Appendix D

Geology and Soils

Appendix D.1

Geotechnical Feasibility Report Update

GEOTECHNICAL
PROFESSIONALS INC.

September 11, 2020

Sares Regis Group
3501 Jamboree Road, Suite 3000
Newport Beach, California 92660

Attention: Dave Powers
Senior Vice President

Subject: Geotechnical Feasibility Report Update
Proposed Paseo Marina Mixed-Use Development
13450 W. Maxella Avenue (Maxella and Glencoe Avenues)
Palms-Mar Vista-Del Rey Planning Area, Los Angeles, California
GPI Project No. 2962.2I

Dear Dave:

As requested, this report presents the results of supplemental geotechnical services performed by Geotechnical Professionals Inc. (GPI) for the subject project. Specifically, this report presents an update to the feasibility-level geotechnical investigation report prepared for a prior, similar project at the site by others (Golder, 2017a). In 2019, GPI performed field infiltration testing for the project (GPI, 2019) and in 2005, GPI performed a geotechnical investigation for the adjacent Vons Pavilion addition, which included cone penetration tests and borings (GPI, 2005).

GPI reviewed the data and reports referenced herein and is prepared to assume the role as Geotechnical Engineer of Record for the proposed project. As stated in the prior feasibility-level reports, the conclusions and recommendations presented herein are based on limited explorations, laboratory testing and analyses. Additional field explorations, laboratory testing and analyses will be required to develop design-level geotechnical recommendations.

PROJECT DESCRIPTION AND BACKGROUND

We understand that Sares Regis Group is planning to construct a mixed-use development at the subject project site. The location of the site is shown on Figure 1. The currently proposed development (Option B) will be a podium style structure that will include three separate buildings for residential, retail, and office space uses that will be supported on a common two-level subterranean parking garage. Option B is an update from the initial Option A that was covered in the prior geotechnical feasibility report by others.

The ground floor level will include retail, parking, residential apartments and amenities, and open space for a park. Buildings 1 and 3 will have apartments and common space on levels 2 through 7 and 2 through 6, respectively. Building 2 will have 3 levels of office space above the retail space on the ground floor. A third subterranean parking garage level is being considered that may be included within a portion of the structure to accommodate additional parking spaces for the development.

The proposed ground floor level is anticipated to be at or near existing grades. The lower floor level for the two-level subterranean parking garage is anticipated to extend approximately 25 feet below existing grades. If a third level of subterranean parking is included, the lower floor level is anticipated to extend approximately 37 feet below existing grades. Considering the structure will be supported on a 3-foot thick mat foundation, site excavations are anticipated to extend approximately 28 feet below existing ground surface for the 2-level subterranean parking garage, and approximately 40 feet below existing ground surface where the subterranean parking garage extends to three levels.

We understand that a new stormwater infiltration system is being considered generally in the proposed park/green area off the southeast side of the proposed building, along Glencoe Avenue. Additional details of the proposed stormwater infiltration system have not yet been established. Based on our 2019 study (GPI, 2019), the upper soils are not considered conducive to infiltration due to the low infiltration rates and the potential for mounding in relatively thin layers of potentially permeable soils overlying practically impermeable layers or groundwater.

The existing and proposed site plans are shown on Figures 2 and 3, respectively, with the approximate locations of prior explorations by others and GPI. Two cross sections through the proposed building are shown on Figure 4.

SITE DESCRIPTION

The project site is an approximately 6.06-acre portion of the existing Marina Marketplace shopping center located at 13450 West Maxella Avenue in the Palms–Mar Vista–Del Rey Community Planning Area of the City of Los Angeles. The site is located at the intersection of Maxella Avenue and Glencoe Avenue as shown on Figure 1. The site is generally bounded by Maxella Avenue to the north, Glencoe Avenue to the east, the existing Pavilions grocery store and associated parking within the Marina Marketplace shopping center to the south, and the Stella Apartments to the west. The site is currently occupied by three single- and two-story retail buildings and at-grade paved parking.

The Stella Apartments building has 6 levels above grade and 1 (possibly 2) subterranean levels for parking. The Pavilions store is a single-story building with no basement level. The proposed subterranean parking garage will be located approximately 30 feet from the Pavilions building and approximately 45 feet from the Stella Apartment building.

The ground surface gently slopes downward from about Elevation +24 feet along Glencoe Avenue, the northeast border of the site, to about Elevation +20 feet, along the southwest border of the site.

SCOPE OF SERVICES

Our scope of work included review of published information and geotechnical data from prior explorations at and near the site and preparation of this update to the existing geotechnical feasibility report by Golder (Golder, 2017a) for use in the entitlements/EIR process. This feasibility level report update intends to address the current project configuration (Option B) with respect to potential impact issues outlined in the CEQA/EIR checklist for Geology and Soils. This feasibility-level update report also provides updated feasibility-level geotechnical design and construction recommendations and considerations.

PRIOR REPORTS AND EXPLORATIONS

GPI reviewed subsurface data presented in prior studies for the project site and an adjacent site. The subsurface data reviewed were presented in the following reports:

- A geotechnical feasibility report for the project site by Golder (Golder, 2017a). The report by Golder also presented data from two borings drilled at the Stella Apartments site in May 2003 and logged by Group Delta.

- Addenda and responses to City of Los Angeles review comments by Golder (Golder, 2017b and 2017d)

- A percolation/infiltration testing study at the project site by GPI (GPI, 2019)

- A geotechnical investigation report for the adjacent Pavilions grocery store addition by GPI (GPI, 2005)

SUBSURFACE CONDITIONS

Based on the data reviewed, the subsurface soil conditions at the site consist of alluvial deposits that increase in density and stiffness with depth to the depth explored, approximately 59 feet. From just below the existing pavement to about 16 to 20 feet below existing grade, the subsurface soils consist of interbedded layers of medium stiff to stiff sandy silts and clays and localized layers of medium dense silty sands and clayey sands. The upper layer of interbedded deposits is underlain by a layer of medium dense to dense sand and silty sand that was encountered to about 25 feet below existing grade, which was underlain by a layer of dense to very dense sand with gravel that extended to about 32 to 40 feet below existing grade. The dense to very dense layer of sand with gravel is underlain by very dense clayey sand and very stiff to hard sandy clay and clay to the depth explored. The natural clay soils were moist to very moist, the upper sandy soils were moist to wet, and the deeper sands were wet.

Groundwater was encountered at a depth of approximately 16 to 17 feet below the existing ground surface in our 2019 study at the site and in our 2005 study for the adjacent Pavilions expansion. Groundwater was encountered approximately 19 to 19.5 feet below the existing ground surface in the borings by Golder in 2017. Based on data from the State of California (CDMG, 1998), the historical shallowest depth to groundwater within the site vicinity is approximately 6 feet below existing grades.

CONCLUSIONS AND RECOMMENDATIONS

GPI reviewed the data and reports referenced herein and is prepared to assume the role as Geotechnical Engineer of Record for the proposed project. Our review of the referenced geotechnical feasibility-level report (Golder, 2017a) and supplemental data review and analyses were conducted in order to update the feasibility level geotechnical conclusions and recommendations for the current project and to update the conclusions and recommendation regarding the requirements of Section VI. Geology and Soils of CEQA Appendix G: Environmental Checklist.

Based on our review, we generally concur with the feasibility-level findings presented by Golder in the referenced reports except where updated and/or addressed in this report. Additional field explorations, laboratory testing and analyses will be required to develop design level geotechnical recommendations. The following sections provide the results of our updated geologic and seismic hazards evaluation for the proposed development.

UPDATED GEOLOGIC-SEISMIC HAZARDS

Seismic Design

The site is located in a seismically active area of Southern California and is likely to be subjected to strong ground shaking due to earthquakes on nearby faults.

We assume the seismic design of the proposed development will be in accordance with the 2020 Los Angeles Building Code (LABC) criteria. For the 2020 LABC, a Site Class D may be used. Using the Site Class, which is dependent on geotechnical issues, and the appropriate internet website (https://seismicmaps.org/), the corresponding seismic design parameters from the LABC are as follows:

$S_S = 1.863g$	$S_{MS} = F_a * S_S = 1.863g$	$S_{DS} = 2/3 * S_{MS} = 1.242g$
$S_1 = 0.660g$	$S_{M1} = F_V * S_1 = 1.122g$	$S_{D1} = 2/3 * S_{M1} = 0.748g$

In accordance with the 2020 LABC (and the 2019 CBC), site-specific response spectra are required for structures located in a Site Class D (with S_1 greater than or equal to 0.2) unless, per the exceptions detailed in Section 11.4 8 of ASCE 7-16, the structure is designed using seismic response coefficient (Cs) determined by either:

- Equation 12.8-2 for values of $T \leq 1.5\ T_S$,
- 1.5 times the value computed by Equation 12.8-3 for values of $T_L \geq T > 1.5\ T_S$, or
- 1.5 times the value computed by Equation 12.8-4 for values of $T > T_L$.

If this exception is not taken and the structure will still be designed in accordance with the 2020 LABC, GPI should be notified that site-specific response spectra is requested. Based on the mapped seismic parameters, the T_S period is approximately 0.6 seconds (therefore 1.5*T_S is approximately 0.9 seconds).

The above seismic code values should be confirmed by the Project Structural Engineer using the values above and the pertinent internet website and tables from the building code. The Project Structural Engineer should also evaluate the period of the proposed structure with respect to the T_S value above when reviewing whether a site-specific response analysis will be requested.

Strong Ground Motion Potential

Based on published information (earthquake.usgs.gov), the most significant fault in the proximity of the site is the Santa Monica Fault, which is located about 3.4 miles from the subject site.

During the life of the project, the site will likely be subject to strong ground motions due to earthquakes on nearby faults. Based on the OSHPD website (https://seismicmaps.org/), we computed that the site could be subjected to a peak ground acceleration (PGA_M) of 0.88g for a magnitude 6.8 earthquake (Santa Monica Fault). This acceleration has been computed using the mapped Maximum Considered Geometric Mean peak ground acceleration from ASCE 7-16 (ASCE, 2017) and a site coefficient (F_{PGA}) based on Site Class. The predominant earthquake magnitude was determined using a 2-percent probability of exceedance in a 50-year period, or an average return period of 2,475 years. The structural design will need to incorporate measures to mitigate the effects of strong ground motion.

Liquefaction and Seismic Settlement

As stated in the prior referenced reports, the site is located within an area mapped by the State of California as having a potential for soil liquefaction (CGS, 1999). This section presents the results of our updated liquefaction analyses based on updated seismic parameters discussed in the preceding section.

Groundwater was encountered at a depth of approximately 19.3 feet below the ground surface during a 2017 geotechnical investigation by others. During our prior 2019 infiltration investigation, we encountered groundwater at a depth of approximately 16.5 feet below the existing ground surface. Based on historical data from the State of California (CDMG, 1998), the shallowest depth to groundwater within the site vicinity is approximately 6 feet below existing grades. A groundwater depth of 6 feet was used in our analyses.

Revisions to the 2020 Los Angeles Building Code, ASCE 7-16, and Special Publication 117A (CGS, 2008) require that the ground motion used for this evaluation be based on the Peak Ground Acceleration (PGAM), adjusted for site class effects. This value is computed using the mapped Maximum Considered Geometric Mean (MCEG) peak ground acceleration and a site coefficient, F_{PGA}, based on a Site Class D. In accordance with the 2020 LABC, we considered a ground acceleration of 0.88g for a magnitude 6.8 earthquake (Santa Monica Fault) for our analyses, which corresponds to the PGAM obtained using the methods described above.

The potential for liquefaction was evaluated using the methods presented by the NCEER and updated by Robertson (Robertson, 2009) and modifications provided in Special Publication 117A. Criterion for liquefaction susceptibility of the fine-grained soils was based on methods presented in Bray and Sancio (2006).

The materials encountered at the site generally consisted of approximately 16 to 20 feet of stiff to very stiff clays and silts with localized layers of medium dense silty sands and clayey sands. The upper silts and clays are underlain by medium dense to very dense sands with silt and gravel to approximately 32 to 40 feet below grade which are underlain by very dense clayey sand and very stiff to hard sandy clay and clay to the depth explored. We understand the proposed development will include two subterranean levels with the bottom of foundation extending to depths of approximately 28 feet below surface grades. The bottom of foundation is anticipated to extend on the order of 40 feet below grade if a 3rd subterranean level is incorporated into design.

Based on prior CPT data (six locations on site by Golder and two locations by GPI at the adjacent Pavilions), we computed an overall potential seismic-induced liquefaction settlement at the ground surface of ½ to 1½ inches. Based on prior boring data (two borings by Golder, B-17-01 and B-17-02) we computed overall potential seismic-induced liquefaction settlement at the ground surface of approximately 5 to 8 inches. Taking into account the planned site excavations for the two-level (or 3 level) subterranean garage, and with the exception of Boring B-17-01, the total seismic-induced liquefaction settlements below foundations depths are estimated to be less than ½ inch. At B-17-01, the calculated total seismic-induced liquefaction settlements below foundations is on the order of 2½ inches. CPT-4, which is adjacent to B-17-01, indicates a liquefaction induced settlement of ¼-inch at the foundation level.

The estimated 2½ inches of liquefaction settlement in Golder Boring B-17-01 occurred in a clayey sand material encountered between depths of 40 and 51.5 feet below grade. As noted in Golder's Addendum 1 response (Golder, 2017d), laboratory testing of this clayey sand material indicated 46-percent fines and a plasticity index of 25. Per Bray and Sancio (2006), this material would be considered non-liquefiable. We should note that liquefaction analyses of the CPT data from the Golder investigations also finds these clayey sand materials to be non-liquefiable.

Differential seismic settlement is estimated to be less 50 percent of the total seismic settlement, which would conservatively be less than 1¼ inch across a span of 40 feet when considering the current results of B-17-01 and less than a ½ inch across a span of 40 feet when considering the remainder of available data. We note that these values are relatively consistent with the estimate liquefaction settlements presented by Golder in their Addendum 1 response (Golder, 2017d).

As part of a supplemental field and laboratory investigation in order to develop design-level geotechnical recommendations, GPI would be able to further assess the liquefaction potential of this material. If laboratory testing confirms this material is likely non-liquefiable, the estimated total and differential seismic-induced liquefaction settlements would be less than ½ inch for a 2 level subterranean garage and less for a 3 level subterranean garage.

Seismic Ground Subsidence

Seismic ground subsidence, not related to liquefaction, occurs when loose, granular soils above the groundwater are densified during strong earthquake shaking. The 2020 LABC and ASCE 7-16 (ASCE, 2017) require that the ground motion used to evaluate liquefaction and seismic settlement be based on the Peak Ground Acceleration (PGA_M) adjusted for site class effects. This value is computed using the mapped Maximum Considered Geometric Mean (MCE_G) peak ground acceleration for Site Class B and a site coefficient, F_{PGA}. Accordingly, we considered a ground acceleration of 0.88g for a magnitude 6.8 earthquake as detailed previously.

Due to the historical shallow depth of groundwater (about 6 feet below grade) and that the proposed structure will include multiple subterranean levels, we consider the potential for dry seismic settlement to negatively impact the proposed development to be nonexistent.

Tsunamis, Seiches, and Flooding

Various types of seismically induced flooding, which may be considered as potential hazards to a particular site, include flooding due to a tsunami (seismic sea wave), a seiche, or failure of a major water retention structure upstream of the project. The site is located approximately 0.25 miles from the marina and about 1.6 miles inland from the Pacific Ocean at an elevation of approximately 24 feet above mean sea level. As mapped by the California Emergency Management Agency, the subject site is not located in a tsunami inundation area (CEMA, 2009). The closest tsunami inundation line to the subject site, as mapped by CEMA, is approximately 0.2 miles to the southwest.

The site does not lie in proximity to reservoirs or other significant water retention structures. The closest reservoir is the Stone Creek Reservoir, which is located approximately 8.2 miles to the north and at an elevation of roughly +850 feet. The subject site is located in a Potential Inundation Area as mapped in the City of Los Angeles Seismic Safety Element (1996). Based on this map, the inclusion of the site in a Potential Inundation Area appears to be related to a potential failure of the Stone Canyon Reservoir.

As such, the probability of flooding due to tsunami, seiche-like waves, or failure of water retention structures is considered to be low.

Methane

The subject site is located in a Methane Buffer Zone as mapped by the City of Los Angeles (NavigateLA; LADPW, 2004). The nearest Methane Zone is located immediately south of the subject site, within the adjacent property. Because the site is located within a Methane Buffer Zone, site testing of the concentration and pressure of methane gas is required to establish the Design Methane Concentration and Design Methane Pressure. We understand that a methane study for the site is being conducted by others. Detectable odors were not noted in prior geotechnical investigation reports by others and were not encountered during our prior infiltration investigation at the subject site (GPI, 2019).

PRELIMINARY GEOTECHNICAL CONCLUSIONS AND RECOMMENDATIONS

Based on our review, we generally concur with the feasibility-level geotechnical design recommendations and construction considerations presented in the referenced reports except where updated and/or addressed below.

Foundations and Walls Below Grade

We generally concur that the recommendations for foundations and walls below grade presented in the prior feasibility report. The proposed two-level subterranean structure is recommended to be supported on a mat foundation. A mat foundation is also recommended if a portion of the structure is deepened to have a 3rd subterranean level. The foundation will need to be stepped between the 2nd and 3rd subterranean level or the walls of the 3rd subterranean level will need to be designed to resist the surcharge load from the 2nd level foundation.

During final design, we anticipate that updated modulus and estimated settlement values will be applicable based on additional explorations, lab testing and analyses. At this time, the current recommended values are considered applicable for preliminary design.

Uplift Pressure

The mat foundation for two and three levels below grade will need to be designed to resist uplift pressure due to buoyant forces as the bottom of foundations are expected to be 28 and 40 feet below existing grade and below the current and historical high groundwater levels. The historical high groundwater elevation of 6 feet below ground surface, approximate Elevation +18 feet, should be used in design.

Shoring of Temporary Excavations

Excavations are anticipated to extend to depths of 28 feet below grade for the two level subterranean garage and 40 feet below grade if a portion of the subterranean garage is extended to three levels below grade. Temporary braced shoring can be used for two- and three-level temporary excavations as space is not anticipated to be available for sloped excavations. Raker (internal) or tie-back anchor (external) bracing may both be used.

Soldier Piles

Shoring may consist of steel soldier piles placed in drilled holes and backfilled with concrete. Due to the granular nature of the site soils and depth to groundwater, continuous lagging is recommended. Use of continuous sheet piles is not considered feasible due to anticipated difficulties in driving the sheet piles though the very dense sand and gravel layers and potential vibration and noise nuisance to adjacent properties caused by pile driving and vibratory hammers.

Dewatering associated with this type of shoring system will likely require a system of wells around the perimeter of the excavation that will draw down and maintain the groundwater level below the bottom of the excavation. Temporary dewatering of the sand layer is anticipated to collect a substantial volume of water. If a 3rd subterranean level is incorporated into design, there will likely be a need to install gravel filled trenches at the base of the soldier pile wall as the deeper clay layers may reduce the effectiveness of the wells in lowering the groundwater level. The trenches act as a sump and pump system for water not collected by the wells.

Challenges and considerations associated with use of a conventional soldier pile and lagging system of shoring with dewatering wells include the following:

- Soldier pile excavations below groundwater and/or in granular deposits will likely require use of drilling mud or polymers to maintain stability of the drilled excavation prior to backfilling with cement and/or slurry.

- Installation of wood lagging will be difficult in layers with wet granular deposits due to increased caving potential.

- Temporary dewatering wells are anticipated to collect a substantial volume of water from the sand layer from prior to installation of soldier piles until subterranean construction is completed. The collected water may need to be treated prior to disposal offsite, which would require on-site treatment equipment.

A primary concern for the type of shoring and dewatering system used is the potential impacts that lowering the groundwater will have on adjacent sites, including ground settlement caused by lowering of the groundwater. At the perimeter and interior of the excavation, and considering groundwater is currently at approximately 16 feet below grade, groundwater levels are anticipated to be lowered 15 to 18 feet for a 2 level subterranean garage and 27 to 30 feet for a three-level subterranean garage.

Because groundwater has reportedly fluctuated at the site to at least 19 feet below grade in the past and the soil conditions discussed herein, it is our opinion that lowering groundwater at the site as described above for a two-level subterranean garage is anticipated to have a negligible effect on the adjacent structures. Potential settlement of adjacent structures is anticipated to be within tolerable limits. This may also be the case for a three-level subterranean garage, but additional subsurface and existing building data is needed to further evaluate this condition.

Soil Cement Cutoff Wall

An alternative shoring system could consist of a continuous soils-cement cutoff wall with embedded H beams. The continuous soil cement wall could be constructed with overlapping soil-cement panels or overlapping soil-cement columns. The cutoff wall will likely extend into the deeper clay layer for both the two- and three- level subterranean garage excavations in order to reduce the volume of water entering the site from the sand layers and thereby reducing the volume of water that will be collected during construction. The embedded length of the soldier pile walls will be dependent on the depth of excavation. Some form of dewatering, likely widely spaced dewatering wells, will be required to initially lower the groundwater within the site after the cutoff wall is installed and to maintain the lowered water level inside the excavation during subterranean construction.

Due to the depth of the excavation, the cutoff wall will need to be braced during construction until the earth loads can be transferred to the structure. Bracing could consist of rakers and/or tie-back anchors.

Considerations associated with use of a cutoff wall shoring system:

- These systems are less common in southern California and cost significantly more than conventional shoring systems.

- The associated dewatering effort is considerably less because the volume of water that will be collect is much less than the system that would be used with traditional soldier piles and lagging shoring systems.

With the cutoff wall system, the groundwater level outside the excavation is minimally impacted by the dewatering inside the excavation. Accordingly, the potential for settlement of adjacent structures caused by the dewatering program is significantly reduced for both the two- and three-level subterranean garage. Potential settlement of adjacent structures is anticipated to be within tolerable limits when a cutoff wall is used.

LIMITATIONS

The report and other materials resulting from GPI's efforts were prepared exclusively for use by Sares Regis Group and their consultants in feasibility level design of the proposed development. The report is not suitable for a project other than the currently proposed development.

Soil deposits may vary in type, strength, and many other important properties between points of exploration due to non-uniformity of the geologic formations or to man-made cut and fill operations. While we cannot evaluate the consistency of the properties of materials in areas not explored, the conclusions drawn in this report are based on the assumption that the data reviewed are reasonably representative of field conditions and are conducive to interpolation and extrapolation.

As noted previously, additional geotechnical investigations will be needed for design and construction. Furthermore, our recommendations were developed with the assumption that a proper level of field observation and construction review will be provided by a qualified geotechnical consulting firm during grading, excavation, and foundation construction. If design- and construction-phase geotechnical services are performed by others they must accept full responsibility for all geotechnical aspects of the project.

Our investigation and evaluations were performed using generally accepted engineering approaches and principles available at this time and the degree of care and skill ordinarily exercised under similar circumstances by reputable Geotechnical Engineers practicing in this area. No other representation, either expressed or implied, is included or intended in our report.

Respectfully submitted,
Geotechnical Professionals Inc.

Justin J Kempton, G.E.
Associate

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Enclosures: References
Figure 1 - Site Location Map
Figure 2 - Existing Site Plan
Figure 3 - Proposed Site Plan
Figure 4 - Building Sections

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0 2000 4000 FEET

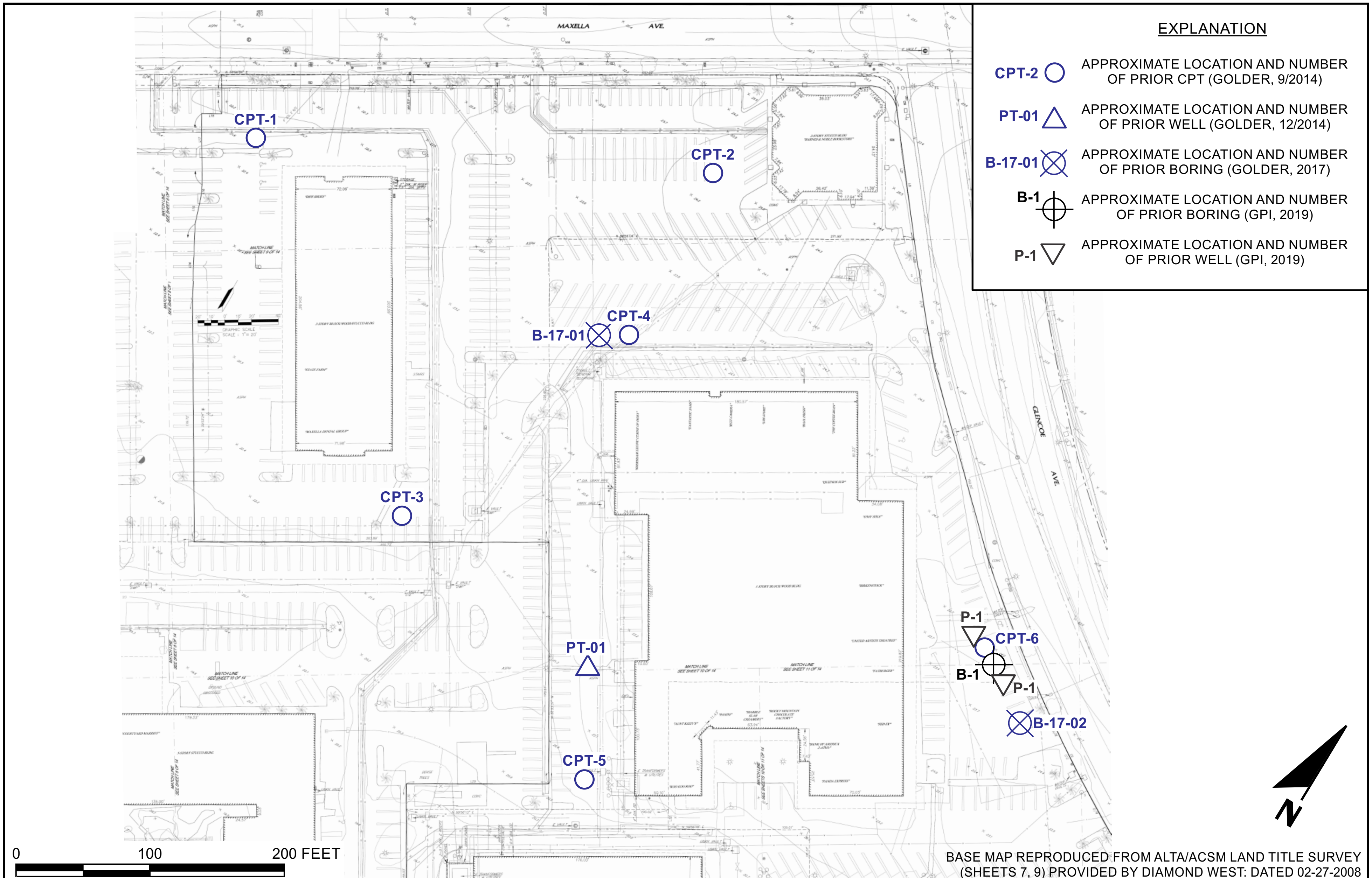
BASE MAP REPRODUCED FROM USGS 7.5' TOPO MAPS © CALTOPO

GPI GEOTECHNICAL PROFESSIONALS, INC.

SITE LOCATION MAP

PASEO MARINA

GPI PROJECT NO.: 2962.2I	SCALE: 1" = 2000'	FIGURE 1



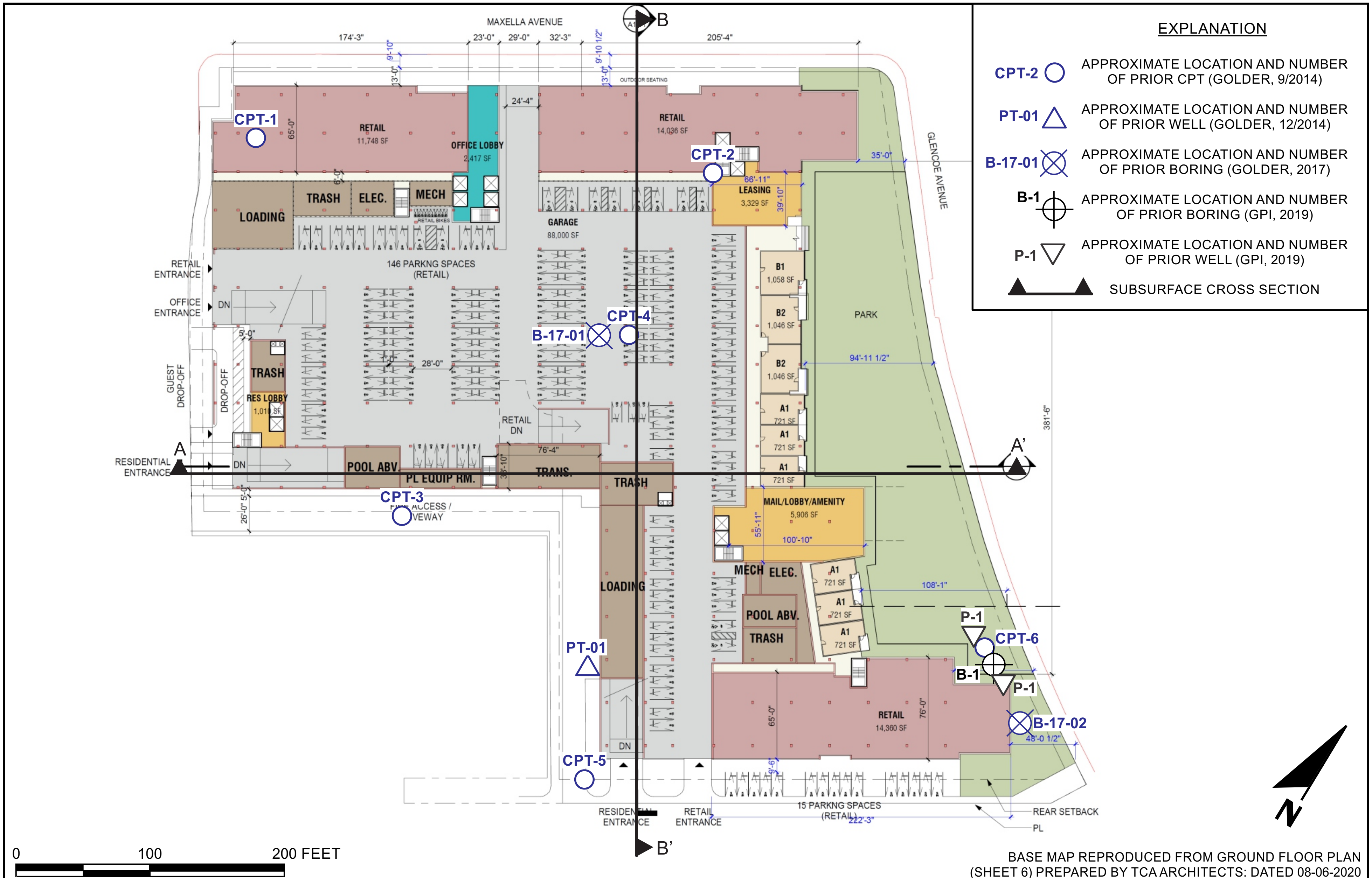
GPI GEOTECHNICAL PROFESSIONALS, INC.

PASEO MARINA

GPI PROJECT NO.: 2962.2I	SCALE: 1" = 100'

EXISTING SITE PLAN

FIGURE 2



GEOTECHNICAL PROFESSIONALS, INC.

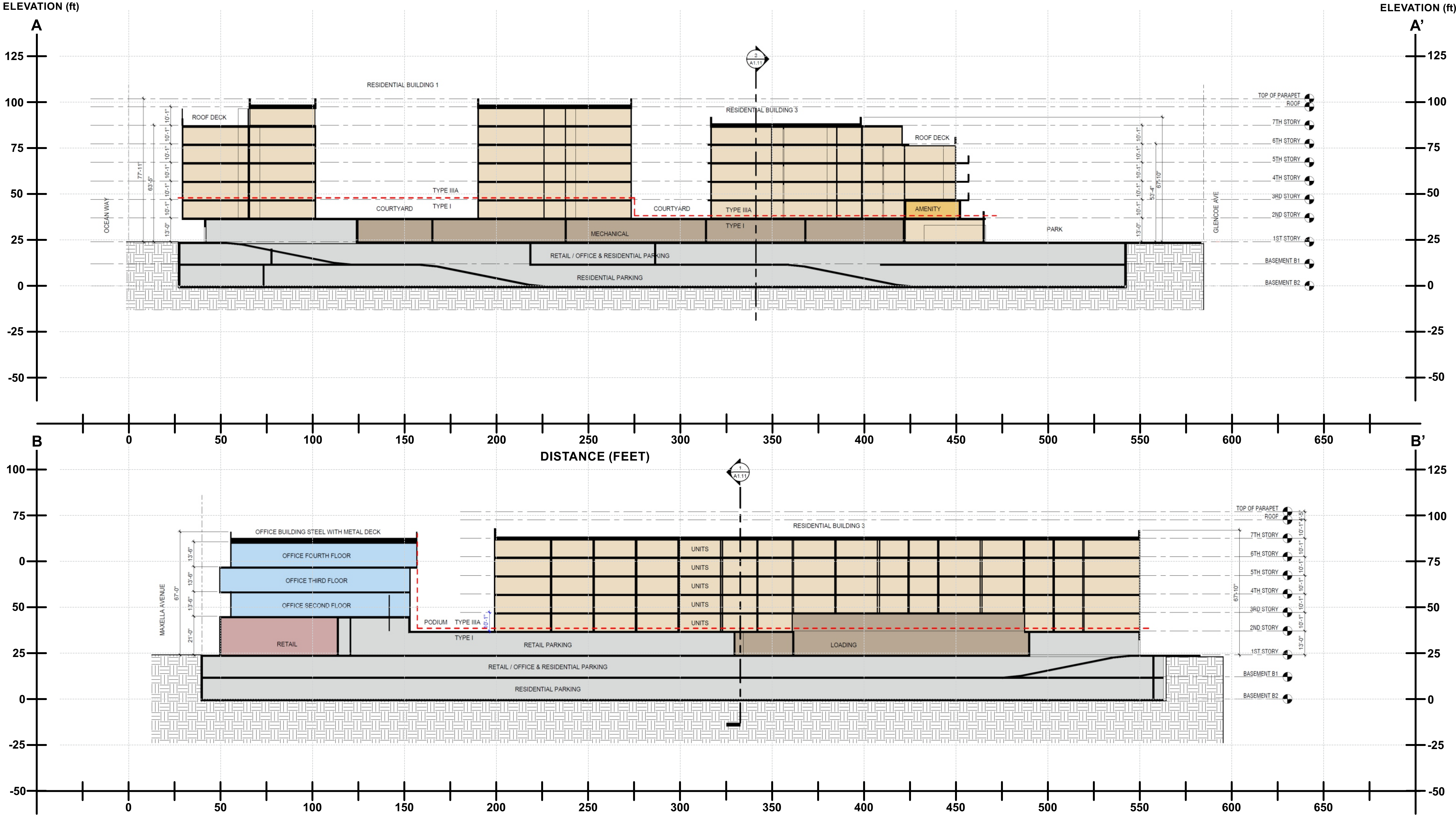
PASEO MARINA

GPI PROJECT NO.: 2962.2I

SCALE: 1" = 100'

PROPOSED SITE PLAN

FIGURE 3



BASE MAP REPRODUCED FROM SITE SECTIONS (SHEET 13) PREPARED BY TCA ARCHITECTS: DATED 08-06-2020

Note: This section is based upon information obtained at borings and CPTs obtained during geotechnical investigations by GPI and others. The section is based upon limited geotechnical data and localized variations should be anticipated. This section is intended for descriptive purposes only.

GEOTECHNICAL PROFESSIONALS, INC.

PASEO MARINA

GPI PROJECT NO.: 2962.2I | SCALE: 1" = 50'

BUILDING SECTIONS: A-A' AND B-B'

FIGURE 4

Appendix D.2

Geotechnical Feasibility Report

REPORT

GEOTECHNICAL FEASIBILITY REPORT

Marina Marketplace Phase III
13450 W. Maxella Avenue, Marina del Rey, California

Submitted To: Sares-Regis Group
18802 Bardeen Avenue
Irvine, CA 92612

Submitted By: Golder Associates Inc.
3 Corporate Park, Suite 200
Irvine, CA 92606

January 16, 2015 (Revised March 16, 2017) **1403929**

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1.0 INTRODUCTION

This report presents the results of the geotechnical feasibility study performed by Golder Associates Inc. (Golder) for the Marina Marketplace Phase III project to be located at 13450 West Maxella Avenue in Marina del Rey, California (the Site). The Site location is shown on Figure 1. This report presents a project description, a summary of Golder's limited geotechnical field investigation, and preliminary geotechnical engineering recommendations for the proposed development. Prior to final design of the project, it will be necessary to perform a design-level geotechnical study for the Site, which will include final geotechnical design recommendations for the project.

1.1 Existing Site Conditions

The Site has a net area of approximately 6 acres and is located at the intersection of Maxella Avenue and Glencoe Avenue in the Marina del Rey area of the City of Los Angeles, California, as shown on Figure 2. The Site is bordered to the north by the Tierra del Rey Apartments and the Villa Velletri Townhouses, to the west by the Marina Marketplace (Gelsons and AMC) and the Stella Apartments, to the east by the Marina Marketplace Phase I (Pavilions) and to the south by Hotel MdR Marina del Rey – a DoubleTree by Hilton. The Site is currently occupied by several retail buildings and at-grade paved parking lots. The existing ground surface at the Site is relatively flat and gently slopes down toward the south and east.

1.2 Proposed Development

The proposed project consists of the re-entitlement of the Site to construct approximately 660 apartment units and approximately 25,000 square feet of retail space. The project currently consists of a multistory residential development with up to seven levels above ground and 1.5 to 2 levels below ground. We have assumed that the total depth of the excavation will be approximately 18 to 20 feet below current grade. The project may also include a stormwater infiltration system.

1.3 Previous Investigations

Golder reviewed available geotechnical information for nearby structures at the City of Los Angeles Building Department. Several reports were available, including a geotechnical report performed at the Site for an expansion of the existing retail. These reports included both geotechnical borings and cone penetration test data.

1.4 Objective and Scope of Work

The objective of Golder's current study was to provide preliminary geotechnical recommendations for the preliminary design of the proposed residential development. In particular, the objective was to identify geologic conditions at the Site that could make the project uneconomic. Golder's scope of work included performing a data review, limited field exploration, and geologic characterization of the Site and providing preliminary geotechnical engineering design recommendations. The results of Golder's study are provided in the following sections of this report.

2.0 LIMITED GEOTECHNICAL EXPLORATION

2.1 Utility Clearance and Data Review

Golder performed a visual reconnaissance of the Site on September 22, 2014 to mark out cone penetration test (CPT) locations. Underground Service Alert of Southern California (Dig Alert) was notified by Golder of the proposed CPT locations as required by law. Golder did not contract the services of any utility location company during this phase of the project.

A drilling permit was obtained from the County of Los Angeles Public Health Department because subsurface exploration depths penetrated the groundwater table. A copy of the drilling permit is included in Appendix A.

Geologic and geotechnical data available for the region and Site were gathered from the following sources:

- "State of California Seismic Hazard Zones Map, Venice Quadrangle," prepared by the State of California Department of Conservation, Division of Mines and Geology, dated March 25, 1999.
- Geotechnical Investigation, Proposed Building Expansion, Existing Vons Store, 4365 Glencoe Avenue, Los Angeles, California.
- Additional Explorations, Proposed Hardscapes and Pavement Improvements, Phase 2 Villa Marina Market Place, 13455 Maxella Avenue, Marina del Rey, California.
- Geotechnical Feasibility Letter, Proposed Villa Marina, 13400 – 13490 W. Maxella Avenue, Los Angeles, California.

2.2 Limited Field Investigation

The purpose of the limited geotechnical field investigation was to evaluate the subsurface conditions within the proposed project Site in order to evaluate the engineering characteristics of the underlying soils for feasibility-level purposes. The limited geotechnical investigation consisted of advancing six CPT soundings (CPT-1 through CPT-6) and one soil boring (PT-01).

2.2.1 Cone Penetration Test (CPT) Soundings

CPT soundings were advanced by Kehoe Testing and Engineering of Huntington Beach, California on September 25, 2014. The CPT's were advanced using a 30-ton thrust capacity truck-mounted CPT rig. Data was collected in accordance with ASTM D5778 using a standard 15 square centimeter electronic cone system. Tip resistance and sleeve friction data were recorded continuously at approximately 2.5 centimeter depth intervals.

The upper 5 feet of each CPT location were hand augered to confirm the absence of utilities. A total of six CPT soundings were advanced at the locations shown on Figure 2. The planned investigation included advancing five (5) CPTs to a depth of 50 feet below the existing ground surface (bgs) and one CPT to a depth of 75 feet bgs. The actual depths of CPT soundings ranged from 26 to 60 feet bgs. Four of the CPT soundings (CPT-2, CPT-4, CPT-5, and CPT-6) hit refusal before the planned termination depth. The CPT data graphs are presented in Appendix B.

All CPT soundings were backfilled with bentonite pellets and the upper 6 inches were capped with cold-patch asphalt mix.

2.2.2 Soil Test Boring

One soil test boring was drilled on December 17, 2014 using a truck mounted hollow stem auger drill rig provided by Martini Drilling Corporation of Huntington Beach, California. The boring was drilled to an approximate depth of 12 feet bgs. The boring was drilled in the location of the proposed stormwater infiltration basin. Figure 2 shows the location of the test boring.

The soil cuttings from the boring were visually logged in the field by a Golder engineer. In addition, two standard penetration test (SPT) soil samples were collected from depths of 6 ft bgs and 12 ft bgs.

The log for the soil boring is presented in Appendix C. The log (Record of Borehole) describes the earth materials encountered and the samples obtained. The log also shows the boring number, drilling date, and the name of the Golder engineer that logged the boring. The soils were described in general accordance with ASTM D2488. The boundaries between different soil types shown on the log are approximate because the actual transition between soil layers may be gradual.

2.2.3 Previous Investigations

Geotechnical Professionals, Inc. performed a geotechnical investigation for a proposed Vons store expansion adjacent to and southwest of the Site in 2005. The investigation included two geotechnical borings drilled to depths of 26.5 and 51 feet bgs and two CPTs advanced to depths of 36 and 50 feet bgs. Group Delta Consultants, Inc. performed a geotechnical investigation for a proposed Villa Marina development. The investigation included two geotechnical borings drilled to depths of 41 and 58.8 feet bgs and two CPTs advanced to depths of 42 and 55 feet bgs. Copies of the boring logs from the previous investigation are included in Appendix D.

3.0 GEOLOGIC CONDITIONS

3.1 Site Subsurface Conditions

The Site is located on alluvial soils derived from the nearby Ballona Creek. The alluvial soils are vertically and horizontally discontinuous as a result of periods of alluvial deposition.

Golder's geotechnical exploration confirmed that the area within the Site is underlain by alluvial soils to the depths explored. From an interpretation of the CPT data, the alluvial soils generally consist of approximately 17 to 20 feet of silt and clay. The silt and clay contained layers/lenses of sand and silty sand. Below the silt and clay lies a medium dense to dense sand layer. This sand layer, where penetrated, was approximately 20 to 25 feet thick. Below the sand is another silt and clay layer approximately 5 to 15 feet thick. The interpretation of the CPT data is consistent with the borings drilled on the adjacent sites.

3.2 Groundwater

According to the groundwater level contour map prepared by the California Division of Mines and Geology (CDMG, 1998) and presented in the Seismic Hazard Zone Report for the Venice 7.5-Minute Quadrangle, the historical high groundwater level at the Site is approximately 6 feet bgs. Geotechnical borings on the properties adjacent to the Site encountered groundwater at a depth of approximately 17 feet bgs. The depth to groundwater can fluctuate with the time of year; however, the water table is likely controlled by the ocean located approximately 1,000 feet to the southwest of the Site. The depth of the groundwater table should be determined during final design.

The City of Los Angeles typically requires that infiltration basins are located a minimum of 10 feet above the current groundwater table. We understand that for this project the City of Los Angeles will allow the infiltration basin to be located a minimum of 5 feet above the current groundwater table. A percolation test was performed in the area of the proposed basin at a depth of 12 feet bgs. The results of the percolation testing are presented in Section 3.3.

3.3 Percolation Testing

The percolation testing was performed in soil test boring PT-01 in accordance with the County of Los Angeles Department of Public Works guidelines as outlined in the Low Impact Development (LID) Manual. After the test boring was drilled, the augers were removed from the borehole and approximately two inches of No. 3 coarse grained sand was placed at the bottom of the hole. A 2-inch diameter, 10-foot long slotted PVC pipe was then placed into the center of the borehole. Six feet of No.3 coarse grained sand was used to fill the annular space between the PVC pipe and the borehole walls. Five gallons of water was poured into the PVC pipe and the borehole was allowed to pre-soak for several hours.

The percolation test was performed in the borehole on the same day the boring was drilled and pre-soaked (i.e., December 17, 2014). The percolation test was performed by pouring 5 gallons of clear water into the

PVC pipe installed in the borehole and then measuring the rate at which the water level in the borehole dropped. The water level in the borehole was measured using an electronic water level indicator.

Measurements of the water levels in the borehole were taken in 30 minute intervals over a period of 2.5 hours. The percolation rate (in minutes per inch) in the borehole was then calculated for each increment of time. The infiltration rate (in inches per hour) was calculated from the percolation test data using the following equation:

$$I_t = \frac{\Delta H(60r)}{\Delta t(r + 2H_{avg})}$$

where:

I_t = infiltration rate computed from test results (inches/hour)
ΔH = change in height of water in borehole during time interval (inches)
r = borehole radius (inches)
Δt = time interval over which calculation is being performed (minutes)
H_{avg} = average height of water in borehole during time interval (inches)

Appendix E contains the percolation test data (time intervals, measured water levels, and heights of water in the borehole) and results. Based on the percolation test data, the percolation rate is 7.8 minutes per inch and the calculated infiltration rate is 0.8 inches per hour. It is noted that the use of these values in stormwater infiltration design will require the use of appropriate factors of safety to account for subsurface variability, long-term performance, and other factors.

3.4 Potential Geologic Hazards

3.4.1 Surface Fault

The Site is not located in an Alquist-Priolo Earthquake Fault Zone (*Los Angeles General Plan Safety Element, Exhibit A, Alquist-Priolo Special Study Zones & Fault Rupture Study Areas, page 47, November 1996)*. The closest known active faults to the Site are the Santa Monica fault located approximately 4 miles to the north and the Newport-Inglewood fault located approximately 4 miles to the east. Accordingly, surface fault rupture is not a significant hazard at the Site.

3.4.2 Faults within 20 Miles of the Site

Faults are zones of weakness in the earth's crust. Faults that accommodate horizontal movement are referred to as strike-slip faults. Vertical movements occur on reverse and normal faults. Oblique faults accommodate both horizontal and vertical movements. Faults that have moved within the last 11,000 years are considered active.

Major active strike-slip faults and reverse faults are located within 20 miles of the Site. Table 1 lists the known active faults within 20 miles of the Site. The faults closest to the Site are the Santa Monica fault, the Newport-Inglewood fault, and the Palos Verdes fault, which are all located within 5 miles of the Site. These three faults are shown on Figure 3 and discussed further below.

For faults located at distances greater than 20 miles from the Site, the seismic ground motions at the Site resulting from earthquakes on these distant faults are expected to be small (i.e., less than 0.1 g). In addition, Section 3.5.2 confirms that the ground motion hazard at the Site is controlled by the faults located closest to the Site (i.e., less than 10 miles from the Site).

Table 1. Holocene-Active Faults with Surface Rupture within 20 Miles of the Site

Fault Name[1]	Distance to Site (miles)[2]	Fault Type[1]	Last Historical Event (year)	Maximum Magnitude (M)[1,3]	Median Deterministic PGA (g)
Santa Monica	4	R	---	6.6	0.29
Newport- Inglewood – north Los Angeles Basin section	4.3	RLSS	1920 (M 4.9)	6.9	0.30
Palos Verdes – Santa Monica Basin section	4.5	RLSS	---	7.1	0.31
Hollywood	6.8	R/LLSS	---	6.5	0.19
Redondo Canyon	16.5	R	---	6.4	0.08
Raymond	17.2	LLSS	---	6.8	0.10
Newport- Inglewood – south Los Angeles Basin section	18	RLSS	1812; 1933 (M 6.3)	7.0	0.11

Notes:

1) Data from U.S. Geological Survey Fault and Fold Database (Petersen et al., 2008)
2) As measured using Google Earth™ from the Site (located at 33.9863, -118.4402)
3) Evaluated from values in Petersen et al (2008) using earthquake scaling relationships presented in Stirling et al. (2013)

3.4.2.1 Santa Monica Fault

The Santa Monica fault is an ENE-trending reverse-oblique fault located along the southern flank of the Santa Monica Mountains. It extends offshore of Santa Monica to the west to Malibu and to the east it extends to the intersection with the West Beverly Hills Lineament (the northern extent of the Newport-Inglewood Fault). Attenuation equations indicate that the Santa Monica fault is capable of generating a median peak horizontal ground acceleration (PGA) of 0.29 g at the Site.

3.4.2.2 Newport-Inglewood Fault System

The Newport-Inglewood fault is right lateral strike slip fault. The Newport-Inglewood fault zone is a part of the fault system that extends from Beverly Hills to San Diego. South of Newport Beach the fault is located offshore. North of Newport Beach the fault is divided into two segments: the North Los Angeles Basin segment and the South Los Angeles Basin segment. The Los Angeles River forms an approximate boundary between these two segments. Attenuation equations indicate that the Newport-Inglewood fault is capable of generating a median PGA of 0.30 g at the Site.

3.4.2.3 Palos Verdes Fault System

The Palos Verdes fault is a right lateral strike-slip fault. The Palos Verdes fault zone is part of a fault system that extends from Santa Monica Bay to San Diego Bay. The fault is located offshore over most of its length. A small onshore segment is located east of San Pedro and Palos Verdes. Attenuation equations indicate that the Palos Verdes fault is capable of generating a median PGA of 0.31 g at the Site.

3.4.3 Historical Seismicity

Instrumental and reported historic records from the late 1900s through January 2015 reveal that at least 162 earthquakes of magnitude **M** ≥ 4.0 having epicenters located within about 62 miles (100 km) of the Site have occurred in this timeframe. Earthquake magnitudes and epicenter locations were taken from catalogs maintained by the U.S. Geological Survey National Earthquake Information Center (http://neic.usgs.gov/). Twenty-two (22) earthquakes of **M** ≥ 5.0 have been recorded from the late 19th Century through January 2011, and 3 of these earthquakes were of **M** ≥ 6.0. Most of the recorded earthquakes have occurred at distances of more than about 20 miles (32 km) from the Site.

The largest earthquakes near the Site are the 1933 **M** 6.3 Long Beach Earthquake, the 1971 **M** 6.6 Sylmar Earthquake, and the 1994 **M** 6.7 Northridge Earthquake. The shortest distance from the Site to the zone of energy release for these earthquakes is estimated to be 4, 18, and 22 miles, respectively. Using strong motion recordings located throughout the Los Angeles basin, Stewart et al. (1994) estimate the PGA at the Site during the Northridge Earthquake was between 0.2 and 0.3 g.

3.4.4 Landslides

The Site is relatively flat and located in Marina del Rey near the coast. The Site and surrounding areas are fully developed and generally characterized by gently sloping topography that would not be susceptible to landslides. There are no known landslides near the Site, nor is the Site in the path of any known or potential landslides. Furthermore, the Site is not mapped as an Earthquake-Induced Landslide Area as designated by the CDMG (1998), nor is the Site mapped as a landslide area by the City of Los Angeles.[1,2]

3.4.5 Tsunamis, Seiches, and Flooding

Tsunamis are very large waves in the ocean caused by seismic events, landslides, or volcanic eruptions. The Site is located less than one mile from the marina at an elevation of approximately 24 feet above mean

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[1] Los Angeles General Plan Safety Element, Exhibit C, Landslide Inventory & Hillside Areas, page 51 (November 1996).

[2] City of Los Angeles Department of City Planning, ZIMAS, Parcel Profile Report for 13450 Maxella http://zimas.lacity.org/, accessed March 14, 2017.

sea level. The Site is not located in a Tsunami Inundation Zone as mapped by the California Geological Survey (2009). On this basis, the tsunami hazards are not significant at the Site.

Seiches are large waves generated in enclosed bodies of water in response to ground shaking. No major water-retaining structures or land-locked bodies of water are located immediately up gradient from the Site. Therefore, the risk of flooding from a seiche is considered to be remote.

The Site is not located within a flood influence area of the City of Los Angeles Seismic Safety Element (1996) or a FEMA flood hazard zone.

3.4.6 Subsidence

SoCal Gas operates a natural gas storage field below Playa del Rey south of the Site. The storage field was originally an oil field that produced in the 1930s. Oil production lasted approximately 10 years. In 1942, the United States government began using the field for natural gas storage. In 1955, a predecessor of SoCal Gas purchased the field and SoCal Gas has been operating it since 1955. The natural gas storage area is not located below the Site. Natural gas is injected and withdrawn from 54 active wells operated by SoCal Gas.

Removal of oil and gas from geologic formations can cause surface subsidence. Because the oil extraction stopped 72 years ago, Golder expects that subsidence from oil extraction is substantially complete. SoCal Gas has been monitoring subsidence from the operation of the gas field since 2009. The monitoring has indicated that minor subsidence may occur with the operation of the field. However, the potential damage to surface structures from subsidence is low.

Subsidence can also occur when groundwater is withdrawn from unconsolidated aquifers. There is no indication that groundwater withdrawal is currently taking place in the area surrounding the Site. Therefore, the potential for subsidence is low.

3.5 Other Seismic Considerations

3.5.1 Ground Shaking

As with all of Southern California, the Site would be subject to potential strong ground motions if a moderate to strong earthquake were to occur on a local or regional fault. Design of the proposed structures in accordance with the provisions of the California Building Code will mitigate the potential effects of strong ground shaking.

The bases for the 2016 California Building Code (CBC) seismic design are 5%-damped spectral accelerations for 0.2 seconds (S_S) and 1 second (S_1) at a rock site (Site Class B). These 5%-damped spectral accelerations are established for a risk-adjusted Maximum Considered Earthquake (MCE_R). Typically, the MCE_R spectral accelerations have a mean return period of 2,475 years (i.e., 2% probability of being exceeded in 50 years). At some locations, the 2,475-year ground motions are capped by deterministic ground motions. The values for S_S and S_1 were evaluated using the US Seismic Design Maps application (http://earthquake.usgs.gov/designmaps/us/application.php) provided by the United States Geological Survey (USGS). Site coefficients (F_a and F_v) were used to scale the spectral accelerations as a function of Site Class to develop a site-specific, 5%-damped acceleration response spectrum. Table 2 provides the recommended 2016 CBC seismic design parameters for the Site based on the results of Golder's geotechnical exploration and on Section 1613 of the 2016 CBC.

Table 2. 2016 California Building Code (CBC) Seismic Design Parameters

2016 CBC Seismic Design Parameter	**Value**
Site Class	D
5%-damped, 0.2-sec spectral acceleration (S_S)	1.672 g
5%-damped, 1-sec spectral acceleration (S_1)	0.658 g
Site Class D, 5%-damped , maximum considered earthquake geometric mean (MCE_G) peak ground acceleration (PGA_M)	0.63 g
Site Coefficient, F_a	1.0
Site Coefficient, F_v	1.5
Site Coefficient, F_{pga}	1.0

3.5.2 Liquefaction Potential and Seismic Settlement

The Site is located within an area mapped as a Liquefaction Hazard Zone by the CDMG (1998). The 2016 CBC requires that liquefaction potential evaluations for soil Site Class D through F be developed based on either a site-specific study taking into account soil amplification effects or using mapped peak ground accelerations (PGA) adjusted for site effects (F_{PGA}), PGA_M. The mapped PGA values represent maximum considered earthquake geometric mean (MCE_G) peak ground accelerations, rather than risk-targeted values. F_{PGA} and PGA values were evaluated using tools provided by the USGS. The PGA_M at the Site (0.63 g) was evaluated from the 2008 model for the United States developed by the USGS. Deaggregation of the seismic hazard indicates that the PGA is associated with an **M** 6.8 earthquake located approximately 9 km from the Site.

Liquefaction potential at the Site was assessed using procedures presented by Youd et al. (2001) for CPT data. The results of the liquefaction analysis are included in Appendix F. The evaluation indicated that liquefaction is likely to occur at the Site in thin layers/lenses generally below 20 feet bgs. The liquefiable layers above 26 to 27 feet bgs (depending on the thickness of mat foundation) will be removed during the basement excavation. The liquefaction-induced settlement was calculated using the procedure proposed by Idriss and Boulanger (2008). The total estimated liquefaction settlement is one-half of an inch or less. A differential settlement equal to one-half of the total settlement should be expected. The significance of the estimated seismic settlement is discussed in Section 4.1.2.

4.0 GEOTECHNICAL DESIGN RECOMMENDATIONS

4.1 Preliminary Foundation Design

4.1.1 Uplift Pressures

The proposed building includes two levels below grade. We have assumed that base of the excavation is approximately 20 feet bgs. This is approximately 3 feet below the current groundwater level. As a result, the foundation will be subjected to hydrostatic uplift pressures. The historic high groundwater table at the Site is approximately 6 feet bgs. The hydrostatic uplift pressures should be calculated based on the historic high groundwater table of 6 feet bgs.

4.1.2 Mat Foundations

Golder recommends that mat foundations bearing on the native soils be designed for a preliminary static allowable net bearing pressure of 4,500 psf. This bearing pressure assumes the mat will be founded on the medium dense to dense sand layer located approximately 20 feet bgs. The recommended bearing value is for equivalent gross loads and may be increased by one-third for wind, seismic, or other transient loading conditions.

The net bearing pressure does not include the weight of the mat foundation. However, the weight of soil excavated to construct the mat will be much greater than the weight of the mat.

The recommended allowable bearing pressure given above is based on a total settlement of one inch or less. A differential settlement equal to one-half of the total settlement can be expected. The City of Los Angeles limits the total allowable settlement (including seismic settlement) to 4 inches and the total allowable differential settlement (including seismic settlement) to 2 inches. The total and differential settlements of the mat foundation (including seismic) are less that the limits prescribed by the City of Los Angeles, so impacts regarding seismic settlement would be less than significant.

4.1.3 Modulus of Subgrade Reaction

The modulus of subgrade reaction, commonly required for the design of mat foundations, is not an intrinsic property of the soil since it also depends on the dimensions and stiffness of the mat and the applied stress level. The coefficient of subgrade reaction, k_1, for a 1-foot diameter plate may be taken as 2,000 kcf for design purposes. The coefficient of subgrade reaction for the mat foundation, k, can then be calculated using the equation:

$$k = k_1 \left(\frac{B+1}{2B}\right)^2$$

where B is the effective diameter of the mat's reaction area in feet. B may be estimated using the following equation:

$$B = \frac{4h}{\pi}\sqrt[3]{\frac{E}{E_S}}$$

where E and E_S are the elastic moduli of the concrete and soil, respectively, and h is the thickness of the mat in feet. Golder recommends that an E_S of 1,000 kips per square foot (ksf) be used to evaluate the modulus of subgrade reaction for the mat foundation.

Waterproofing on the base and sides of the mat foundation is recommended.

4.1.3.1 Lateral Resistance

A mat foundation located below grade may derive lateral load resistance from passive resistance along the vertical sides of the mat, friction acting on the base of the mat, or a combination of the two. An allowable passive resistance of 230 psf per foot of depth up to a maximum of 4,000 psf may be used for design. Golder recommends that the upper 1 foot of soil cover be neglected in the passive resistance calculations. An ultimate friction factor of 0.50 between the base of the mat foundation and the native soils can be used for sliding resistance using the dead load forces. Friction and passive resistance may be combined without reduction.

4.2 Walls

4.2.1 Basement Walls

The basement walls can be designed for an earth pressure represented by an equivalent fluid weight of 60 pounds per cubic foot (pcf). Walls below the groundwater table can be designed for a total earth and water pressure represented by an equivalent fluid weight of 90 pcf. The basement walls should be backfilled with granular soils. The fine fraction of the soil should have a liquid limit of 25 or less and a plasticity index of 12 or less. The soil should be uniformly graded with no greater than 30 percent of the particles passing the No. 200 sieve and no particles greater than 6 inches in dimension.

Under earthquake loading, basement retaining walls will be subjected to an additional lateral force equal to $14H^2$ pounds per linear foot of wall, where H is the height of the wall in units of feet. This force should be applied at a point located 0.6H above the base of the wall and it acts in addition to the static lateral pressures discussed above.

Waterproofing of basement walls is recommended to prevent moisture intrusion and water seepage through the walls due to the shallow groundwater table. In addition, a drainage layer should be placed against the wall above the groundwater table. The drainage layer may consist of a geosynthetic drain placed against the basement wall.

4.2.2 Retaining Walls

Active earth pressures may be used for deign of retaining walls that are free to rotate at least 0.1 percent of the wall height. The active earth pressures can be computed using an equivalent fluid weight of 35 pcf. Retaining walls restrained against rotation should be designed for the higher at-rest earth pressure conditions. For design purposes, the at-rest earth pressure exerted on retaining walls can be taken as that exerted by an equivalent fluid weight of 60 pcf. These recommended values do not include compaction-, truck-, or building-induced wall pressures or water pressures (see below). Additional loads on retaining walls may be imposed by surcharges. Golder should be contacted when development plans are finalized for review of wall, backfill, and surcharge conditions on a case-by-case basis.

Care must be taken during compaction operations not to overstress the retaining wall. Heavy construction equipment should be kept at least 3 feet away from the wall while the backfill soils are being placed. Hand-operated compaction equipment should be used to compact the backfill soils within the 3-foot-wide zone adjacent to the walls. Soil at the toes of retaining walls should be in place and compacted prior to backfilling behind the walls.

Under earthquake loading, retaining walls will be subjected to an additional lateral force equal to $14H^2$ pounds per linear foot of wall, where H is the height of the wall in units of feet. This force should be applied at a point located 0.6H above the base of the wall and it acts in addition to the static lateral pressures discussed above.

The recommended lateral earth pressures provided herein assume that adequate drainage is provided behind the walls to prevent the buildup of hydrostatic pressures. Walls should be provided with backdrains to prevent the buildup of hydrostatic pressure behind the walls. Backdrains could consist of a 2-foot wide zone of Caltrans Class 2 permeable material located immediately behind the wall and extending to within 1 foot of the ground surface. A perforated pipe could be installed at the base of the backdrain and sloped to discharge to a suitable collection point. Alternatively, commercially available synthetic drainage layers could be used for drainage of the wall backfill. The synthetic manufacturer’s recommendations should be followed in the installation of synthetic drainage layers or backdrains.

4.3 Soil Corrosivity

Geotechnical Professionals, Inc. tested one soil sample for corrosion. Based on Caltrans guidelines for structural elements (Caltrans, 2012), the Site soils are corrosive. A corrosive environment is defined by either a chloride content greater than 500 ppm, a sulfate content greater than 1,000 ppm, or a pH less than 5.5. The test indicated the soils had a higher chloride content and sulfate content than the Caltrans defined minimums. Similar corrosive soils should be expected at the Site. Corrosivity testing of on-Site soils should be performed during final design. Type V cement should be used for concrete in contact with the existing on-Site corrosive soils.

Golder recommends that the concrete mix design be reviewed by a qualified corrosion engineer to evaluate the general corrosion potential at the Site. Buried metallic structures and elements are recommended to have corrosion protection designed by a qualified corrosion engineer.

5.0 CONSTRUCTION CONSIDERATIONS

5.1 Existence of Unsuitable Soils

Geotechnical Professionals, Inc. performed an expansion index test on one bulk soil sample. The expansion index value was 31. According to the 1997 Uniform Building Code, an expansion index of less than 50 indicates the soil has a low expansion potential. The on-Site soils should be tested for expansion during final design.

Because of the low expansion potential, Golder does not recommend that expansion pressures on the basement walls be included in the wall design.

5.2 Excavations

Golder assumes that the depth of the excavation will be approximately 18 to 20 feet bgs. The borings performed at the Site were advanced using a track-mounted hollow stem auger drill rig. Drilling was completed with low effort through the existing native alluvium. Therefore, conventional earth moving equipment (i.e., scrapers, dozers, excavators) will be capable of performing a portion of the excavations required for the development. All surface water should be diverted away from excavations.

Basement excavations should be sloped no steeper than 1.5H:1V (horizontal:vertical).

5.3 Shoring

If the basement excavations cannot be sloped, shoring can be used to support the sides of the excavations. Cantilever and tied-back shoring systems should be designed to resist lateral earth pressures calculated as an equivalent fluid weighing 35 pcf. A vertical surcharge load of 250 psf should be applied to the ground surface immediately behind the shoring system to represent construction and street traffic.

An allowable passive earth pressure of 230 psf per foot of depth below the bottom of the excavation should be used for design of the shoring system. The allowable passive pressure can be assumed to act over two times the concreted pile diameter or the pile spacing, whichever is less. For piles spaced closer than three diameters, a reduction in the allowable passive earth pressure may be necessary. Golder recommends that the upper 1 foot below the bottom of the excavation be neglected in the passive resistance calculations. The passive pressure should not exceed 4,000 psf.

The basement excavation is likely to extend into the groundwater table. Groundwater control during construction should be anticipated. In the silt and clay soils, groundwater control may be achieved through the use of sumps and local pumps. Dewatering wells may be required to locally lower the groundwater table in the sand layer. Because the soil below a depth of 17 feet is primarily sand with little fines, the influence zone around a dewatering well will be relatively narrow and the depth of dewatering will be less than 5 feet. As a result, the potential for dewatering induced settlement impacting adjacent structures is considered low.

Movement of shoring walls is a function of many factors including the soil and groundwater conditions, changes in groundwater level, the depth and shape of the excavation, type and stiffness of the wall and its supports, methods of construction of the wall and adjacent facilities, surcharge loads, and the duration of wall exposure among others (Clough and O'Rourke, 1990). Typical horizontal wall movements in these types of soils available in the literature tend to average about 0.2% of the wall height (Clough and O'Rourke, 1990) for walls with good workmanship. The range of possible horizontal wall movements is approximately 0.5 inches to 2.5 inches. Typical vertical movements behind the wall in these types of soils available in the literature tend to average about 0.15% of the wall height (Clough and O'Rourke, 1990) for walls with good workmanship. Movements are largest immediately behind the wall. The movements are typically minimal at a distance beyond the wall equal to the depth of the excavation.

6.0 LIMITATIONS

This report has been prepared for the proposed development at the 13450 West Maxella Avenue in Marina del Rey, California. The findings, conclusions, and recommendations presented in this report were prepared in a manner consistent with that level of care and skill ordinarily exercised by other members of the geotechnical engineering profession currently practicing under similar conditions subject to the time limits and financial, physical, and other constraints applicable to the scope of work. No warranty, expressed or implied, is made. Appendix G contains further information regarding the proper use and interpretation of this geotechnical report.

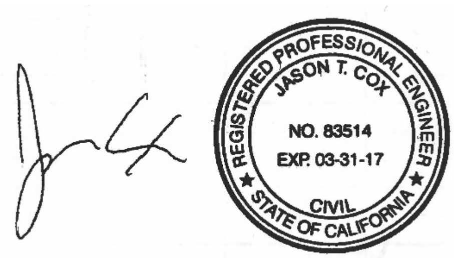
The Owner has the responsibility to see that all parties to the project, including the designer, contractor, subcontractors, etc., are made aware of this report in its entirety. This report contains information that may be useful in the preparation of contract specifications and contractor cost estimates. However, this report is not written as a specification document and may not contain sufficient information for this use without proper modification.

7.0 CLOSING

The preliminary geotechnical recommendations contained herein are based on Golder's current understanding of the proposed project. If changes are made to the proposed project, then it will be necessary for Golder to review this report and make changes accordingly.

Golder appreciates the opportunity to perform this study. If there are any questions regarding this report, please contact the undersigned.

GOLDER ASSOCIATES INC.



Jason Cox, PE
Project Engineer

Ryan Hillman, PE
Senior Engineer

8.0 REFERENCES

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California Division of Mines and Geology (CDMG), 1998, Seismic Hazard Zone Report for the Venice 7.5 – Minute Quadrangle, Los Angeles County, California.

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FIGURES

CLIENT
RELATED CALIFORNIA
IRVINE, CALIFORNIA

PROJECT
MARINA MARKETPLACE - PHASE III
MARINA DEL REY, CALIFORNIA

CONSULTANT

Golder Associates

YYYY-MM-DD	2014-09-11
PREPARED	KJK
DESIGN	AA
REVIEW	AA
APPROVED	AH

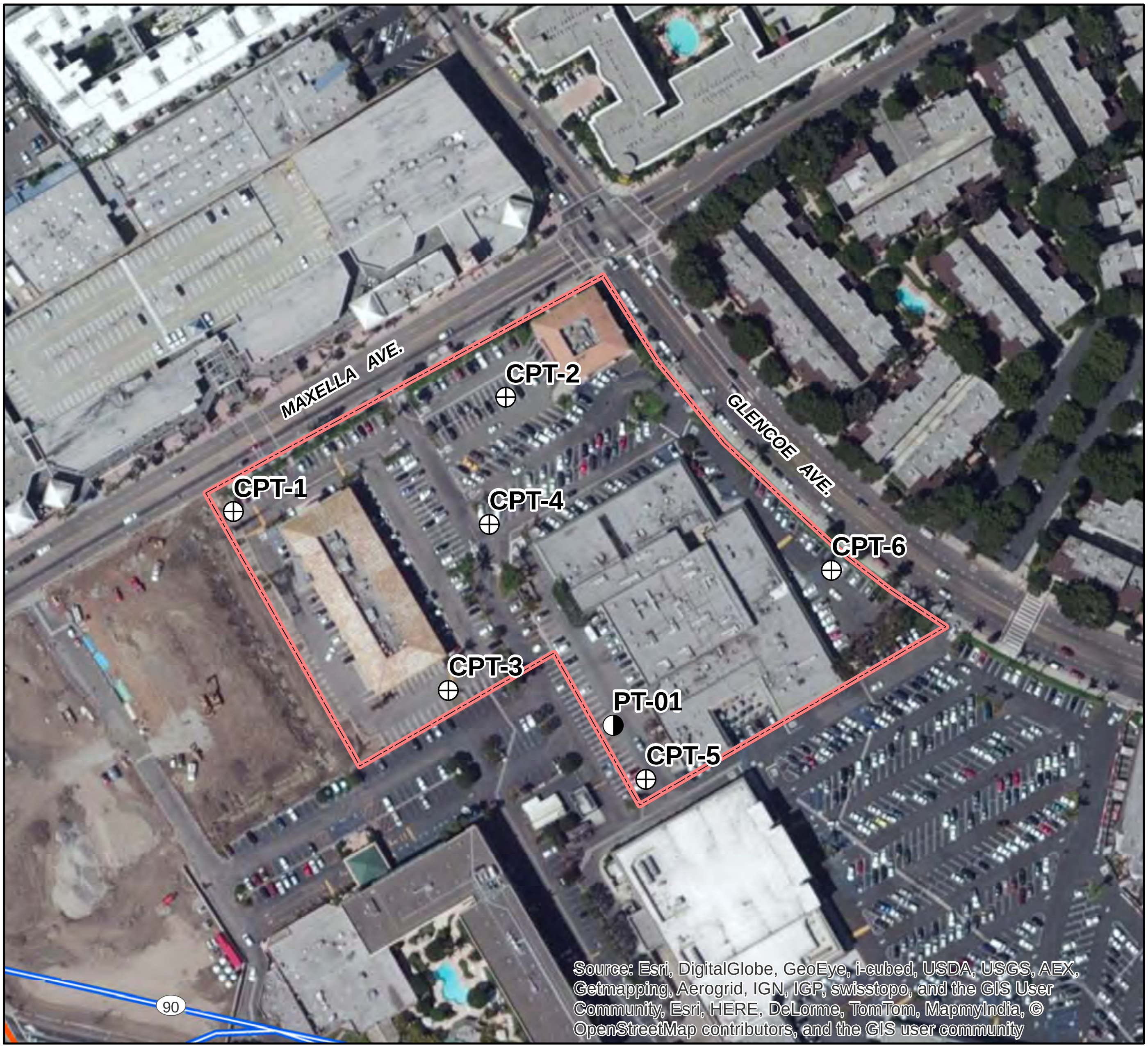
TITLE
SITE LOCATION MAP

PROJECT No.	CONTROL	Rev.	FIGURE
1403929			1

Path: M:\Projects\RelatedCalifornia\MarinaMarketplace\99_PROJECTS\1403929_MarinaMarketPlace-PhaseIII\02_PRODUCTION\1400329PHIII-01.mxd

IF THIS MEASUREMENT DOES NOT MATCH WHAT IS SHOWN, THE SHEET HAS BEEN MODIFIED FROM: ANSI A

0 1 in



LEGEND

APPROXIMATE LIMITS OF PROJECT

CPT-6 DESIGNATION AND APPROXIMATE LOCATION OF CPT ADVANCED SEPTEMBER 25, 2014

PT-01 PERCOLATION TEST PERFORMED DECEMBER 17, 2014

180 90 0 180
1 " = 180 ' FEET

CLIENT
RELATED CALIFORNIA
IRVINE, CALIFORNIA

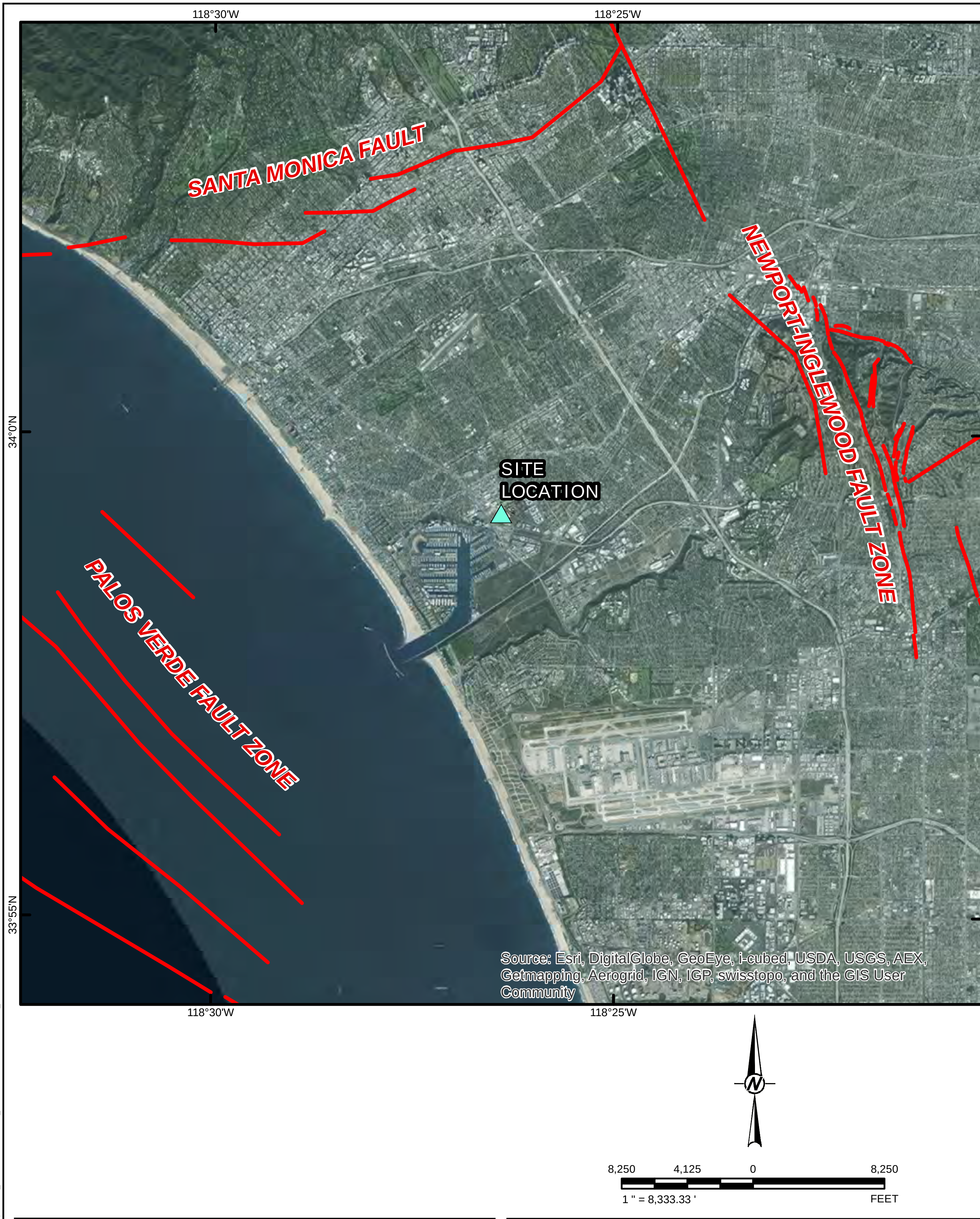
CONSULTANT
Golder Associates

YYYY-MM-DD	2015-01-19
PREPARED	KJK
DESIGN	AA
REVIEW	AA
APPROVED	AH

PROJECT
MARINA MARKETPLACE - PHASE III
MARINA DEL REY, CALIFORNIA

TITLE
BORING LOCATION MAP

PROJECT No.	CONTROL	Rev.	FIGURE
1403929			2



CLIENT
RELATED CALIFORNIA
IRVINE, CALIFORNIA

CONSULTANT

Golder Associates

YYYY-MM-DD	2014-09-11
PREPARED	KJK
DESIGN	AA
REVIEW	AA
APPROVED	AH

PROJECT
MARINA MARKETPLACE - PHASE III
MARINA DEL REY, CALIFORNIA

TITLE
FAULT MAP

PROJECT No.	CONTROL	Rev.	FIGURE
1403929			3

APPENDIX A

COUNTY OF LOS ANGELES PUBLIC HEALTH DEPARTMENT PERMIT

ENVIRONMENTAL HEALTH

Drinking Water Program

COUNTY OF LOS ANGELES Public Health

5050 Commerce Drive, Baldwin Park, CA 91706
Telephone: (626) 430-5420 • Facsimile: (626) 813-3013 • Email: waterquality@ph.lacounty.gov
http://publichealth.lacounty.gov/eh/ep/dw/dw_main.htm

Well Permit Approval

TO BE COMPLETED BY APPLICANT:

WORK SITE ADDRESS	CITY	ZIP	EMAIL ADDRESS FOR WELL PERMIT APPROVAL
13450 Maxella Ave	Marina del Rey	90262	[redacted]

NOTICE:

- WORK PLAN APPROVALS ARE VALID FOR 180 DAYS. 30 DAY EXTENSIONS OF WORK PLAN APPROVALS ARE CONSIDERED ON AN INDIVIDUAL (CASE-BY-CASE) BASIS AND MAY BE SUBJECT TO ADDITIONAL PLAN REVIEW FEES (HOURLY RATE AS APPLICABLE).
- WORK PLAN MODIFICATIONS MAY BE REQUIRED IF WELL AND GEOLOGIC CONDITIONS ENCOUNTERED AT THE SITE INSPECTION ARE FOUND TO DIFFER FROM THE SCOPE OF WORK PRESENTED TO THE DEPARTMENT OF PUBLIC HEALTH—DRINKING WATER PROGRAM.
- THIS WELL PERMIT APPROVAL IS LIMITED TO COMPLIANCE WITH THE CALIFORNIA WELL STANDARDS AND THE LOS ANGELES COUNTY CODE AND DOES NOT GRANT ANY RIGHTS TO CONSTRUCT, RENOVATE, OR DECOMMISSION ANY WELL. THE APPLICANT IS RESPONSIBLE FOR SECURING ALL OTHER NECESSARY PERMITS SUCH AS WATER RIGHTS, PROPERTY RIGHTS, COASTAL COMMISSION APPROVALS, USE COVENANTS, ENCROACHMENT PERMISSIONS, UTILITY LINE SETBACKS, CITY/COUNTY PUBLIC WORKS RIGHTS OF WAY, ETC.
- ALL FIELD WORK MUST BE CONDUCTED UNDER THE DIRECT SUPERVISION OF A PROFESSIONAL GEOLOGIST LICENSED IN THE STATE OF CALIFORNIA.
- THIS PERMIT IS NOT COMPLETE UNTIL ALL OF THE FOLLOWING REQUIREMENTS ARE SIGNED BY THE DEPUTY HEALTH OFFICER. WORK SHALL NOT BE INITIATED WITHOUT A WORK PLAN APPROVAL STAMPED BY THE DEPARTMENT OF PUBLIC HEALTH—DRINKING WATER PROGRAM.
- **NOTIFY THE DRINKING WATER PROGRAM BY EMAIL 3 BUSINESS DAYS BEFORE WORK IS SCHEDULED TO BEGIN.**

Juan Rodriguez 626-430-5386 or [redacted]

TO BE COMPLETED BY DEPARTMENT OF PUBLIC HEALTH—DRINKING WATER PROGRAM:

☐ WORK PLAN INCOMPLETE; SUBMIT THE FOLLOWING:	☑ WORK PLAN APPROVED	DATE: 9/16/14
	Los Angeles County Drinking Water stamp REGISTERED ENVIRONMENTAL HEALTH SPECIALIST, COUNTY OF LOS ANGELES, CALIFORNIA REHS NO. 6330 Juan Rodriguez	ADDITIONAL APPROVAL CONDITIONS: On 9/11/14 $780.00 was paid for Permit #893507 to advance 6 soil borings into groundwater at above site.

☐ ANNULAR SEAL FINAL INSPECTION REQUIRED		☐ WELL COMPLETION LOG REQUIRED	
DATE ACCEPTED:	REHS signature	DATE ACCEPTED:	REHS signature

☐ WATER QUALITY—BACTERIOLOGICAL STANDARDS REQUIRED		☐ WATER QUALITY—CHEMICAL STANDARDS REQUIRED	
DATE ACCEPTED:	REHS signature	DATE ACCEPTED:	REHS signature

☐ WATER SUPPLY YIELD REQUIRED		☐ OTHER REQUIREMENT	
DATE ACCEPTED:	REHS signature	DATE ACCEPTED:	REHS signature

Revised: October 2012

APPENDIX B

CONE PENETRATION TEST RESULTS

SUMMARY

OF

CONE PENETRATION TEST DATA

Project:

13450 Maxella Avenue
Marina Del Rey, CA
September 25, 2014

Prepared for:

Mr. Tony Augello
Golder Associates Inc.
230 Commerce, Ste 200
Irvine, CA 92602
Office (714) 508-4400 / Fax (714) 508-4401

Prepared by:

KEHOE TESTING & ENGINEERING
5415 Industrial Drive
Huntington Beach, CA 92649-1518
Office (714) 901-7270 / Fax (714) 901-7289
www.kehoetesting.com

TABLE OF CONTENTS

SUMMARY

OF

CONE PENETRATION TEST DATA

1. INTRODUCTION

This report presents the results of a Cone Penetration Test (CPT) program carried out for the project located at 13450 Maxella Avenue in Marina Del Rey, California. The work was performed by Kehoe Testing & Engineering (KTE) on September 25, 2014. The scope of work was performed as directed by Golder Associates Inc. personnel.

2. SUMMARY OF FIELD WORK

The fieldwork consisted of performing CPT soundings at six locations to determine the soil lithology. Groundwater measurements and hole collapse depths provided in **TABLE 2.1** are for information only. The readings indicate the apparent depth to which the hole is open and the apparent water level (if encountered) in the CPT probe hole at the time of measurement upon completion of the CPT. KTE does not warrant the accuracy of the measurements and the reported water levels may not represent the true or stabilized groundwater levels.

LOCATION	DEPTH OF CPT (ft)	COMMENTS/NOTES:
CPT-1	50	Groundwater @ 17.0 ft
CPT-2	26	Refusal, groundwater @ 17.0 ft
CPT-3	50	Refusal, groundwater @ 17.0 ft
CPT-4	60	Refusal, hole open to 1.0 ft (dry)
CPT-5	26	Refusal, hole open to 19.0 ft (dry)
CPT-6	33	Refusal, groundwater @ 17.5 ft

TABLE 2.1 - Summary of CPT Soundings

3. FIELD EQUIPMENT & PROCEDURES

The CPT soundings were carried out by **KTE** using an integrated electronic cone system manufactured by Vertek. The CPT soundings were performed in accordance with ASTM standards (D5778). The cone penetrometers were pushed using a 30-ton CPT rig. The cone used during the program was a 15 cm^2 cone and recorded the following parameters at approximately 2.5 cm depth intervals:

- Cone Resistance (qc)
- Sleeve Friction (fs)
- Dynamic Pore Pressure (u)
- Inclination
- Penetration Speed

The above parameters were recorded and viewed in real time using a laptop computer. Data is stored at the KTE office for future analysis and reference. A complete set of baseline readings was taken prior to each sounding to determine temperature shifts and any zero load offsets. Monitoring base line readings ensures that the cone electronics are operating properly.

4. CONE PENETRATION TEST DATA & INTERPRETATION

The Cone Penetration Test data is presented in graphical form in the attached Appendix. These plots were generated using the CPeT-IT program. Penetration depths are referenced to ground surface. The soil classification on the CPT plots is derived from the attached CPT Classification Chart (Robertson) and presents major soil lithologic changes. The stratigraphic interpretation is based on relationships between cone resistance (qc), sleeve friction (fs), and penetration pore pressure (u). The friction ratio (Rf), which is sleeve friction divided by cone resistance, is a calculated parameter that is used along with cone resistance to infer soil behavior type. Generally, cohesive soils (clays) have high friction ratios, low cone resistance and generate excess pore water pressures. Cohesionless soils (sands) have lower friction ratios, high cone bearing and generate little (or negative) excess pore water pressures.

Tables of basic CPT output from the interpretation program CPeT-IT are provided for CPT data averaged over one foot intervals in the Appendix. Spreadsheet files of the averaged basic CPT output and averaged estimated geotechnical parameters are also included for use in further geotechnical analysis. We recommend a geotechnical engineer review the assumed input parameters and the calculated output from the CPeT-IT program. A summary of the equations used for the tabulated parameters is provided in the Appendix.

It should be noted that it is not always possible to clearly identify a soil type based on qc, fs and u. In these situations, experience, judgement and an assessment of the pore pressure data should be used to infer the soil behavior type.

If you have any questions regarding this information, please do not hesitate to call our office at (714) 901-7270.

Sincerely,

KEHOE TESTING & ENGINEERING

Richard W. Koester, Jr.
General Manager

09/29/14-kk-5210

APPENDIX

Kehoe Testing and Engineering
714-901-7270
rich@kehoetesting.com
www.kehoetesting.com

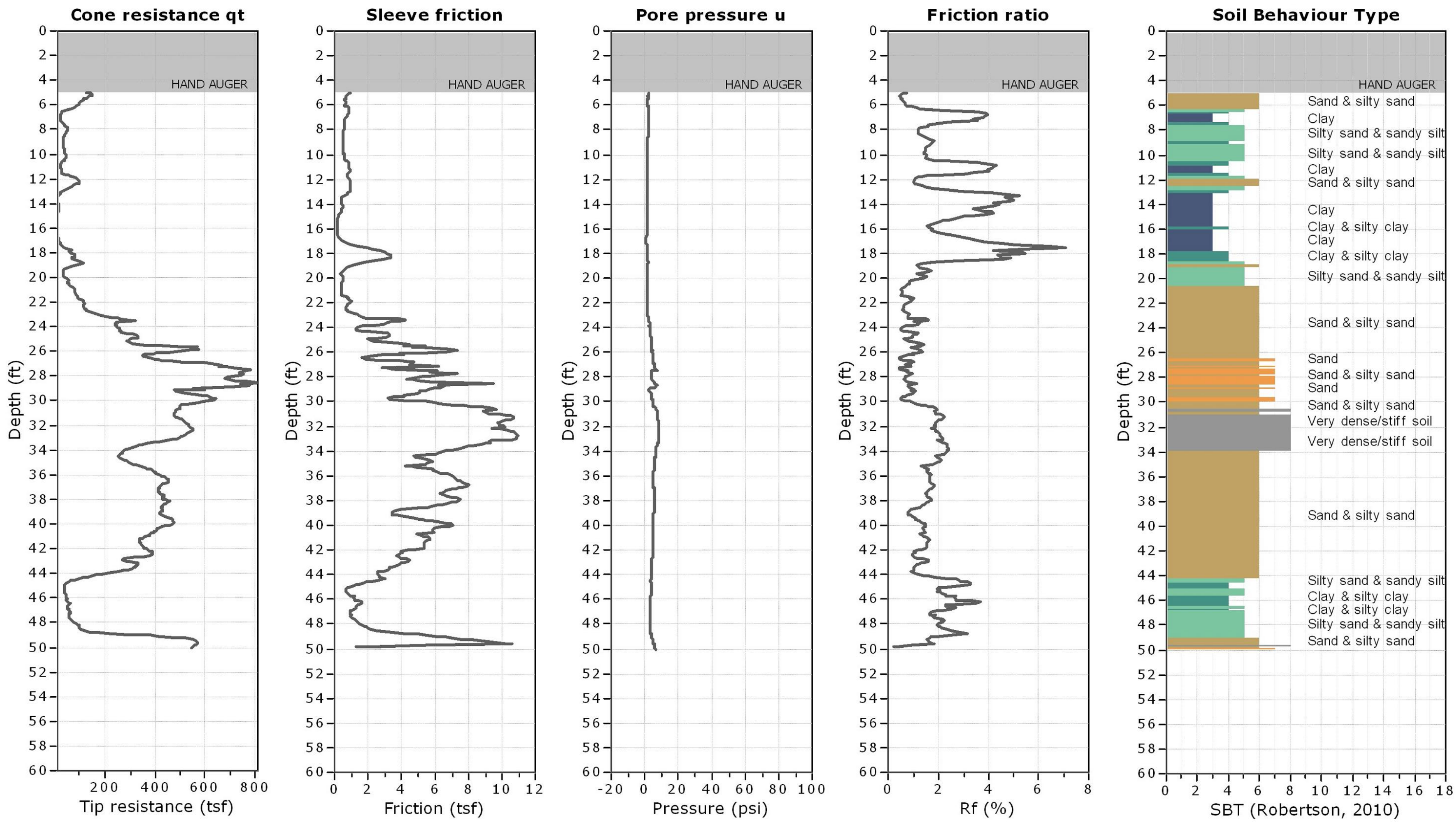
CPT: CPT-1

Total depth: 50.02 ft, Date: 9/25/2014

Cone Type: Vertek

Project: Golder Associates, Inc.

Location: 13450 Maxella Ave. Marina Del Rey, CA



Kehoe Testing and Engineering
714-901-7270
rich@kehoetesting.com
www.kehoetesting.com

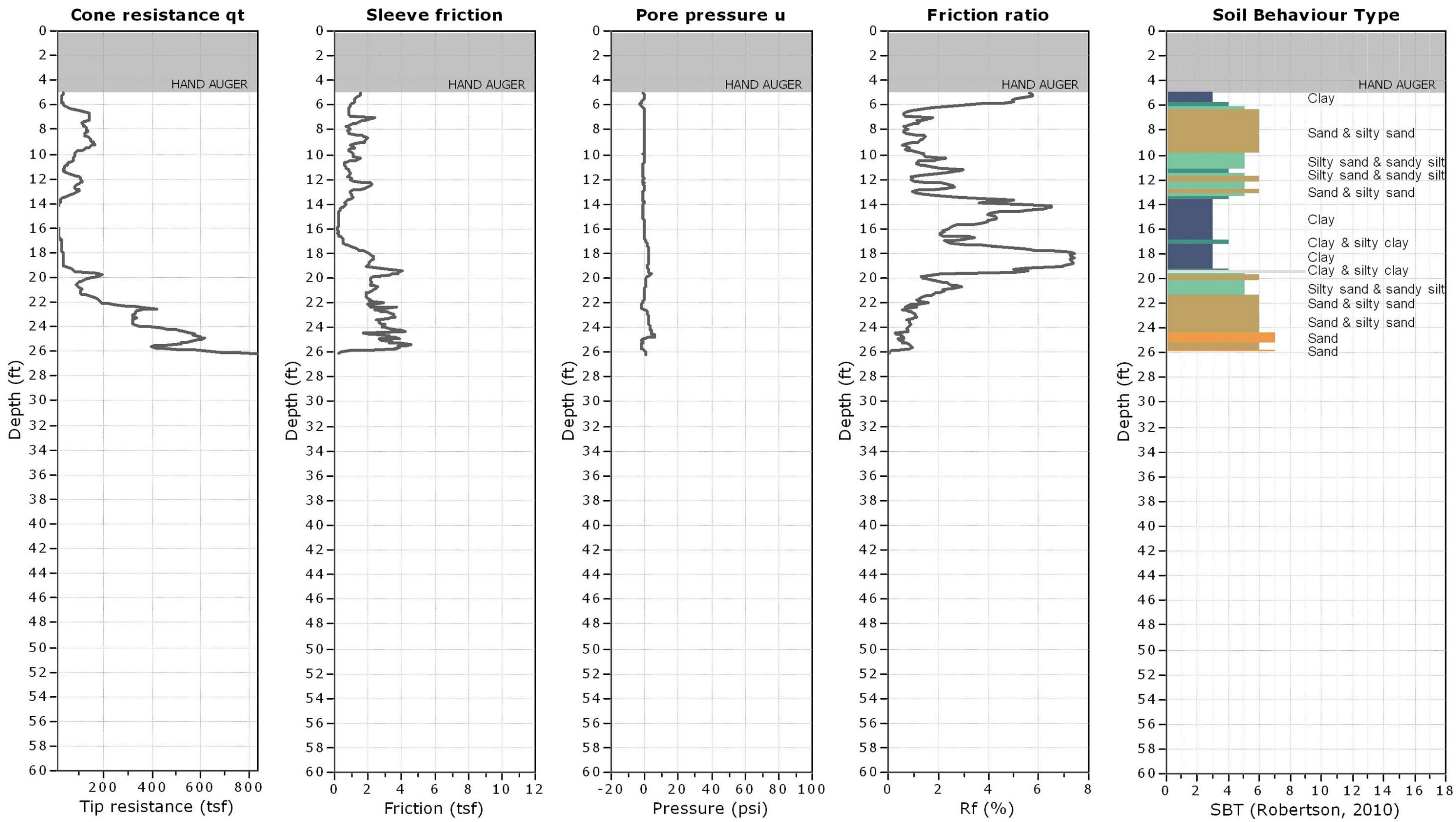
CPT: CPT-2

Total depth: 26.18 ft, Date: 9/25/2014

Cone Type: Vertek

Project: Golder Associates, Inc.

Location: 13450 Maxella Ave. Marina Del Rey, CA



Kehoe Testing and Engineering
714-901-7270
rich@kehoetesting.com
www.kehoetesting.com

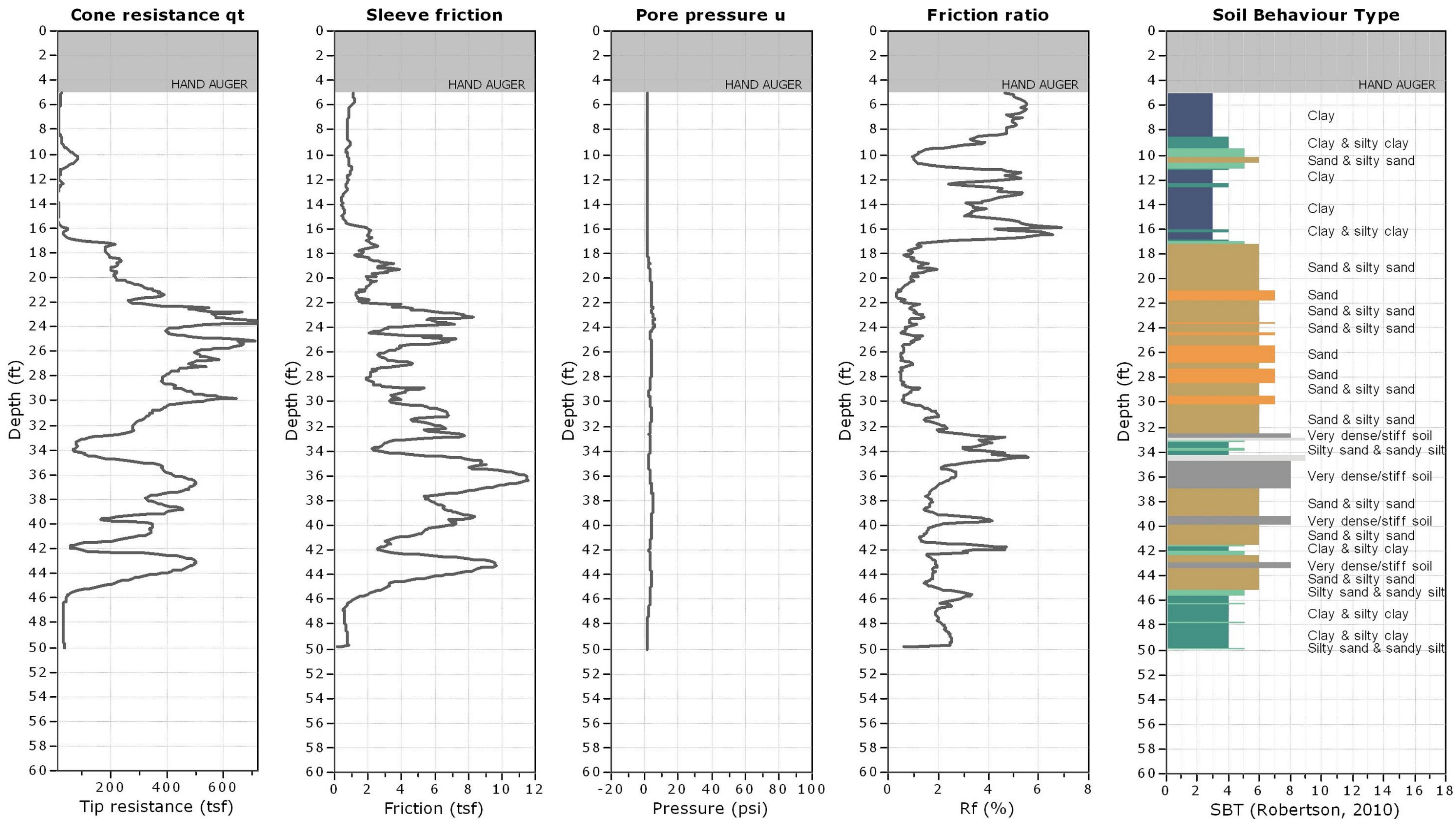
CPT: CPT-3

Total depth: 50.06 ft, Date: 9/25/2014

Cone Type: Vertek

Project: Golder Associates, Inc.

Location: 13450 Maxella Ave. Marina Del Rey, CA



Kehoe Testing and Engineering
714-901-7270
rich@kehoetesting.com
www.kehoetesting.com

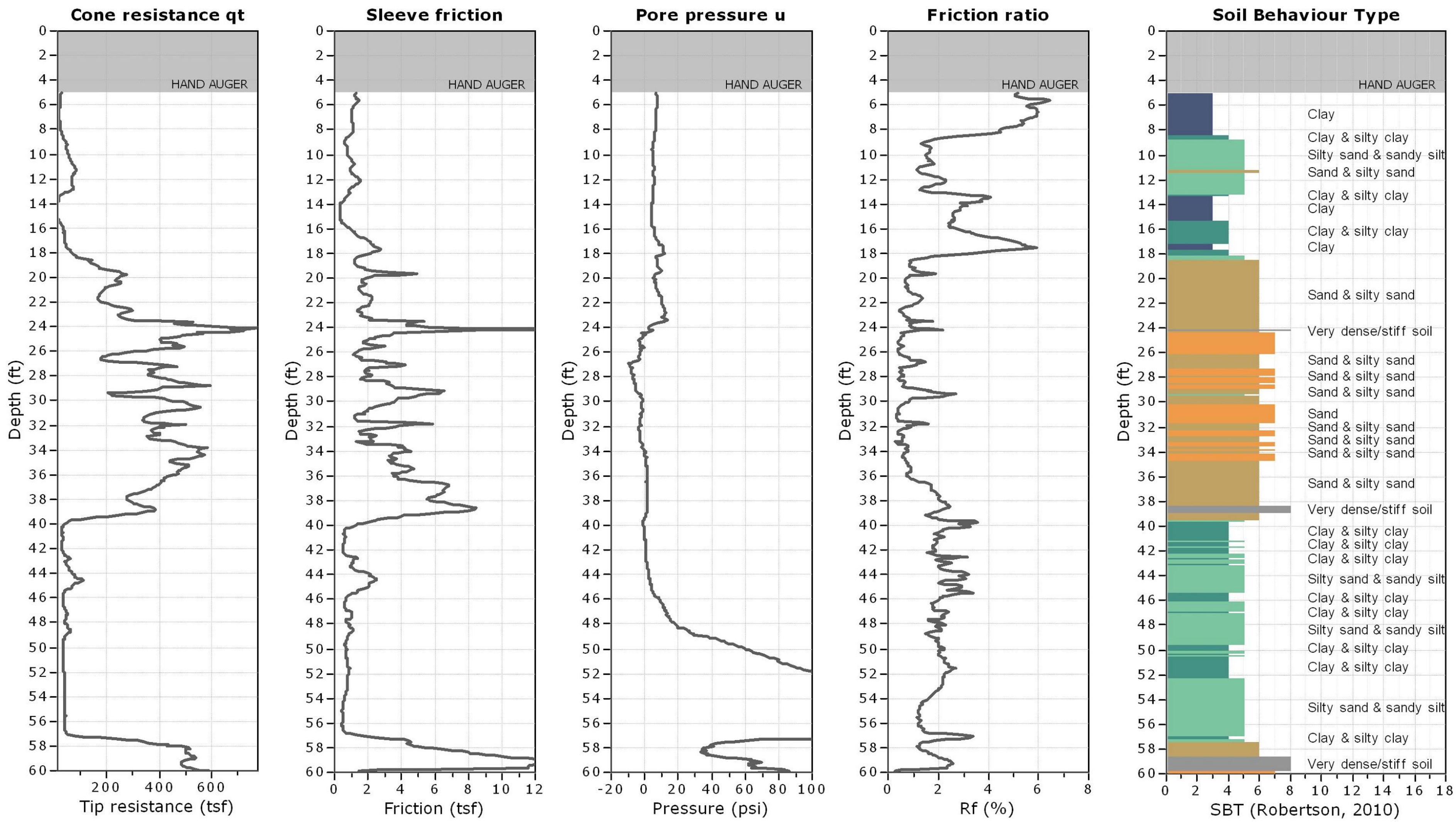
CPT: CPT-4

Project: **Golder Associates, Inc.**

Location: **13450 Maxella Ave. Marina Del Rey, CA**

Total depth: 60.03 ft, Date: 9/25/2014

Cone Type: Vertek



Kehoe Testing and Engineering
714-901-7270
rich@kehoetesting.com
www.kehoetesting.com

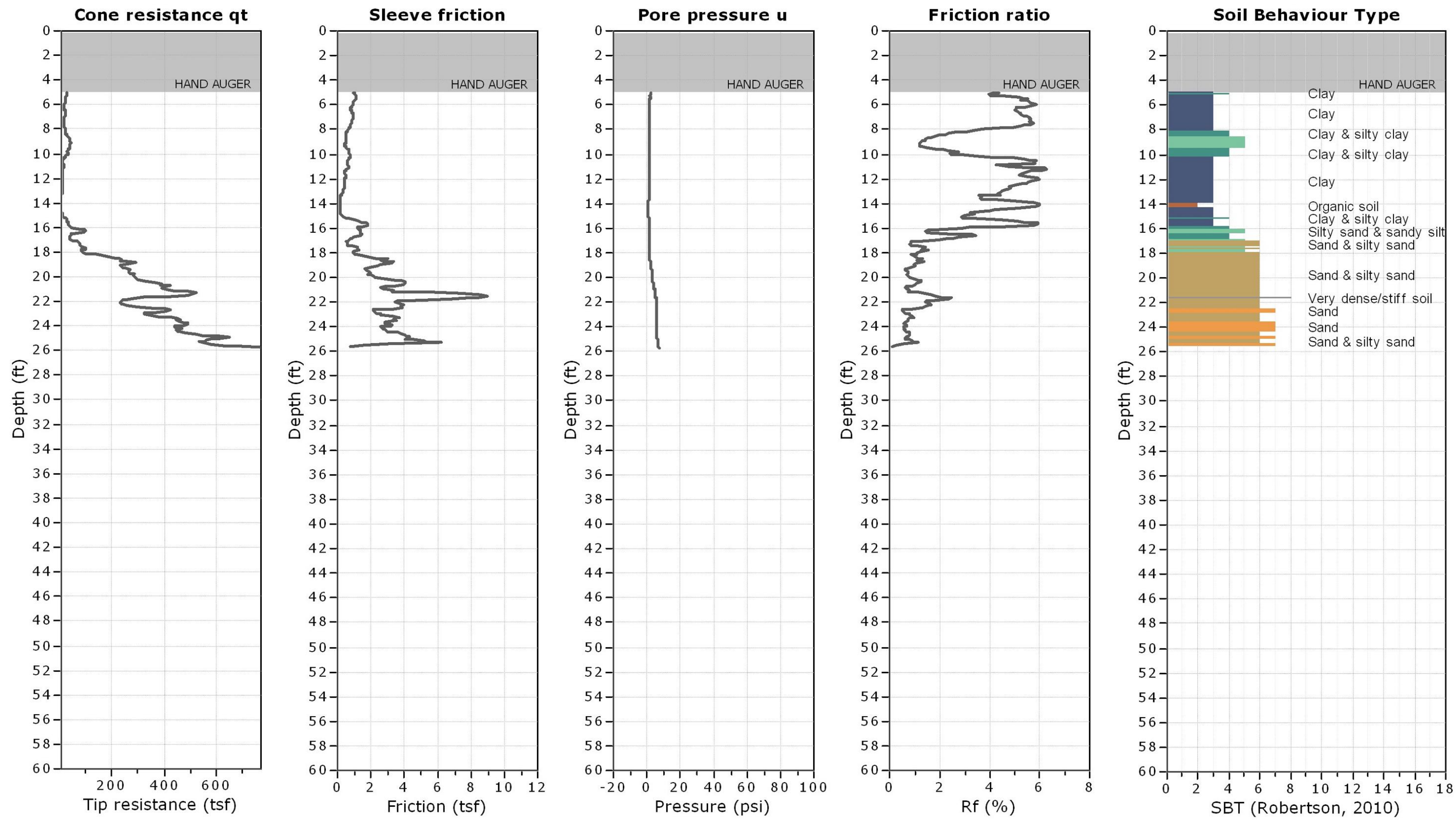
CPT: CPT-5

Total depth: 25.72 ft, Date: 9/25/2014

Cone Type: Vertek

Project: Golder Associates, Inc.

Location: 13450 Maxella Ave. Marina Del Rey, CA



Kehoe Testing and Engineering
714-901-7270
rich@kehoetesting.com
www.kehoetesting.com

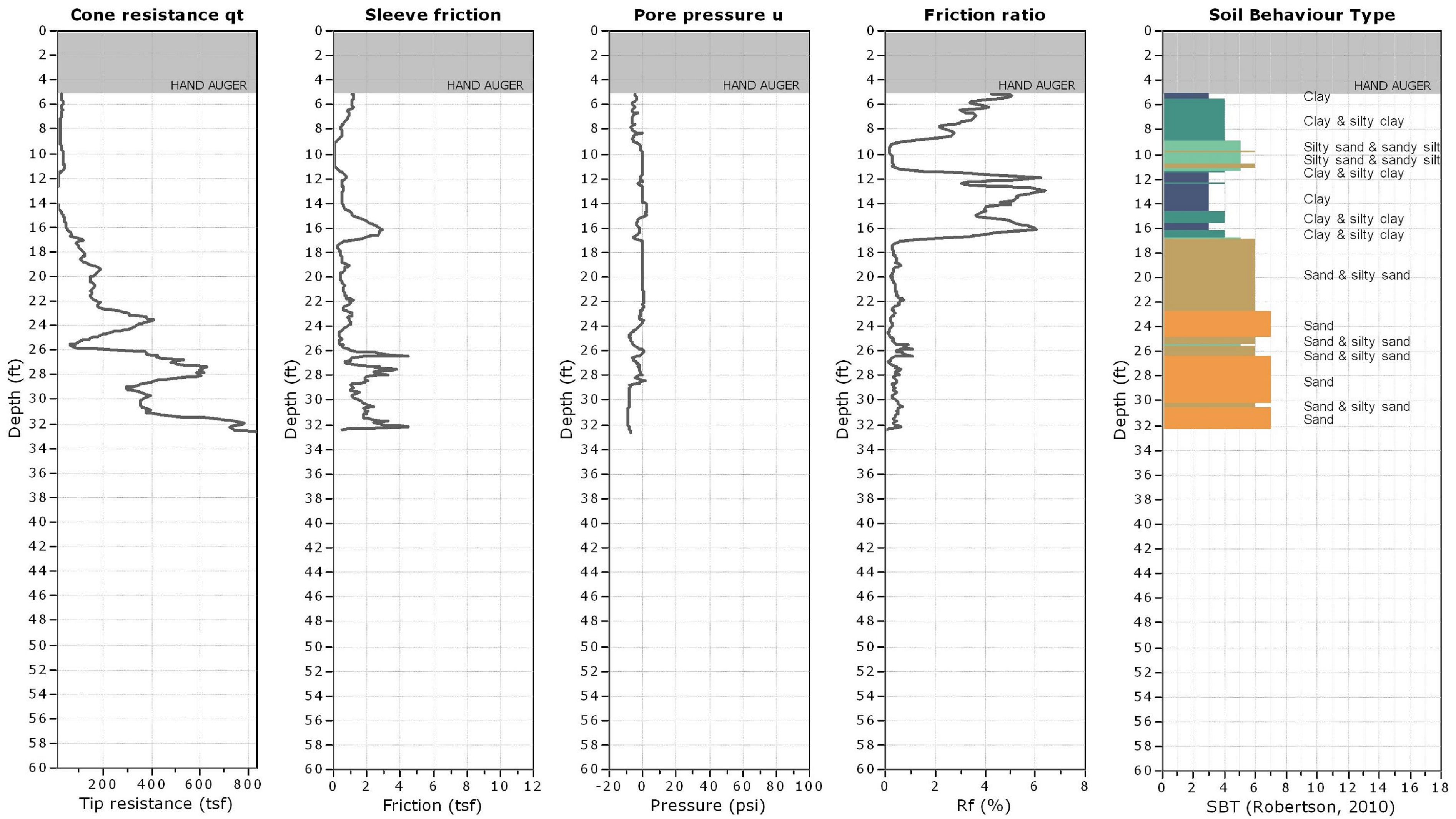
CPT: CPT-6

Project: Golder Associates, Inc.

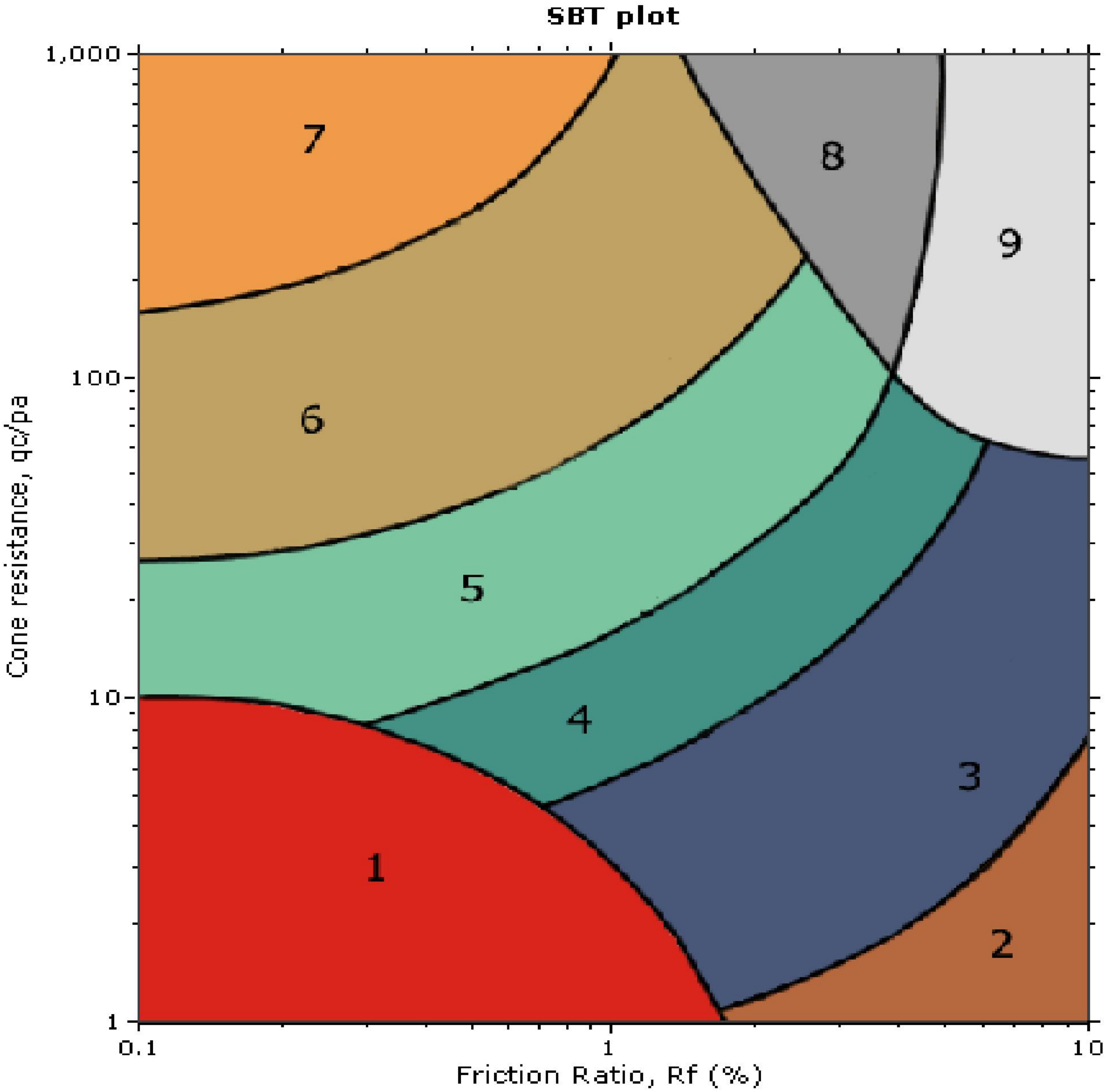
Location: 13450 Maxella Ave. Marina Del Rey, CA

Total depth: 32.60 ft, Date: 9/25/2014

Cone Type: Vertek



Kehoe Testing and Engineering
714-901-7270
rich@kehoetesting.com
www.kehoetesting.com



SBT legend

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

	CPT-1	In situ data										Basic output data								
Depth (ft)	qc (tsf)	fs (tsf)	u (psi)	Other	qt (tsf)	Rf(%)	SBT	Ic SBT	ã (pcf)	ó,v (tsf)	u0 (tsf)	ó',vo (tsf)	Qt1	Fr (%)	Bq	SBTn	n	Cn	Ic	Qtn
1	24.4	0.87	1.01	-0.13	24.4124	3.5638	4	2.75296	117.0452	0.05852	0	0.0585	416.14	3.5723	0.003	8	0.6816	7.1937	2.1699	165.5739
2	17.7	0.74	0.82	-0.04	17.71	4.1784	3	2.90435	115.0782	0.11606	0	0.1161	151.59	4.206	0.0034	9	0.7654	5.4283	2.3861	90.26102
3	12.6	0.29	1.15	0.17	12.6141	2.299	4	2.86896	107.3962	0.16976	0	0.1698	73.305	2.3304	0.0067	5	0.7697	4.0895	2.3909	48.09612
4	30.6	0.62	1.55	0.54	30.619	2.0249	4	2.52272	115.1189	0.22732	0	0.2273	133.7	2.04	0.0037	5	0.6909	2.8937	2.1779	83.11451
5	122.7	0.93	2.2	1.15	122.727	0.7578	6	1.78457	121.4718	0.28806	0	0.2881	425.05	0.7596	0.0013	6	0.4682	1.839	1.586	212.7942
6	87.1	0.6	1.96	1.47	87.124	0.6887	6	1.8803	117.4295	0.34677	0	0.3468	250.24	0.6914	0.0016	6	0.5085	1.7635	1.6848	144.6266
7	17.4	0.69	2.35	1.58	17.4288	3.959	3	2.89496	114.5272	0.40403	0	0.404	42.137	4.0529	0.0099	4	0.8729	2.3173	2.6353	37.28515
8	46.6	0.57	2.08	1.73	46.6255	1.2225	5	2.24563	115.5293	0.4618	0	0.4618	99.965	1.2347	0.0032	5	0.6577	1.7251	2.0638	75.26382
9	27.9	0.49	1.88	1.81	27.923	1.7548	4	2.51806	113.1723	0.51838	0	0.5184	52.865	1.788	0.0049	5	0.766	1.7273	2.3405	44.73709
10	39.1	0.58	1.8	1.95	39.122	1.4825	5	2.35646	115.2286	0.576	0	0.576	66.92	1.5047	0.0034	5	0.7209	1.5502	2.2147	56.47294
11	18.2	0.76	1.72	2.02	18.2211	4.171	3	2.89431	115.3427	0.63367	0	0.6337	27.755	4.3213	0.007	3	0.9315	1.6122	2.7591	26.79636
12	83.3	0.9	1.64	2.1	83.3201	1.0802	6	2.01197	120.2874	0.69381	0	0.6938	119.09	1.0892	0.0014	6	0.618	1.298	1.9298	101.3573
13	28.6	0.9	1.39	2.17	28.617	3.145	4	2.6652	117.6808	0.75265	0	0.7527	37.021	3.2299	0.0036	4	0.8705	1.3452	2.5844	35.42348
14	10.3	0.46	1.39	2.32	10.317	4.4587	3	3.10631	110.2816	0.80779	0	0.8078	11.772	4.8374	0.0105	3	1	1.3099	3.0633	11.77182
15	7.8	0.25	1.31	2.45	7.81603	3.1986	3	3.12146	105.1429	0.86037	0	0.8604	8.0845	3.5942	0.0136	3	1	1.2298	3.1174	8.08454
16	8.5	0.15	1.31	2.56	8.51603	1.7614	3	2.9537	101.6144	0.91117	0	0.9112	8.3462	1.9724	0.0124	3	1	1.1613	2.9648	8.34623
17	14.5	0.53	1.11	2.83	14.5136	3.6518	3	2.93582	112.1505	0.96725	0	0.9673	14.005	3.9125	0.0059	3	1	1.0939	2.947	14.00502
18	57.4	3.15	1.86	3.13	57.4228	5.4856	4	2.61732	128.5459	1.03152	0.0312	1.0003	56.373	5.586	0.0018	4	0.8932	1.0514	2.614	56.03606
19	90.3	0.99	1.7	3.14	90.3208	1.0961	6	1.9887	121.1815	1.09211	0.0624	1.0297	86.654	1.1095	0.0007	6	0.6569	1.018	1.9901	85.849
20	39.5	0.5	1.55	3.09	39.519	1.2652	5	2.31289	114.1673	1.1492	0.0936	1.0556	36.349	1.3031	0.0005	5	0.7876	1.0019	2.3301	36.33058
21	75	0.4	1.72	3.14	75.0211	0.5332	6	1.87586	114.0979	1.20625	0.1248	1.0815	68.256	0.5419	-1E-05	6	0.6214	0.9865	1.8906	68.82178
22	111.7	0.91	1.8	3.75	111.722	0.8145	6	1.83603	121.0836	1.26679	0.156	1.1108	99.439	0.8239	-2E-04	6	0.6085	0.9709	1.8531	101.3481
23	169.7	1.6	1.96	4.28	169.724	0.9427	6	1.7396	126.2326	1.3299	0.1872	1.1427	147.36	0.9502	-3E-04	6	0.574	0.9568	1.7584	152.2732
24	247.5	1.34	2.86	4.25	247.535	0.5413	6	1.45639	125.8555	1.39283	0.2184	1.1744	209.58	0.5444	-5E-05	6	0.4679	0.9524	1.4759	221.5443
25	300.8	1.8	4.01	4.36	300.849	0.5983	6	1.42355	128.4906	1.45708	0.2496	1.2075	247.95	0.6012	0.0001	6	0.4579	0.9413	1.4454	266.349
26	475.7	5.91	4.22	3.26	475.752	1.2422	6	1.54754	137.28	1.52572	0.2808	1.2449	380.93	1.2462	5E-05	6	0.5066	0.9209	1.5687	412.748
27	614.9	4.53	5.85	2.46	614.972	0.7366	7	1.29618	136.9875	1.59421	0.312	1.2822	478.38	0.7385	0.0002	7	0.4123	0.9239	1.3168	535.5511
28	729.7	5.21	4.31	2.83	729.753	0.7139	7	1.24554	137.28	1.66285	0.3432	1.3197	551.73	0.7156	-5E-05	7	0.3949	0.9165	1.2665	630.626
29	551.6	5.61	1.39	3.51	551.617	1.017	6	1.43985	137.28	1.73149	0.3744	1.3571	405.19	1.0202	-5E-04	6	0.4739	0.8888	1.4692	461.8756
30	637.7	5.48	4.59	3.54	637.756	0.8593	6	1.34459	137.28	1.80013	0.4056	1.3945	456.04	0.8617	-1E-04	6	0.4394	0.8858	1.3741	532.3667
31	476.3	9.74	7.68	4.25	476.394	2.0445	8	1.73479	137.28	1.86877	0.4368	1.432	331.38	2.0526	0.0002	6	0.594	0.8355	1.7752	374.69
32	536.1	10.11	8.37	4.28	536.202	1.8855	8	1.67983	137.28	1.93741	0.468	1.4694	363.59	1.8923	0.0003	6	0.5751	0.8279	1.7209	418.0325
33	493.8	10.86	8.23	4.38	493.901	2.1988	8	1.75552	137.28	2.00605	0.4992	1.5069	326.44	2.2078	0.0002	8	0.6079	0.8066	1.8024	374.9717
34	280.5	6.61	6.76	4.34	280.583	2.3558	6	1.90526	137.28	2.07469	0.5304	1.5443	180.35	2.3734	-2E-04	5	0.6746	0.7749	1.9728	203.9558
35	295.7	5.51	5.73	4.38	295.77	1.8629	6	1.80787	136.6352	2.14301	0.5616	1.5814	185.67	1.8765	-5E-04	6	0.6402	0.7732	1.8778	214.5569
36	411.4	7.03	5	4.49	411.461	1.7086	6	1.69849	137.28	2.21165	0.5928	1.6189	252.8	1.7178	-6E-04	6	0.5977	0.7756	1.7615	299.9767
37	420.1	7.4	5.49	4.55	420.167	1.7612	6	1.70512	137.28	2.28029	0.624	1.6563	252.3	1.7708	-6E-04	6	0.6031	0.7632	1.7712	301.4079
38	437.8	7.41	5.97	4.55	437.873	1.6923	6	1.68107	137.28	2.34893	0.6552	1.6937	257.14	1.7014	-5E-04	6	0.5966	0.7553	1.7493	310.8825
39	426.4	3.42	5.73	4.59	426.47	0.8019	6	1.4182	134.0381	2.41595	0.6864	1.7296	245.18	0.8065	-7E-04	6	0.4991	0.7825	1.489	313.6017
40	467.8	7.02	5.38	4.66	467.866	1.5004	6	1.62144	137.28	2.48459	0.7176	1.767	263.38	1.5084	-7E-04	6	0.5789	0.7432	1.6939	326.8555
41	368	5.66	5.27	4.66	368.065	1.5378	6	1.68572	137.28	2.55323	0.7488	1.8044	202.56	1.5485	-0.001	6	0.6104	0.7219	1.7719	249.3873
42	361	3.66	4.95	4.66	361.061	1.0137	6	1.54295	134.1283	2.62029	0.78	1.8403	194.77	1.0211	-0.001	6	0.559	0.7339	1.6325	248.6091
43	269.8	4.06	4.29	4.65	269.853	1.5045	6	1.756	134.177	2.68738	0.8112	1.8762	142.4	1.5197	-0.002	6	0.6491	0.6895	1.8644	174.1005
44	196	2.66	3.92	4.54	196.048	1.3568	6	1.80956	130.3037	2.75253	0.8424	1.9101	101.19	1.3761	-0.003	6	0.6789	0.6697	1.9383	122.3316
45	38.4	0.85	3.83	4.31	38.4469	2.2108	5	2.46874	117.9828	2.81152	0.8736	1.9379	18.388	2.3853	-0.017	4	0.9775	0.5535	2.7185	18.64111
46	45.1	1.2	3.44	4.18	45.1421	2.6583	4	2.46782	120.8975	2.87197	0.9048	1.9672	21.488	2.8389	-0.016	4	0.9754	0.5461	2.7095	21.81774
47	53.8	0.83	3.22	4.12	53.8394	1.5416	5	2.25657	118.6298	2.93129	0.936	1.9953	25.514	1.6304	-0.014	5	0.8927	0.5677	2.4874	27.3108
48	78.3	1.56	3.27	4.17	78.34	1.9913	5	2.2067	124.1617	2.99337	0.9672	2.0262	37.187	2.0704	-0.01	5	0.8654	0.57	2.4135	40.58576
49	336.6	6.29	4.19	4.16	336.651	1.8684	6	1.77771	137.28	3.06201	0.9984	2.0636	161.65	1.8856	-0.002	6	0.67	0.6392	1.8961	201.5162
50	549.8	0	6.41	4.08	549.878	0	0	0	769.6	3.44681	1.0296	2.4172	226.06	0	-0.001	0	1	0.4377	0	0

	CPT-2	In situ data									Basic output data									
Depth (ft)	**qc (tsf)**	**fs (tsf)**	**u (psi)**	**Other**	**qt (tsf)**	**Rf(%)**	**SBT**	**Ic SBT**	**ã (pcf)**	**ó,v (tsf)**	**u0 (tsf)**	**ó',vo (tsf)**	**Qt1**	**Fr (%)**	**Bq**	**SBTn**	**n**	**Cn**	**Ic**	**Qtn**
1	23.5	1.1	-2.39	-2.7	23.4708	4.6867	3	2.84372	118.6656	0.05933	0	0.0593	394.58	4.6986	-0.007	9	0.7144	7.8311	2.2573	173.2681
2	20.8	0.83	10.44	-2.98	20.9278	3.966	3	2.83405	116.3252	0.1175	0	0.1175	177.12	3.9884	0.0361	9	0.7465	5.1582	2.3362	101.4483
3	17.5	0.83	-1.77	-8.85	17.4783	4.7487	3	2.94426	115.8859	0.17544	0	0.1754	98.627	4.7969	-0.007	4	0.8095	4.2826	2.5007	70.03134
4	25.3	1.05	-0.11	-8.17	25.2987	4.1504	3	2.78436	118.5082	0.23469	0	0.2347	106.79	4.1893	-3E-04	4	0.7852	3.2624	2.4282	77.27794
5	27.5	1.57	-1.17	-9.4	27.4857	5.7121	3	2.85175	121.6539	0.29552	0	0.2955	92.008	5.7742	-0.003	9	0.8335	2.8954	2.5458	74.40288
6	30.7	0.99	-2.59	-10.36	30.6683	3.2281	4	2.64966	118.5471	0.35479	0	0.3548	85.44	3.2659	-0.006	5	0.7768	2.3367	2.3897	66.94458
7	136.3	2.16	-0.21	-5.8	136.297	1.5848	6	1.96622	127.8935	0.41874	0	0.4187	324.49	1.5897	-1E-04	6	0.5624	1.6842	1.8189	216.2844
8	126.7	0.91	0.06	-10.2	126.701	0.7182	6	1.75935	121.3905	0.47943	0	0.4794	263.27	0.721	3E-05	6	0.4935	1.4779	1.6308	176.3022
9	151.1	1.77	-0.46	-9.73	151.094	1.1715	6	1.84141	126.6879	0.54278	0	0.5428	277.37	1.1757	-2E-04	6	0.5368	1.431	1.7359	203.603
10	78	1.12	-0.71	-9.84	77.9913	1.4361	5	2.11295	121.7263	0.60364	0	0.6036	128.2	1.4473	-7E-04	6	0.6411	1.4331	2.0018	104.8117
11	38.7	0.67	-0.93	-9.66	38.6886	1.7318	5	2.40076	116.2569	0.66177	0	0.6618	57.462	1.7619	-0.002	5	0.7541	1.4247	2.2911	51.20007
12	102.1	1.02	-0.75	-11.53	102.091	0.9991	6	1.92206	121.6987	0.72262	0	0.7226	140.28	1.0062	-5E-04	6	0.5897	1.2522	1.852	119.9604
13	94.7	0.99	-0.83	-14.29	94.6898	1.0455	6	1.95984	121.2967	0.78327	0	0.7833	119.89	1.0542	-6E-04	6	0.6123	1.2022	1.9036	106.6944
14	14.6	0.66	-1.01	-12.96	14.5876	4.5244	3	2.99152	113.768	0.84015	0	0.8402	16.363	4.8009	-0.005	3	1	1.2594	2.9505	16.36309
15	5.5	0.24	-0.7	-11.59	5.49143	4.3704	3	3.32426	103.9832	0.89214	0	0.8921	5.1553	5.2182	-0.011	3	1	1.186	3.3703	5.15532
16	8.7	0.21	-0.22	-12.6	8.69731	2.4145	3	3.01626	104.1277	0.94421	0	0.9442	8.2112	2.7086	-0.002	3	1	1.1206	3.0435	8.21122
17	22.5	0.51	0.75	-11.14	22.5092	2.2657	4	2.65898	112.9394	1.00068	0	1.0007	21.494	2.3712	0.0025	4	0.9141	1.0523	2.6688	21.39116
18	27.4	2.02	2.37	-11.63	27.429	7.3645	3	2.92995	123.4929	1.06242	0.0312	1.0312	25.568	7.6612	0.0053	3	1	1.0261	2.9464	25.56825
19	27.5	1.91	1.25	-11.75	27.5153	6.9416	3	2.91075	123.0909	1.12397	0.0624	1.0616	24.861	7.2372	0.0011	3	1	0.9967	2.9374	24.86068
20	153.9	2.12	0.57	-11.3	153.907	1.3775	6	1.88575	128.0531	1.188	0.0936	1.0944	139.55	1.3882	-3E-04	6	0.6244	0.9792	1.8969	141.3249
21	93.4	2.27	-0.22	-11.17	93.3973	2.4305	5	2.21391	127.3351	1.25166	0.1248	1.1269	81.772	2.4635	-0.002	5	0.7554	0.9536	2.2365	83.04098
22	187.7	2.8	-1.71	-11.39	187.679	1.4919	6	1.85301	130.5726	1.31695	0.156	1.161	160.53	1.5025	-0.002	6	0.6188	0.9442	1.8739	166.3025
23	313.6	3.18	2	-12.31	313.624	1.014	6	1.58094	132.7561	1.38333	0.1872	1.1961	261.04	1.0184	-1E-04	6	0.5166	0.9386	1.6012	276.9812
24	388.7	2.77	3.44	-10.89	388.742	0.7126	6	1.40347	132.2698	1.44946	0.2184	1.2311	314.6	0.7152	8E-05	6	0.4513	0.934	1.4251	341.852
25	609.4	2.54	-1.93	-10.77	609.376	0.4168	7	1.09959	132.7319	1.51583	0.2496	1.2662	480.06	0.4179	-6E-04	7	0.3359	0.9415	1.1182	540.8561
26	648.2	0	0.81	-11.1	648.21	0	0	0	769.6	1.90063	0.2808	1.6198	399	0	-3E-04	0	1	0.6532	0	0

	CPT-3	In situ data										Basic output data								
Depth (ft)	qc (tsf)	fs (tsf)	u (psi)	Other	qt (tsf)	Rf(%)	SBT	Ic SBT	ã (pcf)	ó,v (tsf)	u0 (tsf)	ó',vo (tsf)	Qt1	Fr (%)	Bq	SBTn	n	Cn	Ic	Qtn
1	16.1	0.42	1.01	0.06	16.1124	2.6067	4	2.81227	110.7033	0.05535	0	0.0554	290.09	2.6157	0.0045	5	0.6774	7.3784	2.1681	111.9683
2	31.8	1.08	1.79	-0.31	31.8219	3.3939	4	2.65183	119.2738	0.11499	0	0.115	275.74	3.4062	0.0041	8	0.6933	4.6586	2.1969	139.5979
3	10.8	0.37	1.46	-0.58	10.8179	3.4203	3	3.02163	108.8042	0.16939	0	0.1694	62.863	3.4747	0.0099	4	0.8216	4.5049	2.5278	45.33559
4	14.6	0.39	1.58	-0.86	14.6193	2.6677	4	2.85251	109.9239	0.22435	0	0.2244	64.162	2.7093	0.0079	5	0.791	3.4105	2.4463	46.39848
5	24.8	1.13	1.79	-0.83	24.8219	4.5524	3	2.81718	118.999	0.28385	0	0.2839	86.447	4.6051	0.0053	4	0.8141	2.9189	2.4969	67.69068
6	18.4	1.01	1.79	-0.69	18.4219	5.4826	3	2.96765	117.4503	0.34258	0	0.3426	52.774	5.5865	0.0071	3	0.8831	2.7071	2.6697	46.25452
7	16.5	0.85	1.68	-0.56	16.5206	5.1451	3	2.98542	115.9226	0.40054	0	0.4005	40.246	5.2729	0.0075	3	0.9053	2.4096	2.7206	36.70911
8	16	0.76	1.41	-0.46	16.0173	4.7449	3	2.97313	115.0283	0.45805	0	0.4581	33.968	4.8846	0.0065	3	0.9174	2.1556	2.7423	31.69796
9	25.4	0.95	1.38	-0.44	25.4169	3.7377	4	2.75301	117.7872	0.51695	0	0.517	48.167	3.8153	0.004	4	0.8535	1.8429	2.57	43.36867
10	66.9	0.81	1.46	-0.37	66.9179	1.2104	5	2.11735	118.9818	0.57644	0	0.5764	115.09	1.221	0.0016	6	0.6364	1.4719	1.993	92.28385
11	42.6	1.01	1.3	-0.22	42.6159	2.37	5	2.45383	119.4958	0.63619	0	0.6362	65.987	2.4059	0.0022	5	0.7702	1.4797	2.3364	58.7049
12	14.5	0.78	1.46	-0.18	14.5179	5.3727	3	3.04046	114.9786	0.69367	0	0.6937	19.929	5.6423	0.0076	3	1	1.5253	2.9322	19.92848
13	15.1	0.69	1.27	-0.13	15.1155	4.5648	3	2.98194	114.1799	0.75076	0	0.7508	19.134	4.8034	0.0064	3	0.9904	1.4048	2.9001	19.07075
14	14.6	0.47	1.22	-0.01	14.6149	3.2159	3	2.90021	111.2884	0.80641	0	0.8064	17.123	3.4037	0.0064	3	0.9715	1.302	2.8436	16.99162
15	14.5	0.44	1.3	-0.02	14.5159	3.0312	3	2.88737	110.7892	0.8618	0	0.8618	15.844	3.2225	0.0069	3	0.9784	1.2223	2.8546	15.77344
16	35.3	1.9	1.33	0.19	35.3163	5.38	3	2.75583	123.6612	0.92363	0	0.9236	37.236	5.5244	0.0028	3	0.9351	1.1355	2.7334	36.90915
17	84.3	1.89	1.56	0.32	84.3191	2.2415	5	2.21971	125.7452	0.98651	0	0.9865	84.472	2.268	0.0014	5	0.7391	1.0532	2.2111	82.94203
18	182	1.72	1.86	0.89	182.023	0.9449	6	1.71835	126.9324	1.04997	0.0312	1.0188	177.64	0.9504	0.0006	6	0.5517	1.0211	1.7154	174.6468
19	221.9	2.87	2.69	1.07	221.933	1.2932	6	1.75839	131.1621	1.11555	0.0624	1.0532	209.67	1.2997	0.0006	6	0.5706	1.0027	1.7607	209.2504
20	210.3	1.83	3.19	1.24	210.339	0.87	6	1.64839	127.7387	1.17942	0.0936	1.0858	192.63	0.8749	0.0007	6	0.5323	0.9863	1.6561	194.9714
21	347.9	1.38	4.15	1.3	347.951	0.3966	7	1.25616	126.9012	1.24287	0.1248	1.1181	310.09	0.398	0.0005	7	0.385	0.979	1.2654	320.7876
22	262	1.79	4.09	1.45	262.05	0.6831	6	1.50665	128.1131	1.30693	0.156	1.1509	226.55	0.6865	0.0005	6	0.4845	0.9601	1.5224	236.5867
23	557.3	8.06	4.82	2.57	557.359	1.4461	6	1.57005	137.28	1.37557	0.1872	1.1884	467.85	1.4497	0.0003	6	0.5097	0.9425	1.5839	495.2601
24	462	3.26	5.48	3.44	462.067	0.7055	6	1.3529	133.883	1.44251	0.2184	1.2241	376.29	0.7077	0.0004	6	0.4306	0.9392	1.3717	408.8507
25	625.8	6.47	4	3.21	625.849	1.0338	6	1.41814	137.28	1.51115	0.2496	1.2616	494.9	1.0363	6E-05	6	0.4573	0.9227	1.4371	544.46
26	502.6	3.04	4.24	4.08	502.652	0.6048	7	1.27769	133.5771	1.57794	0.2808	1.2971	386.29	0.6067	5E-05	7	0.4075	0.9204	1.3023	435.8408
27	476.8	4.54	4.03	4.17	476.849	0.9521	6	1.45014	136.3832	1.64613	0.312	1.3341	356.19	0.9554	-5E-05	6	0.4769	0.8954	1.4798	402.1106
28	383.4	2	3.44	3.38	383.442	0.5216	7	1.30698	129.8532	1.71106	0.3432	1.3679	279.07	0.5239	-3E-04	6	0.4265	0.8963	1.3433	323.3505
29	434.2	4.86	2.94	3.31	434.236	1.1192	6	1.53109	136.6533	1.77938	0.3744	1.405	307.8	1.1238	-4E-04	6	0.5145	0.8643	1.5699	353.2247
30	533.7	3.05	3.02	3.85	533.737	0.5714	7	1.24221	133.7475	1.84626	0.4056	1.4407	369.2	0.5734	-4E-04	7	0.4052	0.8824	1.2786	443.5859
31	342.7	6.75	4.08	3.55	342.75	1.9694	6	1.79274	137.28	1.9149	0.4368	1.4781	230.59	1.9804	-4E-04	6	0.6232	0.8119	1.8461	261.5391
32	279.6	6.42	2.91	3.64	279.636	2.2958	6	1.89672	137.28	1.98354	0.468	1.5155	183.2	2.3123	-9E-04	6	0.6686	0.7865	1.9606	206.3672
33	123	5.3	2.93	3.61	123.036	4.3077	9	2.32612	134.2116	2.05064	0.4992	1.5514	77.982	4.3807	-0.002	4	0.8471	0.7231	2.424	82.68138
34	66.1	2.71	2.85	3.66	66.1349	4.0977	4	2.4821	127.7896	2.11454	0.5304	1.5841	40.413	4.233	-0.005	4	0.9208	0.6896	2.6136	41.72546
35	303.4	8.76	2.31	3.68	303.428	2.887	8	1.96189	137.28	2.18318	0.5616	1.6216	185.77	2.9079	-0.001	8	0.7026	0.7409	2.0365	210.9254
36	420.9	11.39	3.02	3.76	420.937	2.7059	8	1.8675	137.28	2.25182	0.5928	1.659	252.37	2.7204	-9E-04	8	0.6655	0.7413	1.9347	293.3348
37	482.8	9.05	4.16	3.92	482.851	1.8743	6	1.69877	137.28	2.32046	0.624	1.6965	283.25	1.8833	-7E-04	6	0.6023	0.7525	1.7639	341.7607
38	320.1	5.71	4.89	4	320.16	1.7835	6	1.77287	137.0893	2.389	0.6552	1.7338	183.28	1.7969	-1E-03	6	0.6397	0.7291	1.8575	218.9746
39	404.3	7.25	4.66	4.11	404.357	1.793	6	1.72033	137.28	2.45764	0.6864	1.7712	226.9	1.8039	-9E-04	6	0.6193	0.7268	1.7995	276.0656
40	349.5	6.93	4.52	4.25	349.555	1.9825	6	1.79064	137.28	2.52628	0.7176	1.8087	191.87	1.997	-0.001	6	0.6518	0.7051	1.8801	231.2447
41	294.8	3.79	3.75	4.38	294.846	1.2854	6	1.67835	133.8895	2.59323	0.7488	1.8444	158.45	1.2968	-0.002	6	0.6148	0.7106	1.7783	196.2763
42	55.7	2.64	2.77	4.5	55.7339	4.7368	4	2.57872	127.1808	2.65682	0.78	1.8768	28.28	4.9739	-0.011	3	0.9992	0.564	2.7834	28.29301
43	500.8	9.55	3.26	4.59	500.84	1.9068	8	1.69784	137.28	2.72546	0.8112	1.9143	260.21	1.9172	-0.001	6	0.6192	0.6928	1.7813	326.1167
44	379.3	6.68	4.24	4.67	379.352	1.7609	6	1.72815	137.28	2.7941	0.8424	1.9517	192.94	1.774	-0.001	6	0.6389	0.6763	1.8283	240.6746
45	173.6	3.15	3.53	4.68	173.643	1.8141	6	1.93936	131.2448	2.85972	0.8736	1.9861	85.988	1.8444	-0.004	5	0.7387	0.628	2.086	101.3667
46	41.2	1.18	2.94	4.77	41.236	2.8616	4	2.51845	120.5538	2.92	0.9048	2.0152	19.014	3.0797	-0.018	4	1	0.5251	2.7783	19.01351
47	29.2	0.57	2.38	4.78	29.2291	1.9501	4	2.52902	114.3904	2.97719	0.936	2.0412	12.861	2.1713	-0.029	4	1	0.5184	2.8278	12.86107
48	30	0.64	1.79	4.82	30.0219	2.1318	4	2.54308	115.3032	3.03485	0.9672	2.0677	13.052	2.3715	-0.031	4	1	0.5118	2.8438	13.05208
49	30.4	0.76	1.48	4.89	30.4181	2.4985	4	2.5812	116.5926	3.09314	0.9984	2.0947	13.045	2.7813	-0.033	3	1	0.5051	2.8834	13.04456
50	34.1	0	1.58	4.93	34.1193	0	0	0	769.6	3.47794	1.0296	2.4483	12.515	0	-0.03	0	1	0.4322	0	0

	CPT-4	In situ data										Basic output data								
Depth (ft)	qc (tsf)	fs (tsf)	u (psi)	Other	qt (tsf)	Rf(%)	SBT	Ic SBT	ã (pcf)	ó,v (tsf)	u0 (tsf)	ó',vo (tsf)	Qt1	Fr (%)	Bq	SBTn	n	Cn	Ic	Qtn
1	21.7	0.78	9.96	0.07	21.8219	3.5744	4	2.79124	115.9726	0.05799	0	0.058	375.33	3.5839	0.033	8	0.6901	7.4185	2.1917	152.5881
2	13.1	0.52	9.23	0.03	13.213	3.9355	3	2.98796	111.7821	0.11388	0	0.1139	115.03	3.9697	0.0507	4	0.7839	5.7397	2.4346	71.05538
3	17.2	0.66	7.42	-0.13	17.2908	3.8171	3	2.88773	114.1826	0.17097	0	0.171	100.13	3.8552	0.0312	4	0.7874	4.2005	2.438	67.96335
4	28.9	0.99	13.2	-0.2	29.0616	3.4066	4	2.68262	118.4158	0.23018	0	0.2302	125.26	3.4338	0.033	5	0.749	3.1346	2.3345	85.41289
5	29.9	1.34	5.29	-0.42	29.9648	4.4719	3	2.7515	120.7055	0.29053	0	0.2905	102.14	4.5157	0.0128	4	0.7961	2.7982	2.4487	78.47404
6	21	1.2	7.59	-0.38	21.0929	5.6891	3	2.93452	119.0418	0.35005	0	0.3501	59.257	5.7851	0.0264	3	0.8755	2.6339	2.6489	51.63511
7	17.7	1.02	6.63	-0.23	17.7812	5.7364	3	2.99218	117.436	0.40877	0	0.4088	42.499	5.8714	0.0275	3	0.9114	2.3794	2.7355	39.06501
8	24.8	1.09	6.54	-0.12	24.8801	4.381	3	2.80534	118.741	0.46814	0	0.4681	52.147	4.465	0.0193	4	0.8614	2.0186	2.5967	46.57192
9	42.6	0.61	5.36	-0.1	42.6656	1.4297	5	2.31679	115.8091	0.52604	0	0.526	80.107	1.4476	0.0092	5	0.6968	1.6273	2.1579	64.80911
10	51.8	0.79	4.91	0	51.8601	1.5233	5	2.26609	118.1771	0.58513	0	0.5851	87.63	1.5407	0.0069	5	0.6914	1.5062	2.1362	72.98907
11	76.8	1.01	5.42	0.12	76.8663	1.314	5	2.09285	120.9345	0.6456	0	0.6456	118.06	1.3251	0.0051	6	0.6403	1.3721	1.9944	98.83829
12	65.7	1.47	5.65	0.21	65.7692	2.2351	5	2.29638	123.3003	0.70725	0	0.7073	91.993	2.2594	0.0063	5	0.7262	1.3398	2.212	82.38404
13	56.9	0.89	4.97	0.23	56.9608	1.5625	5	2.24117	119.278	0.76689	0	0.7669	73.275	1.5838	0.0064	5	0.7141	1.2584	2.1729	66.83282
14	12.4	0.37	4.59	0.29	12.4562	2.9704	3	2.93624	109.1481	0.82146	0	0.8215	14.163	3.1801	0.0284	3	0.9897	1.2847	2.8895	14.12661
15	13.2	0.34	4.01	0.36	13.2491	2.5662	3	2.87795	108.6799	0.8758	0	0.8758	14.128	2.7479	0.0233	4	0.9786	1.2033	2.8535	14.07084
16	34.7	0.94	5.57	0.43	34.7682	2.7036	4	2.55822	118.4739	0.93504	0	0.935	36.184	2.7783	0.0119	4	0.8619	1.1125	2.5398	35.57089
17	41.1	2.02	7.19	0.47	41.188	4.9043	4	2.68026	124.4845	0.99728	0	0.9973	40.3	5.026	0.0129	4	0.9178	1.0558	2.6788	40.10454
18	54.3	2.28	11.34	0.53	54.4388	4.1882	4	2.54673	126.0507	1.06031	0.0312	1.0291	51.869	4.2714	0.0147	4	0.8708	1.0245	2.5516	51.68282
19	163	1.35	7.76	0.62	163.095	0.8277	6	1.71422	124.8923	1.12275	0.0624	1.0604	152.75	0.8335	0.0031	6	0.555	0.9988	1.7188	152.8973
20	244.2	1.83	5.83	0.49	244.271	0.7492	6	1.55653	128.1034	1.1868	0.0936	1.0932	222.36	0.7528	0.0013	6	0.4978	0.9839	1.5646	226.0338
21	190.1	1.5	7.37	-0.31	190.19	0.7887	6	1.65061	126.0381	1.24982	0.1248	1.125	167.94	0.7939	0.0022	6	0.5376	0.9676	1.6652	172.7733
22	177.9	2.15	10.15	-0.57	178.024	1.2077	6	1.80077	128.511	1.31408	0.156	1.1581	152.59	1.2167	0.0033	6	0.5988	0.9474	1.8216	158.2171
23	248.7	1.61	11.62	-0.46	248.842	0.647	6	1.50655	127.2115	1.37768	0.1872	1.1905	207.87	0.6506	0.0026	6	0.4886	0.944	1.5283	220.784
24	645.6	7.2	2.59	-0.14	645.632	1.1152	6	1.44041	137.28	1.44632	0.2184	1.2279	524.61	1.1177	-5E-05	6	0.4629	0.9334	1.4562	568.2734
25	399.6	2	-3.37	0.26	399.559	0.5006	7	1.2817	129.9536	1.5113	0.2496	1.2617	315.48	0.5025	-0.001	7	0.4072	0.9309	1.3059	350.1757
26	315.8	1.33	-2.49	0.38	315.77	0.4212	7	1.30518	126.3945	1.5745	0.2808	1.2937	242.87	0.4233	-0.001	6	0.4203	0.919	1.3364	272.8815
27	347.4	4.11	-8.92	0.99	347.291	1.1835	6	1.60687	134.8819	1.64194	0.312	1.3299	259.9	1.1891	-0.003	6	0.5385	0.8842	1.6421	288.824
28	369.3	1.88	-6.75	1.46	369.217	0.5092	7	1.31106	129.3082	1.70659	0.3432	1.3634	269.56	0.5116	-0.002	6	0.4279	0.8972	1.3476	311.6255
29	403.5	4.77	-5.73	2.15	403.43	1.1824	6	1.56879	136.337	1.77476	0.3744	1.4004	286.82	1.1876	-0.002	6	0.5292	0.8622	1.6088	327.2808
30	451.9	3.67	-1.84	2.99	451.877	0.8122	6	1.40744	134.6955	1.84211	0.4056	1.4365	313.28	0.8155	-0.001	6	0.4696	0.8663	1.448	368.4397
31	384.4	1.5	-2.31	3.65	384.372	0.3903	7	1.219	127.7541	1.90599	0.4368	1.4692	260.32	0.3922	-0.002	7	0.4016	0.8765	1.2653	316.8261
32	473.5	3.91	-3.94	3.58	473.452	0.8259	6	1.40133	135.2727	1.97362	0.468	1.5056	313.14	0.8293	-0.002	6	0.4726	0.8465	1.4474	377.1648
33	342.4	1.97	-3.36	3.33	342.359	0.5754	6	1.37188	129.4662	2.03836	0.4992	1.5392	221.11	0.5789	-0.002	6	0.4676	0.8393	1.4298	269.9377
34	550.6	4.55	0.07	3.36	550.601	0.8264	6	1.36427	136.7501	2.10673	0.5304	1.5763	347.96	0.8295	-1E-03	6	0.4627	0.8316	1.4124	431.0709
35	438.7	3.46	0.51	3.86	438.706	0.7887	6	1.40498	134.1922	2.17383	0.5616	1.6122	270.76	0.7926	-0.001	6	0.4838	0.8157	1.4634	336.5146
36	461	3.6	1.61	4.05	461.02	0.7809	6	1.38856	134.6034	2.24113	0.5928	1.6483	278.33	0.7847	-0.001	6	0.4799	0.8084	1.4487	350.4978
37	380.7	6.63	1.44	4.13	380.718	1.7415	6	1.72323	137.28	2.30977	0.624	1.6858	224.47	1.7521	-0.001	6	0.6139	0.7513	1.7959	268.6915
38	278.2	5.72	1.63	4.22	278.22	2.0559	6	1.8582	136.7597	2.37815	0.6552	1.723	160.1	2.0737	-0.002	6	0.6735	0.7201	1.9476	187.7259
39	354.6	6.39	1.18	4.3	354.614	1.802	6	1.75226	137.28	2.44679	0.6864	1.7604	200.05	1.8145	-0.002	6	0.6326	0.7247	1.8357	241.191
40	39.6	1.26	-0.64	4.35	39.5922	3.1825	4	2.56246	120.9346	2.50726	0.7176	1.7897	20.722	3.3976	-0.021	4	0.9921	0.5937	2.7743	20.80792
41	30.4	0.66	0.25	4.06	30.4031	2.1708	4	2.54356	115.5591	2.56504	0.7488	1.8162	15.327	2.3709	-0.026	4	0.9974	0.5834	2.7857	15.34919
42	28.8	0.54	0.66	3.89	28.8081	1.8745	4	2.52388	113.9594	2.62202	0.78	1.842	14.216	2.0622	-0.028	4	0.9959	0.5757	2.7783	14.24834
43	43.6	1.09	1.24	3.91	43.6152	2.4991	5	2.46127	120.1101	2.68207	0.8112	1.8709	21.879	2.6629	-0.018	4	0.9612	0.5782	2.6839	22.36893
44	69.1	2.22	2.6	3.96	69.1318	3.2113	5	2.39164	126.4383	2.74529	0.8424	1.9029	34.887	3.3441	-0.01	4	0.9253	0.581	2.5854	36.45073
45	63.8	2	4.4	4.04	63.8539	3.1322	5	2.4079	125.481	2.80803	0.8736	1.9344	31.558	3.2762	-0.009	4	0.9376	0.568	2.6138	32.76868
46	32.1	0.67	8.82	4.11	32.208	2.0802	4	2.51242	115.8098	2.86594	0.9048	1.9611	14.962	2.2834	-0.009	4	1	0.5395	2.7855	14.96175
47	41.7	1.01	12.82	4.17	41.8569	2.413	5	2.46484	119.452	2.92566	0.936	1.9897	19.567	2.5943	-3E-04	4	0.98	0.5386	2.7188	19.81524
48	38.1	0.92	18.16	4.22	38.3223	2.4007	4	2.49265	118.5539	2.98494	0.9672	2.0177	17.513	2.6035	0.0096	4	0.998	0.5251	2.7623	17.53638
49	45.2	0.79	38.26	4.31	45.6683	1.7299	5	2.34365	117.867	3.04387	0.9984	2.0455	20.838	1.8534	0.0412	4	0.9381	0.5388	2.6011	21.706
50	32.2	0.74	60.04	4.38	32.9349	2.2469	4	2.52545	116.5913	3.10217	1.0296	2.0726	14.394	2.4805	0.1104	4	1	0.5105	2.8198	14.39409
51	33.5	0.76	79.37	4.41	34.4715	2.2047	4	2.50483	116.8977	3.16062	1.0608	2.0998	14.911	2.4273	0.1486	4	1	0.5039	2.8018	14.91124
52	34.4	0.82	122.69	4.48	35.9017	2.284	4	2.50072	117.5528	3.21939	1.092	2.1274	15.363	2.509	0.2369	4	1	0.4974	2.7995	15.36262
53	36.1	0.82	132.74	4.54	37.7247	2.1736	5	2.47046	117.6736	3.27823	1.1232	2.155	15.984	2.3805	0.2449	4	1	0.491	2.7723	15.98424
54	34.7	0.62	152.71	4.62	36.5692	1.6954	5	2.4147	115.552	3.33601	1.1544	2.1816	15.233	1.8656	0.2961	4	0.9933	0.4874	2.7284	15.30754
55	36.1	0.47	208.93	4.69	38.6573	1.2158	5	2.31094	113.6608	3.39284	1.1856	2.2072	15.977	1.3328	0.393	4	0.9541	0.4959	2.6228	16.52578
56	36	0.49	261.04	4.77	39.1951	1.2502	5	2.31286	113.9994	3.44984	1.2168	2.233	16.007	1.3708	0.4918	4	0.9577	0.4891	2.6292	16.52164
57	44	1.4	224.23	4.83	46.7446	2.995	4	2.49152	122.1105	3.51089	1.248	2.2629	19.106	3.2382	0.3446	4	1	0.4676	2.7902	19.10551
58	443.5	5.58	26.27	4.93	443.822	1.2573	6	1.56807	137.28	3.57953	1.2792	2.3003	191.38	1.2675	0.0014	6	0.6034	0.6259	1.6917	260.4042
59	538.1	12.81	63.29	4.96	538.875	2.3772	8	1.76908	137.28	3.64817	1.3104	2.3378	228.95	2.3934	0.0061	8	0.6791	0.5837	1.8858	295.2701
60	575.4	0	73.28	4.98	576.297	0	0	0	769.6	4.03297	1.3416	2.6914	212.63	0	0.0069	0	1	0.3932	0	0

	CPT-5	In situ data										Basic output data								
Depth (ft)	qc (tsf)	fs (tsf)	u (psi)	Other	qt (tsf)	Rf(%)	SBT	Ic SBT	ã (pcf)	ó,v (tsf)	u0 (tsf)	ó',vo (tsf)	Qt1	Fr (%)	Bq	SBTn	n	Cn	Ic	Qtn
1	15	0.37	0.41	-0.5	15.005	2.4658	4	2.82364	109.6022	0.0548	0	0.0548	272.81	2.4749	0.002	5	0.6774	7.4288	2.1687	104.9632
2	15.3	0.77	1.42	-0.33	15.3174	5.027	3	3.00408	115.015	0.11231	0	0.1123	135.39	5.0641	0.0067	9	0.7947	5.945	2.4638	85.43042
3	14.4	0.24	1.55	0.11	14.419	1.6645	4	2.7445	106.3377	0.16548	0	0.1655	86.136	1.6838	0.0078	5	0.7256	3.8431	2.2749	51.76902
4	22.6	0.65	1.72	1.34	22.6211	2.8734	4	2.71969	114.7263	0.22284	0	0.2228	100.51	2.902	0.0055	5	0.7528	3.2307	2.3462	68.38821
5	22.5	0.99	2.04	1.45	22.525	4.3951	3	2.83871	117.7944	0.28174	0	0.2817	78.95	4.4508	0.0066	4	0.8191	2.9563	2.5104	62.14566
6	16	0.94	1.95	1.36	16.0239	5.8663	3	3.03259	116.5846	0.34003	0	0.34	46.125	5.9934	0.009	3	0.9041	2.7907	2.725	41.36573
7	16.7	0.93	1.88	1.42	16.723	5.5612	3	3.00338	116.6105	0.39834	0	0.3983	40.982	5.6969	0.0083	3	0.9117	2.4367	2.7375	37.59374
8	17.9	0.66	1.63	1.47	17.92	3.6831	3	2.86592	114.2698	0.45547	0	0.4555	38.344	3.7791	0.0067	4	0.8766	2.0936	2.6384	34.55629
9	38.3	0.49	1.47	1.58	38.318	1.2788	5	2.32654	113.9442	0.51244	0	0.5124	73.775	1.2961	0.0028	5	0.6963	1.6567	2.1583	59.19214
10	29.6	0.72	1.47	1.71	29.618	2.431	4	2.58282	116.132	0.57051	0	0.5705	50.915	2.4787	0.0036	5	0.8026	1.6418	2.4298	45.07088
11	13.6	0.68	1.39	1.79	13.617	4.9938	3	3.04172	113.8185	0.62742	0	0.6274	20.703	5.235	0.0077	3	0.9849	1.6732	2.9007	20.54012
12	7.7	0.46	1.39	1.88	7.71701	5.9609	3	3.28301	109.5734	0.6822	0	0.6822	10.312	6.5389	0.0142	3	1	1.551	3.1904	10.31188
13	7.3	0.32	1.39	1.99	7.31701	4.3734	3	3.22189	106.7882	0.7356	0	0.7356	8.947	4.8622	0.0152	3	1	1.4384	3.1588	8.94702
14	3.2	0.19	0.98	1.97	3.212	5.9153	3	3.59091	100.9658	0.78608	0	0.7861	3.0861	7.8321	0.0291	2	1	1.3461	3.6541	3.08609
15	8.6	0.25	1.08	2.05	8.61322	2.9025	3	3.06302	105.3797	0.83877	0	0.8388	9.2689	3.2157	0.01	3	1	1.2615	3.0411	9.26886
16	39.6	1.06	1.72	2.22	39.6211	2.6754	4	2.51216	119.6717	0.89861	0	0.8986	43.092	2.7374	0.0032	4	0.838	1.1468	2.4817	41.96624
17	42.5	0.64	1.55	2.81	42.519	1.5052	5	2.33132	116.152	0.95668	0	0.9567	43.444	1.5399	0.0027	5	0.7784	1.0816	2.318	42.48483
18	79	0.89	1.72	2.77	79.0211	1.1263	6	2.04128	120.0764	1.01672	0.0312	0.9855	79.15	1.141	0.0012	6	0.6712	1.0489	2.033	77.32217
19	253.6	2.39	2.39	2.3	253.629	0.9423	6	1.61707	130.1486	1.0818	0.0624	1.0194	247.74	0.9464	0.0004	6	0.5132	1.0193	1.6141	243.2874
20	286.4	2.23	3.19	2.21	286.439	0.7785	6	1.52031	129.9383	1.14676	0.0936	1.0532	270.89	0.7817	0.0005	6	0.4797	1.0023	1.5221	270.2313
21	387.4	3.06	4.76	1.78	387.458	0.7898	6	1.43881	132.9903	1.21326	0.1248	1.0885	354.85	0.7922	0.0006	6	0.4517	0.9873	1.4442	360.4002
22	236.4	3.25	5.72	1.08	236.47	1.3744	6	1.76084	132.2267	1.27937	0.156	1.1234	209.36	1.3819	0.0011	6	0.5789	0.9659	1.7738	214.7049
23	324.2	2.53	6.17	1.29	324.276	0.7802	6	1.48476	131.1644	1.34496	0.1872	1.1578	278.93	0.7835	0.0008	6	0.4762	0.9581	1.4997	292.3938
24	448.9	2.69	6.05	1.47	448.974	0.5991	7	1.30559	132.4067	1.41116	0.2184	1.1928	375.23	0.601	0.0005	7	0.4099	0.9521	1.3214	402.7169
25	607.6	3.5	6.36	0.86	607.678	0.576	7	1.21099	135.0709	1.47869	0.2496	1.2291	493.21	0.5774	0.0003	7	0.3757	0.9453	1.227	541.5573

	CPT-6	In situ data										Basic output data								
Depth (ft)	qc (tsf)	fs (tsf)	u (psi)	Other	qt (tsf)	Rf(%)	SBT	Ic SBT	ã (pcf)	ó,v (tsf)	u0 (tsf)	ó',vo (tsf)	Qt1	Fr (%)	Bq	SBTn	n	Cn	Ic	Qtn
1	44.9	0.15	-0.12	-0.37	44.8985	0.3341	6	1.9868	105.6691	0.05283	0	0.0528	848.8	0.3345	-2E-04	6	0.42	3.5209	1.4947	149.2249
2	12.8	0.11	0	-0.06	12.8	0.8594	4	2.65169	100.3388	0.103	0	0.103	123.27	0.8664	0	5	0.6481	4.5253	2.0859	54.30285
3	22	0.18	-0.14	0.26	21.9983	0.8183	5	2.43211	105.263	0.15564	0	0.1556	140.34	0.8241	-5E-04	6	0.6195	3.2784	1.9946	67.6765
4	12.8	0.09	-0.16	0.5	12.798	0.7032	4	2.61502	98.87015	0.20507	0	0.2051	61.408	0.7147	-9E-04	5	0.6927	3.1162	2.1833	37.08753
5	26.3	0.96	-4.35	0.65	26.2468	3.6576	4	2.73629	117.9422	0.26404	0	0.264	98.404	3.6948	-0.012	4	0.7783	2.9459	2.4062	72.33839
6	30.4	1.11	-5.94	0.48	30.3273	3.6601	4	2.6892	119.3569	0.32372	0	0.3237	92.684	3.6996	-0.014	4	0.7827	2.5269	2.4093	71.65368
7	25.4	0.91	-5.98	0.42	25.3268	3.593	4	2.74305	117.4638	0.38245	0	0.3825	65.222	3.6481	-0.017	4	0.8159	2.2939	2.4886	54.07821
8	19.6	0.46	-6.7	0.36	19.518	2.3568	4	2.7191	111.8366	0.43837	0	0.4384	43.524	2.411	-0.025	4	0.8177	2.0555	2.4862	37.06385
9	21.4	0.13	-4.64	0.28	21.3432	0.6091	5	2.387	102.8082	0.48977	0	0.4898	42.578	0.6234	-0.016	5	0.7065	1.7232	2.1885	33.96199
10	30.7	0.06	-0.59	0.15	30.6928	0.1955	6	2.07154	98.03684	0.53879	0	0.5388	55.966	0.199	-0.001	6	0.6028	1.502	1.9103	42.804
11	37.9	0.11	-0.47	0.11	37.8943	0.2903	6	2.03399	102.986	0.59029	0	0.5903	63.196	0.2949	-9E-04	6	0.6014	1.4205	1.8999	50.08085
12	12.2	0.74	-1.03	0.01	12.1874	6.0719	3	3.13285	114.1667	0.64737	0	0.6474	17.826	6.4125	-0.006	3	1	1.6345	3.0054	17.82603
13	8.7	0.55	-0.3	-0.12	8.69633	6.3245	3	3.25785	111.1723	0.70296	0	0.703	11.371	6.8807	-0.003	3	1	1.5052	3.1721	11.3711
14	11	0.53	1.29	-0.14	11.0158	4.8113	3	3.10378	111.4779	0.75869	0	0.7587	13.519	5.1672	0.0091	3	1	1.3946	3.0346	13.51941
15	31.6	1.13	1.99	-0.25	31.6244	3.5732	4	2.66867	119.5897	0.81849	0	0.8185	37.637	3.6681	0.0047	4	0.8839	1.2548	2.6119	36.53231
16	46.2	2.76	-2.21	-0.37	46.173	5.9775	3	2.70842	127.047	0.88201	0	0.882	51.35	6.0939	-0.004	3	0.9098	1.1801	2.6722	50.51339
17	116	0.98	-2.1	-0.55	115.974	0.845	6	1.83322	121.717	0.94287	0	0.9429	122	0.8519	-0.001	6	0.5863	1.0699	1.8154	116.3182
18	115.4	0.4	-0.35	-0.67	115.396	0.3466	6	1.62142	115.1481	1.00045	0.0312	0.9693	118.03	0.3497	-5E-04	6	0.5091	1.0457	1.6094	113.0504
19	134.3	0.82	-0.35	-0.71	134.296	0.6106	6	1.69669	120.7705	1.06083	0.0624	0.9984	133.44	0.6155	-7E-04	6	0.5413	1.0319	1.6905	129.9378
20	152.8	0.37	-0.4	-1.23	152.795	0.2422	6	1.44296	115.2624	1.11846	0.0936	1.0249	148	0.2439	-8E-04	6	0.4477	1.0144	1.4416	145.4105
21	154.3	0.65	0.01	-1.36	154.3	0.4213	6	1.55542	119.4092	1.17817	0.1248	1.0534	145.36	0.4245	-8E-04	6	0.4938	1.0022	1.5592	145.0343
22	173.5	0.95	0.75	-1.53	173.509	0.5475	6	1.57927	122.4721	1.2394	0.156	1.0834	159.01	0.5515	-6E-04	6	0.5061	0.9881	1.5878	160.8742
23	300.8	0.98	-0.82	-1.15	300.79	0.3258	7	1.25301	124.0415	1.30142	0.1872	1.1142	268.79	0.3272	-8E-04	7	0.3837	0.9804	1.2626	277.4842
24	332.1	0.8	-2.45	-0.82	332.07	0.2409	7	1.14436	122.7978	1.36282	0.2184	1.1444	288.97	0.2419	-0.001	7	0.3449	0.9733	1.1568	304.2058
25	150.7	0.51	-7.37	-0.78	150.61	0.3386	6	1.51516	117.5753	1.42161	0.2496	1.172	127.29	0.3419	-0.005	6	0.4922	0.9509	1.5398	134.0764
26	274.6	1.24	0.82	-0.43	274.61	0.4516	6	1.37108	125.5412	1.48438	0.2808	1.2036	226.93	0.454	-8E-04	6	0.4379	0.9452	1.3933	243.9691
27	475.7	0.62	-3.21	0.1	475.661	0.1304	7	0.88327	121.8093	1.54528	0.312	1.2333	384.43	0.1308	-0.001	7	0.2514	0.9622	0.9006	431.1481
28	599.3	2.69	-2.13	0.11	599.274	0.4489	7	1.12896	133.111	1.61184	0.3432	1.2686	471.1	0.4501	-8E-04	7	0.3474	0.9389	1.1482	530.3303
29	305.6	1.21	-8.02	-0.37	305.502	0.3961	7	1.29918	125.622	1.67465	0.3744	1.3003	233.67	0.3983	-0.003	6	0.4189	0.9173	1.3319	263.3944
30	360.6	1.59	-8.25	-0.35	360.499	0.4411	7	1.27534	128.0241	1.73866	0.4056	1.3331	269.12	0.4432	-0.003	7	0.4117	0.9093	1.309	308.2975
31	373.6	2.01	-8.6	-0.54	373.495	0.5382	7	1.32468	129.8256	1.80358	0.4368	1.3668	271.95	0.5408	-0.003	6	0.4334	0.895	1.3616	314.394
32	776.8	2.69	-9.45	-0.72	776.684	0.3463	7	0.97057	133.7434	1.87045	0.468	1.4025	552.47	0.3472	-0.001	7	0.2954	0.9202	0.995	673.7941

Presented below is a list of formulas used for the estimation of various soil properties. The formulas are presented in SI unit system and assume that all components are expressed in the same units.

:: Unit Weight, g (kN/m³) ::

$$g = g_w \cdot \left(0.27 \cdot \log(R_f) + 0.36 \cdot \log(\frac{q_t}{p_a}) + 1.236\right)$$

where g_w = water unit weight

:: Permeability, k (m/s) ::

$I_c < 3.27$ and $I_c > 1.00$ then $k = 10^{0.952-3.04 \cdot I_c}$

$I_c \leq 4.00$ and $I_c > 3.27$ then $k = 10^{-4.52-1.37 \cdot I_c}$

:: N_{SPT} (blows per 30 cm) ::

$$N_{60} = \left(\frac{q_c}{P_a}\right) \cdot \frac{1}{10^{1.1268-0.2817 \cdot I_c}}$$

$$N_{1(60)} = Q_{tn} \cdot \frac{1}{10^{1.1268-0.2817 \cdot I_c}}$$

:: Young's Modulus, Es (MPa) ::

$$(q_t - \sigma_v) \cdot 0.015 \cdot 10^{0.55 \cdot I_c + 1.68}$$

(applicable only to $I_c < I_{c\_cutoff}$)

:: Relative Density, Dr (%) ::

$$100 \cdot \sqrt{\frac{Q_{tn}}{k_{DR}}}$$

(applicable only to SBT_n: 5, 6, 7 and 8 or $I_c < I_{c\_cutoff}$)

:: State Parameter, ψ ::

$$\psi = 0.56 - 0.33 \cdot \log(Q_{tn,cs})$$

:: Peak drained friction angle, φ (°) ::

$$\varphi = 17.60 + 11 \cdot \log(Q_{tn})$$

(applicable only to SBT_n: 5, 6, 7 and 8)

:: 1-D constrained modulus, M (MPa) ::

If $I_c > 2.20$

$\alpha = 14$ for $Q_{tn} > 14$

$\alpha = Q_{tn}$ for $Q_{tn} \leq 14$

$M_{CPT} = \alpha \cdot (q_t - \sigma_v)$

If $I_c \leq 2.20$

$M_{CPT} = (q_t - \sigma_v) \cdot 0.0188 \cdot 10^{0.55 \cdot I_c + 1.68}$

:: Small strain shear Modulus, Go (MPa) ::

$$G_0 = (q_t - \sigma_v) \cdot 0.0188 \cdot 10^{0.55 \cdot I_c + 1.68}$$

:: Shear Wave Velocity, Vs (m/s) ::

$$V_s = \left(\frac{G_0}{\rho}\right)^{0.50}$$

:: Undrained peak shear strength, Su (kPa) ::

$N_{kt} = 10.50 + 7 \cdot \log(F_r)$ or user defined

$$S_u = \frac{(q_t - \sigma_v)}{N_{kt}}$$

(applicable only to SBT_n: 1, 2, 3, 4 and 9 or $I_c > I_{c\_cutoff}$)

:: Remolded undrained shear strength, Su(rem) (kPa) ::

$S_{u(rem)} = f_s$ (applicable only to SBT_n: 1, 2, 3, 4 and 9 or $I_c > I_{c\_cutoff}$)

:: Overconsolidation Ratio, OCR ::

$$k_{OCR} = \left[\frac{Q_{tn}^{0.20}}{0.25 \cdot (10.50 \cdot + 7 \cdot \log(F_r))}\right]^{1.25}$$ or user defined

$OCR = k_{OCR} \cdot Q_{tn}$

(applicable only to SBT_n: 1, 2, 3, 4 and 9 or $I_c > I_{c\_cutoff}$)

:: In situ Stress Ratio, Ko ::

$$K_O = (1 - \sin\varphi') \cdot OCR^{\sin\varphi'}$$

(applicable only to SBT_n: 1, 2, 3, 4 and 9 or $I_c > I_{c\_cutoff}$)

:: Soil Sensitivity, S_t ::

$$S_t = \frac{N_S}{F_r}$$

(applicable only to SBT_n: 1, 2, 3, 4 and 9 or $I_c > I_{c\_cutoff}$)

:: Effective Stress Friction Angle, φ' (°) ::

$$\varphi' = 29.5° \cdot B_q^{0.121} \cdot (0.256 + 0.336 \cdot B_q + \log Q_t)$$

(applicable for $0.10 < B_q < 1.00$)

References

- Robertson, P.K., Cabal K.L., Guide to Cone Penetration Testing for Geotechnical Engineering, Gregg Drilling & Testing, Inc., 5th Edition, November 2012
- Robertson, P.K., Interpretation of Cone Penetration Tests - a unified approach., Can. Geotech. J. 46(11): 1337–1355 (2009)

APPENDIX C

LOG OF SOIL BORING

REPORT OF BOREHOLE: PT-01

CLIENT:	Jones Lang Lasalle IP Inc (JLL)
PROJECT:	Marina Marketplace Phase III Geotechnical Evaluation
LOCATION:	13400 Glencoe Avenue #240
PROJECT NO.:	140-3929

DRIVE WEIGHT: 140 lbs.
DROP DISTANCE: 30 inches
N: E:
ELEVATION: DATUM: GS
INCLINATION: -90°
BOREHOLE DIAMETER: 8 inches

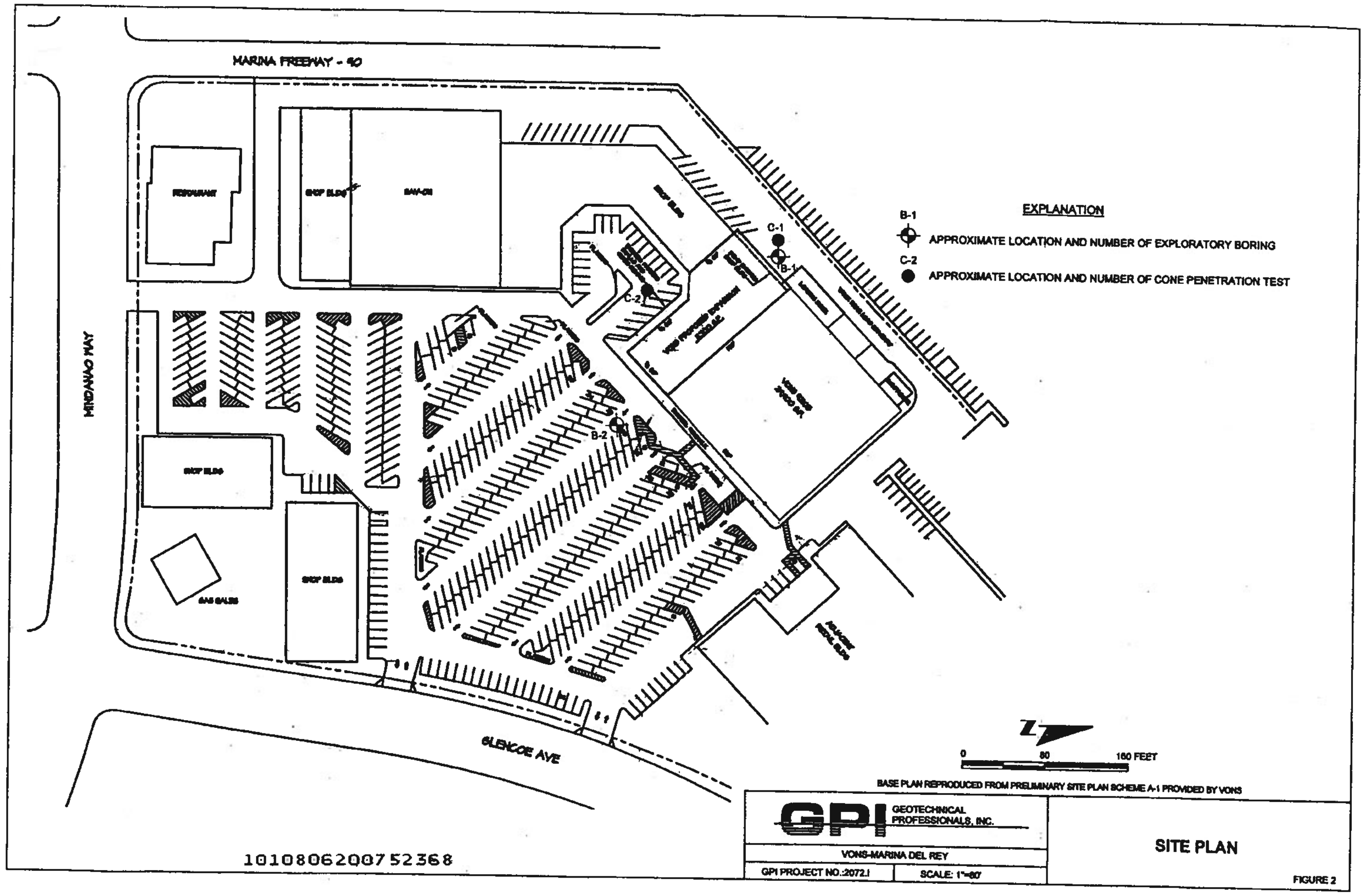
SHEET: 1 OF 1
DRILLER: Martini Drilling Corp.
DRILL RIG: CME 75
LOGGED: LG DATE: 12/17/14
CHECKED: AJA DATE: 1/7/15

Drilling: METHOD	DRILL DATE/TIME	WATER	DEPTH feet	LAYER ELEVATION	Sampling: RUN	SAMPLE TYPE	RECOVERY (ft)	GRAPHIC LOG	USCS	Material Description: (SYMBOL) SOIL NAME, particle size, gradation, shape, minor components; color, contamination; behaviour, moisture, density/consistency	MOISTURE	DRY DENSITY (pcf)	ADDITIONAL LAB TESTING
Hand Auger			0							3-inch asphalt pavement			
				0.5					SM	FILL: (SM), SILTY SAND, fine to medium grained, dark brown, non-cohesive, trace of clay, moist			
				0.7					CL	(CL), silty CLAY, medium plasticity, dark brown, cohesive, w~PL			
Hollow Stem Auger			5–6.5		S-1		1.5			-brown, some fine sand			
			10–11.5		S-2		1.5						
				13.5						Bottom of borehole at 12.0 feet. No groundwater encountered. Drilled borehole, sampled, and installed well. Performed percolation test, backfilled with coarse and patched with asphalt.			

Report of borehole must be read in conjunction with accompanying notes and abbreviations

APPENDIX D

PREVIOUS GEOTECHNICAL INVESTIGATIONS



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APPENDIX B

EXPLORATORY BORINGS

The subsurface conditions at the site were investigated by drilling and sampling three exploratory borings. The boring locations are shown on the Site Plan, Figure 2. The borings were advanced to depths of 26 and 51 feet below the existing site grades.

The borings were drilled using truck-mounted hollow-stem auger equipment. Relatively undisturbed samples were obtained using a brass-ring lined sampler (ASTM D3550), driven into the soil by a 140-pound hammer dropping 30 inches. The number of blows needed to drive the sampler 12 inches into the soil was recorded as the penetration resistance. Due to the use of a "free-fall" hammer (rather than a hammer attached to a rope), the blow-counts recorded with the drive (D) sampler are approximately equal to the Standard Penetration Test blow-count (N60).

The field explorations for the investigation were performed under the continuous technical supervision of GPI's representative, who visually inspected the site, maintained detailed logs of the borings, classified the soils encountered, and obtained relatively undisturbed samples for examination and laboratory testing. The soils encountered in the borings were classified in the field and through further examination in the laboratory in accordance with the Unified Soils Classification System. Detailed logs of the borings are presented in Figures B-1 to B-2 in this appendix.

When drilling below the groundwater depth, a head of water above the groundwater depth was maintained by the driller to help mitigate against any heaving or instability of the soils at the sampling depth due to excess hydrostatic pressure.

The borings were laid out in the field by measuring from existing site features. Existing ground surface elevations at the site were determined by USGS topographic map and should be considered very approximate. All borings were backfilled with bentonite chips above the groundwater depth where the hole did not cave.

	MOISTURE (%)	DRY DENSITY (PCF)	PENETRATION RESISTANCE (BLOWS/FOOT)	SAMPLE TYPE	DEPTH (FEET)	DESCRIPTION OF SUBSURFACE MATERIALS	ELEVATION (FEET)
						This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	
					0	3 inches AC over 3 inches AB	
				B		Fill: **SILTY CLAY (CL)** dark brown/black, moist, very stiff, trace organics	15
	17.6	111	18	D	5	@ 5 feet, mixed fill with SILTY SAND (SM), brown, moist, fine to medium grained sand, with shell fragments	
							10
	14.7	116	21	D		Natural?: **SILTY CLAY (CL)** brown, moist, very stiff, with sand, slightly porous	
	14.8	108	12	D	10	**SANDY CLAY (CL)/SANDY SILT (ML)** brown, very moist, stiff, fine to coarse grained sand	
							5
					15	**SILTY SAND (SM)** brown, wet, medium dense	
	14.3		12	S			
						SAND (SP) brown, wet, medium dense, fine to coarse grained, with gravel	0
					20		
	16.2		30	S			-5
	6.0		39	S	25		
							-10
	12.3		65	D	30	-becomes dense	
							-15
	26.1		9	S		**CLAY (CL)** dark grey, moist, stiff	
						@ 34 feet, with fine sand, slightly porous	
					35		
				D			-20

SAMPLE TYPES
- C Rock Core
- S Standard Split Spoon
- D Drive Sample
- B Bulk Sample
- T Tube Sample

DATE DRILLED:
11-2-05

EQUIPMENT USED:
8" Hollow Stem Auger

GROUNDWATER LEVEL (ft):
Water at 16'6"

GPI

PROJECT NO.: 2072.I
VONS-MARINA DEL REY

LOG OF BORING NO. B-1

FIGURE B-1

	MOISTURE (%)	DRY DENSITY (PCF)	PENETRATION RESISTANCE (BLOWS/FOOT)	SAMPLE TYPE	DEPTH (FEET)	DESCRIPTION OF SUBSURFACE MATERIALS This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	ELEVATION (FEET)
					40	**CLAYEY SAND (SC)** dark grey, very moist, medium dense, fine to coarse grained sand, porous	
	23.0		10	S			-25
	22.4	99	46	D	45	**CLAY (CL)** dark grey, moist, hard	-30
	39.1		9	D	50	@ 50 feet, stiff @ 51 feet, clayey sand, fine grained sand, slightly porous Total Depth 51 feet	

SAMPLE TYPES
- C Rock Core
- S Standard Split Spoon
- D Drive Sample
- B Bulk Sample
- T Tube Sample

DATE DRILLED: 11-2-05

EQUIPMENT USED: 8" Hollow Stem Auger

GROUNDWATER LEVEL (ft): Water at 16'6"

GPI

PROJECT NO.: 2072.I

VONS-MARINA DEL REY

LOG OF BORING NO. B-1

FIGURE B-1

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	MOISTURE (%)	DRY DENSITY (PCF)	PENETRATION RESISTANCE (BLOWS/FOOT)	SAMPLE TYPE	DEPTH (FEET)	DESCRIPTION OF SUBSURFACE MATERIALS This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	ELEVATION (FEET)
					0	3 inches AC over 3 inches AB	
				B		Fill: **CLAYEY SILT (ML)** dark brown, moist, very stiff, organic	15
						SILTY CLAY (CL) brown, moist, very stiff, with shale fragments	
	14.9	115	18	D	5		
							10
	20.1	106	12	D		Natural?: **SILTY SAND (SM)/SANDY SILT (ML)** brown, moist, medium dense, fine grained sand, porous	
	16.4	112	12	D	10	@ 10 feet, with angular gravel	
							5
	23.2		4	S	15	**SANDY CLAY (CL)** brown, very moist, soft to firm, fine to medium grained sand, porous	
							0
	23.2	100	22	D	20		
			35	S		**SAND (SP)** light brown, wet,medium dense, fine to medium grained @ 22 feet, with gravel	-5
	17.4		38	S	25		
						Total Depth 26 ½ feet	

SAMPLE TYPES
- C Rock Core
- S Standard Split Spoon
- D Drive Sample
- B Bulk Sample
- T Tube Sample

DATE DRILLED:
11-2-05

EQUIPMENT USED:
8" Hollow Stem Auger

GROUNDWATER LEVEL (ft):
Water at 16'6"

GPI

PROJECT NO.: 2072.I
VONS-MARINA DEL REY

LOG OF BORING NO. B-2

FIGURE B-2

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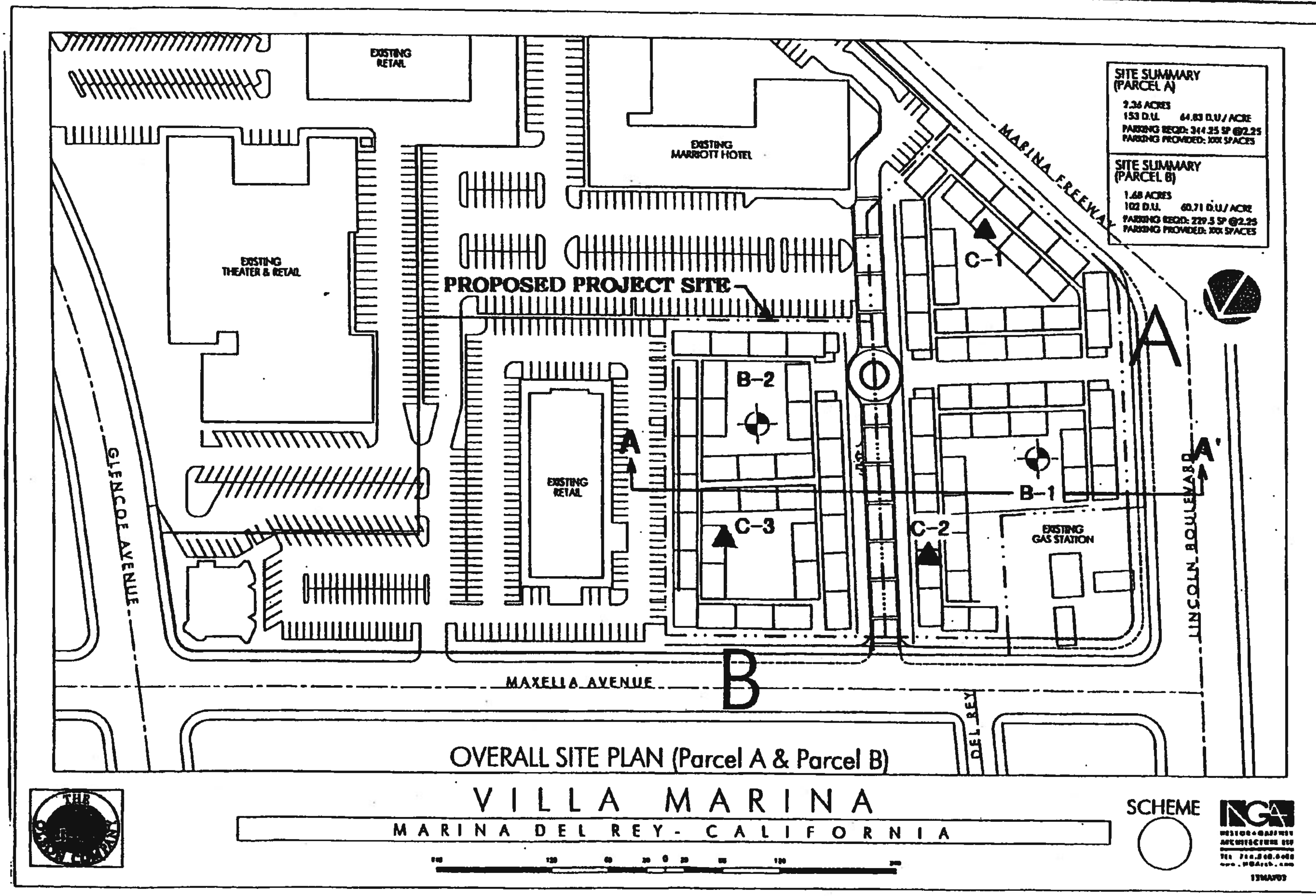

LEGEND

- BORING LOCATION
- CPT LOCATION

Reference: Drawing provided by, NGA, dated 5/13/03

DATE: 05/27/03	DRAWN BY: T Ybarra	GROUP DELTA CONSULTANTS, INC. 2291 W. 205th Street Suite 105 Torrance, CA 90501	PROJECT NUMBER: L-453
REVISED:	APPROVED BY: T Armstrong	**BORING LOCATIONS** The Olson Company - Villa Marina Marina Del Rey, California	SCALE: AS SHOWN
REVISED:			FIGURE NUMBER: 2

GROUP DELTA CONSULTANTS

Geotechnical Engineering

Geology

HydroGeology

Earthquake Engineering

Materials Testing & Inspection

Forensic Services

APPENDIX A
FIELD EXPLORATION

The subsurface conditions at the proposed improvement site were investigated on May 15 and May 19, 2003 by drilling two mud rotary wash borings and three Cone Penetration Test (CPT) soundings at the locations shown on Figure 2. The borings were advanced to a depth of about 41 and 59 feet. The CPT soundings were performed to depths between approximately 9 feet to 65 feet. Subsurface materials were visually classified and logged by our field engineer in accordance with the Unified Soil Classification System (USCS). Boring logs are presented in Figures A-1 and A-2. A key to the boring logs is presented in Figure A-0. CPT soundings are presented in Figures A-3 through A-5.

Relatively undisturbed drive samples and large samples of the materials encountered in the borings were obtained at the depth intervals noted on the boring logs. The drive samples were obtained with a 3-inch O.D. slit-barrel sampler lined with 1-inch metal rings. The samples were sealed to prevent moisture loss and returned to our laboratory for additional visual examination and laboratory testing. The sampler was driven into the soil using a 300-pound hammer falling a distance of 18 inches. The number of blows required to drive the sampler 12 inches is recorded on the boring logs. In addition, Standard Penetration Tests (SPT) were also conducted in accordance with ASTM D 1586, using a standard 2-inch outside diameter, 1.375-inch inside diameter, split-spoon sampler. The SPT sampler was driven into the soil using a 140-pound hammer free-falling 30 inches. The N-value blowcounts are shown directly on the boring logs.

Results of moisture content and dry density tests and pocket penetrometer tests are shown on the boring logs. Additional laboratory tests performed are indicated on the boring logs in the column labeled "Other Tests". The following abbreviations are used to identify these tests:

DS Direct Shear
WA Percent Passing No. 200 Sieve (-200 wash)
CN Consolidation

The following are attached and complete this appendix:

Figure A-0	Key to Log of Borings
Figures A-1 and A-2	Log of Borings
Figures A-3 through A-5	CPT Sounding

LOG OF TEST BORING

PROJECT NAME	PROJECT NUMBER	BORING
The Olson Company - Lincoln/Maxella	L-453	LEGEND

SITE LOCATION	START	FINISH	SHEET NO.
Marina Del Rey, CA	5/19/03	5/19/03	1 of 1

DRILLING COMPANY	DRILLING METHOD	LOGGED BY	CHECKED BY
A&W Drilling	Rotary Wash	N. Nghiem	T. Armstrong

DRILLING EQUIPMENT	BORING DIA. (in)	TOTAL DEPTH (ft)	GROUND ELEV (ft)	DEPTH/ELEV. GROUND WATER (ft)
Mayhew 1000	6	32		▼

SAMPLING METHOD	NOTES
SPT: Hammer: 140 lbs., Drop 30 in., Ring 300 lbs., Drop 18 in.	

DEPTH (ft)	ELEVATION (ft)	SAMPLE TYPE	SAMPLE NO.	PENETRATION RESISTANCE (BLOWS/ft)	DRY DENSITY (pcf)	MOISTURE (%)	OTHER TEST	GRAPHIC LOG	DESCRIPTION AND CLASSIFICATION
									GRAB, CAL, SPT - Refers to the sampling method as described below
5									GRAB - Refers to collecting sample by method of placing disturbed soil cuttings into a plastic bag
10									CAL (CALIFORNIA MODIFIED) - A 3.0" o.d. split tube sampler lined with 2.42" i.d. metal sample rings generally driven into the soil by a free falling hammer
15									SPT (STANDARD PENETRATION TEST) - A 2.0" o.d. split spoon sampler with a 1.375" i.d. generally driven into the soil with a 140# hammer free falling a height of 30"
20									ABBREVIATIONS FOR OTHER TESTS:
25									
30									

ABBREVIATIONS FOR OTHER TESTS:

- AL = Atterberg Limits
- CN = Consolidation
- CO = Corrosivity
- CP = Laboratory Compaction
- DS = Direct Shear
- LL = Liquid Limit
- GS = Grain Size Analyses
- PP = Pocket Pen
- RV = R-Value
- WA = Wash on #200 Sieve
- EI = Expansion Index

GROUP DELTA

GROUP DELTA CONSULTANTS, INC.

THIS SUMMARY APPLIES ONLY AT THE LOCATION OF THIS BORING AND AT THE TIME OF DRILLING. SUBSURFACE CONDITIONS MAY DIFFER AT OTHER LOCATIONS AND MAY CHANGE AT THIS LOCATION WITH THE PASSAGE OF TIME. THE DATA PRESENTED IS A SIMPLIFICATION OF THE ACTUAL CONDITIONS ENCOUNTERED.

FIGURE A-0

GDC_LOG_BORING_4C L453.GPJ GDC_WLOG.GDT 6/18/03

LOG OF TEST BORING

PROJECT NAME	PROJECT NUMBER	BORING
The Olson Company - Lincoln/Maxella	L-453	B-1

SITE LOCATION	START	FINISH	SHEET NO.
Marina Del Rey, CA	5/19/03	5/19/03	1 of 2

DRILLING COMPANY	DRILLING METHOD	LOGGED BY	CHECKED BY
A&W Drilling	Rotary Wash	N. Nghiem	T. Armstrong

DRILLING EQUIPMENT	BORING DIA. (in)	TOTAL DEPTH (ft)	GROUND ELEV (ft)	DEPTH/ELEV. GROUND WATER (ft)
Mayhew 1000	6	41	15	▼ 17.00 / -2.0

SAMPLING METHOD	NOTES
SPT: Hammer: 140 lbs., Drop 30 in., Ring 300 lbs., Drop 18 in.	

DEPTH (ft)	ELEVATION (ft)	SAMPLE TYPE	SAMPLE NO.	PENETRATION RESISTANCE (BLOWS/ft)	DRY DENSITY (pcf)	MOISTURE (%)	OTHER TEST	GRAPHIC LOG	DESCRIPTION AND CLASSIFICATION
									3" of Asphalt.
									3" of Gravel.
									FILL: Silty Sand (SM/SC), dark gray, interbedded with brown silty sand, with clay.
									Silty Sand (SM), medium dense, black, with organics, with some coarse gravel up to 2".
5	10	Ring	1	22	118	14.9			
		Ring	2	13	112	15.1			Medium dense, brown, w/some gravel.
									NATIVE: Lean Sandy CLAY (CL), medium stiff, brown, moist, fine to medium grained sand, with some gravel.
10	5								
		Ring	3	9	114	14.6	CN		
15	0								Stiff.
		SPT	4	14					SAND (SP), loose to medium dense, brown, fine to coarse grained. ▼
									Gravelly SAND (SP), very dense, brown, fine to coarse grained, poorly grade.
20	-5	SPT	5	50		11.7			
25	-10	SPT	6	65					Very dense.
30	-15	SPT	7	57		25.2			Very dense.
									Clayey SAND (SC), very dense, olive, fine to coarse grained sand.

GDC_LOG_BORING_4C L453.GPJ GDC_WLOG.GDT 6/16/03

GROUP DELTA

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FIGURE A-1 a

LOG OF TEST BORING

PROJECT NAME: The Olson Company - Lincoln/Maxella		PROJECT NUMBER: L-453	BORING: B-1	
SITE LOCATION: Marina Del Rey, CA	START: 5/19/03	FINISH: 5/19/03	SHEET NO.: 2 of 2	
DRILLING COMPANY: A&W Drilling	DRILLING METHOD: Rotary Wash	LOGGED BY: N. Nghiem	CHECKED BY: T. Armstrong	
DRILLING EQUIPMENT: Mayhew 1000	BORING DIA. (in): 6	TOTAL DEPTH (ft): 41	GROUND ELEV (ft): 15	DEPTH/ELEV. GROUND WATER (ft): ▼ 17.00 / -2.0
SAMPLING METHOD: SPT: Hammer: 140 lbs., Drop 30 in., Ring 300 lbs., Drop 18 in.		NOTES		

DEPTH (ft)	ELEVATION (ft)	SAMPLE TYPE	SAMPLE NO.	PENETRATION RESISTANCE (BLOWS/ft)	DRY DENSITY (pcf)	MOISTURE (%)	OTHER TEST	GRAPHIC LOG	DESCRIPTION AND CLASSIFICATION
35	-20		8	73	109	19.1			Very dense.
									SAND (SP), very dense, brown to olive, fine to medium grained, poorly graded.
40	-25		9	96/6"					Becomes very dense, olive brown, fine to coarse grained, poorly graded.
									Bottom of Boring B-1 @ 41 feet. Boring backfilled with soil cuttings, grout & capped with asphalt.
45	-30								
50	-35								
55	-40								
60	-45								

GDC_LOG_BORING_4C L453.GPJ GDC_WLOG.GDT 6/18/03

GROUP DELTA CONSULTANTS, INC.

THIS SUMMARY APPLIES ONLY AT THE LOCATION OF THIS BORING AND AT THE TIME OF DRILLING. SUBSURFACE CONDITIONS MAY DIFFER AT OTHER LOCATIONS AND MAY CHANGE AT THIS LOCATION WITH THE PASSAGE OF TIME. THE DATA PRESENTED IS A SIMPLIFICATION OF THE ACTUAL CONDITIONS ENCOUNTERED.

FIGURE A-1 b

LOG OF TEST BORING

PROJECT NAME	PROJECT NUMBER	BORING
The Olson Company - Lincoln/Maxella	L-453	B-2

SITE LOCATION	START	FINISH	SHEET NO.
Marina Del Rey, CA	5/19/03	5/19/03	1 of 2

DRILLING COMPANY	DRILLING METHOD	LOGGED BY	CHECKED BY
A&W Drilling	Rotary Wash	N. Nghiem	T. Armstrong

DRILLING EQUIPMENT	BORING DIA. (in)	TOTAL DEPTH (ft)	GROUND ELEV (ft)	DEPTH/ELEV. GROUND WATER (ft)
Mayhew 1000	6	58.8	15	⊻ 17.00 / -2.0

SAMPLING METHOD	NOTES
SPT: Hammer: 140 lbs., Drop 30 in., Ring 300 lbs., Drop 18 in.	

DEPTH (ft)	ELEVATION (ft)	SAMPLE TYPE	SAMPLE NO.	PENETRATION RESISTANCE (BLOWS/ft)	DRY DENSITY (pcf)	MOISTURE (%)	OTHER TEST	GRAPHIC LOG	DESCRIPTION AND CLASSIFICATION
									3" of Asphalt.
									3" of Gravel.
									FILL: Silty SAND (SM), dense, black, fine to medium grained, with some fine gravel.
		☒	1	21	118	13.4			
5	10								
		☒	2	21	119	15.4	DS		Sandy CLAY (CL), dense, dark gray, fine to medium grained.
									NATIVE: Sandy SILT (ML), brown, fine to medium grained.
10	5	☒	3	8		20.9			Loose to firm.
15	0	☒	4	17	120	16.9	DS		Medium dense.
								⊻	
									Gravelly.
20	-5	☒	5	35					Gravelly SAND (SP), dense, brown, fine to coarse grained, poorly graded.
25	-10	☒	6	80					Very dense.
30	-15	☒	7	76					

GDC_LOG_BORING_4C L453.GPJ GDC_WLOG.GDT 6/18/03

GROUP DELTA

GROUP DELTA CONSULTANTS, INC.

THIS SUMMARY APPLIES ONLY AT THE LOCATION OF THIS BORING AND AT THE TIME OF DRILLING. SUBSURFACE CONDITIONS MAY DIFFER AT OTHER LOCATIONS AND MAY CHANGE AT THIS LOCATION WITH THE PASSAGE OF TIME. THE DATA PRESENTED IS A SIMPLIFICATION OF THE ACTUAL CONDITIONS ENCOUNTERED.

FIGURE A-2 a

LOG OF TEST BORING

PROJECT NAME: The Olson Company - Lincoln/Maxella	PROJECT NUMBER: L-453	BORING: B-2	
SITE LOCATION: Marina Del Rey, CA	START: 5/19/03	FINISH: 5/19/03	SHEET NO.: 2 of 2
DRILLING COMPANY: A&W Drilling	DRILLING METHOD: Rotary Wash	LOGGED BY: N. Nghiem	CHECKED BY: T. Armstrong
DRILLING EQUIPMENT: Mayhew 1000	BORING DIA. (in): 6	TOTAL DEPTH (ft): 58.8	GROUND ELEV (ft): 15
DEPTH/*ELEV.* GROUND WATER (ft): ▼ 17.00 / *-2.0*			
SAMPLING METHOD: SPT: Hammer: 140 lbs., Drop 30 in., Ring 300 lbs., Drop 18 in.	NOTES:		

DEPTH (ft)	ELEVATION (ft)	SAMPLE TYPE	SAMPLE NO.	PENETRATION RESISTANCE (BLOWS/ft)	DRY DENSITY (pcf)	MOISTURE (%)	OTHER TEST	GRAPHIC LOG	DESCRIPTION AND CLASSIFICATION
35	-20	⊠	8	84/6"					Grades to fine to medium grain.
									Clayey SILT and fine to medium Sandy SILT (ML), stiff, gray.
40	-25	■	9	17	98	28.3	CN		
									Clayey SAND (SC), dense, olive gray, fine to medium grained, sand with clay lense.
									SAND (SP), very dense, gray, fine to medium grained, poorly graded.
45	-30	⊠	10	50/6"					
50	-35	■	NSR	49					Medium dense to dense.
		■	11	55					
55	-40								@ 54 feet: Interbedding of Fat Clayey Silt, gray
		⊠	12	92/3"					Very dense, gray.
60	-45								Bottom of Boring B-2 @ 58.8 feet. Groundwater encountered @ 17 feet. Boring backfilled with soil cuttings and capped with asphalt.

GROUP DELTA CONSULTANTS, INC.

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FIGURE A-2 b

GDC_LOG_BORING_4C L453.GPJ GDC_WLOG.GDT 6/16/03

APPENDIX E

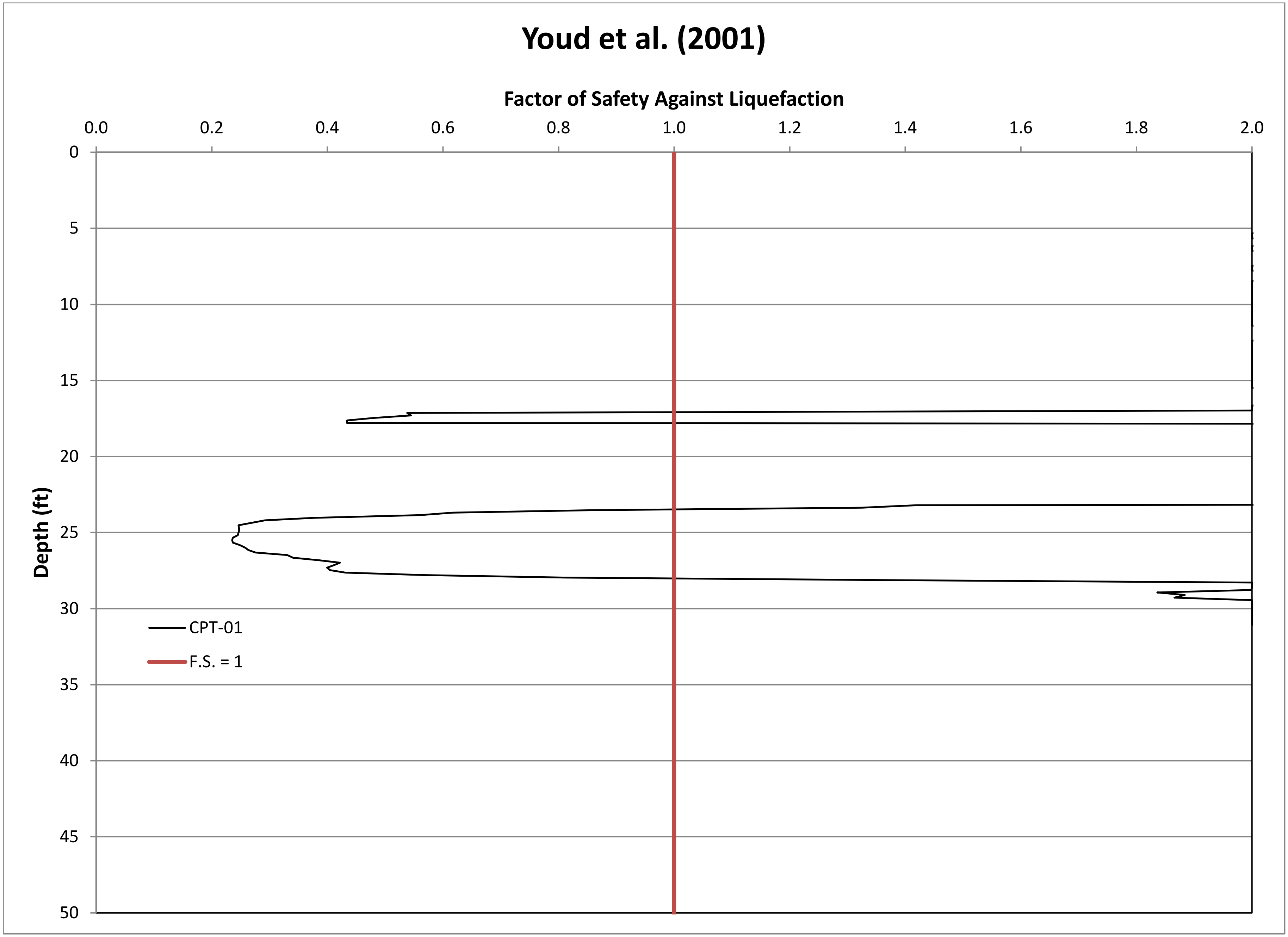
PERCOLATION TEST RESULTS

Percolation Test: PT-01

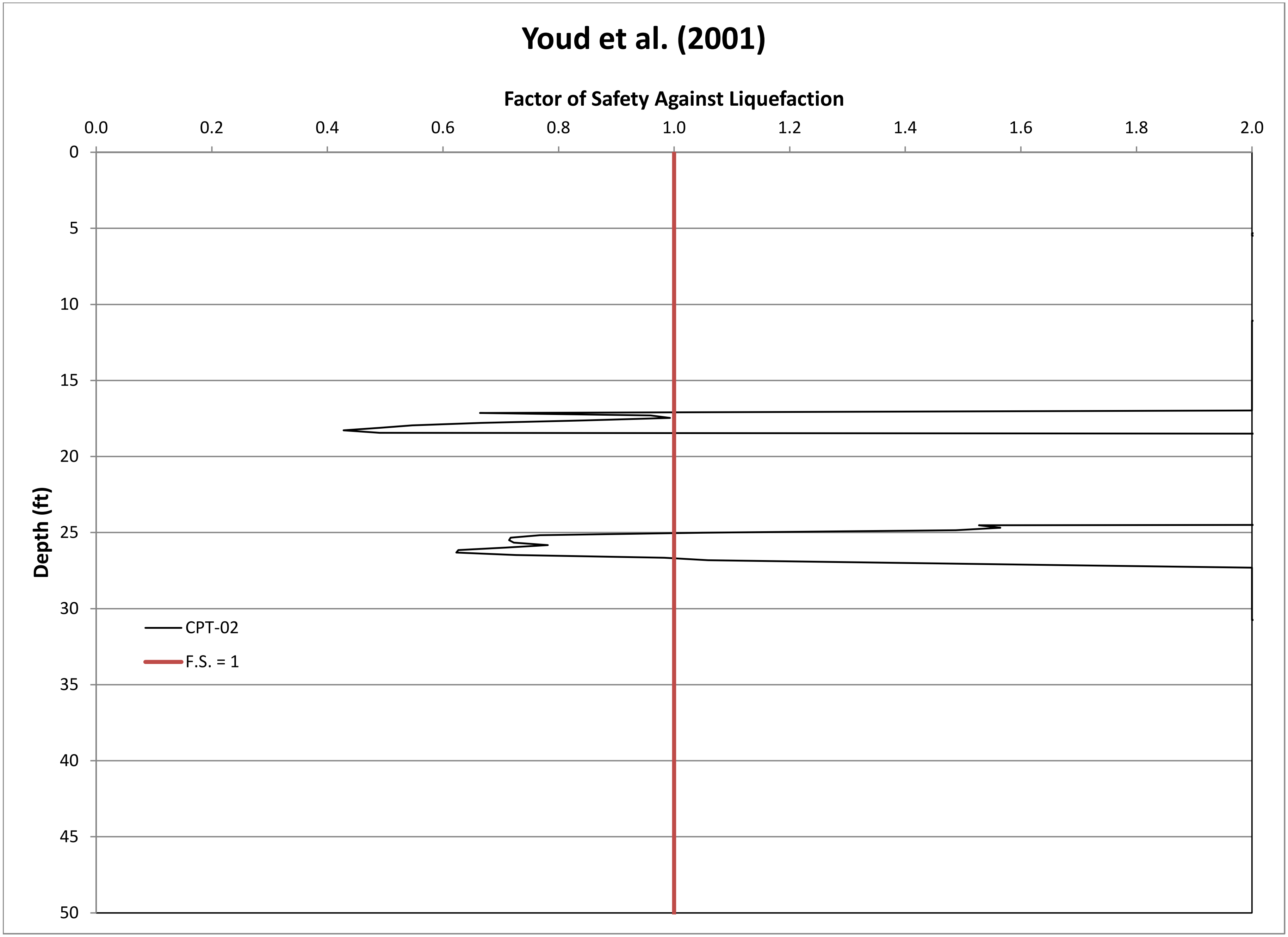
Pre-Soak (5 gallons)			Percolation Test (5 gallons)				
Elapsed Time (minutes)	Depth to Water Level (inches)	Water Level Height (inches)	Elapsed Time (minutes)	Depth to Water Level (inches)	Water Level Height (inches)	Percolation Rate (minutes/inch)	Infiltration Rate (inches/hour)
0	91.3	52.7	0	91.3	52.7	-	-
2	91.6	52.4	30	97.8	46.2	4.6	0.5
4	91.8	52.2	60	117.2	26.8	1.5	2.0
6	92.2	51.8	90	123.8	20.2	4.5	1.0
8	92.2	51.8	120	127.7	16.3	7.8	0.8
10	92.3	51.7	150	130.7	13.3	10.0	0.7
15	92.5	51.5					
20	92.8	51.2					
25	93.0	51.0					
30	93.0	51.0					
35	93.2	50.8					
40	93.4	50.6					
45	93.5	50.5					
50	93.5	50.5					
55	93.7	50.3					
60	93.7	50.3					
65	93.8	50.2					

APPENDIX F

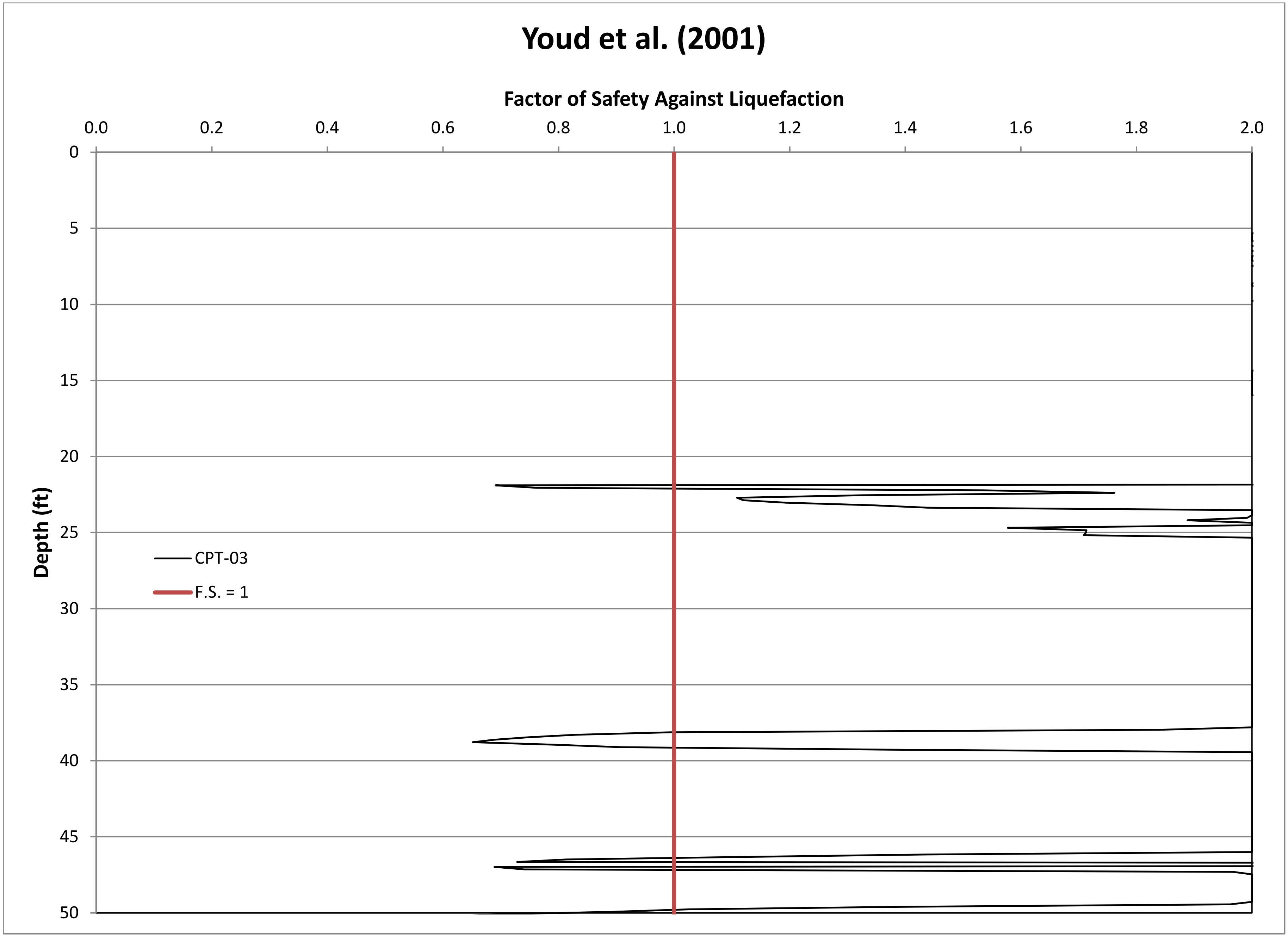
RESULTS OF LIQUEFACTION EVALUATION



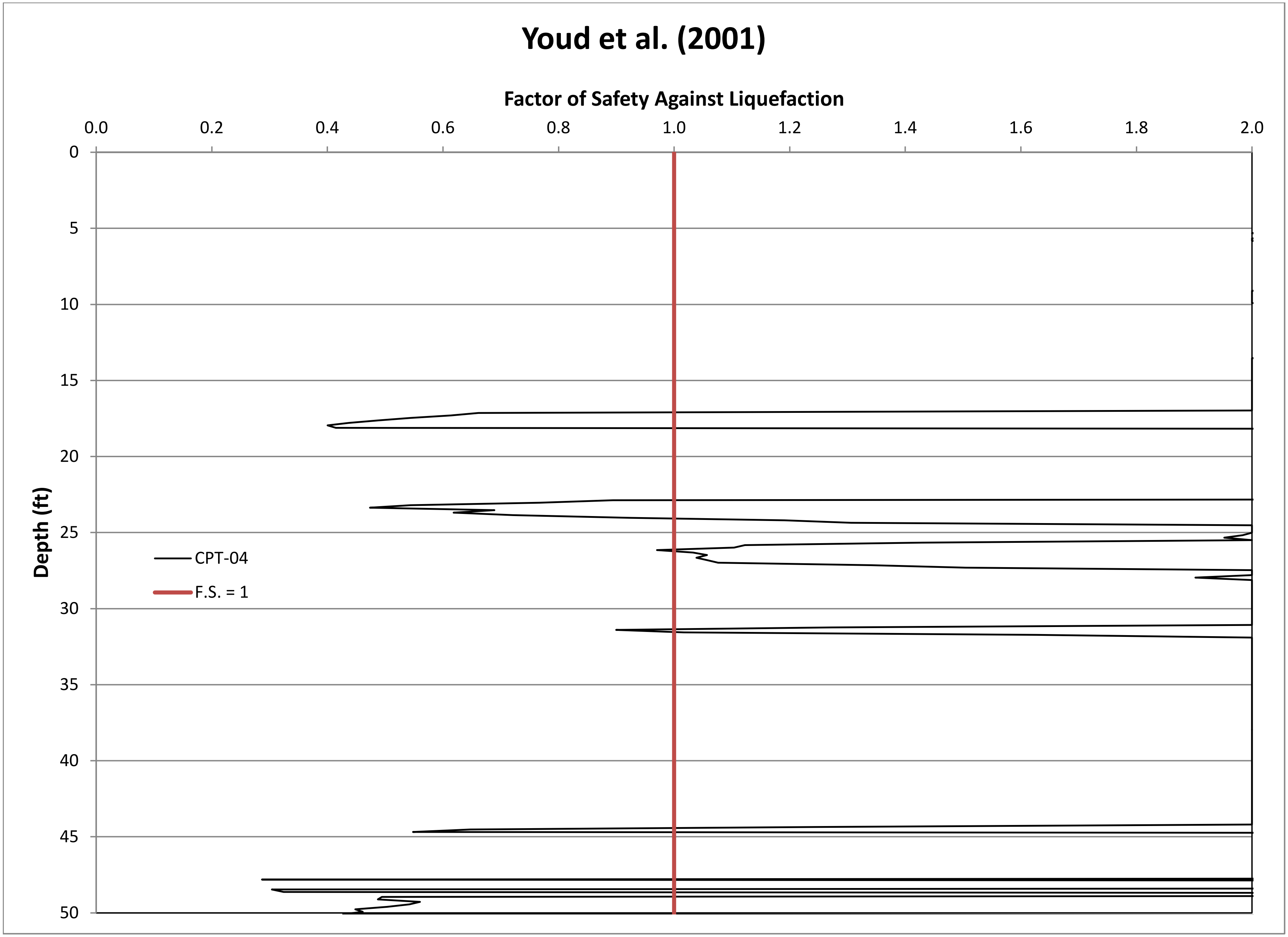
Youd et al. (2001)
Factor of Safety Against Liquefaction
0.0
0.2
0.4
0.6
0.8
1.0
1.2
1.4
1.6
1.8
2.0
Depth (ft)
0
5
10
15
20
25
30
35
40
45
50
CPT-01
F.S. = 1

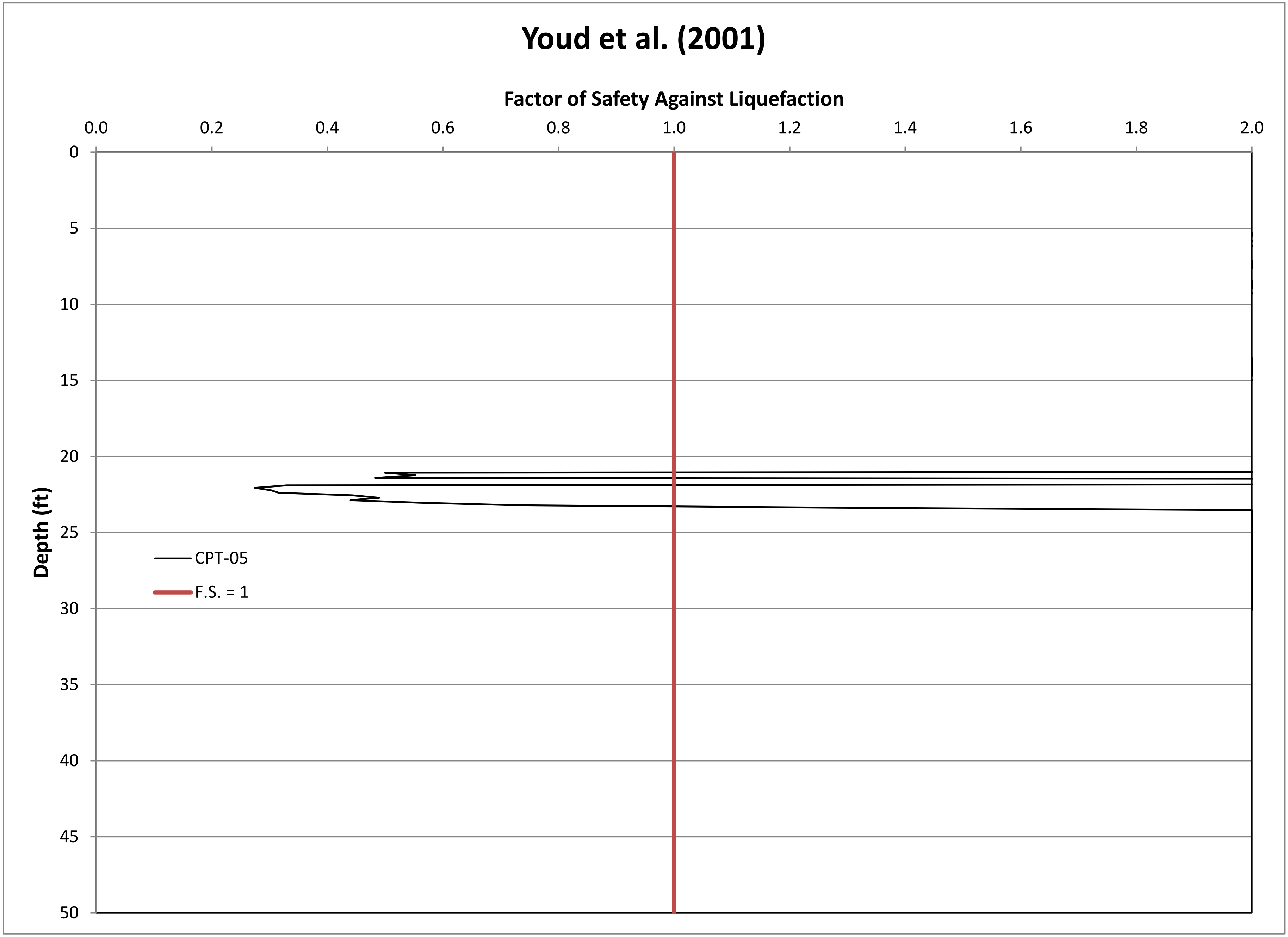


Youd et al. (2001)
Factor of Safety Against Liquefaction
0.0
0.2
0.4
0.6
0.8
1.0
1.2
1.4
1.6
1.8
2.0
0
5
10
15
20
25
30
35
40
45
50
Depth (ft)
CPT-02
F.S. = 1

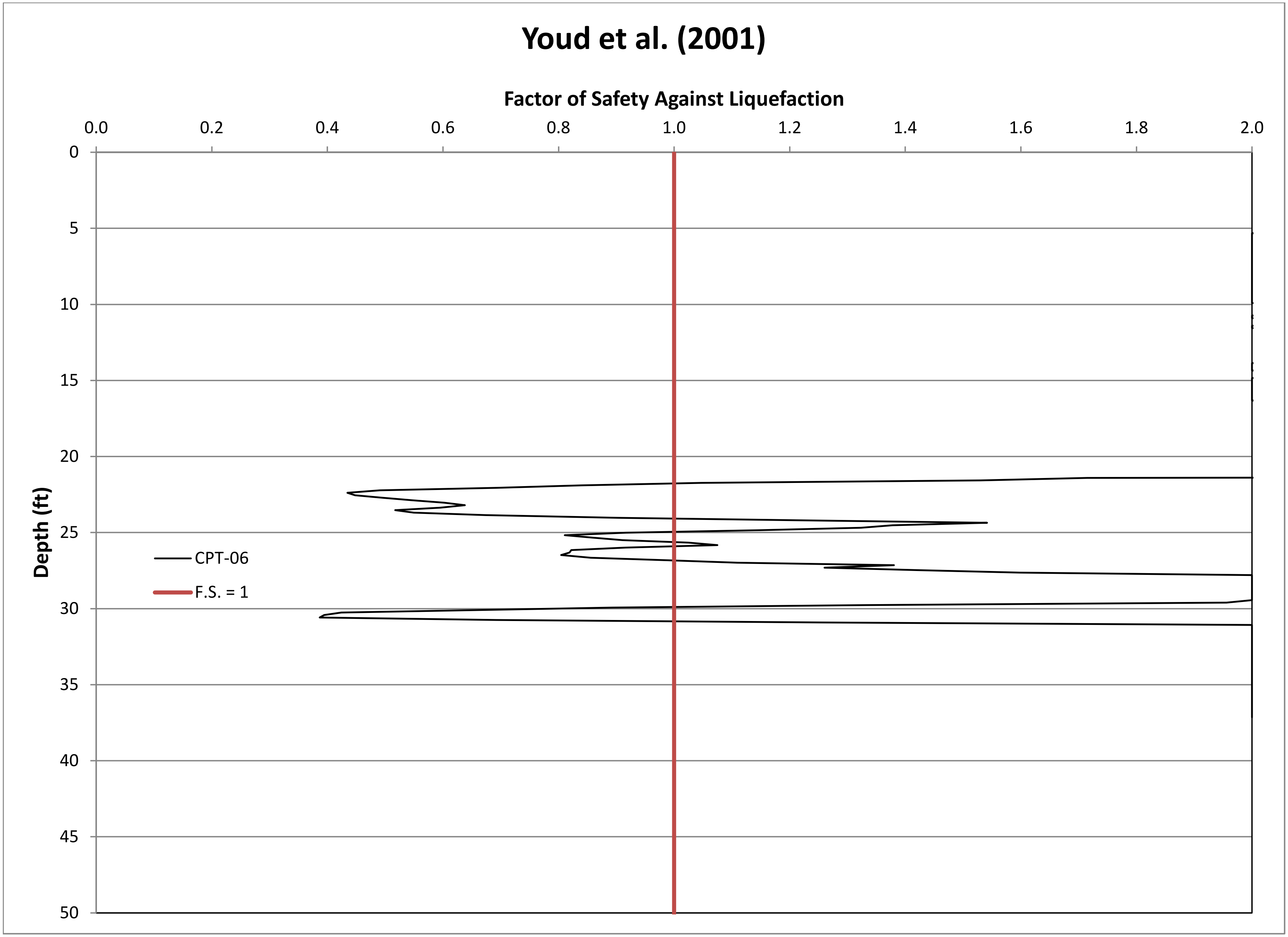


Youd et al. (2001)
Factor of Safety Against Liquefaction
0.0
0.2
0.4
0.6
0.8
1.0
1.2
1.4
1.6
1.8
2.0
Depth (ft)
0
5
10
15
20
25
30
35
40
45
50
CPT-03
F.S. = 1


Youd et al. (2001)
Factor of Safety Against Liquefaction
0.0
0.2
0.4
0.6
0.8
1.0
1.2
1.4
1.6
1.8
2.0
0
5
10
15
20
25
30
35
40
45
50
Depth (ft)
CPT-04
F.S. = 1



Youd et al. (2001)
Factor of Safety Against Liquefaction
0.0
0.2
0.4
0.6
0.8
1.0
1.2
1.4
1.6
1.8
2.0
Depth (ft)
0
5
10
15
20
25
30
35
40
45
50
CPT-05
F.S. = 1


Youd et al. (2001)
Factor of Safety Against Liquefaction
0.0
0.2
0.4
0.6
0.8
1.0
1.2
1.4
1.6
1.8
2.0
0
5
10
15
20
25
30
35
40
45
50
Depth (ft)
CPT-06
F.S. = 1

APPENDIX G

IMPORTANT INFORMATION ABOUT YOUR GEOTECHNICAL ENGINEERING REPORT
(by ASFE)

Important Information About Your
Geotechnical Engineering Report

Subsurface problems are a principal cause of construction delays, cost overruns, claims, and disputes.
The following information is provided to help you manage your risks.

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

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

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

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

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

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

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

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

Subsurface Conditions Can Change

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

Most Geotechnical Findings Are Professional Opinions

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

A Report's Recommendations Are *Not* Final

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

A Geotechnical Engineering Report Is Subject To Misinterpretation

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

Do Not Redraw the Engineer's Logs

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

Give Contractors a Complete Report and Guidance

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

Read Responsibility Provisions Closely

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

Geoenvironmental Concerns Are Not Covered

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

Rely on Your Geotechnical Engineer for Additional Assistance

Membership in ASFE exposes geotechnical engineers to a wide army of risk management techniques that can be of genuine benefit for everyone involved with a construction project. Confer with your ASFE-member geotechnical engineer for more information.

ASFE

8811 Colesville Road Suite 3106 Silver Spring. MD 20910
Telephone: 301-565-2733 Facsimile: 301-589-2017
email: info@asde.org www.asfe.org

At Golder Associates we strive to be the most respected global group of companies specializing in ground engineering and environmental services. Employee owned since our formation in 1960, we have created a unique culture with pride in ownership, resulting in long-term organizational stability. Golder professionals take the time to build an understanding of client needs and of the specific environments in which they operate. We continue to expand our technical capabilities and have experienced steady growth with employees now operating from offices located throughout Africa, Asia, Australasia, Europe, North America and South America.

Africa	+ 27 11 254 4800
Asia	+ 852 2562 3658
Australasia	+ 61 3 8862 3500
Europe	+ 356 21 42 30 20
North America	+ 1 800 275 3281
South America	+ 55 21 3095 9500

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Irvine, CA 92606
Tel: (714) 508-4400
Fax: (949) 483-2339

Appendix D.3

Soils Report Approval Letter

DEPARTMENT OF
BUILDING AND SAFETY
201 NORTH FIGUEROA STREET
LOS ANGELES, CA 90012

FRANK M. BUSH
GENERAL MANAGER
SUPERINTENDENT OF BUILDING

OSAMA YOUNAN, P.E.
EXECUTIVE OFFICER

SOILS REPORT APPROVAL LETTER

October 25, 2017

LOG # 94712-02
SOILS/GEOLOGY FILE - 2
LIQ

Rarz-Marina Market Place, LLC
13450 W. Maxella Ave
Los Angeles, CA 90292

TRACT:	P M 3391
LOT(S):	A & FR B (ARB 2)
LOCATION:	13450 W. Maxella Ave

CURRENT REFERENCE REPORT/LETTER(S)	REPORT No.	DATE(S) OF DOCUMENT	PREPARED BY
Addendum Report	1657237	10/02/2017	Golder Associates

PREVIOUS REFERENCE REPORT/LETTER(S)	REPORT No.	DATE(S) OF DOCUMENT	PREPARED BY
Dept. Correction Letter	94712-01	08/28/2017	LADBS
Soils Report	1657237	08/08/2017	Golder Associates
Laboratory Data	GLDL-17-014	07/24/2017	Hushmand Associates, INC
Dept. Correction Letter	94712	09/23/2016	LADBS
Soils Report	1403929	01/16/2015	Golder Associates
CPT Data	--	9/25/2014	Kehoe Testing & Engineering

The Grading Division of the Department of Building and Safety has reviewed the referenced reports that provide liquefaction evaluation and preliminary foundation recommendations for the proposed 660 unit apartment structure with 25,000 S.F. of retail space. The evaluation is for the purpose of filling a vesting tentative tract (VTT-74415) with the Department of City Planning. The structure will be 7 levels above ground and up to 2 levels below grade (9 levels total). The site is currently occupied by several retail buildings and a surface parking lot. The earth materials at the subsurface exploration locations consist of up to 3 feet of uncertified fill underlain by native. The consultants recommend to support the proposed structure(s) on mat-type foundations bearing on native undisturbed soils.

The site is located in a designated liquefaction hazard zone as shown on the Seismic Hazard Zones map issued by the State of California. The Liquefaction study included as a part of the report/s demonstrates that the site soils are subject to liquefaction. The earthquake induced total a settlements is calculated to be 2.2. To mitigate the earthquake induced settlements it is proposed to use a mat foundation. The requirements of the 2017 City of Los Angeles Building Code have been satisfied.

The referenced reports are acceptable, provided the following conditions are complied with during site development:

(Note: Numbers in parenthesis ( ) refer to applicable sections of the 2017 City of LA Building Code. P/BC numbers refer the applicable Information Bulletin. Information Bulletins can be accessed on the internet at LADBS.ORG.)

1. This referenced reports are approved for the purpose of filing a vesting tentative tract with the Department of City Planning only. No building or grading permits shall be issued based on the referenced reports and this approval letter.

2. Prior to any issuance of permit a design-level geotechnical study shall be submitted to the Grading Department. Geotechnical recommendations and calculations for temporary excavations, shoring, permanent basement walls and Mat-foundations shall be provided.

3. Prior to the issuance of any permit, secure approval from the Division of Land Unit of the Department of City Planning for the proposed lot split and residential development of the of the property. The Division of Land Unit of the Planning Department is located in City Hall, 200 N. Spring Street, Room # 750 - Phone (213) 978-1362.

4. This approval does ***not*** extend to the use of an on-site infiltration systems. If an on-site infiltration system is proposed, the consultant shall provide an evaluation on the items discussed in Information Bulletin P/BC 2017-118 in a supplemental report with plans drawn to scale and suitable for reproduction and archiving purposes that clearly shows the location of the infiltration facility, all property lines, proposed and existing grades and structures, and the location of the proposed infiltration system. The plan shall be provided on the soils consultant's stationary or shall be signed and stamped by the soils engineer. Note: On-site infiltration systems are required to be a minimum of 10 feet (in any direction) from any foundation, and a minimum of 10 feet horizontally from private property lines.

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