ALBUS-KEEFE & ASSOCIATES, INC
GEOTECHNICAL CONSULTANTS

April 5, 2019
J.N.: 2789.00

Mr. Mitchell Gardner
G3 Urban
15235 S. Western Avenue
Gardena, CA 90249

Subject: **Preliminary Geotechnical Investigation, Proposed Commercial and Residential Development, 2129 W. Rosecrans Avenue, Gardena, California**

Dear Mr. Gardner,

Pursuant to your request, ***Albus-Keefe & Associates, Inc***. is pleased to present to you our preliminary geotechnical investigation report for the subject development. This report presents the results of our field investigation, laboratory testing, engineering analyses, as well as our preliminary geotechnical recommendations for design and construction of the subject development.

We appreciate this opportunity to be of service to you. If you have any questions regarding the contents of this report, please do not hesitate to call this office.

Sincerely,

ALBUS-KEEFE & ASSOCIATES, INC.

Paul Kim
Associate Engineer

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1.0 INTRODUCTION

1.1 PURPOSE AND SCOPE

The purposes of our preliminary geotechnical investigation were to evaluate geotechnical conditions within the project area and to provide conclusions and recommendations relevant to the design and construction of the proposed improvements at the subject site. The scope of this investigation included the following:

- Review of the referenced historical aerial photographs and environmental report
- Review of published geologic and seismic data for the site and surrounding area
- Exploratory drilling and soil sampling
- Laboratory testing of selected soil samples
- Engineering analyses of data obtained from our review, exploration, and laboratory testing
- Evaluation of site seismicity, liquefaction, and settlement potential
- Preparation of this report

1.2 SITE LOCATION AND DESCRIPTION

The site is located at 2129 West Rosecrans Avenue, in the city of Gardena, California. The site consists of approximately 5 acres of land. Currently, the site consists of a taxi-cab facility that also provides auto maintenance of their vehicles. The site is developed with a single-story commercial building and a shed area where maintenance of auto vehicles take place. A small car wash facility is located at the northeast portion of the site. Undeveloped land is also present at the southeast corner of the site. Concrete- and asphalt-paving is present in the remainder of the site. The site is bordered by West Rosecrans Avenue to the south and industrial buildings to the west, north, and east. In addition, a large industrial structure is situated along the east property line. The location of the site and its relationship to the surrounding areas is shown on the Site Location Map, Figure 1.

The site is relatively level with elevations that vary from approximately 49 feet above mean sea level (MSL) to 52 feet above MSL based on Google Earth 2018. Drainage at the site appears to be directed toward concrete v-gutters located throughout the site which is then directed to the south towards West Rosecrans Avenue. Vegetation within the taxi cab facility is sparse and consists of a few trees within the surface lot. Ground cover and a few trees are also located within the undeveloped portion of the site.

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SITE LOCATION MAP

G3 Urban
Proposed Commercial and Residential Development
2129 W. Rosecrans Avenue
Gardena, California

NOT TO SCALE

FIGURE 1

1.3 PROPOSED DEVELOPMENT

We understand that the site will be redeveloped for residential use. It is anticipated that the proposed site development will consist of 105 residential units and associated interior driveways, perimeter/retaining walls and underground utilities. Minor rough grading is also anticipated. We also understand that approximately 0.56 acre of the southwest portion of the site will be developed with a retail building.

No grading or structural plans were available in preparing this report. However, we anticipate that minor rough grading of the site will be required to achieve future surface configuration and we expect the proposed residential dwellings will be up to 3-story, wood-framed structures with concrete slabs on grade yielding relatively light foundation loads. For the retail building, we have assumed maximum column loads of less than 75 kips per column and maximum wall loads of less than 3 kips per linear foot.

2.0 INVESTIGATION

2.1 RESEARCH

We have reviewed the referenced geologic publications, geologic maps, environmental report, and historic aerial photos (see references). Data from these sources were utilized to develop some of the findings and conclusions presented herein.

Based on our review, the site was divided into several sections since at least 1962. A car storage lot was located at the west portion of the site, a waste facility to the southeast, and an undeveloped land to the northeast. By 1963, the waste facility was expanded to the northeast to the northern limits of the site. Between 1972 to 1980, the current industrial building at the southwest corner of the site was constructed. In addition, the car storage lot was removed and the waste facility was expanded to the west to the property line. The waste facility was later removed by 1994 leaving the southeast portion of the site undeveloped and the current shed area was constructed to the northwest. By 2003, the west parking lot was expanded. The site has remained relatively unchanged since 2003.

2.2 SUBSURFACE EXPLORATION

Subsurface exploration for this investigation was conducted on January 22, 2019 and March 28, 2019. Our initial investigation consisted of four (4) exploratory borings and six (6) cone penetration test soundings to depths ranging from approximately 21.5 to 51.5 feet below the existing ground surface (bgs). The borings were drilled using a truck-mounted, continuous flight, hollow-stem-auger drill rig.

The CPT soundings were advanced using a 30-ton CPT truck. As the cone is advanced through the soil, direct measurements are obtained and recorded for tip resistance, side resistance and porewater measurements. The relationship between the tip resistance and the side resistance allows a determination of the general soil type. Following completion of the CPT boring, a log is generated that provides a continuous profile of the tip resistance, side resistance and porewater measurements. Copies of the CPT logs are provided in Appendix A.

Our secondary investigation was completed to investigate the magnetic anomalies determined by the geophysical survey performed on the site. This investigation consisted of excavating twelve (12) test pits (TP-1 through TP-12) to the depths of 3 to 7 feet bgs. The test pits were excavated using a rubber-tired backhoe. The locations of the test pits are shown on the *Geophysical Interpretation Map DRAFT* provided by the environmental consultant.

A representative of *Albus-Keefe & Associates, Inc.* logged the exploratory borings and test pits. Visual and tactile identifications were made of the materials encountered within the borings and test pits, and their descriptions are presented in the Exploration Logs in Appendix A. The approximate locations of the exploratory excavations completed by this firm are shown on the enclosed Geotechnical Map, Plate 1 and Plot Plan, Plate 2.

Bulk, relatively undisturbed and Standard Penetration Test (SPT) samples were obtained at selected depths within the exploratory borings and test pits for subsequent laboratory testing. Relatively undisturbed samples were obtained using a 3-inch O.D., 2.5-inch I.D., California split-spoon soil sampler lined with brass rings. SPT samples were obtained from the borings using a standard, unlined SPT soil sampler. During each sampling interval, the sampler was driven 18 inches with successive drops of a 140-pound automatic hammer falling 30 inches. The number of blows required to advance the sampler was recorded for each six inches of advancement. The total blow count for the lower 12 inches of advancement per soil sample is recorded on the exploration log. Samples were placed in sealed containers or plastic bags and transported to our laboratory for analyses. The borings were backfilled with auger cuttings upon completion of sampling. Borings within asphalt-paved areas were capped with asphalt cold patch.

In addition, a percolation test well, P-1 was utilized for percolation testing. The percolation test well was excavated to an approximated depth of 15 feet in the vicinity of exploratory boring B-1. The percolation test well was later backfilled upon completion of testing. Results of our percolation testing are discussed in a separate report.

2.3 LABORATORY TESTING

Selected samples of representative earth materials from our borings and test pits were tested in the laboratory. Tests consisted of USCS classification, in-situ moisture content and dry density, maximum dry density and optimum moisture content, consolidation/collapse, direct shear strength, sieve analysis, expansion index, Atterberg Limits, corrosivity (pH, chloride, and resistivity), percent passing No. 200 sieve, and soluble sulfate content. Descriptions of laboratory testing and the test results are presented in Appendix B and on the Exploration Logs in Appendix A.

3.0 GEOLOGIC CONDITIONS

3.1 SOIL CONDITIONS

Descriptions of the earth materials encountered during our investigation are summarized below and are presented in detail on the Exploration Logs presented in Appendix A.

The soils encountered within the site generally consist of artificial fill materials overlying older alluvial deposits. The artificial materials were observed in borings B-1 through B-4 to be up to approximately 7.5 feet thick. The artificial fill materials generally consisted of gray, brown, and black clay, silty sand, and clayey sand that are typically medium dense /stiff to very stiff. Debris was observed within the artificial fill. Specifically, the debris was observed in all of the test pits and borings B-1 and B-2. On average, the amount of debris is estimated to be less than 5% by volume. Debris is likely anticipated within the upper 2 to 3 feet of the artificial fill and generally limited to the eastern 2/3 of the site where the waste facility was located. Debris may be likely more present within the southern portion where the waste facility was originally located.

The older alluvial materials were encountered beneath the artificial fills to the maximum depth explored, 51.5 feet below the existing ground surface. The alluvial materials are alternating fine-grained and coarse-grained material. The fine-grained material consisted of brown clay and silt with varying amounts of sand that are damp to wet and very stiff to hard. The coarse-grained material consisted of brown silty and clayey sand that are damp to wet and medium dense to dense.

3.2 GROUNDWATER CONDITIONS

Groundwater was encountered during this firm's investigation to the depth of 23.6 feet below the existing ground surface. A review of the referenced Seismic Hazard Zone Report 027 indicates that historical high groundwater level for the general site area was estimated at approximately 25 feet below the existing ground surface.

3.3 FAULTING

Based on our review of the referenced publications and seismic data, no active faults are known to project through or immediately adjacent the site and the site does not lie within an "Earthquake Fault Zone" as defined by the State of California in the Alquist-Priolo Earthquake Fault Zoning Act. Table 3.1 presents a summary of known seismically active faults within 10 miles of the site, based on the 2008 National Seismic Hazard Maps.

TABLE 3.1
SUMMARY OF ACTIVE FAULTS

Name	Distance (miles)	Slip Rate (mm/yr.)	Preferred Dip (degrees)	Slip Sense	Rupture Top (km)	Fault Length (km)
Newport-Inglewood, alt 1	1.65	1	88	strike slip	0	65

Newport Inglewood Connected alt 1	1.65	1.3	89	strike slip	0	208
Newport Inglewood Connected alt 2	1.93	1.3	90	strike slip	0	208
Puente Hills (LA)	6.84	0.7	27	thrust	2.1	22
Palos Verdes	7.36	3	90	strike slip	0	99
Palos Verdes Connected	7.36	3	90	strike slip	0	285

4.0 ANALYSES

4.1 SEISMICITY

We have performed probabilistic seismic analyses utilizing the U.S. Seismic Design Maps web application by the U.S. Geological Survey (USGS). From our analyses, we obtain a PGA of 0.597g in accordance with Figure 22-7 of ASCE 7-10. The site factor for Site Class D in this range of PGA is F_{PGA} = 1.0. Therefore, the PGA_M = 1.0 x 0.597 = 0.60g. The mean event associated with a probability of exceedance equal to 2% over 50 years has a moment magnitude of 6.73 with a mean distance to the seismic source of 7.0 miles.

4.2 SETTLEMENT

Analyses were performed to evaluate potential for static settlement especially on the existing artificial fill and alluvial deposits. Our analyses were based on the results of consolidation tests performed on selected samples from our borings as well as the recorded blow counts during the exploration. Results of our testing indicate the native site materials has a relatively low compressibility. Due to its unsuitability, we have assumed that the artificial fill will be removed and recompacted. We estimate that settlement of foundations would undergo a total settlement less than 1 inch.

4.3 LIQUEFACTION

We have performed engineering analyses to evaluate the liquefaction potential at the site should the risk-targeted maximum earthquake event occur. We reviewed the subsurface data from both the borings and CPT soundings and have determined that the site is underlain relatively uniform materials. As a result, we utilized the information obtained from CPT-1 in our CLiq analyses. The analyses followed the guidelines presented in the CGS Special Publication 117A (2008), as modified in the procedures by Youd, et al. (2001) using seismicity parameters discussed in Section 4.1 above.

The results of CPT-based liquefaction analyses and its consequences are also provided in Appendix C which includes the related CLiq report, based on Robertson (NCEER 2001). Groundwater was assumed at depth 23.6 feet below ground surface corresponding to the groundwater level encountered during the investigation, and shallower than historically high groundwater level per CGS Seismic Hazard Zone Report 027. Soils with a plasticity index above 12 or Soil Behavior Type Index, IC, greater than 2.6 were assumed not susceptible to liquefaction.

Based on our analyses, liquefaction may occur below the site during periods of strong ground shaking. Our analyses indicate liquefaction could lead to a total settlement of the ground surface of less than approximately 0.5 inch due to seismic consolidation during liquefaction. Differential settlement due to seismic settlement would likely be on the order of ½ of the total settlement or approximately 0.25 inch over 30 feet. The details of the liquefaction-induced settlement analyses are provided within Appendix C.

5.0 CONCLUSIONS

5.1 FEASIBILITY OF PROPOSED DEVELOPMENT

From a geotechnical point of view, the proposed site improvements are considered feasible provided the recommendations presented in this report are incorporated into the design and construction of the project. Furthermore, it is also our opinion that the proposed development will not adversely impact the stability of adjoining properties if the recommendations presented in this report are incorporated into site development. Key issues that could have significant fiscal impacts on the geotechnical aspects of the proposed site development are discussed in the following sections of this report.

5.2 GEOLOGIC HAZARDS

5.2.1 Ground Rupture

No active faults are known to project through the site nor does the site lie within the bounds of an "Earthquake Fault Zone" as defined by the State of California in the Alquist-Priolo Earthquake Fault Zoning Act. As such, the potential for ground rupture due to fault displacement beneath the site is considered very low. The nearest zoned fault is the Newport-Inglewood fault located approximately 1.65 miles to the east.

5.2.2 Ground Shaking

The site is located in a seismically active area that has historically been affected by moderate to occasionally high levels of ground motion. The site lies in relatively close proximity to several seismically active faults; therefore, during the life of the proposed development, the property will probably experience moderate to occasionally high ground shaking from these fault zones, as well as some background shaking from other seismically active areas of the southern California region. Design of proposed structures in accordance with the current CBC is anticipated to adequately mitigate concerns with ground shaking.

5.2.3 Liquefaction

Based on our engineering analyses discussed previously, a number of thin layers of granular soils below groundwater are susceptible to liquefaction. The results of our analyses indicate a total seismic settlement of less than 0.5 inch and differential settlement of 0.5 inches over 30 feet.

Based on the State of California Special Publication 117A, hazards from liquefaction should be mitigated to the extent required to reduce seismic risk to “acceptable levels”. The acceptable level of risk means, “that level that provides reasonable protection of the public safety” [California Code of Regulations Title 14, Section 3721 (a)]. Protection of public safety does not require that structures be resistant to cracking or general distress due to differential movements. As such, a greater allowance for differential movement during liquefaction events is acceptable compared to the design requirements for static conditions.

The use of well-reinforced foundations, such as robust post-tensioned slabs, grade beams with structural slabs, or mat foundations have been proven to adequately provide basal support for similar structures during liquefaction events comparable to the predicted site event. Specific design recommendations for such systems are provided in Section 6.3.6.

5.3 STATIC SETTLEMENT

Provided grading and construction are performed in accordance with the recommendations provided herein, estimated total and differential settlement of proposed site improvements are anticipated to be less than 1 inch and ½ inch over 30 feet, respectively. These magnitudes of settlement are considered within tolerable limits of proposed site development.

5.4 EARTHWORK AND MATERIAL CHARACTERISTICS

The subsurface soils are anticipated to be relatively easy to excavate with conventional heavy earthmoving equipment. Most of the existing fill materials are near or above optimum moisture content and may require drying and/or mixing to achieve proper compaction.

Debris from the previous site development are expected within the upper 2 to 3 feet of the existing artificial fill. Deleterious debris, such as scrap metal, wires, etc. will require removal. Some of the debris, such as brick, asphalt and concrete, may be incorporated into the engineered fill.

Offsite improvements exist near the property lines. The presence of the existing improvements may limit removals of unsuitable materials adjacent the property lines. Special grading techniques, such as slot cutting or other acceptable criteria, may be required when grading adjacent the property lines. Specific recommendations can be provided by the geotechnical consultant upon request.

Onsite disposal systems, clarifiers and other underground improvements may be present beneath the site. If encountered during future rough grading, these improvements will require proper abandonment or removal.

5.5 SHRINKAGE/BULKAGE AND SUBSIDENCE

Volumetric changes in earth quantities will occur when excavated onsite soil materials are replaced as properly compacted fill. We estimate the existing fill soils will shrink less than 5 percent. In addition, excavations into site materials located within the eastern 2/3 of the site is also anticipated to expose deleterious debris within the upper 2 to 3 feet. These debris will require removal and have a potential for loss of materials less than 5 percent. Subsidence is not anticipated during removals. We estimate an overall loss of about 5 percent due to shrinkage. The estimates are intended as an aid for project engineers in determining earthwork quantities. However, these estimates should be used with some caution since they are not absolute values. Contingencies should be made for balancing earthwork quantities based on actual bulkage and debris excavated that occurs during the grading process.

5.6 SOIL EXPANSION

Based on our laboratory test results and experience of the surrounding area, the near-surface soils within the site are generally anticipated to possess **Low to High** expansion potentials. Additional testing for soil expansion will be required subsequent to rough grading and prior to construction of foundations and other concrete flatwork to confirm these conditions. The presence of expansive soils will tend to swell when wetted and shrink when dried. This characteristic will result in differential movement of structures and other site improvements. Specific recommendations to mitigate the adverse effects of expansive soils are provided in the following sections.

6.0 RECOMMENDATIONS

6.1 EARTHWORK

6.1.1 General Earthwork and Grading Specifications

All earthwork and grading should be performed in accordance with all applicable requirements of the grading codes of the City of Gardena, California and CAL OSHA, in addition to recommendations presented herein.

6.1.2 Pre-Grade Meeting and Geotechnical Observation

Prior to commencement of earthwork operations and foundation installation, we recommend a meeting be held between City Inspector, general contractor, civil engineer, and geotechnical consultant to discuss proposed earthwork and logistics.

We also recommend that a geotechnical consultant be retained to provide soil engineering and engineering geologic services during site development. This is to observe compliance with the design specifications and recommendations, and to allow design changes in the event that subsurface conditions differ from those anticipated. If conditions are encountered during construction that appears to be different than those indicated in this report, the project geotechnical consultant should be notified immediately. Design and construction revisions may be required.

6.1.3 Site Clearing

All previous structures, foundation elements, vegetation, and deleterious materials should be removed from areas to receive fill placement. The project geotechnical consultant should be notified at the appropriate times to provide observation services during clearing operations to verify compliance with the above recommendations. Voids created by clearing should be left open for observation by the geotechnical consultant. Any unusual soil conditions or subsurface structures encountered during site clearing and/or grading should be brought to the immediate attention of the project geotechnical consultant for corrective recommendations.

Deleterious debris located within the eastern 2/3 of the site should be removed. The amount of deleterious debris is anticipated to be less than 5%. A track mounted vehicle should be utilized to excavated the upper 2 to 3 feet of surficial soil to expose the deleterious debris. The exposed deleterious debris should then be removed and not placed within the fill. A thin layer of asphalt was observed within the artificial fill and may be considered for re-use provided it is crushed to less than

4 inches and thoroughly blended into the engineered fill. Concrete and brick fragments may also be incorporated provided they are crushed to less than 4 inches.

6.1.4 Site Preparation (Removals and Overexcavations)

In general, all artificial fill is considered unsuitable for support of proposed engineered fill and site improvements. These materials should be removed from proposed building pads, retaining walls and any other "structural" areas, and replaced as engineered compacted fill. The depth of removal is anticipated to range from approximately 2 to 7.5 feet below existing grades. The observed artificial fill depths within the borings and test pits are provided in the *Geotechnical Map*, Plate 1. In addition to general removal of unsuitable soils above, the existing soils should be over-excavated to a depth of at least 2 foot below the bottom of footings for the structures.

Locally deeper removals of fill are anticipated throughout the site. Based on the referenced Phase II report by Fulcrum Resources Environmental, two underground storage tanks (UST) were removed within the northern portion of the site. These USTs were excavated and removed before 1991. The excavations for the removal of the two USTs measured approximately 200 feet by 75 feet and 200 feet by 50 feet. In addition, a couple of more USTs were mapped along the west-central and south-central perimeters of the site. The excavation depth was not indicated within the report nor the details of backfill of the abandoned void. The actual depth of removal of the existing fill should be determined by the geotechnical consultant during grading.

Within the limits of pavement, retaining walls less than 3 feet in height and free-standing walls, a minimum 1 foot of engineered fill should be provided below the proposed footings.

The lateral extent of removals should extend at least 5 feet beyond the limits of the proposed structures or a distance equal to the depth of overexcavation below the bottom of the structural elements, whichever is greater.

Removals for pavement and free-standing retaining walls may be limited to the edge of the foundations or pavement where lateral restrictions to removals are present such as property lines. The actual depth of removals should be verified by the geotechnical consultant during site grading.

Where removals are limited by existing structures, protected trees or property lines, special considerations may be required in the construction of affected improvements. Under such conditions, specific recommendations should be provided by this firm.

All removal excavations should be evaluated by the geotechnical consultant during grading to confirm the exposed conditions are as anticipated and to provide supplemental recommendations if required.

Following removals/overexcavation, the exposed grade should first be scarified to a depth of 6 inches, brought to at least 120 percent of the optimum moisture content, and then compacted to at least 90 percent of the laboratory standard (ASTM D 1557).

6.1.5 Fill Placement

In general, materials excavated from the site may be reused as fill provided they are free of deleterious materials, metallic debris, and particles greater than 4 inches in maximum dimension (oversized materials). Asphaltic and concrete debris generated during site demolition can likely be reduced to no more than 4 inches in maximum dimension and incorporated within fill soils during earthwork operations. Such materials should be mixed thoroughly with fill soils to prevent nesting. All fill should be placed in lifts no greater than 8 inches in loose thickness, moisture conditioned to a uniform moisture of at least 120 percent of the optimum moisture content, then compacted in place to at least 90 percent of the laboratory standard. Each lift should be treated in a similar manner. Subsequent lifts should not be placed until the project geotechnical consultant has approved the preceding lift.

6.1.6 Import Materials

If import materials are required to achieve the proposed finish grades, the proposed import soils should have an Expansion Index (EI, ASTM D 4829) less than 90 and possess negligible soluble sulfate concentrations. Import sources should be indicated to the geotechnical consultant prior to hauling the materials to the site so that appropriate testing and evaluation of the fill materials can be performed in advance.

6.1.7 Temporary Excavations

Temporary construction slopes or trench excavations in site materials may be cut vertically up to a height of 4 feet provided that no surcharging of the excavations is present. Temporary slopes over 4 feet in height but no more than 10 feet in height should be laid back at a maximum gradient of 1:1 (H:V) or properly shored. If steeper cuts are required to avoid existing site improvements, then additional analyses by the geotechnical consultant will be required or the excavation should be shored.

Excavations should not be left open for prolonged periods of time. The project geotechnical consultant should observe all temporary cuts to confirm anticipated conditions and to provide alternate recommendations if conditions dictate. All excavations should conform to the requirements of CAL OSHA.

Where temporary excavations cannot accommodate a 1:1 layback or where surcharging occurs, shoring, slot cutting, underpinning, or other methods should be used. Consideration should be given to perform slot-cutting during rough grading along the east property due to the adjacent structure being separated from planned development approximately 5 feet. Specific recommendations for other options if considered should be provided by the geotechnical consultant based on review of the final design plans.

6.2 SEISMIC DESIGN PARAMETERS

For design of the project in accordance with Chapter 16 of the 2016 CBC, the following table presents the seismic design factors:

TABLE 6.1
2016 CBC Seismic Design Parameters

Parameter	Value
Site Class	D
Mapped MCE Spectral Response Acceleration, short periods, S_S	1.624

Mapped MCE Spectral Response Acceleration, at 1-sec. period, S_1	0.599
Site Coefficient, Fa	1.0
Site Coefficient, Fv	1.5
Adjusted MCE Spectral Response Acceleration, short periods, S_{MS}	1.624
Adjusted MCE Spectral Response Acceleration, at 1-sec. period, S_{M1}	0.899
Design Spectral Response Acceleration, short periods, S_{DS}	1.082
Design Spectral Response Acceleration, at 1-sec. period, S_{D1}	0.599

MCE = Maximum Considered Earthquake

6.3 FOUNDATION DESIGN

The following recommendations are provided for preliminary design purposes. These recommendations have been based on the site materials exposed during our investigation, our understanding of the proposed development, and the assumption that the recommendations presented herein are incorporated into the design and construction of the project. Final recommendations should be provided by the project geotechnical consultant following review of final foundation plans as well as observation and testing of site materials during grading. Depending upon the design plans and actual site conditions, the recommendations provided herein may require modification.

6.3.1 Soil Expansion

Expansion potential of existing site materials is expected to vary from **Low** to **High**. As such, we are providing recommendation for both conventional footings and post-tension foundation slabs. Additional testing of site soils should be performed by the project geotechnical consultant to confirm the basis of these recommendations during site grading.

6.3.2 Settlement

Foundations should be designed for total and differential static settlement up to 1 inch and ½-inch over 30 feet, respectively. For seismic considerations on the site, foundation systems should also be designed to prevent collapse or failure of structures due to the estimated liquefaction-induced total seismic settlement of up to 0.5 inch and differential settlement of 0.25 inch over 30 feet.

6.3.3 Allowable Bearing Value

Provided foundations are bearing into engineered fill, a bearing value of 2,500 pounds per square foot (psf) may be used for continuous and pad footings that have a minimum width of 12 inches and founded at a minimum depth of 12 inches below the lowest adjacent grade. This value may be increased by 150 psf and 450 psf for each additional foot in width and depth, respectively, up to a maximum value of 4,000 psf. Recommended allowable bearing values include both dead and live loads, and may be increased by one-third for wind and seismic forces.

6.3.4 Lateral Resistance

Provided site grading is performed and that foundations are founded in engineered fill, a passive earth pressure of 300 pounds per square foot per foot of depth (psf/ft) up to a maximum value of 1800 pounds per square foot (psf) may be used to determine lateral bearing for footings. This value may be increased by one-third when designing for wind and seismic forces. A coefficient of friction of 0.28

times the dead load forces may also be used between concrete and the supporting soils to determine lateral sliding resistance. No increase in the coefficient of friction should be used when designing for wind and seismic forces.

The above values are based on footings placed directly against engineered fill. In the case where footing sides are formed, all backfill against the footings should be compacted to at least 90 percent of the laboratory standard.

6.3.5 Post-Tensioned Slabs/Mat on Grade

Perimeter edge beams for the post-tensioned slabs should have a minimum effective width of 12 inches and be founded at a minimum depth of 18 inches below the lowest adjacent final ground surface. Interior beams may be founded at a minimum depth of 12 inches below the tops of the finish floor slabs. Where a post-tensioned mat is utilized, the exterior edge of the mat should be embedded at least 8 inches below the lowest adjacent grade. The thickness of the floor slab/mat should be determined by the project structural engineer; however, we recommend a minimum slab thickness of 5.0 inches.

Design of the mat may be based on a modulus of subgrade reaction (Kv1) of 95 pounds per cubic inch (pci). The modulus is based on an effective loading area of 1 foot by 1 foot. The modulus may be adjusted for other effective loading areas using the equation provided below.

$k_b(pci) = 95\left\{\frac{b+1}{2b}\right\}^2$ where "b" is the effective width of loading (minimum dimension) in feet.

Concrete floor slabs in areas to receive carpet, tile, or other moisture sensitive coverings should be underlain with a minimum of 10-mil moisture vapor retarder conforming to ASTM E 1745, Class A. The membrane should be properly lapped, sealed, and underlain within a layer of sand at least 4 inches thick. One inch of sand may be placed over the membrane to aid in the curing of the concrete. The sand should have a SE no less than 30. This vapor retarder system is anticipated to be suitable for most flooring finishes that can accommodate some vapor emissions. However, this system may emit more than 4 pounds of water per 1000 sq. ft. and therefore, may not be suitable for all flooring finishes. Additional steps should be taken if such vapor emission levels are too high for anticipated flooring finishes. Where a mat is utilized, the sand may be reduced to 1 inch provided the mat is at least 6 inches thick.

Prior to placing concrete, subgrade soils below slab-on-grade/mat areas should be thoroughly moistened to provide moisture contents that are at least 120 percent of the optimum moisture content to a depth of 12 inches.

Based on the guidelines provided in the "Design of Post-Tensioned Slabs-on-Ground" 3rd Edition by Post-Tensioning Institute, the em and ym values are summarized below:

TABLE 6.2
PTI Design Parameters

Parameter	Value
Edge Lift Moisture Variation Distance, e_m	3.4 feet
Edge Lift, y_m	2.838 inches
Center Lift Moisture Variation Distance, e_m	5.7 feet
Center Lift, y_m	2.164 inches

6.3.6 Foundation Observations

Foundation excavation should be observed by the project geotechnical consultant to verify that they have been excavated into competent bearing soils and to the minimum embedment recommended above. These observations should be performed prior to placement of forms or reinforcement. The excavations should be trimmed neat, level and square. Loose, sloughed or moisture-softened materials and debris should be removed prior to placing concrete.

6.4 RETAINING/SCREEN WALLS

6.4.1 General

The following preliminary design and construction recommendations are provided for general retaining and screen walls. Final wall designs specific to the site development should be provided for review once completed. The structural engineer and architect should provide appropriate recommendations for sealing at all joints and applying moisture-proofing material on the back of the walls.

6.4.2 Allowable Bearing Value and Lateral Resistance

Retaining walls may be supported by conventional spread footings that utilize the bearing capacities and lateral resistance values provided in Sections 6.3.3 and 6.3.4. The passive pressure used for lateral bearing should be reduced by 50% for walls that have a descending slope below the face of the wall or for walls along property lines.

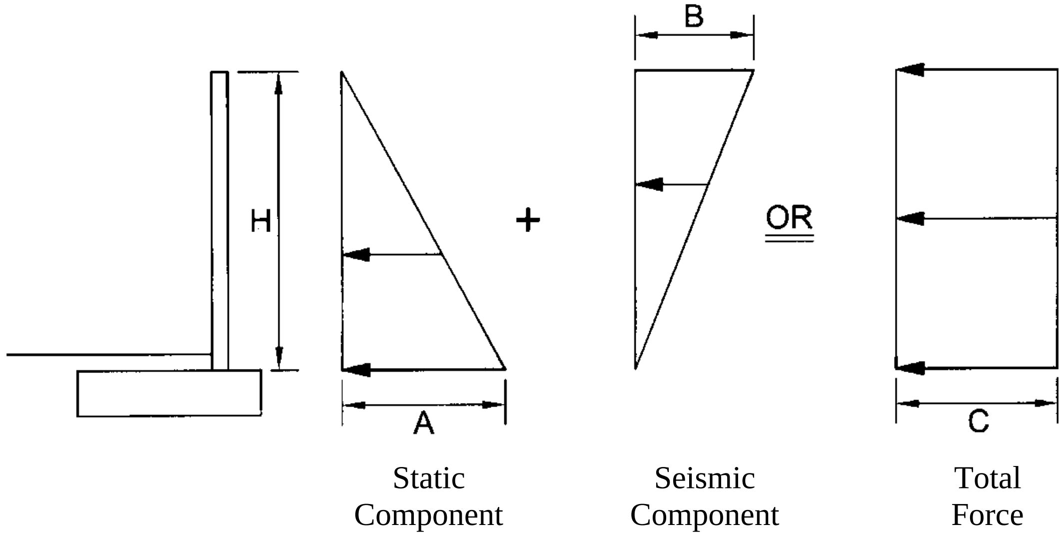
The above values are based on footings placed directly against properly compacted fill or competent native soils and embedded a minimum of 18 inches below the lowest adjacent grade. In the case where footing sides are formed, all backfill against the footings should be compacted to at least 90 percent of the Modified Proctor standard.

6.4.3 Active Earth Pressures

Static and seismic earth pressures for level and 2:1 (H:V) backfill conditions are provided in Table 6.3. Seismic earth pressures provided herein are based on the method provided by Seed & Whitman (1970) using a peak ground acceleration (PGA) of 0.39g for 10% probability of exceedance in 50 years. As indicated in the 2016 CBC, retaining walls supporting 6 feet of backfill or less are not required to be designed for seismic earth pressures. Two sets of values are provided in the following table; one for select import with Expansion Index (EI) less than 20, and one for onsite materials with an expansion index between 20 and 60. The backfill material should be placed within a 1:1 plane projected up from the base of the wall stem. In addition, the values are based on drained backfill conditions and do not consider hydrostatic pressure. Furthermore, retaining walls should be designed to support adjacent surcharge loads imposed by other nearby footings or traffic loads in addition to the earth pressure.

TABLE 6.3

SEISMIC EARTH PRESSURES
Pressure Diagram



Active Pressure Values
Walls Using Select Import Backfill
(Soils with EI <20 & <30% passing 200 sieve)

Value	Level Backfill	2:1 Backfill
A	35H	60H
B	12H	12H
C	23.5H	36H

Active Pressure Values
Walls Using Select Onsite Soil Backfill
(Soils with 20≤EI <60)

Value	Backfill Condition	
	Level	**2H:1V Slope**
A	50H	85H
B	12H	12H
C	31H	48.5H

Note: H is in feet and resulting pressure is in psf. Design may utilize either the sum of the static component and the seismic component force diagrams or the total force diagram above. SEAOSC has suggested using a load factor of 1.7 for the static component and 1.0 for the seismic component. The actual load factors should be determined by the structural engineer.

6.4.4 Footing Reinforcement

All continuous footings should be reinforced with a minimum of two No. 4 bars on top and two No. 4 bars on the bottom. The structural engineer may require different reinforcement and should dictate if greater than the recommendations provided herein. Where recommended removals are limited due to space restrictions, greater reinforcement may be recommended. Specific recommendations should be provided by the geotechnical consultant during grading based on as-built conditions exposed in the field.

6.4.5 Footing Observations

Footing excavations should be observed by the project geotechnical consultant to verify that they have been excavated into competent bearing soils and to the minimum embedment recommended herein. These observations should be performed prior to placement of forms or reinforcement. The excavations should be trimmed neat, level, and square. Loose, sloughed or moisture-softened materials and debris should be removed prior to placing concrete.

6.4.6 Drainage and Moisture-Proofing

Retaining walls should be constructed with a perforated pipe and gravel subdrain to prevent entrapment of water in the backfill. The perforated pipe should consist of 4-inch-diameter, ABS SDR-35 or PVC Schedule 40 with the perforations laid down. The pipe should be embedded in ¾- to 1½-inch open-graded gravel wrapped in filter fabric. The gravel should be at least one foot wide and extend at least one foot up the wall above the footing and drainage outlet. Drainage gravel and piping should not be placed below outlets and weepholes. Filter fabric should consist of Mirafi 140N, or equal. Outlet pipes should be directed to positive drainage devices.

The use of weepholes may be considered in locations where aesthetic issues from potential nuisance water are not a concern. Weepholes should be 2 inches in diameter and provided at least every 6 feet on center. Where weepholes are used, perforated pipe may be omitted from the gravel subdrain.

Retaining walls supporting backfill should also be coated with a moisture-proofing compound or covered with such material to inhibit infiltration of moisture through the walls. Moisture-proofing material should cover any portion of the back of wall that will be in contact with soil and should lap over and onto the top of footing. A drainage panel should be provided between the soil backfill and water proofing. The panel should extend from the top of the backdrain gravel up to within 12 inches of finish grade. The top of footing should be finished smooth with a trowel to inhibit the infiltration of water through the wall. The project structural engineer should provide specific recommendations for moisture-proofing, water stops, and joint details.

If select backfill soil is used, the backfill should be placed within the zone defined by a 1:1 plane projected up from the back of the footing. Active pressures may be used for walls free to move at the top. For walls restrained from movement at the time of backfilling, at-rest pressures should be used.

6.4.7 Retaining Wall Backfill

Some of the onsite soils are generally not suitable for use as backfill of retaining walls. Select onsite soils having expansion index (EI) in the range of $20 \leq EI < 60$ or select imported soils having $EI < 20$ may be used for backfill behind retaining walls provided the wall has been designed for earth pressures as discussed in Section 6.4.3. The project geotechnical consultant should approve the backfill used for retaining walls. Wall backfill should be thoroughly moistened to provide moisture contents slightly over optimum moisture content; placed in lifts no greater than 12 inches in thickness, and then mechanically compacted with appropriate equipment to at least 90 percent of the laboratory standard. Hand-operated compaction equipment should be used to compact the backfill placed immediately adjacent the wall to avoid damage to the wall. Flooding or jetting of backfill material is not recommended.

6.4.8 Wall Jointing

All site walls should be provided with cold joints through the masonry block section at horizontal spacing generally not exceeding 20 feet. If walls will be constructed in locations where removal of unsuitable soils was restricted to less than a 1 to 1 projection down from the foundation (such as property boundaries) the joints should be provided every 10 feet or other mitigation as recommended by the project geotechnical consultant. Joints should not extend through the footing nor should they be covered by a brittle finish such as stucco. Joints may be filled with a mastic caulking or covered by a facing strip attached to one side of the wall at the joint.

6.5 EXTERIOR FLATWORK

Exterior flatwork should be a minimum 4 inches thick. Cold joints or saw cuts should be provided at least every 5 feet in each direction. Flatwork more than 5 feet in width across the minimum dimension should be reinforced with 4” by 4”, W4 by W4 welded wire mesh or No 4 bars spaced 18 inches center to center in both directions. Cold joints should be keyed or provided with dowels spaced 24 inches on center. Flatwork that meets the structure at points of entry should be doweled into the footing or grade beam of the structure. Consideration should also be given to doweling flatwork into curbs where they meet. Special jointing detail should be provided in areas of block-outs, notches, or other irregularities to avoid cracking at points of high stress. Subgrade soils below flatwork should be thoroughly moistened to a moisture content of at least 120 percent of optimum to a depth of 12 inches. Moistening should be accomplished by lightly spraying the area over a period of a few days just prior to pouring concrete.

Drainage from flatwork areas should be directed to local area drains and/or other appropriate collection devices designed to carry runoff water to the street or other approved drainage structures. The concrete flatwork should also be sloped at a minimum gradient of 2% away from building foundations and masonry walls.

The geotechnical consultant should observe and verify the density and moisture content of subgrade soils prior to pouring concrete to verify the recommended pre-moistening recommendations have been met.

6.6 CONCRETE MIX DESIGN

Laboratory testing of onsite soil indicates **negligible** soluble sulfate content. Concrete designed to follow the procedures provided in ACI 318, Section 4.3, Table 4.3.1 for negligible sulfate exposure are anticipated to be adequate for mitigation of sulfate attack on concrete. Upon completion of rough grading, an evaluation of as-graded conditions and further laboratory testing will be required for the site to confirm or modify the conclusions provided in this section.

6.7 CORROSION

Results of preliminary testing of soils for pH, chloride content, and minimum resistivity indicate the site is potentially **Severely Corrosive** to metals that are in contact or close proximity to onsite soils. As such, structures fabricated from metals should have appropriate corrosion protection if they will be in direct contact with site soils. Under such conditions, a corrosion specialist should provide specific recommendations.

6.8 PRELIMINARY PAVEMENT DESIGN

6.8.1 Pavement Structural Sections

Based on the soil conditions present at the site and estimated traffic index, preliminary pavement structural sections are recommended in Table 6.4 below. Soil conditions vary significantly with respect to R-value. An assumed “R-value” of 5 was used for this preliminary pavement design to represent the typical condition we anticipate to be present following site grading. The sections provided below are for planning purposes only and should be re-evaluated subsequent to site grading. Final pavement sections should be based on actual R-value testing of in-place soils and analysis of anticipated traffic.

TABLE 6.4
PRELIMINARY PAVEMENT STRUCTURAL SECTIONS

Location	Traffic Index	Asphaltic Concrete (inches)	Concrete Pavers (mm)	Portland Cement Concrete (inches)	Aggregate Base (inches)
Entry Way and Main Driveway	6.0	4.0	--	--	13.0
		--	80	--	16.0
		--	--	9.0	--
Alley Ways	5.0	4.0	--	--	8.0
		--	80	--	12.0
		--	--	7.5	--

		4.0	--	--	12.0
Commercial Drive Aisles	6.0		80	--	16.0
		--	--	9.0	--
Parking Stalls	N/A	4.0	--	--	6.0

6.8.1 Subgrade Preparation

Prior to placement of paving elements, subgrade soils should be scarified 6 inches, moisture-conditioned to at least 120 percent of the optimum moisture content then compacted to at least 90 percent of the maximum dry density determined in accordance with ASTM D1557. Areas observed to pump or yield under vehicle traffic should be removed and replaced with firm and unyielding engineered compacted soil or aggregate base materials.

6.8.2 Aggregate Base

Aggregate base should be moisture conditioned to slightly over the optimum moisture content, placed in lifts no greater than 6 inches in thickness, then compacted to at least 95 percent of the laboratory standard (ASTM D 1557). Aggregate base materials should be Class 2 Aggregate Base conforming to Section 26-1 of the latest edition of the Caltrans Standard Specifications, Crushed Aggregate Base conforming to Section 200-2.2 of the latest edition of the Standard Specifications for Public Works Construction (Greenbook) or Crushed Miscellaneous Base conforming to Section 200-2.4 of the Greenbook.

6.8.3 Asphaltic Concrete

Aggregate base should be moisture conditioned to slightly over the optimum moisture content, placed in lifts no greater than 6 inches in thickness, then compacted to at least 95 percent of the laboratory standard (ASTM D 1557). Aggregate base materials should be Class 2 Aggregate Base conforming to Section 26-1 of the latest edition of the Caltrans Standard Specifications, Crushed Aggregate Base conforming to Section 200-2.2 of the latest edition of the Standard Specifications for Public Works Construction (Greenbook) or Crushed Miscellaneous Base conforming to Section 200-2.4 of the Greenbook.

6.8.4 Asphaltic Concrete

Aggregate base should be moisture conditioned to slightly over the optimum moisture content, placed in lifts no greater than 6 inches in thickness, then compacted to at least 95 percent of the laboratory standard (ASTM D 1557). Aggregate base materials should be Class 2 Aggregate Base conforming to Section 26-1 of the latest edition of the Caltrans Standard Specifications, Crushed Aggregate Base conforming to Section 200-2.2 of the latest edition of the Standard Specifications for Public Works Construction (Greenbook) or Crushed Miscellaneous Base conforming to Section 200-2.4 of the Greenbook.

6.8.5 Concrete Paver

Concrete pavers should conform to the requirements of ASTM C 936. Construction of the pavers, including bedding sand, should follow manufacturer's specifications. Typical thickness of bedding sand is about 1 inch. The gradation of bedding sand should meet the requirement in Table 6.5.

TABLE 6.5
Gradation for Sand Bedding

Sieve Size	Percent Passing
⅜"	100
No. 4	95 - 100
No. 8	80 - 100
No. 16	50 - 85
No. 30	25 - 60
No. 50	5 - 30
No. 100	0 - 10
No. 200	0 - 1

6.9 POST GRADING CONSIDERATIONS

6.9.1 Site Drainage and Irrigation

The ground immediately adjacent to foundations should be provided with positive drainage away from the structures in accordance with 2016 CBC, Section 1804.4. No rain or excess water should be allowed to pond against structures such as walls, foundations, flatwork, etc.

Excessive irrigation water can be detrimental to the performance of the proposed site development. Water applied in excess of the needs of vegetation will tend to percolate into the ground. Such percolation can lead to nuisance seepage and shallow perched groundwater. Seepage can form on slope faces, on the faces of retaining walls, in streets, or other low-lying areas. These conditions could lead to adverse effects such as the formation of stagnant water that breeds insects, distress or damage of trees, surface erosion, slope instability, discoloration and salt buildup on wall faces, and premature failure of pavement. Excessive watering can also lead to elevated vapor emissions within buildings that can damage flooring finishes or lead to mold growth inside the home.

Key factors that can help mitigate the potential for adverse effects of overwatering include the judicious use of water for irrigation, use of irrigation systems that are appropriate for the type of vegetation and geometric configuration of the planted area, the use of soil amendments to enhance moisture retention, use of low-water demand vegetation, regular use of appropriate fertilizers, and seasonal adjustments of irrigation systems to match the water requirements of vegetation. Specific recommendations should be provided by a landscape architect or other knowledgeable professional.

6.9.2 Utility Trenches

Trench excavations should be constructed in accordance with the recommendations contained in Section 6.1.7 of this report. Trench excavations must also conform to the requirements of Cal/OSHA.

Trench backfill materials and compaction criteria should conform to the requirements of the local municipalities. As a minimum, utility trench backfill should be compacted to at least 90 percent of the laboratory standard. Materials placed within the pipe zone (6 inches below and 12 inches above the pipe) should consist of particles no greater than ¾ inches and have a SE of at least 30. The materials within the pipe zone should be moisture-conditioned and compacted by hand-operated compaction equipment. Above the pipe zone (>1 foot above pipe), the backfill may consist of general fill materials. Trench backfill should be moisture-conditioned to slightly over the optimum moisture content, placed in lifts no greater than 12 inches in thickness, and then mechanically compacted with appropriate equipment to at least 90 percent of the laboratory standard. For trenches with sloped walls, backfill material should be placed in lifts no greater than 8 inches in loose thickness, and then compacted by rolling with a sheepsfoot roller or similar equipment. The project geotechnical consultant should perform density testing along with probing to verify that adequate compaction has been achieved.

Within shallow trenches (less than 18 inches deep) where pipes may be damaged by heavy compaction equipment, imported clean sand having a SE of 30 or greater may be utilized. The sand should be placed in the trench, thoroughly watered, and then compacted with a vibratory compactor. For utility trenches located below a 1:1 (H:V) plane projecting downward from the outside edge of the adjacent footing base or crossing footing trenches, concrete or slurry should be used as trench backfill.

6.10 PLAN REVIEW AND CONSTRUCTION SERVICES

We recommend ***Albus-Keefe & Associates, Inc.*** be engaged to review any future development plans, including foundation plans prior to construction. This is to verify that the assumptions of this report are valid and that the preliminary conclusions and recommendations contained in this report have been properly interpreted and are incorporated into the project plans and specifications. If we are not provided the opportunity to review these documents, we take no responsibility for misinterpretation of our preliminary conclusions and recommendations.

We recommend that a geotechnical consultant be retained to provide soil engineering services during construction of the project. These services are to observe compliance with the design, specifications or recommendations, and to allow design changes in the event that subsurface conditions differ from those anticipated prior to the start of construction.

If the project plans change significantly from the assumed development described herein, the project geotechnical consultant should review our preliminary design recommendations and their applicability to the revised construction. If conditions are encountered during construction that appear to be different than those indicated in this report or subsequent design reports, the project geotechnical consultant should be notified immediately. Design and construction revisions may be required.

7.0 LIMITATIONS

This report is based on the proposed development and geotechnical data as described herein. The materials encountered on the project site, described in other literature, and utilized in our laboratory testing for this investigation are believed representative of the total project area, and the conclusions and recommendations contained in this report are presented on that basis. However, soil and bedrock materials can vary in characteristics between points of exploration, both laterally and vertically, and those variations could affect the conclusions and recommendations contained herein. As such, observation and testing by a geotechnical consultant during the grading and construction phases of the project are essential to confirming the basis of this report.

This report has been prepared consistent with that level of care being provided by other professionals providing similar services at the same locale and time period. The contents of this report are professional opinions and as such, are not to be considered a guaranty or warranty.

This report should be reviewed and updated after a period of one year or if the site ownership or project concept changes from that described herein.

This report has been prepared for the exclusive use of **G3 Urban** and their project consultants in the planning and design of the proposed development. This report has not been prepared for use by parties or projects other than those named or described herein. This report may not contain sufficient information for other parties or other purposes.

This report is subject to review by the controlling governmental agency.

Respectfully submitted,

ALBUS-KEEFE & ASSOCIATES, INC

Mark Principe
Staff Engineer

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Plans

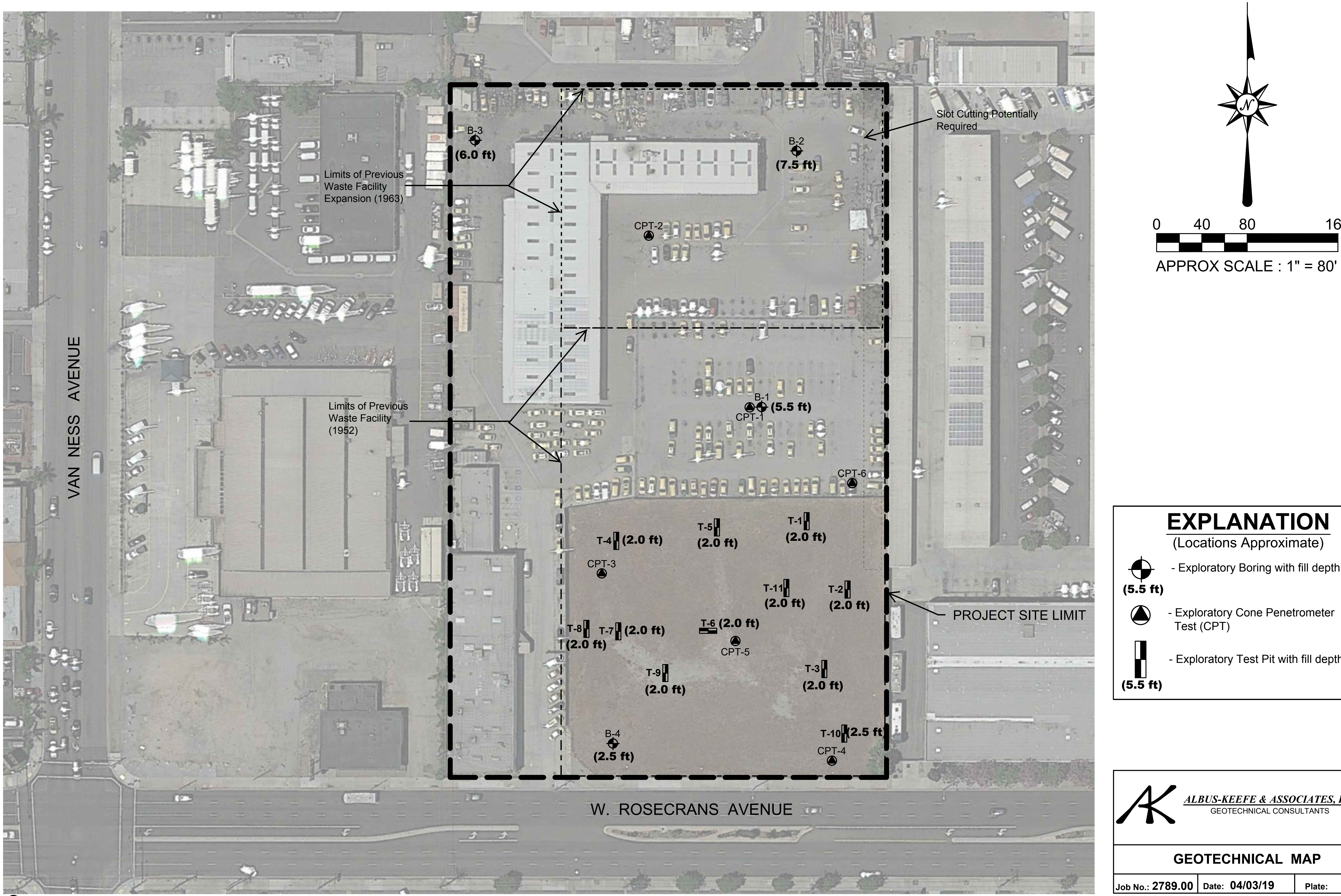
Conceptual Site Plan, Rosecrans & Van Ness, prepared by Angeleno Associates, Inc., dated March 5, 2019

Reports

Phase II Subsurface Investigation, 2101 & 2129 West Rosecrans Avenue, Gardena, California 90249, prepared for John Von Helms, by Fulcrum Resources Environmental, Project number: 201710-3988, dated December 11, 2017

Maps

Geophysical Interpretation Map Draft, Former Salvage Yard, 2129 Rosecrans Avenue, Gardena, California, prepared by Spectrum Geophysics, Figure 1, scale 1 inch = 30 feet, dated September 7, 2018, Project No. 1808031R



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EXPLANATION
(Locations Approximate)

(5.5 ft) - Exploratory Boring with fill depth

- Exploratory Cone Penetrometer Test (CPT)

(5.5 ft) - Exploratory Test Pit with fill depth

ALBUS-KEEFE & ASSOCIATES, INC.
GEOTECHNICAL CONSULTANTS

GEOTECHNICAL MAP

Job No.: 2789.00	Date: 04/03/19	Plate: 1

APPENDIX A

EXPLORATION BORING LOGS

E X P L O R A T I O N   L O G

Project:		Location:
Address:		Elevation:
Job Number:	Client:	Date:
Drill Method:	Driving Weight:	Logged By:

Depth (feet)	Lith-ology	Material Description	Water	Samples: Blows Per Foot	Samples: Core	Samples: Bulk	Laboratory Tests: Moisture Content (%)	Laboratory Tests: Dry Density (pcf)	Laboratory Tests: Other Lab Tests
5									
10									
15									
20									

EXPLANATION

Solid lines separate geologic units and/or material types.

Dashed lines indicate unknown depth of geologic unit change or material type change.

Solid black rectangle in Core column represents California Split Spoon sampler (2.5in ID, 3in OD).

Double triangle in core column represents SPT sampler.

Vertical Lines in core column represents Shelby sampler.

Solid black rectangle in Bulk column respresents large bag sample.

Other Laboratory Tests:

Max = Maximum Dry Density/Optimum Moisture Content
EI = Expansion Index
SO4 = Soluble Sulfate Content
DSR = Direct Shear, Remolded
DS = Direct Shear, Undisturbed
SA = Sieve Analysis (1" through #200 sieve)
Hydro = Particle Size Analysis (SA with Hydrometer)
200 = Percent Passing #200 Sieve
Consol = Consolidation
SE = Sand Equivalent
Rval = R-Value
ATT = Atterberg Limits

Albus-Keefe & Associates, Inc.

EXPLORATION LOG

Project:		Location: B-1
Address: 2129 Rosecrans Ave, Gardena, CA 90249		Elevation: 50.9
Job Number: 2789.00	Client: G3 Urban	Date: 1/22/2019
Drill Method: Hollow-Stem Auger	Driving Weight: 140 lbs / 30 in	Logged By: PK

Depth (feet)	Material Description	Water	Samples: Blows Per Foot	Laboratory Tests: Moisture Content (%)	Laboratory Tests: Dry Density (pcf)	Laboratory Tests: Other Lab Tests
0	<u>Asphalt Concrete (AC):</u> 4 inches					SO4 ATT EI
	<u>Crushed Aggregate Base (CAB):</u> 5 inches					
	ARTIFICIAL FILL (Af) <u>Silty Sand (SM):</u> Brown, moist, medium dense, medium to coarse grained sand		17	15.6	110.7	
	<u>Clayey Sand (SC):</u> Mottled: brown, light brown, and reddish brown, damp to moist, medium dense, presence of trash		35	23.9	102.8	
5	<u>Lean Clay (CL):</u> Gray, damp to moist, stiff, presence of wire		43	13.3	119.5	Consol
	OLDER ALLUVIUM (Qoal) <u>Clayey Sand (SC):</u> brown and gray mottling, damp to moist, dense @ 6 ft, medium to coarse grained sand					
10	@ 10 ft, Brown with white specs, moist, medium dense, coarse grained sand, scattered gravel		39	8.1	115.5	SA Hydro
	<u>Sandy Lean Clay (CL):</u> Brown with scattered gray mottling, damp to moist, very stiff, with silt					
15			19			SA Hydro ATT
	@ 18 ft, brown, olive brown, dark gray brown, hard					
20			27			ATT
	@ 23 ft, brown with dark brown mottling, moist, very stiff, micaceous					

Albus-Keefe & Associates, Inc.

Plate A-2

EXPLORATION LOG

Project:		Location: B-1
Address: 2129 Rosecrans Ave, Gardena, CA 90249		Elevation: 50.9
Job Number: 2789.00	Client: G3 Urban	Date: 1/22/2019
Drill Method: Hollow-Stem Auger	Driving Weight: 140 lbs / 30 in	Logged By: PK

Depth (feet)	Lith-ology	Material Description	Water	Samples: Blows Per Foot	Core	Bulk	Laboratory Tests: Moisture Content (%)	Dry Density (pcf)	Other Lab Tests
				10					200 ATT
		Silty Sand (SM): Brown, wet, dense, medium grained sand							
30				20					
		Sandy Lean Clay (CL): Brown, moist, hard, fine to medium grained sand							200 ATT
		Silty Sand (SM): Brown, wet, dense, fine to medium grained sand, trace coarse sand							
35				20					
		Sandy Silt (ML): Brown with orange brown staining, moist, hard, trace clay							
		Silty Sand (SM): Brown, wet, dense, trace clay binder							
40				23					
		@ 41 ft, brown with gray, moist, with clay							
45		@ 45 ft, brown, wet, medium dense, presence of clay binder		17					200
		@ 45.5 ft, moist, with clay							

EXPLORATION LOG

Project:		Location: B-1
Address: 2129 Rosecrans Ave, Gardena, CA 90249		Elevation: 50.9
Job Number: 2789.00	Client: G3 Urban	Date: 1/22/2019
Drill Method: Hollow-Stem Auger	Driving Weight: 140 lbs / 30 in	Logged By: PK

Depth (feet)	Lith-ology	Material Description	Water	Samples: Blows Per Foot	Samples: Core	Samples: Bulk	Laboratory Tests: Moisture Content (%)	Laboratory Tests: Dry Density (pcf)	Laboratory Tests: Other Lab Tests
		@ 50 ft, wet, dense, some coarse grained sand		20					
		Silt (ML): Brown and gray, damp to moist, hard, with clay							
		Clay (CL): Brown with orange brown staining, damp to moist, hard							
		End of boring at 51.5 feet. Groundwater encountered at 25 feet. Backfilled with soil cuttings.							

EXPLORATION LOG

Project:		Location: B-2
Address: 2129 Rosecrans Ave, Gardena, CA 90249		Elevation: 51.6
Job Number: 2789.00	Client: G3 Urban	Date: 1/22/2019
Drill Method: Hollow-Stem Auger	Driving Weight: 140 lbs / 30 in	Logged By: PK

Depth (feet)	Material Description	Water	Blows Per Foot	Moisture Content (%)	Dry Density (pcf)	Other Lab Tests
	ARTIFICIAL FILL (Af) Silty Sand (SM): Mottled brown and gray, damp to moist, medium dense					
	Sandy Clay (CL): Black, damp, very stiff, scattered gravel, asphalt fragments		26	16.4	112.7	
	Clay (CL): Gray to black, damp to moist, stiff		20	24.5	99	
5						
	@ 6 ft, very stiff		27	21.3	102.8	Consol
	OLDER ALLUVIUM (Qoal) Clayey Sand (SC): Mottled : brown, light brown, and dark orange brown, damp to moist, dense					
10	@ 10 ft, Brown, medium to coarse grained sand, scattered gravel		56	9.9	125.5	
	Silty Sand (SM): Brown to orange brown, damp to moist, medium dense, fine grained sand					
15			18			
	Clay (CL): Grayish brown, damp, very stiff					
20			17			
	End of boring at 21.5 feet. No groundwater encountered. Backfilled with soil cuttings.					

EXPLORATION LOG

Project:		Location: B-3
Address: 2129 Rosecrans Ave, Gardena, CA 90249		Elevation: 51.4
Job Number: 2789.00	Client: G3 Urban	Date: 1/22/2019
Drill Method: Hollow-Stem Auger	Driving Weight: 140 lbs / 30 in	Logged By: PK

Depth (feet)	Lith-ology	Material Description	Water	Samples: Blows Per Foot	Samples: Core	Samples: Bulk	Laboratory Tests: Moisture Content (%)	Laboratory Tests: Dry Density (pcf)	Laboratory Tests: Other Lab Tests
		<u>Asphalt (SC):</u> 3 inches							
		<u>Crushed Aggregate Base (CAB):</u> 5 inches							
		ARTIFICIAL FILL (Af) <u>Clay (CL):</u> Gray and greenish gray, damp to moist, very stiff, with silt		26			16	111.7	
		@ 4 ft, gray, damp, stiff		16			20.4	103	
5									
		OLDER ALLUVIUM (Qoal) <u>Clayey Sand (SC):</u> Brown to orange brown with grayish brown mottling, damp to moist, dense, medium to coarse grained sand, scattered gravel		58			11.3	124.1	
10		@ 10 ft, medium dense		39			11.8	122	
		<u>Silty Sand (SM):</u> Brown, damp to moist, medium dense, clay binder							
15		@ 15.5 ft, fine grained sand, no clay binder		20					
		<u>Clay (CL):</u> Grayish brown, damp, very stiff							
20		@ 20 ft, hard		29					
		<u>Clayey Sand (SC):</u> Brown mottled with: dark brown, reddish brown, and orange brown, damp to moist, dense							
		End of boring at 21.5 feet. No groundwater encountered. Backfilled with soil cuttings.							

EXPLORATION LOG

Project:	Location: B-4
Address: 2129 Rosecrans Ave, Gardena, CA 90249	Elevation: 49.1
Job Number: 2789.00 / Client: G3 Urban	Date: 1/22/2019
Drill Method: Hollow-Stem Auger / Driving Weight: 140 lbs / 30 in	Logged By: PK

Depth (feet)	Lith-ology	Material Description	Water	Samples: Blows Per Foot	Samples: Core	Samples: Bulk	Laboratory Tests: Moisture Content (%)	Laboratory Tests: Dry Density (pcf)	Laboratory Tests: Other Lab Tests
		ARTIFICIAL FILL (Af) Sandy Fat Clay (CH): Orange brown and dark brown, damp to moist, hard, coarse grained sand							SO4 DS ATT pH Resist Ch EI Max
		OLDER ALLUVIUM (Qoal) Clayey Sand (SC): Orange brown to brown mottled with dark brown, damp, dense		60			15.8	112.9	
5				50/ 7"			13.2	115.7	
		@ 6 ft, very dense		84			9.2	122	
10		@ 10 ft, dense		59			20.3	110.5	
		Clay (CL): Grayish brown, damp to moist, very stiff							
15				20					
20		@ 20 ft, brown		18					

EXPLORATION LOG

Project:		Location: B-4
Address: 2129 Rosecrans Ave, Gardena, CA 90249		Elevation: 49.1
Job Number: 2789.00	Client: G3 Urban	Date: 1/22/2019
Drill Method: Hollow-Stem Auger	Driving Weight: 140 lbs / 30 in	Logged By: PK

Depth (feet)	Lith-ology	Material Description	Water	Samples: Blows Per Foot	Samples: Core	Samples: Bulk	Laboratory Tests: Moisture Content (%)	Laboratory Tests: Dry Density (pcf)	Laboratory Tests: Other Lab Tests
		Silty Sand (SM): Brown to grayish brown, moist, dense, fine to medium grained sand, wet at top of the sample							
				28					
		Clay (CL): Brown, damp to moist, very stiff							
30				14					
		Silty Sand (SM): Brown to orange brown, moist to very moist, medium dense, trace clay binder							
		End of boring at 31.5 feet. Groundwater at 23.6 feet. Backfilled with soil cuttings							

Albus-Keefe & Associates, Inc.

Plate A-8

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

CPT-1

Project: **Albus-Keefe**
Location: **2129 Rosecrans Avenue, Gardena, CA**

Total depth: 50.54 ft, Date: 1/22/2019
Cone Type: Vertek

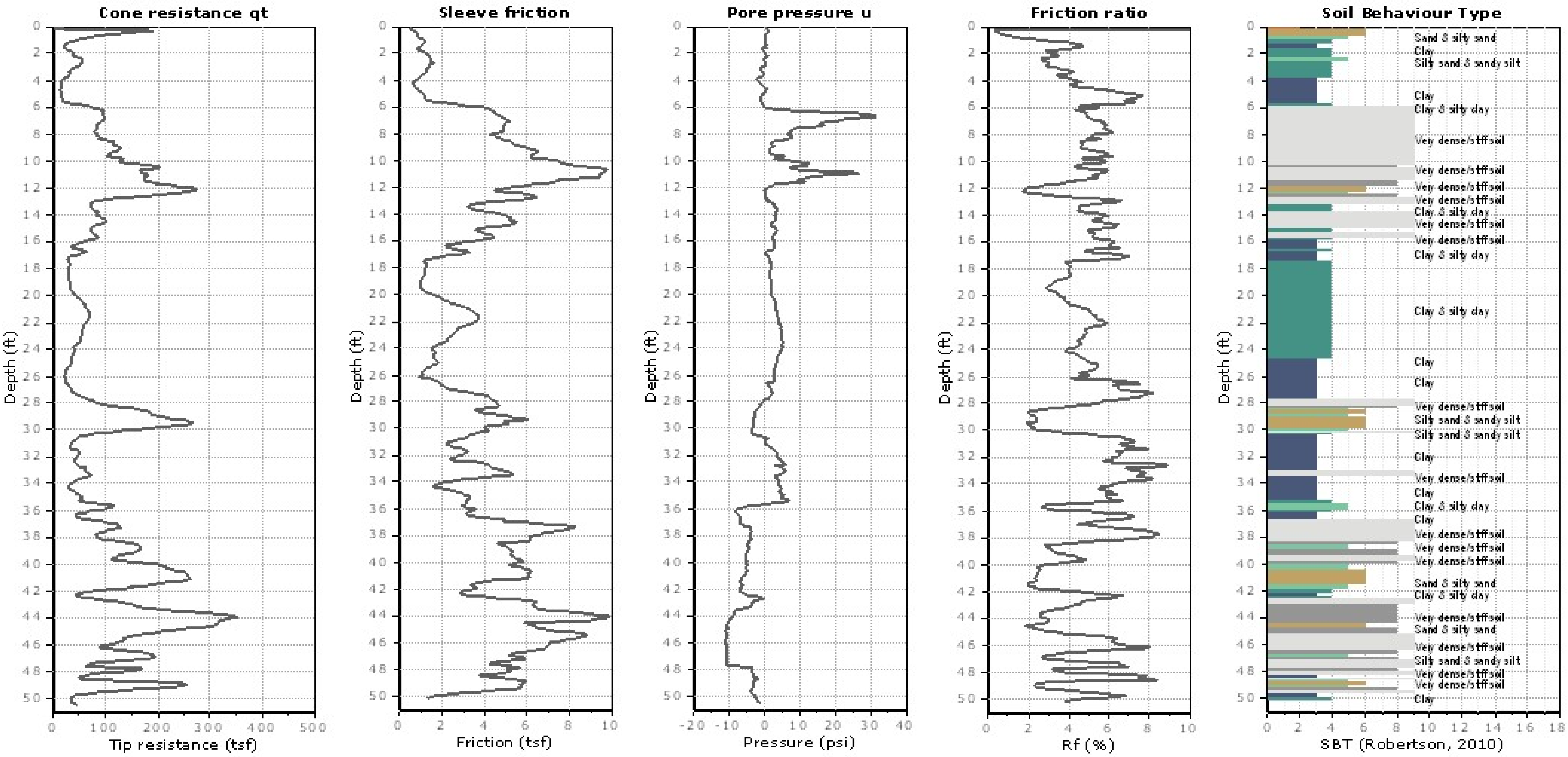


Plate 9

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

CPT-2

Project: **Albus-Keefe**
Location: **2129 Rosecrans Avenue, Gardena, CA**

Total depth: 40.43 ft, Date: 1/22/2019
Cone Type: Vertek

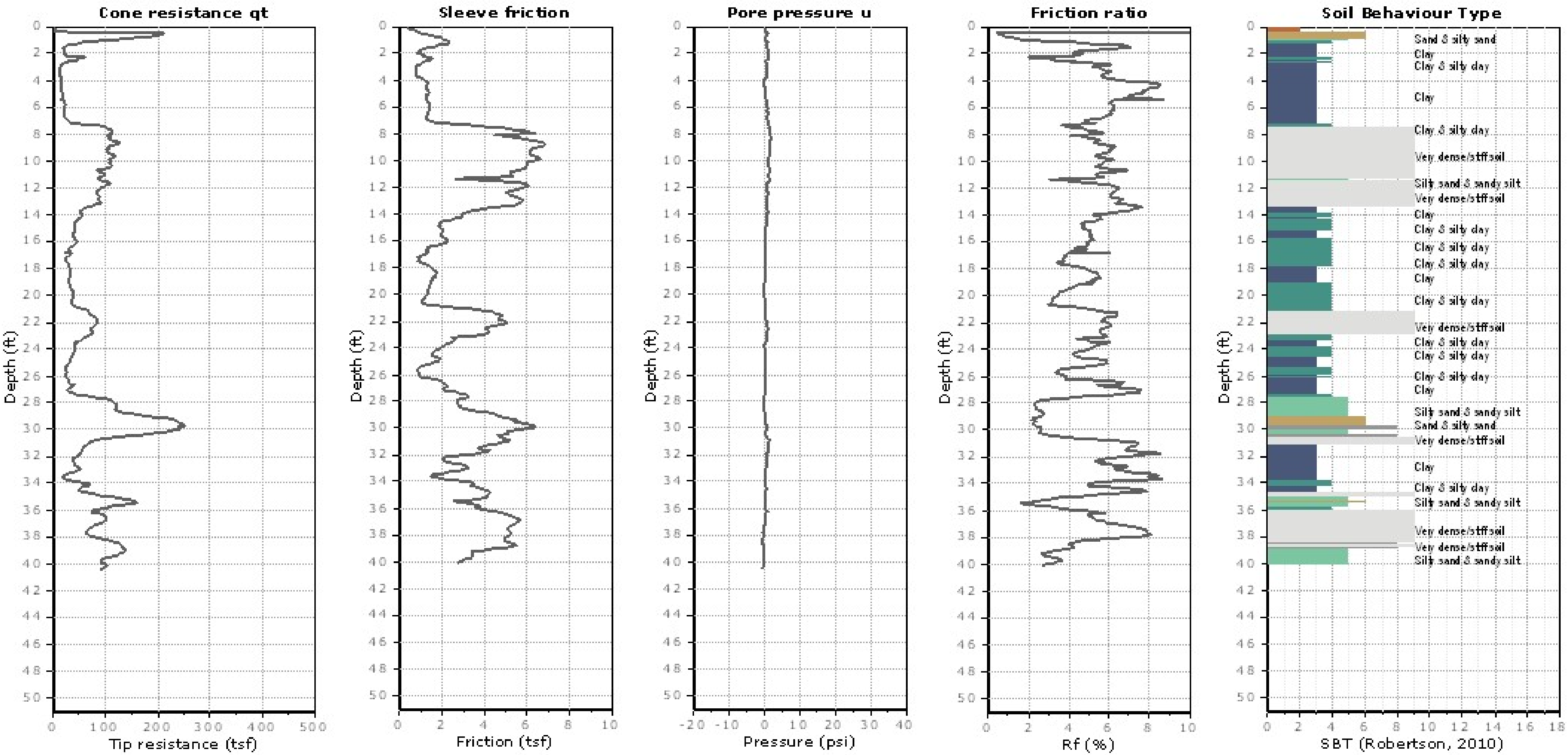


Plate A-10

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

CPT-3

Project: **Albus-Keefe**
Location: **2129 Rosecrans Avenue, Gardena, CA**

Total depth: 40.18 ft, Date: 1/22/2019
Cone Type: Vertek

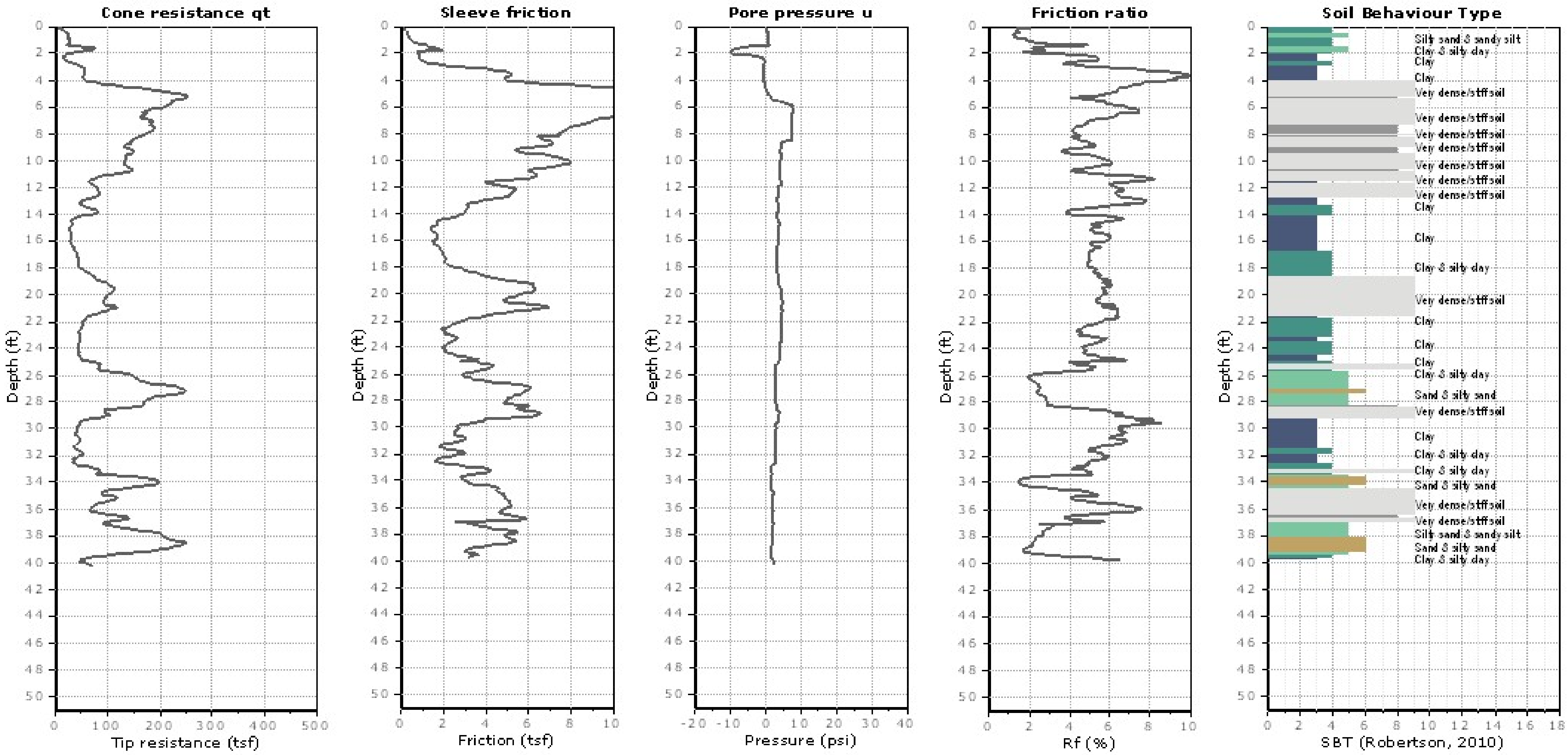


Plate A-11

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

Project: **Albus-Keefe**

Location: **2129 Rosecrans Avenue, Gardena, CA**

CPT-4

Total depth: 43.51 ft, Date: 1/22/2019

Cone Type: Vertek

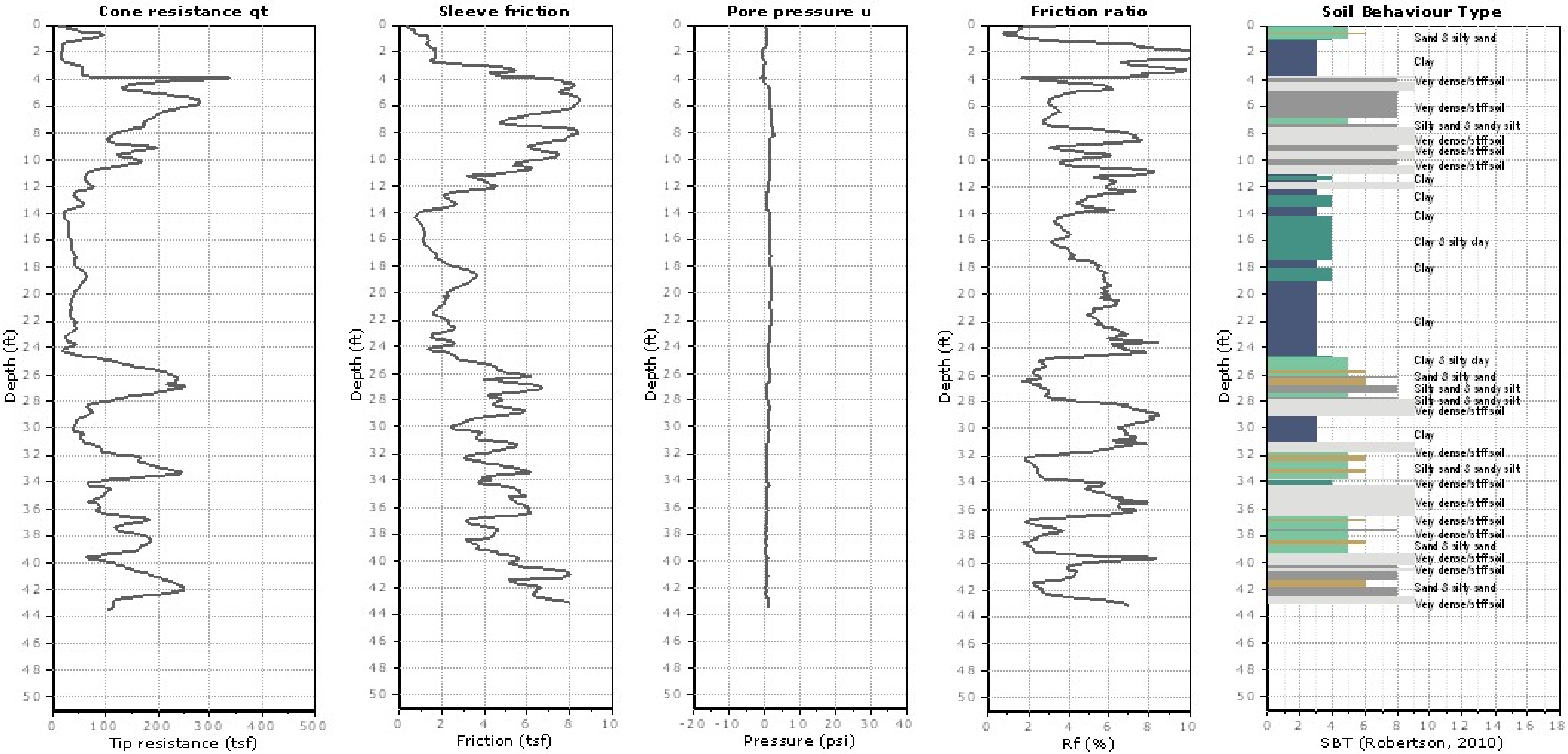


Plate A-12

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

CPT-5

Project: Albus-Keefe

Location: 2129 Rosecrans Avenue, Gardena, CA

Total depth: 30.45 ft, Date: 1/22/2019

Cone Type: Vertek

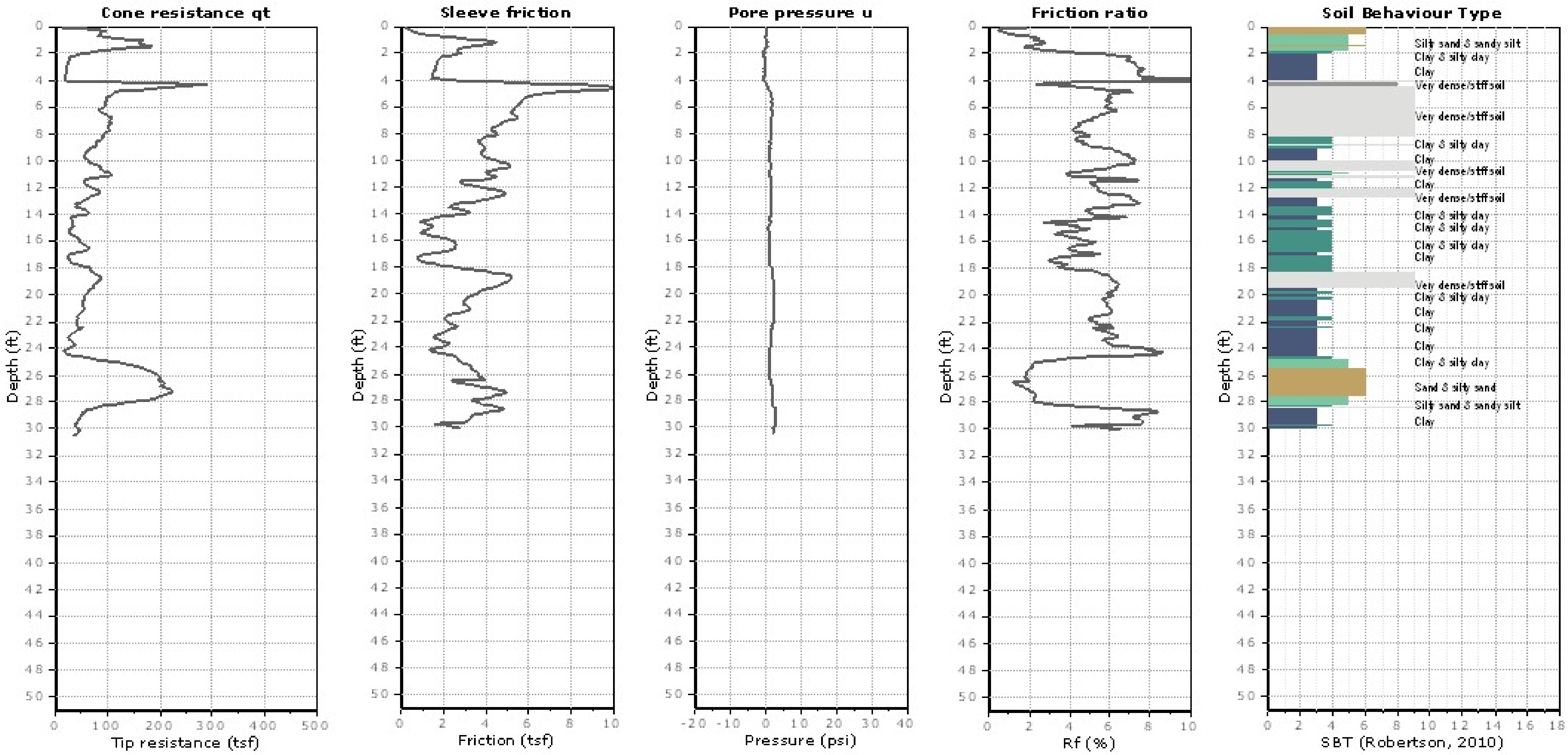


Plate A-13

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

CPT-6

Project: **Albus-Keefe**
Location: **2129 Rosecrans Avenue, Gardena, CA**

Total depth: 30.34 ft, Date: 1/22/2019
Cone Type: Vertek

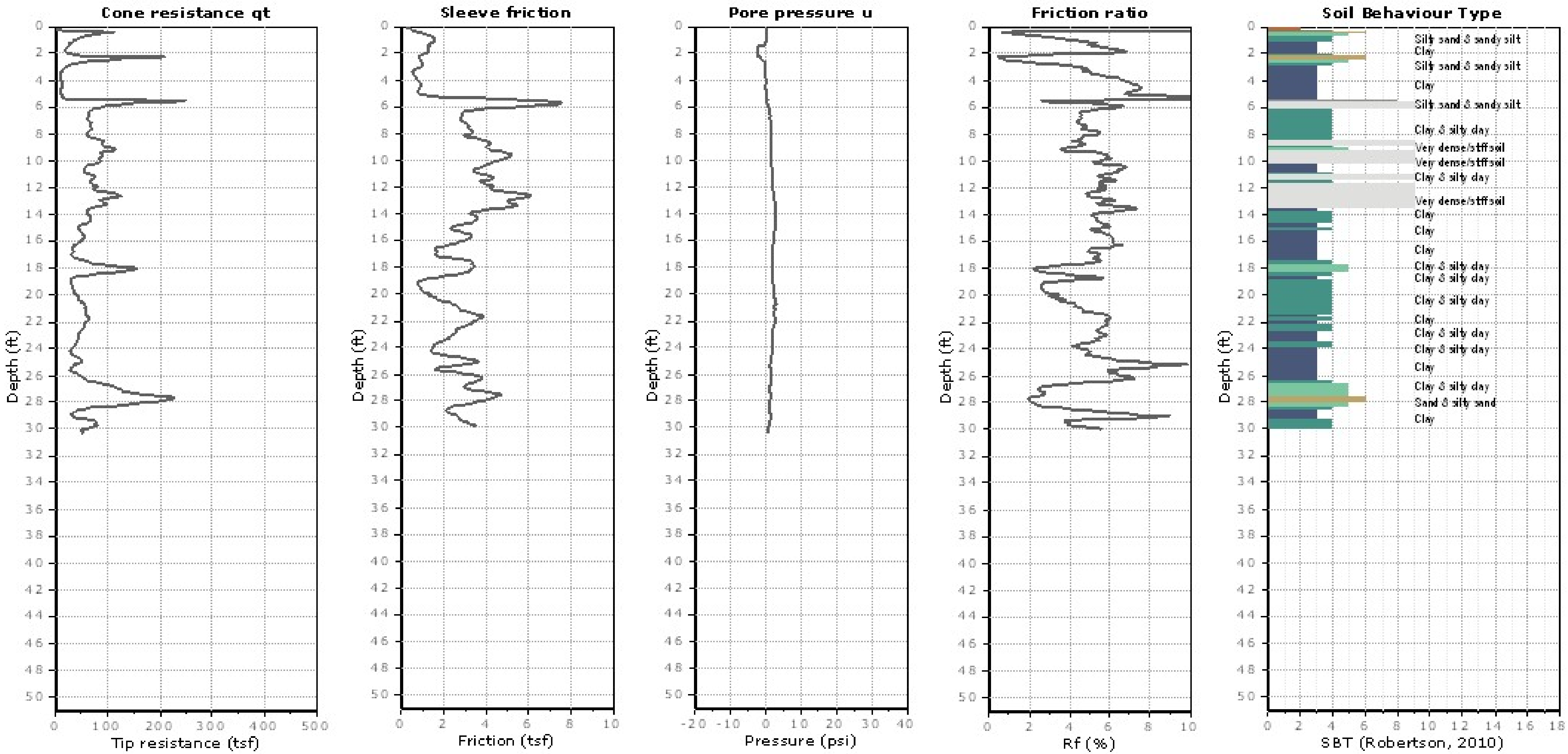


Plate A-14

DESCRIPTIVE LOG OF TRENCH EXCAVATION

Trench Number	Depth (Feet)	U.S.C.S. Symbol	Field Description
			Artificial Fill (AF):
T-1	0.0-2.0	CL	Clay: Dark bluish gray mottled with brown, moist, stiff, stiff, trace fine sand, metal debris present, trace fine gravel, rootlets present
			Older Alluvium (Qoal):
	2.0-4.0	CL	Clay: Dark bluish gray, moist, stiff
	4.0-5.0	CL	Sandy Clay: Tan mottled with gray, dry, stiff, trace fine sand, trace medium sand, orangish brown iron oxide staining present, porous, rootlets present
	5.0-7.0	SC	Clayey Sand: Grayish brown with light orangish brown oxidation staining, dry to damp, medium dense, fine sand, trace medium sand, porous, rootlets present
			Total Depth: 7.0 feet No Caving No Groundwater Backfill not compacted
			Artificial Fill:
T-2	0.0-2.0	CL	Clay: Dark bluish gray mottled with brown, moist, stiff, trace medium to coarse sand, metal debris present, trace fine gravel, rootlets present
			Older Alluvium (Qoal):
	2.0-4.0	CL	Clay: Dark bluish gray, moist, very stiff, trace minor orange oxidation staining
	4.0-5.0	SC/CL	Clayey Sand / Sandy Clay: Light orangish brown mottled with grayish brown, dry to damp, very stiff / dense, fine sand, some medium sand, reddish brown oxidation staining, very porous
			Total Depth: 5.0 feet No Caving No Groundwater Backfill not compacted

DESCRIPTIVE LOG OF TRENCH EXCAVATION

Trench Number	Depth (Feet)	U.S.C.S. Symbol	Field Description
			Artificial Fill (Af):
T-3	0.0-2.0	CL	Clay: Dark bluish gray mottled with brown, moist, stiff, trace fine sand, trace metal debris
			Older Alluvium (Qoal):
	2.0-4.5	CL	Clay: Dark bluish gray, moist, very stiff, trace minor orange oxidation staining
	4.5-6.0	SC/CL	Clayey Sand / Sandy Clay: Reddish brown mottled with grayish brown, damp, very stiff/dense, fine sand, some medium sand, very porous, rootlets present @5.5': brownish gray with slight orange oxidation staining, reduced pores
			Total Depth: 6.0 feet No Caving No Ground Water Backfill not compacted
			Artificial Fill (Af):
T-4	0.0-2.0	CL	Clay: Dark bluish gray, moist, stiff, trace fine to medium sand, trace metal debris, trace fine gravel
			Older Alluvium (Qoal):
	2.0-4.5	CL	Clay: Dark bluish gray, moist, very stiff, trace minor orange oxidation staining
	4.5-6.0	SC	Clayey Sand: Brown mottled with brownish gray, moist, fine to medium sand, medium dense, trace fine gravel, pores present
			Total Depth: 6.0 feet No Caving No Ground Water Backfill not compacted

DESCRIPTIVE LOG OF TRENCH EXCAVATION

Trench Number	Depth (Feet)	U.S.C.S. Symbol	Field Description
			Artificial Fill (Af):
T-5	0.0-2.0	CL	Sandy Clay: Grayish brown and brownish gray, damp to moist, stiff, fine to medium sand, fine gravel, construction and scrap metal debris, alternating 1-3 inch-thick layers of increased sand and clay, rootlets present, 4 inch layer of asphalt at base.
			Older Alluvium (Qoal)
	2.0-4.5	CL	Clay: Dark bluish gray, moist, stiff to very stiff, trace rootlets, trace pinhole pores
	4.5-6.0	CL	Sandy Clay: Brownish gray mottled with light reddish brown, slightly moist, very stiff to hard, fine sand, trace medium sand, porous, trace reddish brown oxidation staining Total Depth: 6.0 feet No Caving No Groundwater Backfill not compacted
			Artificial Fill (Af):
T-6	0.0-2.0	SC/CL	Clayey Sand/Sandy Clay: Alternating layers of brown and dark grayish brown, dry to damp, medium dense/stiff, fine sand, some medium sand, porous, trace scrap metal and brick debris, broken layer of asphalt at base
	2.0-2.5	CL/SC	Sandy Clay/Clayey Sand: Brownish gray, moist, stiff/medium dense, fine sand, stiff, construction debris present
			Older Alluvium (Qoal):
	2.5-4.0	CL	Clay: Dark bluish gray, moist, stiff, trace fine sand, trace pinhole pores
	4.0-5.5	SC	Clayey Sand: Light reddish brown mottled with brownish gray, slightly moist, medium dense, fine sand, some medium sand, porous, 6 inch layer of reddish brown iron oxide cemented layer at 4 feet Total Depth: 5.5 feet No Caving No Groundwater Backfill not compacted

DESCRIPTIVE LOG OF TRENCH EXCAVATION

Trench Number	Depth (Feet)	U.S.C.S. Symbol	Field Description
			Artificial Fill (Af):
T-7	0.0-2.0	CL	Sandy Clay: Light reddish brown, moist, stiff fine sand, trace coarse sand, fine gravel, trace construction debris, 4-inch asphalt layer on west wall
			Older Alluvium (Qoal):
	2.0-3.5	CL	Clay: Dark bluish gray, moist, very stiff, trace pinhole pores, @1'-2', trace fine sand
	3.5-5.0	CL	Sandy Clay: Light brown mottled with gray, slightly moist to moist, stiff, fine sand, some medium sand, porous, trace fine gravel, reddish brown oxidation staining in upper 1 foot
			Total Depth: 5.0 feet No Caving No Groundwater Backfill not compacted
			Artificial Fill:
TP-8	0.0-2.0	SC	Clayey Sand: Light reddish brown to tan, slightly moist, medium dense, fine to medium sand, construction debris present, trace reddish brown oxidation staining
			Older Alluvium (Qoal):
	2.0-3.5	CL	Clay: Dark bluish gray, moist, very stiff, trace orange oxidation staining, trace pinhole pores
	3.5-4.5	SC/CL	Clayey Sand/Sandy Clay: Brown mottled with gray, damp, very stiff/dense, fine sand, some medium sand, pores present
			Total Depth: 4.5 feet No Caving No Groundwater Backfill not compacted

<u>DESCRIPTIVE LOG OF TRENCH EXCAVATION</u>

Trench Number	Depth (Feet)	U.S.C.S. Symbol	Field Description
			Artificial Fill (Af):
T-9	0.0-2.0	CL	Sandy Clay: Light reddish brown to tan, slightly moist, stiff, fine sand, metal debris present, trace fine gravel @2’, 4-inch asphalt layer
			Older Alluvium (Qoal):
	2.0-3.0	CL	Clay: Dark bluish gray, moist, very stiff, trace pinhole pores
			Total Depth: 3.0 feet No Caving No Groundwater Backfill not compacted
			Artificial Fill (Af):
T-10	0.0-2.5	CL	Sandy Clay: Brown mottled with gray, damp, stiff, fine sand, some medium sand, metal debris present @1’, A.C. debris
			Older Alluvium (Qoal):
	2.5-3.0	CL	Clay: Dark bluish gray, moist, stiff
			Total Depth: 3.0 feet No Caving No Groundwater Backfill not compacted

DESCRIPTIVE LOG OF TRENCH EXCAVATION

Trench Number	Depth (Feet)	U.S.C.S. Symbol	Field Description
			Artificial Fill (Af):
T-11	0.0-2.0	SM	Silty Sand: Brown mottled with gray, moist, medium dense, fine sand, with clay, metal debris present
			Older Alluvium (Qoal):
	2.0-3.0	CL	Clay: Dark bluish gray, moist, very stiff, trace brown staining, trace pinhole pores
			Total Depth: 3.0 feet No Caving No Groundwater Backfill not compacted

APPENDIX B

LABORATORY TEST PROGRAM

LABORATORY TESTING PROGRAM

Soil Classification

Soils encountered within the exploratory borings were initially classified in the field in general accordance with the visual-manual procedures of the Unified Soil Classification System (ASTM D2488). The samples were re-examined in the laboratory and classifications reviewed and then revised where appropriate. The assigned group symbols are presented in the Boring Logs provided in Appendix A.

In Situ Moisture and Density

Moisture content and dry density of in-place soil materials were determined in representative strata. Test data are summarized on the Boring Logs provided in Appendix A.

Maximum Dry Density and Optimum Moisture Content

Maximum dry density and optimum moisture content of onsite soils were determined for one selected sample in general accordance with Method A of ASTM D1557. Pertinent test values are given on Table B.

Particle-Size Analyses

Particle-size analyses were performed on selected samples in accordance with ASTM D 422-63. The results are presented graphically on the attached Plates B-1 and B-2.

Consolidation

Consolidation tests were performed for selected soil samples in general conformance with ASTM D 2435. Axial loads were applied in several increments to a laterally restrained 1-inch-high sample. Loads were applied in geometric progression by doubling the previous load, and the resulting deformations were recorded at selected time intervals. The test samples were inundated at selected loads to evaluate the effects of a sudden increase in moisture content (hydro-consolidation potential). Results of the tests are graphically presented on Plates B-3 to B-4.

Direct Shear

Direct shear tests were performed for samples remolded to 90 percent of the maximum dry density. These tests were performed in general accordance with ASTM D3080. Three specimens were prepared for each test. The test specimens were artificially saturated, and then sheared under varied normal loads at a constant rate. Results are graphically presented on Plate B-5.

Percent Passing the No. 200 Sieve

Percent of material passing the No. 200 sieve was determined on selected samples to verify visual classifications performed in the field. These tests were performed in accordance with ASTM D 1140. Test results are presented on Table B-1.

Atterberg Limits

Atterberg Limits (Liquid Limit, Plastic Limit, and Plasticity Index) were performed in accordance with Test Method ASTM D-4318. Pertinent test values are presented within Table B-1.

Expansion Potential

Expansion index testing was performed on selected samples. The test was performed in conformance with ASTM D 4829-11. The test results are presented on Table B.

Soluble Sulfate Content

A chemical analysis was performed on a selected soil sample to determine soluble sulfate content. The test was performed in accordance with California Test Method (CTM) 417. The test result is included in Table B.

Corrosion

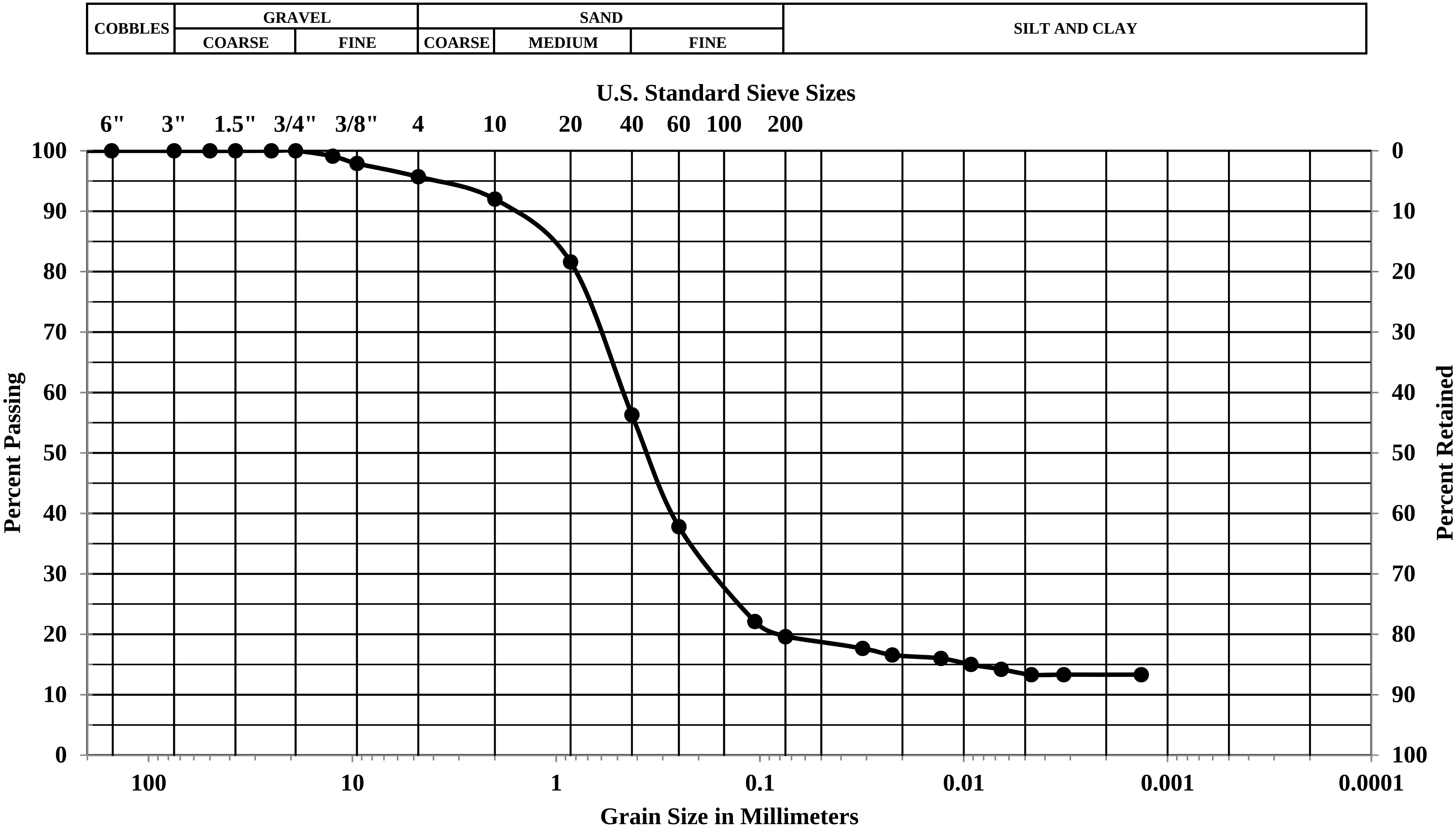
Select samples were tested for minimum resistivity, chloride, and pH in accordance with California Test Method (CTM) 643. Results of these tests are provided in Table B.

TABLE B
SUMMARY OF LABORATORY TEST RESULTS

Boring Number	Depth (feet)	Soil Type	Test Results	
B-1	0 – 5	Sandy Lean Clay (CL)	Maximum Dry Density (pcf): Optimum Moisture Content (%): Soluble Sulfate Content (%): Sulfate Exposure: Expansion Index: Expansion Potential: Liquid Limit: Plastic Index:	115.5 16.0% 0.002% Negligible 39 Low 29 % 13 %
B-1	15	Sandy Lean Clay (CL)	Liquid Limit: Plastic Index:	35 % 17 %
B-1	20	Sandy Lean Clay (CL)	Liquid Limit: Plastic Index:	33 % 18 %
B-1	25	Sandy Lean Clay (CL)	Liquid Limit: Plastic Index:	31 % 10 %
B-1	31	Sandy Lean Clay (CL)	Liquid Limit: Plastic Index:	29% 9 %
B-4	0 – 5	Sandy Fat Clay (CH)	Expansion Index: Expansion Potential: Minimum Resistivity: pH: Chloride: Liquid Limit: Plastic Index:	96 High 700 Ohm-cm 7.5 24.3 ppm 58 % 41%
T-8	2.5	Clay (CL)	Expansion Index: Expansion Potential:	89 High

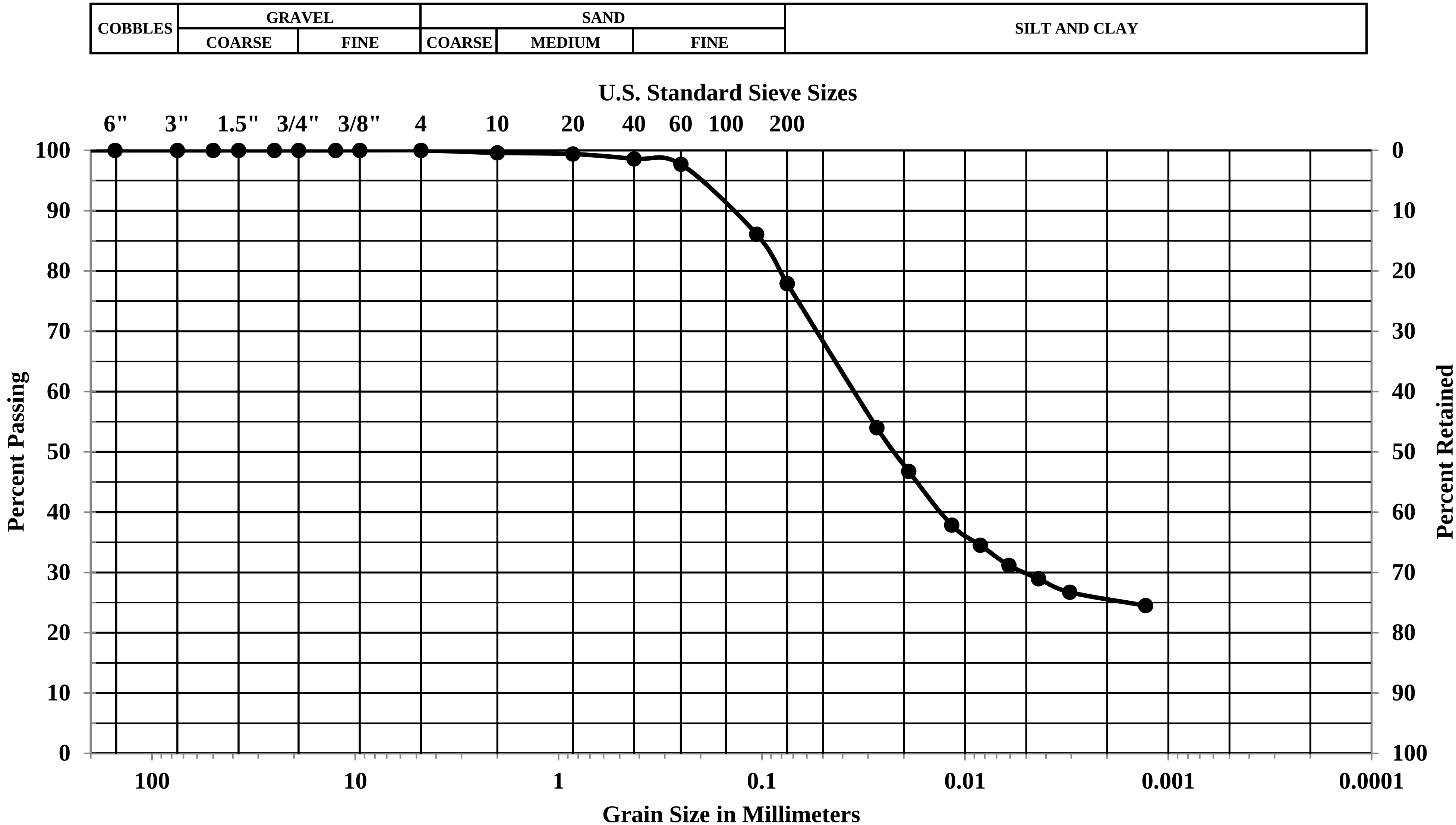
Additional laboratory test results are provided on the boring logs provided in Appendix A and on the Plates that follow.

GRAIN SIZE DISTRIBUTION



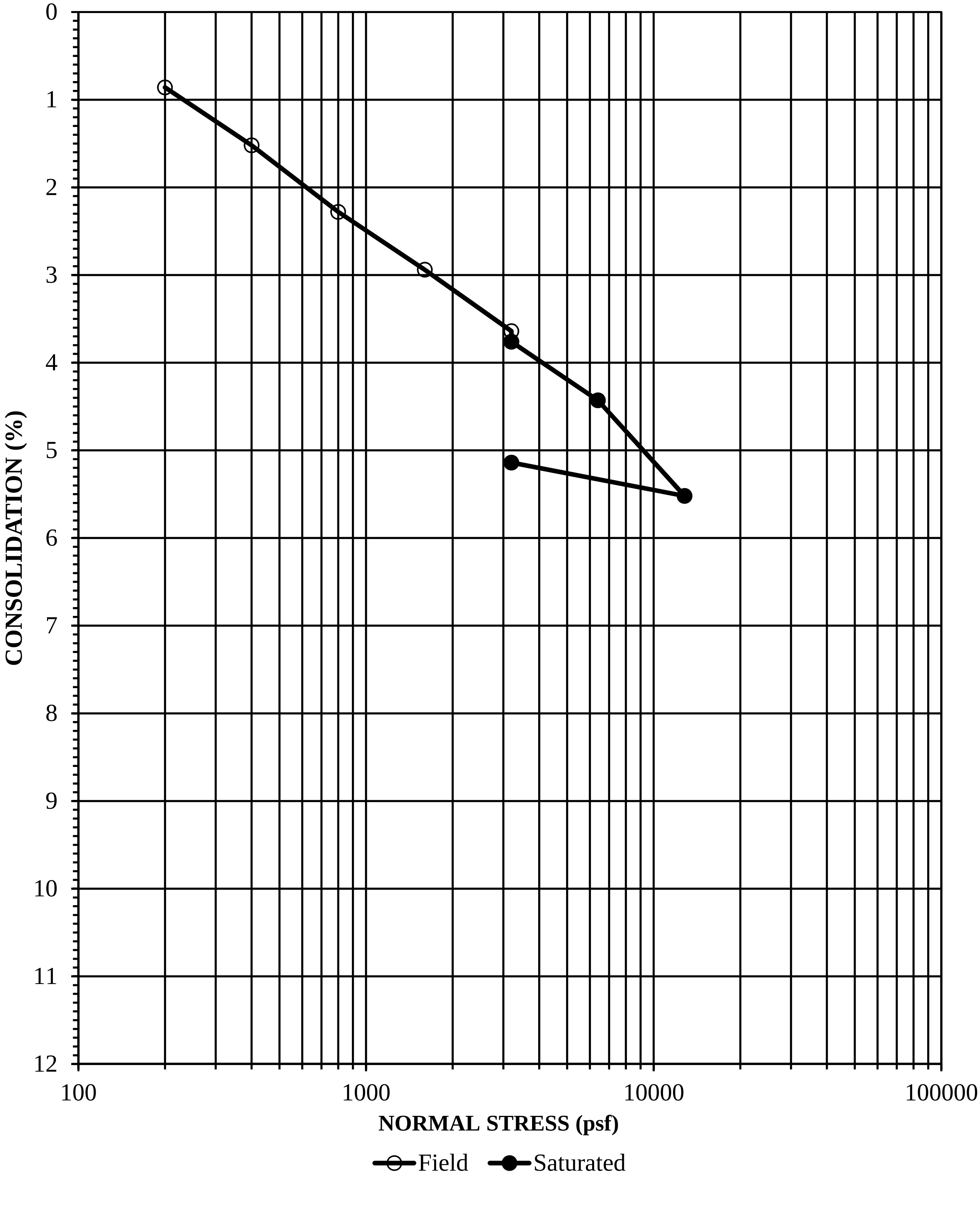
Job Number	Location	Depth	Description
2789.00	B-1	10	Clayey Sand (SC)

GRAIN SIZE DISTRIBUTION



Job Number	Location	Depth	Description
2789.00	B-1	15	Sandy Lean Clay (CL)

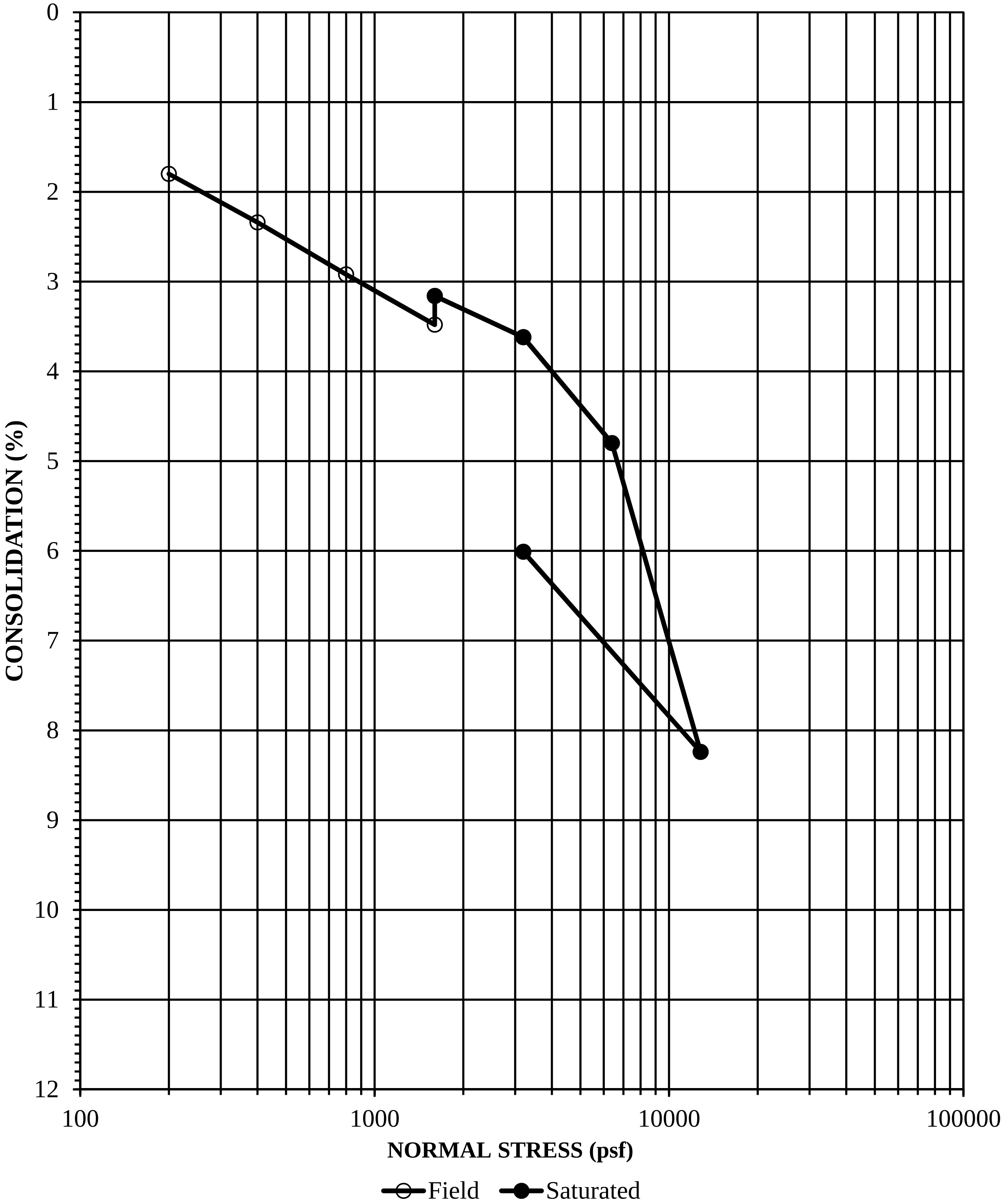
CONSOLIDATION



Job Number	Location	Depth	Description
2789.00	B-1	6	Clayey Sand-Sandy Clay (SC-Cl)

Initial Dry Density (pcf)	Initial Moisture Content (%)	Final Moisture Concent (%)
116.9	14.2	14.1

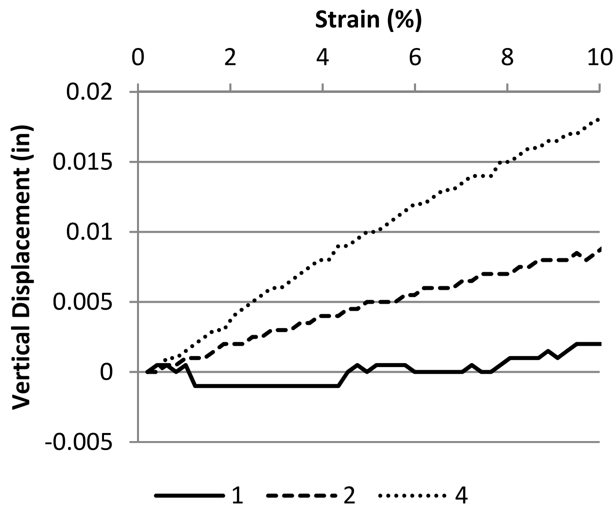
CONSOLIDATION

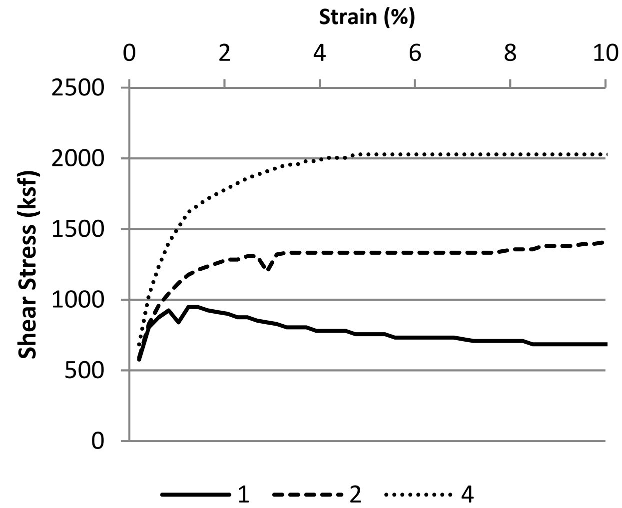


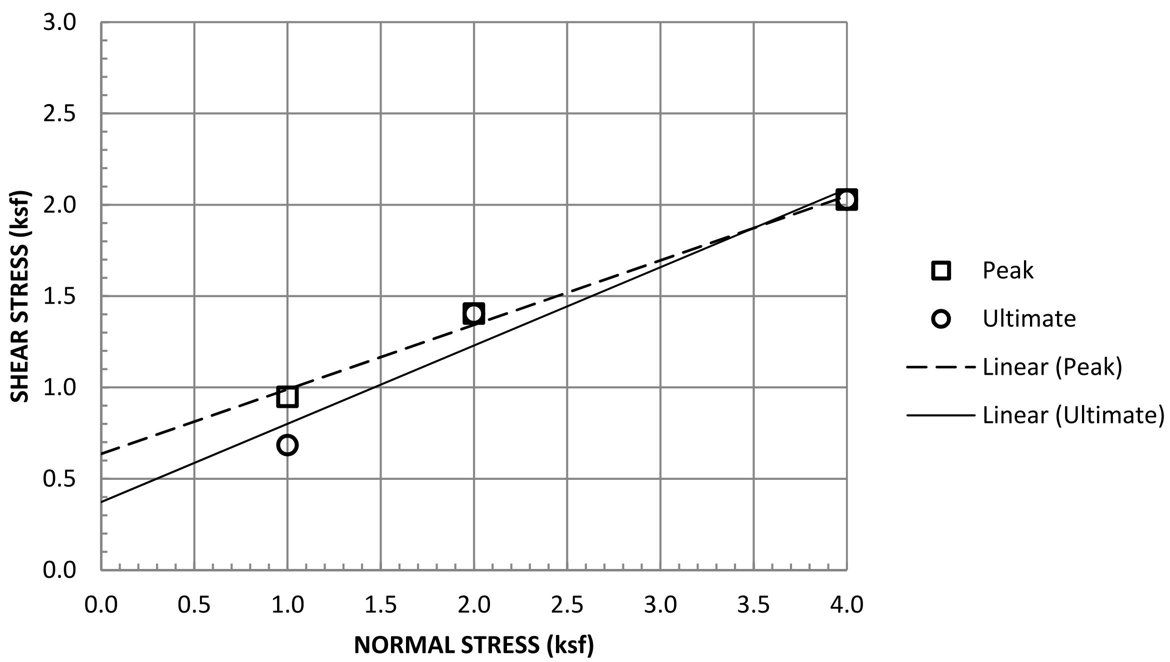
Job Number	Location	Depth	Description
2789.00	B-2	6	Sandy Clay (CL)

Initial Dry Density (pcf)	Initial Moisture Content (%)	Final Moisture Concent (%)
101.5	22.4	22.6

DIRECT SHEAR







Sample Type:	Remolded 90% of 115.5 @ 16%, Saturated		
Normal Stress (ksf)	1	2	4
Peak Shear Stress (ksf)	0.948	1.404	2.028
Peak Displacement (in)	0.003	0.009	0.018
Ultimate Shear Stress (ksf)	0.684	1.404	2.028
Ultimate Displacement (in)	0.25	0.25	0.25
Initial Dry Density (pcf)	104	104	104
Initial Moisture Content (%)	16	16	16
Final Moisture Content (%)	19.8	19.5	19.3
Strain Rate (in/min)	.005		

Job Number	Location	Depth	Description
2789.00	B-4	0-5	Sandy Fat Clay (CH)

APPENDIX C

LIQUEFACTION ANALYSES

GeoLogismiki
Geotechnical Engineers
Merarhias 56
http://www.geologismiki.gr

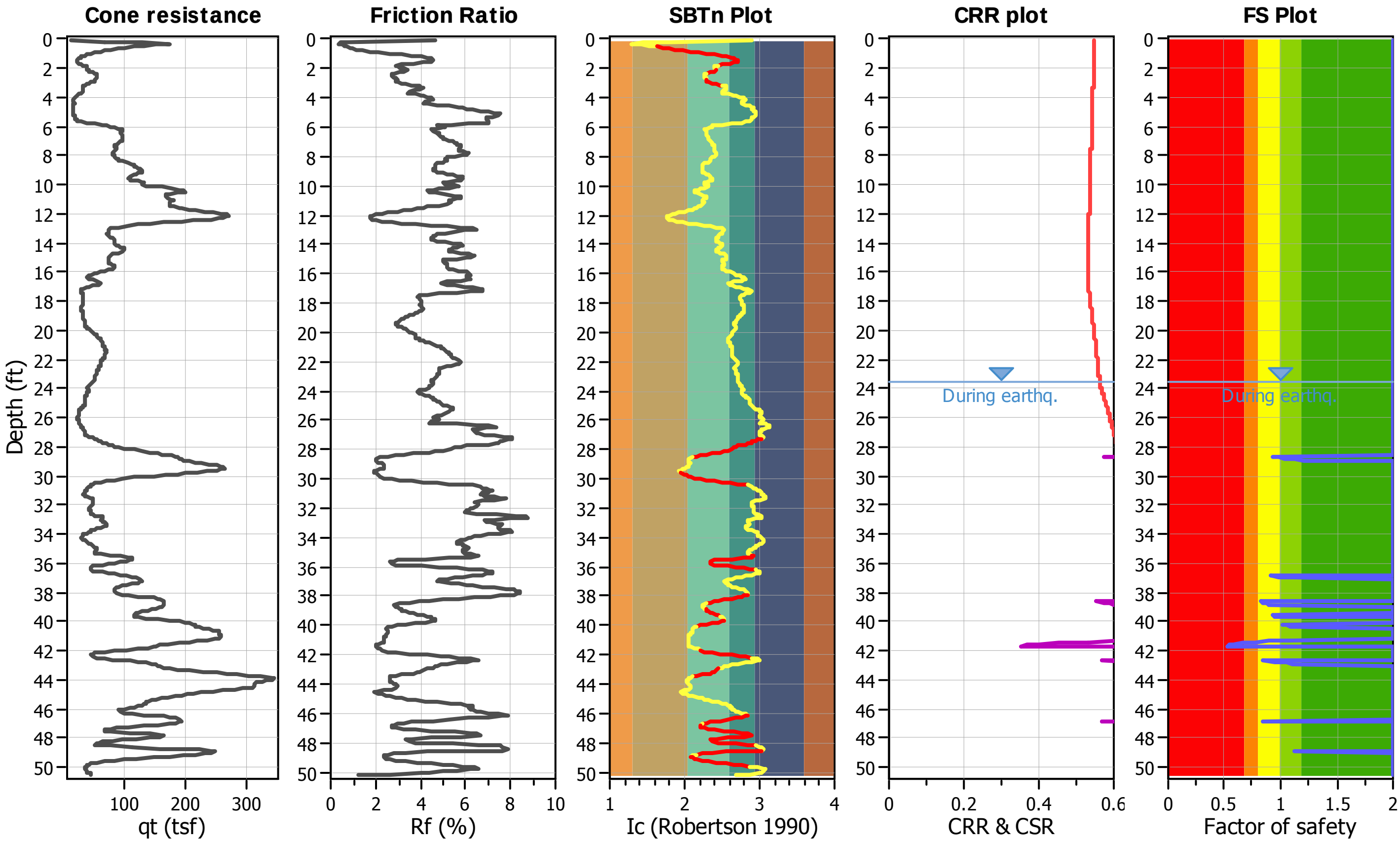
LIQUEFACTION ANALYSIS REPORT

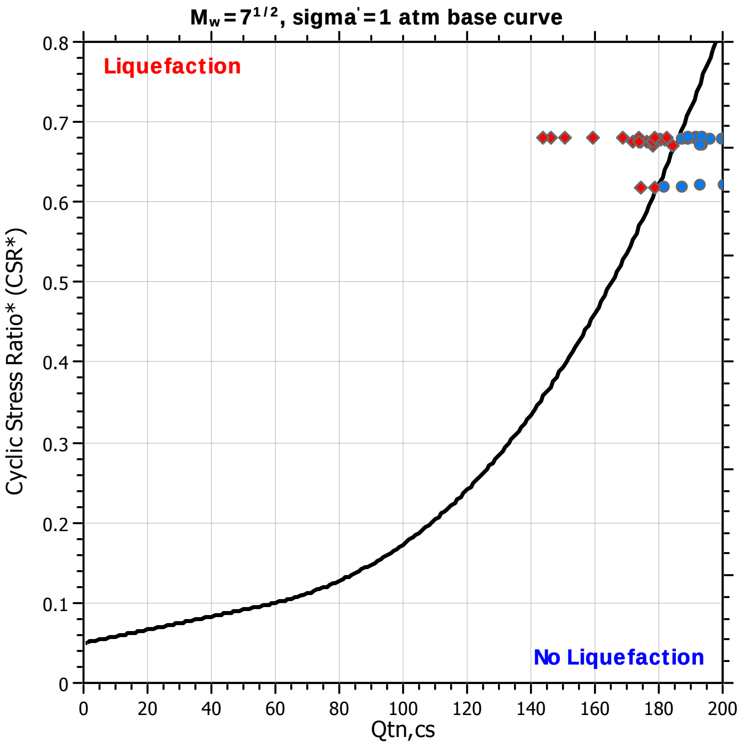
Project title : **Location :**

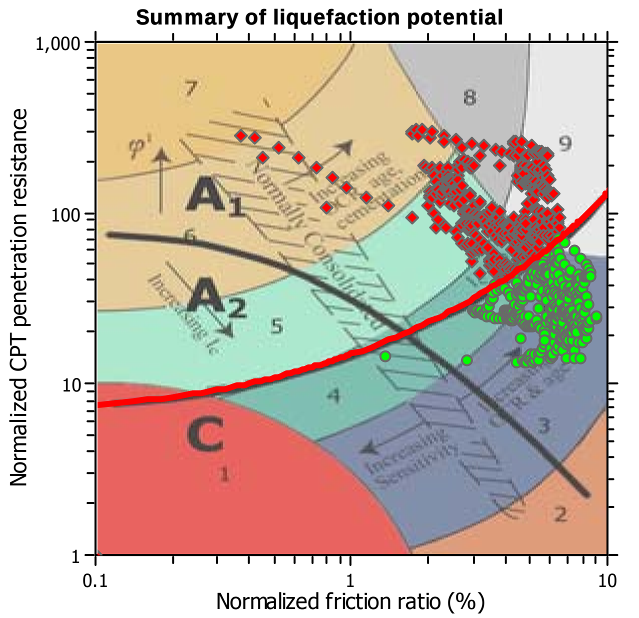
CPT file : CPT-1

Input parameters and analysis data

Analysis method:	NCEER (1998)	G.W.T. (in-situ):	23.60 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	23.60 ft	Fill height:	N/A	Limit depth applied:	Yes
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	50.00 ft
Earthquake magnitude M_w:	7.73	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.60	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

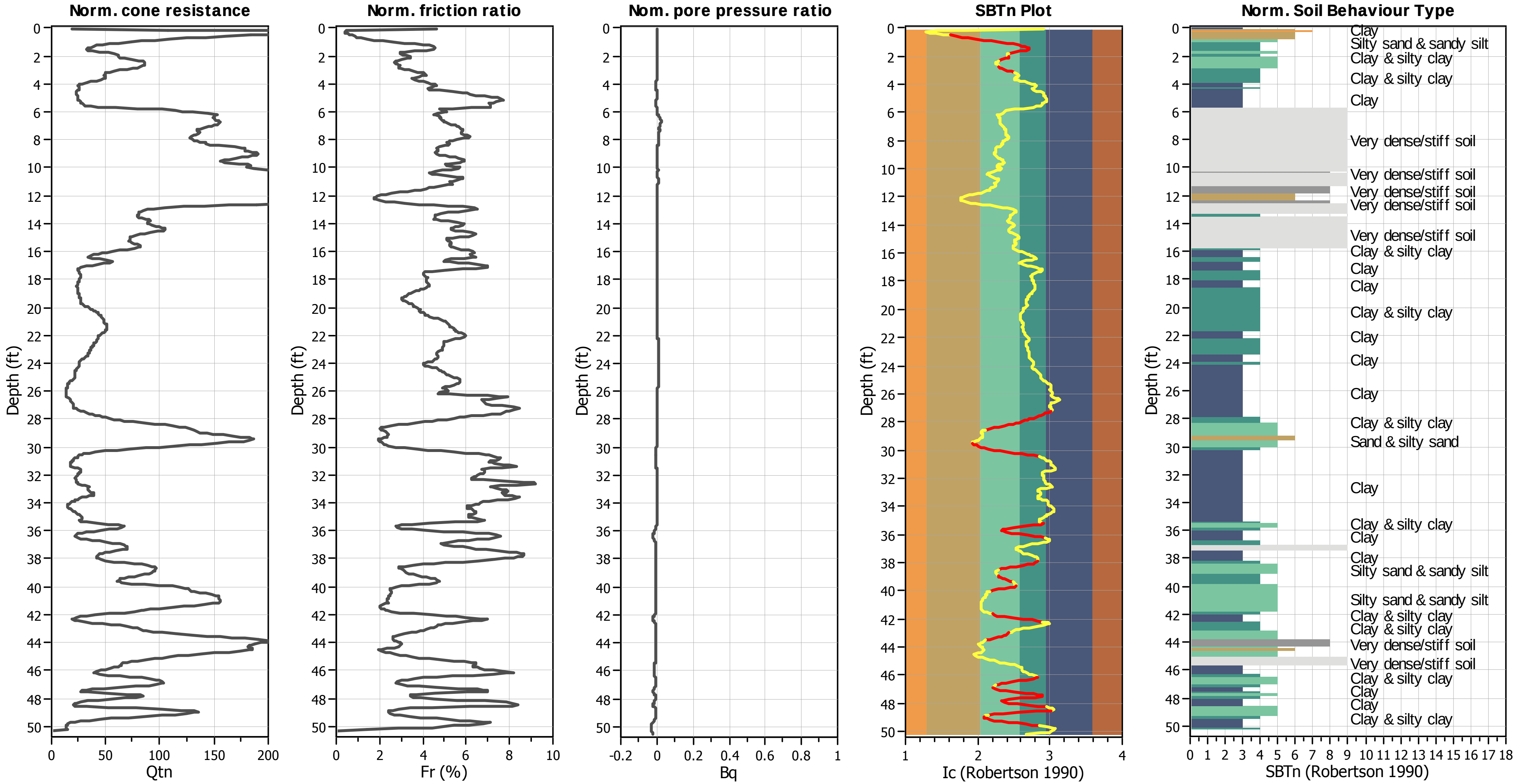






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

CPT basic interpretation plots (normalized)



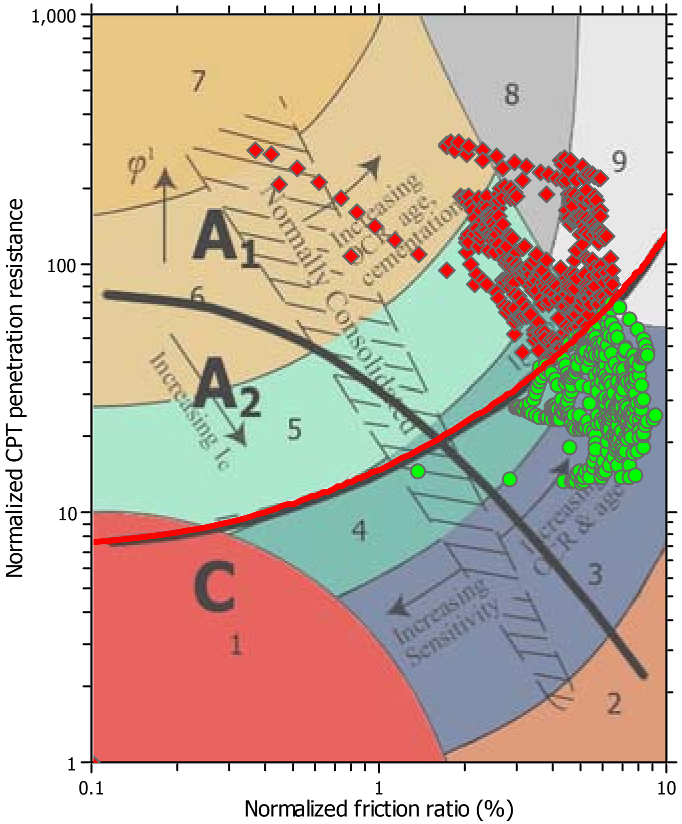
Input parameters and analysis data

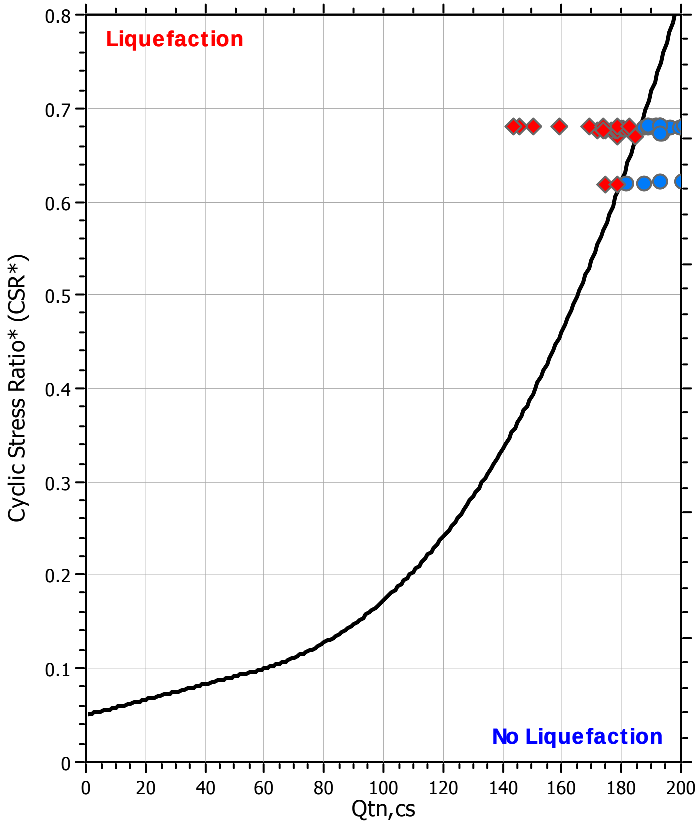
Analysis method:	NCEER (1998)	Depth to water table (erthq.):	23.60 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.73	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.60	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	23.60 ft	Fill height:	N/A	Limit depth:	50.00 ft

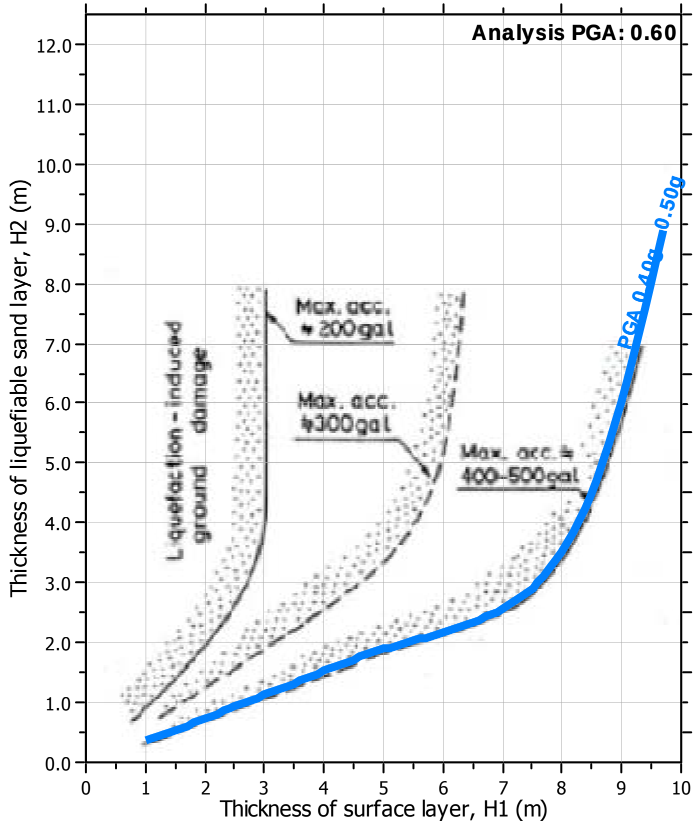
SBTn legend

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

Liquefaction analysis summary plots



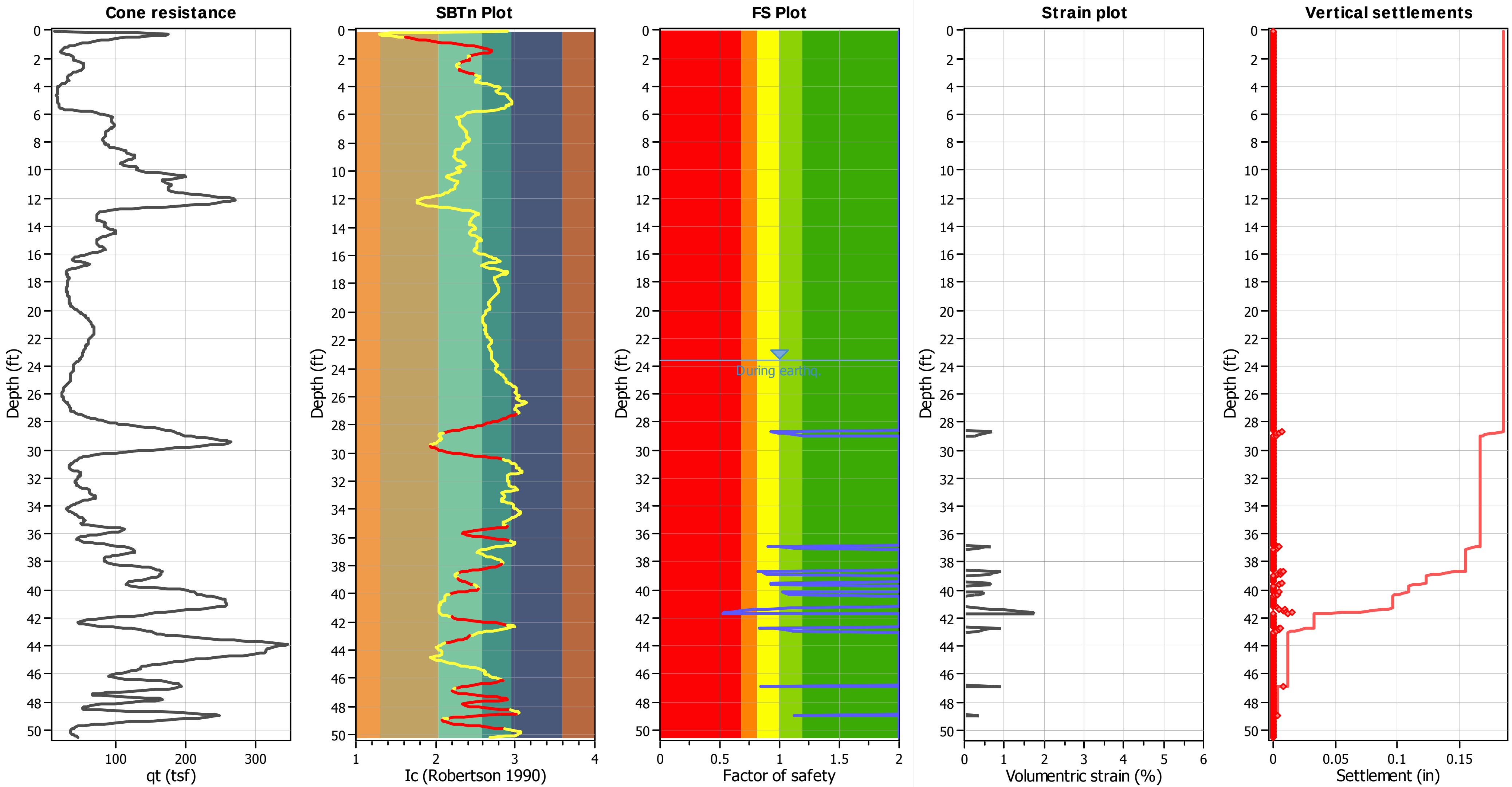




Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	23.60 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_{σ} applied:	Yes
Earthquake magnitude M_w:	7.73	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.60	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	23.60 ft	Fill height:	N/A	Limit depth:	50.00 ft

Estimation of post-earthquake settlements



Abbreviations

q_t:	Total cone resistance (cone resistance q_c corrected for pore water effects)
I_c:	Soil Behaviour Type Index
FS:	Calculated Factor of Safety against liquefaction
Volumentric strain:	Post-liquefaction volumentric strain