ALBUS & ASSOCIATES

July 1, 2025
J.N.: 3341.00

Mr. Chad Brown
Melia Homes
8951 Research Drive
Irvine, California 92618

Subject: **Preliminary Geotechnical Investigation Report, Proposed Residential Development, 9822 Russell Avenue, Garden Grove, California**

Dear Mr. Brown,

Albus & Associates, Inc. is pleased to present to you our preliminary geotechnical investigation report for the proposed development at the subject site. This report presents the results of our literature review, subsurface exploration, laboratory testing, and engineering analyses. Conclusions relevant to the feasibility of the proposed site development are also presented herein based on the findings of our work.

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.

Sincerely,

ALBUS & ASSOCIATES, INC.

David E. Albus
Principal Engineer

REPORT

FIGURES AND PLATES

APPENDICES

1.0 INTRODUCTION

1.1 PURPOSE AND SCOPE

The purpose of our work was to evaluate the feasibility of the proposed site development in order to assist you in your land acquisition evaluation and due-diligence review. The scope of our work for this investigation was focused primarily on the geotechnical issues that we expect could have significant fiscal impacts on future site development. While this report is comprehensive for feasibility purposes, it is not intended for final design purposes. As such, additional geotechnical studies may be warranted based on our review of future rough grading plans and foundation plans. The scope of our work for this investigation included the following:

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

1.2 SITE LOCATION AND DESCRIPTION

The property is located at 9822 Russell Avenue within the city of Garden Grove. The site is rectangularly shaped and bordered by Kerry Street to the west, Russel Avenue to the north, residential properties to the south, and Sunnyside Elementary School to the east. The location of the site and its relationship to the surrounding areas are shown on Figure 1, Site Location Map.

The site is currently occupied by a church located within the northwest corner of the site. Additional structures are also present within the northwest portion of the site and appear to be associated with a child day care center. The remainder of the site is covered in asphalt with driveways and parking bays associated with the onsite improvements. Chain link fencing is present along the east and a portion of the south. A masonry block wall is also present along the south property line.

The site is relatively flat with elevations ranging from 70 to 72 feet above Mean Sea Level (based on Google Earth). Drainage is generally directed as sheet flow to either Russell Avenue or to Kerry Street. Site vegetation consists of bushes and moderate sized trees.

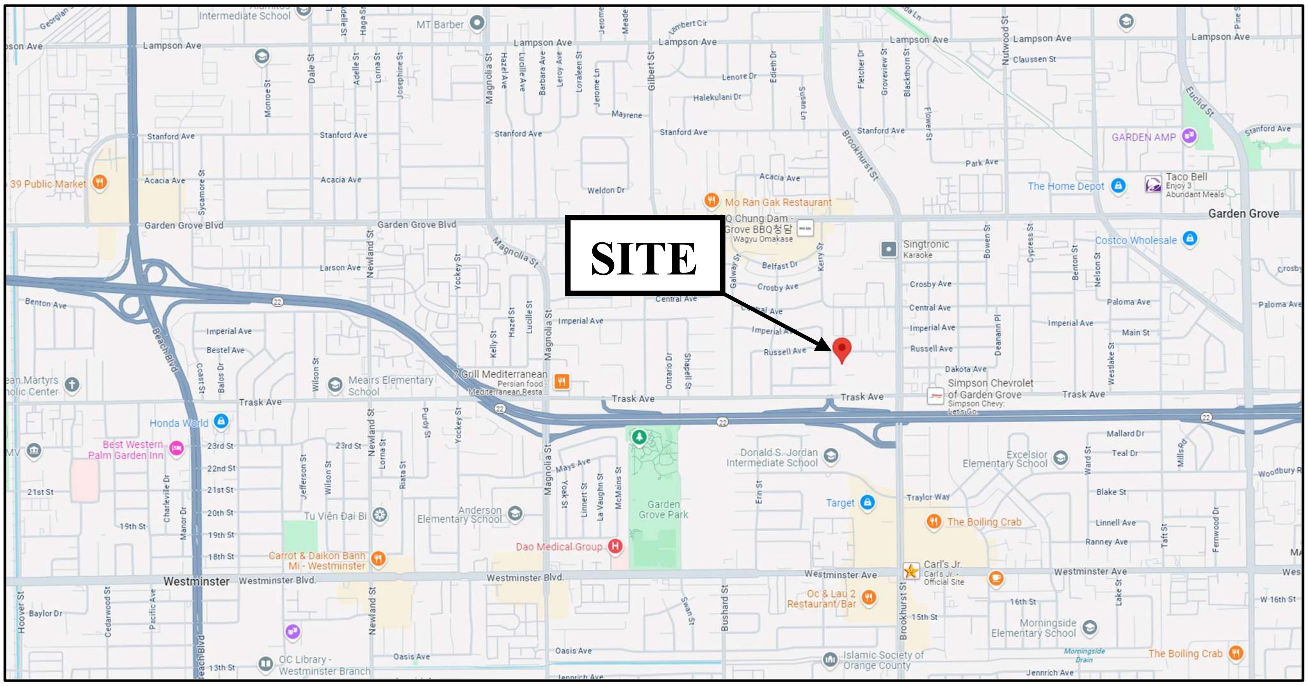

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FIGURE 1-SITE LOCATION MAP

Proposed Residential Development
9822 Russell Avenue, Garden Grove, California

NOT TO SCALE

1.3 PROPOSED DEVELOPMENT

Review of the conceptual site plan provided suggests the site will be constructed with (26) twenty-six 2-story townhomes at grade within six buildings. Associated interior driveways, parking, underground utilities, and decorative hardscape and landscape areas are also anticipated.

No grading or structural plans were available in preparing this report. However, we anticipate minor cuts and filling of the site will be required to achieve future surface configuration, and we expect future foundation loads will be relatively light.

2.0 INVESTIGATION

2.1 RESEARCH

We have reviewed referenced geologic publications, maps, and historical aerial photos of the vicinity. Data from these sources were utilized for the development of some of our findings and conclusions presented in this report.

A cursory review of the geologic map of Orange County by Morton (1981) in the vicinity of the site suggests the site is underlain by alluvium and colluvium (Qac).

Historical imagery suggests that the site was undeveloped as far back as 1953. By 1963, the site appears to have been developed into its current configuration. The site appears to be relatively unchanged since the date of this proposal.

2.2 SUBSURFACE EXPLORATION

Subsurface exploration for this investigation was completed on June 10, 2025. Three (3) exploratory borings were drilled to depths of approximately 11.5 to 51.5 feet below existing ground surface utilizing a truck-mounted, hollow-stem-auger drill rig. A representative of ***Albus & Associates, In*c.** logged the exploratory excavation. Visual and tactile identifications were made of the materials encountered, and their descriptions are presented on the Exploration Log in Appendix A.

The CPT soundings were advanced using a 30-ton CPT truck, utilizing an external subcontactor. 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 for the determination of the general soil type. Following the completion of the CPT soundings, logs are generated that provide a relatively continuous profile of the tip resistance, side resistance, and pore water measurements. Copies of the CPT logs are provided in Appendix A.

Bulk, relatively undisturbed, and Standard Penetration Test (SPT) samples were obtained at selected depths 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 using a standard SPT soil sampler. During each sampling interval, the samplers were 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 and testing. The borings were backfilled with soil cuttings in a non-compacted state, and asphalt was patched upon completion of drilling.

The approximate locations of the exploratory excavations completed by this firm are shown on the enclosed Geotechnical Map, Plate 1.

2.3 LABORATORY TESTING

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

3.0 SUBSURFACE CONDITIONS

3.1 SUBSOIL CONDITIONS

Artificial fills (Af) were encountered within the upper 2 feet across the site within the areas of our exploration. The fills were predominately sands with silts, damp, and fine- to medium-grained. Site exploration was limited to areas within the existing asphalt due to the existing buildings onsite. We anticipate deeper, localized fills below and surrounding the existing buildings which would have resulted from previous grading activities.

Alluvium (Qal) was encountered below the fills for the maximum depth explored of 51.5 feet. The alluvium consisted of sands with silt, sandy silts, silty sands / sandy silts, and silty sands.

A more detailed description of the interpreted soil profile at each of the boring locations is presented in Appendix A. The stratigraphic descriptions in the logs represent the predominant materials encountered and relatively thin, often discontinuous layers of different material may occur within the major divisions.

3.2 GROUNDWATER

Groundwater was encountered during this firm’s subsurface exploration at a depth of 14 feet. The CDMG Special Report 03 suggests that historic high groundwater for the subject site is approximately 13-14 feet.

3.3 FAULTING

Geologic literature and field exploration do not indicate the presence of active faulting within the site. The site does not lie within an "Earthquake Fault Zone" as defined by the State of California in the Earthquake Fault Zoning Act. Table 3.1 presents a summary of all the known seismically active faults within 10 miles of the site based on the 2008 National Seismic Hazards Maps.

TABLE 3.1
SUMMARY OF FAULTS

Name	Distance (miles)	Slip Rate (mm/yr.)	Preferred Dip (degrees)	Slip Sense	Rupture Top (km)	Fault Length (km)
San Joaquin hills	5.22	0.5	23	thrust	2	27
Newport-Inglewood, alt 1	6.46	1	88	strike slip	0	65
Newport Inglewood Connected alt 1	6.46	1.3	89	strike slip	0	208
Newport Inglewood Connected alt 2	6.49	1.3	90	strike slip	0	208
Puente Hills (Coyote Hills)	7.25	0.7	26	thrust	2.8	17

4.0 ANALYSES

4.1 SEISMICITY AND SEISMIC DESIGN PARAMETERS

The 2022 CBC requires seismic parameters in accordance with ASCE 7-16. Unless noted otherwise, all section numbers cited in the following refer to the sections in ASCE 7-16.

The site is underlain by soil strata that are susceptible to liquefaction. As such, per item 1 in Section 20.3.1, the project site should be designated Site Class F. However, because the proposed developments are anticipated to have fundamental periods of less than 0.5 seconds, per the same (referenced item) the site is exempted from Site Class F designation. Instead, site class can be designated per Section 20.3. Using weighted average SPT blow count N > 15 (across the upper 100 ft of the soil profile) in Table 20.3-1, Site Class D is assigned. (or: Using weighted average SPT blow count N < 15 (across the upper 100 ft of the soil profile) in Table 20.3-1, Site Class E is assigned.). We used the OSHPD seismic hazard tool to obtain the basic mapped acceleration parameters, including short periods (S_S) and 1-second period (S_1) MCE_R Spectral Response Accelerations. Section 11.4.8 requires site-specific ground hazard analysis for structures on Site Class E with S_S greater than or equal to 1.0 or Site Class D or E with S_1 greater than or equal to 0.2. Based on the mapped values of S_S and S_1 the project site falls within this category, requiring site specific hazard analysis in accordance with Section 21.2.

According to Section 21.2.3 (Supplement 1), the site-specific Risk Targeted Maximum Considered Earthquake (MCE_R) spectral response acceleration at any period is the lesser of the probabilistic and the deterministic response accelerations, subject to the exception specified in the same section. The probabilistic response spectrum was developed using the computer program OpenSHA (Field et al., 2013), which implements Method 1 as described on Section 21.2.1.1. Fault Models 3.1 and 3.2 from the Third Uniform California Earthquake Rupture Forecast (UCERF3) were used as the earthquake rupture forecast models for the PSHA. In addition to known fault sources, background seismicity was also included in the PSHA. The ground motion Prediction Equations (GMPEs) selected for use in this analysis are those developed for the Pacific Earthquake Engineering Research Center (PEER) Next

Generation Attenuation (NGA) West 2 project. Four GMPEs - Abrahamson et al. (2014), Boore et al. (2014), Campbell and Bozorgnia (2014), and Chiou and Youngs (2014) were used to perform the analysis.

In accordance with Section 21.2.2 (Supplement 1), the deterministic spectral response acceleration at each period was calculated as the 84th percentile, 5% damped response acceleration, using NGA-West2 GMPE Worksheet. For this, information from at least three causative faults with the greatest contribution per deaggregation analysis were used and the larger acceleration spectrum among these was selected as the deterministic response spectrum. The deterministic spectrum was adjusted per requirements in Section 21.2.2 (Supplement 1) where applicable. Both probabilistic and deterministic spectra were subjected to the maximum direction scale factors specified in Section 21.2 to produce the maximum acceleration spectra.

Design response spectrum was developed by subjecting the site-specific MCE_R response spectrum to the provisions outlined in Section 21.3. This process included comparison with 80% code-based design spectrum determined in accordance with Section 11.4.6. The short period and long period site coefficient (Fa and Fv, respectively) were determined per Section 21.3 in conjunction with Table 11.4-1. Site specific design acceleration parameters (S_{MS}, S_{M1}, S_{DS}, and S_{D1}) were calculated according to Section 21.4.

Per Section 11.2 (definitions on Page 79 of ASCE7-16) for evaluation of liquefaction, lateral spreading, seismic settlements, and other soil-related issues, Maximum Considered Earthquake Geometric Mean (MCE_G) peak ground acceleration PGA_M shall be used. The site-specific PGA_M is calculated per Section 21.5.3, as the lesser of the probabilistic PGA_M (Section 21.5.1) and deterministic PGA_M (Section 21.5.2), but no less than 80% site modified peak ground acceleration, PGA_M, obtained from OSHPD seismic hazard tool. According to Section 21.5.3 of ASCE 7, either mapped or site-specific PGAm may be used for soil related analysis. For this investigation, a mapped PGAm of 0.67 was incorporated into the analysis.

4.2 STATIC SETTLEMENT

Analyses were performed for the potential settlement of the underlying soils encountered during our investigation. The site is predominately granular in nature. As such, analyses of settlement were based on the elastic method using estimated moduli correlated from N_{60} blow counts, as well as consideration from consolidation testing. Two analyses were performed to evaluate settlement of the structures. The first model was based on a conventional shallow strip footing 1.2 feet wide and a wall load of 3,000 plf. The second model was based on a conventional square footing with 3 feet width and a column load of 27 kips. Both models yielded a total settlement of less than 0.5 inches.

4.3 LIQUEFACTION

Engineering research of soil liquefaction potential (Youd, et al., 2001) indicates that generally three basic factors must exist concurrently in order for liquefaction to occur. These factors include:

- A source of ground shaking, such as an earthquake, capable of generating soil mass distortions.
- A relatively loose silty and/or sandy soil.

- A relative shallow groundwater table (within approximately 50 feet below ground surface) or completely saturated soil conditions that will allow positive pore pressure generation.

The liquefaction susceptibility of the onsite subsurface soils was evaluated by analyzing the potential concurrent occurrence of the above-mentioned three basic factors. The liquefaction evaluation for the site was completed under the guidance of Special Publication 117A: Guidelines for Evaluating and Mitigating Seismic Hazards in California (CDMG, 2008).

The liquefaction analyses were based on field and laboratory testing data results from CPT-1 and CPT-2. The liquefaction analyses were performed utilizing the CLiq software by GeoLogismiki which incorporates the methods by NCEER (1998) and Robertson (2009) for the CPT data.

Historic high groundwater was assumed at a depth of 13 feet below the existing ground surface based on our discussion in Section 3.2. Fine-grained soils that do not have a Plasticity Index (PI) less than 12 and field moisture contents greater than 85% of liquid limit (LL) or soils with corrected blow counts greater than 30 per foot were assumed to be not susceptible to liquefaction. Based on our analysis, some layers of granular soils below a depth of 13 feet have factors of safety below 1.3 are prone to liquefaction during the design earthquake event. The results of our liquefaction analyses are presented in Appendix C.

5.0 CONCLUSIONS

5.1 FEASIBILITY OF PROPOSED DEVELOPMENT

From a geotechnical point of view, the proposed site development is 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 grading and construction is performed in accordance with the recommendations presented in this report. Key issues that could have significant 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 known active faults are known to project through the subject site, nor do the sites lie within the boundaries of an "Earthquake Fault Zone" as defined by the State of California in the Alquist-Priolo Earthquake Fault Zoning Act. The closest known active fault is the San Joaquin hills fault, located approximately 5.22 miles away. Therefore, the potential for ground rupture due to an earthquake beneath the sites is considered low.

5.2.2 Ground Shaking

The site is situated in a seismically active area that has historically been affected by generally 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 structures, the property will probably experience similar 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 and construction in accordance with the current California Building Code (C.B.C.) requirements are anticipated to adequately address potential ground shaking. Potential ground accelerations have been estimated for the site and are presented in Section 4.1 of this report.

5.2.3 Liquefaction

Based on our analyses, liquefaction may occur below the site during periods of strong ground motion. Our analyses indicate liquefaction could lead to a total seismic settlement (saturated and dry) of the ground surface of up to approximately 5.5 inches due to seismic consolidation during liquefaction. Given this condition, differential settlement due to seismic settlement would likely be on the order of half of the total seismic settlement or approximately 2.75 inches over 30 feet or approximately 0.01L.

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)]. The use of well-reinforced foundations, such as post-tensioned slabs, grade beams with structural slabs, or mat foundations, have been proven to adequately provide basal support for similar structures during comparable liquefaction events. Further, ASCE 7-16, Section 12.13.9.2 allows for use of properly designed foundations for mitigation of seismic settlement. Since the estimated differential settlement is less than 0.015L, structural mitigation is permitted. Specific recommendations for mitigation of differential seismic settlement are provided in Section 6.3.

5.3 STATIC SETTLEMENT

Provided the existing near surface soils are removed and recompacted, total and differential static settlement can likely be limited to a maximum of 1 inch and ½-inch over 30 feet, respectively. These estimated magnitudes of static settlements are considered within tolerable limits for the proposed residential structures. Specific recommendations for ground preparation are provided in Section 6.1.4.

5.4 EXCAVATION AND MATERIAL CHARACTERISTICS

Onsite earth materials are anticipated to be relatively easy to excavate with conventional heavy earthmoving equipment. The site earth materials are generally considered suitable for reuse as fill provided they are cleared of deleterious debris and oversized rocks (greater than 4 inches in greatest dimension). If encountered, portions of concrete debris and asphalt can likely be reduced in size (4" minus) and incorporated within fill soils during earthwork operations.

Temporary construction slopes will be required to complete removal of unsuitable soils and for construction of underground utilities. Site materials are typically coarse-grained and may be prone to caving. Such excavations will require laybacks where they are surcharged or where they exceed 4 feet in height. Specific recommendations are provided in Section 6.1.5.

Most of the near-surface soils are below optimum moisture content. As such, the addition of water to these materials will be required during placement and compaction as engineered fills.

5.5 SHRINKAGE AND SUBSIDENCE

Volumetric changes in earth quantities will occur when excavated onsite soil materials are replaced as properly compacted fill. We estimate the existing upper 4 feet of earth materials will shrink up to approximately 8 to 13 percent. The subsidence of removal bottoms is estimated to be on the order of 0.15 feet. The estimates of shrinkage and/or bulkage 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 swelling and bulkage that occurs during the grading process.

5.6 SOIL EXPANSION

Based on USCS visual manual classification, the near-surface soils within the site are generally anticipated to possess a **Very Low** expansion potential. Additional testing for soil expansion will be required subsequent to rough grading and prior to construction of foundations and other concrete work to confirm these conditions.

6.0 RECOMMENDATIONS

6.1 EARTHWORK

6.1.1 General Earthwork and Grading Specifications

All earthwork and grading should be performed in accordance with applicable requirements of Cal/OSHA, applicable specifications of the Grading Codes of the City of Garden Grove, California in addition to the recommendations presented herein.

6.1.2 Pre-Grade Meeting and Geotechnical Observation

Prior to commencement of grading, we recommend a meeting be held between the developer, City Inspector, grading contractor, civil engineer, and geotechnical consultant to discuss the proposed grading and construction logistics. We also recommend that a geotechnical consultant be retained to provide soil engineering and engineering geologic services during site grading and foundation construction. 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 that appear 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 existing site improvements, oversized materials, vegetation and other deleterious materials should be removed from the areas to be developed. Existing underground improvements such as utility lines, septic tanks, seepage pits, etc. are also anticipated at the site. If encountered during site development, these improvements should also be completely removed from the site and seepage pits should be properly abandoned in accordance with the requirements established by the governing agencies as well as recommendations made in the field by the project geotechnical consultant.

In general, seepage pits that are open should be cleared of any fluids and then filled with 2-sack cement slurry up to within 5 feet of proposed grades. Any brick lining that remains in the upper 5 feet should be removed and the remainder of the pit filled with engineered fill in accordance with Section 6.1.6. Seepage pits that are presently backfilled with soil should be removed to a depth of 10 feet below pad grade and be capped with 2-sack cement slurry. The slurry cap should be at least 5 feet thick and should extend at least 12 inches outside the perimeter of the seepage pit. The remaining 5 feet should be filled with engineered fill in accordance with Section 6.1.6.

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 and excavation should be left open for observation by the geotechnical consultant. Should any unusual soil conditions or subsurface structures be encountered during site clearing or grading that are not described or anticipated herein, these conditions should be brought to the immediate attention of the project geotechnical consultant for corrective recommendations as needed.

Concrete debris generated by site demolition can be reduced to no more than 4 inches in maximum dimension and uniformly incorporated with fill soils during earthwork operations.

6.1.4 Ground Preparation

Within the building pad areas and retaining walls, the existing artificial fills are considered unsuitable for supporting proposed structures and site improvements. Therefore, the upper 2 feet of the existing earth materials should be removed and replaced as engineered compacted fills. In addition to general removal of unsuitable soils, the existing soils should be over-excavated to a minimum depth of 1 foot below the bottom of footings for residential structures and one foot below retaining walls.

Existing soil within roadways and parking areas should be removed to at least 12 inches below the proposed pavement subgrade and replaced with engineered compacted fill.

Removals should extend laterally beyond the limits of the proposed buildings, a distance equal to the depth of removal (i.e. 1:1 projection) but not less than 5 feet. Existing soils below proposed pavement should be removed laterally to at least the edge of the pavement. 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.

The grading contractor should take appropriate measures when excavating adjacent any existing improvements to remain in-place to avoid disturbing or compromising support of existing structures.

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

6.1.5 Temporary Excavations

Temporary construction slopes and trench excavations may be cut vertically up to a height of 4 feet provided that no surcharging of the excavations is present. Temporary excavations greater than 4 feet in height but no more than 10 feet should be laid back to a 1:1 (H:V) or flatter or shored to mitigate the potential for instability. Where temporary excavations expose friable granular soils, the excavation should be laid back to a gradient of 1.5:1 (H:V).

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.

The grading contractor should take appropriate measures when excavating adjacent existing improvements to avoid disturbing or compromising support of existing structures.

6.1.6 Fill Placement

Materials excavated from the site may be reused as fill provided they are free of deleterious materials and particles greater than 4 inches in maximum dimension (oversized materials). Asphaltic and concrete debris generated during site demolition or encountered within the existing fill can be incorporated within new fill soils during earthwork operations provided they are reduced to no more than 4 inches in maximum dimension. 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 at least 100 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.

6.1.7 Import Materials

If import materials are required to achieve the proposed finish grades, the import soils should have an Expansion Index (EI) less than 21 (ASTM D 4829) and negligible soluble sulfate content. Import sources should be indicated to the geotechnical consultant at least 3 days prior to hauling the materials to the site so that appropriate testing and evaluation of the fill materials can be performed in advance.

6.2 SITE SPECIFIC SEISMIC DESIGN PARAMETERS

6.2.1 Mapped Seismic Design Parameters

For design of the project in accordance with Chapter 16 of the 2022 CBC, the mapped seismic parameters may be taken as presented in the tables below.

TABLE 6.1
2022 CBC Mapped Seismic Design Parameters

Parameter	Value
Site Class	D
Mapped MCE_R Spectral Response Acceleration, short periods, S_S	1.374
Mapped MCE_R Spectral Response Acceleration, at 1-second period, S_1	0.488
Site Coefficient, F_a	1.0
Site Coefficient, F_v *	1.712
Adjusted MCE_R Spectral Response Acceleration, short periods, S_{MS}	1.374
Adjusted MCE_R Spectral Response Acceleration, at 1-second period, S_{M1} *	1.253
Design Spectral Response Acceleration, short periods, S_{DS}	0.916
Design Spectral Response Acceleration, at 1-second period, S_{D1} *	0.835
Long-Period Transition Period, T_L (sec.)	8
Seismic Design Category for Risk Categories I-IV	II

MCE_R = Risk-Targeted Maximum Considered Earthquake

*According to Section 11.4.8 in ASCE 7-16 and supplement 3, “a ground motion hazard analysis shall be performed in accordance with Section 21.2 for the following structures on Site Class D site with S_1 greater than or equal to 0.2.” However, “A ground motion hazard analysis is not required for structures where the value of the parameter S_{M1} determined by Eq. (11.4-2) is increased by 50% for all applications of S_{M1} in the Standard. The resulting value of the parameter S_{D1} determined by Eq. (11.4-4) shall be used for all applications of S_{D1} in this Standard.” Should this exception not be met, the site-specific seismic design parameters provided in the next section should be used.

6.2.2 Site-Specific Seismic Design Parameters

In addition to the Code Spectra parameters presented in Table 6.1, we have performed a site-specific ground motion hazard analysis in accordance with Chapter 21 of ASCE 7-16 to obtain site-specific seismic design acceleration parameters, the risk-targeted maximum considered earthquake response spectrum, and the design earthquake response spectrum. The site-specific seismic design parameters are presented below.

TABLE 6.2
2022 CBC Site-Specific Seismic Design Parameters

Parameter	Value
Site Class	D
Site Coefficient, F_a	1.0
Site Coefficient, F_v	2.5
Adjusted MCE Spectral Response Acceleration, short periods, S_{MS}	1.683
Adjusted MCE Spectral Response Acceleration, at 1-second period, S_{M1}	1.488
Design Spectral Response Acceleration, short periods, S_{DS}	1.122
Design Spectral Response Acceleration, at 1-second period, S_{D1}	0.992

MCE = Maximum Considered Earthquake

6.3 FOUNDATION DESIGN

6.3.1 General

The following design parameters are provided to assist the project structural engineer to design foundation systems to support the proposed structures at the site. Recommendations for design of other foundation systems will be provided upon request. These design parameters are based on typical site materials encountered during subsurface exploration and are provided for preliminary design and estimating purposes. Depending on actual materials encountered during site grading and actual foundation loads, the design parameters presented herein may require modification.

The site is subject to the adverse effects of liquefaction. The site may be subject to seismic settlement that will require additional mitigation through design considerations in the foundation and slab on grade systems. We assume the foundation system will consist of either shallow foundations that are tied together with grade beams or a post-tension slab system. Therefore, the provision of ASCE 7-16, Section 12.13.9.2 should be incorporated into the design in addition to other recommendations provided below. Where the requirements of ASCE 7-16 are greater, the provisions of ASCE 7-16 should govern.

6.3.2 Soil Expansion

The recommendations presented herein are based on soils with a **Very Low** expansion potential (EI < 21). Following site grading, additional testing of site soils should be performed by the project geotechnical consultant to confirm the basis of these recommendations. If site soils with higher expansion potentials are encountered or imported to the site, the recommendations contained herein may require modification.

6.3.3 Settlement

Under normal static conditions, the foundation system should be designed to tolerate a total settlement of 1 inch and a differential settlement of ½-inch over 30 feet. The foundations should also be designed for total and differential seismic settlement of 5.5 inches and 2.75 inches over 30 feet, respectively. Requirements for conventional shallow footings should incorporate the requirements of Section 12.13.9 of ASCE 7-16. The PTI design parameters presented in Section 6.3.7 incorporate the estimated seismic settlements.

6.3.4 Allowable Bearing Value

Provided site grading is performed as recommended herein, a bearing value of 2,000 pounds per square foot (psf) may be used for continuous beams or isolated pad footings. The bearing value is based on beams having a minimum width of 12 inches and founded at a minimum of 12 inches below the lowest adjacent grade. The above value may be increased by 250 psf and 1000 psf for each additional foot in width and depth, respectively, up to a maximum value of 3,000 psf. Recommended allowable bearing values include both dead and live loads and may be increased by one-third for wind and seismic forces.

6.3.5 Lateral Resistance

Provided site grading is performed in accordance with the recommendations provided by the project geotechnical consultant, a passive earth pressure of 330 pounds per square foot per foot of depth up to a maximum value of 1,000 pounds per square foot may be used to determine lateral bearing for beams. This value may be increased by one-third when designing for wind and seismic forces. A coefficient of friction of 0.37 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. Where lateral removals cannot be performed beyond the foundation, the passive resistance values should be decreased by 25% such as for design of property line walls.

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

6.3.6 Shallow Footings and Slabs on Grade

Exterior and interior building footings should be founded at a minimum depth of 12 inches and 12 inches, respectively, below the lowest adjacent grade. All continuous footings should be reinforced with a minimum of two No. 4 bars, one top and one bottom. The structural engineer may require different reinforcement and should dictate if greater than the recommendations provided herein.

Interior isolated pad footings should be a minimum of 24 inches square and founded at minimum depths of 12 inches below the lowest adjacent final grade. Exterior isolated pad footings intended for support of patio covers or similar construction should be a minimum of 24 inches square and founded at a minimum depth of 12 inches below the lowest adjacent final grade. Isolated footings should be tied to nearby continuous footings in both directions with grade beams in accordance with the requirements of ASCE 7-16, Section 12.13.9.

Interior concrete slabs constructed on grade should be a minimum 4 inches thick and should be reinforced with a minimum of No. 3 bars spaced 30 inches on center, each way. Care should be taken to ensure the placement of reinforcement at mid-slab height. The structural engineer may recommend a greater slab thickness and reinforcement based on proposed use and loading conditions and such recommendations should govern if greater than the recommendations presented herein. Reinforcement and stiffening beams should be provided as required by ASCE 7-16, Section 12.13.9 for effects of liquefaction settlement.

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 1745-11, Class A. The membrane should be properly lapped, sealed, and underlain with at least 2 inches of sand having a SE no less than 30. One inch of this sand may be placed over the membrane to aid in the curing of the concrete. 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.

Special consideration should be given to slabs in areas to receive ceramic tile or other rigid, crack-sensitive floor coverings. Design and construction of such areas should mitigate hairline cracking as recommended by the structural engineer.

6.3.7 Post-Tensioned Slab on Grade

Due to potential seismic settlement, the proposed structures may be supported by a post-tension slab. 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 12 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 125 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.

$K_b(pci) = Kv1\left\{\frac{b+1}{2b}\right\}^2$; 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 2 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. If a post tension mat is used and is at least 8 inches in thickness, sand below the slab may be omitted.

Prior to placing concrete, subgrade soils below slab-on-grade/mat areas should be thoroughly moistened to provide moisture contents 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 in Table 6.3. These values already consider differential settlement from liquefiable soils.

TABLE 6.3
PTI Design Parameters

Parameter	Value
Edge Lift Moisture Variation Distance, e_m	5.1 feet
Edge Lift, y_m	1.66 inches
Center Lift Moisture Variation Distance, e_m	9.0 feet
Center Lift, y_m	1.10 inches

6.3.8 Foundation Observations

Foundation 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 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 AND SCREENING 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 site development should be provided to the project geotechnical consultant 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.

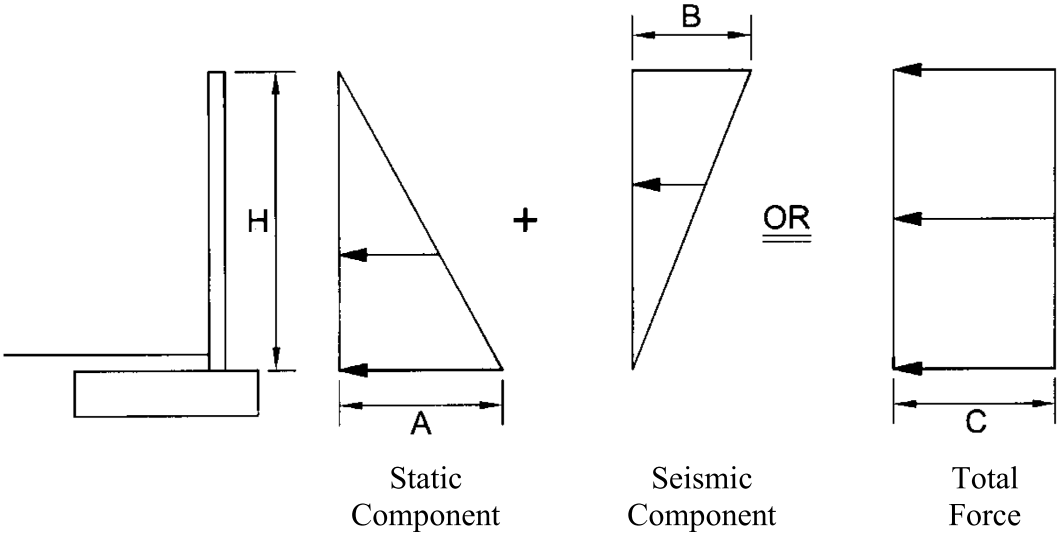
6.4.2 Allowable Bearing Value and Lateral Resistance

Design of retaining and screen walls may utilize the bearing and lateral resistance values provided in Section 6.3.4 and 6.3.5. The passive earth pressure for walls along property lines, where lateral removals are likely restricted, should be reduced by 25%.

6.4.3 Earth Pressures

Static and seismic earth pressures for level and 2:1 (H:V) backfill conditions are provided in Table 6.4. Seismic earth pressures provided herein are based on the method provided by Seed & Whitman (1970) using a peak ground acceleration (PGA) of 0.39 g for 10% probability of exceedance in 50 years. As indicated in Section 1807.2.2of the 2022 CBC, retaining walls supporting 6 feet of backfill or less are not required to be designed for seismic earth pressures. The values provided in the following table do not consider hydrostatic pressure. Retaining walls should also be designed to support adjacent surcharge loads imposed by other nearby footings or traffic loads in addition to the earth pressure.

TABLE 6.4
SEISMIC EARTH PRESSURES
Pressure Diagram



Earth Pressure Values
Walls Up to 10 Feet in Height

Value	Backfill Condition	
	Level	**2H:1V Slope**
A	33H	54H
B	12H	12H
C	22H	33H

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 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 pipes 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 the wall that will be in contact with soil and should lap over and onto the top of the 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 smoothly 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.

6.4.5 Footing Reinforcement

All continuous footings should be reinforced with a minimum of four No. 4 bars, two top and two 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 warranted. Such specific recommendations should be provided by our firm during grading as-built conditions observed in the field.

6.4.6 Wall Jointing

All free-standing, exterior site walls should be provided with cold joints through the masonry block section at horizontal spacing generally not exceeding 40 feet. The joints should not extend through the footing. Retaining walls that are integral to the building should be provided joints based on recommendations by the structural engineer.

6.4.7 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.

6.4.8 Wall Backfill

Onsite soils may be used for backfill behind retaining walls. 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 to the wall to avoid damage to the wall.

6.5 EXTERIOR FLATWORK

Concrete sidewalks, patios, and similar flatwork should be a nominal 4 inches thick and provided with saw cuts or expansion joints at spacing no greater than 10 feet in each direction. 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 100 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 0.5% away from building foundations and masonry walls.

6.6 CONCRETE MIX DESIGN AND CORROSION

Laboratory testing of existing near-surface soils for soluble sulfate content indicates soluble sulfate concentration less than 0.15%. We recommend following the procedures provided in ACI 318, Section 4.3, Table 4.3.1 for **negligible** sulfate exposure. Upon completion of rough grading, an evaluation of as-graded conditions and further laboratory testing should be completed for the site to confirm or modify the recommendations provided in this section.

Laboratory testing of onsite soil indicates a minimum resistivity of 4,700 ohm-cm, chloride content of 25 ppm, and a pH of 8.3. Based on laboratory test results, site soils are **Moderately Corrosive** to metals. Structures fabricated from metals should have appropriate corrosion protection if they are in direct contact with site soils. Under such conditions, a corrosion specialist should provide specific recommendations.

6.7 POST GRADING CONSIDERATIONS

6.7.1 Site Drainage and Irrigation

Positive drainage devices, such as sloping concrete flatwork, graded swales or area drains, should be provided around the new construction to collect and direct all surface water to suitable discharge areas. In general, the site should be graded to conform to the requirements of Section 1804.4 of the 2022 California Building Code. However, the slope of the ground surface may be reduced to a maximum of 2% based on soils and environmental conditions of the site. No rain or excess water should be directed toward or 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.

6.7.2 Utility Trenches

Trench excavations should be constructed in accordance with the recommendations contained in Section 6.1.5 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. Trench backfill should be brought to moisture content slightly over optimum, 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. The project geotechnical consultant should perform density testing, along with probing, to test compaction. Jetting should not be completed without prior approval from the project geotechnical consultant.

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.

6.8 PRELIMINARY PAVEMENT DESIGN

6.8.1 Preliminary Structural Sections

Based on the soil conditions present at the site and estimated traffic indices, preliminary pavement sections are provided in Table 6.5 below. A preliminary "R-value" of 25 was used for the near-surface soil in this preliminary pavement design. 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. It should be noted that the preliminary paving sections provided below are considered suitable for support of a 94,000 pound fire apparatus.

TABLE 6.5
PRELIMINARY PAVEMENT STRUCTURAL SECTIONS FOR RESIDENTIAL DEVELOPMENT

Location	Traffic Index	AC (inches)	Paver Thickness (mm)	Portland Cement Concrete (inches)	AB (inches)
Driveway	5.0	3.0 4.0	--	--	7.0 4.0
		--	80	--	8.0
		--	--	6.0	--
Parking Stalls	--	3.0	--	---	4.0

6.8.1 Subgrade Preparation

Prior to placement of pavement elements, subgrade soils should be moisture-conditioned to at least 110 percent of the optimum moisture content then compacted to at least 90 percent of the laboratory determined maximum dry density. Areas observed to pump or yield under vehicle traffic should be removed and replaced with firm and unyielding 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

Paving asphalt should be PG 64-10. Asphaltic concrete materials should conform to Section 203-6 of the Greenbook and construction should conform to Section 302 of the Greenbook. Where traffic will traverse over cold joints in asphaltic concrete such as against concrete ribbon gutters and concrete paver sections, the asphaltic concrete section should be thickened by 1 additional inch from the values indicated in the above Table 6.5 within 2 feet of cold joints.

6.8.4 Concrete Pavers

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 6.6.

TABLE 6.6
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

Construction of edge restraints should also follow manufacturer’s specifications. As a minimum, restraints should be provided along the perimeter of concrete pavers and where there is a change in the paving materials. The proposed concrete bands should extend to the bottom of the base course underlying the concrete pavers. Portland cement concrete used to construct concrete bands should conform to Section 201 of the Greenbook and should have a minimum compressive strength of 2,500 pounds per square inch (psi) at 28 days. Reinforcement and jointing of concrete pavement sections should be designed according to the minimum recommendations provided by the Portland Cement Association (PCA). For rigid pavement, transverse and longitudinal contraction joints should be provided at spacing no greater than 15 feet. Score joints may be constructed by saw cutting to a depth of ¼ of the slab thickness. Expansion/cold joints may be used in lieu of score joints. However, cold joints should be provided with dowels or keyways are recommended by PCA.

6.8.5 Portland Cement Concrete (PCC)

Portland cement concrete used to construct concrete paving should conform to Section 201 of the Greenbook and should have a minimum compressive strength of 3,000 pounds per square inch (psi) at 28 days. Reinforcement and jointing of concrete pavement sections should be designed according to the minimum recommendations provided by the Portland Cement Association (PCA). For rigid pavement, transverse and longitudinal contraction joints should be provided at spacing no greater than 15 feet. Score joints may be constructed by saw cutting to a depth of ¼ of the slab thickness. Expansion/cold joints may be used in lieu of score joints. Such joints should be properly sealed. Where traffic will traverse over cold joints or edges of concrete paving, the edges should be thickened by 20% of the design thickness toward the edge over a horizontal distance of 5 feet.

Trash pickup areas should be provided with a concrete slab where the bins will be picked up and extend at least 3 feet past the front wheel landing areas. The slab should be at least 8 inches thick and be reinforced with No. 4 bars spaced at 24 inches on centers, both ways. The slabs should be provided transverse and longitudinal joints spacing as specified above. Dowels or a keyway should be provided at all cold joints.

6.9 PLAN REVIEW AND CONSTRUCTION SERVICES

We recommend ***Albus & Associates, Inc.*** be engaged to review any future development plans, including revisions to the grading plans, foundation plans and proposed structural loads, 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 described herein and in other literature are believed representative of the total project area, and the conclusions contained in this report are presented on that basis. However, soil 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 prior to and during the grading and construction phases of the project are essential to confirming the basis of this report.

This report summarizes several geotechnical topics that should be beneficial for project planning and budgetary evaluations. The information presented herein is intended only for a preliminary feasibility evaluation and is not intended to satisfy the requirements of a site specific and detailed geotechnical investigation required for further planning and permitting.

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 **Melia Homes** to assist the project consultants in determining the feasibility 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.

Respectfully submitted,

ALBUS & ASSOCIATES, INC

Daniel Albus
Project Engineer

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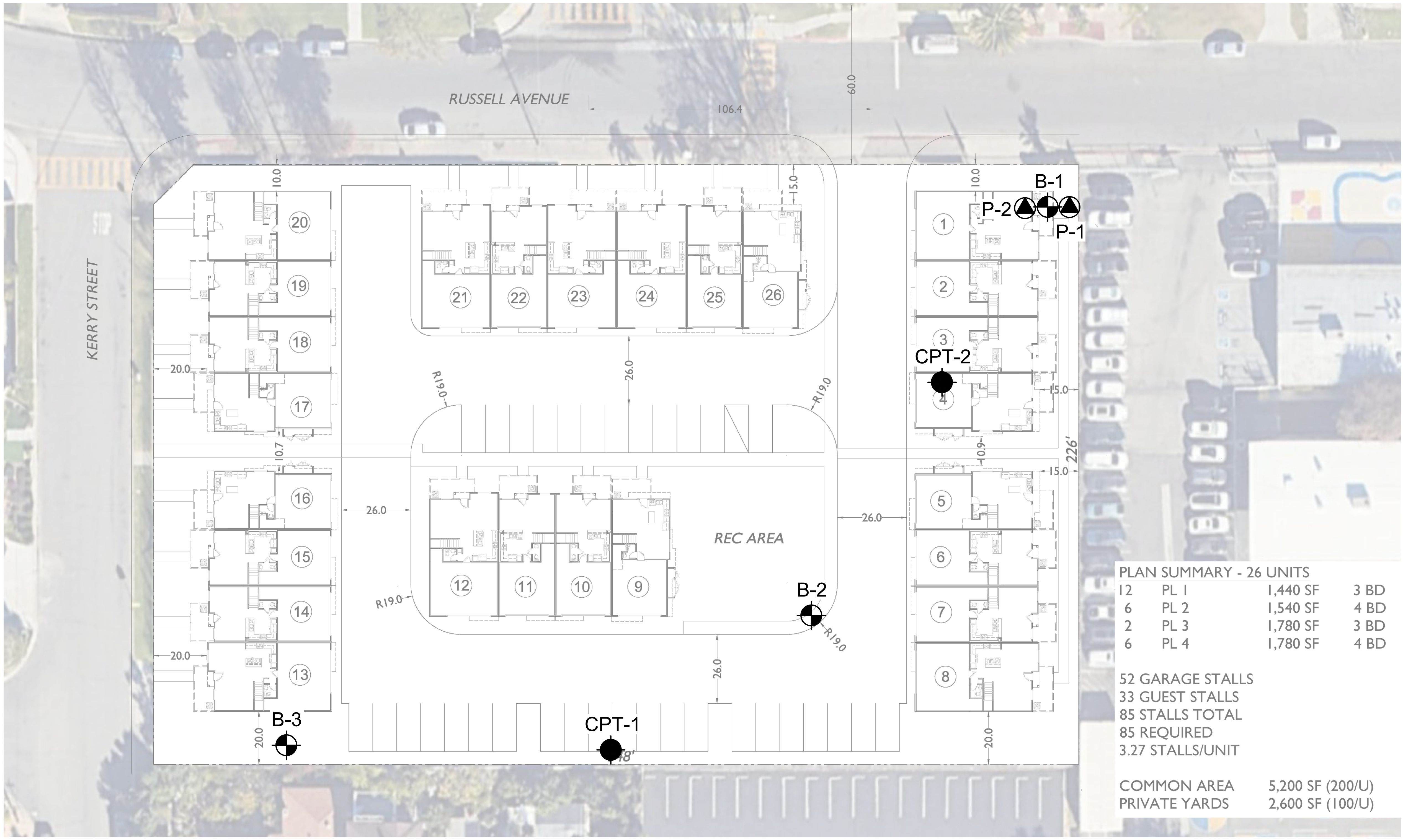

PLAN SUMMARY - 26 UNITS

12	PL 1	1,440 SF	3 BD
6	PL 2	1,540 SF	4 BD
2	PL 3	1,780 SF	3 BD
6	PL 4	1,780 SF	4 BD

52 GARAGE STALLS
33 GUEST STALLS
85 STALLS TOTAL
85 REQUIRED
3.27 STALLS/UNIT

COMMON AREA	5,200 SF (200/U)
PRIVATE YARDS	2,600 SF (100/U)

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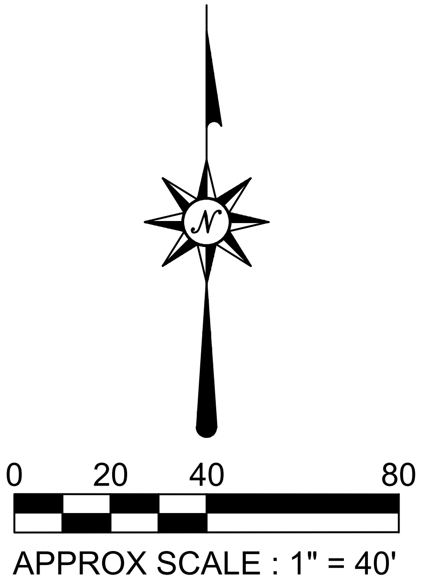


APPROX SCALE : 1" = 40'

EXPLANATION
(Locations Approximate)

- ⊕ - Exploratory Boring
- ▲ - Percolation Testing
- ● - CPT

Base Provided By Summa Architecture

ALBUS & ASSOCIATES

GEOTECHNICAL MAP

Job No.: 3341.00 | Date: 06/18/2025 | Plate: 1

APPENDIX A

EXPLORATION LOGS

Field Identification Sheet

- Light gray
- Gray
- Dark gray
- Gray Brown
- Light brown
- Brown
- Dark Brown
- Olive brown
- Olive
- Yellow
- Yellowish brown
- Yellowish red
- Red
- Reddish Brown
- Tan

Description Order:

Description, Color, Moisture, Density, Grain Size, Additional Description

Description	%	Example
	0-5	Sand
trace	5-15	Sand trace Silt
with	15-30	Sand with Silt
	30+	Silty Sand

More Examples

Sand with Silt trace Clay

Sand trace Silt and Clay

Sand with Silt and Clay

Gravelly Sand with Silt trace Clay

Silty Clay with Sand trace Gravel

Moisture

Dry	absence of water
Damp	below optimum
Moist	near optimum
Very Moist	above optimum
Wet	free water visible

Density (Navfac)

Coarse grained soils	SPT	CA
Very Loose	0-3	0-5
Loose	3-8	5-13
Medium Dense	8-14	13-22
Dense	14-25	22-40
Very Dense	25>	40>

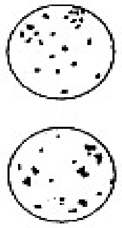
Fine grained soils		
Very Soft	2<	0-3
Soft	2-4	3-6
Medium Stiff	4-8	6-13
Stiff	8-15	13-24
Very Stiff	15-30	24-48
Hard	30>	48>

Grain Size

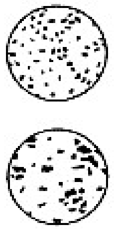
Description		Sieve Size	Approx. Size
Boulders		>12"	Larger than basketball
Cobbles		3-12"	Fist to basketball
Gravel	coarse	3/4-3"	Thumb to Fist
	fine	#4-3/4"	Pea to Thumb
Sand	coarse	#10-4	Rock Salt to Pea
	medium	#40-10	Sugar to Rock Salt
	fine	#200-40	Flour to Sugar
Fines		Pass #200	Smaller than Flour

Additional Description (ie. roots, pinhole pores, debris, etc.)

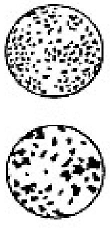
Trace 5%



Moderate 15%



Abundant 30%



EXPLORATION LOG

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

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

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

5
10
15
20

Albus & Associates, Inc.

ALBUS & ASSOCIATES

EXPLORATION LOG B-1

JOB NO.	CLIENT/PROJECT	DAY	DATE
3341.00	Melia Homes	Tuesday	2025-06-10

LOCATION	LATITUDE	LONGITUDE	ELEVATION
9822 Russell Avenue, Garden Grove	33.76847	-117.95723	70.9

LOGGED BY	DRILLER	DRILL METHOD	DRIVING WEIGHT
klopez	2R Drilling	Hollow-Stem Auger	140 lbs / 30 in

DEPTH	LITHO	DESCRIPTION	H2O	COR	BAG	BLOW COUNT	MC (%)	DD (pcf)	LAB
		Asphalt							max ei so4 sa ph resist ch
1		**Artificial Fill (Af)** Sand with Silt (SP-SM): tan, damp, medium grained							
2				▨		20	3	96	
3		**Alluvium (Qal)** Sand (SP): tan, damp, medium dense, fine to medium grained							
4				▨		20	2.4	97.7	
5									
6		Sand trace Silt (SP): gray brown, very moist, loose, fine to medium grained		▨		6	7.6	93.1	200
7									
8									
9									
10		Sandy Silt (ML): gray brown, very moist, very loose, fine to medium grained		▨		5			
11									
12									
13									
14			▼						
15		@ 15 ft, wet, loose		⧗		3			200
16									
17									
18									
19									
20		Silty Sand / Sandy Silt (SM / ML): gray, wet, medium dense / very stiff, fine grained		⧗		14			200
21									
22									
23									
24									
25		Silty Sand (SM): gray, wet, loose, fine grained		⧗		6	25.5		att
26									
27									
28									
29									

EXPLORATION LOG B-1

JOB NO.	CLIENT/PROJECT	DAY	DATE
3341.00	Melia Homes	Tuesday	2025-06-10

LOCATION	LATITUDE	LONGITUDE	ELEVATION
9822 Russell Avenue, Garden Grove	33.76847	-117.95723	70.9

LOGGED BY	DRILLER	DRILL METHOD	DRIVING WEIGHT
klopez	2R Drilling	Hollow-Stem Auger	140 lbs / 30 in

DEPTH	LITHO	DESCRIPTION	H2O	COR	BAG	BLOW COUNT	MC (%)	DD (pcf)	LAB
		Silty Sand / Sandy Silt (SM / ML): gray, wet, loose / stiff, fine grained		⧗		6			
31									
32									
33									
34									
35		@ 35 ft, medium dense		⧗		9			200
36									
37									
38									
39									
40				⧗		8			
41									
42									
43									
44									
45		Silty Sand with Clay (SM-SC): gray, wet, medium dense, fine grained		⧗		11	27.8		att
46									
47									
48									
49									
50				⧗		8	34.3		
51									
52		Total Depth 51.5 feet Groundwater at 14 feet							
53									
54									
55									
56									
57									
58									
59									

EXPLORATION LOG B-2

JOB NO.	CLIENT/PROJECT	DAY	DATE
3341.00	Melia Homes	Tuesday	2025-06-10

LOCATION	LATITUDE	LONGITUDE	ELEVATION
9822 Russell Avenue, Garden Grove	33.76805	-117.95752	70.7

LOGGED BY	DRILLER	DRILL METHOD	DRIVING WEIGHT
klopez	2R Drilling	Hollow-Stem Auger	140 lbs / 30 in

DEPTH	LITHO	DESCRIPTION	H2O	COR	BAG	BLOW COUNT	MC (%)	DD (pcf)	LAB
		Asphalt							
1		**Artificial Fill (Af)** Sand with Silt (SP-SM): light brown, damp, fine to medium grained							
2				▨		19	6.8	99.9	
3		**Alluvium (Qal)** Sand (SP): light brown, damp, medium dense, fine to medium grained							
4				▨		12	8.8	96.9	consol
5		@ 4 ft, gray brown, moist, loose							
6				▨		6	12.4	89.8	
7									
8									
9									
10		Sandy Silt trace Clay (ML): gray brown, very moist to wet, very loose, fine grained		▨		4	31.6	89.1	sa hydro
11		Total Depth 11.5 feet No Groundwater							
12									
13									
14									
15									
16									
17									
18									
19									
20									
21									
22									
23									
24									
25									
26									
27									
28									
29									

EXPLORATION LOG B-3

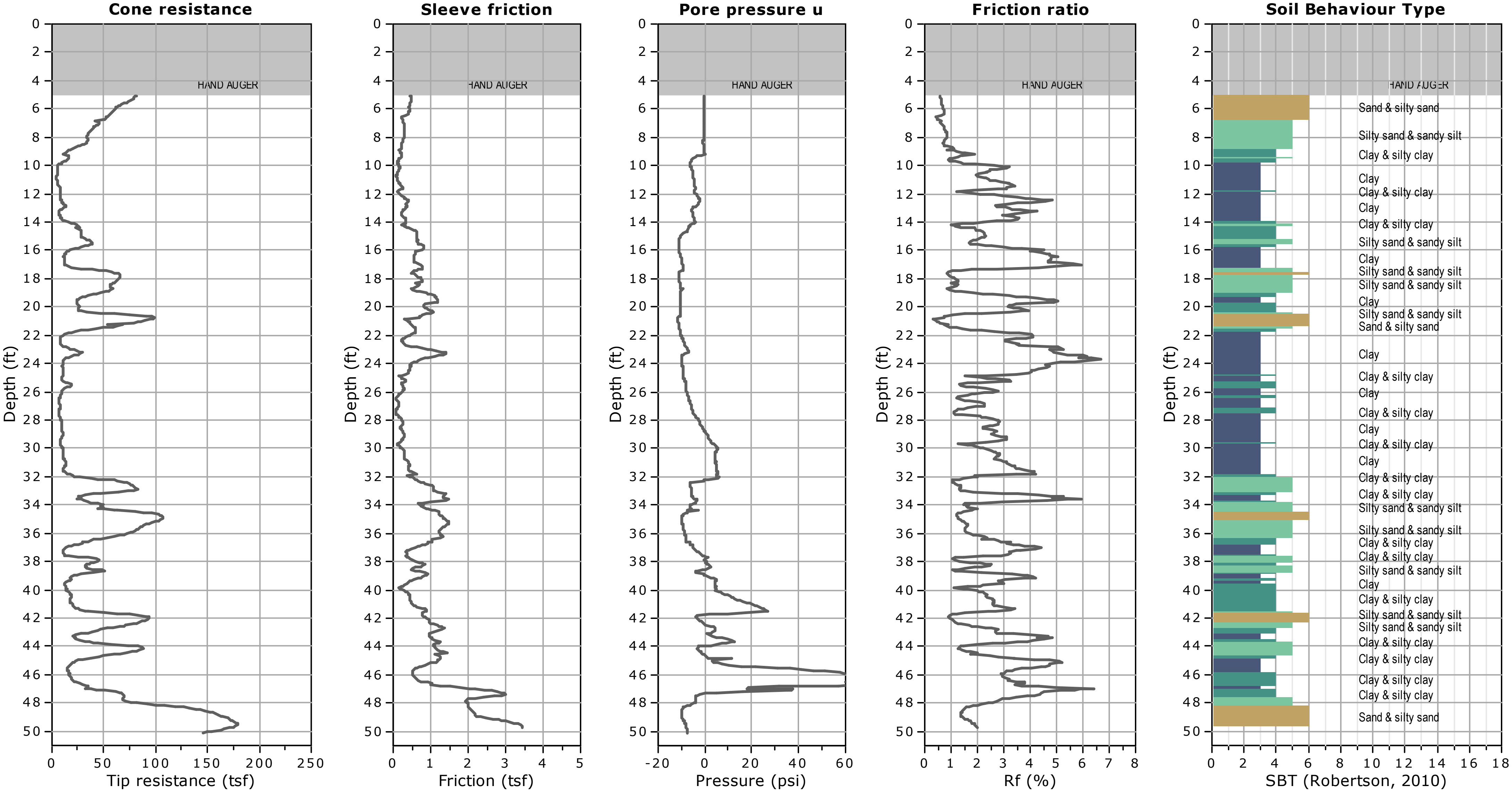
JOB NO. 3341.00
CLIENT/PROJECT Melia Homes
DAY Tuesday
DATE 2025-06-10
LOCATION 9822 Russell Avenue, Garden Grove
LATITUDE 33.76792
LONGITUDE -117.95817
ELEVATION 71.3
LOGGED BY klopez
DRILLER 2R Drilling
DRILL METHOD Hollow-Stem Auger
DRIVING WEIGHT 140 lbs / 30 in

DEPTH	LITHO	DESCRIPTION	H2O	COR	BAG	BLOW COUNT	MC (%)	DD (pcf)	LAB
		Asphalt							
1		**Artificial Fill (Af)** Sand with Silt (SP-SM): light brown, moist, fine to medium grained							
2				▨		13	4.7	100.4	
3		**Alluvium (Qal)** Sand (SP): brown, damp to moist, loose, fine to medium grained		▨					
4				▨		16	8.4	100.2	
5		@ 4 ft, medium dense		▨					
6		@ 6 ft, loose		▨		12	12.4	87.4	
7				▨					
8									
9									
10		Sandy Silt with Clay (ML): gray brown, moist, soft, fine grained		▨		4	28.5	93.2	
11				▨					
12		Total Depth 11.5 feet No Groundwater							
13									
14									
15									
16									
17									
18									
19									
20									
21									
22									
23									
24									
25									
26									
27									
28									
29									

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

Project: **Albus & Associates**
Location: **9822 Russell Avenue, Garden Grove, CA**

CPT-1
Total depth: 50.07 ft, Date: 6/10/2025



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

Project: **Albus & Associates**
Location: **9822 Russell Avenue, Garden Grove, CA**

CPT-2
Total depth: 50.07 ft, Date: 6/10/2025

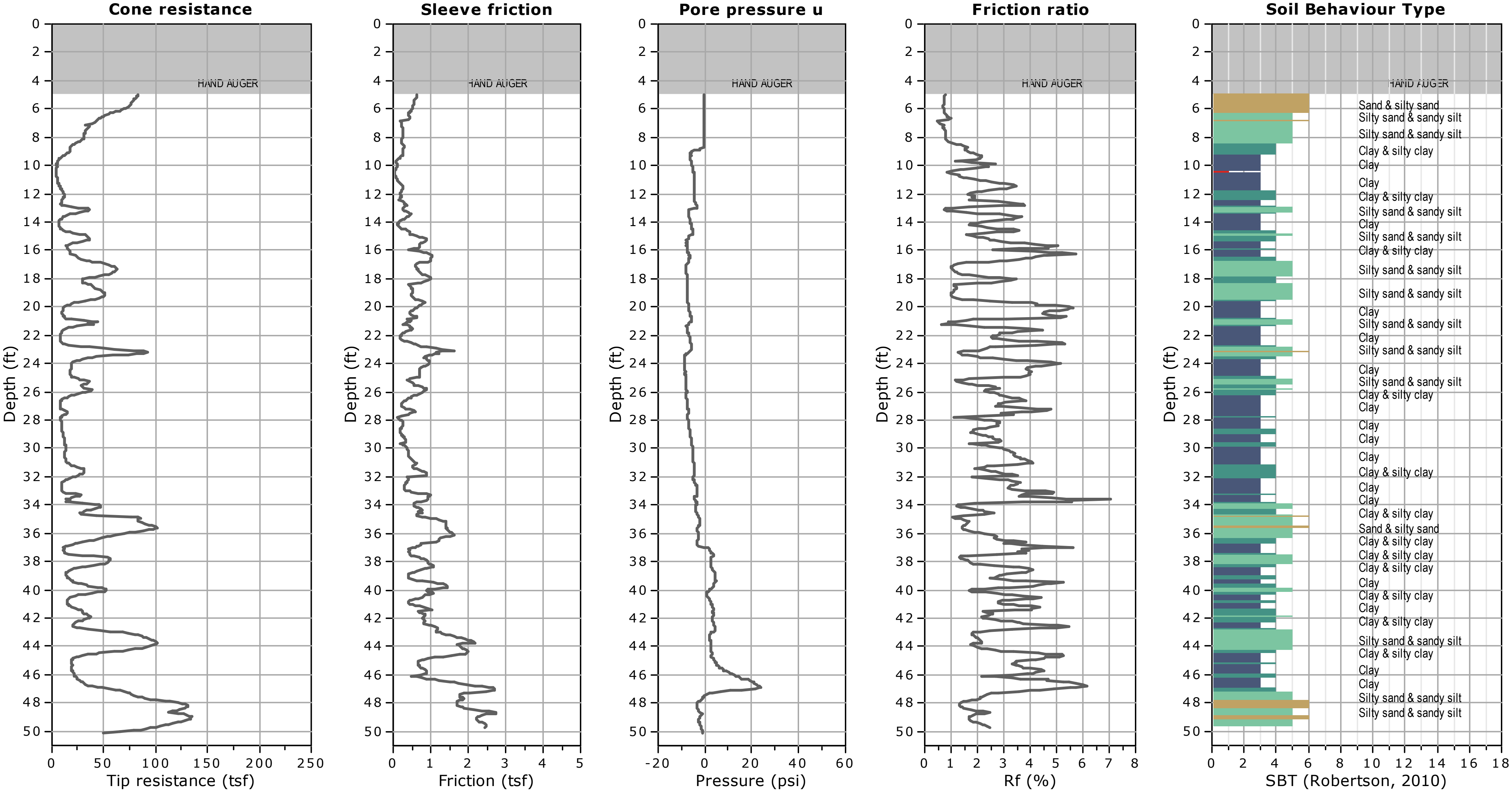


Plate A-7

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-1.

Expansion Potential

Expansion index testing was performed on a selected sample. The test was performed in accordance with ASTM D4829. The test result is presented on Table B-1.

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-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.

Particle-Size Analyses and Hydrometer

Particle-size analyses were performed on selected samples in accordance with ASTM D 422. 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. Results of the tests are graphically presented on Plate B-3.

Direct Shear

The Coulomb shear strength parameters, angle of internal friction and cohesion, were determined for a bulk sample obtained from one our borings. The tests were performed in general conformance with Test Method ASTM D 3080. The sample was remolded to 90 percent of maximum dry density and at the optimum moisture content. Three specimens were prepared for each test, artificially saturated, and then sheared under varied loads at an appropriate constant rate of strain. Results are graphically presented on Plate B-4.

Percent Passing the No. 200 Sieve

The percentage 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 D1140. Test results are presented on Table B-1.

Corrosion

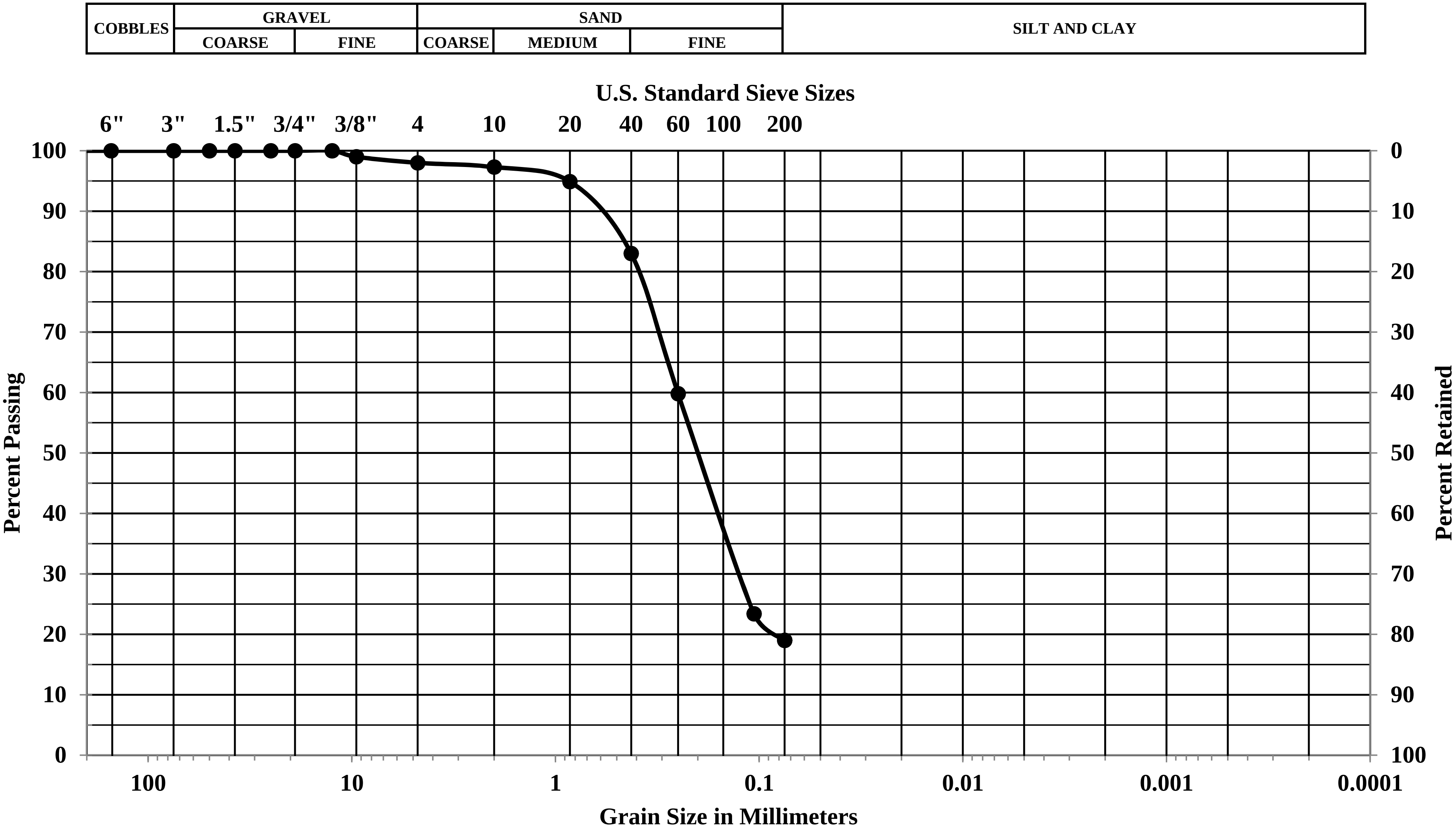
Select samples were tested for minimum resistivity, chloride, and pH in accordance with California Test Method 643. Results of these tests are provided in Table B-1.

TABLE B-1
SUMMARY OF LABORATORY TEST RESULTS

Boring No.	Sample Depth (ft.)	Soil Description	Test Results	
B-1	0-5	Sand with Silt (SP-M)	Maximum Dry Density (pcf):	121
			Optimum Moisture Content (%):	10
			Expansion Index:	0
			Expansion Potential:	Very Low
			Soluble Sulfate Content (%):	0.015
			Sulfate Exposure:	Negligible
			pH:	8.3
			Chloride (ppm):	25
			Minimum Resistivity (ohm-cm):	4700
B-1	6	Sand trace Silt (SP)	Passing No. 200 Sieve (%):	2.6
B-1	15	Sandy Silt (ML)	Passing No. 200 Sieve (%):	59.1
B-1	20	Silty Sand / Sandy Silt (SM/ML)	Passing No. 200 Sieve (%):	51
B-1	25	Silty Sand (SM)	Liquid Limit:	29
			Plastic Index:	5
B-1	35	Silty Sand / Sandy Silt (SM/ML)	Passing No. 200 Sieve (%):	45.5
B-1	45	Silty Sand with Clay (SM-SC)	Liquid Limit:	36
			Plastic Index:	12

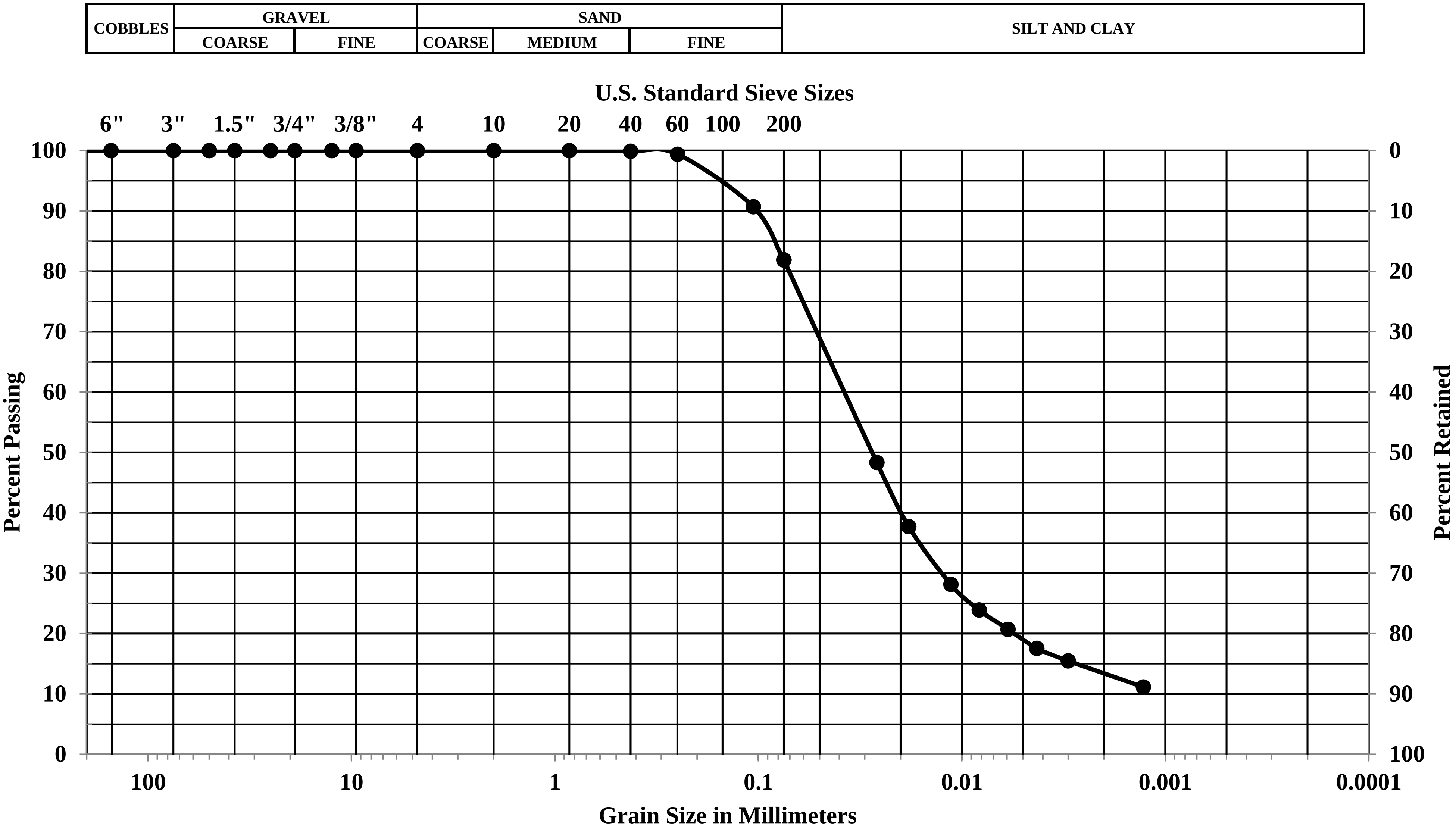
Note: Additional laboratory test results are provided on the boring logs provided in Appendix A.

GRAIN SIZE DISTRIBUTION



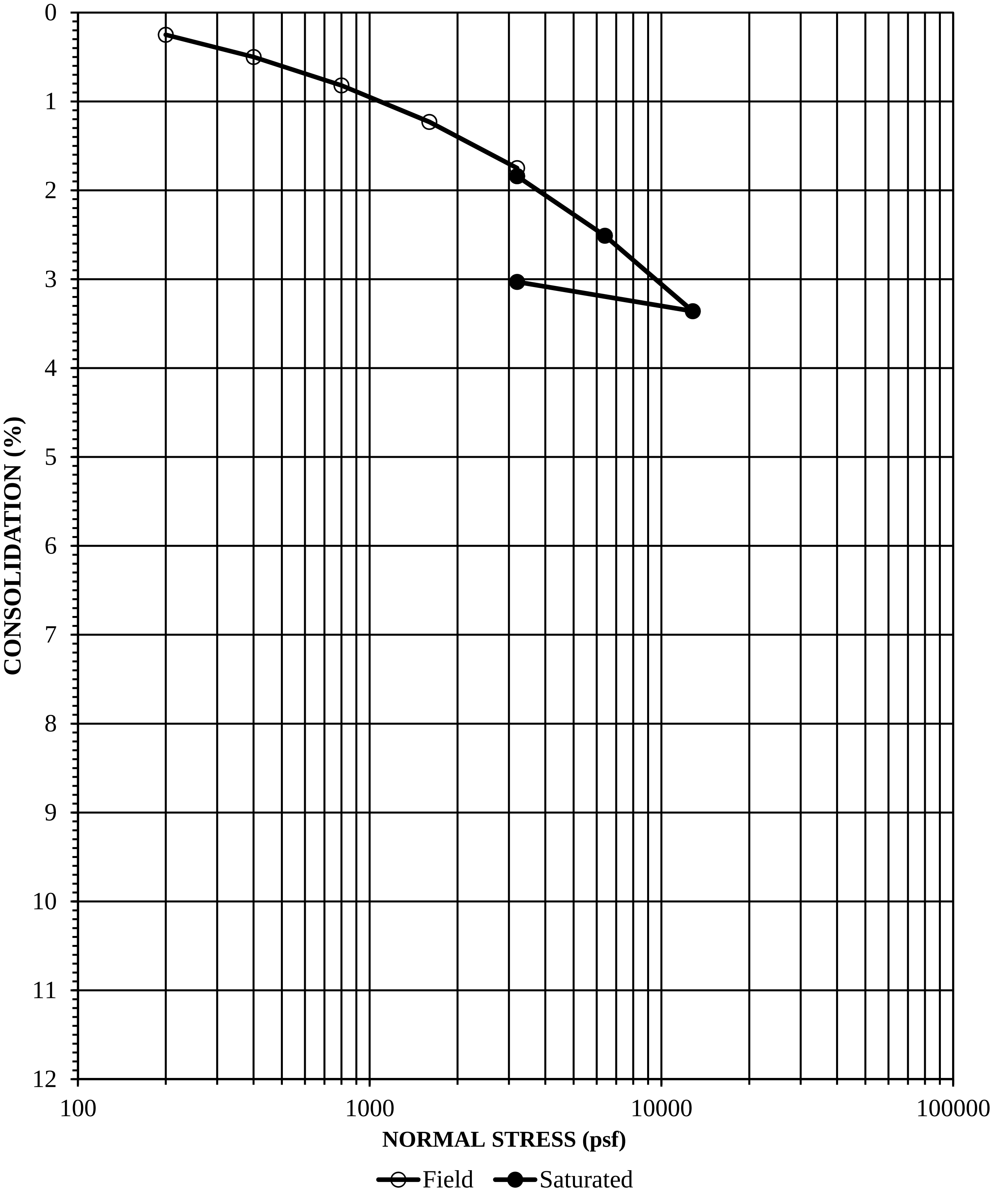
Job Number	Location	Depth	Description
3341.00	B-1	0-5	Sand with Silt (SP-SM)

GRAIN SIZE DISTRIBUTION



Job Number	Location	Depth	Description
3341.00	B-2	10	Silt trace Sand and Clay (ML)

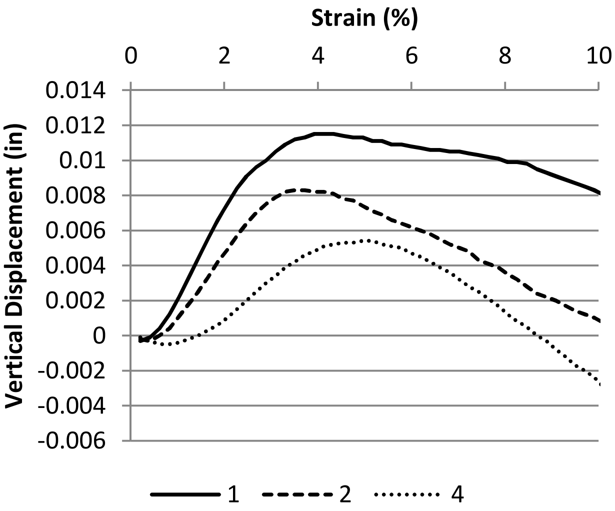
CONSOLIDATION

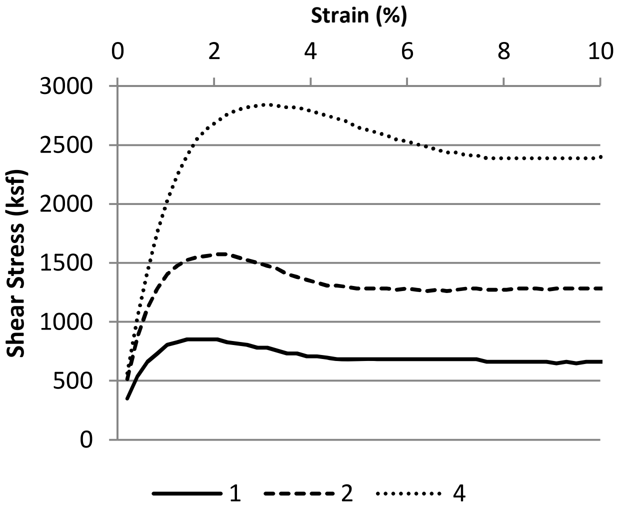


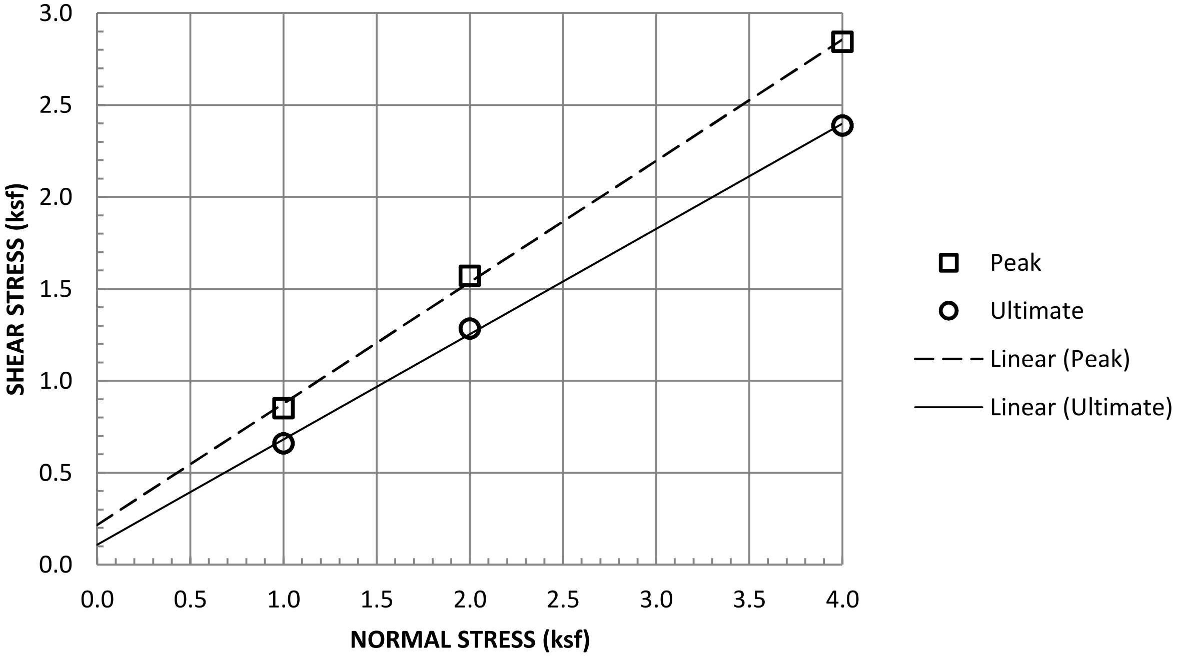
Job Number	Location	Depth	Description
3341.00	B-2	4	Silty Sand (SM)

Initial Dry Density (pcf)	Initial Moisture Content (%)	Final Moisture Concent (%)
96.0	10.4	17

DIRECT SHEAR







Sample Type:	Remolded to 90% of 121 pcf, Saturated		
Normal Stress (ksf)	1	2	4
Peak Shear Stress (ksf)	0.852	1.572	2.844
Peak Displacement (in)	0.012	0.008	0.005
Ultimate Shear Stress (ksf)	0.66	1.284	2.388
Ultimate Displacement (in)	0.25	0.25	0.25
Initial Dry Density (pcf)	108.7	108.7	108.7
Initial Moisture Content (%)	10	10	10
Final Moisture Content (%)	15.2	15.1	14.3
Strain Rate (in/min)	0.01		

Job Number	Location	Depth	Description
3341.00	B-1	0-5	Sand with Silt (SP-SM)

APPENDIX C

LIQUEFACTION ANALYSIS

TABLE OF CONTENTS

Kehoe Testing and Engineering
Albus & Associates, Inc.
1011 N. Armando St., Anaheim, CA
www.albusgeo.net

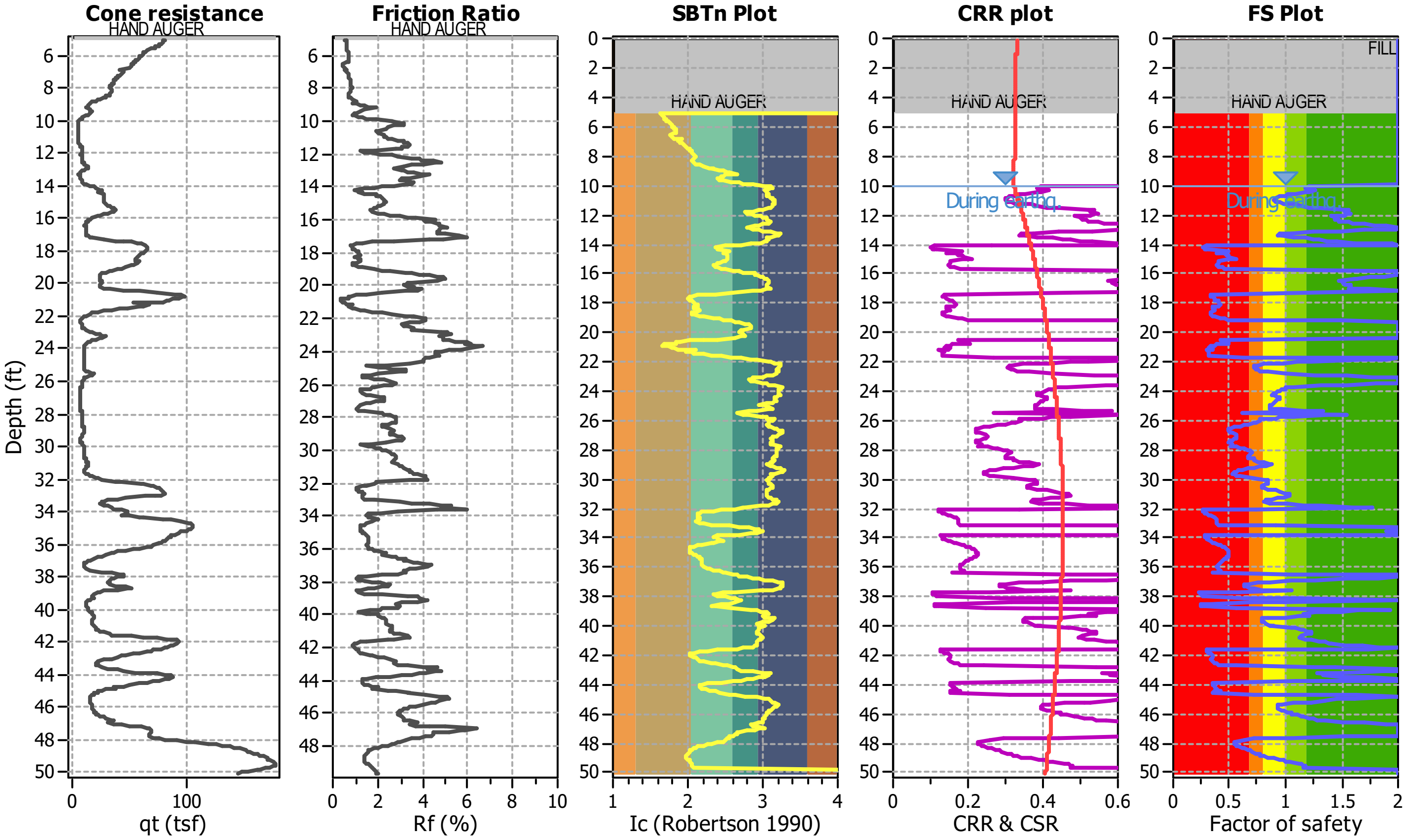
LIQUEFACTION ANALYSIS REPORT

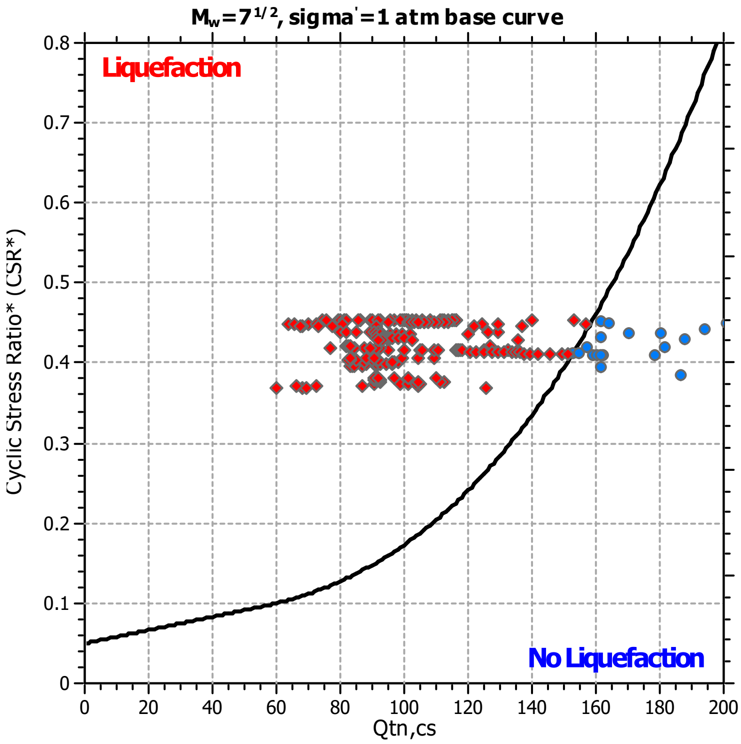
Project title : 3341.00 Melia Homes -- Garden Grove **Location : 9822 Russell Avenue, Garden Grove, CA**

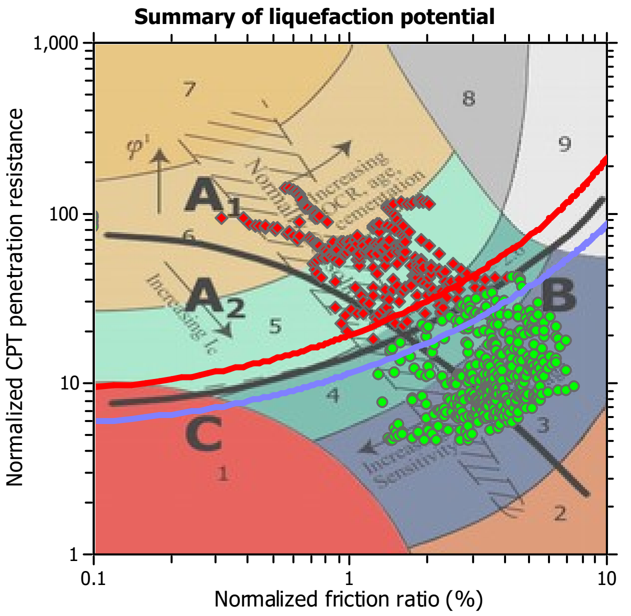
CPT file : CPT-1

Input parameters and analysis data

Analysis method:	Robertson (2009)	G.W.T. (in-situ):	14.00 ft	Use fill:	Yes	Clay like behavior applied:	All soils
Fines correction method:	Robertson (2009)	G.W.T. (earthq.):	13.00 ft	Fill height:	3.00 ft	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	1	Fill weight:	120.00 lb/ft³	Limit depth:	N/A
Earthquake magnitude M_w:	6.74	Ic cut-off value:	2.60	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.67	Unit weight calculation:	Based on SBT	K_σ applied:	No		

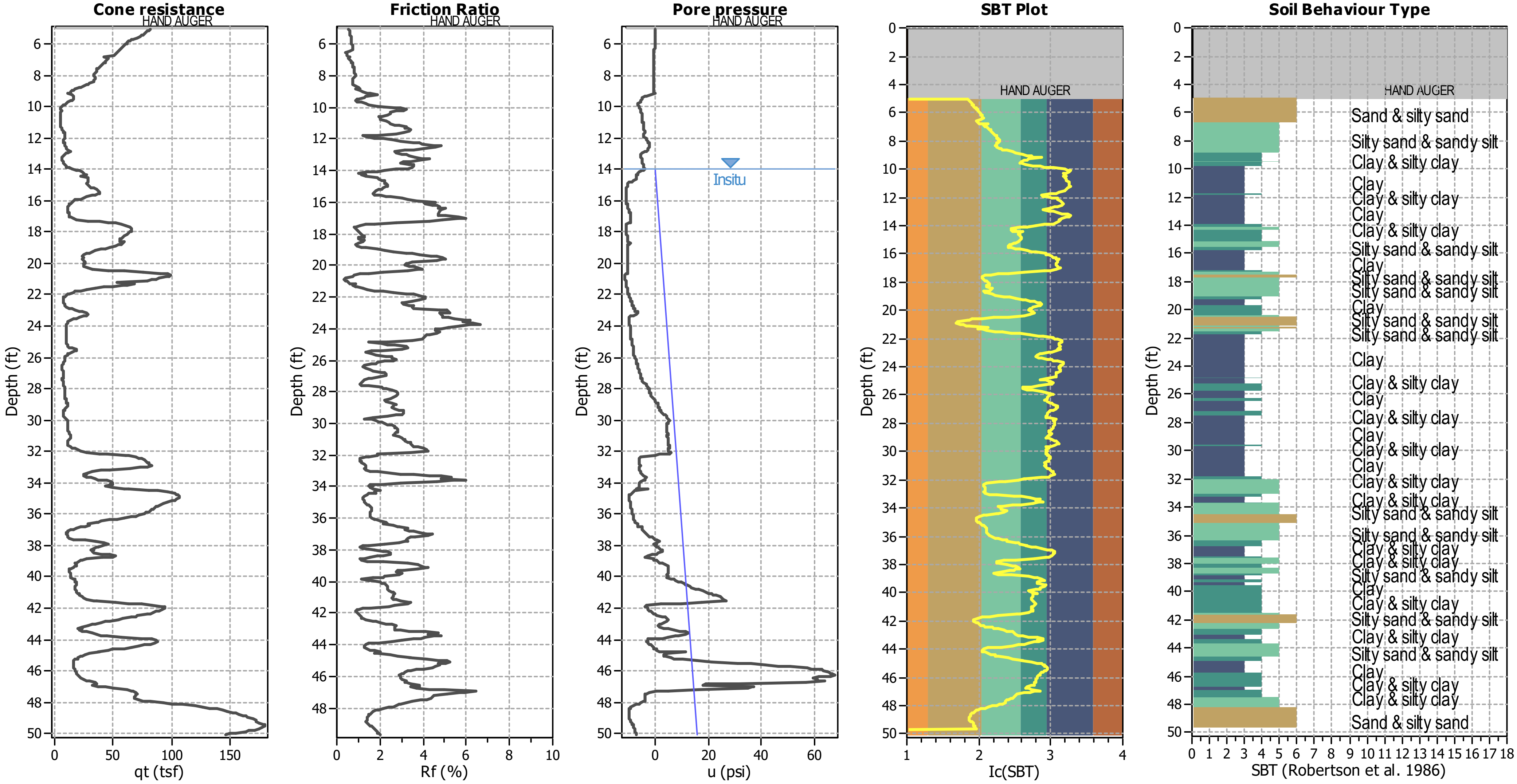






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and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

CPT basic interpretation plots



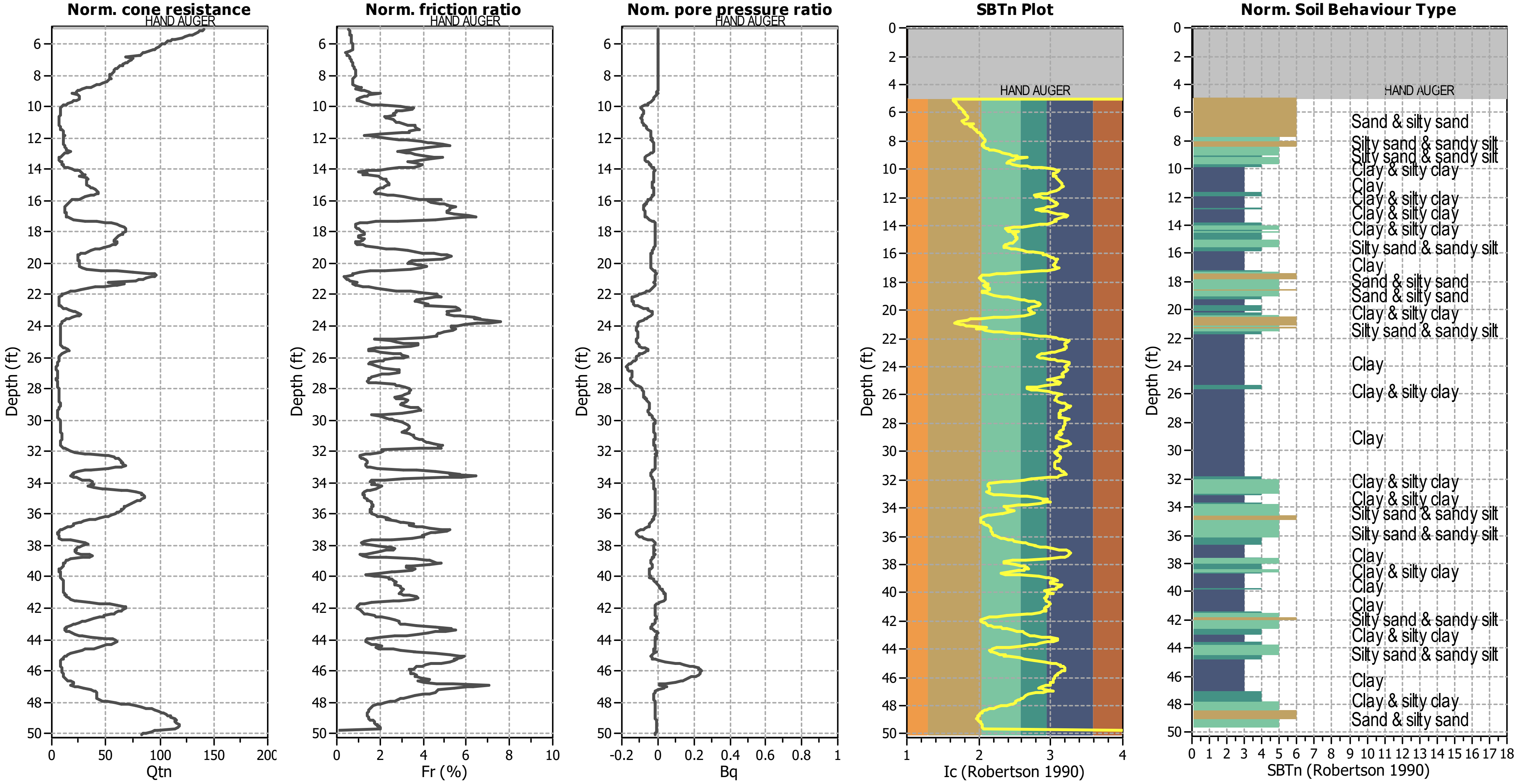
Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	13.00 ft	Fill weight:	120.00 lb/ft³
Fines correction method:	Robertson (2009)	Average results interval:	1	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	No
Earthquake magnitude M_w:	6.74	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.67	Use fill:	Yes	Limit depth applied:	No
Depth to water table (insitu):	14.00 ft	Fill height:	3.00 ft	Limit depth:	N/A

SBT legend

🟥 1. Sensitive fine grained	🟩 4. Clayey silt to silty	🟧 7. Gravely sand to sand
🟫 2. Organic material	🟩 5. Silty sand to sandy silt	⬛ 8. Very stiff sand to
🟦 3. Clay to silty clay	🟨 6. Clean sand to silty sand	⬜ 9. Very stiff fine grained

CPT basic interpretation plots (normalized)



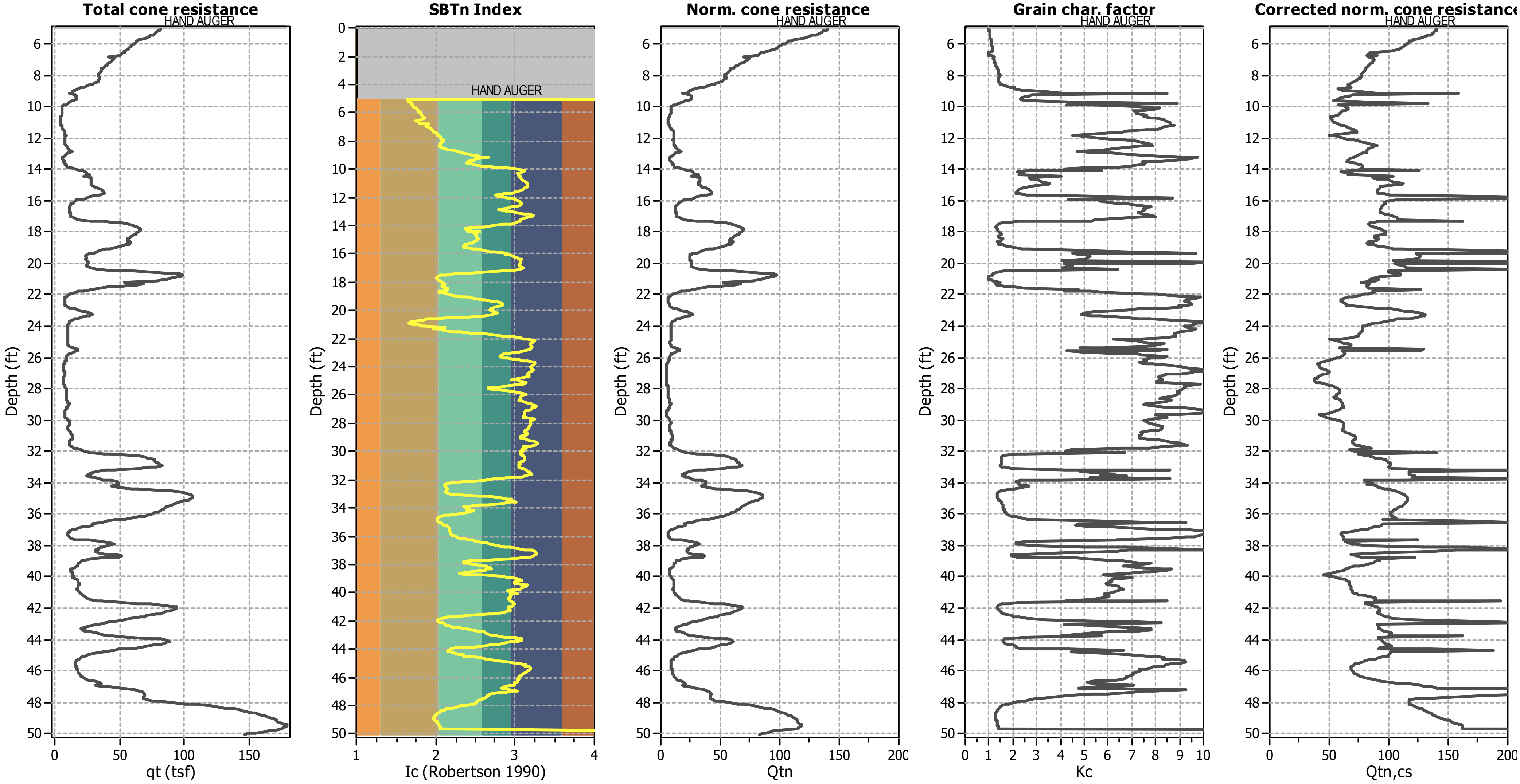
Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	13.00 ft	Fill weight:	120.00 lb/ft^3
Fines correction method:	Robertson (2009)	Average results interval:	1	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	No
Earthquake magnitude M_w:	6.74	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.67	Use fill:	Yes	Limit depth applied:	No
Depth to water table (insitu):	14.00 ft	Fill height:	3.00 ft	Limit depth:	N/A

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

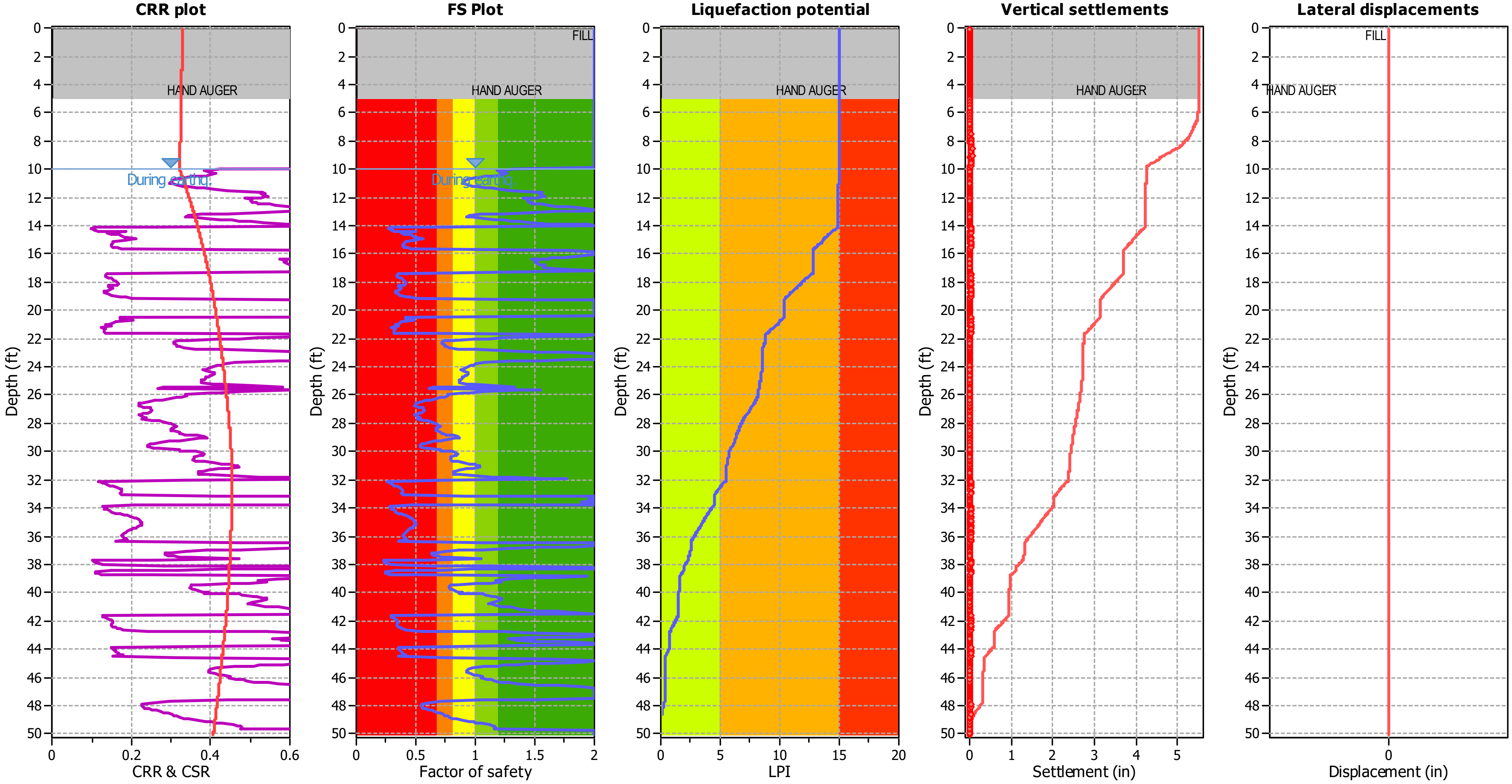
Liquefaction analysis overall plots (intermediate results)



Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	13.00 ft	Fill weight:	120.00 lb/ft^3
Fines correction method:	Robertson (2009)	Average results interval:	1	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	No
Earthquake magnitude M_w:	6.74	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.67	Use fill:	Yes	Limit depth applied:	No
Depth to water table (insitu):	14.00 ft	Fill height:	3.00 ft	Limit depth:	N/A

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	13.00 ft	Fill weight:	120.00 lb/ft^3
Fines correction method:	Robertson (2009)	Average results interval:	1	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	No
Earthquake magnitude M_w:	6.74	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.67	Use fill:	Yes	Limit depth applied:	No
Depth to water table (insitu):	14.00 ft	Fill height:	3.00 ft	Limit depth:	N/A

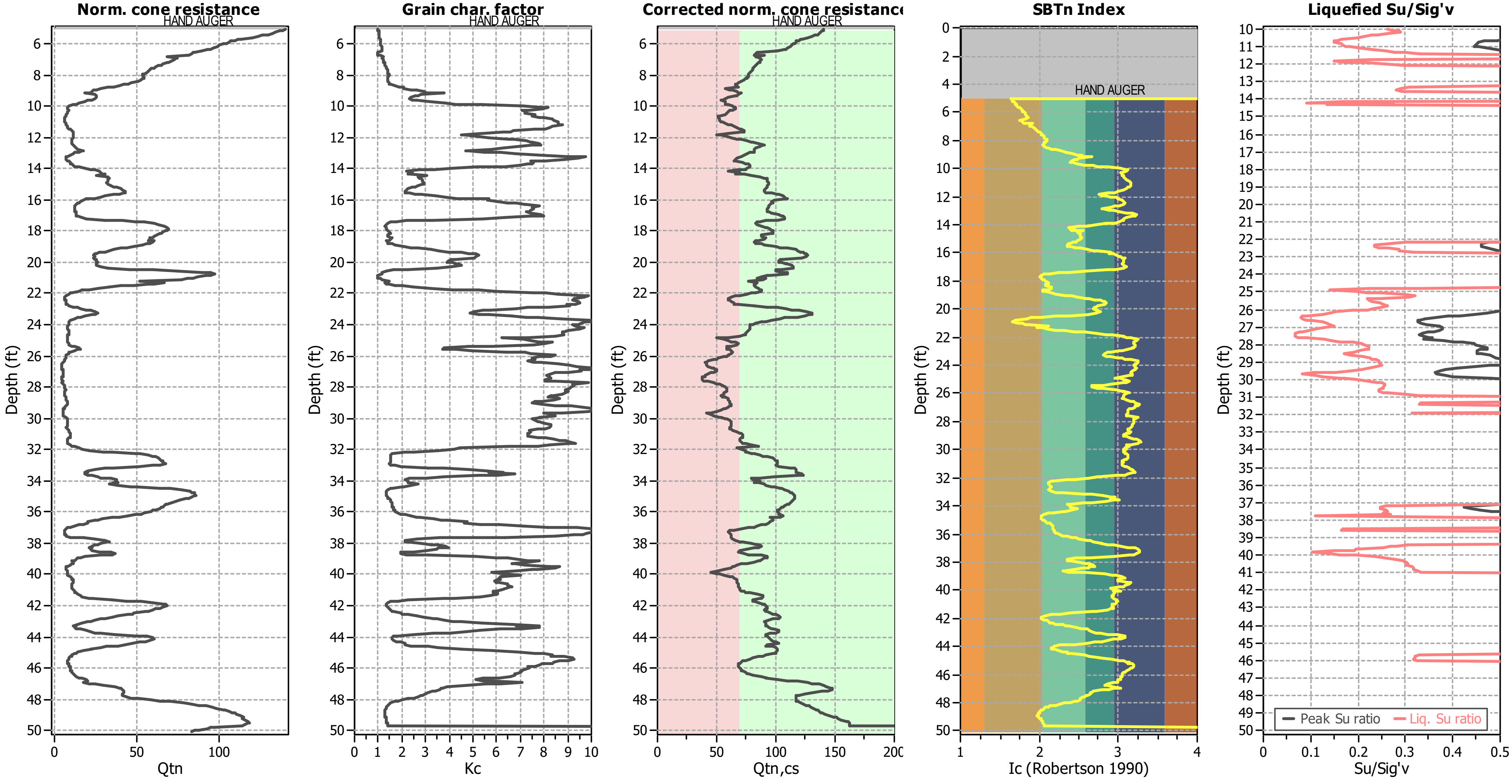
F.S. color scheme

- 🟥 Almost certain it will liquefy
- 🟧 Very likely to liquefy
- 🟨 Liquefaction and no liq. are equally likely
- 🟩 Unlike to liquefy
- 🟩 Almost certain it will not liquefy

LPI color scheme

- 🟥 Very high risk
- 🟧 High risk
- 🟩 Low risk

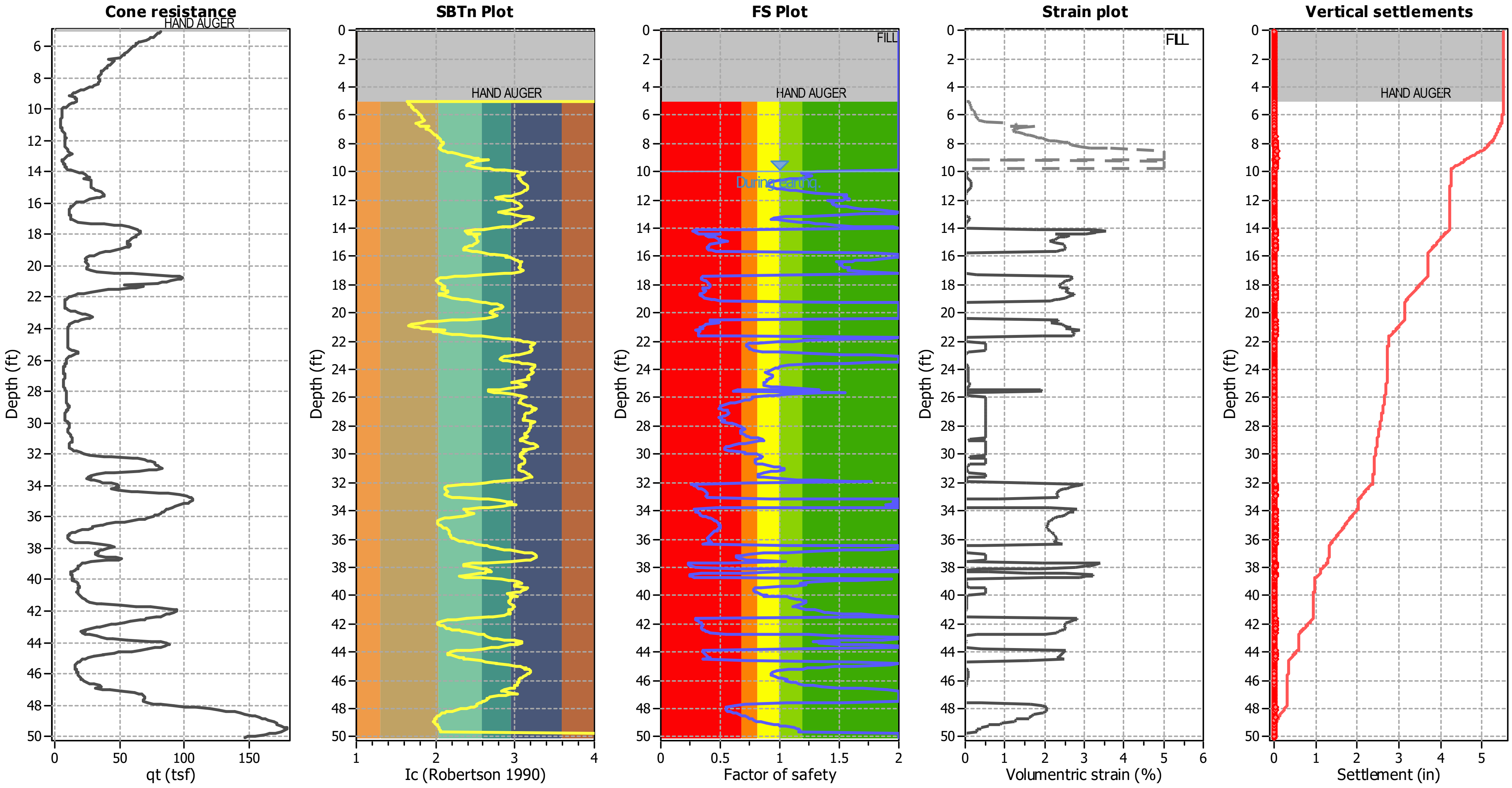
Check for strength loss plots (Robertson (2010))



Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	13.00 ft	Fill weight:	120.00 lb/ft³
Fines correction method:	Robertson (2009)	Average results interval:	1	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	No
Earthquake magnitude M_w:	6.74	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.67	Use fill:	Yes	Limit depth applied:	No
Depth to water table (insitu):	14.00 ft	Fill height:	3.00 ft	Limit depth:	N/A

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

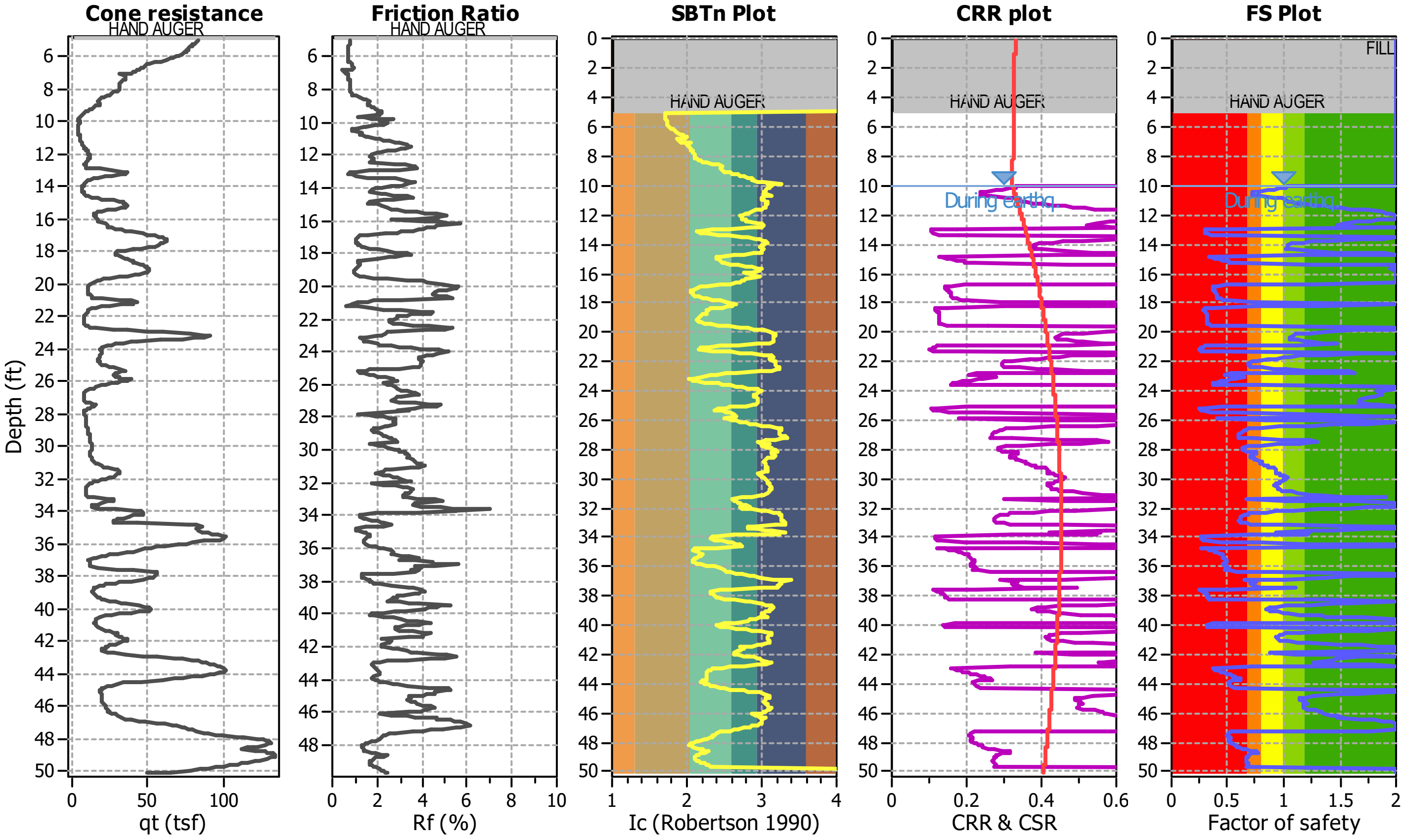
LIQUEFACTION ANALYSIS REPORT

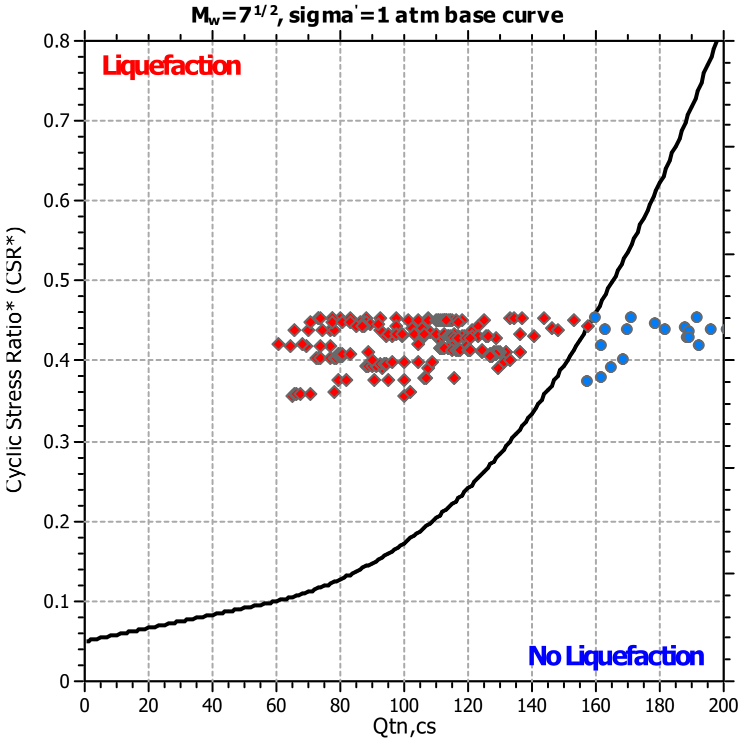
Project title : 3341.00 Melia Homes -- Garden Grove **Location : 9822 Russell Avenue, Garden Grove, CA**

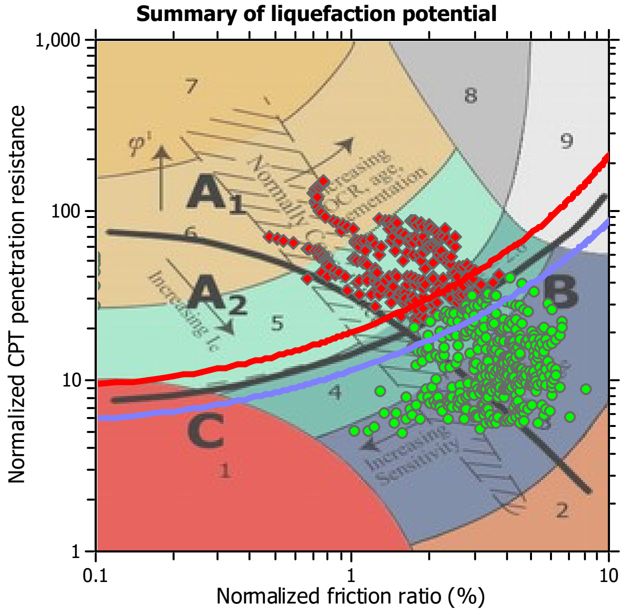
CPT file : CPT-2

Input parameters and analysis data

Analysis method:	Robertson (2009)	G.W.T. (in-situ):	14.00 ft	Use fill:	Yes	Clay like behavior applied:	All soils
Fines correction method:	Robertson (2009)	G.W.T. (earthq.):	13.00 ft	Fill height:	3.00 ft	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	1	Fill weight:	120.00 lb/ft³	Limit depth:	N/A
Earthquake magnitude M_w:	6.74	Ic cut-off value:	2.60	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.67	Unit weight calculation:	Based on SBT	K_σ applied:	No		

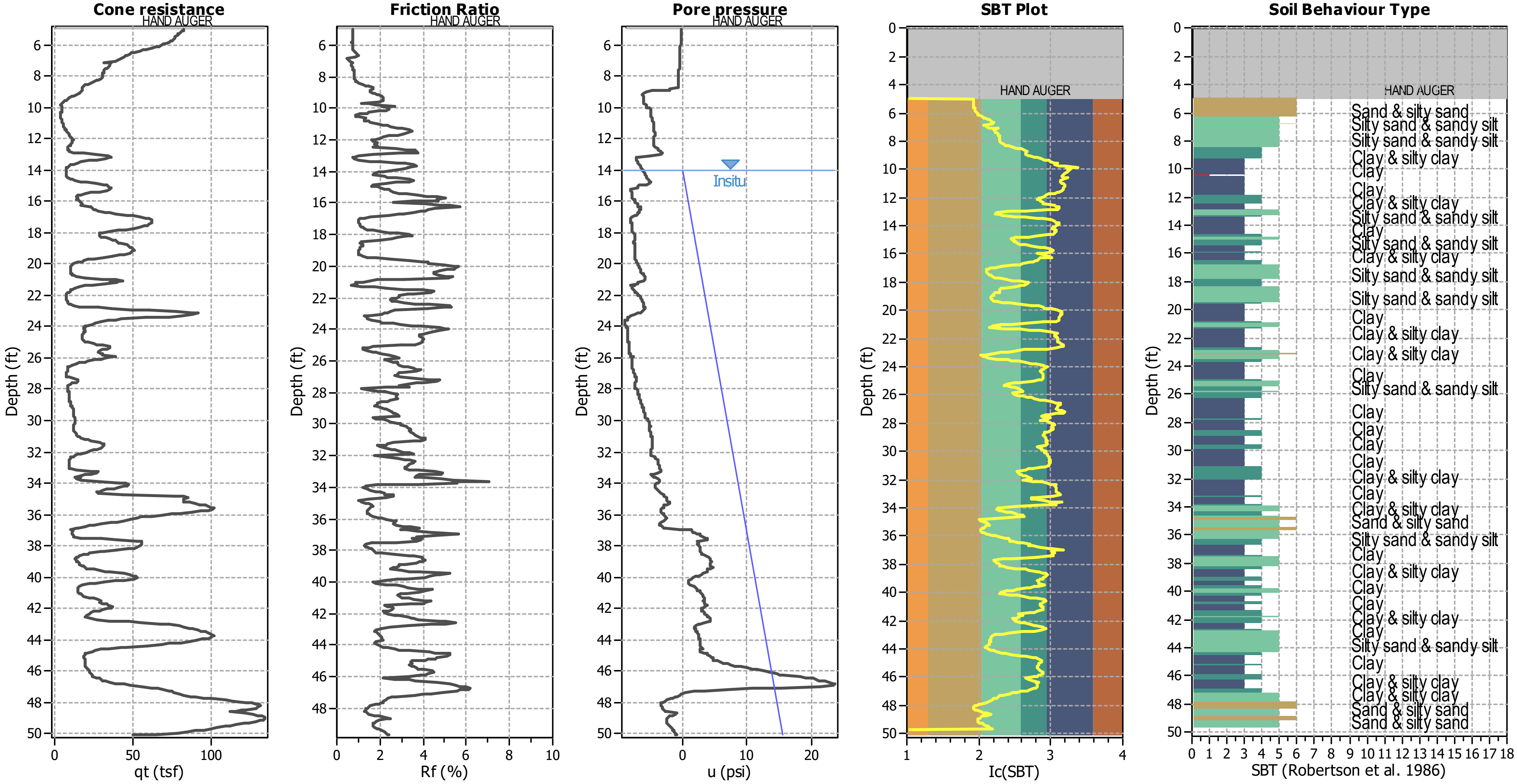






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and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

CPT basic interpretation plots



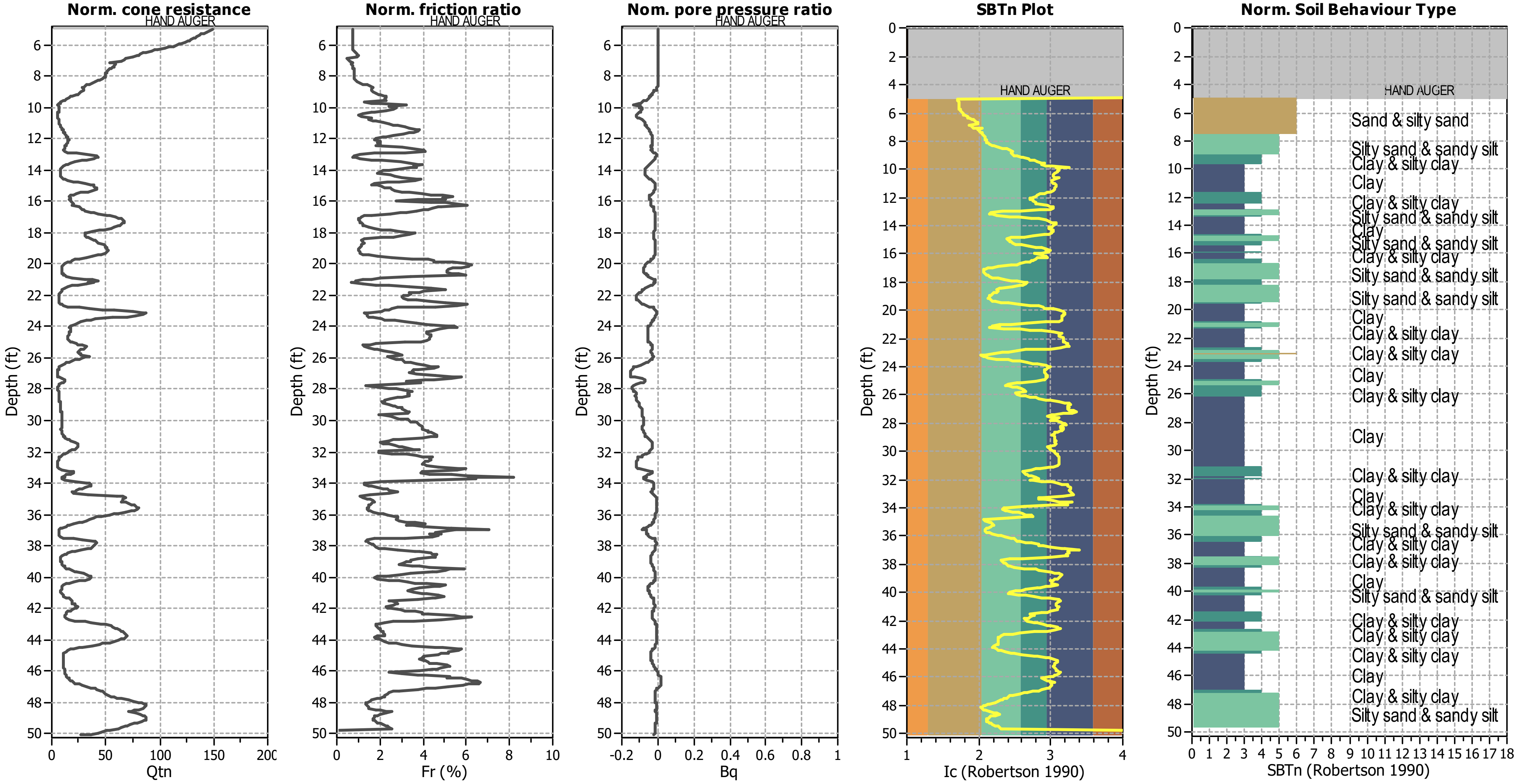
Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	13.00 ft	Fill weight:	120.00 lb/ft^3
Fines correction method:	Robertson (2009)	Average results interval:	1	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	No
Earthquake magnitude M_w:	6.74	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.67	Use fill:	Yes	Limit depth applied:	No
Depth to water table (insitu):	14.00 ft	Fill height:	3.00 ft	Limit depth:	N/A

SBT legend

🟥 1. Sensitive fine grained	▨ 4. Clayey silt to silty	🟧 7. Gravely sand to sand
🟫 2. Organic material	🟩 5. Silty sand to sandy silt	■ 8. Very stiff sand to
🟦 3. Clay to silty clay	🟨 6. Clean sand to silty sand	□ 9. Very stiff fine grained

CPT basic interpretation plots (normalized)



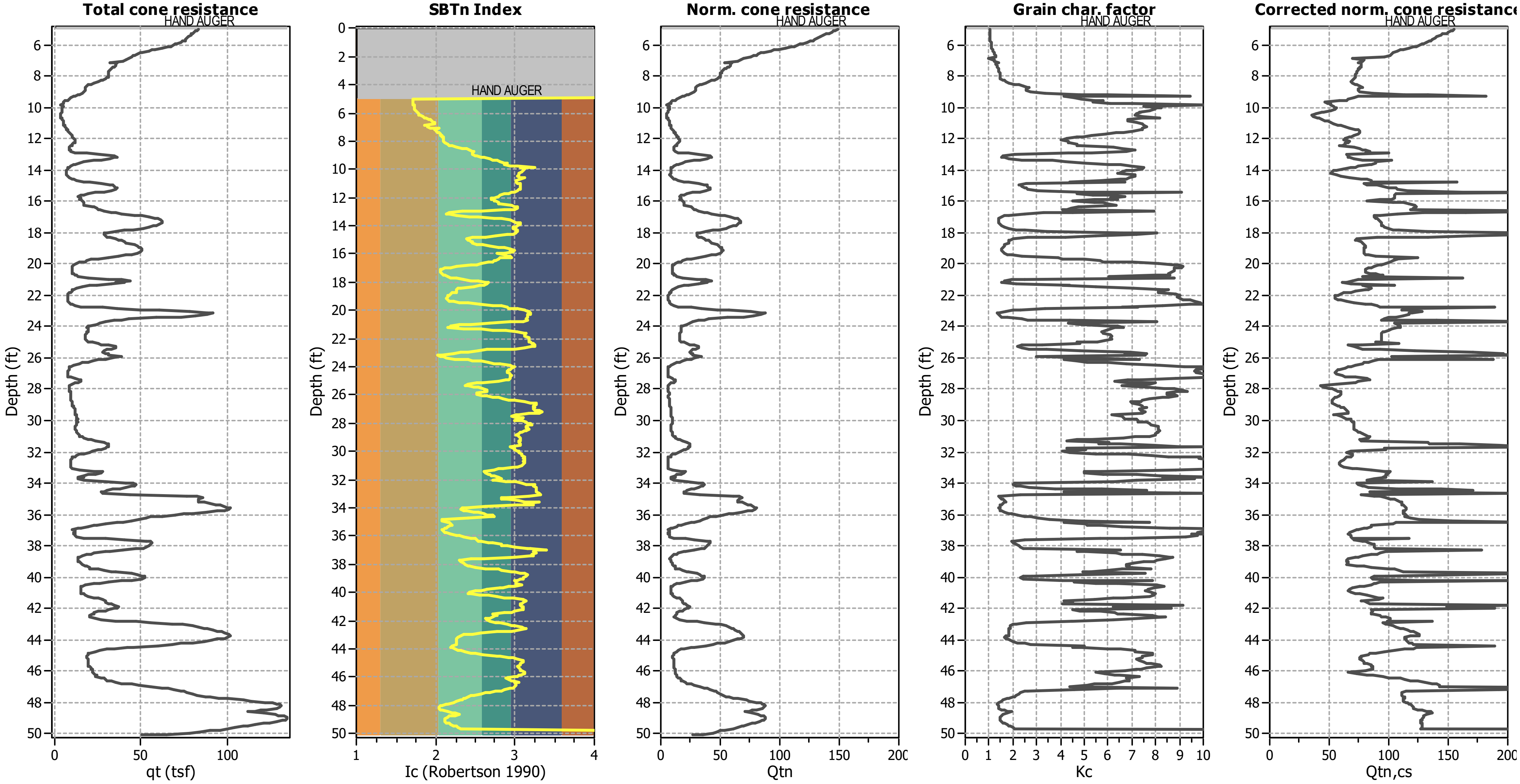
Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	13.00 ft	Fill weight:	120.00 lb/ft³
Fines correction method:	Robertson (2009)	Average results interval:	1	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	No
Earthquake magnitude M_w:	6.74	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.67	Use fill:	Yes	Limit depth applied:	No
Depth to water table (insitu):	14.00 ft	Fill height:	3.00 ft	Limit depth:	N/A

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

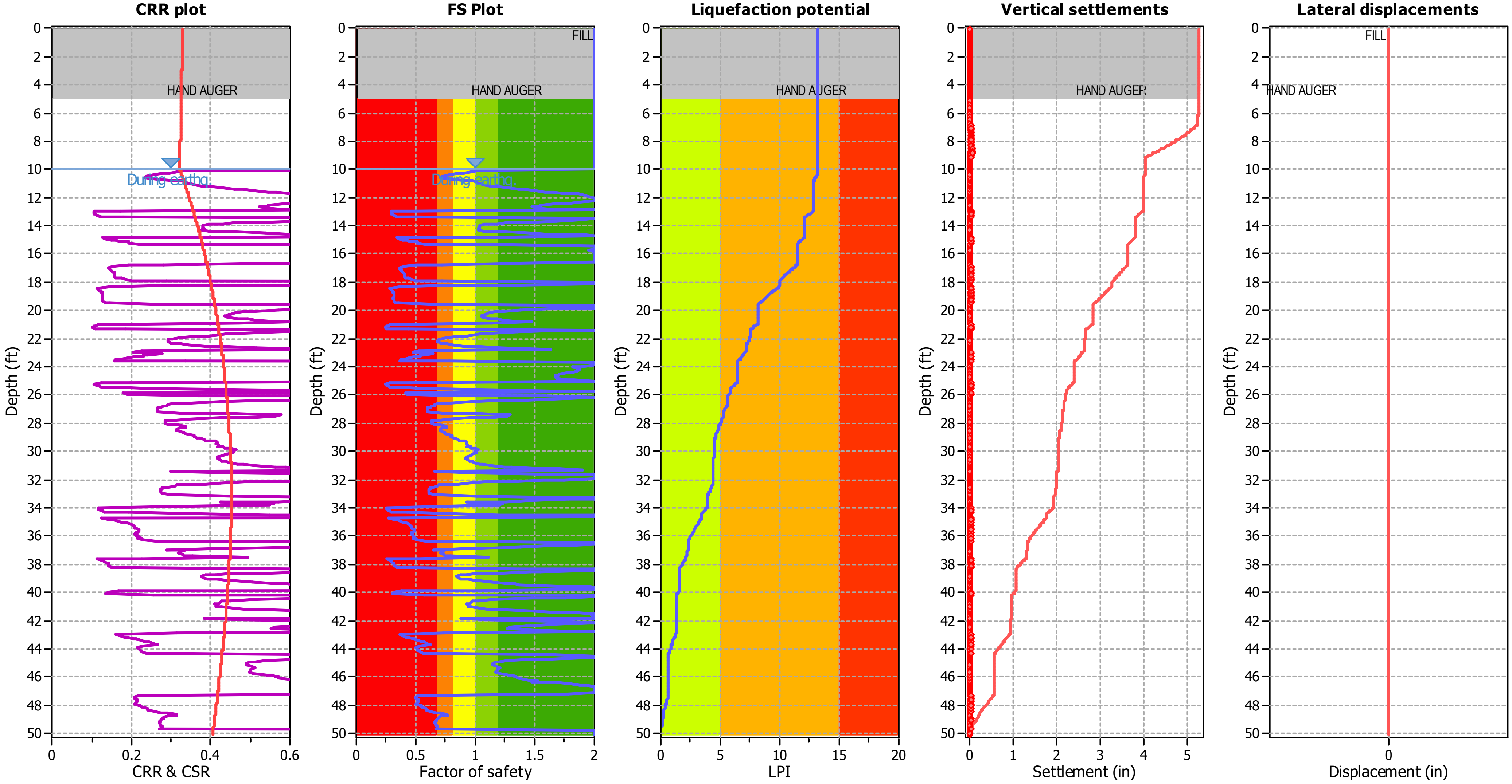
Liquefaction analysis overall plots (intermediate results)



Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	13.00 ft	Fill weight:	120.00 lb/ft^3
Fines correction method:	Robertson (2009)	Average results interval:	1	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	No
Earthquake magnitude M_w:	6.74	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.67	Use fill:	Yes	Limit depth applied:	No
Depth to water table (insitu):	14.00 ft	Fill height:	3.00 ft	Limit depth:	N/A

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	13.00 ft	Fill weight:	120.00 lb/ft³
Fines correction method:	Robertson (2009)	Average results interval:	1	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	No
Earthquake magnitude M_w:	6.74	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.67	Use fill:	Yes	Limit depth applied:	No
Depth to water table (insitu):	14.00 ft	Fill height:	3.00 ft	Limit depth:	N/A

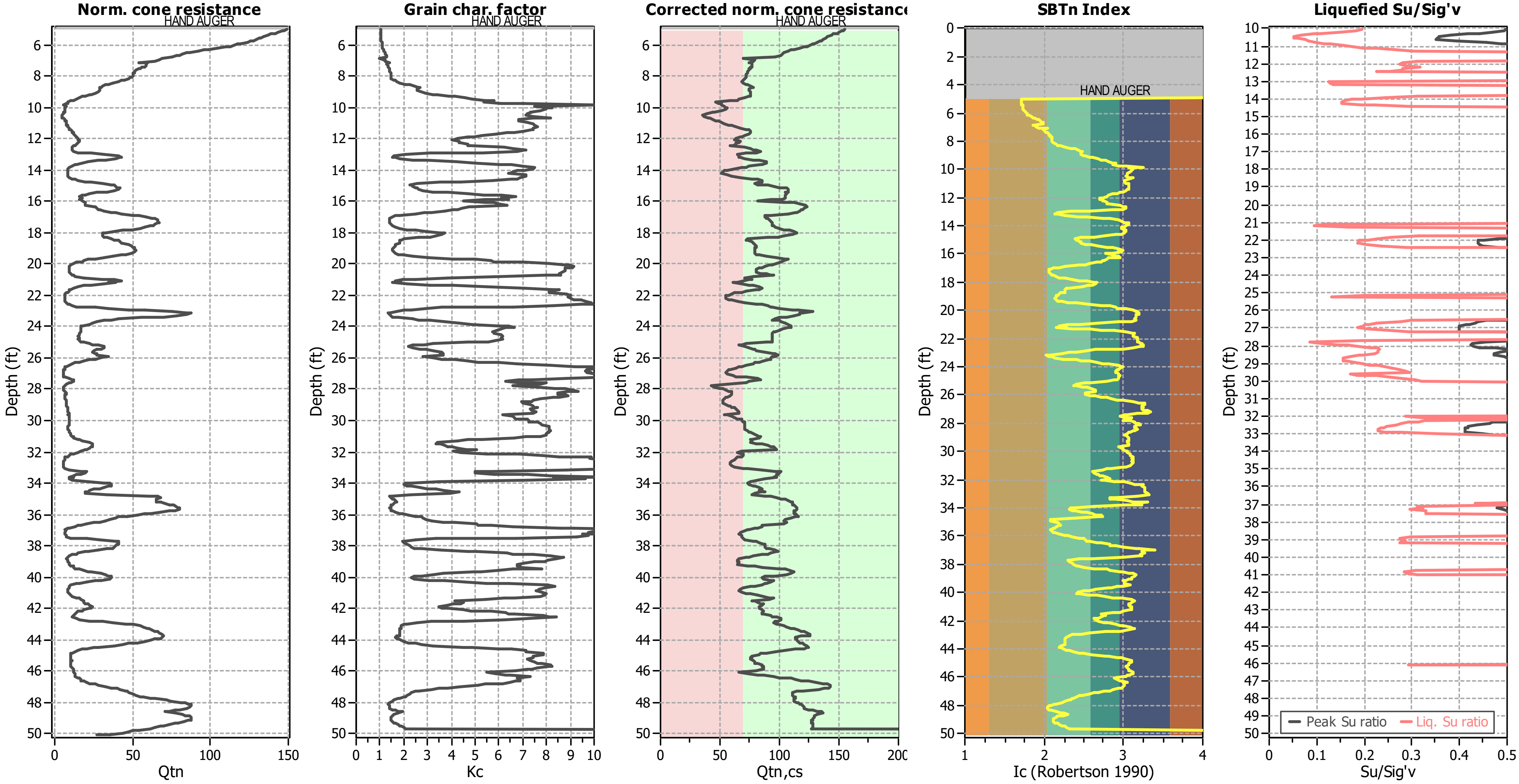
F.S. color scheme

- 🟥 Almost certain it will liquefy
- 🟧 Very likely to liquefy
- 🟨 Liquefaction and no liq. are equally likely
- 🟩 Unlike to liquefy
- 🟩 Almost certain it will not liquefy

LPI color scheme

- 🟥 Very high risk
- 🟧 High risk
- 🟨 Low risk

Check for strength loss plots (Robertson (2010))



Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	13.00 ft	Fill weight:	120.00 lb/ft^3
Fines correction method:	Robertson (2009)	Average results interval:	1	Transition detect. applied:	No
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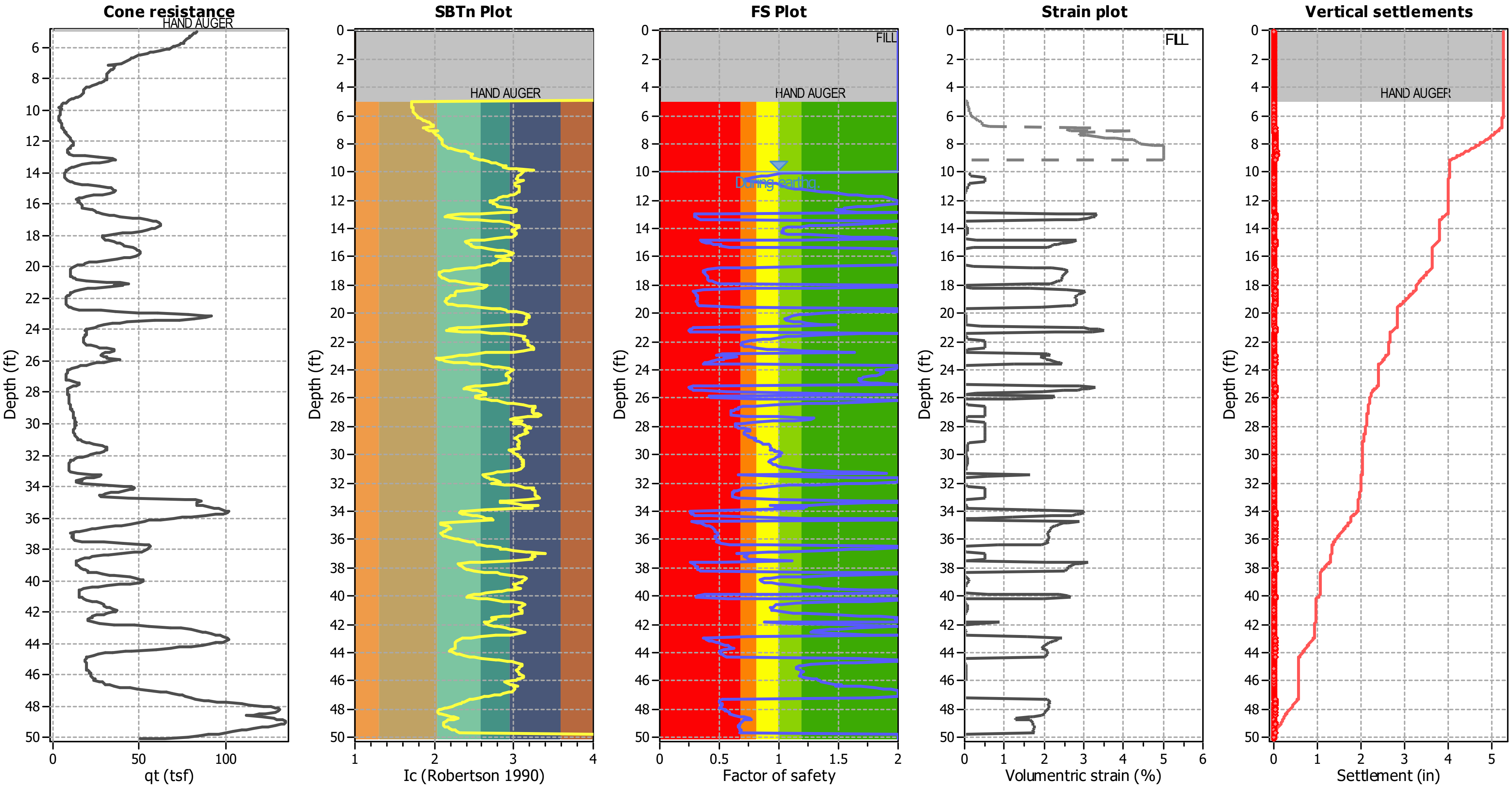
Earthquake magnitude M_w:	6.74	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.67	Use fill:	Yes	Limit depth applied:	No
Depth to water table (insitu):	14.00 ft	Fill height:	3.00 ft	Limit depth:	N/A

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

Project title : 3341.00 Melia Homes -- Garden Grove

Location : 9822 Russell Avenue, Garden Grove, CA

Overall vertical settlements report