APPENDIX D

GEOTECHNICAL EVALUATION

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UPDATED GEOTECHNICAL EVALUATION
FOR
PROPOSED RESIDENTIAL DEVELOPMENT
ORANGE AVENUE SENIORS
9470 MOODY STREET AND 5081 ORANGE AVENUE
CITY OF CYPRESS, ORANGE COUNTY, CALIFORNIA

PREPARED FOR

MELIA HOMES
8951 RESEARCH DRIVE
IRVINE, CALIFORNIA 92618

PREPARED BY

GEOTEK, INC.
1548 N. MAPLE STREET
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PROJECT NO. 2573-CR **JANUARY 4, 2021**

GEOTEK

GEOTEK

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January 4, 2021
Project No. 2573-CR

Melia Homes
8151 Research Drive
Irvine, California 92618

Attention: Mr. Chad Brown

Subject: Updated Geotechnical Evaluation
Proposed Residential Development
Orange Avenue Seniors
9470 Moody Street and 5081 Orange Avenue
City of Cypress, Orange County, California

Dear Mr. Brown:

We are pleased to provide herein the results of our updated geotechnical evaluation for the subject site located in the city of Cypress, County of Orange, California. This report presents a discussion of our evaluation and provides preliminary geotechnical recommendations for earthwork, foundation design, and construction. In our opinion, site development appears feasible from a geotechnical viewpoint provided that the recommendations included herein are incorporated into the design and construction phases of site development.

The opportunity to be of service is sincerely appreciated. If you should have any questions, please do not hesitate to call our office.

Respectfully submitted,
GeoTek, Inc.

Edward H. LaMont
CEG 1892, Exp. 07/31/22
Principal Geologist

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Distribution: .pdf file sent to addressee via email

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1. PURPOSE AND SCOPE OF SERVICES

The purpose of this study was to complete a geotechnical evaluation of the existing geotechnical conditions of the project site. Services provided for this study included the following:

- Research and review of available geologic and geotechnical data, and general information pertinent to the site,
- Site reconnaissance,
- Site exploration consisting of the excavation, logging and sampling of ten exploratory hollow-stem borings,
- Collection of relatively undisturbed and bulk soil samples of the onsite materials,
- Laboratory testing of the soil samples collected from the site,
- Review and evaluation of site seismicity, and
- Compilation of this updated geotechnical report which presents our findings, conclusions, and recommendations for site development.

2. SITE DESCRIPTION AND PROPOSED DEVELOPMENT

2.1 SITE DESCRIPTION

The rectangular-shaped site is located on the north side of Orange Avenue and east of Moody Street in the city of Cypress, Orange County, California. The site is addressed as 9470 Moody Street and 5081 Orange Avenue. The roughly 6.3-acre site area is currently being used as a Cypress School District facility. The western-half of the site contains various one-story administration/educational buildings and associated parking/drive areas, while the eastern half consists of a maintenance yard. The maintenance yard includes numerous structures, vehicles, equipment, and materials and is asphalt concrete paved. Near the north property line, stockpiled soil up to eight feet high with a grass area further north and west are present. Access to the site is via driveways off Orange Avenue and Moody Street.

The site is relatively flat (not taking into consideration the stockpiled soil) with surface drainage directed to the south-southwest. Topographic relief across the site is less than two feet.

A perimeter wall marks the limits of the site to the north and east. The site limits to the south and west are partially defined by a chain-link fence.

The property is bounded by single-family residences to the north; a church facility to the east; Orange Avenue with single-family dwellings beyond to the south; and Moody Street with an animal hospital and single-family homes to the west.

The general location of the site is shown in Figure 1. The current conditions of the site and site topography are shown in the Exploration Location Map presented as Figure 2. Figure 2 uses the Boundary and Topographic Survey Plan, prepared by Salazar Surveying, dated September 12, 2017, as a base map.

2.2 PROPOSED DEVELOPMENT

It is our understanding that proposed development will consist of rough grading of the site and subsequent construction of various multi-family buildings for senior housing. The buildings are anticipated to be up to two-story in height and of wood-framed and stucco construction resting on shallow foundations and concrete floors. Structural loads are anticipated to be typical for this type of construction. Site improvements will include interior streets, driveways, a pool area, underground utilities, possibly small interior retaining walls and landscaped areas. In addition, we expect that cuts and fills up two feet in height to be required to achieve the proposed site grades.

If site development differs from the assumptions made herein, the recommendations included in this report should be subject to further review and evaluation. Site development plans should be reviewed by GeoTek when they become available. Additional geotechnical field exploration, analyses and recommendations may be necessary upon review of site development plans.

3. REPORT REVIEW

NMG Geotechnical, Inc. (NMG) issued on November 4, 2011 a report entitled *Preliminary Geotechnical Exploration and Design Parameters for Potential Residential Development, District Office Site, Cypress School District, City of Cypress, California*. This evaluation included the excavation of two exploratory borings to depths of 51.5 feet below existing ground surface and two Cone Penetration Test (CPT) soundings to depths of 50 feet within the project site. NMG described the earth materials at the site as younger alluvial deposits consisting of silty and clayey sand which were moist to saturated and loose to dense. Groundwater was reportedly present in the borings at depths of 8.25 and 10 feet. In addition, NMG noted that the upper 15 feet of the alluvium had relatively lower blow counts per foot as well as dry densities ranging from 78.3 to 108.2 pounds per cubic foot (pcf) and water content ranging from 5.5 to 31.5 percent. NMG performed a liquefaction assessment for the site and found some liquefaction-prone layers below 10 feet in depth, with thicknesses between 1 and 3 feet. Total and differential seismic settlements were estimated to be 1.5 inches and 0.75 inches, respectively. Removal and recompaction on the order of 5 feet deep was suggested by NMG to mitigate the soft/loose upper alluvium. Very low expansion potential (EI = 2) and high R-value (RV = 64) were also reported by NMG for the upper alluvium.

Logs of the exploratory borings and soundings as well as laboratory test results by NMG are included in Appendix A.

4. FIELD EXPLORATION AND LABORATORY TESTING

4.1 FIELD EXPLORATION

GeoTek first investigated the subsurface soil conditions of the eastern-half portion of the property on September 25, 2017 via five exploratory borings to depths ranging from 11.5 to 13 feet. On December 8, 2020, the western half of the site was explored via five additional borings excavated to depths ranging from 7 to 8 feet. The approximate locations of our borings and the previous borings and CPT soundings by NMG are shown in the Exploration Location Map, presented as Figure 2. Logs of the exploratory borings performed by GeoTek are included in Appendix B. GeoTek collected relatively undisturbed and bulk samples of onsite soil materials from the borings and transported to our in-house geotechnical laboratory for laboratory testing.

4.2 LABORATORY TESTING

Laboratory testing was performed on selected relatively undisturbed and bulk soil samples collected during the field exploration. The purpose of the laboratory testing was to confirm the field classification of the soil materials encountered and to evaluate the soils physical properties for use in the engineering design and analysis. Results of the laboratory testing program along with a brief description and relevant information regarding testing procedures are included in Appendix C.

5. GEOLOGIC AND SOILS CONDITIONS

5.1 REGIONAL SETTING

The subject property is situated in the Peninsular Ranges geomorphic province. The Peninsular Ranges province is one of the largest geomorphic units in western North America. Basically, it extends roughly 975 miles from the north and extends from the Transverse Ranges geomorphic province to the tip of Baja California, from north to south. This province varies in width from about 30 to 100 miles. It is bounded on the west by the Pacific Ocean, on the south by the Gulf of California and on the east by the Colorado Desert Province.

The Peninsular Ranges are essentially a series of northwest-southeast oriented fault blocks. Several major fault zones are found in this province. The Elsinore Fault zone and the San Jacinto Fault zone trend northwest-southeast and are found in the near the middle of the province. The San Andreas Fault zone borders the northeasterly margin of the province.

More specific to the subject property, the site is located in an area geologically mapped to be underlain by younger alluvial fan deposits (Sasucedo, G.J., Greene, G.H., Kennedy, M.P., and Bezore, S.P. 2016). The closest fault to the subject site is the Newport-Inglewood Fault located approximately 5.5 miles to the southwest.

5.2 GENERAL SOIL/GEOLOGIC CONDITIONS

A brief description of the earth materials encountered below the site and within the area of anticipated construction is presented in the following section. Based on our field exploration and exploration by NMG (2011), the area of anticipated improvements is underlain by undocumented fill covering younger alluvium.

5.2.1 Undocumented Fill

Undocumented fill was encountered in some of our borings to approximately 3 to 4 feet below grade. The fill consisted of brown, moist, loose silty sand. The fill is likely associated with the current use of the site and may be thicker beneath existing building areas. In addition, stockpile soil up to 8 feet in height is situated within the northeastern portion of the site (See Figure 2).

5.2.2 Younger Alluvial Fan Deposits

Younger alluvium was encountered in our borings below the fill or at near ground surface and extended to the maximum depth explored of about 13 feet. The alluvium encountered generally consisted of brown to brownish gray, moist to very moist, soft sandy silt and loose to medium silty sand. The logs of the exploratory borings and data from the CPT soundings performed by

NMG show similar conditions with soft/loose alluvial materials up to a depth of 15 feet. Below 15 feet, the alluvium has alternating layers of silty sand, sandy silt, poorly graded sand, clay, and silty clay which are in a medium dense to dense/stiff state.

According to the test results by NMG and GeoTek, the near surface site-soils have a "very low" expansion potential when tested and classified in accordance with ASTM D 4829. Also, consolidation tests performed by GeoTek on relatively undisturbed samples of the upper 10 feet of the alluvium showed that these soils are moderately compressible and have slight to moderate potential for collapse. Laboratory test results are shown in Appendix C.

Detailed boring logs are provided in Appendices A and B.

5.3 SURFACE WATER AND GROUNDWATER

5.3.1 Surface Water

If encountered during the earthwork construction, surface water on this site is the result of precipitation or surface run-off from surrounding sites. Overall drainage in the area is variable, and most commonly directed toward the south-southwest. Provisions for surface drainage will need to be accounted for by the project civil engineer.

5.3.2 Groundwater

Groundwater was not encountered in our exploratory borings performed to a maximum depth of 13 feet. However, groundwater was found in NMG's borings at 8 and 10 feet below the ground surface at the time of drilling (see logs in Appendix A).

Historically highest groundwater at the site is reported to be about 10 feet below ground surface based on the *Seismic Hazard Zone Report for Los Alamitos 7.5-Minute Quadrangle* (California Department of Conservation, 1998).

The GeoTracker database (https://geotracker.waterboards.ca.gov/) indicates that groundwater monitoring wells were installed on site to evaluate the impact of an old leaking underground fuel tank. Groundwater levels were reported to fluctuate between 4 and 8 feet below grade.

It is possible that seasonal variations (temperature, rainfall, etc.) will cause fluctuations in the groundwater level. The groundwater levels presented in this report are the levels that were measured at the time of our field activities. It is recommended that the contractor determine the actual groundwater levels at the site at the time of the construction activities to determine the impact, if any, on the construction procedures.

5.4 FAULTING AND SEISMICITY

The geologic structure of the entire southern California area is dominated mainly by northwest-trending faults associated with the San Andreas system. The site is in a seismically active region. No active or potentially active fault is known to exist at this site nor is the site situated within an *"Alquist-Priolo"* Earthquake Fault Zone (Bryant and Hart, 2007; CGS, 1986). The subject property is not located within a State of California Seismic Hazard Zone for earthquake induced landsliding; however, the site is located within a State of California Seismic Hazard Zone for liquefaction (CGS, 1998).

5.4.1 Seismic Design Parameters

The site is located at approximately 33.8250 degrees Latitude and -118.0445 degrees Longitude. Site spectral accelerations (S_a and S_1), for 0.2 and 1.0 second periods for a Class "D" site, was determined from the SEAOC/OSHPD web interface that utilizes the USGS web services and retrieves the seismic design data and presents that information in a report format. Using the ASCE 7-16 option on the SEAOC/OSHPD website results in the values for S_{M1} and S_{D1} reported as "null-See Section 11.4.8" (of ASCE 7-16). As noted in ASCE 7-16, Section 11.4.8, a site-specific ground motion procedure is recommended for Site Class D when the value S_1 exceeds 0.2. The value S_1 for the subject site exceeds 0.2.

For a site Class "D", an exception to performing a site-specific ground motion analysis is allowed in ASCE 7-16 where S_1 exceeds 0.2 provided the value of the seismic response coefficient, Cs, is conservatively calculated by Eq 12.8-2 of ASCE 7-16 for values of $T \leq 1.5Ts$ and taken as equal to 1.5 times the value computed in accordance with either Eq. 12.8-3 for $T_L \geq T > 1.5Ts$ or Eq. 12.8-4 for $T > T_L$.

The results, based on the 2015 NEHRP and the 2019 CBC, are presented in the following table and we have assumed that the exception as allowed in ASCE 7-16 is applicable. If the exception is deemed not appropriate, a site-specific ground motion analysis will be required.

SITE SEISMIC PARAMETERS	
Mapped 0.2 sec Period Spectral Acceleration, S_S	1.476g
Mapped 1.0 sec Period Spectral Acceleration, S_1	0.524g
Site Coefficient for Site Class "D", F_a	1.0
Site Coefficient for Site Class "D", F_v	1.776
Maximum Considered Earthquake Spectral Response Acceleration for 0.2 Second, S_{MS}	1.476g
Maximum Considered Earthquake Spectral Response Acceleration for 1.0 Second, S_{M1}	0.931g
5% Damped Design Spectral Response Acceleration Parameter at 0.2 Second, S_{DS}	0.984g
5% Damped Design Spectral Response Acceleration Parameter at 1 second, S_{D1}	0.621g
Peak Ground Acceleration Adjusted for Site Class Effects, PGA_M	0.695g
Seismic Design Category	D

5.5 LIQUEFACTION AND SEISMICALLY INDUCED SETTLEMENT

Liquefaction describes a phenomenon in which cyclic stresses, produced by earthquake-induced ground motion, create excess pore pressures in relatively cohesionless and low plastic soils. These soils may thereby acquire a high degree of mobility, which can lead to lateral movement, sliding, consolidation and settlement of loose sediments, sand boils and other damaging deformations. This phenomenon occurs only below the water table, but, after liquefaction has developed, the effects can propagate upward into overlying non-saturated soil as excess pore water dissipates.

The factors known to influence liquefaction potential include soil type and grain size, relative density, groundwater level, confining pressures, and both intensity and duration of ground shaking. In general, materials that are susceptible to liquefaction are loose, saturated granular soils having low fines content under low confining pressures and some low plastic silts and clays.

Based on the review of onsite groundwater data, a historic high groundwater depth of 4 feet was used in our analysis. The soil profiles identified within CPT-1 and CPT-2 soundings by NMG were used for our liquefaction assessment. A mean magnitude weighted (Mw) seismic event of 6.72 (based on a 2 percent exceedance in 50 years) and a PGA_M value of 0.695g were used in our assessment. We assumed that the grading of the proposed residential pads will not incorporate significant cuts and/or fill and therefore the current confining stress will remain unchanged. The CPT data was used for the liquefaction analysis since the CPT provides a continuous log of the subsurface soils and is deemed a superior means of evaluation as compared to conventional borings. GeoTek evaluated the liquefaction potential of the on-site soils using the computer program *Cliq Version 2.0*.

The results of the analyses indicated the presence of some layers of loose sands and silty sands that would be prone to liquefaction and settlement. The following table summarizes the amount of total settlement (liquefaction settlement plus settlement of dry sands) estimated at each CPT location:

ESTIMATED SEISMICALLY INDUCED TOTAL SETTLEMENT	
CPT Sounding	Total Settlement (inches)
1	1.3
2	1.5

As noted above, seismically induced settlement of up to about 1.5 inches total and 0.75-inch differential over a 30-foot span is estimated for the property. The results of the liquefaction and seismic settlement analyses are presented within Appendix D.

Based on relationships developed by Ishihara (1985) with respect to the thickness of potentially liquefiable soils relative to the thickness of the non-liquefiable soils, it is our opinion that a potential does exist for surface manifestations (sand boils and/or loss of bearing support) to occur during the design level earthquake. Recommendations presented in subsequent sections of this report have been prepared to reduce the potential for surface manifestations.

Due to the flat topography of the site, the potential for lateral spreads is considered nil.

5.6 OTHER SEISMIC HAZARDS

Evidence of ancient landslides or slope instabilities at this site was not observed during our investigation. Thus, the potential for landslides is considered negligible.

The potential for secondary seismic hazards such as a seiche or tsunami is considered negligible due to site elevation and distance to an open body of water.

6. CONCLUSIONS AND RECOMMENDATIONS

6.1 GENERAL

The anticipated site development appears feasible from a geotechnical viewpoint provided that the following recommendations, and those provided by this firm at a later date are incorporated into the design and construction phases of development. Site development and grading and foundation plans should be reviewed by GeoTek, Inc. when they become available.

Site excavations indicate that the property contains loose undocumented fills and soft/loose to stiff/medium dense alluvial deposits. Therefore, to provide a dense, homogeneous support for the proposed structures, all undocumented fill and upper loose/soft alluvium should be removed and recompacted within the proposed structural grading limits. General removals on the order of 5 feet are anticipated.

Our analyses also indicate that the site may be subject to liquefaction and settlement during the design level earthquake. This could result in seismically induced settlement of up to about 1.5 inches total and 0.75-inch differential over a 30-foot span. Surface manifestation of liquefaction (sand boils and/or loss of bearing support) are also possible to occur.

According to ASCE 7-16, a maximum differential settlement of 3.6 inches over a 30-foot span can be tolerated by multistory structures with Risk Category II (structures of ordinary occupancy such as residential buildings). However, ASCE 7-16 indicates that standard shallow foundations may be designed where the settlement does not exceed one-fourth of the differential settlement threshold (i.e. 0.9 inches). Given that the estimated differential settlement exceeds one-fourth of the threshold and given that the property may be subject to potential manifestations of liquefaction, we recommend that site buildings be supported by either shallow footings with foundation ties, post-tensioned slabs, or mat foundations.

After the completion of the recommended remedial grading, we anticipate a total static settlement of less than 1-inch and a maximum differential static settlement of less than 0.5-inch in a 30-foot span for residential buildings resting on shallow footings with foundation ties. For structures resting on mat foundations or post-tensioned systems, a total settlement of about 2 inches and a differential settlement of about 1 inch over a horizontal distance of 30 feet are estimated. These static settlements along with the anticipated seismically induced settlements will result in up to 2.5 inches of combined (static plus seismic) total settlement and up to 1.3 inches of combined (static plus seismic) differential settlements over a horizontal distance of 30 feet for the future residential structures resting on shallow footings with foundation ties. For structures resting on mat foundations or post-tensioned systems, up to 3.5 inches of combined (static plus seismic) total settlement and up to 1.8 inches of combined (static plus seismic) differential settlements over a horizontal distance of 30 feet are estimated.

6.2 EARTHWORK CONSIDERATIONS

Earthwork and grading should be performed in accordance with the applicable grading ordinances of the City of Cypress and/or County of Orange, the *2019 California Building Code* (CBC), and recommendations contained in this report. Site grading plans should be reviewed by this office when they become available. Additional recommendations will likely be offered subsequent to review of these plans. The General Grading Guidelines included in Appendix E outline general procedures and do not anticipate all site-specific situations. In the event of conflict, the recommendations presented in the text of this report should supersede those contained in Appendix E.

6.2.1 Site Clearing

The site should be cleared of existing vegetation, roots, stockpiled soil, and debris. All foundations, slabs, utilities, and underground improvements associated to the existing buildings should also be removed. These materials should be properly disposed of off-site. Voids resulting from site clearing should be backfilled with engineered fill.

6.2.2 Site Preparation

As a minimum, the upper 5 feet of existing soils or 3 feet below footing base, whichever is deeper, should be completely removed within the structural grading limits. The depth of over-excavation should be extended, where needed, to remove all undocumented fill. Removal bottoms should expose relatively uniform, moist alluvium that is not visibly porous or highly compressible. As a minimum, removals should extend down and away from foundation elements at a 1:1 projection, to the recommended removal depth.

All existing fill should be removed from surface improvement areas. A minimum of 12 inches of engineered fill should be provided below asphaltic concrete pavement and Portland cement concrete hardscape areas. The horizontal extent of removals should extend at least two feet beyond the edge. Development plans should be reviewed by this firm when available. Depending on actual field conditions encountered during grading, locally deeper areas of removal may be recommended.

The bottom of all removals should be scarified to a minimum depth of 12 inches, brought to slightly above the optimum moisture content, and then recompacted to at least 90 percent of the soil's maximum dry density, per ASTM D 1557. The bottoms of removals should be observed by a GeoTek representative prior to scarification.

The bottom of removals will likely encounter very moist and soft soils that may require stabilization. If necessary, removal bottoms may be stabilized with a layer of gravel and/or geogrid supplemented with gravel, prior to placing engineered compacted fill. A 12-inch thick layer of gravel has been successfully used on similar project sites that GeoTek has provided services on in the past.

6.2.3 Engineered Fill

The onsite soils are considered suitable for reuse as engineered fill provided they are free from vegetation, debris and other deleterious material. Rock fragments greater than 6 inches in maximum dimension should not be incorporated into engineered fill.

At the time of our field investigation, the on-site soils were very moist (approximately 5 to 18 percent above optimum water content). To be suitable for placement as engineered fill, these materials should be dried to approximately optimum moisture content.

Concrete generated from the demolition of existing site improvements may be incorporated into site fills provided the following guidelines are implemented: 1) concrete should be free of rebar or other deleterious materials and should be broken down to a maximum dimension of six inches; 2) concrete should not be placed within three feet of finish grade in the building pad areas or within one foot of subgrade elevations in the street/drive areas; 3) concrete should be distributed in the fill and should not be "nested" or placed in concentrated pockets.

Engineered fill materials should be placed in horizontal lifts not exceeding eight inches in loose thickness, moisture conditioned to over the optimum moisture content and compacted to a minimum relative compaction of 90 percent (ASTM D 1557).

6.2.4 Excavation Characteristics

Excavation in the onsite soil materials is expected to be easy using heavy-duty grading equipment in good operating conditions.

All temporary excavations for grading purposes and installation of underground utilities should be constructed in accordance with local and Cal-OSHA guidelines. Temporary excavations within the onsite materials should be stable at 1:1 (h:v) inclinations for cuts less than five feet in height.

6.2.5 Shrinkage and Subsidence

Several factors will impact earthwork balancing on the site, including shrinkage, bulking, subsidence, trench spoil from utilities and footing excavations, as well as the accuracy of topography.

Shrinkage, bulking and subsidence are primarily dependent upon the degree of compactive effort achieved during construction. For planning purposes, a shrinkage factor of 5 to 15 percent may be considered for the materials requiring removal and/or recompaction. Site balance areas should be available in order to adjust project grades, depending on actual field conditions at the conclusion of site earthwork construction. Bulking is not considered to be a significant factor with the underlying materials within the vicinity of the anticipated construction. Subsidence on the order of up to 0.2 foot could occur.

6.2.6 Trench Excavations and Backfill

Temporary excavations within the onsite materials should be stable at 1:1 (h:v) inclinations for short durations during construction, and where cuts do not exceed 5 feet in height. Temporary cuts to a maximum height of four feet can be excavated vertically, but local sloughing and/or failure could occur due to the granular nature of some of the soils at this site. Increased caution should be applied when working near or within any excavations at this site.

Trench excavations should conform to Cal-OSHA regulations. The contractor should have a competent person, per OSHA requirements, on site during construction to observe conditions and to make the appropriate recommendations.

Utility trench backfill should be compacted to at least 90 percent relative compaction (as determined per ASTM D 1557). Under-slab trenches should also be compacted to project specifications. Where applicable, based on jurisdictional requirements, the top 12 inches of backfill below subgrade for onsite pavements should be compacted to at least 95 percent relative compaction. Onsite materials may not be suitable for use as bedding material but should be suitable as backfill provided particles larger than six± inches are removed.

Compaction should be achieved with a mechanical compaction device. Ponding or jetting of trench backfill is not recommended. If backfill soils have dried out, they should be thoroughly moisture conditioned prior to placement in trenches.

6.2.7 Slopes

Slopes at the site constructed at gradients of 2:1 (h:v) or flatter, in accordance with industry standards, are anticipated to be grossly stable. Fill placed on sloping ground should be properly benched into competent soils.

6.3 DESIGN RECOMMENDATIONS

6.3.1 Foundation Design Criteria

Our exploratory borings encountered relatively granular soils within the upper five to ten feet at the site. Our test results and results by NMG also indicate that the near surface soils have a "very low" ($0 \leq EI \leq 20$) potential for expansion in accordance with ASTM D 4829. However, verification testing should be performed after site remedial grading.

The foundation elements for the proposed structures should bear entirely in engineered fill soils and should be designed in accordance with the *2019 CBC*.

Because of the potential for liquefaction induced settlement and surface manifestations of liquefaction, it is our recommendation that foundation systems such as shallow footings with foundation ties, post-tensioned slabs, or mat foundations be used to support the planned buildings. After the completion of the recommended remedial grading, we anticipate a total static settlement of less than 1-inch and a maximum differential static settlement of less than 0.5-inch in a 30-foot span for residential buildings resting on shallow footings with foundation ties. For structures resting on mat foundations or post-tensioned systems, a total settlement of about 2 inches and a differential settlement of about 1 inch over a horizontal distance of 30 feet are estimated. These static settlements along with the anticipated seismically induced settlements will result in up to 2.5 inches of combined (static plus seismic) total settlement and up to 1.3 inches of combined (static plus seismic) differential settlements over a horizontal distance of 30 feet for the future residential structures resting on shallow footings with foundation ties. For structures resting on mat foundations or post-tensioned systems, up to 3.5 inches of combined (static plus seismic) total settlement and up to 1.8 inches of combined (static plus seismic) differential settlements over a horizontal distance of 30 feet are estimated.

Shallow Footings with Foundation Ties

A summary of our foundation design recommendations is presented in the following table.

MINIMUM DESIGN REQUIREMENTS FOR CONVENTIONALLY REINFORCED SHALLOW FOUNDATIONS	
Design Parameter	"Very Low" Expansion Potential
Foundation Depth or Minimum Perimeter Beam Depth (inches below lowest adjacent grade)	One-and Two-Story – 12
Minimum Foundation Width (Inches)*	One-story and Two-Story – 12
Minimum Slab Thickness (Inches)	4 – Actual
Minimum Slab Reinforcing	6" x 6" – W1.4/W1.4 welded wire fabric placed in middle of slab
Minimum Reinforcement for Continuous Footings, Grade Beams, and Retailing Wall Footings	Two No. 4 reinforcing bars, one placed near the top and one near the bottom
Effective Plasticity Index	NA**
Presaturation of Subgrade Soil (Percent of Optimum/Depth in Inches)	Minimum 100% of the optimum moisture content to a depth of at least 12 inches prior to placing concrete

*Code minimums per Table 1809.7 of the 2019 CBC should be complied with
**Effective Plasticity Index should be verified at the completion of the site remedial grading

These are minimal recommendations and are not intended to supersede the design by the project structural engineer. In addition, design of foundations on liquefiable sites should follow the provisions in ASCE 7-16 Section 12.13.9. Per ASCE 7-16 Section 12.13.9, shallow foundations underlain by potentially liquefiable soils are required to be interconnected by ties so that the differential settlement is reduced. Foundation ties are required when the estimated differential settlement exceeds one-fourth of the differential settlement threshold specified by Table 12.13-3 of ASCE 7-16.

In general, an allowable bearing capacity of 1,500 psf may be used for design of continuous and perimeter footings 12 inches deep and 12 inches wide, and pad footings 24 inches square and 12 inches deep. Additionally, an increase of one-third may be applied when considering short-term live loads (e.g. seismic and wind loads).

The passive earth pressure may be computed as an equivalent fluid having a density of 225 psf per foot of depth, to a maximum earth pressure of 2,500 psf for footings founded in engineered fill. A coefficient of friction between engineered fill and concrete of 0.40 may be used with dead load forces. The upper one foot of soil below the adjacent grade should not be used in calculating passive pressure. When combining passive pressure and frictional resistance, the passive pressure component should be reduced by one-third.

Post-Tensioned Slabs

The slab designer may choose the post-tension design methodology. Since the CBC indicated Post Tensioning Institute (PTI) design methodology is intended for expansive soils conditions which do not apply, no em or ym parameters as used in the PTI methodology are provided. However, the slab design should consider the estimated static and liquefaction induced settlements as noted above.

MINIMUM DESIGN REQUIREMENTS FOR POST-TENSIONED FOUNDATIONS	
Foundation Design Parameter	"Very Low" Expansion Potential
Foundation Depth or Minimum Perimeter Beam Depth (inches below lowest adjacent grade)	One- or Two-Story – 12 inches
Minimum Foundation Width	One- or Two-Story – 12 inches
Minimum Slab Thickness (actual)	5 inches
Presaturation of Subgrade Soil (Percent of Optimum)	Minimum 100% to a depth of 12 inches

It should be noted that the above recommendations are based on soil support characteristics only. The structural engineer should design the slab and beam reinforcement based on actual loading conditions.

An allowable bearing capacity of 1,000 psf may be used for design of post-tensioned slab foundations.

The passive earth pressure may be computed as an equivalent fluid having a density of 225 psf per foot of depth, to a maximum earth pressure of 2,500 psf for footings founded on engineered fill. A coefficient of friction between soil and concrete of 0.40 may be used with dead load forces. The upper one foot of soil below the adjacent grade should not be used in calculating passive pressure.

Mat Foundations

The mat foundations should have a minimum embedment depth of 12 inches and may be designed using an allowable bearing capacity of 1,000 psf. Reinforcement within the mat foundation should be determined by the structural engineer. Structural design should consider the estimated static and liquefaction induced settlements as noted above.

For resistance to lateral loads, an allowable coefficient of friction of 0.40 between the base of the foundation elements and underlying compacted fill material is recommended. In addition, an allowable passive resistance equal to an equivalent fluid density of 225 pcf acting against the foundations may be used to resist lateral forces. The top foot of passive resistance at foundations should be neglected unless the ground surface is covered by concrete or pavement.

Where the mat foundation is to be designed as a beam on an elastic foundation, it is our opinion that a modulus of subgrade reaction (k-value) of 200 pounds per cubic inch (pci) may be considered for design based on a presumed value for a 1-foot by 1-foot plate load test. Dependent upon how the mat slab is loaded, the subgrade modulus value may need to be geometrically modified.

A moisture and vapor retarding system should be placed below slabs-on-grade where moisture migration through the slab is undesirable. Guidelines for these are provided in the 2019 California Green Building Standards Code (CALGreen) Section 4.505.2 and the 2019 CBC Section 1907.1 and ACI 360R-10. The vapor retarder design and construction should also meet the requirements of ASTM E1643. A portion of the vapor retarder design should be the implementation of a moisture vapor retardant membrane.

It should be realized that the effectiveness of the vapor retarding membrane can be adversely impacted as a result of construction related punctures (e.g. stake penetrations, tears, punctures from walking on the aggregate layer, etc.). These occurrences should be limited as much as possible during construction. Thicker membranes are generally more resistant to accidental puncture than thinner ones. Products specifically designed for use as moisture/vapor retarders may also be more puncture resistant. Although the CBC specifies a six-mil vapor retarder membrane, it is GeoTek's opinion that a minimum ten mil thick membrane with joints properly overlapped and sealed should be considered, unless otherwise specified by the slab design professional. The membrane should consist of Stego wrap or the equivalent.

Moisture and vapor retarding systems are intended to provide a certain level of resistance to vapor and moisture transmission through the concrete, but do not eliminate it. The acceptable level of moisture transmission through the slab is to a large extent based on the type of flooring used and environmental conditions. Ultimately, the vapor retarding system should be comprised of suitable elements to limit migration of water and reduce transmission of water vapor through the slab to acceptable levels. The selected elements should have suitable properties (i.e., thickness, composition, strength, and permeability) to achieve the desired performance level. Consideration should be given to consulting with an individual possessing specific expertise in this area for additional evaluation.

Moisture retarders can reduce, but not eliminate, moisture vapor rise from the underlying soils up through the slab. Moisture retarders should be designed and constructed in accordance with applicable American Concrete Institute, Portland Cement Association, Post-Tensioning Concrete Institute, ASTM and California Building Code requirements and guidelines.

GeoTek recommends that a qualified person, such as the flooring contractor, structural engineer, and/or architect be consulted to evaluate the general and specific moisture vapor transmission paths and associated potential impact.

In addition, the recommendations in this report and our services in general are not intended to address mold prevention, since we along with geotechnical consultants in general, do not practice in areas of mold prevention. If specific recommendations are desired, a professional mold prevention consultant should be contacted.

6.3.2 Miscellaneous Foundation Recommendations

- To minimize moisture penetration beneath the slab on grade areas, utility trenches should be backfilled with engineered fill, lean concrete, or concrete slurry where they intercept the perimeter footing or thickened slab edge.
- Soils from the footing excavations should not be placed in the slab-on-grade areas unless properly compacted and tested. The excavations should be free of loose/sloughed materials and be neatly trimmed at the time of concrete placement.
- Under-slab utility trenches should be compacted to project specifications. Compaction should be achieved with a mechanical compaction device. If backfill soils have dried out, they should be thoroughly moisture conditioned prior to placement in trenches.

6.3.3 Foundation Set Backs

Foundations should comply with the following setbacks. Improvements not conforming to these setbacks are subject to the increased likelihood of excessive lateral movements and/or differential settlements. If large enough, these movements can compromise the integrity of the improvements. The following recommendations are presented:

- The outside bottom edge of all footings should be set back a minimum of H/2 (where H is the slope height) from the face of any ascending slope. The setback should be at least 5 feet and need not to exceed 15 feet. Where a retaining wall is constructed at the toe of the slope, the height of the slope should be measured from top of the wall to the top of the slope.

- The outside bottom edge of all footings should be set back a minimum of H/3 from the face of any descending slope. The setback should be at least 7 feet and need not exceed 40 feet.
- The bottom of all footings for structures near retaining walls should be deepened so as to extend below a 1:1 (h:v) projection upward from the bottom inside edge of the wall footing.
- The bottom of any existing foundations for structures should be deepened so as to extend below a 1:1 (h:v) projection upward from the bottom of the nearest excavation.

6.3.4 Retaining Wall Design and Construction

6.3.4.1 General Design Criteria

Recommendations presented in this report apply to typical masonry or concrete retaining walls with a maximum retained soil height of 6 feet. Additional review and recommendations should be requested for higher walls. These are typical design criteria and are not intended to supersede the design by the structural engineer.

Retaining wall foundations should be embedded a minimum of 12 inches into engineered fill. Walls should be designed using an allowable bearing capacity of 1,500 psf. An increase of one-third may be applied when considering short-term live loads (e.g. seismic and wind loads). The passive earth pressure may be computed as an equivalent fluid having a density of 225 psf per foot of depth, to a maximum earth pressure of 2,500 psf. A coefficient of friction between soil and concrete of 0.40 may be used with dead load forces. When combining passive pressure and frictional resistance, the passive pressure component should be reduced by one-third.

All earth retention structure plans, as applicable, should be reviewed by this office prior to finalization. The seismic design parameters as discussed in this report remain applicable to all proposed earth retention structures at this site, and should be properly incorporated into the design and construction of the structures.

Earthwork considerations, site clearing and remedial earthwork for all earth retention structures should meet the requirements of this report, unless specifically provided otherwise, or more stringent requirements or recommendations are made by the designer. The backfill material placement for all earth retention structures should meet the requirement of Section 6.3.4.4 in this report.

In general, cantilever earth retention structures, which are designed to yield at least 0.001H, where H is equal to the height of the earth retention structure to the base of its footing, may be designed using the active condition. Rigid earth retention structures (including but not limited to rigid walls, and walls braced at top, such as typical basement walls) should be designed using the at-rest condition.

In addition to the design lateral forces due to retained earth, surcharges due to improvements, such as an adjacent building or traffic loading, should be considered in the design of the earth retention structures. Loads applied within a 1:1 (h:v) projection from the surcharge on the stem and footing of the earth retention structure should be considered in the design.

Final selection of the appropriate design parameters should be made by the designer of the earth retention structures.

6.3.4.2 Cantilevered Walls

The recommendations presented below are for cantilevered walls retaining up to 6 feet of soil. Active earth pressure may be used for retaining wall design, provided the top of the wall is not restrained from minor deflections. An equivalent fluid pressure approach may be used to compute the horizontal pressure against the wall. Appropriate fluid unit weights are given below for specific slope gradients of the retained material. These do not include other superimposed loading conditions such as traffic, structures, or adverse geologic conditions.

ACTIVE EARTH PRESSURES	
Surface Slope of Retained Materials (h:v)	Equivalent Fluid Pressure (pcf) (Native Backfill)*
Level	37
2:1	60

* The design pressures assume the backfill material has an expansion index less than or equal to 20. Backfill zone includes area between back of the wall to a plane (1:1 h:v) up from bottom of the wall foundation (on the backside of the wall) to the (sloped) ground surface.

6.3.4.3 Restrained Retaining Walls

Retaining walls that will be restrained at the top that support level backfill or that have reentrant or male corners, should be designed for an equivalent at-rest fluid pressure of 60 pcf, plus any applicable surcharge loading for level backfill conditions. For areas of male or reentrant corners, the restrained wall design should extend a minimum distance of twice the height of the wall laterally from the corner, or a distance otherwise determined by the project structural engineer.

6.3.4.4 Retaining Wall Backfill and Drainage

Retaining walls should be provided with an adequate pipe and gravel back drain system to help prevent buildup of hydrostatic pressures. Backdrains should consist of a four-inch diameter perforated collector pipe (Schedule 40, SDR 35, or approved equivalent) embedded in a minimum of one-cubic foot per linear foot of ¾- to 1-inch clean crushed rock or an approved equivalent, wrapped in filter fabric (Mirafi 140N or an approved equivalent). The drain system should be connected to a suitable outlet. Waterproofing of site walls should be performed where moisture migration through the wall is undesirable.

Retaining wall backfill should be placed in lifts no greater than eight inches in thickness and compacted to a minimum of 90 percent relative compaction in accordance with ASTM Test Method D 1557. The wall backfill should also include a minimum one-foot wide section of ¾- to 1-inch clean crushed rock (or an approved equivalent). The rock should be placed immediately adjacent to the back of the wall and extend up from a back drain to within approximately two feet of the finish grade. The rock should be separated from the earth with filter fabric. The upper two feet should consist of compacted on-site soil.

As an alternative to the drain rock and fabric, Miradrain 2000, or approved equivalent, may be used behind the retaining wall. The Miradrain 2000 should extend from the base of the wall to within two feet of the ground surface. The subdrain should be placed at the base of the wall in direct contact with the Miradrain 2000.
The presence of other materials might necessitate revision to the parameters provided and modification of the wall designs. Proper surface drainage needs to be provided and maintained.

6.3.4.5 Other Design Considerations

- Wall design should consider the additional surcharge loads from superjacent slopes and/or footings, where appropriate.
- No backfill should be placed against concrete until minimum design strengths are evident by compression tests of cylinders.
- The retaining wall footing excavations, backcuts, and backfill materials should be approved the project geotechnical engineer or their authorized representative.

6.3.5 Pool Construction

Because of the presence of relatively shallow groundwater, dewatering systems may be required to facilitate the excavation of the pool area. We recommend that the water table be lowered to a least two feet below the deepest excavation and maintained at that depth until the pool is constructed and filled.

The proposed swimming pool should derive support entirely from engineered fill. A minimum 12 inches of engineered fill should be provided below the pool shell.

The pool walls should be designed for at-rest soil conditions using an equivalent fluid density of 60 pcf for at-rest conditions. Pool walls surcharged by adjacent structures should be designed for additional pressures. Alternatively, the pool walls may be designed as freestanding walls using the active soil state conditions provided that some lateral movement of the pool walls would be acceptable. If the active state is to be used, an equivalent fluid density of 37 pcf is considered suitable. These pressures are recommended for sections of the pool walls above the groundwater table and are based on drained conditions. Below the groundwater level (about six to eight feet), the pool walls should then be designed for an equivalent fluid density of 83 pcf for the active condition and 94 pcf for the at-rest condition. These values include the hydrostatic pressure.

Due to the high groundwater table under the site, positive drainage below the pool may not be feasible. We recommend that the pool walls be designed to include the hydrostatic pressure as indicated above. Also, buoyancy of the pool should be evaluated by the pool designer using a groundwater level of about five feet below the existing ground surface. The pool designer should consider hydrostatic relief valves or equivalent in order to prevent the effects of hydrostatic pressure on an empty pool shell.

Pool decking supported on grade should be separated from the pool bond beam by a full-depth, mastic construction joint. If it is desired to extend the pool deck over the bond beam, consideration should be given to designing the deck as a structural slab supported by the pool shell. This will reduce the possibility of deck cracking occurring along the outer edge of the bond beam. We also recommend that the area of the pool decking be pre-saturated prior to concrete placement. The subgrade soils should be moisture conditioned to at least 100 percent of the soil's optimum moisture content to a depth of 12 inches, prior to concrete placement. Testing by the geotechnical engineer is recommended to confirm that the soils have been adequately moisture treated.

Pool decking may consist of five-inch thick concrete and the use of reinforcement is suggested. Control joints should be placed in two directions and located a distance apart approximately equal to 24 to 36 times the slab thickness. The project structural engineer should provide final design recommendations.

6.3.6 Underground Utility Considerations

Due to the high groundwater table under the site and in the vicinity, underground utilities deeper than 5 feet should consider buoyancy in their design.

6.3.7 Pavement Design Considerations

Pavement design for proposed street improvements was conducted per Caltrans *Highway Design Manual* guidelines for flexible pavements. Based on a design R-value of 50 (NMG, 2011) for Traffic Indices (TIs) of 5.0 and 6.0 generally linked to roads with light vehicular traffic with occasional heavy truck traffic, the following sections were calculated:

PRELIMINARY PAVEMENT SECTIONS		
Traffic Index	Thickness of Asphalt Concrete (feet)	Thickness of Aggregate Base (Feet)
5.0	0.25*	0.50*
6.0	0.25*	0.50*

*Minimum thickness per City of Cypress Street Standards.

Traffic Indices (TIs) used in our pavement design are considered reasonable values for the proposed pavement areas and should provide a pavement life of approximately 20 years with a normal amount of flexible pavement maintenance. Irrigation adjacent to pavements, without a deep curb or other cutoff to separate landscaping from the paving will result in premature pavement failure. Traffic parameters used for design were selected based upon engineering judgment and not upon information furnished to us such as an equivalent wheel load analysis or a traffic study.

The recommended pavement sections provided are intended as a minimum guideline and final selection of pavement cross section parameters should be made by the project civil engineer, based upon the local laws and ordinates, expected subgrade and pavement response, and desired level of conservatism. If thinner or highly variable pavement sections are constructed, increased maintenance and repair could be expected. Final pavement design should be checked by testing of soils exposed at subgrade (the upper one foot) after final grading has been completed.

Asphalt concrete and aggregate base should conform to current Caltrans Standard Specifications Section 39 and 26-1.02, respectively. As an alternative, asphalt concrete can conform to Section 203-6 of the current Standard Specifications for Public Work (Green Book). Crushed aggregate base or crushed miscellaneous base can conform to Section 200-2.2 and 200-2.4 of the Green Book, respectively. Pavement base should be compacted to at least 95 percent of the ASTM D1557 laboratory maximum dry density (modified proctor).

All pavement installation, including preparation and compaction of subgrade, compaction of base material, placement and rolling of asphaltic concrete, should be done in accordance with the City of Cypress specifications, and under the observation and testing of GeoTek and a City Inspector where required. Jurisdictional minimum compaction requirements in excess of the aforementioned minimums may govern.

Deleterious material, excessive wet or dry pockets, oversized rock fragments, and other unsuitable yielding materials encountered during grading should be removed. Once existing compacted fill are brought to the proposed pavement subgrade elevations, the subgrade should be proof-rolled in order to check for a uniform and unyielding surface. The upper 12 inches of pavement subgrade soils should be scarified, moisture conditioned at or near optimum moisture content, and recompacted to at least 95 percent of the laboratory maximum dry density (ASTM D1557). Rock fragments over six inches in one dimensions should not be placed within the upper 12 inches of the subgrade. If loose or yielding materials are encountered during construction, additional evaluation of these areas should be carried out by GeoTek. All pavement section changes should be properly transitioned.

6.3.8 Soil Corrosivity

The soil resistivity was tested in the laboratory on two samples collected during our field exploration. The results of the testing (5,226 and 8,040 ohm-cm) indicate that the tested soil samples are "moderately corrosive" to buried metals, based on the guidelines provided in *Corrosion Basics: An Introduction* (Roberge, 2005). Soil resistivity testing performed by NMG (2011) revealed a "severely" corrosive to "moderately" corrosive category for the on-site materials (73 to 2,320 ohm-cm).

Chloride content of the samples tested by GeoTek (20 and 126 ppm) was found to be negligible. Chloride concentration measured by NMG (2011) was noted to range from "negligible" to "corrosive" (11 to 685 ppm). Consideration should be given to consulting with a corrosion engineer.

6.3.9 Soil Sulfate Content

The sulfate content was determined in the laboratory for two soil samples obtained during our field investigation. The results (0.0003 and 0.0054 percent) indicate that the tested water-soluble sulfate is negligible, per Table 4.2.1 of ACI 318. NMG reported similar findings regarding the soil sulfate content. Based upon the test results, no special concrete mix design is required by Code for sulfate attack resistance. Additional sampling and testing should be performed once the site grading is complete.

6.3.10 Import Soils

Import soils should have an Expansion Index of less than 20 (very low) and should not possess oversized or deleterious materials. GeoTek also recommends that, as a minimum, proposed import soils be tested for soluble sulfate content. GeoTek should be notified a minimum of 72 hours of potential import sources so that appropriate sampling and laboratory testing can be performed.

6.3.11 Concrete Flatwork

6.3.11.1 Exterior Concrete Slabs, Sidewalks, and Driveways

Exterior concrete slabs, sidewalks, and driveways should be designed using a four-inch minimum thickness. No specific reinforcement is required due to the non-structural nature. However, the use of some reinforcement should be considered. Recommendations can be provided upon request. Some shrinkage and cracking of the concrete should be anticipated as a result of typical mix designs and curing practices commonly utilized in residential construction.

Sidewalks and driveways may be under the jurisdiction of the governing agency. If so, jurisdictional design and construction criteria would apply, if more restrictive than the recommendations presented herein.

Subgrade soils, classified as having "very low" expansion potential, should be pre-moistened prior to placing concrete. The subgrade soils below exterior slabs, sidewalks, driveways, etc. at the subject site should be pre-saturated to a minimum of 100 percent of optimum moisture content to a depth of 12 inches.

All concrete installation, including preparation and compaction of subgrade, should be done in accordance with the City of Cypress/County of Orange specifications, and under the observation and testing of GeoTek and a City Inspector, if necessary.

6.3.11.2 Concrete Performance

Concrete cracks should be expected. These cracks can vary from sizes that are essentially unnoticeable to more than 1/8 inch in width. Most cracks in concrete, while unsightly, do not significantly impact long-term performance. While it is possible to take measures (proper concrete mix, placement, curing, control joints, etc.) to reduce the extent and size of cracks that occur, some cracking will occur despite the best efforts to minimize it. Concrete can also undergo chemical processes that are dependent on a wide range of variables, which are difficult, at best, to control. Concrete, while seemingly a stable material, is also subject to internal expansion and contraction due to external changes over time.

One of the simplest means to control cracking is to provide weakened control joints for cracking to occur along. These do not prevent cracks from developing; they simply provide a relief point for the stresses that develop. These joints are a widely accepted means to control cracks but are not always effective. Control joints are more effective the more closely spaced they are. GeoTek suggests that control joints be placed in two directions and located a distance apart roughly equal to 24 to 36 times the slab thickness.

Exterior concrete flatwork (patios, walkways, driveways, etc.) is often some of the most visible aspects of site development. They are typically given the least level of quality control, being considered "non-structural" components. We suggest that the same standards of care be applied to these features as to the structure itself.

6.4 POST CONSTRUCTION CONSIDERATIONS

6.4.1 Landscape Maintenance and Planting

Water has been shown to weaken the inherent strength of soil, and slope stability is significantly reduced by overly wet conditions. Positive surface drainage away from graded slopes should be maintained and only the amount of irrigation necessary to sustain plant life should be provided for planted slopes. Controlling surface drainage and runoff, and maintaining a suitable vegetation cover can minimize erosion. Plants selected for landscaping should be lightweight, deep-rooted types that require little water and are capable of surviving the prevailing climate.

Overwatering should be avoided. The soils should be maintained in a solid to semi-solid state as defined by the materials Atterberg Limits. Care should be taken when adding soil amendments to avoid excessive watering. Leaching as a method of soil preparation prior to planting is not recommended. An abatement program to control ground-burrowing rodents should be implemented and maintained. This is critical as burrowing rodents can decreased the long-term performance of slopes.

It is common for planting to be placed adjacent to structures in planter or lawn areas. This will result in the introduction of water into the ground adjacent to the foundation. This type of landscaping should be avoided. If used, then extreme care should be exercised with regard to the irrigation and drainage in these areas.

6.4.2 Drainage

The need to maintain proper surface drainage and subsurface systems cannot be overly emphasized. Positive site drainage should be maintained at all times. Drainage should not flow uncontrolled down any descending slope. Water should be directed away from foundations and not allowed to pond or seep into the ground. Pad drainage should be directed toward approved area(s) and not be blocked by other improvements.

It is the owner's responsibility to maintain and clean drainage devices on or contiguous to their lot. In order to be effective, maintenance should be conducted on a regular and routine schedule and necessary corrections made prior to each rainy season.

6.5 PLAN REVIEW AND CONSTRUCTION OBSERVATIONS

We recommend that site grading plans, pool plans, retaining wall plans, foundation plans, and relevant project specifications be reviewed by this office prior to construction to check for conformance with the recommendations of this report. We also recommend that GeoTek representatives be present during site grading and foundation construction to check for proper implementation of the geotechnical recommendations. The owner/developer should verify that GeoTek representatives perform at least the following duties:

- Observe site clearing and grubbing operations for proper removal of unsuitable materials.
- Observe and test bottom of removals prior to fill placement.
- Evaluate the suitability of onsite and import materials for fill placement, and collect soil samples for laboratory testing where necessary.
- Observe the fill for uniformity during placement, including utility trenches.
- Perform field density testing of the fill materials.
- Observe and probe foundation excavations to confirm suitability of bearing materials.

If requested, a construction observation and compaction report can be provided by GeoTek, which can comply with the requirements of the governmental agencies having jurisdiction over the project. We recommend that these agencies be notified prior to commencement of construction so that necessary grading permits can be obtained.

7. INTENT

It is the intent of this report to aid in the design and construction of the proposed development. Implementation of the advice presented in Section 6 of this report is intended to reduce risk associated with construction projects. The professional opinions and geotechnical advice contained in this report are not intended to imply total performance of the project or guarantee that unusual or variable conditions will not be discovered during or after construction.

The scope of our evaluation is limited to the boundaries of the subject site. This review does not and should in no way be construed to encompass any areas beyond the specific area of the proposed construction as indicated to us by the client. Further, no evaluation of any existing site improvements is included. The scope is based on our understanding of the project and the client's needs, our fee estimate (P-1006220-CR) dated October 15, 2020 and geotechnical engineering standards normally used on similar projects in this region.

8. LIMITATIONS

The materials observed on the project site appear to be representative of the area; however, soil materials vary in character between excavations or conditions exposed during site construction. Site conditions may vary due to seasonal changes or other factors. GeoTek, Inc. assumes no responsibility or liability for work, testing or recommendations performed or provided by others.

Since our recommendations are based on the site conditions observed and encountered, and laboratory testing, our conclusion and recommendations are professional opinions that are limited to the extent of the available data. Observations during construction are important to allow for any change in recommendations found to be warranted. These opinions have been derived in accordance with current standards of practice and no warranty is expressed or implied. Standards of practice are subject to change with time.

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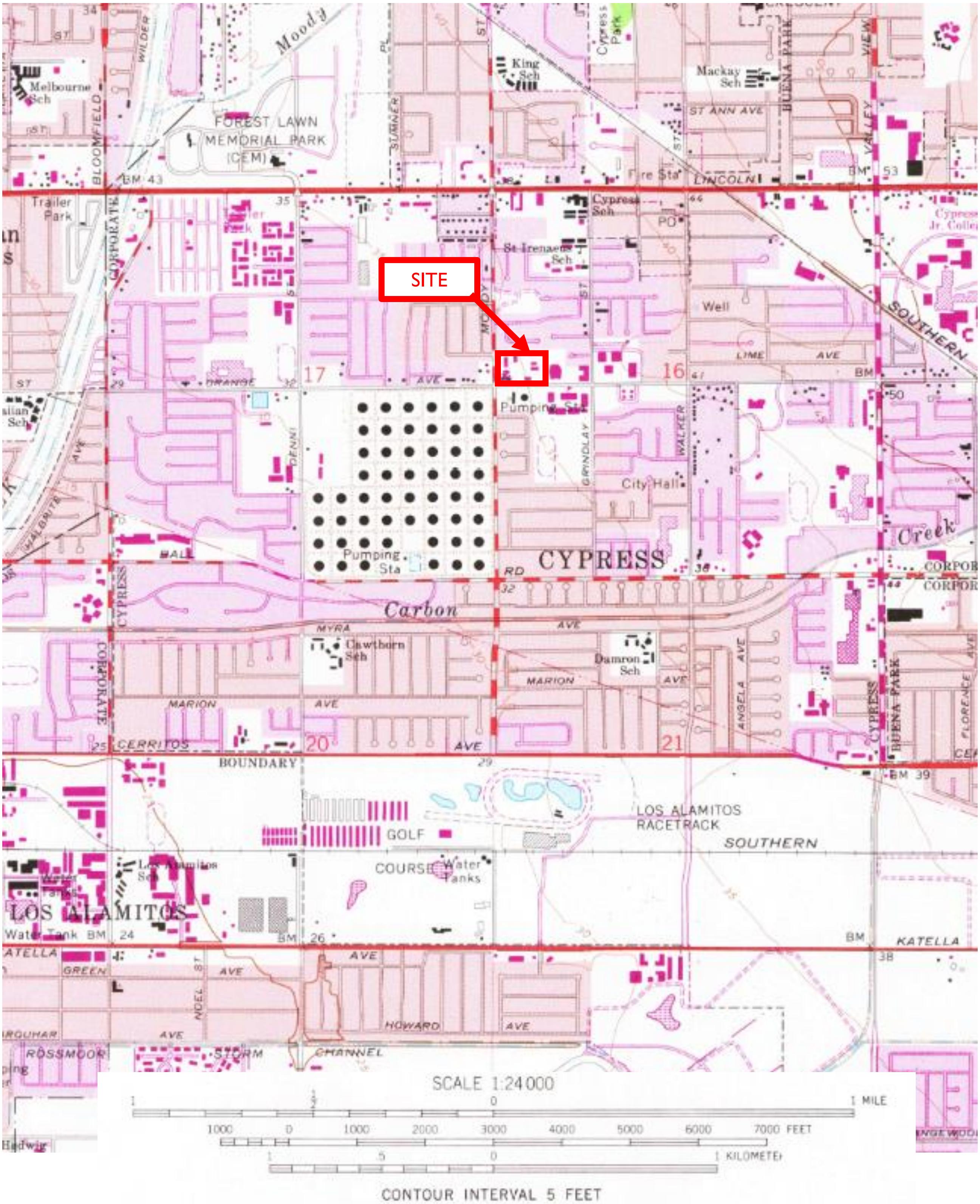

Melia Homes
Orange Avenue Seniors
9470 Moody Street and 5081 Orange Avenue
City of Cypress, Orange County, California

Project No. 2573-CR

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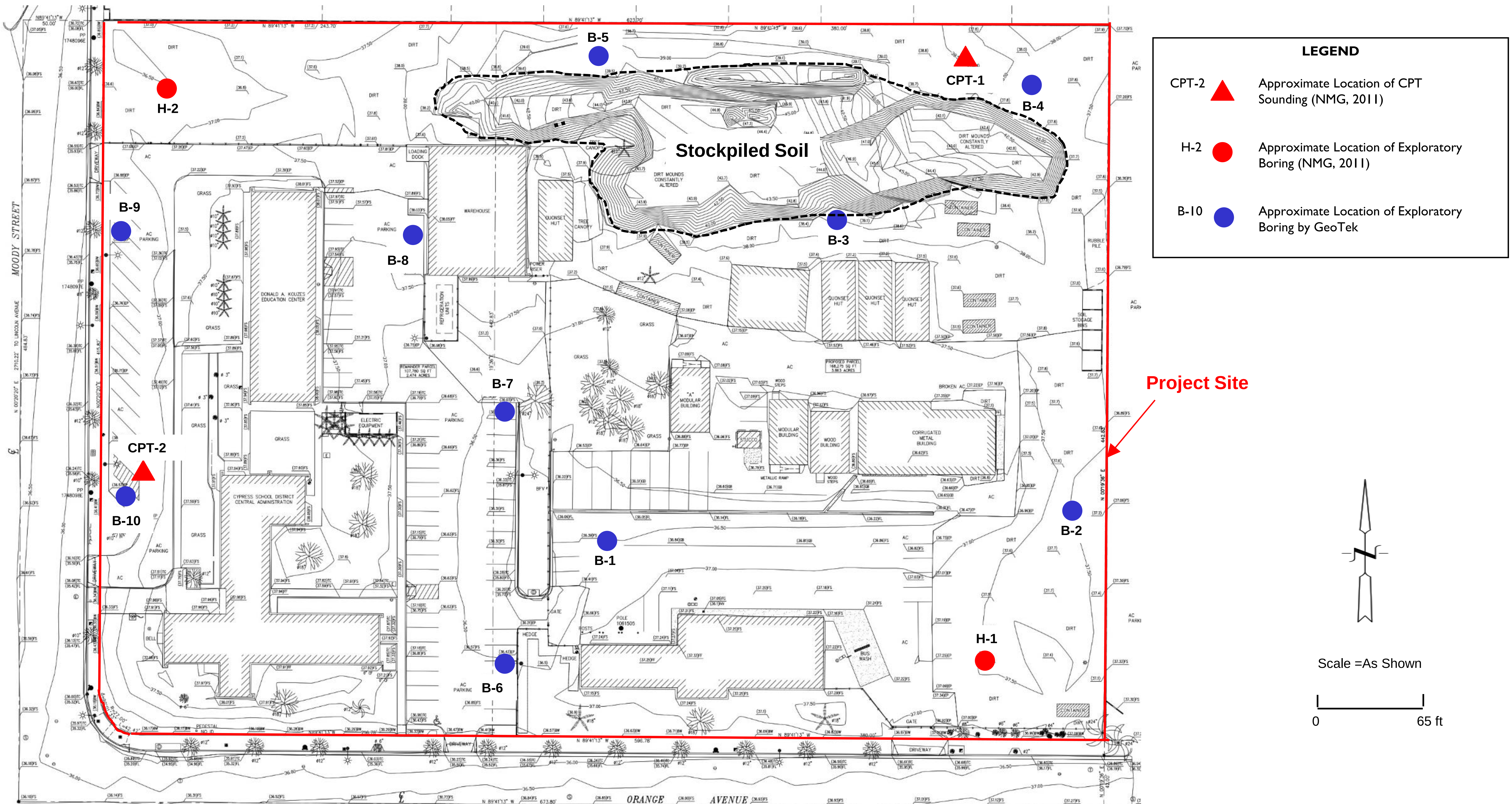
Modified from USGS 7.5-minute Los Alamitos Topographic Map

Figure 1
Site Location Map

GEOTEK



Melia Homes
Orange Avenue Seniors
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Figure 2

Exploration Location Map

APPENDIX A

BORING AND CPT LOGS

LABORATORY TEST RESULTS BY NMG (2011)

Updated Geotechnical Evaluation

Orange Avenue Seniors, Cypress, California

Project No. 2573-CR

GEOTEK

Date(s) Drilled	10/4/11	Logged By	LC
Drilling Company	2R Drilling	Drill Bit Size/Type	8"
Drill Rig Type	CME 55	Hammer Data	140 llbs @ 30"
Sampling Method(s)	Modified California, Bulk		
Approximate Groundwater Depth:	Groundwater at 8.25 Feet after 3 Hours.		
Comments			

H-1

Total Depth Drilled (ft)	51.5
Approximate Ground Surface Elevation (ft)	37.0

Elevation (ft)	Depth (ft)	SAMPLES Type	SAMPLES Number	SAMPLES Blows per foot	Graphic Log	USCS	MATERIAL DESCRIPTION	Moisture Content (%)	Dry Density (pcf)	OTHER TESTS and REMARKS
	0					SM	@Surface: 1/2" Crushed rock and rounded gravel. **Alluvium (Qal)**			B-1 (0'-5')
			B-1							
			D-1	25			@2.5 Grayish brown silty SAND, moist, medium dense, scattered gravel in top, slightly porous.	13.8	105.9	
							@4' Pale yellowish brown SAND and silty SAND.			
	5		D-2	10			@5' Grayish brown silty bSAND, moist, loose, friable, micaceous.	21.6	78.3	
30										
			D-3	3			@7.5' Gray silty SAND, very moist, loose, friable, micaceous, slightly porous.	31.5	88.5	CN
	10		D-4	11		SM/SC	@10' Grayish brown silty clayey SAND, very moist to saturated, loose, friable, micaceous, slightly porous.	26.3	96.7	
	15		D-5	17		SP/SM	@15' Pale yellowish brown silty SAND and poorly graded SAND, saturated, loose to medium dense, friable, micaceous.	27.6	95.1	GS
20										
	20		D-6	21			@20' Olive gray silty SAND and fine grained SAND, saturated, medium dense, friable, micaceous, slightly bedded.	25.9	99.8	
	25		D-7	18		SM/ML	@25' Dark olive gray SILT and very fine grained SAND with some clay, very moist to saturated, medium stiff / medium dense, micaceous, friable.	24.1	102.2	
10										
	30									

LOG OF BORING

CSD/ District Office
Cypress, California
PROJECT NO. 11050-02

NMG

CSD/ District Office Cypress, California

H-1

Elevation (ft)	Depth (ft)	SAMPLES Type	SAMPLES Number	SAMPLES Blows per foot	Graphic Log	USCS	MATERIAL DESCRIPTION	Moisture Content (%)	Dry Density (pcf)	OTHER TESTS and REMARKS
	30		D-8	16		SP/SM	@30' Dark olive gray SAND and silty SAND, saturated, loose to medium dense, highly micaceous, slightly porous, scattered fine organic fibers.	19.7	109.5	
	35		D-9	25			@35' Dark olive gray SAND and silty SAND, medium dense, saturated, friable, highly micaceous.	23.4	102.3	
0										
	40		D-10	11			@40' Pale yellowish brown SAND, very moist to saturated, loose, micaceous, layers of silty SAND.	20.2	105.2	
	45		D-11	70		SP	@45' Light olive gray SAND with scattered pebbles, saturated, dense, friable, micaceous.	23.0	104.4	
-10										
	50		D-12	28		SM/ML	@50' Olive gray SILT and silty fine grained SAND, saturated, stiff / medium dense, highly micaceous, scattered small gravel.	24.1	101.0	
							Notes: Total Depth: 51.5 Feet. Groundwater at 8.25 Feet after 3 Hours. Backfilled With Cuttings, Bentonite and Sand Mix.			
	55									
-20										
	60									
	65									

LOG OF BORING
CSD/ District Office
Cypress, California

PROJECT NO. 11050-02

NMG

Date(s) Drilled	10/4/11	Logged By	LC
Drilling Company	2R Drilling	Drill Bit Size/Type	8"
Drill Rig Type	CME 55	Hammer Data	140 llbs @ 30"
Sampling Method(s)	Modified California, Bulk		
Approximate Groundwater Depth:	Groundwater Encountered at 10 Feet.		
Comments			

H-2

Total Depth Drilled (ft)	51.5
Approximate Ground Surface Elevation (ft)	37.0

Elevation (ft)	Depth (ft)	SAMPLES Type	Number	Blows per foot	Graphic Log	USCS	MATERIAL DESCRIPTION	Moisture Content (%)	Dry Density (pcf)	OTHER TESTS and REMARKS
	0					SM/SC	@Surface: Grass **Alluvium (Qal)**	8.4		B-1 (0'-5) AL, CC, EI, GS, MD, RV
			B-1 D-1	13			@2.5' Pale brown silty and clayey fine grained SAND and sandy SILT, dry to damp, loose / medium stiff, friable, micaceous, scattered root hairs.	5.5	97.7	
	5		D-2	11		ML	@5' Olive gray SILT and clayey SILT, very moist, medium stiff, highly micaceous.	25.7	93.9	CN, DS
30										
			D-3	8		SC/CL	@7.5' Gray clayey SAND / sandy CLAY, saturated, loose / soft to medium stiff, highly micaceous.	28.6	100.6	
	10		D-4	8		CL	@10' Gray sandy CLAY, saturated, soft to medium stiff, highly micaceous, slightly porous, scattered pebbles to 1/4".	31.5	92.8	GS
	15		D-5	18		SM	@15' Olive gray silty fine grained SAND, very moist to saturated, loose to medium dense, friable, highly micaceous.	25.8	96.7	
20										
	20		D-6	27		SM/ML	@20' Olive gray silty fine grained SAND and sandy SILT, saturated, medium dense / stiff, highly micaceous.	25.2	100.8	
	25		D-7	19		SC/ML	@25' Olive gray clayey SAND and sandy SILT, saturated, medium dense / stiff, micaceous, slightly porous.	27.1	102.7	
10										
	30									

Report: HOLLOW STEM; Project: P:\2011\11050-02\GINTW\11050-02-DISTRICT.GPJ; Data Template: NMGNOV98.GDT; Printed: 10/28/11

Elevation (ft)	Depth (ft)	SAMPLES Type	Number	Blows per foot	Graphic Log	USCS	MATERIAL DESCRIPTION	Moisture Content (%)	Dry Density (pcf)	OTHER TESTS and REMARKS
	30		D-8	16		SC/CL	@30' Olive gray silty CLAY and clayey SAND, very moist to saturated, stiff / medium dense, highly micaceous, scattered calcium carbonate nodules.	23.5	99.6	
	35		D-9	17		SC	@35' Olive brown clayey SAND, saturated, medium dense, micaceous.	21.8	108.2	
0										
	40		D-10	25		SP/SM	@40' Olive gray SAND and silty fine grained SAND, saturated, medium dense, highly micaceous, friable.	24.5	98.0	
	45		D-11	28		SP	@45' Olive gray SAND, saturated, medium dense, friable, micaceous.	20.8	106.0	
-10										
	50		D-12	23		SC/ML	@50' Olive gray SILT and clayey fine grained SAND, very moist, stiff / medium dense, highly micaceous.	26.4	98.4	
							Notes: Total Depth: 51.5 Feet. Groundwater Encountered at 10 Feet. Backfilled With Cuttings, Bentonite and Sand Mix.			
	55									
-20										
	60									
	65									

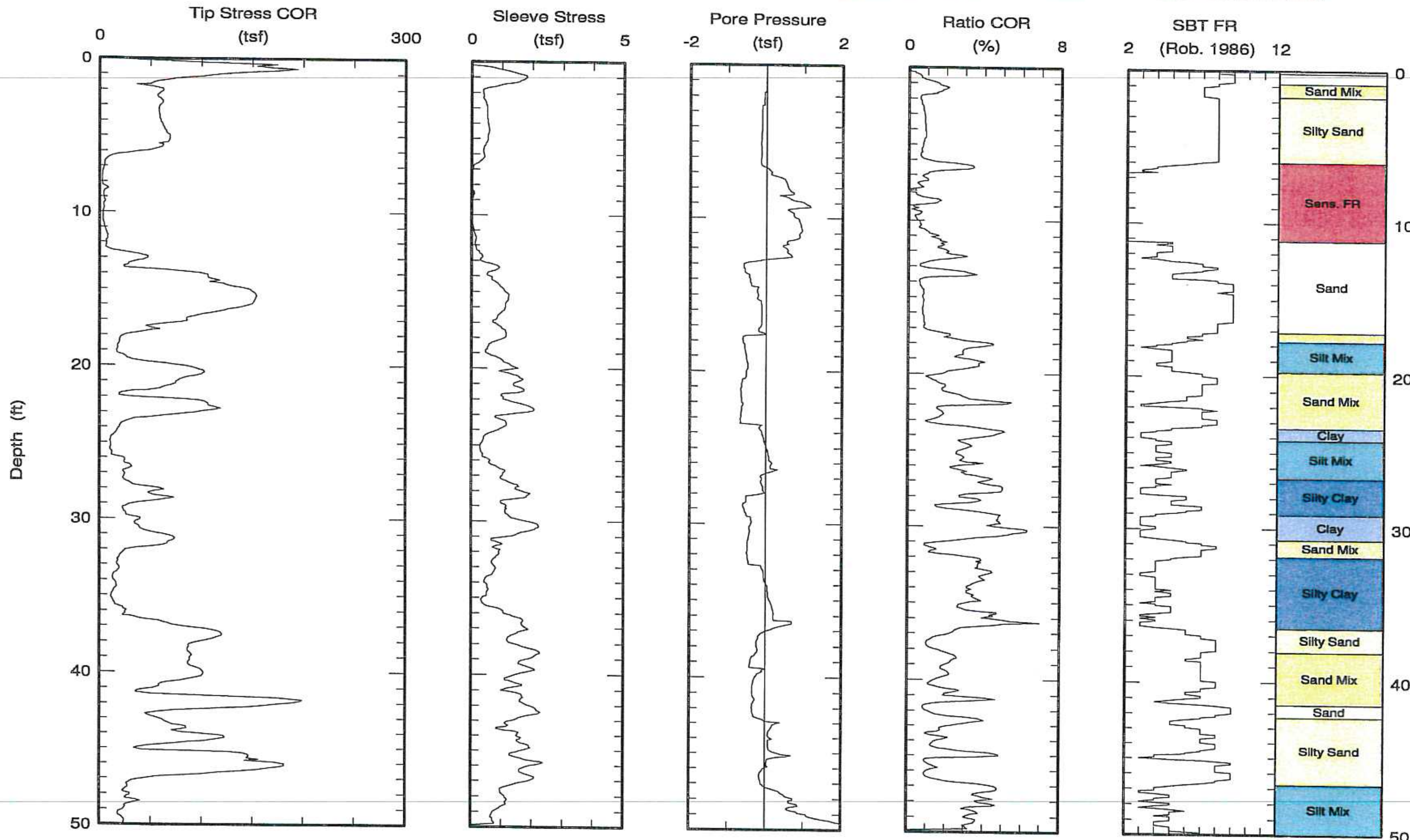
LOG OF BORING

CSD/ District Office
Cypress, California

PROJECT NO. 11050-02

NMG

KTE Kehoe Testing & Engineering Office: (714) 901-7270 Fax: (714) 901-7289 rich@kehoetesting.com www.kehoetesting.com	CPT Data 30 ton rig	Date: 04/Oct/2011 Test ID: CPT-1 Project: Cypress
	Customer: NMG Geotechnical, Inc. Job Site: Cypress School District	



Maximum depth: 50.17 (ft)
Page 1 of 2

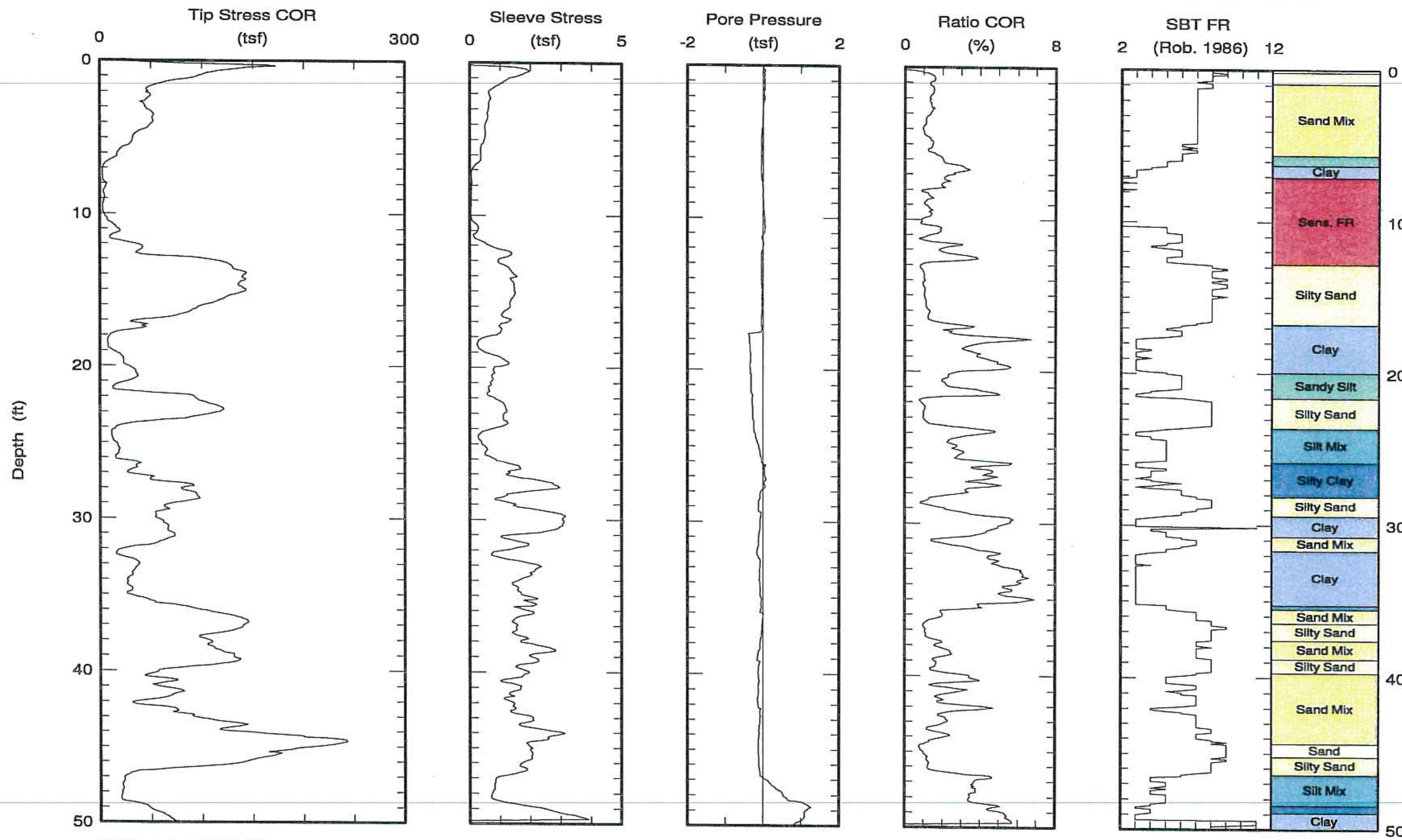
Test ID: CPT-1
File: Z04O1101C.ECP

Kehoe Testing & Engineering
Office: (714) 901-7270
Fax: (714) 901-7289
rich@kehoetesting.com
www.kehoetesting.com

CPT Data
30 ton rig

Date: 04/Oct/2011
Test ID: CPT-2
Project: Cypress

Customer: NMG Geotechnical, Inc.
Job Site: Cypress School District



Maximum depth: 50.14 (ft)
Page 1 of 2

Test ID: CPT-2
File: Z04O1102C.ECP

Sample	*Compacted Moisture (%)*	*Compacted Dry Density (pcf)*	*Final Moisture (%)*	*Volumetric Swell (%)*	*Expansion Index[1] Value/Method*		*Expansive Classification[2]*	*Soluble Sulfate (%)*	*Sulfate Exposure[3]*
HA-2 B-1 0-5'	8.4	103.0	19.9	0.2	2	A	Very Low	--	--

Test Method:

ASTM D4829 / UBC Standard 18-2

HACH SF-1 (Turbidimetric)

Notes:

1. Expansion Index (EI) method of determination:
   [A] E.I. determined by adjusting water content to achieve a 50±1% degree of saturation
   [B] E.I. calculated based on measured saturation within the range of 40% and 60%
2. 1994 UBC Table 18-1-B
3. 1994 UBC Table 19-A-3

Expansion Index and Soluble Sulfate Test Results

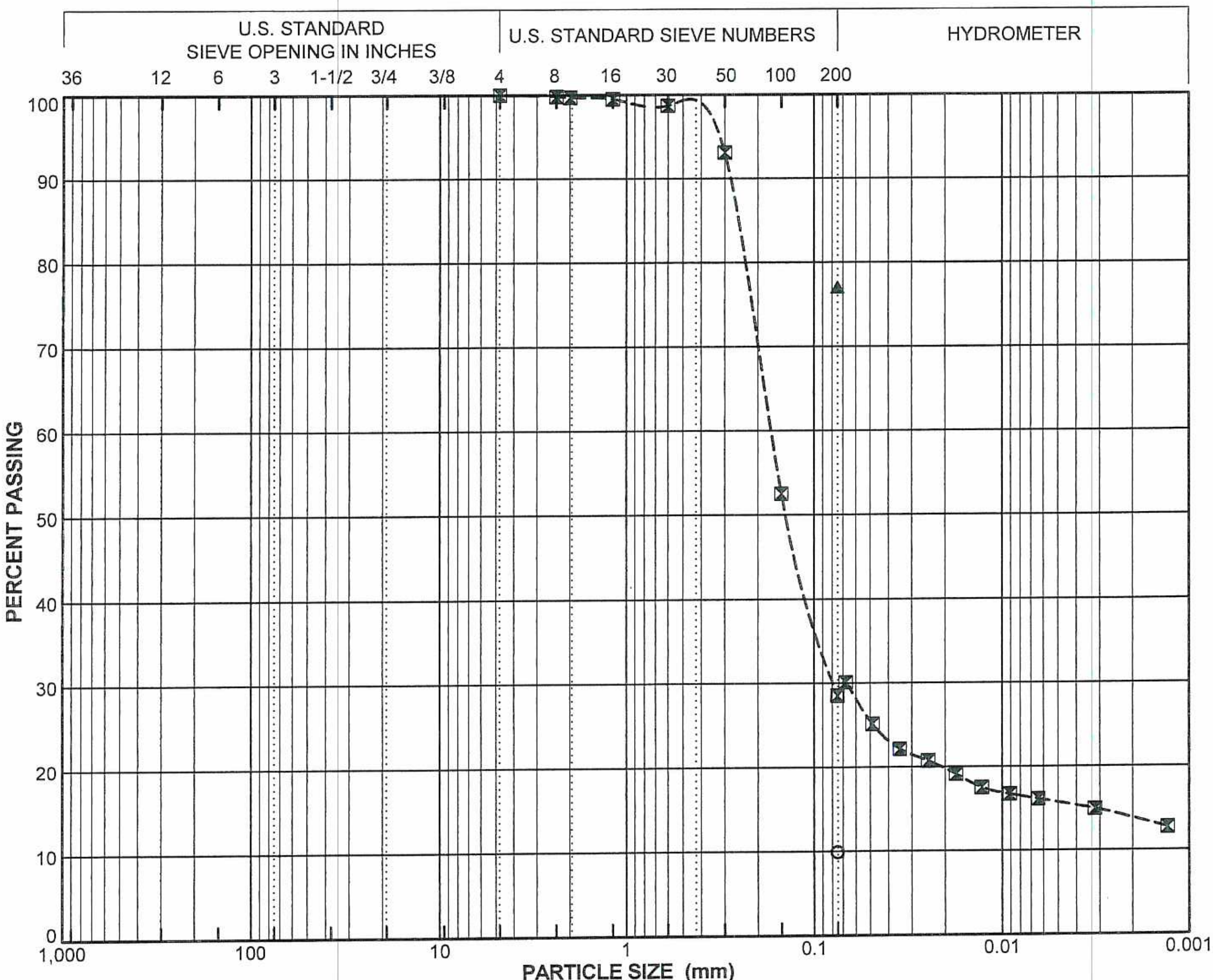
(FRM001a Rev.4)

Project No. 11050-02

Project Name: CSD / District Office

NMG

BOULDERS	COBBLES	GRAVEL		SAND			SILT OR CLAY
		coarse	fine	coarse	medium	fine	



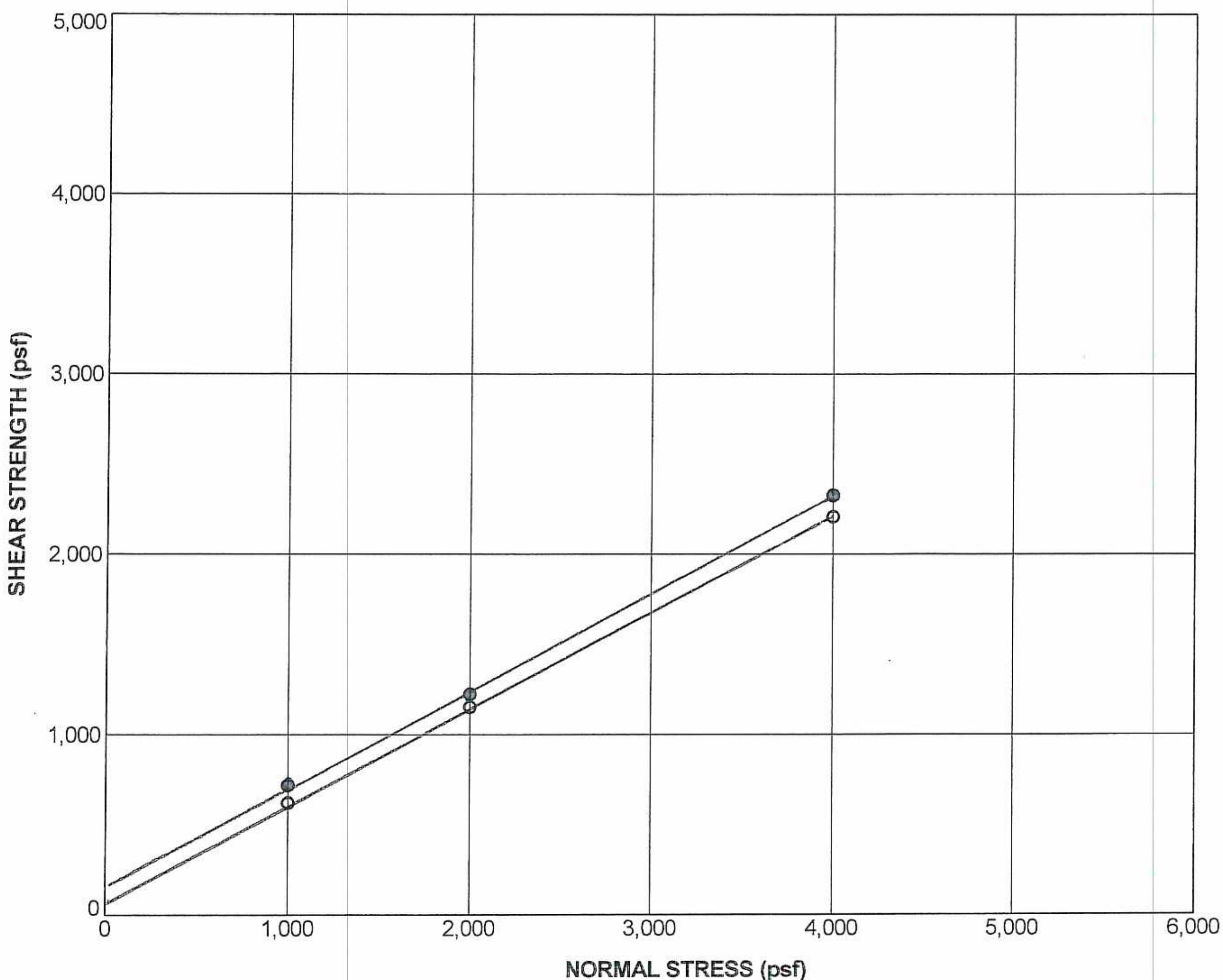
Symbol	Boring Number	Sample Number	Depth (feet)	Field Moisture (%)	LL	PI	Activity PI/-2μ	C_u	C_c	Passing No. 200 Sieve (%)	Passing 2μ (%)	USCS
○	H-1	D-5	15.0	28						10		SP
⊠	H-2	B-1	1.0	8	NP	NP				29	14	SM
▲	H-2	D-4	10.0	32						77		CL

PARTICLE SIZE DISTRIBUTION

CSD/ District Office
Cypress, California
PROJECT NO. 11050-02

NMG Geotechnical, Inc.

Template: NMSIV; Prj ID: 11050-02-DISTRICT.GPJ; Printed: 10/27/11



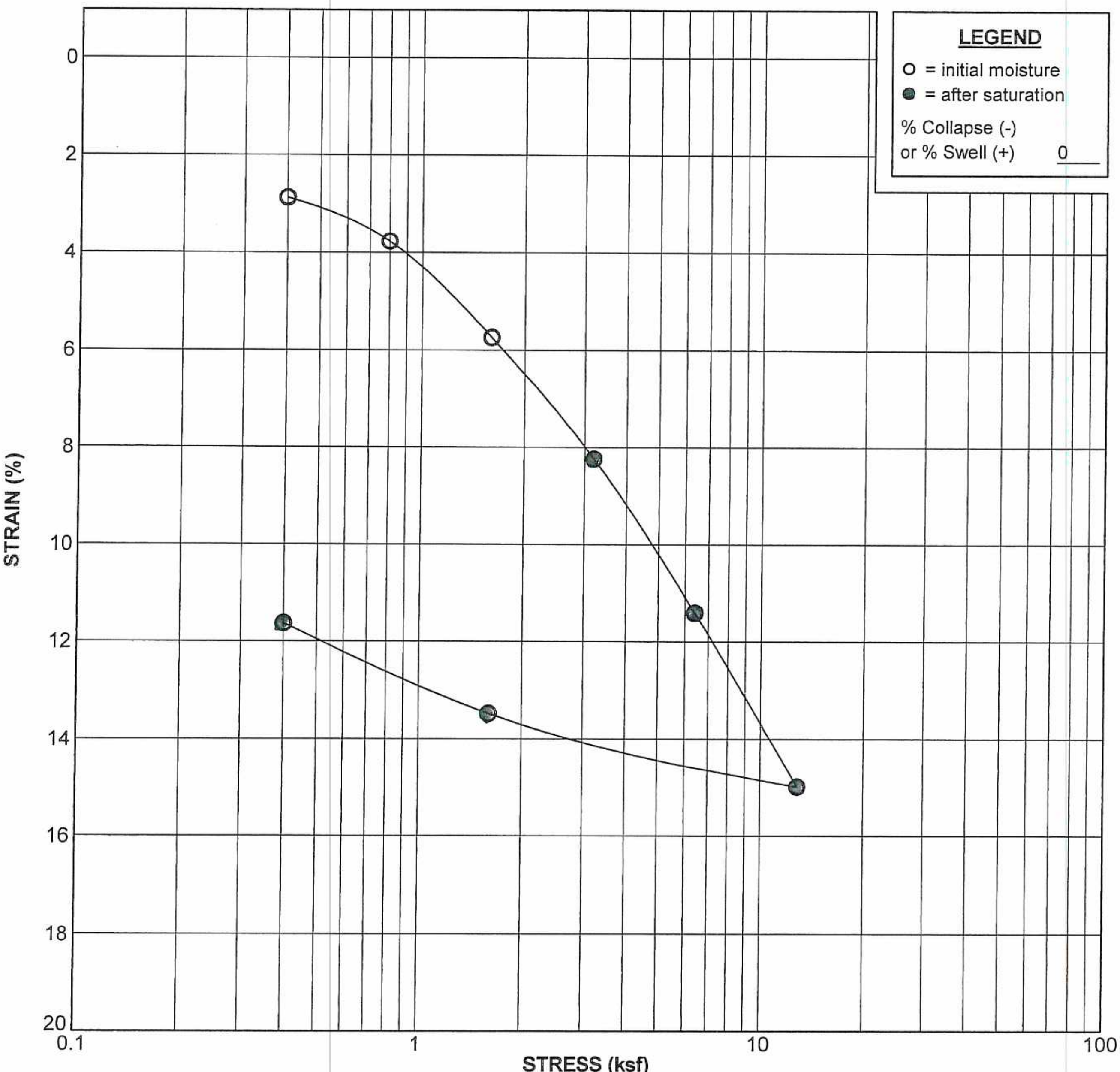
Boring No. H-2	Sample No. D-2	Depth: 5.0 ft
Sample Description: (Qal) Dark gray sandy SILT		
Liquid Limit:	Plasticity Index:	Percent Passing No. 200 Sieve:
Moisture Content (%): 39.4	Dry Density (pcf): 86.9	Degree of Saturation (%): 100
Sample Type: Undisturbed	Rate of Shear (in./min.): 0.005	

SHEAR STRENGTH PARAMETERS		
Parameter	Peak ●	Ultimate ○
Cohesion (psf)	180	80
Friction Angle (degrees)	28	27.0

DIRECT SHEAR TEST RESULTS

CSD/ District Office
Cypress, California
PROJECT NO. 11050-02

NMG Geotechnical, Inc.



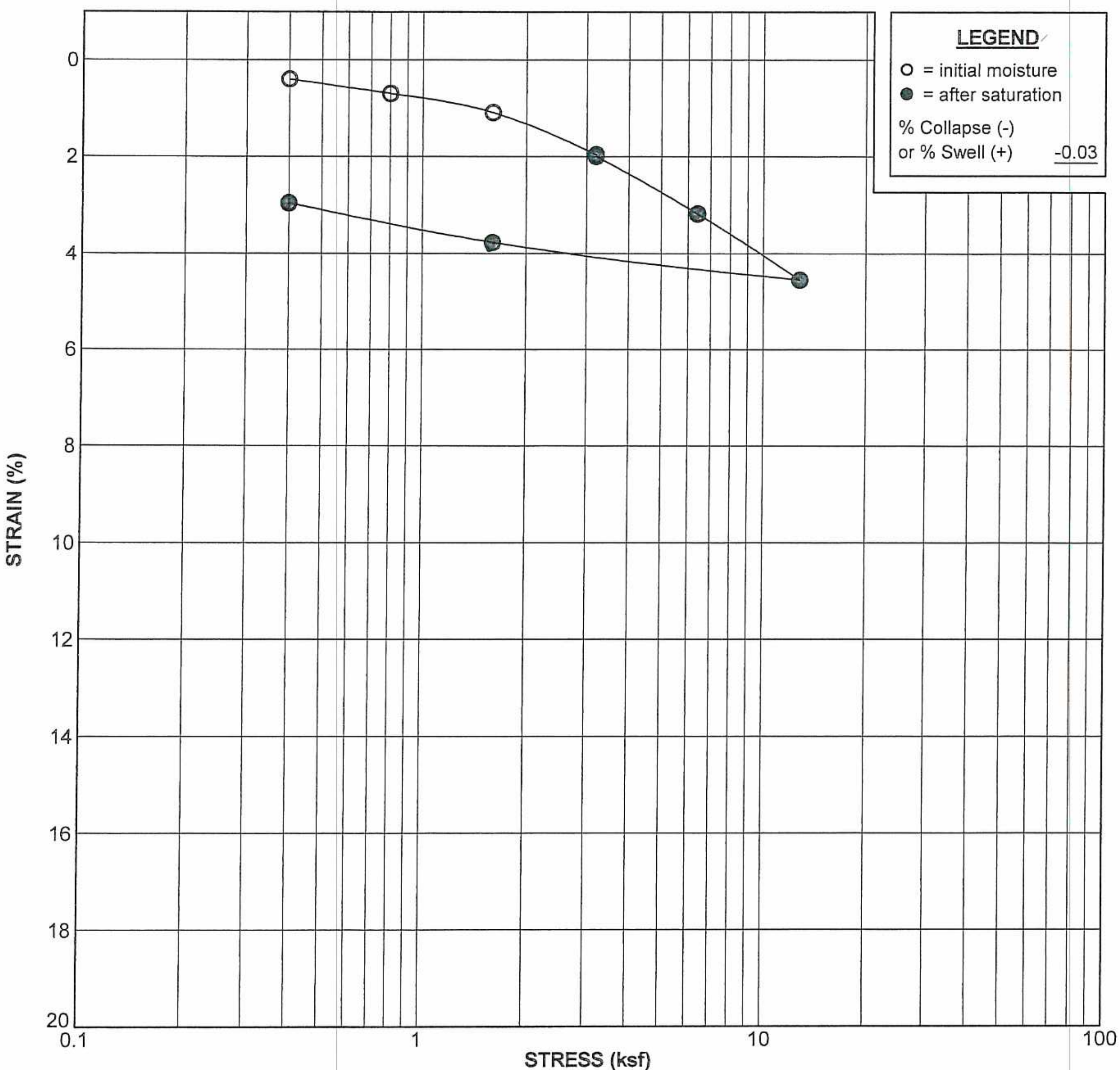
Boring No. H-1	**Sample No. D-3**	**Depth: 7.5 ft**
Sample Description:	(Qal) Grayish brown silty SAND	
Liquid Limit:	**Plasticity Index:**	**Percent Passing No. 200 Sieve:**

Test Stage	Moisture Content (%)	Dry Density (pcf)	Degree of Saturation (%)	Void Ratio
Initial	41.5	79.1	100.9	1.091
Final	34.2	88.3	103.8	0.873

CONSOLIDATION TEST RESULTS

CSD/ District Office
Cypress, California
PROJECT NO. 11050-02

NMG Geotechnical, Inc.



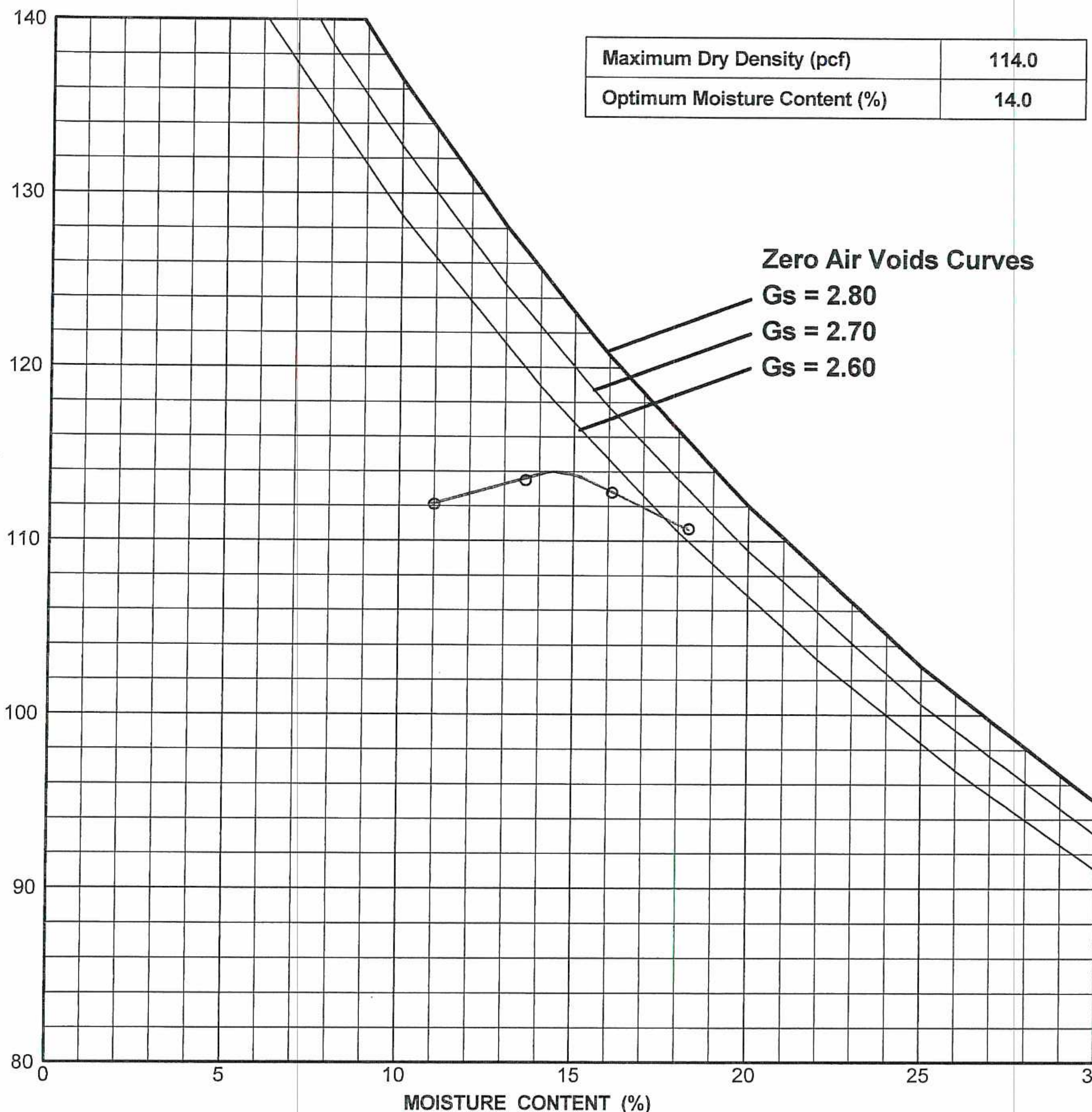
Boring No. H-2	Sample No. D-2	Depth: 5.0 ft
Sample Description:	(Qal) Dark gray sandy SILT	
Liquid Limit:	Plasticity Index:	Percent Passing No. 200 Sieve:

Test Stage	Moisture Content (%)	Dry Density (pcf)	Degree of Saturation (%)	Void Ratio
Initial	27.0	90.8	74.1	1.130
Final	31.0	93.5	89.9	1.069

CONSOLIDATION TEST RESULTS

CSD/ District Office
Cypress, California
PROJECT NO. 11050-02

NMG Geotechnical, Inc.



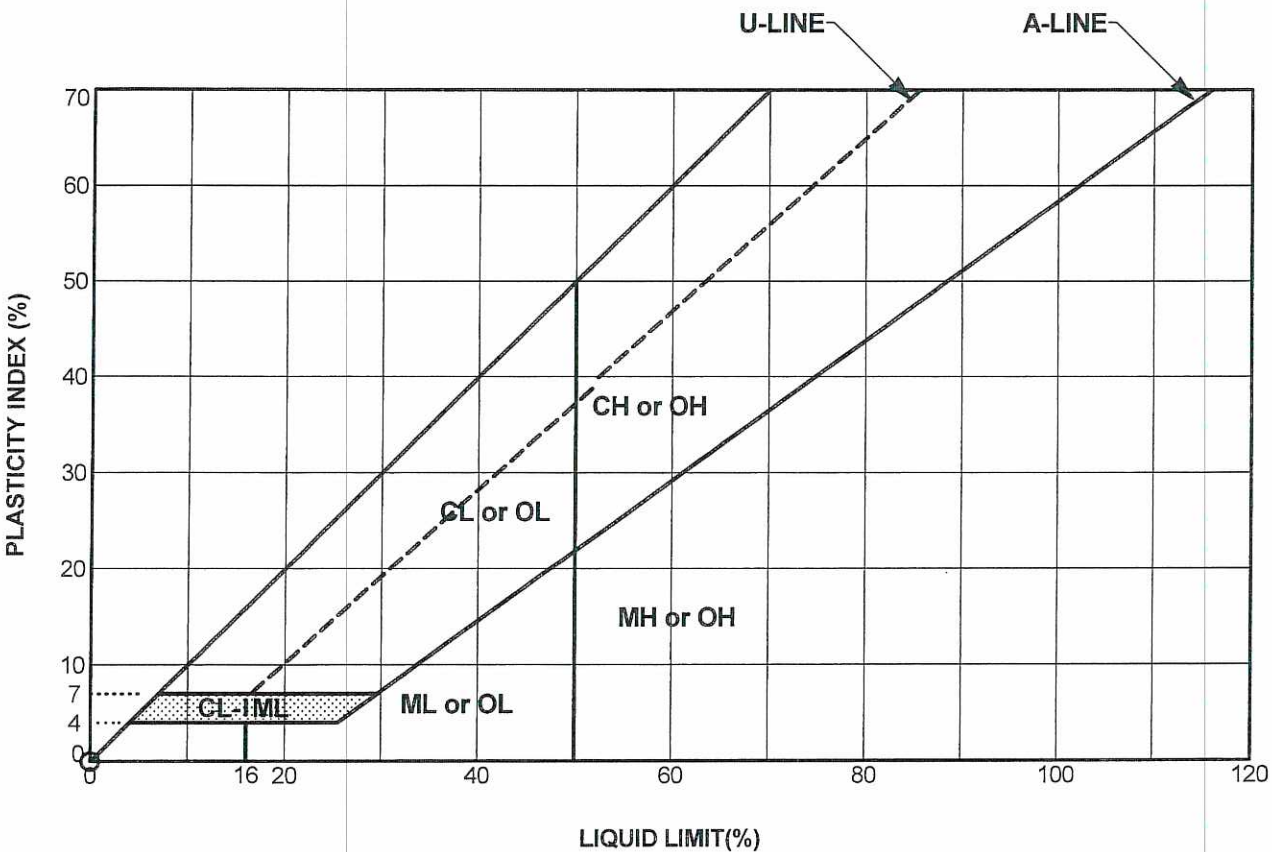
Boring No. H-2	Sample No. B-1	Depth: 1.0 ft
Sample Description: (Qal) Pale brown silty SAND		

Liquid Limit:	NP	Plasticity Index:	NP	Percent Passing No. 200 Sieve:	29
Comments:	1557A				

COMPACTION TEST RESULTS

CSD/ District Office
Cypress, California
PROJECT NO. 11050-02

NMG Geotechnical, Inc.



Symbol	Boring Number	Depth (feet)	Sample Number	Passing No. 200 Sieve (%)	LL	PI	USCS	Description
O	H-2	1.0	B-1	29	NP	NP	SM	(Qal) Pale brown silty SAND

PLASTICITY CHART

CSD/ District Office
Cypress, California
PROJECT NO. 11050-02

NMG Geotechnical, Inc.

R-VALUE TEST DATA CTM 301

Project:	C.S.D./ District Office	Project No:	11050-02	Date:	10/18/2011
Boring Trench No:	H-2	Sample No:	B-1	Sample Depth:	0-5'
Field Description:	SM				
Lab Description:	Olive brown clayey SILT (ML)				

Specimen Number	1		2		3		4	
Mold Number	7		8		9			
Water Adjustment (g)	+70		+80		+75			
Compactor Pressure (psi)	165		145		155			
Exudation Pressure (psi)	427		203		359			
Gross Weight (g)	3187.2		3175.8		3186.2			
Mold Tare (g)	2132.2		2118.9		2129.8			
Wet Weight (g)	1055		1056.9		1056.4		0	
Sample Height (in)	2.51		2.52		2.51			
Initial Dial Reading	0.0725		0.0356		0.0424			
Final Dial Reading	0.0736		0.0363		0.0432			
Expansion (in x10^{-4})	11		7		8		0	
Stability(psi) at 2,000 lbs (160 psi)	24	39	26	46	25	40		
Turns Displacement	3.5		3.92		3.84			
R-Value Uncorrected	69		61		66		#DIV/0!	
R-Value Corrected	69		61		66			
Moisture Content (%)	13.3		13.9		13.6		#DIV/0!	
Dry Density (pcf)	112.4		111.6		112.3		#DIV/0!	
Assumed Traffic Index	4.0		4.0		4.0		4.0	
G.E. by Stability	0.32		0.40		0.35		1.02	
G.E. by Expansion	0.37		0.23		0.27		0.00	
Gf	1.25							

Moisture Content				
Dish No.	A	B	C	
Weight of Moist Soil and Dish (g)	260.6	243.2	259	
Weight of Dry Soil and Dish (g)	235.8	219.7	234	
Water Loss (g)	24.8	23.5	25	0
Weight of Dish (g)	49.3	50.1	50	
Dry Soil (g)	186.5	169.6	184	0
Moisture Content (%)	13.3	13.9	13.6	#DIV/0!

R-Value by Exudation = 64

R-Value by Expansion = 71

R-Value at Equilibrium = 64 by Exudation

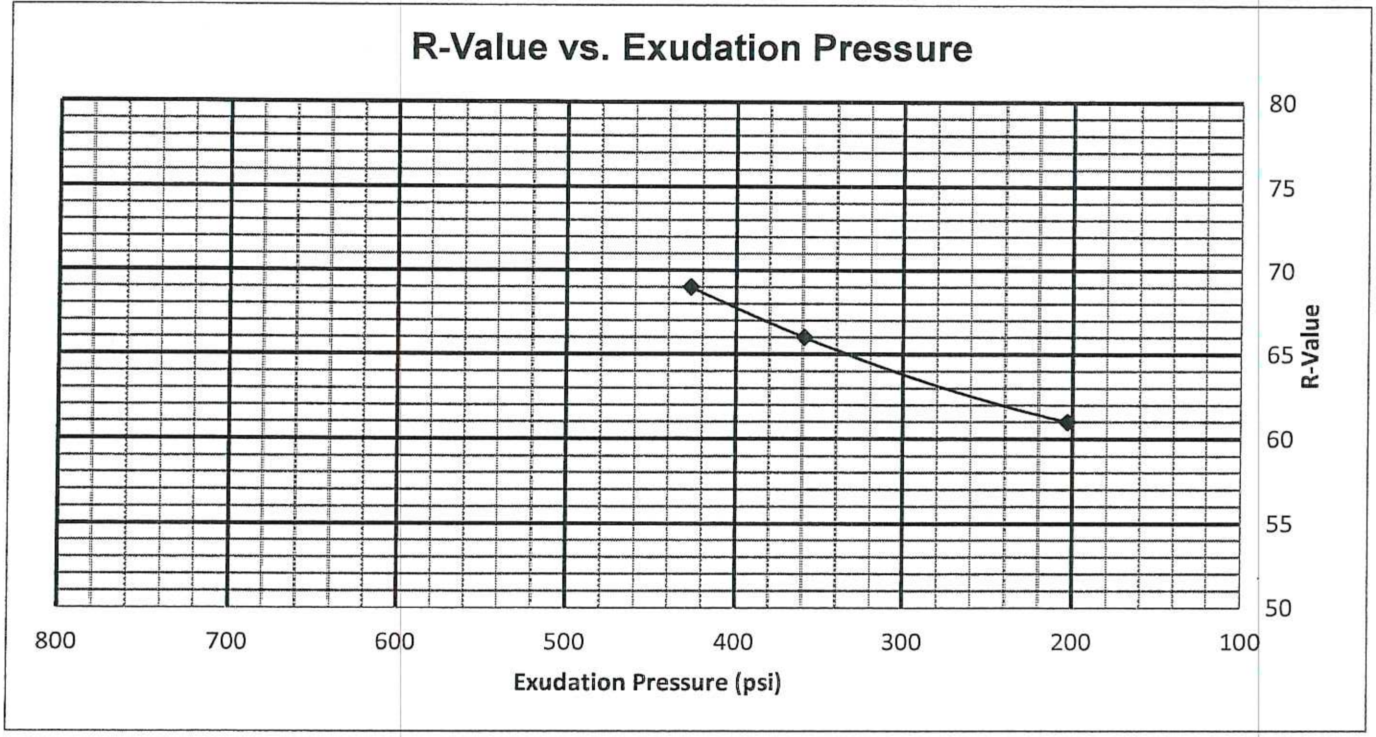
The data above is based upon processing and testing samples as received from the field. Test procedures in accordance with latest revisions to Department of Transportation, State of California, Materials & Research Test Method No. 301

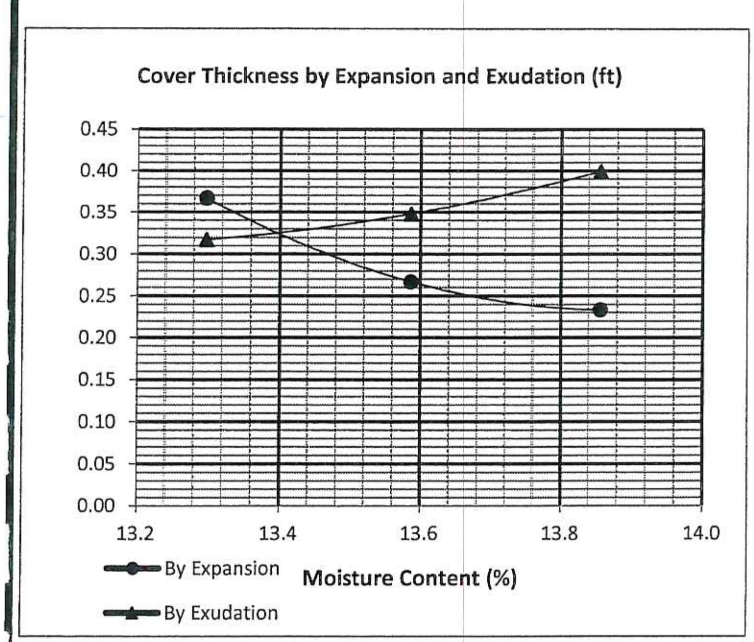
Remarks:			
Set up by:	Run by: GEH/MPD		
Calculated by:	Checked by:	Date Completed:	10/19/2011

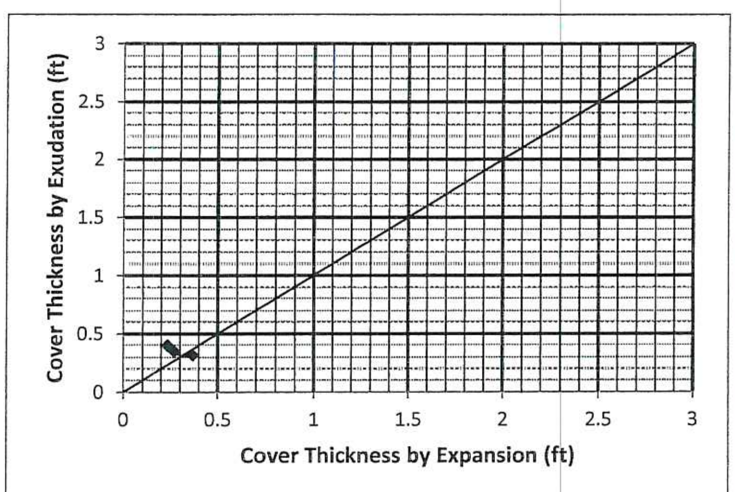
NMG Geotechnical, Inc.

R-VALUE GRAPHICAL PRESENTATION

Project:	C.S.D./ District Office	Project No:	11050-02	Date:	10/18/2011
Boring Trench No:	H-2	Sample No:	B-1	Sample Depth:	0-5'
Field Description:	SM				
Lab Description:	Olive brown clayey SILT (ML)				







Cover Thickness (ft) = 0.3

The data above is based upon processing and testing samples as received from the field. Test procedures in accordance with latest revisions to Department of Transportation, State of California, Materials & Research Test Method No. 301

Remarks:		
Set up by:	Run by:	
Calculated by:	Checked by:	Date Completed:

NMG Geotechnical, Inc.

Table 1 - Laboratory Tests on Soil Samples

NMG Geotechnical, Inc.
HDR|Schiff #11-1071LAB
21-Oct-11

Sample ID			Cawthorn H-1, B-1 @ 0-5' SM	District Office H-2, B-1 @ 0-5' SM, ML	Damron H-1, B-1 @ 0-5' SM, ML
Resistivity		**Units**			
as-received		ohm-cm	2,440	13,200	25,200
saturated		ohm-cm	76	2,080	2,320
pH			8.1	8.2	8.0
Electrical Conductivity		mS/cm	1.76	0.21	0.13
Chemical Analyses					
Cations					
calcium	Ca^{2+}	mg/kg	167	53	72
magnesium	Mg^{2+}	mg/kg	93	11	18
sodium	Na^{1+}	mg/kg	1,561	139	70
potassium	K^{1+}	mg/kg	128	147	40
Anions					
carbonate	CO_3^{2-}	mg/kg	ND	ND	ND
bicarbonate	HCO_3^{1-}	mg/kg	296	479	375
fluoride	F^{1-}	mg/kg	6.6	5.5	8.3
chloride	Cl^{1-}	mg/kg	685	21	11
sulfate	SO_4^{2-}	mg/kg	2,086	58	36
phosphate	PO_4^{3-}	mg/kg	ND	72	0.9
Other Tests					
ammonium	NH_4^{1+}	mg/kg	ND	4.0	ND
nitrate	NO_3^{1-}	mg/kg	178	28	13
sulfide	S^{2-}	qual	na	na	na
Redox		mV	na	na	na

Electrical conductivity in millisiemens/cm and chemical analysis were made on a 1:5 soil-to-water extract.
mg/kg = milligrams per kilogram (parts per million) of dry soil.
Redox = oxidation-reduction potential in millivolts
ND = not detected
na = not analyzed

APPENDIX B

BORING LOGS BY GEOTEK

Updated Geotechnical Evaluation

Orange Avenue Seniors, Cypress, California

Project No. 2573-CR

GEOTEK

GeoTek, Inc.
LOG OF EXPLORATORY BORING

CLIENT:	Melia Homes	**DRILLER:**	2R Drilling	**LOGGED BY:**	DRW
PROJECT NAME:	Cypress	**DRILL METHOD:**	Hollow Stem Auger	**OPERATOR:**	Ish
PROJECT NO.:	1763-CR	**HAMMER:**	140/30"	**RIG TYPE:**	CME 75
LOCATION:	See Exploration Location Map			**DATE:**	9/25/2017

BORING NO.: B-1

Depth (ft)	Sample Type	Blows/ 6 in	Sample Number	USCS Symbol	MATERIAL DESCRIPTION AND COMMENTS	Water Content (%)	Dry Density (pcf)	Others
					UNDOCUMENTED FILL			
	Large Bulk / Ring	8 11 13		SM	Silty f-m SAND, brown, moist, medium dense	13.0	97.0	DS, HC, SA
					ALLUVIUM			
5	Ring	3 2 1		SM	Silty f SAND, gray to grayish brown, very moist, loose approximately 23% fines	30.0	92.0	
10	Ring	1 3 7			No recovery in sampler			
	SPT	1 2 4		ML	F sandy SILT, brownish gray, very moist, loose			
					BORING TERMINATED AT 13 FEET			
					No groundwater encountered Boring backfilled with soil cuttings			
15								
20								
25								
30								

Laboratory Testing

LEGEND

Sample type: ---Ring ---SPT ---Small Bulk ---Large Bulk ---No Recovery ---Water Table

Lab testing:
AL = Atterberg Limits EI = Expansion Index SA = Sieve Analysis RV = R-Value Test
SR = Sulfate/Resisitivity Test SH = Shear Test HC= Consolidation MD = Maximum Density

GeoTek, Inc.

LOG OF EXPLORATORY BORING

CLIENT:	Melia Homes	**DRILLER:**	2R Drilling	**LOGGED BY:**	DRW
PROJECT NAME:	Cypress	**DRILL METHOD:**	Hollow Stem Auger	**OPERATOR:**	Ish
PROJECT NO.:	1763-CR	**HAMMER:**	140/30"	**RIG TYPE:**	CME 75
LOCATION:	See Exploration Location Map			**DATE:**	9/25/2017

BORING NO.: B-2

Depth (ft)	SAMPLES: Sample Type	Blows/ 6 in	Sample Number	USCS Symbol	MATERIAL DESCRIPTION AND COMMENTS	Laboratory Testing: Water Content (%)	Dry Density (pcf)	Others
					UNDOCUMENTED FILL			
				SM	Silty f-m SAND, brown, moist			
	Ring	10 13 16		SM	**ALLUVIUM** Silty f SAND, brownish gray, moist, medium dense	14.0	101	
5								
	Ring	3 3 4			No recovery in sampler			
10	SPT	1 1 2		ML	F sandy SILT, gray, very moist, loose			
					BORING TERMINATED AT 11.5 FEET			
					No groundwater encountered Boring backfilled with soil cuttings			
15								
20								
25								
30								

LEGEND

Sample type: ---Ring ---SPT ---Small Bulk ---Large Bulk ---No Recovery ---Water Table

Lab testing: AL = Atterberg Limits EI = Expansion Index SA = Sieve Analysis RV = R-Value Test SR = Sulfate/Resisitivity Test SH = Shear Test HC= Consolidation MD = Maximum Density

GeoTek, Inc.

LOG OF EXPLORATORY BORING

CLIENT:	Melia Homes	**DRILLER:**	2R Drilling	**LOGGED BY:**	DRW
PROJECT NAME:	Cypress	**DRILL METHOD:**	Hollow Stem Auger	**OPERATOR:**	Ish
PROJECT NO.:	1763-CR	**HAMMER:**	140/30"	**RIG TYPE:**	CME 75
LOCATION:	See Exploration Location Map			**DATE:**	9/25/2017

BORING NO.: B-3

Depth (ft)	SAMPLES: Sample Type	SAMPLES: Blows/ 6 in	SAMPLES: Sample Number	USCS Symbol	MATERIAL DESCRIPTION AND COMMENTS	Laboratory Testing: Water Content (%)	Laboratory Testing: Dry Density (pcf)	Laboratory Testing: Others
					UNDOCUMENTED FILL			
	Ring	11 16 17			Silty f-m SAND, brown to dark brown, moist, medium dense	11.0	122.0	
					ALLUVIUM			
5	Ring	5 7 8		SM	Silty f SAND, brownish gray, moist, loose approximately 15% fines	24.0	92.0	SA
10	Ring	1 1 1		ML	SILT, gray, very moist, loose	36.0	86.0	HC
					BORING TERMINATED AT 11.5 FEET No groundwater encountered Boring backfilled with soil cuttings			
15								
20								
25								
30								

LEGEND

Sample type: ---Ring ---SPT ---Small Bulk ---Large Bulk ---No Recovery ---Water Table

Lab testing:
AL = Atterberg Limits
EI = Expansion Index
SA = Sieve Analysis
RV = R-Value Test
SR = Sulfate/Resisitivity Test
SH = Shear Test
HC= Consolidation
MD = Maximum Density

GeoTek, Inc.

LOG OF EXPLORATORY BORING

CLIENT:	Melia Homes	**DRILLER:**	2R Drilling	**LOGGED BY:**	DRW
PROJECT NAME:	Cypress	**DRILL METHOD:**	Hollow Stem Auger	**OPERATOR:**	Ish
PROJECT NO.:	1763-CR	**HAMMER:**	140/30"	**RIG TYPE:**	CME 75
LOCATION:	See Exploration Location Map			**DATE:**	9/25/2017

BORING NO.: B-4

Depth (ft)	SAMPLES: Sample Type	SAMPLES: Blows/ 6 in	SAMPLES: Sample Number	USCS Symbol	MATERIAL DESCRIPTION AND COMMENTS	Laboratory Testing: Water Content (%)	Laboratory Testing: Dry Density (pcf)	Laboratory Testing: Others
					UNDOCUMENTED FILL			
				SM	Silty f-m SAND, brown, moist			
	Ring	3 4 5		SM	**ALLUVIUM** Silty f SAND to sandy SILT, brownish gray, moist, loose	15.0	102	HC
5								
	Large Bulk							
	Ring	2 1 2		ML	F sandy SILT, gray, very moist, loose	32.0	91	
10					**BORING TERMINATED AT 11.5 FEET**			
					No groundwater encountered Boring backfilled with soil cuttings			
15								
20								
25								
30								

LEGEND

Sample type: ---Ring ---SPT ---Small Bulk ---Large Bulk ---No Recovery ---Water Table

Lab testing:

AL = Atterberg Limits	EI = Expansion Index	SA = Sieve Analysis	RV = R-Value Test
SR = Sulfate/Resisitivity Test	SH = Shear Test	HC= Consolidation	MD = Maximum Density

GeoTek, Inc.
LOG OF EXPLORATORY BORING

CLIENT:	Melia Homes	**DRILLER:**	2R Drilling	**LOGGED BY:**	DRW
PROJECT NAME:	Cypress	**DRILL METHOD:**	Hollow Stem Auger	**OPERATOR:**	Ish
PROJECT NO.:	1763-CR	**HAMMER:**	140/30"	**RIG TYPE:**	CME 75
LOCATION:	See Exploration Location Map			**DATE:**	9/25/2017

BORING NO.: B-5

Depth (ft)	SAMPLES: Sample Type	SAMPLES: Blows/ 6 in	SAMPLES: Sample Number	USCS Symbol	MATERIAL DESCRIPTION AND COMMENTS	Laboratory Testing: Water Content (%)	Laboratory Testing: Dry Density (pcf)	Laboratory Testing: Others
					UNDOCUMENTED FILL			
	Ring	8 6 6			Silty f-m SAND, brown, moist, loose, trace fine gravel	8.0	107.0	
					ALLUVIUM			
5	Ring	5 9 15		ML	F-c SAND, light gray, moist, medium dense	8.0	101.0	
10	Ring	2 2 4			No recovery in sampler			
	SPT	1 1 2		ML	F sandy SILT, gray, very moist, loose			
					BORING TERMINATED AT 13 FEET			
					No groundwater encountered Boring backfilled with soil cuttings			
15								
20								
25								
30								

LEGEND

Sample type: ---Ring ---SPT ---Small Bulk ---Large Bulk ---No Recovery ---Water Table

Lab testing: AL = Atterberg Limits; EI = Expansion Index; SA = Sieve Analysis; RV = R-Value Test; SR = Sulfate/Resisitivity Test; SH = Shear Test; HC= Consolidation; MD = Maximum Density

GeoTek, Inc.
LOG OF EXPLORATORY BORING

CLIENT:	Melia Homes	**DRILLER:**	2R Drilling	**LOGGED BY:**	JE
PROJECT NAME:	9470 Moody Street	**DRILL METHOD:**	8" Hollow Stem	**OPERATOR:**	Ish
PROJECT NO.:	2573-CR	**HAMMER:**	Auto 140#/30"	**RIG TYPE:**	CME 75
LOCATION:	See Exploration Location Map			**DATE:**	12/8/2020

BORING NO.: B-6

Depth (ft)	SAMPLES: Sample Type	SAMPLES: Blows/ 6 in	SAMPLES: Sample Number	USCS Symbol	MATERIAL DESCRIPTION AND COMMENTS	Laboratory Testing: Water Content (%)	Laboratory Testing: Dry Density (pcf)	Laboratory Testing: Others
					4" Asphalt over 3" Base **ALLUVIUM:**			
	Ring	7 12 15		SM	Silty f-m SAND, light brown to brown, moist, medium dense	10.2	93.5	
5	Ring	5 3 3		ML	F-m sandy SILT, gray to grayish brown, wet, loose	28.9	93.7	HC
					BORING TERMINATED AT 7 FEET No groundwater encountered Boring backfilled with soil cuttings and patched with AC			
10								
15								
20								
25								
30								

LEGEND

Sample type: ---Ring ---SPT ---Small Bulk ---Large Bulk ---No Recovery ---Water Table

Lab testing:
AL = Atterberg Limits
EI = Expansion Index
SA = Sieve Analysis
RV = R-Value Test
SR = Sulfate/Resisitivity Test
SH = Shear Test
HC= Consolidation
MD = Maximum Density

GeoTek, Inc.

LOG OF EXPLORATORY BORING

CLIENT:	Melia Homes	**DRILLER:**	2R Drilling	**LOGGED BY:**	JE
PROJECT NAME:	9470 Moody Street	**DRILL METHOD:**	8" Hollow Stem	**OPERATOR:**	Ish
PROJECT NO.:	2573-CR	**HAMMER:**	Auto 140#/30"	**RIG TYPE:**	CME 75
LOCATION:	See Exploration Location Map			**DATE:**	12/8/2020

BORING NO.: B-7

Depth (ft)	SAMPLES: Sample Type	SAMPLES: Blows/ 6 in	SAMPLES: Sample Number	USCS Symbol	MATERIAL DESCRIPTION AND COMMENTS	Laboratory Testing: Water Content (%)	Laboratory Testing: Dry Density (pcf)	Laboratory Testing: Others
					4" Asphalt over 4" Base			
					ALLUVIUM:			MD, EI, SR
	Ring	10 12 12		SM	F-m SAND with silt, light grayish brown to brown, moist, medium dense	10.6	108.6	
	Ring	5 7 11			becomes very moist	24.2	100.1	
5	Ring	4 3 3		SM	Silty f-m SAND, dark brown to brown, very moist, loose.	37.8	84.3	HC
					BORING TERMINATED AT 8 FEET			
					No groundwater encountered Boring backfilled with soil cuttings and patched with AC			
10								
15								
20								
25								
30								

LEGEND

Sample type: ---Ring ---SPT ---Small Bulk ---Large Bulk ---No Recovery ---Water Table

Lab testing:

AL = Atterberg Limits | EI = Expansion Index | SA = Sieve Analysis | RV = R-Value Test
SR = Sulfate/Resisitivity Test | SH = Shear Test | HC= Consolidation | MD = Maximum Density

GeoTek, Inc.
LOG OF EXPLORATORY BORING

CLIENT:	Melia Homes	**DRILLER:**	2R Drilling	**LOGGED BY:**	JE
PROJECT NAME:	9470 Moody Street	**DRILL METHOD:**	8" Hollow Stem	**OPERATOR:**	Ish
PROJECT NO.:	2573-CR	**HAMMER:**	Auto 140#/30"	**RIG TYPE:**	CME 75
LOCATION:	See Exploration Location Map			**DATE:**	12/8/2020

BORING NO.: B-8

Depth (ft)	SAMPLES: Sample Type	Blows/ 6 in	Sample Number	USCS Symbol	MATERIAL DESCRIPTION AND COMMENTS	Laboratory Testing: Water Content (%)	Dry Density (pcf)	Others
					4" Asphalt over 4" Base **ALLUVIUM:**			
	SPT	7 10 13		SM	F-m SAND with silt, light brown, moist, medium dense	12.3	114.2	
5	SPT	7 9 11		SM	Silty f-m SAND, brown to dark brown, very moist, medium dense	28.3	82.7	
					BORING TERMINATED AT 7 FEET No groundwater encountered Boring backfilled with soil cuttings and patched with AC			
10								
15								
20								
25								
30								

LEGEND

Sample type: ---Ring ---SPT ---Small Bulk ---Large Bulk ---No Recovery ---Water Table

Lab testing:
AL = Atterberg Limits, EI = Expansion Index, SA = Sieve Analysis, RV = R-Value Test
SR = Sulfate/Resisitivity Test, SH = Shear Test, HC= Consolidation, MD = Maximum Density

GeoTek, Inc.

LOG OF EXPLORATORY BORING

CLIENT:	Melia Homes	**DRILLER:**	2R Drilling	**LOGGED BY:**	JE
PROJECT NAME:	9470 Moody Street	**DRILL METHOD:**	8" Hollow Stem	**OPERATOR:**	Ish
PROJECT NO.:	2573-CR	**HAMMER:**	Auto 140#/30"	**RIG TYPE:**	CME 75
LOCATION:	See Exploration Location Map			**DATE:**	12/8/2020

BORING NO.: B-9

Depth (ft)	Sample Type	Blows/ 6 in	Sample Number	USCS Symbol	MATERIAL DESCRIPTION AND COMMENTS	Water Content (%)	Dry Density (pcf)	Others
					3" Asphalt over 4" Base			
					ALLUVIUM:			
	Ring	5 11 12		SM	F-m SAND with silt, light brown, moist, medium dense	12.6	114.2	
	Ring	7 10 11			same as above, becomes very moist	27.7	85.5	
5	Ring	5 5 6		SP	F-m SAND, light brown to brown, very moist, loose	24.6	85.9	
					BORING TERMINATED AT 7 FEET			
					No groundwater encountered Boring backfilled with soil cuttings and patched with AC			
10								
15								
20								
25								
30								

Laboratory Testing: Water Content, Dry Density, Others

LEGEND

Sample type: ---Ring ---SPT ---Small Bulk ---Large Bulk ---No Recovery ---Water Table

Lab testing:

AL = Atterberg Limits	EI = Expansion Index	SA = Sieve Analysis	RV = R-Value Test
SR = Sulfate/Resisitivity Test	SH = Shear Test	HC= Consolidation	MD = Maximum Density

GeoTek, Inc.
LOG OF EXPLORATORY BORING

CLIENT:	Melia Homes	**DRILLER:**	2R Drilling	**LOGGED BY:**	JE
PROJECT NAME:	9470 Moody Street	**DRILL METHOD:**	8" Hollow Stem	**OPERATOR:**	Ish
PROJECT NO.:	2573-CR	**HAMMER:**	Auto 140#/30"	**RIG TYPE:**	CME 75
LOCATION:	See Exploration Location Map			**DATE:**	12/8/2020

BORING NO.: B-10

Depth (ft)	SAMPLES: Sample Type	Blows/ 6 in	Sample Number	USCS Symbol	MATERIAL DESCRIPTION AND COMMENTS	Laboratory Testing: Water Content (%)	Dry Density (pcf)	Others
					4" Asphalt over 4" Base			
					ALLUVIUM:			
	Ring	5 9 13		SM	F-c SAND with silt, light brown, very moist, medium dense	15.5	96.8	
5	Ring	3 4 4		SM	Silty f-m SAND, brown to dark brown, very moist, loose	34.4	87.8	
					BORING TERMINATED AT 7 FEET			
					No groundwater encountered Boring backfilled with soil cuttings and patched with AC			
10								
15								
20								
25								
30								

LEGEND

Sample type: ---Ring ---SPT ---Small Bulk ---Large Bulk ---No Recovery ---Water Table

Lab testing:
AL = Atterberg Limits | EI = Expansion Index | SA = Sieve Analysis | RV = R-Value Test
SR = Sulfate/Resisitivity Test | SH = Shear Test | HC= Consolidation | MD = Maximum Density

A - FIELD TESTING AND SAMPLING PROCEDURES

The Standard Penetration Test (SPT)

The SPT is performed in accordance with ASTM Test Method D 1586. The SPT sampler is typically driven into the ground 12 or 18 inches with a 140-pound hammer free falling from a height of 30 inches. Blow counts are recorded for every 6 inches of penetration as indicated on the log of boring. The split-barrel sampler has an external diameter of 2 inches and an unlined internal diameter of 1-3/8 inches. The samples of earth materials collected in the sampler are typically classified in the field, bagged, sealed and transported to the laboratory for further testing.

The Modified Split-Barrel Sampler (Ring)

The ring sampler is driven into the ground in accordance with ASTM Test Method D 3550. The sampler, with an external diameter of 3.0 inches, is lined with 1-inch long, thin brass rings with inside diameters of approximately 2.4 inches. The sampler is typically driven into the ground 12 or 18 inches with a 140-pound hammer free falling from a height of 30 inches. Blow counts are recorded for every 6 inches of penetration as indicated on the log of boring. The samples are removed from the sample barrel in the brass rings, sealed, and transported to the laboratory for testing.

Bulk Samples (Large)

These samples are normally large bags of earth materials over 20 pounds in weight collected from the field by means of hand digging or exploratory cuttings.

Bulk Samples (Small)

These are plastic bag samples which are normally airtight and contain less than 5 pounds in weight of earth materials collected from the field by means of hand digging or exploratory cuttings. These samples are primarily used for determining natural moisture content and classification indices.

B – BORING LOG LEGEND

The following abbreviations and symbols often appear in the classification and description of soil and rock on the logs of borings:

SOILS

USCS	Unified Soil Classification System
f-c	Fine to coarse
f-m	Fine to medium

GEOLOGIC

B: Attitudes	Bedding: strike/dip
J: Attitudes	Joint: strike/dip

C: Contact line

………..	Dashed line denotes USCS material change
———	Solid Line denotes unit / formational change
———	Thick solid line denotes end of boring

(Additional denotations and symbols are provided on the log of borings)

APPENDIX C

LABORATORY TEST RESULTS

Updated Geotechnical Evaluation

Orange Avenue Seniors, Cypress, California

Project No. 2573-CR

GEOTEK

SUMMARY OF LABORATORY TESTING

Classification
Soils were classified visually in general accordance with the Unified Soil Classification System (ASTM Test Method D 2487). The soil classifications are shown on the logs of exploratory borings in Appendix B.

In Situ Moisture Content and Unit Weight
The field moisture content was measured in the laboratory on selected samples collected during the field investigation. The field moisture content is determined as a percentage of the dry unit weight. The dry density was measured in the laboratory on selected ring samples. The results are shown on the logs of exploratory borings in Appendix B.

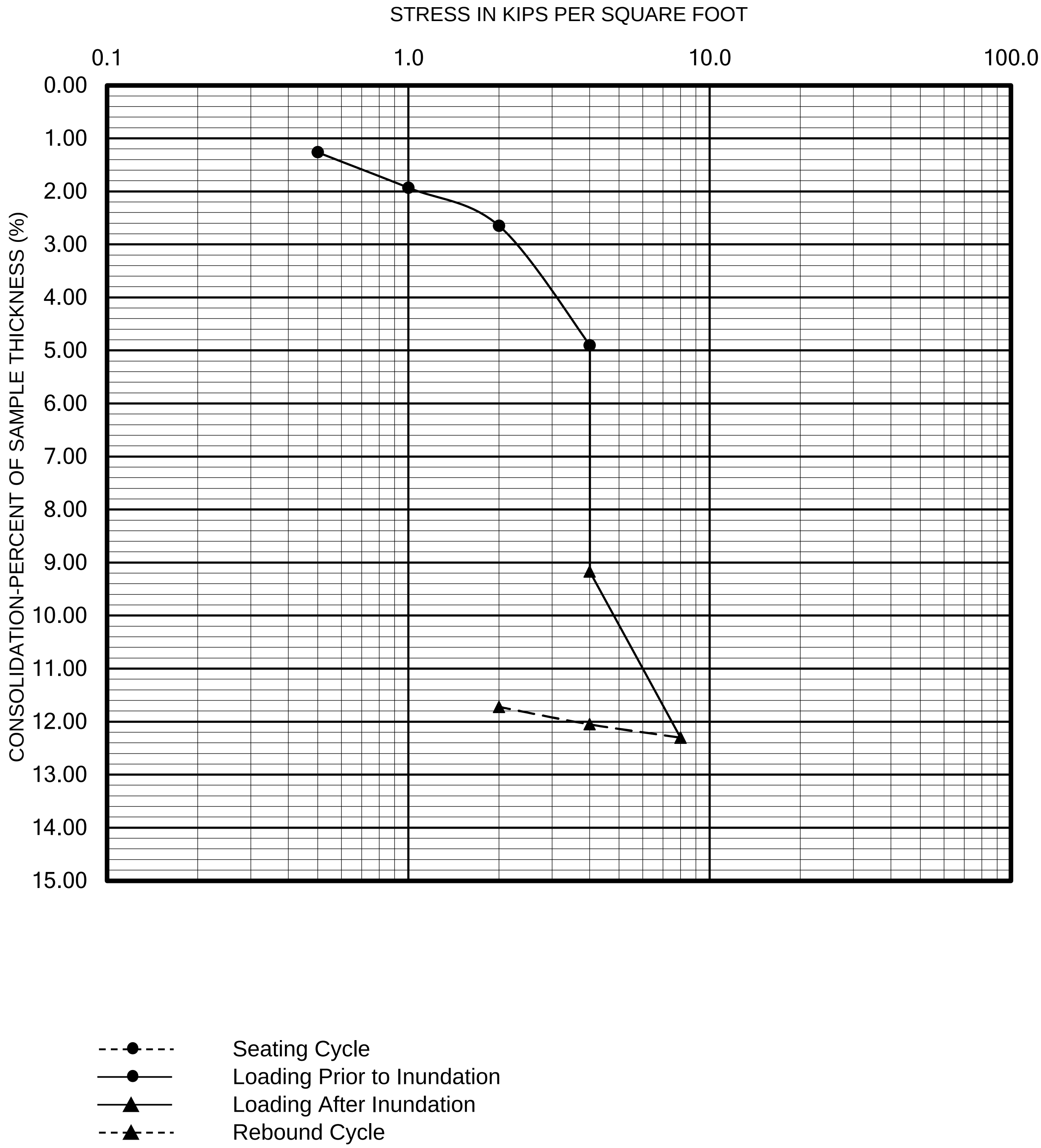
Consolidation/Collapse
Consolidation/collapse testing was performed on selected samples of the site soils according to ASTM Test Method D 2435 and ASTM D4546, respectively. The results of these tests are presented herein.

Direct Shear
Direct shear testing was performed on remolded samples of the surficial soils according to ASTM Test Method D 3080. The results of these tests are presented herein.

Moisture-Density Relationship
Laboratory testing was performed on two samples collected during the subsurface exploration. The laboratory maximum dry density and optimum moisture content for the soil types was determined in general accordance with test method ASTM Test Procedure D 1557. The results are included herein.

Materials Finer Than the No. 200 Sieve
A #200 sieve wash was performed on selected samples of the soils according to ASTM Test Method D 1140. The results of these tests are presented herein.

Sulfate Content, Resistivity and Chloride Content
Testing to determine the water-soluble sulfate content, resistivity testing and the chloride content was performed by others. The test results are presented herein



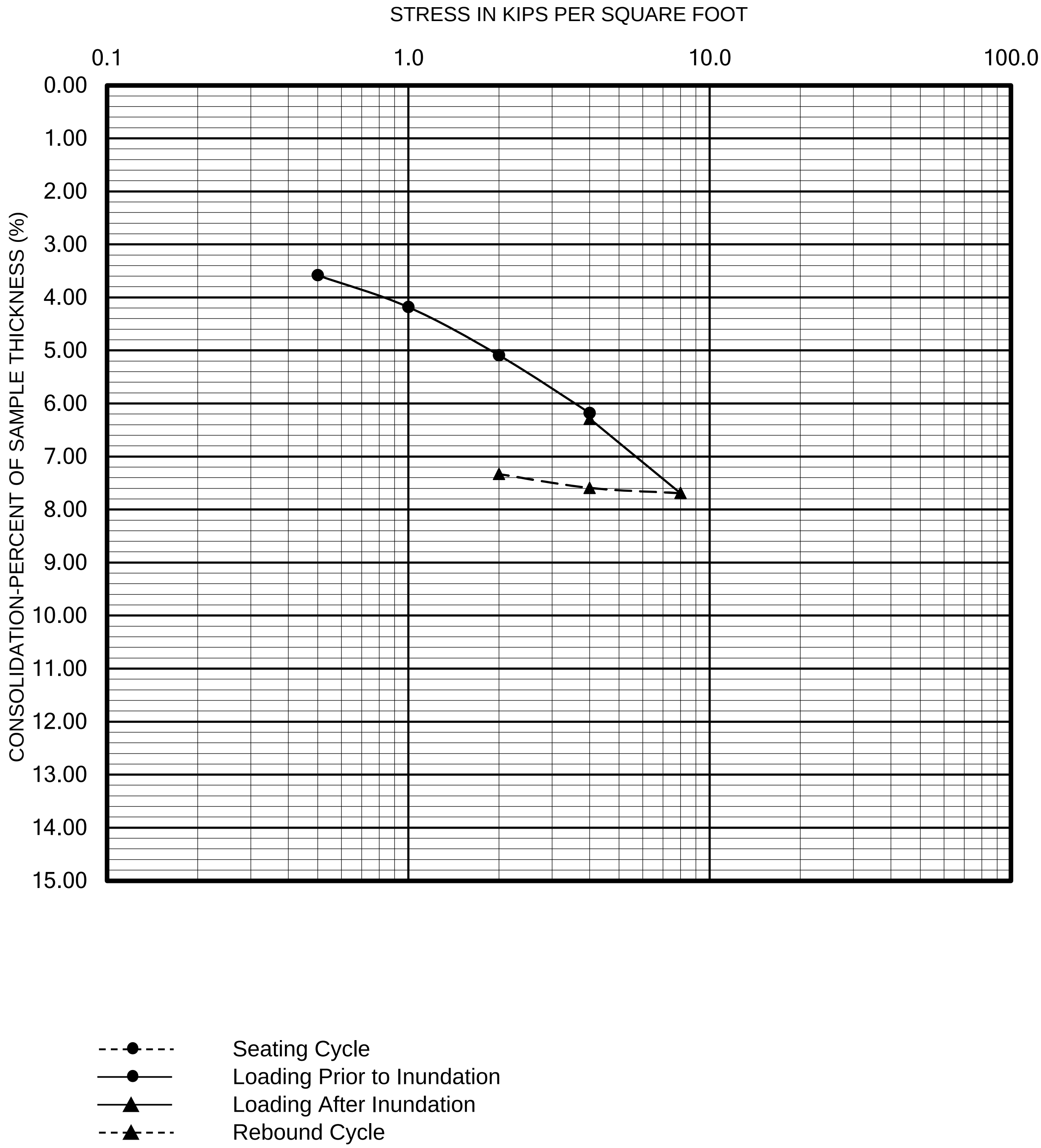
PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 2435

GEOTEK

CONSOLIDATION REPORT

B-1 @ 5 ft

CHECKED BY:	Lab: DI
PROJECT NO.: 1763-CR	Date: 10/4/2017



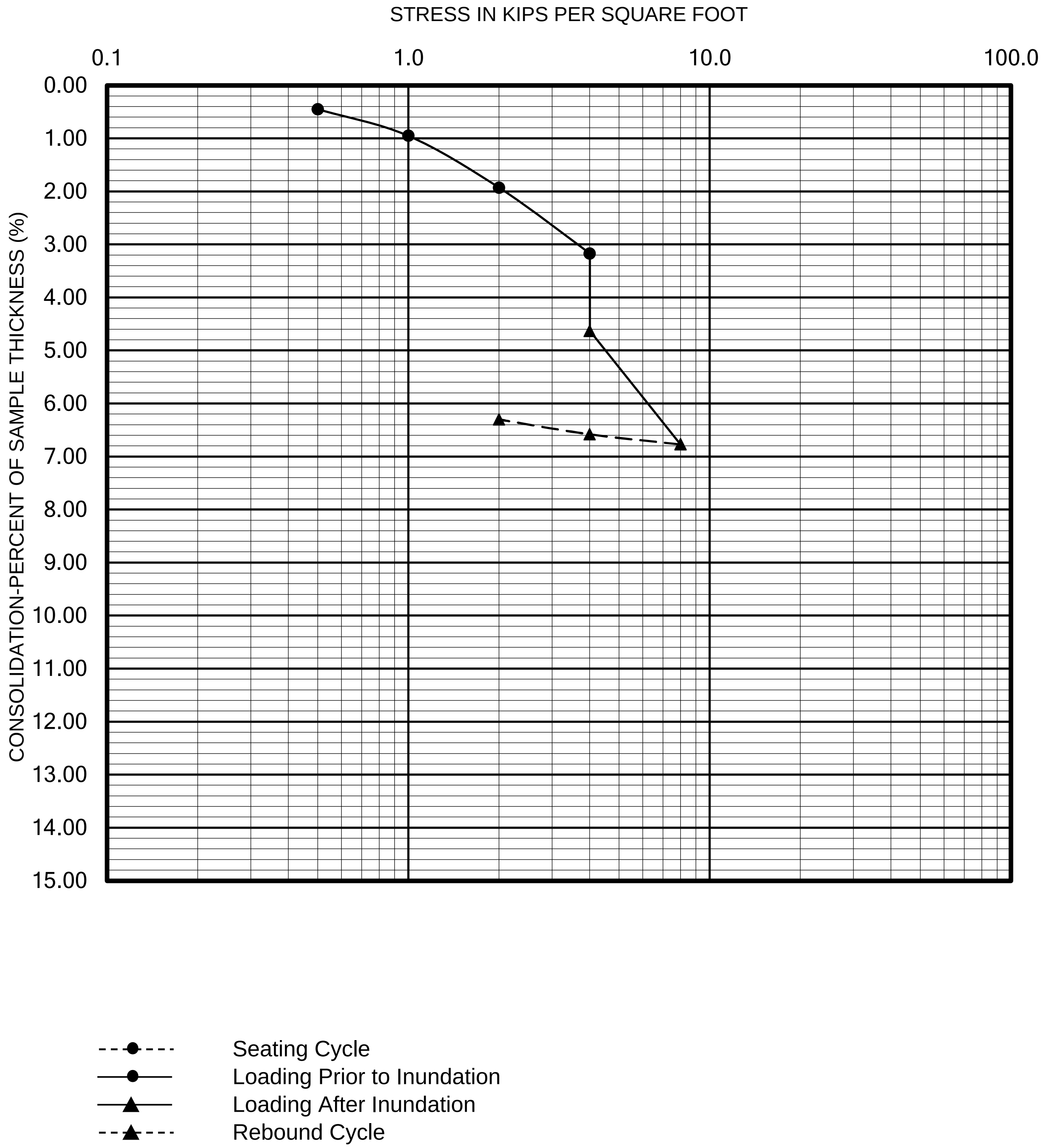
PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 2435

GEOTEK

CONSOLIDATION REPORT

B-3 @ 10ft

CHECKED BY:	Lab: DI
PROJECT NO.: 1763-CR	Date: 10/4/2017



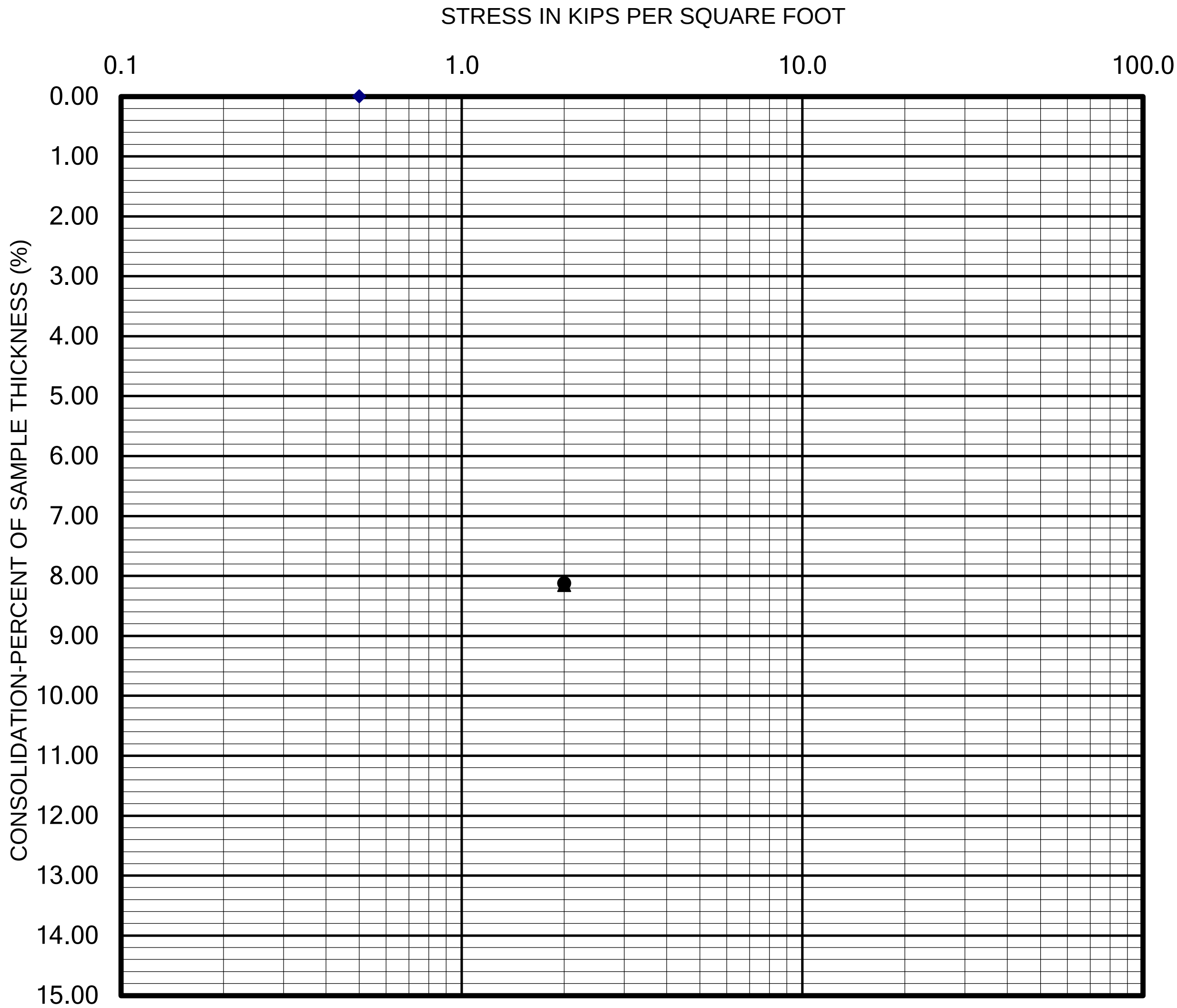
PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 2435

GEOTEK

CONSOLIDATION REPORT

B-4 @ 3 ft

CHECKED BY:	Lab: DI
PROJECT NO.: 1763-CR	Date: 10/4/2017



Seating Cycle
Loading Prior to Inundation
Loading After Inundation
Rebound Cycle

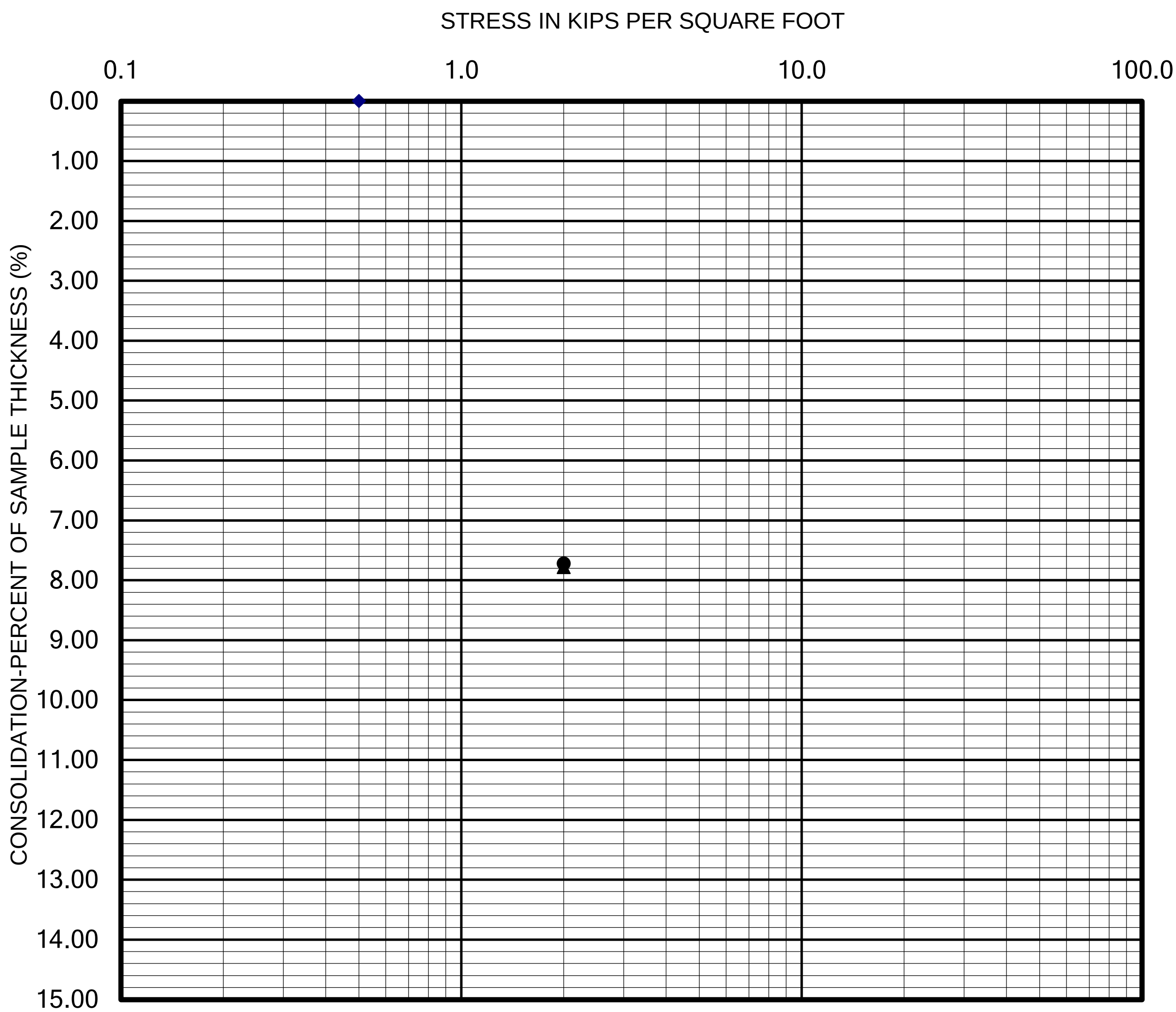
PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 4546

GEOTEK

COLLAPSE REPORT

Sample: B-6 @ 5 feet

CHECKED BY: DA	Lab: Corona
PROJECT NO.: 2573-CR	Date: 12/29/20



Seating Cycle
Loading Prior to Inundation
Loading After Inundation
Rebound Cycle

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 4546

GEOTEK

COLLAPSE REPORT

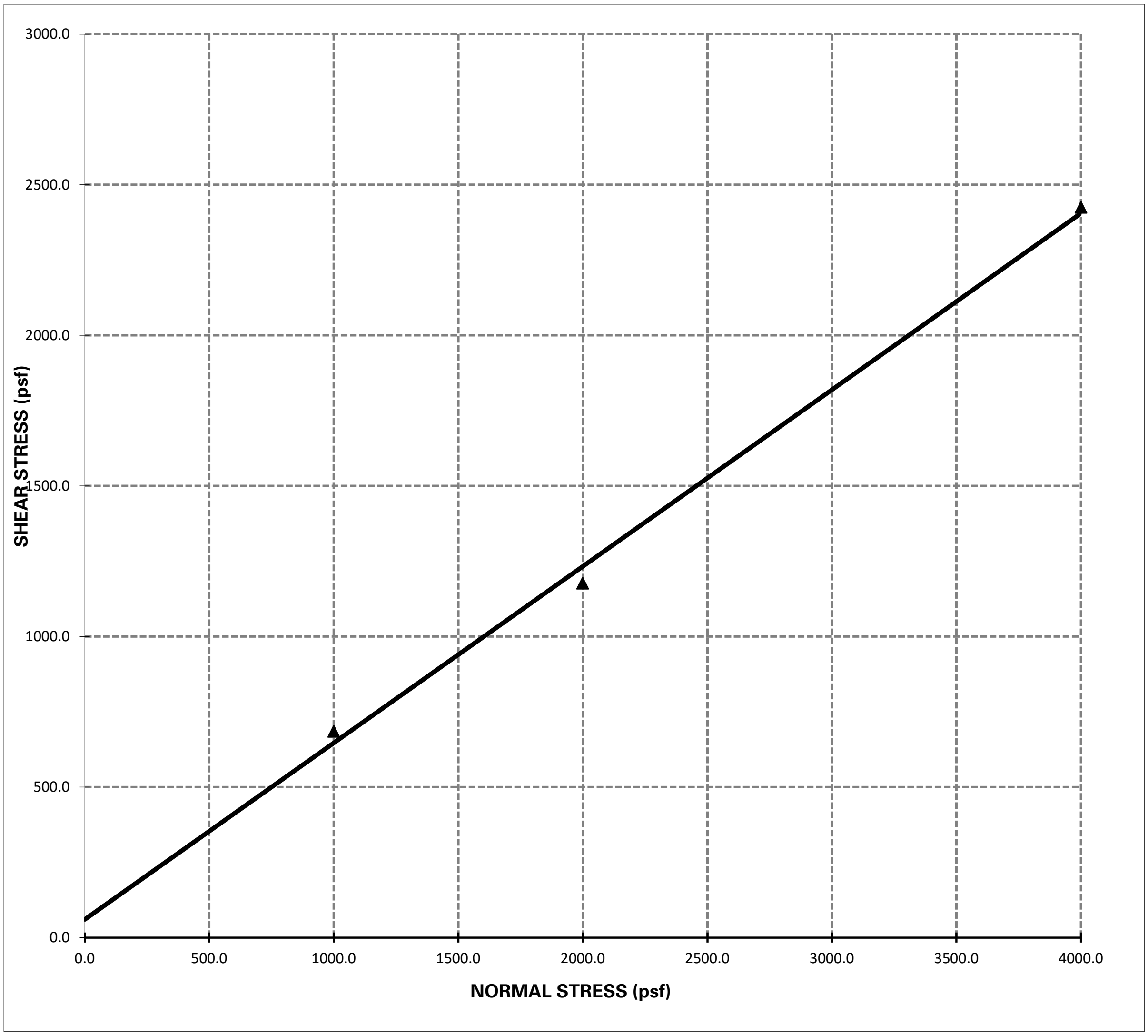
Sample: B-7 @ 5 feet

CHECKED BY: DA	Lab: Corona
PROJECT NO.: 2573-CR	Date: 12/29/20

DIRECT SHEAR TEST

GEOTEK

Project Name:		***Sample Location:***	B-1 @ 1 - 5 ft
Project Number:	1763-CR	***Date Tested:***	10/4/2017



Shear Strength: Φ = **30.4 °** , **C = 60.00 psf**

Notes: 1 - The soil specimen used in the shear box was a ring sample remolded to approximately 90% relative compaction from a bulk sample collected during the field investigation.

2 - The above reflect direct shear strength at saturated conditions.

3 - The tests were run at a shear rate of 0.035 in/min.

MOISTURE/DENSITY RELATIONSHIP

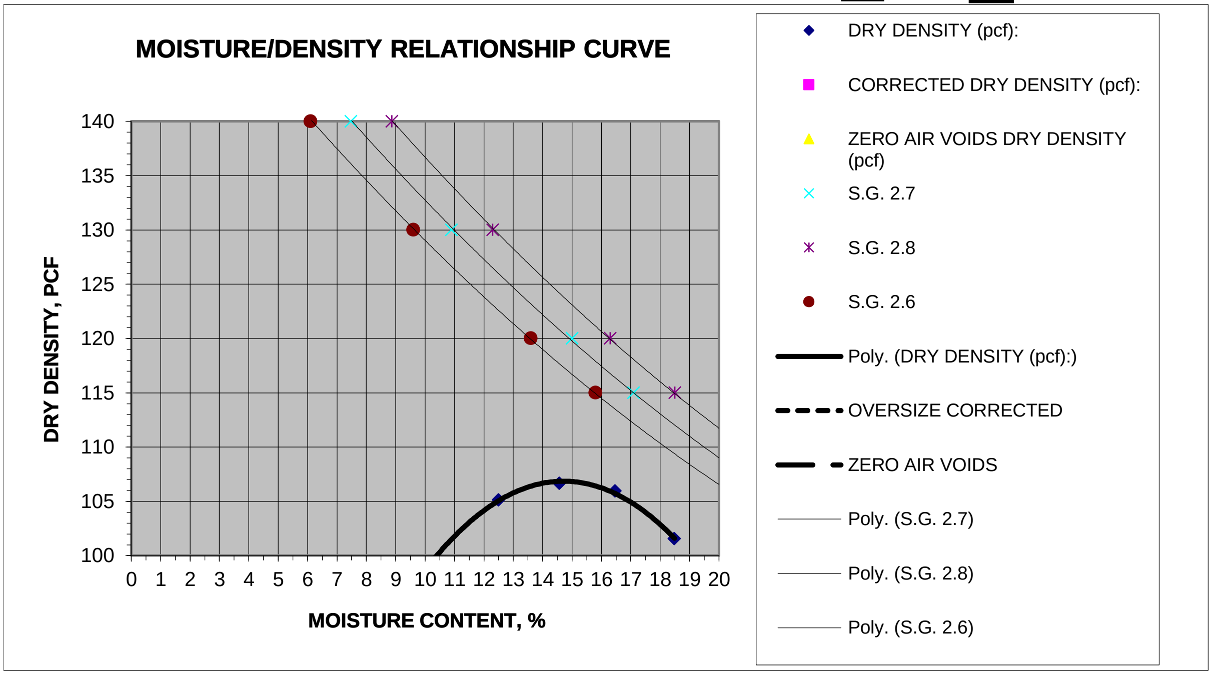
Client: Melia Homes
Project: Cypress Village
Location: Cypress Village
Material Type: Light Brown Fine Sand w/ Silt
Material Supplier:
Material Source:
Sample Location: B-1 @ 1 - 5

Job No.: 1763-CR
Lab No.: Corona

Sampled By: DRW
Received By: DA
Tested By: DA
Reviewed By:

Date Sampled: 0-Jan-00
Date Received:
Date Tested: 26-Sep-17
Date Reviewed:

Test Procedure: ASTM 1557 **Method:** A
Oversized Material (%): 0.0 **Correction Required:** ☐ yes ☒ no



MOISTURE DENSITY RELATIONSHIP VALUES

Maximum Dry Density, pcf	107.0	**@ Optimum Moisture, %**	15.0
Corrected Maximum Dry Density, pcf		**@ Optimum Moisture, %**	

MATERIAL DESCRIPTION

Grain Size Distribution:

	% Gravel (retained on No. 4)
	% Sand (Passing No. 4, Retained on No. 200)
	% Silt and Clay (Passing No. 200)

Atterberg Limits:

	Liquid Limit, %
	Plastic Limit, %
	Plasticity Index, %

Classification:

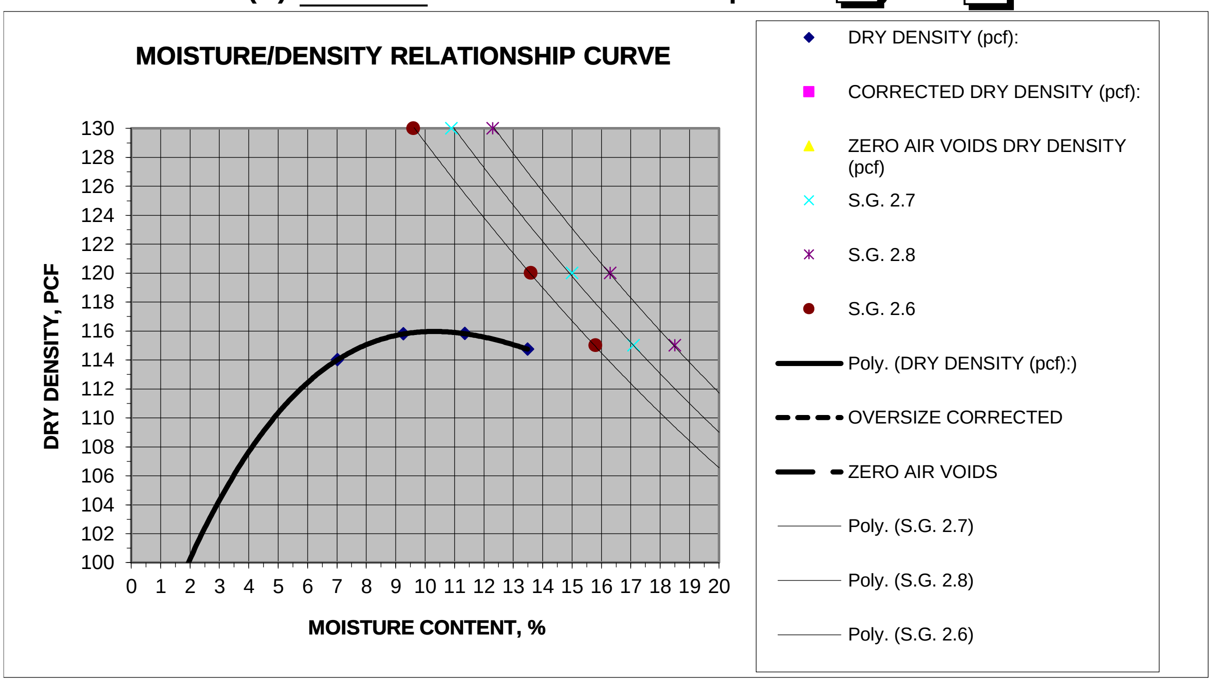
Unified Soils Classification:
AASHTO Soils Classification:

MOISTURE/DENSITY RELATIONSHIP

Client:	Melia Homes	**Job No.:**	2573-CR
Project:	Seniors Development	**Lab No.:**	Corona
Location:	Cypress		
Material Type:	Fine to medium sand with some silt, brown		
Material Supplier:	-		
Material Source:	-		
Sample Location:	B-7 @ 0-5 feet		
	-		
Sampled By:	JE	**Date Sampled:**	12/9/2020
Received By:	DA	**Date Received:**	12/9/2020
Tested By:	FS	**Date Tested:**	12/15/2020
Reviewed By:	DA	**Date Reviewed:**	12/29/2020

Test Procedure: ASTM D1557 **Method:** A

Oversized Material (%): 0.0 **Correction Required:** ☐ yes ☒ no



MOISTURE DENSITY RELATIONSHIP VALUES

Maximum Dry Density, pcf	116.0	**@ Optimum Moisture, %**	10.5
Corrected Maximum Dry Density, pcf		**@ Optimum Moisture, %**	

MATERIAL DESCRIPTION

Grain Size Distribution:

	% Gravel (retained on No. 4)
	% Sand (Passing No. 4, Retained on No. 200)
	% Silt and Clay (Passing No. 200)

Atterberg Limits:

	Liquid Limit, %
	Plastic Limit, %
	Plasticity Index, %

Classification:

Unified Soils Classification:

AASHTO Soils Classification:

GEOTEK

EXPANSION INDEX TEST

(ASTM D4829)

Client:	Melia Homes	***Tested/ Checked By:***	GP Lab No Corona
Project Number:	2573-CR	***Date Tested:***	12/18/2020
Project Location:	Seniors Developments, Cypress	***Sample Source:***	B-7 @ 0-5 feet
		Sample Description:	

Ring #:______ Ring Dia. : 4.01" Ring Ht. 1"

DENSITY DETERMINATION

A	Weight of compacted sample & ring (gm)	748.2
B	Weight of ring (gm)	362.4
C	Net weight of sample (gm)	**385.8**
D	Wet Density, lb / ft3 (C*0.3016)	**116.4**
E	Dry Density, lb / ft3 (D/1.F)	**104.8**

SATURATION DETERMINATION

F	Moisture Content, %	11.0
G	Specific Gravity, assumed	**2.70**
H	Unit Wt. of Water @ 20 °C, (pcf)	**62.4**
I	% Saturation	**48.9**

READINGS

DATE	TIME	READING	
12/18/2020	7:31	0.5770	Initial
12/18/2020	7:41	0.5760	10 min/Dry
12/19/2020		0.5760	Final

FINAL MOISTURE

Final Weight of wet sample & tare	% Moisture
775.2	**18.0**

EXPANSION INDEX = 0

Soil Analysis Lab Results

Client: Geotek Inc
Job Name: Cypress
Client Job Number: 1763-CR
Project X Job Number: S170927F
September 30, 2017

	Method	ASTM G187	ASTM G187	ASTM D516		ASTM D512B		SM 4500-E	SM 4500-C	SM 4500-D	ASTM G200	ASTM G51
Bore# / Description	**Depth**	**As-Rec'd Resistivity**	**Min-Resistivity**	**Sulfates**		**Chlorides**		**Nitrate**	**Ammonia**	**Sulfide**	**Redox**	**pH**
	(ft)	(Ohm-cm)	(Ohm-cm)	(mg/kg)	(wt%)	(mg/kg)	(wt%)	(mg/kg)	(mg/kg)	(mg/kg)	(mV)	
#1	1.0-5.0	8,710	5,226	3	0.0003	126	0.0126	27	26.6	1.65	206	9.35

Unk = Unknown
ND = 0 = Not Detected
NT = Not Tested
mg/kg = milligrams per kilogram (parts per million) of dry soil weight
Chemical Analysis performed on 1:3 Soil-To-Water extract

Please call if you have any questions.

Respectfully Submitted,

Ed

Ed Hernandez, M.Sc., P.E.
Sr. Corrosion Consultant
NACE Corrosion Technologist #16592
Professional Engineer
California No. M37102
ehernandez@projectxcorrosion.com

Soil Analysis Lab Results

Client: GeoTek, Inc.
Job Name: Orange-Ave
Client Job Number: 2573-CR
Project X Job Number: S201216A
December 17, 2020

	Method	ASTM D4327		ASTM D4327		ASTM G187		ASTM D4972	ASTM G200	SM 4500-S2-D	ASTM D4327	ASTM D6919	ASTM D6919	ASTM D6919	ASTM D6919	ASTM D6919	ASTM D6919	ASTM D4327	ASTM D4327
Bore# / Description	**Depth**	**Sulfates**		**Chlorides**		**Resistivity**		**pH**	**Redox**	**Sulfide**	**Nitrate**	**Ammonium**	**Lithium**	**Sodium**	**Potassium**	**Magnesium**	**Calcium**	**Fluoride**	**Phosphate**
		SO_4^{2-}		Cl^-		**As Rec'd**	**Minimum**			S^{2-}	NO_3^-	NH_4^+	Li^+	Na^+	K^+	Mg^{2+}	Ca^{2+}	F_2^{--}	PO_4^{3-}
	(ft)	(mg/kg)	(wt%)	(mg/kg)	(wt%)	(Ohm-cm)	(Ohm-cm)		(mV)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)
B-7	0-5	54.4	0.0054	20.4	0.0020	16,080	8,040	9.4	199	<0.01	0.6	38.6	ND	46.6	141.4	44.2	326.9	3.2	11.6

Cations and Anions, except Sulfide and Bicarbonate, tested with Ion Chromatography
mg/kg = milligrams per kilogram (parts per million) of dry soil weight
ND = 0 = Not Detected | NT = Not Tested | Unk = Unknown
Chemical Analysis performed on 1:3 Soil-To-Water extract

APPENDIX D

LIQUEFACTION AND SETTLEMENT ANALYSES

Updated Geotechnical Evaluation

Orange Avenue Seniors, Cypress, California

Project No. 2573-CR

GEOTEK

GeoTek, Inc.
1548 N. Maple Street
Corona, CA 92880
http://www.geotekusa.com

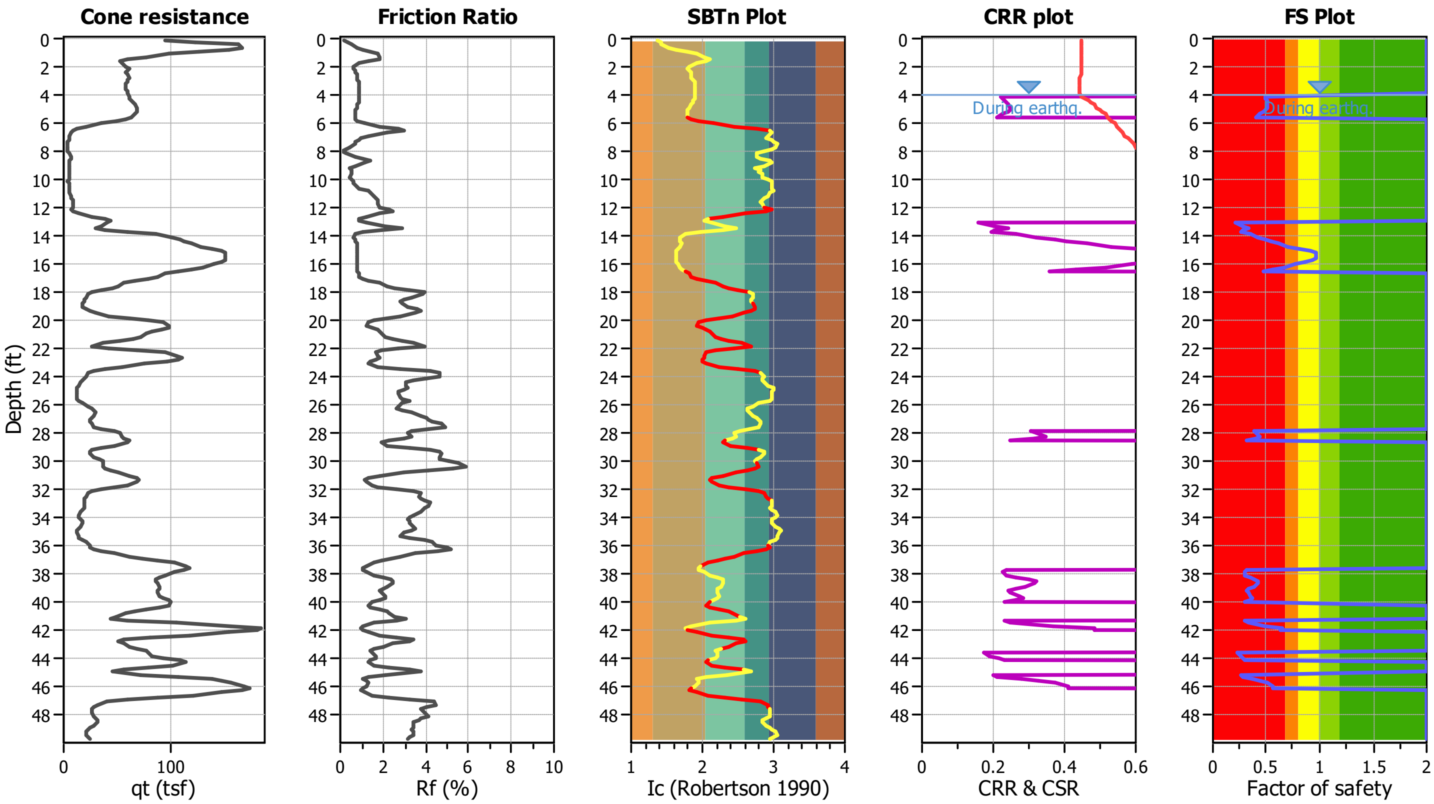
LIQUEFACTION ANALYSIS REPORT

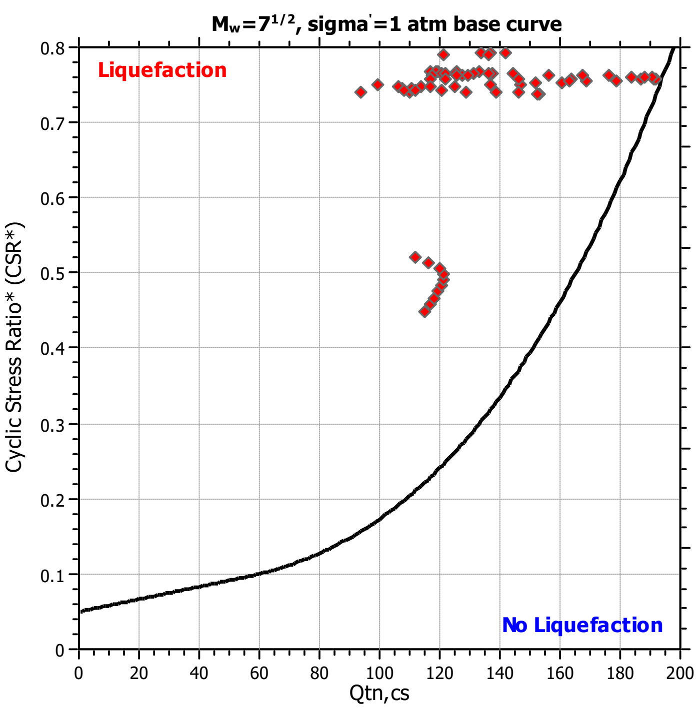
Project title : Orange Avenue Seniors **Location : Cypress, CA**

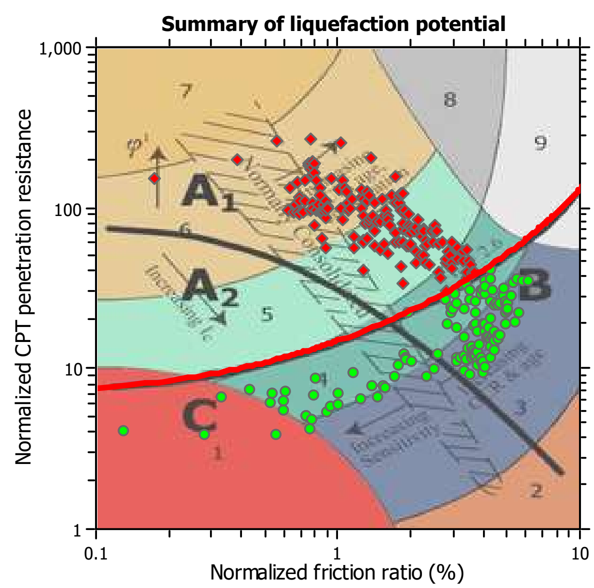
CPT file : CPT-1

Input parameters and analysis data

Analysis method:	NCEER (1998)	G.W.T. (in-situ):	8.00 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	4.00 ft	Fill height:	N/A		
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth applied:	Yes
Earthquake magnitude M_w:	6.72	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	Limit depth:	60.00 ft
Peak ground acceleration:	0.70	Unit weight calculation:	Based on SBT	K_σ applied:	Yes	MSF method:	Method based

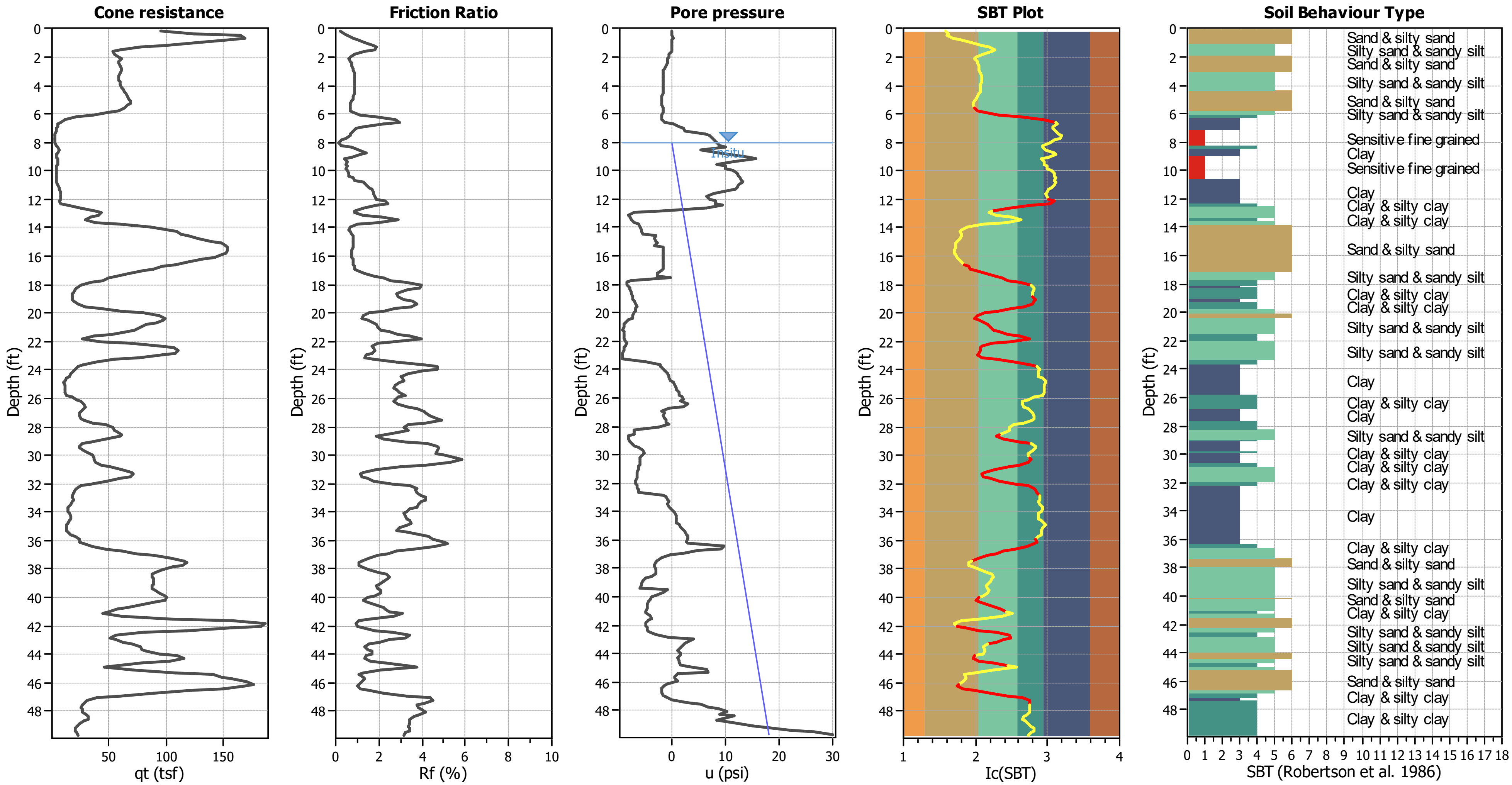






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and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

CPT basic interpretation plots



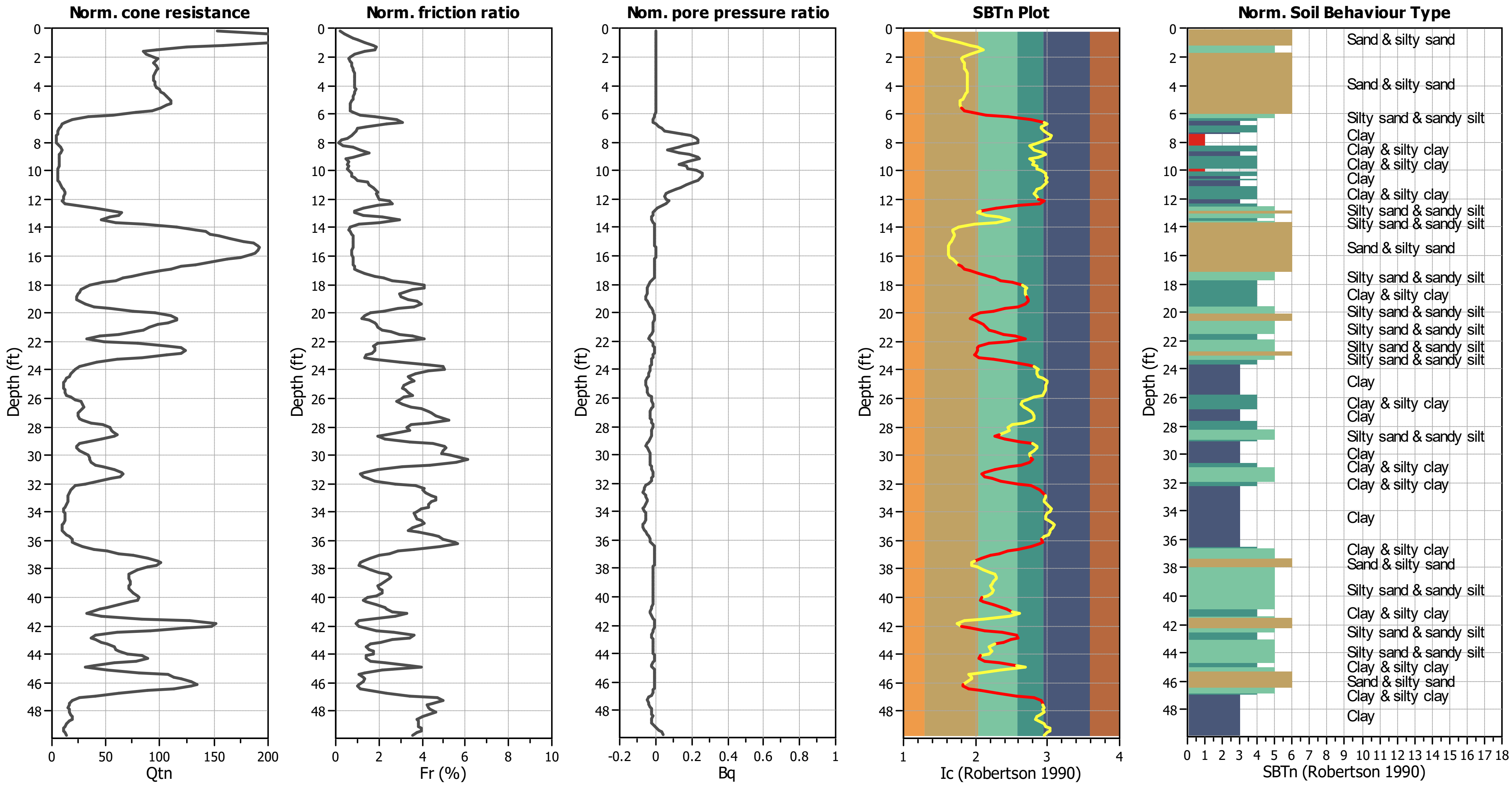
Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	4.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.72	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.70	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	8.00 ft	Fill height:	N/A	Limit depth:	60.00 ft

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)



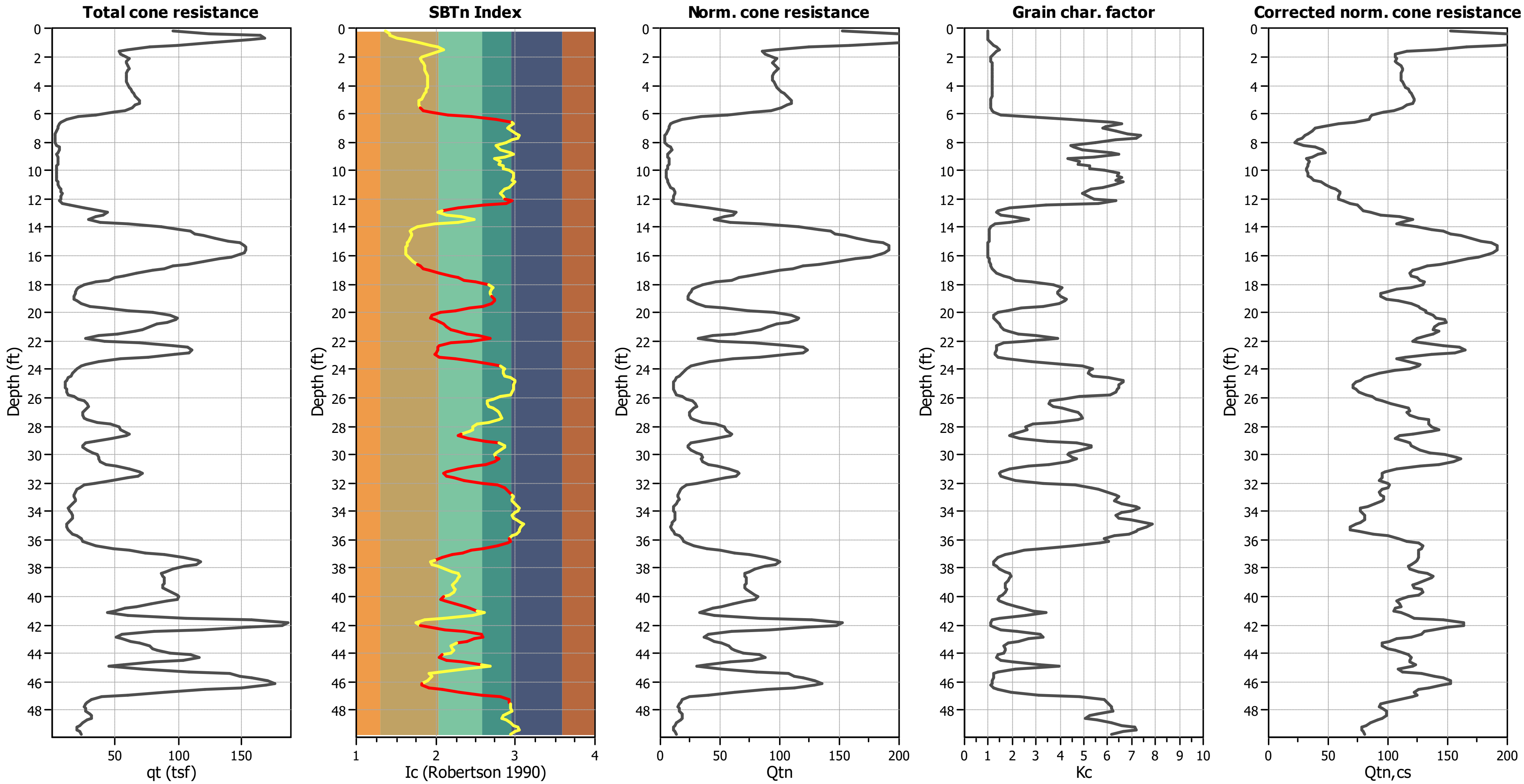
Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	4.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.72	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.70	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	8.00 ft	Fill height:	N/A	Limit depth:	60.00 ft

SBTn legend

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

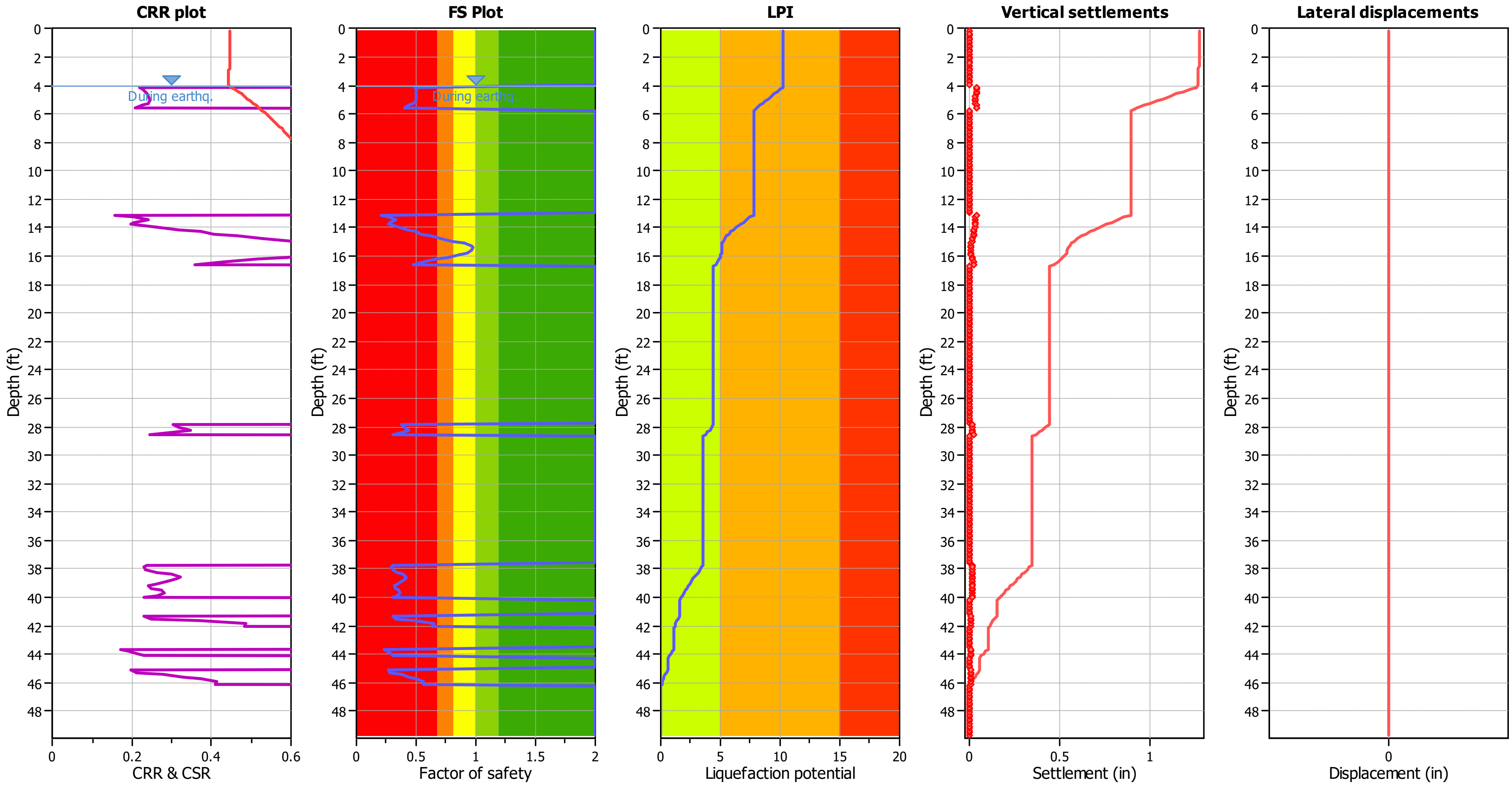
Liquefaction analysis overall plots (intermediate results)



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	4.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.72	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.70	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	8.00 ft	Fill height:	N/A	Limit depth:	60.00 ft

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	4.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_{σ} applied:	Yes
Earthquake magnitude M_w:	6.72	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.70	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	8.00 ft	Fill height:	N/A	Limit depth:	60.00 ft

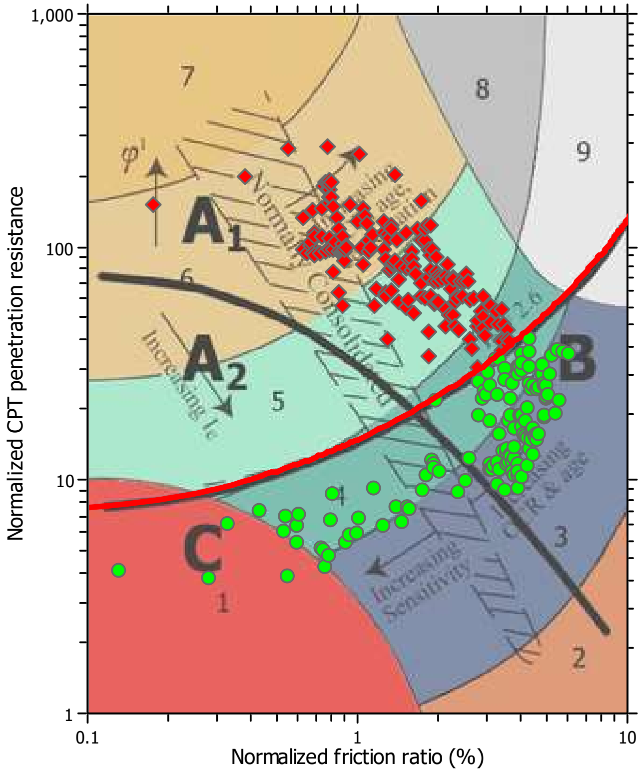
F.S. color scheme

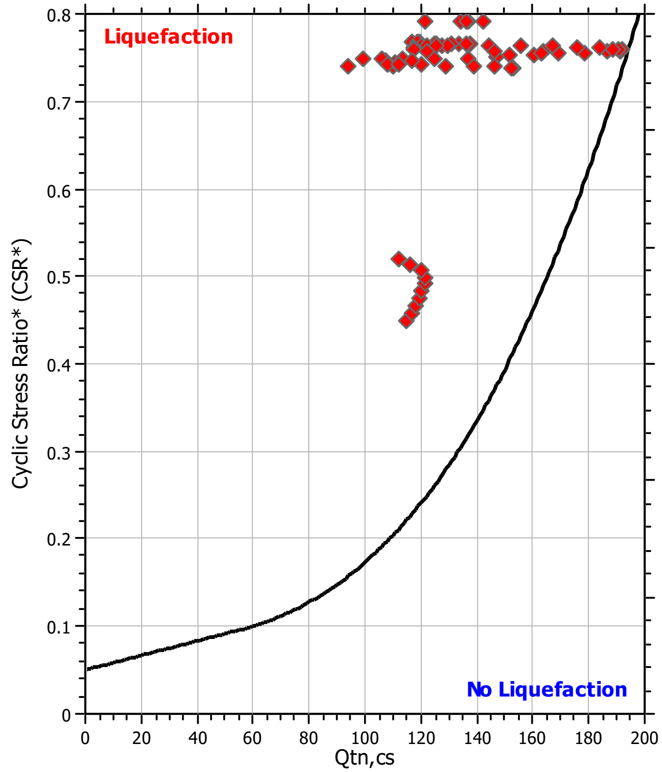
- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

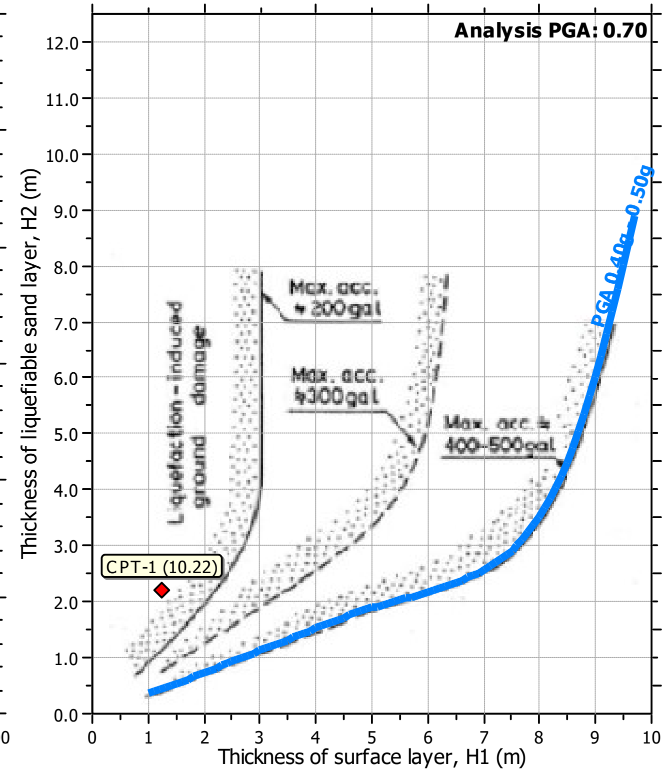
LPI color scheme

- Very high risk
- High risk
- Low risk

Liquefaction analysis summary plots



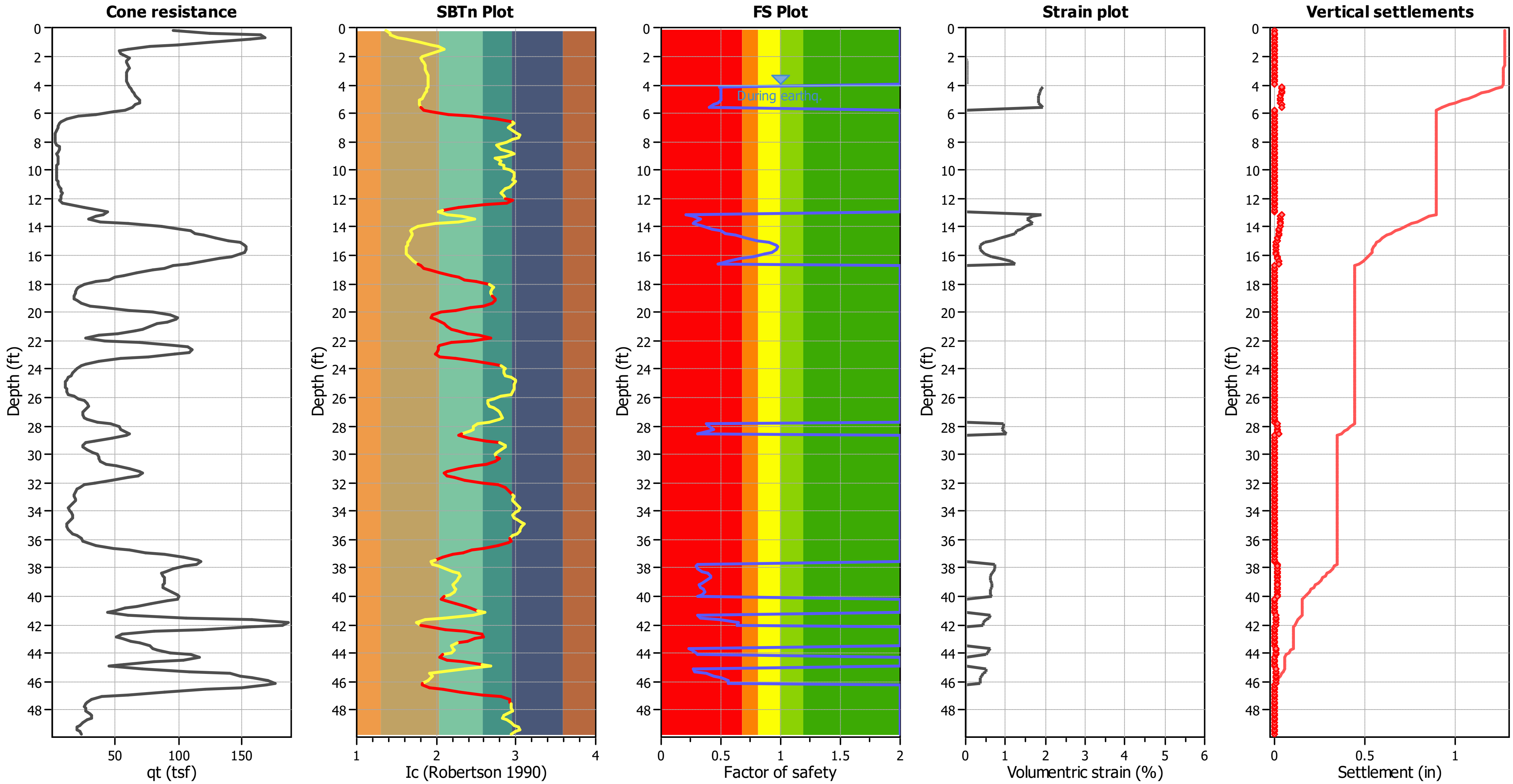




Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	4.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.72	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.70	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	8.00 ft	Fill height:	N/A	Limit depth:	60.00 ft

Estimation of post-earthquake settlements



Abbreviations

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

:: Post-earthquake settlement of dry sands ::

Depth (ft)	Ic	Q_{tn}	Kc	$Q_{tn,cs}$	$N_{1,60}$ (blows)	G_{max} (tsf)	CSR	Shear, γ (%)	$e_{vol(15)}$ (%)	N_c	e_v (%)	Settle. (in)
0.16	1.37	152.94	1.00	152.94	26	483	0.45	0.001	0.00	8.77	0.00	0.000
0.33	1.42	199.84	1.00	199.84	34	676	0.45	0.001	0.00	8.77	0.00	0.000
0.49	1.42	263.87	1.00	263.87	45	897	0.45	0.002	0.00	8.77	0.00	0.000
0.66	1.52	270.12	1.00	270.12	47	1035	0.45	0.002	0.00	8.77	0.00	0.000
0.82	1.63	251.09	1.00	251.09	46	1108	0.45	0.002	0.00	8.77	0.00	0.000
0.98	1.79	203.79	1.10	223.47	43	1096	0.45	0.003	0.00	8.77	0.00	0.000
1.15	1.93	158.52	1.22	193.41	39	1025	0.45	0.004	0.00	8.77	0.00	0.000
1.31	2.03	124.49	1.34	166.26	35	907	0.45	0.005	0.00	8.77	0.00	0.000
1.48	2.10	96.74	1.45	140.27	30	772	0.45	0.007	0.00	8.77	0.00	0.000
1.64	2.05	84.88	1.37	116.26	25	637	0.45	0.012	0.01	8.77	0.01	0.000
1.80	1.93	87.43	1.21	106.14	21	561	0.45	0.017	0.02	8.77	0.01	0.000
1.97	1.82	94.21	1.12	105.61	21	528	0.45	0.022	0.02	8.77	0.02	0.001
2.13	1.80	97.46	1.10	107.62	21	531	0.45	0.024	0.02	8.77	0.02	0.001
2.30	1.83	94.66	1.13	106.67	21	536	0.45	0.027	0.03	8.77	0.02	0.001
2.46	1.84	94.27	1.14	107.47	21	545	0.45	0.029	0.03	8.77	0.02	0.001
2.62	1.85	96.44	1.15	110.53	22	563	0.44	0.029	0.03	8.77	0.02	0.001
2.79	1.85	98.19	1.15	112.70	22	575	0.44	0.030	0.03	8.77	0.02	0.001
2.95	1.87	97.15	1.16	112.47	22	577	0.44	0.033	0.03	8.77	0.02	0.001
3.12	1.88	95.00	1.17	111.39	22	577	0.44	0.036	0.03	8.77	0.02	0.001
3.28	1.89	93.64	1.18	110.70	22	576	0.44	0.039	0.03	8.77	0.03	0.001
3.44	1.89	93.63	1.18	110.84	22	578	0.44	0.042	0.04	8.77	0.03	0.001
3.61	1.89	94.20	1.18	111.50	22	581	0.44	0.045	0.04	8.77	0.03	0.001
3.77	1.89	94.93	1.18	112.25	22	585	0.44	0.048	0.04	8.77	0.03	0.001
3.94	1.89	95.83	1.18	113.26	23	590	0.44	0.050	0.04	8.77	0.03	0.001

Total estimated settlement: 0.01

Abbreviations

Q_{tn}: Equivalent clean sand normalized cone resistance
K_c: Fines correction factor
$Q_{tn,cs}$: Post-liquefaction volumentric strain
G_{max}: Small strain shear modulus
CSR: Soil cyclic stress ratio
γ: Cyclic shear strain
$e_{vol(15)}$: Volumetric strain after 15 cycles
N_c: Equivalent number of cycles
e_v: Volumetric strain
Settle.: Calculated settlement

:: Post-earthquake settlement due to soil liquefaction ::

Depth (ft)	$Q_{tn,cs}$	FS	e_v (%)	DF	Settlement (in)	Depth (ft)	$Q_{tn,cs}$	FS	e_v (%)	DF	Settlement (in)
4.10	114.70	0.49	1.94	0.93	0.04	4.27	116.62	0.50	1.91	0.93	0.04
4.43	118.35	0.50	1.88	0.92	0.04	4.59	119.46	0.50	1.86	0.92	0.04
4.76	120.45	0.50	1.84	0.92	0.04	4.92	121.48	0.50	1.83	0.92	0.04
5.09	121.47	0.49	1.82	0.91	0.04	5.25	120.27	0.48	1.83	0.91	0.04
5.41	116.10	0.44	1.88	0.91	0.04	5.58	111.90	0.40	1.93	0.91	0.04
5.74	105.06	2.00	0.00	0.90	0.00	5.91	96.09	2.00	0.00	0.90	0.00
6.07	86.69	2.00	0.00	0.90	0.00	6.23	84.78	2.00	0.00	0.89	0.00
6.40	83.62	2.00	0.00	0.89	0.00	6.56	72.17	2.00	0.00	0.89	0.00
6.73	58.11	2.00	0.00	0.89	0.00	6.89	45.50	2.00	0.00	0.88	0.00
7.05	39.44	2.00	0.00	0.88	0.00	7.22	36.26	2.00	0.00	0.88	0.00
7.38	32.48	2.00	0.00	0.87	0.00	7.55	30.89	2.00	0.00	0.87	0.00

:: Post-earthquake settlement due to soil liquefaction :: (continued)

Depth (ft)	$Q_{tn,cs}$	FS	e_v (%)	DF	Settlement (in)	Depth (ft)	$Q_{tn,cs}$	FS	e_v (%)	DF	Settlement (in)
7.71	27.82	2.00	0.00	0.87	0.00	7.87	24.26	2.00	0.00	0.87	0.00
8.04	22.43	2.00	0.00	0.86	0.00	8.20	28.83	2.00	0.00	0.86	0.00
8.37	39.36	2.00	0.00	0.86	0.00	8.53	45.35	2.00	0.00	0.86	0.00
8.69	47.03	2.00	0.00	0.85	0.00	8.86	41.22	2.00	0.00	0.85	0.00
9.02	36.40	2.00	0.00	0.85	0.00	9.19	31.71	2.00	0.00	0.84	0.00
9.35	34.17	2.00	0.00	0.84	0.00	9.51	32.96	2.00	0.00	0.84	0.00
9.68	32.91	2.00	0.00	0.84	0.00	9.84	31.59	2.00	0.00	0.83	0.00
10.01	31.44	2.00	0.00	0.83	0.00	10.17	32.60	2.00	0.00	0.83	0.00
10.33	32.63	2.00	0.00	0.82	0.00	10.50	35.06	2.00	0.00	0.82	0.00
10.66	37.33	2.00	0.00	0.82	0.00	10.83	43.80	2.00	0.00	0.82	0.00
10.99	47.02	2.00	0.00	0.81	0.00	11.15	51.93	2.00	0.00	0.81	0.00
11.32	55.55	2.00	0.00	0.81	0.00	11.48	59.43	2.00	0.00	0.81	0.00
11.65	59.86	2.00	0.00	0.80	0.00	11.81	58.94	2.00	0.00	0.80	0.00
11.98	59.05	2.00	0.00	0.80	0.00	12.14	61.98	2.00	0.00	0.79	0.00
12.30	68.76	2.00	0.00	0.79	0.00	12.47	74.01	2.00	0.00	0.79	0.00
12.63	77.33	2.00	0.00	0.79	0.00	12.80	79.87	2.00	0.00	0.78	0.00
12.96	85.34	2.00	0.00	0.78	0.00	13.12	93.94	0.21	1.91	0.78	0.04
13.29	109.82	0.27	1.68	0.77	0.03	13.45	120.32	0.33	1.55	0.77	0.03
13.62	110.84	0.28	1.65	0.77	0.03	13.78	107.64	0.26	1.69	0.77	0.03
13.94	124.96	0.35	1.49	0.76	0.03	14.11	137.06	0.43	1.37	0.76	0.03
14.27	146.76	0.50	1.29	0.76	0.03	14.44	151.97	0.54	1.25	0.76	0.02
14.60	160.48	0.62	1.15	0.75	0.02	14.76	169.05	0.70	0.87	0.75	0.02
14.93	178.66	0.81	0.65	0.75	0.01	15.09	186.77	0.91	0.46	0.74	0.01
15.26	191.02	0.96	0.36	0.74	0.01	15.42	191.84	0.97	0.36	0.74	0.01
15.58	190.91	0.96	0.36	0.74	0.01	15.75	188.40	0.92	0.45	0.73	0.01
15.91	183.85	0.86	0.46	0.73	0.01	16.08	176.04	0.77	0.65	0.73	0.01
16.24	167.20	0.67	0.86	0.72	0.02	16.40	156.00	0.57	1.15	0.72	0.02
16.57	144.25	0.47	1.24	0.72	0.02	16.73	132.73	2.00	0.00	0.72	0.00
16.90	126.09	2.00	0.00	0.71	0.00	17.06	120.81	2.00	0.00	0.71	0.00
17.22	117.70	2.00	0.00	0.71	0.00	17.39	120.01	2.00	0.00	0.71	0.00
17.55	123.85	2.00	0.00	0.70	0.00	17.72	127.36	2.00	0.00	0.70	0.00
17.88	130.03	2.00	0.00	0.70	0.00	18.04	129.42	2.00	0.00	0.69	0.00
18.21	119.55	2.00	0.00	0.69	0.00	18.37	107.01	2.00	0.00	0.69	0.00
18.54	98.22	2.00	0.00	0.69	0.00	18.70	94.24	2.00	0.00	0.68	0.00
18.86	93.67	2.00	0.00	0.68	0.00	19.03	98.40	2.00	0.00	0.68	0.00
19.19	109.12	2.00	0.00	0.67	0.00	19.36	119.31	2.00	0.00	0.67	0.00
19.52	125.58	2.00	0.00	0.67	0.00	19.69	127.03	2.00	0.00	0.67	0.00
19.85	132.11	2.00	0.00	0.66	0.00	20.01	131.99	2.00	0.00	0.66	0.00
20.18	137.27	2.00	0.00	0.66	0.00	20.34	140.41	2.00	0.00	0.66	0.00
20.51	147.46	2.00	0.00	0.65	0.00	20.67	148.20	2.00	0.00	0.65	0.00
20.83	142.89	2.00	0.00	0.65	0.00	21.00	138.53	2.00	0.00	0.64	0.00
21.16	138.12	2.00	0.00	0.64	0.00	21.33	142.56	2.00	0.00	0.64	0.00
21.49	137.95	2.00	0.00	0.64	0.00	21.65	132.32	2.00	0.00	0.63	0.00
21.82	124.56	2.00	0.00	0.63	0.00	21.98	120.24	2.00	0.00	0.63	0.00
22.15	127.80	2.00	0.00	0.62	0.00	22.31	146.86	2.00	0.00	0.62	0.00
22.47	159.63	2.00	0.00	0.62	0.00	22.64	164.28	2.00	0.00	0.62	0.00
22.80	155.93	2.00	0.00	0.61	0.00	22.97	136.77	2.00	0.00	0.61	0.00
23.13	114.67	2.00	0.00	0.61	0.00	23.29	106.80	2.00	0.00	0.61	0.00

:: Post-earthquake settlement due to soil liquefaction :: (continued)

Depth (ft)	$Q_{tn,cs}$	FS	e_v (%)	DF	Settlement (in)	Depth (ft)	$Q_{tn,cs}$	FS	e_v (%)	DF	Settlement (in)
23.46	116.94	2.00	0.00	0.60	0.00	23.62	126.50	2.00	0.00	0.60	0.00
23.79	125.28	2.00	0.00	0.60	0.00	23.95	118.20	2.00	0.00	0.59	0.00
24.11	105.67	2.00	0.00	0.59	0.00	24.28	93.20	2.00	0.00	0.59	0.00
24.44	84.35	2.00	0.00	0.59	0.00	24.61	80.04	2.00	0.00	0.58	0.00
24.77	77.15	2.00	0.00	0.58	0.00	24.93	73.63	2.00	0.00	0.58	0.00
25.10	71.25	2.00	0.00	0.57	0.00	25.26	70.80	2.00	0.00	0.57	0.00
25.43	73.22	2.00	0.00	0.57	0.00	25.59	75.22	2.00	0.00	0.57	0.00
25.75	81.85	2.00	0.00	0.56	0.00	25.92	86.28	2.00	0.00	0.56	0.00
26.08	90.57	2.00	0.00	0.56	0.00	26.25	95.25	2.00	0.00	0.56	0.00
26.41	102.93	2.00	0.00	0.55	0.00	26.57	112.22	2.00	0.00	0.55	0.00
26.74	116.82	2.00	0.00	0.55	0.00	26.90	117.69	2.00	0.00	0.54	0.00
27.07	116.38	2.00	0.00	0.54	0.00	27.23	118.87	2.00	0.00	0.54	0.00
27.40	126.11	2.00	0.00	0.54	0.00	27.56	134.49	2.00	0.00	0.53	0.00
27.72	134.55	2.00	0.00	0.53	0.00	27.89	134.04	0.38	0.97	0.53	0.02
28.05	136.88	0.40	0.95	0.52	0.02	28.22	142.12	0.44	0.91	0.52	0.02
28.38	136.10	0.40	0.94	0.52	0.02	28.54	121.19	0.31	1.03	0.52	0.02
28.71	109.30	2.00	0.00	0.51	0.00	28.87	106.43	2.00	0.00	0.51	0.00
29.04	114.12	2.00	0.00	0.51	0.00	29.20	118.89	2.00	0.00	0.51	0.00
29.36	119.97	2.00	0.00	0.50	0.00	29.53	121.54	2.00	0.00	0.50	0.00
29.69	126.32	2.00	0.00	0.50	0.00	29.86	135.42	2.00	0.00	0.49	0.00
30.02	147.66	2.00	0.00	0.49	0.00	30.18	157.55	2.00	0.00	0.49	0.00
30.35	160.93	2.00	0.00	0.49	0.00	30.51	154.96	2.00	0.00	0.48	0.00
30.68	142.10	2.00	0.00	0.48	0.00	30.84	126.63	2.00	0.00	0.48	0.00
31.00	107.63	2.00	0.00	0.47	0.00	31.17	98.14	2.00	0.00	0.47	0.00
31.33	95.13	2.00	0.00	0.47	0.00	31.50	95.64	2.00	0.00	0.47	0.00
31.66	95.14	2.00	0.00	0.46	0.00	31.82	93.10	2.00	0.00	0.46	0.00
31.99	99.39	2.00	0.00	0.46	0.00	32.15	101.82	2.00	0.00	0.46	0.00
32.32	99.83	2.00	0.00	0.45	0.00	32.48	95.07	2.00	0.00	0.45	0.00
32.64	93.18	2.00	0.00	0.45	0.00	32.81	93.78	2.00	0.00	0.44	0.00
32.97	96.14	2.00	0.00	0.44	0.00	33.14	96.68	2.00	0.00	0.44	0.00
33.30	94.59	2.00	0.00	0.44	0.00	33.46	89.02	2.00	0.00	0.43	0.00
33.63	82.79	2.00	0.00	0.43	0.00	33.79	76.99	2.00	0.00	0.43	0.00
33.96	76.45	2.00	0.00	0.42	0.00	34.12	77.95	2.00	0.00	0.42	0.00
34.28	80.92	2.00	0.00	0.42	0.00	34.45	80.81	2.00	0.00	0.42	0.00
34.61	80.29	2.00	0.00	0.41	0.00	34.78	76.93	2.00	0.00	0.41	0.00
34.94	72.69	2.00	0.00	0.41	0.00	35.10	68.68	2.00	0.00	0.41	0.00
35.27	68.43	2.00	0.00	0.40	0.00	35.43	75.41	2.00	0.00	0.40	0.00
35.60	88.24	2.00	0.00	0.40	0.00	35.76	99.52	2.00	0.00	0.39	0.00
35.93	108.86	2.00	0.00	0.39	0.00	36.09	116.40	2.00	0.00	0.39	0.00
36.25	125.07	2.00	0.00	0.39	0.00	36.42	129.60	2.00	0.00	0.38	0.00
36.58	127.65	2.00	0.00	0.38	0.00	36.75	125.07	2.00	0.00	0.38	0.00
36.91	125.98	2.00	0.00	0.37	0.00	37.07	125.72	2.00	0.00	0.37	0.00
37.24	126.20	2.00	0.00	0.37	0.00	37.40	124.96	2.00	0.00	0.37	0.00
37.57	123.13	2.00	0.00	0.36	0.00	37.73	119.20	0.31	0.73	0.36	0.01
37.89	116.97	0.30	0.74	0.36	0.01	38.06	118.68	0.31	0.72	0.35	0.01
38.22	125.75	0.35	0.68	0.35	0.01	38.39	133.40	0.39	0.64	0.35	0.01
38.55	137.27	0.42	0.62	0.35	0.01	38.71	136.46	0.41	0.62	0.34	0.01
38.88	131.03	0.38	0.64	0.34	0.01	39.04	124.86	0.34	0.66	0.34	0.01

:: Post-earthquake settlement due to soil liquefaction :: (continued)

Depth (ft)	$Q_{tn,cs}$	FS	e_v (%)	DF	Settlement (in)	Depth (ft)	$Q_{tn,cs}$	FS	e_v (%)	DF	Settlement (in)
39.21	120.46	0.32	0.67	0.34	0.01	39.37	122.01	0.33	0.66	0.33	0.01
39.53	127.61	0.36	0.63	0.33	0.01	39.70	129.44	0.37	0.62	0.33	0.01
39.86	125.52	0.35	0.63	0.32	0.01	40.03	117.37	0.30	0.66	0.32	0.01
40.19	110.25	2.00	0.00	0.32	0.00	40.35	107.36	2.00	0.00	0.32	0.00
40.52	109.69	2.00	0.00	0.31	0.00	40.68	111.11	2.00	0.00	0.31	0.00
40.85	106.05	2.00	0.00	0.31	0.00	41.01	104.52	2.00	0.00	0.30	0.00
41.17	111.54	2.00	0.00	0.30	0.00	41.34	117.13	0.30	0.61	0.30	0.01
41.50	121.96	0.33	0.59	0.30	0.01	41.67	146.07	0.49	0.50	0.29	0.01
41.83	163.48	0.64	0.43	0.29	0.01	41.99	163.16	0.64	0.43	0.29	0.01
42.16	150.92	2.00	0.00	0.29	0.00	42.32	137.28	2.00	0.00	0.28	0.00
42.49	130.67	2.00	0.00	0.28	0.00	42.65	128.08	2.00	0.00	0.28	0.00
42.81	118.96	2.00	0.00	0.27	0.00	42.98	107.52	2.00	0.00	0.27	0.00
43.14	101.61	2.00	0.00	0.27	0.00	43.31	94.70	2.00	0.00	0.27	0.00
43.47	94.93	2.00	0.00	0.26	0.00	43.64	99.55	0.23	0.61	0.26	0.01
43.80	106.13	0.26	0.57	0.26	0.01	43.96	113.59	0.29	0.54	0.25	0.01
44.13	117.06	0.31	0.52	0.25	0.01	44.29	119.95	2.00	0.00	0.25	0.00
44.46	117.74	2.00	0.00	0.25	0.00	44.62	118.62	2.00	0.00	0.24	0.00
44.78	123.49	2.00	0.00	0.24	0.00	44.95	120.94	2.00	0.00	0.24	0.00
45.11	108.38	0.27	0.51	0.24	0.01	45.28	112.10	0.28	0.49	0.23	0.01
45.44	128.55	0.37	0.44	0.23	0.01	45.60	138.79	0.44	0.41	0.23	0.01
45.77	146.55	0.50	0.38	0.22	0.01	45.93	152.88	0.56	0.36	0.22	0.01
46.10	152.39	0.55	0.36	0.22	0.01	46.26	145.70	2.00	0.00	0.22	0.00
46.42	134.73	2.00	0.00	0.21	0.00	46.59	125.31	2.00	0.00	0.21	0.00
46.75	122.49	2.00	0.00	0.21	0.00	46.92	124.28	2.00	0.00	0.20	0.00
47.08	120.46	2.00	0.00	0.20	0.00	47.24	109.56	2.00	0.00	0.20	0.00
47.41	99.32	2.00	0.00	0.20	0.00	47.57	93.36	2.00	0.00	0.19	0.00
47.74	93.09	2.00	0.00	0.19	0.00	47.90	95.31	2.00	0.00	0.19	0.00
48.06	98.24	2.00	0.00	0.19	0.00	48.23	98.87	2.00	0.00	0.18	0.00
48.39	99.00	2.00	0.00	0.18	0.00	48.56	95.64	2.00	0.00	0.18	0.00
48.72	91.90	2.00	0.00	0.17	0.00	48.88	85.50	2.00	0.00	0.17	0.00
49.05	81.35	2.00	0.00	0.17	0.00	49.21	78.38	2.00	0.00	0.17	0.00
49.38	78.12	2.00	0.00	0.16	0.00	49.54	79.33	2.00	0.00	0.16	0.00
49.70	80.59	2.00	0.00	0.16	0.00						

Total estimated settlement: 1.26

Abbreviations

$Q_{tn,cs}$: Equivalent clean sand normalized cone resistance
FS: Factor of safety against liquefaction
e_v (%): Post-liquefaction volumentric strain
DF: e_v depth weighting factor
Settlement: Calculated settlement

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Corona, CA 92880
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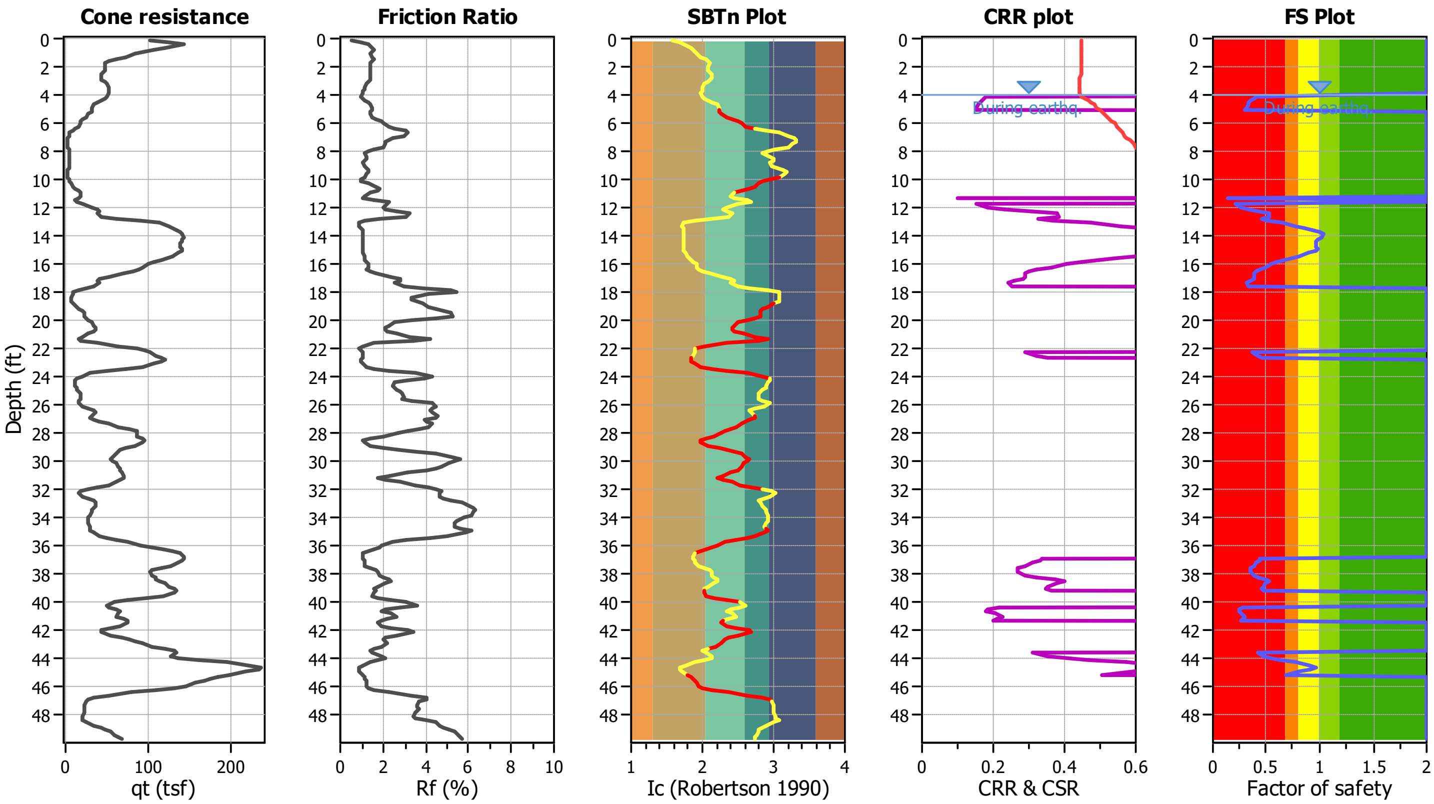
LIQUEFACTION ANALYSIS REPORT

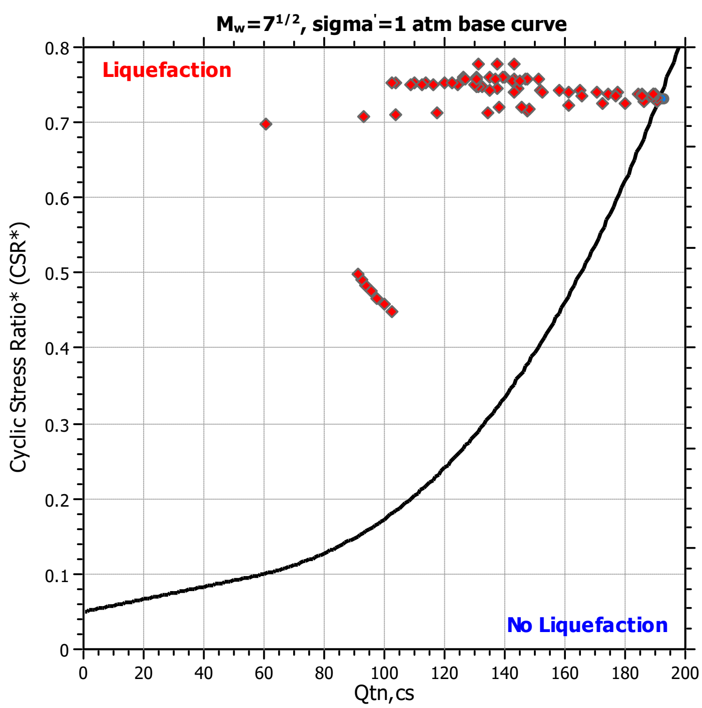
Project title : Orange Avenue Seniors **Location : Cypress, CA**

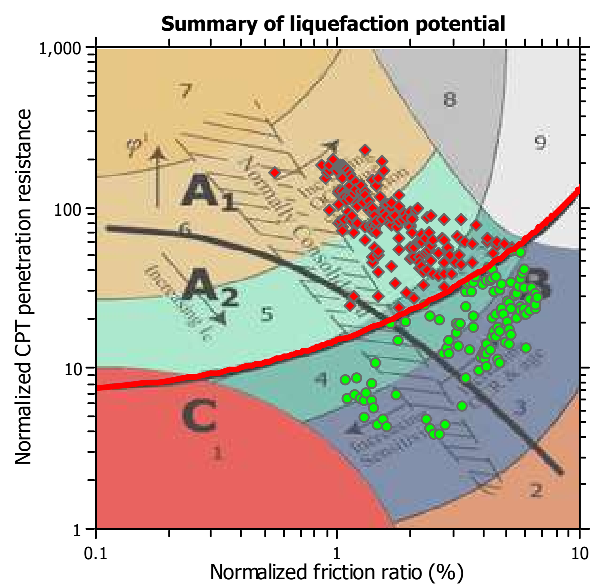
CPT file : CPT-2

Input parameters and analysis data

Analysis method:	NCEER (1998)	G.W.T. (in-situ):	8.00 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	4.00 ft	Fill height:	N/A	Limit depth applied:	Yes
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	60.00 ft
Earthquake magnitude M_w:	6.72	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.70	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

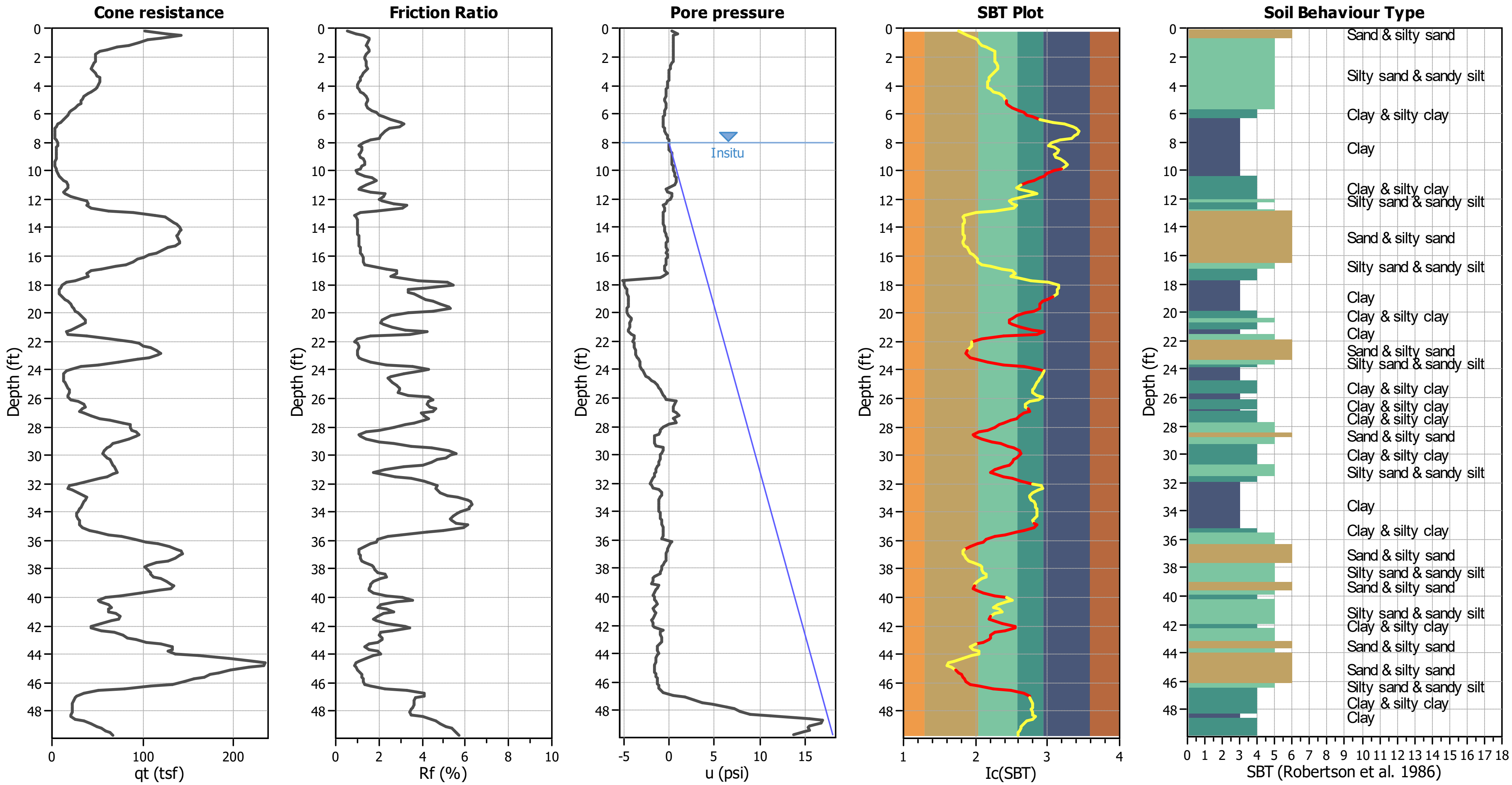






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and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

CPT basic interpretation plots



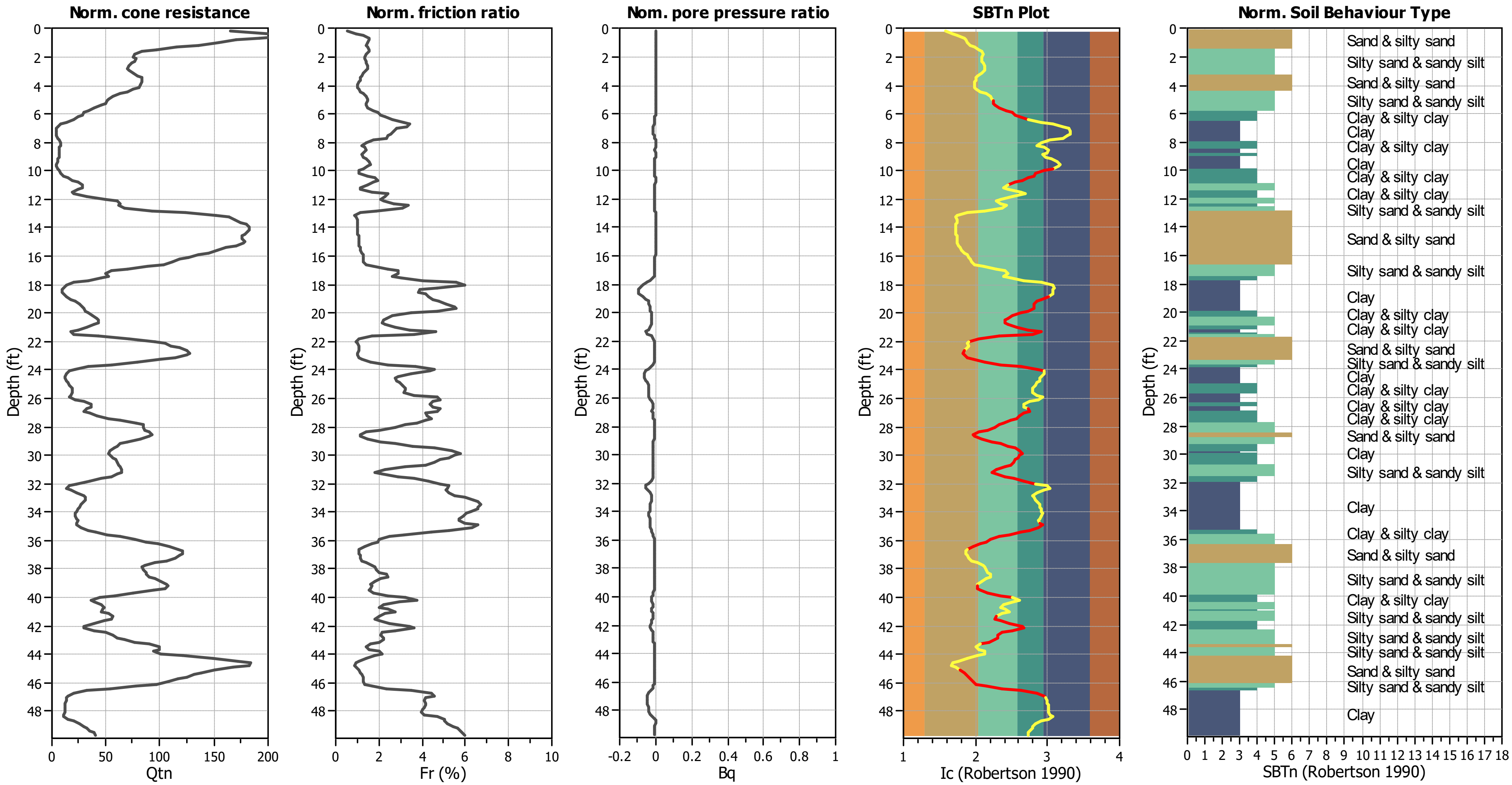
Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	4.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.72	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.70	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	8.00 ft	Fill height:	N/A	Limit depth:	60.00 ft

SBT 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

CPT basic interpretation plots (normalized)



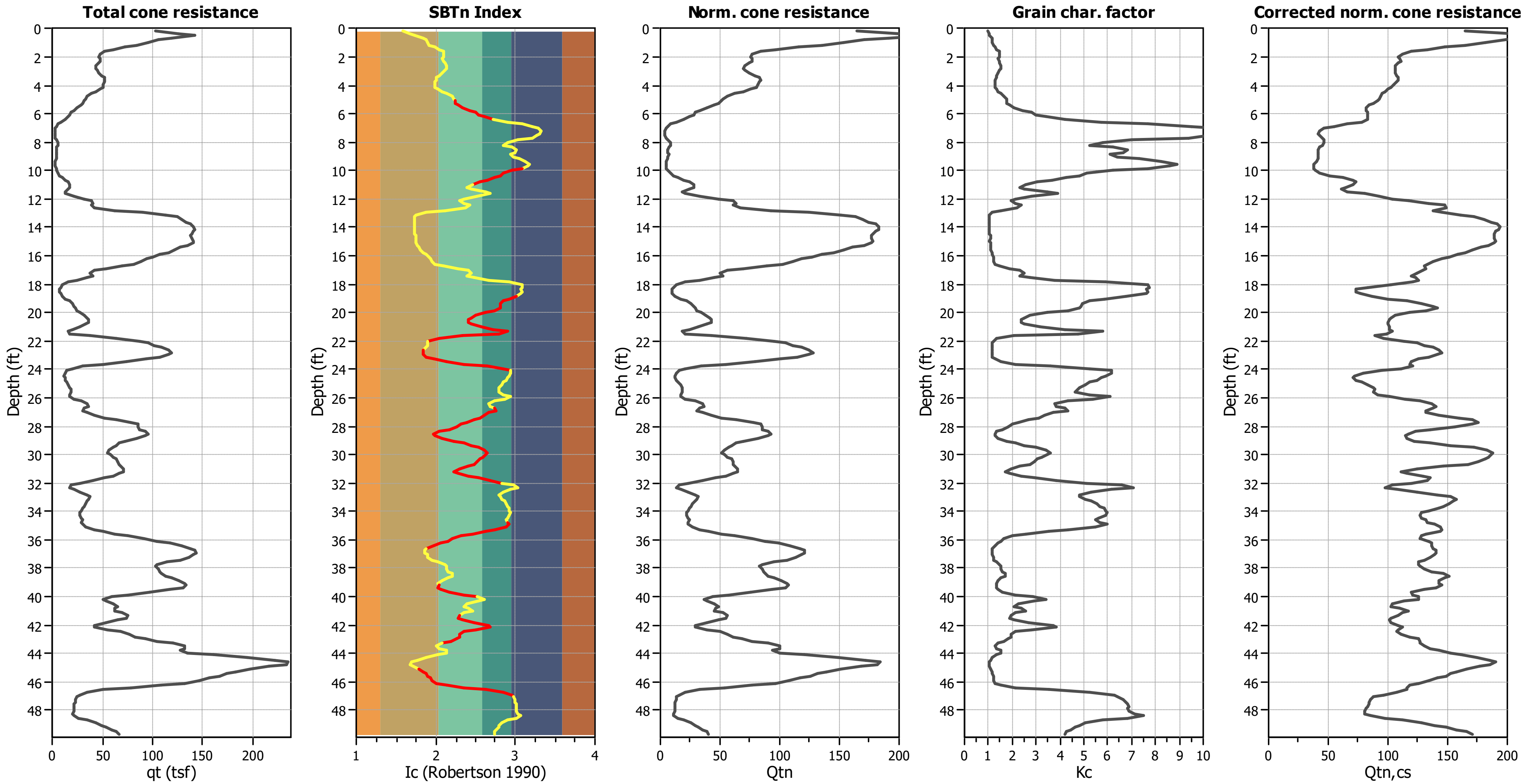
Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	4.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	$K_σ$ applied:	Yes
Earthquake magnitude M_w:	6.72	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.70	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	8.00 ft	Fill height:	N/A	Limit depth:	60.00 ft

SBTn legend

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

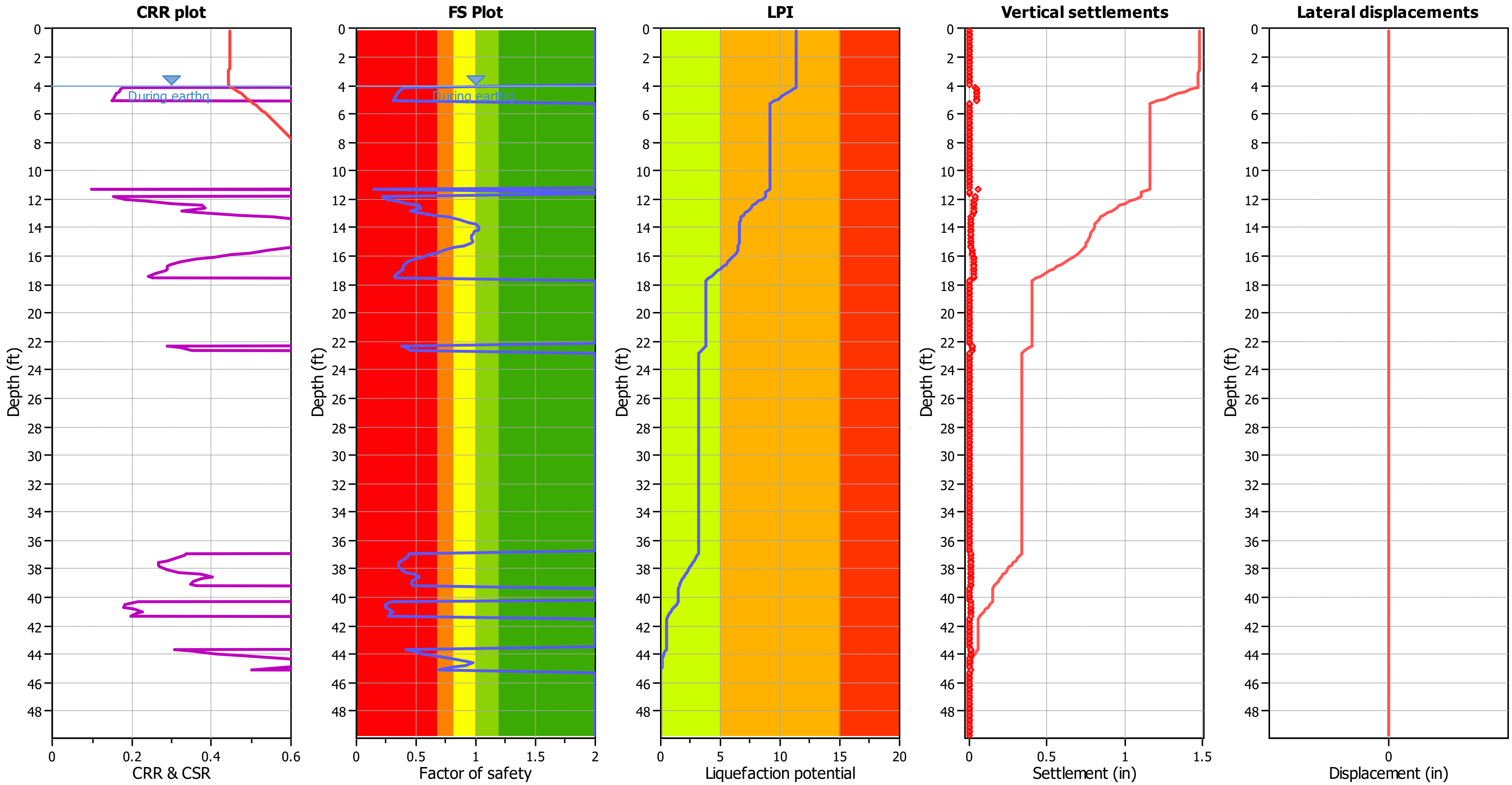
Liquefaction analysis overall plots (intermediate results)



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	4.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.72	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.70	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	8.00 ft	Fill height:	N/A	Limit depth:	60.00 ft

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	4.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.72	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.70	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	8.00 ft	Fill height:	N/A	Limit depth:	60.00 ft

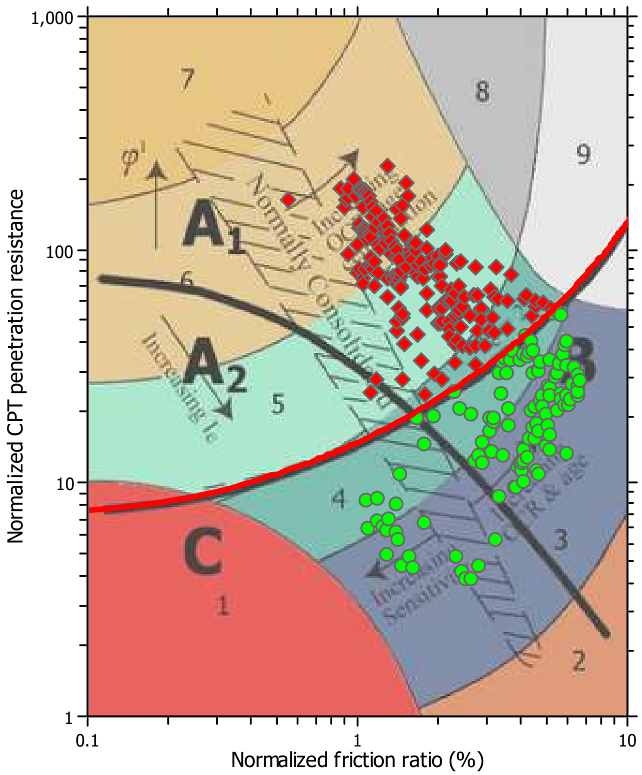
F.S. color scheme

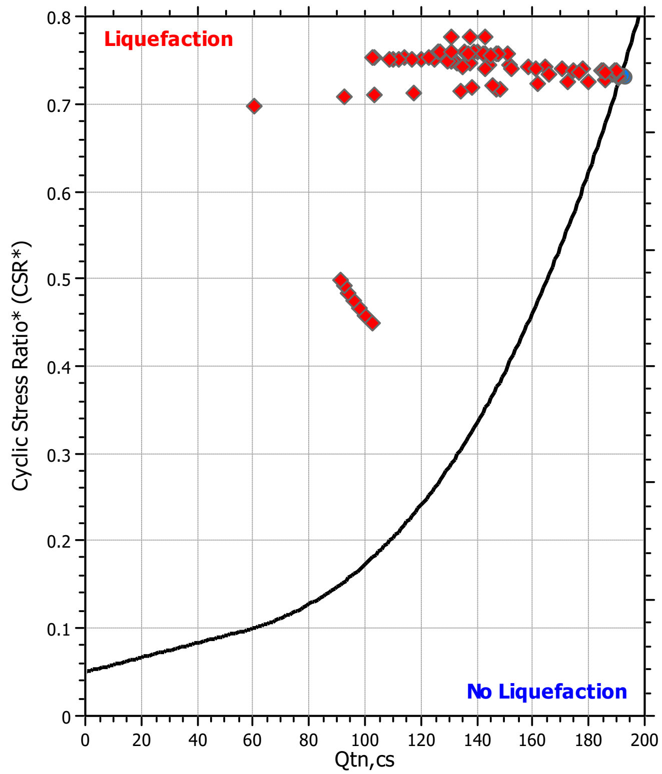
- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

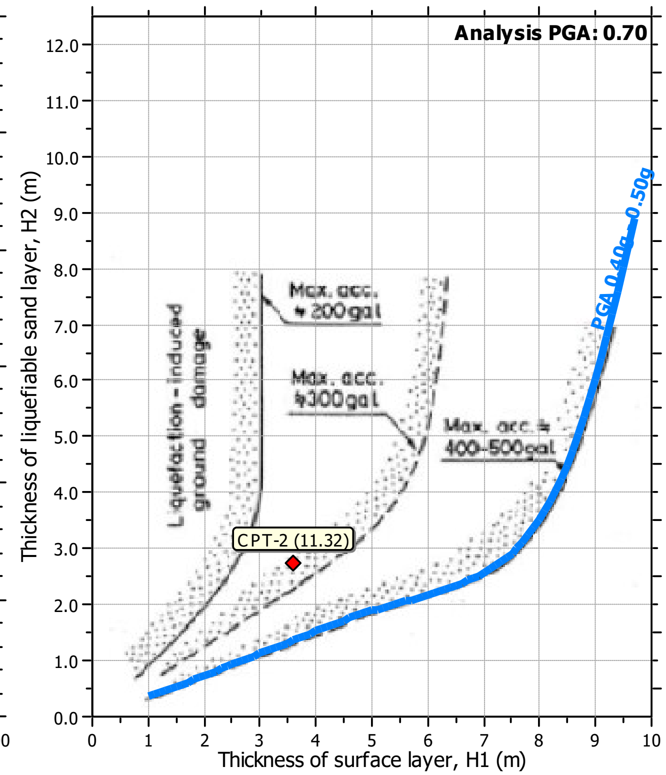
LPI color scheme

- Very high risk
- High risk
- Low risk

Liquefaction analysis summary plots



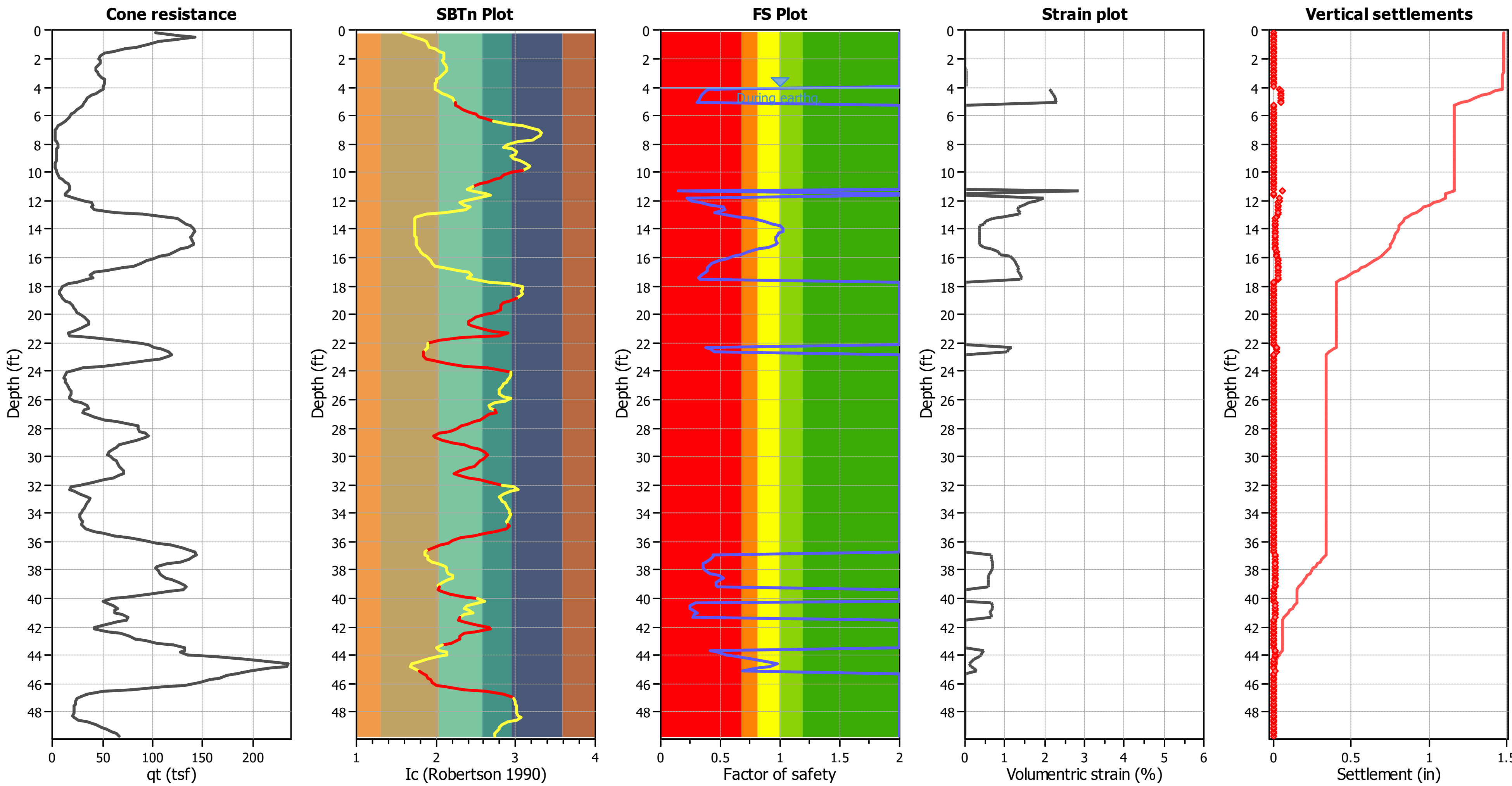




Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	4.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_o applied:	Yes
Earthquake magnitude M_w:	6.72	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.70	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	8.00 ft	Fill height:	N/A	Limit depth:	60.00 ft

Estimation of post-earthquake settlements



Abbreviations

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

:: Post-earthquake settlement of dry sands ::

Depth (ft)	Ic	Q_{tn}	Kc	$Q_{tn,cs}$	$N_{1,60}$ (blows)	G_{max} (tsf)	CSR	Shear, γ (%)	$e_{vol(15)}$ (%)	N_c	e_v (%)	Settle. (in)
0.16	1.58	164.62	1.00	164.62	29	681	0.45	0.001	0.00	8.77	0.00	0.000
0.33	1.68	201.83	1.02	206.06	38	943	0.45	0.001	0.00	8.77	0.00	0.000
0.49	1.73	227.20	1.06	241.15	46	1145	0.45	0.001	0.00	8.77	0.00	0.000
0.66	1.84	191.99	1.14	217.99	43	1103	0.45	0.002	0.00	8.77	0.00	0.000
0.82	1.88	170.23	1.17	198.59	39	1025	0.45	0.003	0.00	8.77	0.00	0.000
0.98	1.89	154.20	1.18	181.92	36	946	0.45	0.004	0.00	8.77	0.00	0.000
1.15	1.92	135.93	1.21	164.07	33	865	0.45	0.005	0.00	8.77	0.00	0.000
1.31	1.98	115.40	1.27	147.05	30	793	0.45	0.006	0.00	8.77	0.00	0.000
1.48	2.05	96.91	1.37	132.50	28	726	0.45	0.008	0.01	8.77	0.00	0.000
1.64	2.09	83.66	1.43	120.02	26	660	0.45	0.011	0.01	8.77	0.01	0.000
1.80	2.10	77.01	1.45	111.92	24	616	0.45	0.014	0.01	8.77	0.01	0.000
1.97	2.09	75.33	1.44	108.43	23	596	0.45	0.017	0.01	8.77	0.01	0.000
2.13	2.08	77.30	1.42	110.12	24	605	0.45	0.018	0.02	8.77	0.01	0.000
2.30	2.09	77.12	1.43	110.62	24	608	0.45	0.020	0.02	8.77	0.01	0.000
2.46	2.11	74.05	1.47	108.98	24	600	0.45	0.023	0.02	8.77	0.01	0.001
2.62	2.13	70.50	1.51	106.45	23	585	0.44	0.027	0.02	8.77	0.02	0.001
2.79	2.14	69.30	1.52	105.68	23	581	0.44	0.030	0.03	8.77	0.02	0.001
2.95	2.12	71.11	1.49	105.81	23	582	0.44	0.033	0.03	8.77	0.02	0.001
3.12	2.08	75.00	1.42	106.17	23	583	0.44	0.036	0.03	8.77	0.02	0.001
3.28	2.04	79.64	1.35	107.54	23	588	0.44	0.038	0.03	8.77	0.02	0.001
3.44	2.01	82.79	1.31	108.84	23	592	0.44	0.041	0.03	8.77	0.03	0.001
3.61	2.00	83.63	1.30	108.90	23	591	0.44	0.044	0.04	8.77	0.03	0.001
3.77	1.99	83.29	1.29	107.35	22	581	0.44	0.050	0.04	8.77	0.03	0.001
3.94	1.98	82.15	1.28	105.02	22	567	0.44	0.058	0.05	8.77	0.04	0.002

Total estimated settlement: 0.01

Abbreviations

Q_{tn}: Equivalent clean sand normalized cone resistance
K_c: Fines correction factor
$Q_{tn,cs}$: Post-liquefaction volumentric strain
G_{max}: Small strain shear modulus
CSR: Soil cyclic stress ratio
γ: Cyclic shear strain
$e_{vol(15)}$: Volumetric strain after 15 cycles
N_c: Equivalent number of cycles
e_v: Volumetric strain
Settle.: Calculated settlement

:: Post-earthquake settlement due to soil liquefaction ::

Depth (ft)	$Q_{tn,cs}$	FS	e_v (%)	DF	Settlement (in)	Depth (ft)	$Q_{tn,cs}$	FS	e_v (%)	DF	Settlement (in)
4.10	102.66	0.40	2.13	0.93	0.04	4.27	100.16	0.38	2.16	0.93	0.04
4.43	97.80	0.36	2.20	0.92	0.04	4.59	95.64	0.34	2.24	0.92	0.04
4.76	93.99	0.33	2.26	0.92	0.04	4.92	92.50	0.31	2.28	0.92	0.04
5.09	90.99	0.30	2.31	0.91	0.05	5.25	87.53	2.00	0.00	0.91	0.00
5.41	84.05	2.00	0.00	0.91	0.00	5.58	81.28	2.00	0.00	0.91	0.00
5.74	81.77	2.00	0.00	0.90	0.00	5.91	82.40	2.00	0.00	0.90	0.00
6.07	82.73	2.00	0.00	0.90	0.00	6.23	83.52	2.00	0.00	0.89	0.00
6.40	82.40	2.00	0.00	0.89	0.00	6.56	77.56	2.00	0.00	0.89	0.00
6.73	66.42	2.00	0.00	0.89	0.00	6.89	54.54	2.00	0.00	0.88	0.00
7.05	45.87	2.00	0.00	0.88	0.00	7.22	42.36	2.00	0.00	0.88	0.00
7.38	41.65	2.00	0.00	0.87	0.00	7.55	42.92	2.00	0.00	0.87	0.00

:: Post-earthquake settlement due to soil liquefaction :: (continued)

Depth (ft)	$Q_{tn,cs}$	FS	e_v (%)	DF	Settlement (in)	Depth (ft)	$Q_{tn,cs}$	FS	e_v (%)	DF	Settlement (in)
7.71	45.09	2.00	0.00	0.87	0.00	7.87	46.90	2.00	0.00	0.87	0.00
8.04	46.27	2.00	0.00	0.86	0.00	8.20	44.65	2.00	0.00	0.86	0.00
8.37	43.02	2.00	0.00	0.86	0.00	8.53	42.04	2.00	0.00	0.86	0.00
8.69	41.18	2.00	0.00	0.85	0.00	8.86	41.28	2.00	0.00	0.85	0.00
9.02	41.23	2.00	0.00	0.85	0.00	9.19	41.00	2.00	0.00	0.84	0.00
9.35	39.68	2.00	0.00	0.84	0.00	9.51	38.38	2.00	0.00	0.84	0.00
9.68	37.52	2.00	0.00	0.84	0.00	9.84	37.95	2.00	0.00	0.83	0.00
10.01	39.38	2.00	0.00	0.83	0.00	10.17	43.00	2.00	0.00	0.83	0.00
10.33	51.61	2.00	0.00	0.82	0.00	10.50	62.37	2.00	0.00	0.82	0.00
10.66	70.97	2.00	0.00	0.82	0.00	10.83	73.60	2.00	0.00	0.82	0.00
10.99	71.23	2.00	0.00	0.81	0.00	11.15	64.47	2.00	0.00	0.81	0.00
11.32	60.47	0.14	2.85	0.81	0.06	11.48	65.93	2.00	0.00	0.81	0.00
11.65	80.74	2.00	0.00	0.80	0.00	11.81	92.85	0.22	1.99	0.80	0.04
11.98	103.64	0.26	1.81	0.80	0.04	12.14	117.58	0.32	1.63	0.79	0.03
12.30	134.22	0.43	1.45	0.79	0.03	12.47	147.23	0.53	1.34	0.79	0.03
12.63	148.21	0.53	1.33	0.79	0.03	12.80	138.21	0.45	1.40	0.78	0.03
12.96	145.42	0.51	1.34	0.78	0.03	13.12	161.43	0.65	0.97	0.78	0.02
13.29	172.56	0.77	0.71	0.77	0.01	13.45	180.19	0.86	0.51	0.77	0.01
13.62	186.09	0.93	0.48	0.77	0.01	13.78	190.92	1.00	0.37	0.77	0.01
13.94	193.32	1.03	0.37	0.76	0.01	14.11	192.71	1.02	0.37	0.76	0.01
14.27	190.68	0.99	0.37	0.76	0.01	14.44	189.38	0.97	0.37	0.76	0.01
14.60	188.70	0.96	0.37	0.75	0.01	14.76	188.74	0.96	0.37	0.75	0.01
14.93	190.08	0.98	0.36	0.75	0.01	15.09	189.15	0.96	0.36	0.74	0.01
15.26	184.51	0.90	0.47	0.74	0.01	15.42	177.54	0.81	0.65	0.74	0.01
15.58	170.73	0.73	0.85	0.74	0.02	15.75	164.76	0.67	0.89	0.73	0.02
15.91	158.26	0.60	1.14	0.73	0.02	16.08	151.62	0.54	1.21	0.73	0.02
16.24	144.56	0.48	1.25	0.72	0.02	16.40	137.58	0.43	1.30	0.72	0.03
16.57	132.71	0.40	1.33	0.72	0.03	16.73	130.78	0.39	1.34	0.72	0.03
16.90	131.19	0.39	1.33	0.71	0.03	17.06	129.85	0.38	1.34	0.71	0.03
17.22	124.54	0.35	1.38	0.71	0.03	17.39	119.86	0.32	1.42	0.71	0.03
17.55	122.76	0.33	1.39	0.70	0.03	17.72	125.53	2.00	0.00	0.70	0.00
17.88	120.46	2.00	0.00	0.70	0.00	18.04	102.69	2.00	0.00	0.69	0.00
18.21	83.44	2.00	0.00	0.69	0.00	18.37	73.14	2.00	0.00	0.69	0.00
18.54	72.59	2.00	0.00	0.69	0.00	18.70	76.97	2.00	0.00	0.68	0.00
18.86	86.79	2.00	0.00	0.68	0.00	19.03	100.18	2.00	0.00	0.68	0.00
19.19	115.53	2.00	0.00	0.67	0.00	19.36	128.27	2.00	0.00	0.67	0.00
19.52	138.27	2.00	0.00	0.67	0.00	19.69	141.15	2.00	0.00	0.67	0.00
19.85	131.97	2.00	0.00	0.66	0.00	20.01	117.78	2.00	0.00	0.66	0.00
20.18	105.60	2.00	0.00	0.66	0.00	20.34	102.52	2.00	0.00	0.66	0.00
20.51	101.24	2.00	0.00	0.65	0.00	20.67	100.22	2.00	0.00	0.65	0.00
20.83	99.58	2.00	0.00	0.65	0.00	21.00	101.50	2.00	0.00	0.64	0.00
21.16	101.50	2.00	0.00	0.64	0.00	21.33	103.70	2.00	0.00	0.64	0.00
21.49	98.41	2.00	0.00	0.64	0.00	21.65	89.08	2.00	0.00	0.63	0.00
21.82	96.70	2.00	0.00	0.63	0.00	21.98	113.80	2.00	0.00	0.63	0.00
22.15	124.25	2.00	0.00	0.62	0.00	22.31	131.02	0.37	1.16	0.62	0.02
22.47	137.50	0.41	1.11	0.62	0.02	22.64	142.85	0.45	1.07	0.62	0.02
22.80	144.65	2.00	0.00	0.61	0.00	22.97	141.54	2.00	0.00	0.61	0.00
23.13	134.28	2.00	0.00	0.61	0.00	23.29	126.07	2.00	0.00	0.61	0.00

:: Post-earthquake settlement due to soil liquefaction :: (continued)

Depth (ft)	$Q_{tn,cs}$	FS	e_v (%)	DF	Settlement (in)	Depth (ft)	$Q_{tn,cs}$	FS	e_v (%)	DF	Settlement (in)
23.46	119.22	2.00	0.00	0.60	0.00	23.62	117.81	2.00	0.00	0.60	0.00
23.79	120.34	2.00	0.00	0.60	0.00	23.95	111.07	2.00	0.00	0.59	0.00
24.11	95.13	2.00	0.00	0.59	0.00	24.28	79.77	2.00	0.00	0.59	0.00
24.44	71.64	2.00	0.00	0.59	0.00	24.61	70.44	2.00	0.00	0.58	0.00
24.77	73.05	2.00	0.00	0.58	0.00	24.93	77.61	2.00	0.00	0.58	0.00
25.10	83.16	2.00	0.00	0.57	0.00	25.26	87.66	2.00	0.00	0.57	0.00
25.43	88.88	2.00	0.00	0.57	0.00	25.59	87.83	2.00	0.00	0.57	0.00
25.75	91.88	2.00	0.00	0.56	0.00	25.92	101.36	2.00	0.00	0.56	0.00
26.08	114.24	2.00	0.00	0.56	0.00	26.25	126.33	2.00	0.00	0.56	0.00
26.41	136.03	2.00	0.00	0.55	0.00	26.57	139.69	2.00	0.00	0.55	0.00
26.74	137.86	2.00	0.00	0.55	0.00	26.90	131.38	2.00	0.00	0.54	0.00
27.07	131.21	2.00	0.00	0.54	0.00	27.23	142.73	2.00	0.00	0.54	0.00
27.40	160.10	2.00	0.00	0.54	0.00	27.56	171.19	2.00	0.00	0.53	0.00
27.72	175.79	2.00	0.00	0.53	0.00	27.89	170.18	2.00	0.00	0.53	0.00
28.05	155.86	2.00	0.00	0.52	0.00	28.22	135.71	2.00	0.00	0.52	0.00
28.38	122.62	2.00	0.00	0.52	0.00	28.54	116.57	2.00	0.00	0.52	0.00
28.71	115.03	2.00	0.00	0.51	0.00	28.87	116.10	2.00	0.00	0.51	0.00
29.04	122.48	2.00	0.00	0.51	0.00	29.20	135.51	2.00	0.00	0.51	0.00
29.36	152.29	2.00	0.00	0.50	0.00	29.53	171.40	2.00	0.00	0.50	0.00
29.69	183.01	2.00	0.00	0.50	0.00	29.86	187.58	2.00	0.00	0.49	0.00
30.02	186.25	2.00	0.00	0.49	0.00	30.18	184.41	2.00	0.00	0.49	0.00
30.35	181.23	2.00	0.00	0.49	0.00	30.51	176.00	2.00	0.00	0.48	0.00
30.68	166.81	2.00	0.00	0.48	0.00	30.84	146.36	2.00	0.00	0.48	0.00
31.00	124.60	2.00	0.00	0.47	0.00	31.17	111.28	2.00	0.00	0.47	0.00
31.33	116.10	2.00	0.00	0.47	0.00	31.50	128.90	2.00	0.00	0.47	0.00
31.66	135.72	2.00	0.00	0.46	0.00	31.82	133.00	2.00	0.00	0.46	0.00
31.99	118.99	2.00	0.00	0.46	0.00	32.15	104.02	2.00	0.00	0.46	0.00
32.32	97.58	2.00	0.00	0.45	0.00	32.48	107.92	2.00	0.00	0.45	0.00
32.64	123.87	2.00	0.00	0.45	0.00	32.81	141.12	2.00	0.00	0.44	0.00
32.97	152.12	2.00	0.00	0.44	0.00	33.14	156.86	2.00	0.00	0.44	0.00
33.30	154.50	2.00	0.00	0.44	0.00	33.46	151.51	2.00	0.00	0.43	0.00
33.63	146.75	2.00	0.00	0.43	0.00	33.79	142.14	2.00	0.00	0.43	0.00
33.96	134.06	2.00	0.00	0.42	0.00	34.12	128.61	2.00	0.00	0.42	0.00
34.28	127.19	2.00	0.00	0.42	0.00	34.45	128.49	2.00	0.00	0.42	0.00
34.61	131.54	2.00	0.00	0.41	0.00	34.78	133.09	2.00	0.00	0.41	0.00
34.94	139.32	2.00	0.00	0.41	0.00	35.10	144.35	2.00	0.00	0.41	0.00
35.27	145.03	2.00	0.00	0.40	0.00	35.43	143.89	2.00	0.00	0.40	0.00
35.60	131.80	2.00	0.00	0.40	0.00	35.76	127.61	2.00	0.00	0.39	0.00
35.93	127.20	2.00	0.00	0.39	0.00	36.09	134.74	2.00	0.00	0.39	0.00
36.25	136.77	2.00	0.00	0.39	0.00	36.42	136.88	2.00	0.00	0.38	0.00
36.58	137.61	2.00	0.00	0.38	0.00	36.75	139.80	2.00	0.00	0.38	0.00
36.91	140.05	0.44	0.66	0.37	0.01	37.07	139.26	0.44	0.66	0.37	0.01
37.24	135.31	0.41	0.67	0.37	0.01	37.40	130.65	0.38	0.69	0.37	0.01
37.57	126.14	0.35	0.70	0.36	0.01	37.73	126.12	0.35	0.70	0.36	0.01
37.89	126.92	0.36	0.69	0.36	0.01	38.06	130.78	0.38	0.67	0.35	0.01
38.22	136.91	0.42	0.64	0.35	0.01	38.39	146.70	0.49	0.60	0.35	0.01
38.55	151.22	0.53	0.58	0.35	0.01	38.71	147.44	0.50	0.58	0.34	0.01
38.88	143.20	0.47	0.59	0.34	0.01	39.04	142.30	0.46	0.59	0.34	0.01

:: Post-earthquake settlement due to soil liquefaction :: (continued)

Depth (ft)	$Q_{tn,cs}$	FS	e_v (%)	DF	Settlement (in)	Depth (ft)	$Q_{tn,cs}$	FS	e_v (%)	DF	Settlement (in)
39.21	144.97	0.48	0.58	0.34	0.01	39.37	140.52	2.00	0.00	0.33	0.00
39.53	129.92	2.00	0.00	0.33	0.00	39.70	119.56	2.00	0.00	0.33	0.00
39.86	120.81	2.00	0.00	0.32	0.00	40.03	125.55	2.00	0.00	0.32	0.00
40.19	125.98	2.00	0.00	0.32	0.00	40.35	113.95	0.29	0.66	0.32	0.01
40.52	103.58	0.24	0.71	0.31	0.01	40.68	102.57	0.24	0.71	0.31	0.01
40.85	109.81	0.27	0.67	0.31	0.01	41.01	116.52	0.30	0.63	0.30	0.01
41.17	112.33	0.28	0.64	0.30	0.01	41.34	108.67	0.27	0.65	0.30	0.01
41.50	102.82	2.00	0.00	0.30	0.00	41.67	101.17	2.00	0.00	0.29	0.00
41.83	102.97	2.00	0.00	0.29	0.00	41.99	109.14	2.00	0.00	0.29	0.00
42.16	112.52	2.00	0.00	0.29	0.00	42.32	108.96	2.00	0.00	0.28	0.00
42.49	107.05	2.00	0.00	0.28	0.00	42.65	110.76	2.00	0.00	0.28	0.00
42.81	118.18	2.00	0.00	0.27	0.00	42.98	125.45	2.00	0.00	0.27	0.00
43.14	127.18	2.00	0.00	0.27	0.00	43.31	127.39	2.00	0.00	0.27	0.00
43.47	130.56	2.00	0.00	0.26	0.00	43.64	134.88	0.42	0.48	0.26	0.01
43.80	143.14	0.48	0.45	0.26	0.01	43.96	152.25	0.55	0.42	0.25	0.01
44.13	161.13	0.63	0.38	0.25	0.01	44.29	174.68	0.78	0.22	0.25	0.00
44.46	185.53	0.91	0.15	0.25	0.00	44.62	189.68	0.97	0.12	0.24	0.00
44.78	185.59	0.92	0.15	0.24	0.00	44.95	176.62	0.81	0.21	0.24	0.00
45.11	165.65	0.68	0.28	0.24	0.01	45.28	157.62	2.00	0.00	0.23	0.00
45.44	153.03	2.00	0.00	0.23	0.00	45.60	148.80	2.00	0.00	0.23	0.00
45.77	141.91	2.00	0.00	0.22	0.00	45.93	134.04	2.00	0.00	0.22	0.00
46.10	125.76	2.00	0.00	0.22	0.00	46.26	118.46	2.00	0.00	0.22	0.00
46.42	115.51	2.00	0.00	0.21	0.00	46.59	115.49	2.00	0.00	0.21	0.00
46.75	107.60	2.00	0.00	0.21	0.00	46.92	96.33	2.00	0.00	0.20	0.00
47.08	88.39	2.00	0.00	0.20	0.00	47.24	85.25	2.00	0.00	0.20	0.00
47.41	84.41	2.00	0.00	0.20	0.00	47.57	84.01	2.00	0.00	0.19	0.00
47.74	83.00	2.00	0.00	0.19	0.00	47.90	81.83	2.00	0.00	0.19	0.00
48.06	80.03	2.00	0.00	0.19	0.00	48.23	80.08	2.00	0.00	0.18	0.00
48.39	85.21	2.00	0.00	0.18	0.00	48.56	97.20	2.00	0.00	0.18	0.00
48.72	112.51	2.00	0.00	0.17	0.00	48.88	127.57	2.00	0.00	0.17	0.00
49.05	139.04	2.00	0.00	0.17	0.00	49.21	148.80	2.00	0.00	0.17	0.00
49.38	157.39	2.00	0.00	0.16	0.00	49.54	165.52	2.00	0.00	0.16	0.00
49.70	170.66	2.00	0.00	0.16	0.00						

Total estimated settlement: 1.47

Abbreviations

$Q_{tn,cs}$: Equivalent clean sand normalized cone resistance
FS: Factor of safety against liquefaction
e_v (%): Post-liquefaction volumentric strain
DF: e_v depth weighting factor
Settlement: Calculated settlement

APPENDIX E

GENERAL GRADING GUIDELINES

Updated Geotechnical Evaluation

Orange Avenue Seniors, Cypress, California

Project No. 2573-CR

GEOTEK

GENERAL GRADING GUIDELINES

Guidelines presented herein are intended to address general construction procedures for earthwork construction. Specific situations and conditions often arise which cannot reasonably be discussed in general guidelines, when anticipated these are discussed in the text of the report. Often unanticipated conditions are encountered which may necessitate modification or changes to these guidelines. It is our hope that these will assist the contractor to more efficiently complete the project by providing a reasonable understanding of the procedures that would be expected during earthwork and the testing and observation used to evaluate those procedures.

General

Grading should be performed to at least the minimum requirements of governing agencies, Chapters 18 and 33 of the California Building Code, CBC (2019) and the guidelines presented below.

Preconstruction Meeting

A preconstruction meeting should be held prior to site earthwork. Any questions the contractor has regarding our recommendations, general site conditions, apparent discrepancies between reported and actual conditions and/or differences in procedures the contractor intends to use should be brought up at that meeting. The contractor (including the main onsite representative) should review our report and these guidelines in advance of the meeting. Any comments the contractor may have regarding these guidelines should be brought up at that meeting.

Grading Observation and Testing

1. Observation of the fill placement should be provided by our representative during grading. Verbal communication during the course of each day will be used to inform the contractor of test results. The contractor should receive a copy of the "Daily Field Report" indicating results of field density tests that day. If our representative does not provide the contractor with these reports, our office should be notified.
2. Testing and observation procedures are, by their nature, specific to the work or area observed and location of the tests taken, variability may occur in other locations. The contractor is responsible for the uniformity of the grading operations; our observations and test results are intended to evaluate the contractor's overall level of efforts during grading. The contractor's personnel are the only individuals participating in all aspect of site work. Compaction testing and observation should not be considered as relieving the contractor's responsibility to properly compact the fill.
3. Cleanouts, processed ground to receive fill, key excavations, and subdrains should be observed by our representative prior to placing any fill. It will be the contractor's responsibility to notify our representative or office when such areas are ready for observation.
4. Density tests may be made on the surface material to receive fill, as considered warranted by this firm.
5. In general, density tests would be made at maximum intervals of two feet of fill height or every 1,000 cubic yards of fill placed. Criteria will vary depending on soil conditions and size of the fill. More frequent testing may be performed. In any case, an adequate number of field density tests should be made to evaluate the required compaction and moisture content is generally being obtained.

6. Laboratory testing to support field test procedures will be performed, as considered warranted, based on conditions encountered (e.g. change of material sources, types, etc.) Every effort will be made to process samples in the laboratory as quickly as possible and in progress construction projects are our first priority. However, laboratory workloads may cause in delays and some soils may require a **minimum of 48 to 72 hours to complete test procedures**. Whenever possible, our representative(s) should be informed in advance of operational changes that might result in different source areas for materials.
7. Procedures for testing of fill slopes are as follows:
   a) Density tests should be taken periodically during grading on the flat surface of the fill, three to five feet horizontally from the face of the slope.
   b) If a method other than over building and cutting back to the compacted core is to be employed, slope compaction testing during construction should include testing the outer six inches to three feet in the slope face to determine if the required compaction is being achieved.
8. Finish grade testing of slopes and pad surfaces should be performed after construction is complete.

Site Clearing

1. All vegetation, and other deleterious materials, should be removed from the site. If material is not immediately removed from the site it should be stockpiled in a designated area(s) well outside of all current work areas and delineated with flagging or other means. Site clearing should be performed in advance of any grading in a specific area.
2. Efforts should be made by the contractor to remove all organic or other deleterious material from the fill, as even the most diligent efforts may result in the incorporation of some materials. This is especially important when grading is occurring near the natural grade. All equipment operators should be aware of these efforts. Laborers may be required as root pickers.
3. Nonorganic debris or concrete may be placed in deeper fill areas provided the procedures used are observed and found acceptable by our representative.

Treatment of Existing Ground

1. Following site clearing, all surficial deposits of alluvium and colluvium as well as weathered or creep effected bedrock, should be removed unless otherwise specifically indicated in the text of this report.
2. In some cases, removal may be recommended to a specified depth (e.g. flat sites where partial alluvial removals may be sufficient). The contractor should not exceed these depths unless directed otherwise by our representative.
3. Groundwater existing in alluvial areas may make excavation difficult. Deeper removals than indicated in the text of the report may be necessary due to saturation during winter months.
4. Subsequent to removals, the natural ground should be processed to a depth of six inches, moistened to near optimum moisture conditions and compacted to fill standards.
5. Exploratory back hoe or dozer trenches still remaining after site removal should be excavated and filled with compacted fill if they can be located.

Fill Placement

1. Unless otherwise indicated, all site soil and bedrock may be reused for compacted fill; however, some special processing or handling may be required (see text of report).

2. Material used in the compacting process should be evenly spread, moisture conditioned, processed, and compacted in thin lifts six (6) to eight (8) inches in compacted thickness to obtain a uniformly dense layer. The fill should be placed and compacted on a nearly horizontal plane, unless otherwise found acceptable by our representative.
3. If the moisture content or relative density varies from that recommended by this firm, the contractor should rework the fill until it is in accordance with the following:
   a) Moisture content of the fill should be at or above optimum moisture. Moisture should be evenly distributed without wet and dry pockets. Pre-watering of cut or removal areas should be considered in addition to watering during fill placement, particularly in clay or dry surficial soils. The ability of the contractor to obtain the proper moisture content will control production rates.
   b) Each six-inch layer should be compacted to at least 90 percent of the maximum dry density in compliance with the testing method specified by the controlling governmental agency. In most cases, the testing method is ASTM Test Designation D 1557.
4. Rock fragments less than eight inches in diameter may be utilized in the fill, provided:
   a) They are not placed in concentrated pockets;
   b) There is a sufficient percentage of fine-grained material to surround the rocks;
   c) The distribution of the rocks is observed by, and acceptable to, our representative.
5. Rocks exceeding eight (8) inches in diameter should be taken off site, broken into smaller fragments, or placed in accordance with recommendations of this firm in areas designated suitable for rock disposal. On projects where significant large quantities of oversized materials are anticipated, alternate guidelines for placement may be included. If significant oversize materials are encountered during construction, these guidelines should be requested.
6. In clay soil, dry or large chunks or blocks are common. If in excess of eight (8) inches minimum dimension, then they are considered as oversized. Sheepsfoot compactors or other suitable methods should be used to break up blocks. When dry, they should be moisture conditioned to provide a uniform condition with the surrounding fill.

Slope Construction

1. The contractor should obtain a minimum relative compaction of 90 percent out to the finished slope face of fill slopes. This may be achieved by either overbuilding the slope and cutting back to the compacted core, or by direct compaction of the slope face with suitable equipment.
2. Slopes trimmed to the compacted core should be overbuilt by at least three (3) feet with compaction efforts out to the edge of the false slope. Failure to properly compact the outer edge results in trimming not exposing the compacted core and additional compaction after trimming may be necessary.
3. If fill slopes are built "at grade" using direct compaction methods, then the slope construction should be performed so that a constant gradient is maintained throughout construction. Soil should not be "spilled" over the slope face nor should slopes be "pushed out" to obtain grades. Compaction equipment should compact each lift along the immediate top of slope. Slopes should be back rolled or otherwise compacted at approximately every 4 feet vertically as the slope is built.
4. Corners and bends in slopes should have special attention during construction as these are the most difficult areas to obtain proper compaction.
5. Cut slopes should be cut to the finished surface. Excessive undercutting and smoothing of the face with fill may necessitate stabilization.

UTILITY TRENCH CONSTRUCTION AND BACKFILL

Utility trench excavation and backfill is the contractors responsibility. The geotechnical consultant typically provides periodic observation and testing of these operations. While efforts are made to make sufficient observations and tests to verify that the contractors' methods and procedures are adequate to achieve proper compaction, it is typically impractical to observe all backfill procedures. As such, it is critical that the contractor use consistent backfill procedures.

Compaction methods vary for trench compaction and experience indicates many methods can be successful. However, procedures that "worked" on previous projects may or may not prove effective on a given site. The contractor(s) should outline the procedures proposed, so that we may discuss them **prior** to construction. We will offer comments based on our knowledge of site conditions and experience.

1. Utility trench backfill in slopes, structural areas, in streets and beneath flat work or hardscape should be brought to at least optimum moisture and compacted to at least 90 percent of the laboratory standard. Soil should be moisture conditioned prior to placing in the trench.
2. Flooding and jetting are not typically recommended or acceptable for native soils. Flooding or jetting may be used with select sand having a Sand Equivalent (SE) of 30 or higher. This is typically limited to the following uses:
   a) shallow (12 + inches) under slab interior trenches and,
   b) as bedding in pipe zone.

   The water should be allowed to dissipate prior to pouring slabs or completing trench compaction.
3. Care should be taken not to place soils at high moisture content within the upper three feet of the trench backfill in street areas, as overly wet soils may impact subgrade preparation. Moisture may be reduced to 2% below optimum moisture in areas to be paved within the upper three feet below sub grade.
4. Sand backfill should not be allowed in exterior trenches adjacent to and within an area extending below a 1:1 projection from the outside bottom edge of a footing, unless it is similar to the surrounding soil.
5. Trench compaction testing is generally at the discretion of the geotechnical consultant. Testing frequency will be based on trench depth and the contractors procedures. A probing rod would be used to assess the consistency of compaction between tested areas and untested areas. If zones are found that are considered less compact than other areas, this would be brought to the contractors attention.

JOB SAFETY

General

Personnel safety is a primary concern on all job sites. The following summaries are safety considerations for use by all our employees on multi-employer construction sites. On ground personnel are at highest risk of injury and possible fatality on grading construction projects. The company recognizes that construction activities will vary on each site and that job site safety is the contractor's responsibility. However, it is, imperative that all personnel be safety conscious to avoid accidents and potential injury.

In an effort to minimize risks associated with geotechnical testing and observation, the following precautions are to be implemented for the safety of our field personnel on grading and construction projects.

1. Safety Meetings: Our field personnel are directed to attend the contractor's regularly scheduled safety meetings.
2. Safety Vests: Safety vests are provided for and are to be worn by our personnel while on the job site.
3. Safety Flags: Safety flags are provided to our field technicians; one is to be affixed to the vehicle when on site, the other is to be placed atop the spoil pile on all test pits.

In the event that the contractor's representative observes any of our personnel not following the above, we request that it be brought to the attention of our office.

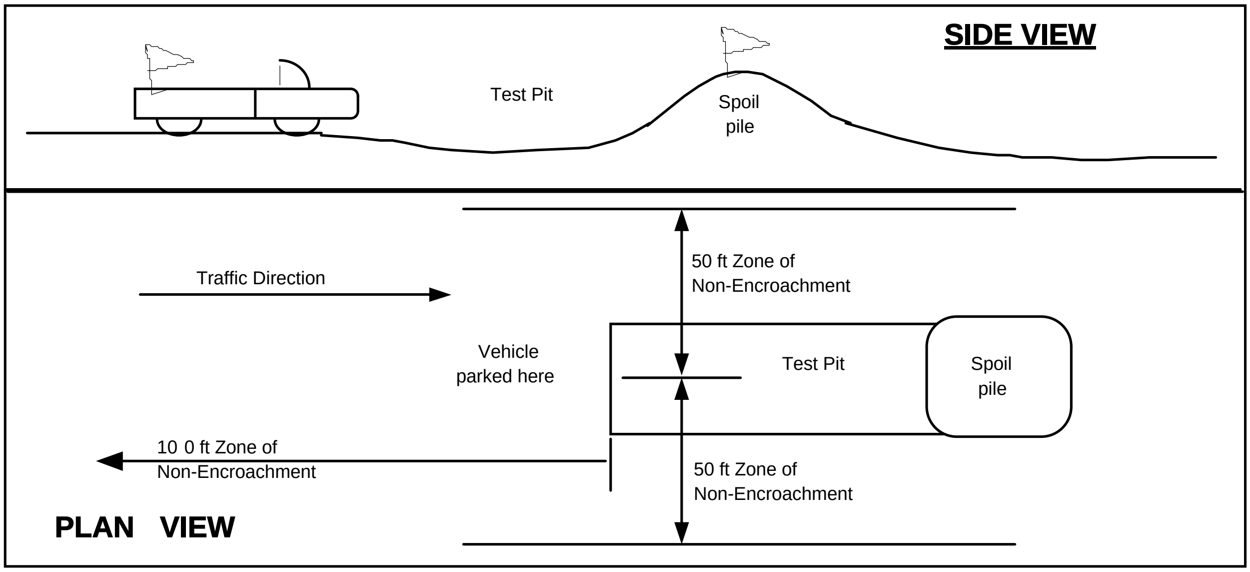
Test Pits Location, Orientation and Clearance

The technician is responsible for selecting test pit locations. The primary concern is the technician's safety. However, it is necessary to take sufficient tests at various locations to obtain a representative sampling of the fill. As such, efforts will be made to coordinate locations with the grading contractors authorized representatives (e.g. dump man, operator, supervisor, grade checker, etc.), and to select locations following or behind the established traffic pattern, preferably outside of current traffic. The contractors authorized representative should direct excavation of the pit and safety during the test period. Again, safety is the paramount concern.

Test pits should be excavated so that the spoil pile is placed away from oncoming traffic. The technician's vehicle is to be placed next to the test pit, opposite the spoil pile. This necessitates that the fill be maintained in a drivable condition. Alternatively, the contractor may opt to park a piece of equipment in front of test pits, particularly in small fill areas or those with limited access.

A zone of non-encroachment should be established for all test pits (see diagram below). No grading equipment should enter this zone during the test procedure. The zone should extend outward to the sides approximately 50 feet from the center of the test pit and 100 feet in the direction of traffic flow. This zone is established both for safety and to avoid excessive ground vibration, which typically decreases test results.

TEST PIT SAFETY PLAN



Slope Tests

When taking slope tests, the technician should park their vehicle directly above or below the test location on the slope. The contractor's representative should effectively keep all equipment at a safe operation distance (e.g. 50 feet) away from the slope during testing.

The technician is directed to withdraw from the active portion of the fill as soon as possible following testing. The technician's vehicle should be parked at the perimeter of the fill in a highly visible location.

Trench Safety

It is the contractor's responsibility to provide safe access into trenches where compaction testing is needed. Trenches for all utilities should be excavated in accordance with CAL-OSHA and any other applicable safety standards. Safe conditions will be required to enable compaction testing of the trench backfill.

All utility trench excavations in excess of 5 feet deep, which a person enters, are to be shored or laid back. Trench access should be provided in accordance with OSHA standards. Our personnel are directed not to enter any trench by being lowered or "riding down" on the equipment.

Our personnel are directed not to enter any excavation which;
1. is 5 feet or deeper unless shored or laid back,
2. exit points or ladders are not provided,
3. displays any evidence of instability, has any loose rock or other debris which could fall into the trench, or
4. displays any other evidence of any unsafe conditions regardless of depth.

If the contractor fails to provide safe access to trenches for compaction testing, our company policy requires that the soil technician withdraws and notifies their supervisor. The contractors representative will then be contacted in an effort to effect a solution. All backfill not tested due to safety concerns or other reasons is subject to reprocessing and/or removal.

Procedures

In the event that the technician's safety is jeopardized or compromised as a result of the contractor's failure to comply with any of the above, the technician is directed to inform both the developer's and contractor's representatives. If the condition is not rectified, the technician is required, by company policy, to immediately withdraw and notify their supervisor. The contractor's representative will then be contacted in an effort to effect a solution. No further testing will be performed until the situation is rectified. Any fill placed in the interim can be considered unacceptable and subject to reprocessing, recompaction or removal.

In the event that the soil technician does not comply with the above or other established safety guidelines, we request that the contractor bring this to technicians attention and notify our project manager or office. Effective communication and coordination between the contractors' representative and the field technician(s) is strongly encouraged in order to implement the above safety program and safety in general.

The safety procedures outlined above should be discussed at the contractor's safety meetings. This will serve to inform and remind equipment operators of these safety procedures particularly the zone of non-encroachment.

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