ATTACHMENT E-1

GEOTECHNICAL DESIGN REPORT

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NMG Geotechnical, Inc.

April 18, 2024

Project No. 23111-01

To: C.J. Segerstrom & Sons
3315 Fairview Road
Costa Mesa, California 92626

Attention: Jeffrey M. Reese

Subject: Geotechnical Design Report for Lake Center Office Park Redevelopment, South Coast Technology Center, 3100 Lake Center Drive, City of Santa Ana, California

In accordance with your request and authorization, NMG Geotechnical, Inc. (NMG) has performed a geotechnical design study that included a site-specific subsurface investigation for the proposed redevelopment for the subject Lake Center office park. The proposed redevelopment consists of the demolition of a 10.2-acre portion of the office park that has three existing office buildings and a parking structure plus an adjacent 5.6-acre vacant field. The redevelopment will consist of three new, larger manufacturing/warehouse buildings that will be the South Coast Technology Center. At this time, we have reviewed preliminary planning design information that includes the third architectural submittal by DRA Architects, typical foundation and structural loading provided by HSA & Associates, Inc. (Consulting Structural Engineers), and the site layout and survey developed by the project civil engineer (Incledon). We also reviewed the foundation plans for the existing buildings and parking structure.

The geotechnical investigation performed included a review of background information, field reconnaissance, drilling of six hollow-stem-auger borings, seven cone-penetrometer test (CPT) soundings, five backhoe trenches, laboratory testing and geotechnical analysis of the collected data. Our study focused on the grading and foundation design considerations, and includes an assessment of groundwater, seismicity, liquefaction, and settlement potential. The prior NMG (2023) report provided preliminary geotechnical and groundwater information for the site along with our assessment for infiltration BMPs.

The project site is underlain by deep Quaternary-aged alluvial deposits, prior compacted fill up to 7.5 feet thick, and some undocumented/stockpile fill that is up to 6.5 feet thick in the vacant field. The current groundwater level is approximately 10 feet deep (elevation 26 feet msl) below existing grades. Between 15 and 35 feet, the alluvium consists mainly of wet, compressible, fine-grained silty and clayey soils and the deeper alluvium consists of alternating coarse- and fine-grained soils with varying thicknesses. There are no mapped faults underlying the property and the closest seismically active fault is the San Joaquin Hills Fault located approximately 4.2 km (2.6 miles) away. The site is mapped in a potential liquefaction seismic hazard zone.

The main geotechnical issues impacting the project site include:

- Removal of prior and undocumented fill and unsuitable surficial soils to provide a uniform cap of certified engineered fill for the building pads. The demolition of existing structures and utilities will require deeper excavations and could result in additional loose, disturbed soil that will need to be backfilled and properly recompacted.
- The underlying silt and clayey alluvial soil is soft, saturated, and compressible that will be subject to static settlement due to the site grading and new building loads. The potential and magnitude of the future settlement is anticipated to be within the range that can be mitigated with foundation design measures.
- The site has wet soil and shallow groundwater conditions. Wet soils and groundwater should be anticipated in excavations deeper than 7 feet and could possibly impact site grading and construction of deeper utilities.
- Potential for strong seismic shaking during an earthquake on a regionally active fault and the potential for seismic settlement.
- The design of the building foundations and site improvements should also consider the presence of moderate soil expansion potential and corrosive soils.

This report presents our geotechnical findings, conclusions and recommendations for grading, design, and construction. This report is based on the 2022 California Building Code (CBC) requirements. We have included a Geotechnical Map (Plate 1) which depicts the collected geologic information. The data from our subsurface exploration (including borings, trench, and CPT logs) and laboratory test results are all included in Appendices B and C, respectively. Appendix D includes the code-based seismic analysis. Appendix E provides the liquefaction analysis. Appendix F includes NMG's general earthwork and grading specifications.

The proposed grading and future development of the site is considered geotechnically acceptable, provided our recommendations are implemented. The recommendations in this report may be revised and/or additional recommendations may be provided once design information and geotechnical review of the grading and foundation plan is performed. Observation and testing during grading and additional testing at the completion of grading should also be performed to confirm the foundation/subgrade soils conditions.

If you have any questions regarding this report, please contact us. We appreciate the opportunity to provide our services.

Respectfully submitted,

NMG GEOTECHNICAL, INC.

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1.0 INTRODUCTION

1.1 Scope of Work

The purpose of our geotechnical study was to evaluate the existing subsurface conditions in light of the proposed redevelopment at the subject site. Our investigation and this report are based upon our review of the preliminary project planning information prepared by DRA Architects and provided by the project team.

Our scope of work included the following:

- Background review of available published and unpublished reports and maps (Appendix A).
- Review of available historic aerial photographs pertinent to the surrounding area (referenced in Appendix A).
- Drilling, logging, sampling, and backfilling of six hollow-stem-auger borings (H-1 to H-6) up to approximately 51.5 feet deep. Approximate boring locations are shown on Plate 1 and the boring logs are included in Appendix B.
- Excavating, logging, sampling, and backfilling of five backhoe trenches (T-1 to T-5) up to 16 feet deep. Approximate trench locations are shown on Plate 1 and the trench logs are included in Appendix B.
- Advancement of seven Cone Penetration Test soundings (CPT-1 to CPT-6). Approximate CPT locations are shown on Plate 1 and the CPT logs are included in Appendix B.
- Laboratory testing of relatively undisturbed ring and bulk soil samples. Corrosivity testing was also performed. Test results are summarized in Appendix C.
- Geotechnical evaluation and analysis of the compiled data with respect to the proposed site grading and redevelopment.
- Preparation of this report including our findings, conclusions, recommendations, and accompanying illustrations.

1.2 Site Location and Existing Conditions

The project site is located at the Lake Center Office Park, south of the W. Lake Center Drive and Susan Street intersection, in the City of Santa Ana, California. The overall site for the new South Coast Technology Center is approximately 15.8 acres and includes an approximate 5.6-acre vacant field southwest of the intersection and the existing 10.2-acre office park (Figure 1). The existing office buildings (three-story) and surrounding parking structures are located at 3100, 3110 and 3120 W. Lake Center Drive and southeast of the intersection. There is a lake feature with fountains situated between the existing buildings and Lake Center Drive. The lake is understood to consist of a cement-treated bottom of some kind. The site is relatively flat with elevations ranging from 34 to 40 feet above mean sea level (msl). The upper level of the existing parking structures site is at elevations of approximately 43 feet msl. The lower level of the parking structure is partially subterranean, and the perimeter walls retain approximately 3.5 feet of soil. Footings for the buildings and parking structure are approximately 2 to 4 feet deep based on the foundation plans reviewed. Also, the existing floor slabs are shown to be 4 inches thick. Existing utilities at the site consist of storm drain, sewer, water, electrical, gas, and telecommunications. The sewer (8- to 12-inch VCP) is indicated to be the deepest utility pipeline and up to ±10 feet deep. The majority of these utilities stem from Lake Center Drive and follow the drive aisles before entering the buildings. The drives and parking lots consist of asphalt cement pavements. There are limited Portland cement improvements for the walks and curbs and gutters, etc. The existing landscape includes turf areas, planters, and trees. The Greenville Banning Urban Runoff Diversion Channel flows from north to south, approximately 180 feet east of the property.

At the time of our investigation, the vacant lot to the west of Susan Street was very wet and had soft soils and ponded water on the surface due to recent rain events. Minor weeds were present on the edges of the active stockpile area. The perimeter of this site is fenced with an entry gate at the northwest corner. The west end of this site abuts an easement for the Southern Pacific Company Railroad.

1.3 Site History

The earliest aerial photographs reviewed were taken in 1953. At that time, the subject site and surrounding areas were being utilized for row-crop farming. No structures or roads existed in these photographs, except for farming access roads. Between 1953 and 1963, the Banning-Greenville Channel was constructed. The channel may have been constructed to aid in the drainage of the adjacent farmland and drains into the Santa Ana River to the southwest.

By 1963, the Southern Pacific Company Railroad was constructed along the western edge of the site. MacArthur Boulevard was constructed by 1972. All but one of the existing buildings and surrounding parking structures were built by 1987. By 1992, a final building and parking structure had been built. The site has remained relatively unchanged since 1992 (NETR, 2024).

According to our conversations with the site facility manager, a portion of the stockpile (end-dump piles) was removed from the vacant lot site prior to our field investigation.

1.4 Proposed Development and Preliminary Design Information

Three of the existing buildings and one parking structure will be demolished for the redevelopment of the site. The new development will consist of construction of three new Class A manufacturing/warehouse buildings. The proposed site will be regraded to create new building pads with associated backbone infrastructure, and paved drives/parking areas. The buildings will each have loading dock areas consisting of recessed truck parking bays. The site will have at-grade vehicle parking. Utility and landscape improvements are also proposed for the redevelopment.

We reviewed the current architectural plan set (3rd submittal dated April 8, 2024) prepared by DRA Architects. The plan set consists of architectural, civil and landscape sheets. The new proposed buildings will be single story buildings with a mezzanine. The buildings will be concrete tilt-ups that are on the order of 80,000 to 125,000 square feet in size. The maximum height of the new buildings will be 48.5 feet and the perimeter wall heights are approximately 39.6 feet. The finish floor elevations for the new buildings are indicated to be between 37.25 to 39.00 feet. We understand that the concrete floor slab thickness will be on the order of 7 to 8 inches.

The preliminary structural foundation loading information provided for the buildings is summarized below:

Columns	***Perimeter Walls and Footings***	***Floor Slab***
Dead Load: 45-50 kips	Wall thickness: 8.5-9.25 inches	Dead Load: 90 – 100 psf
Live Load: 37-46 kips	Dead+Live Loads: 5.5-7.5 klf	Live Load: 500 psf
Spacing: 52-60 feet on center	Footing Width: 2.5 – 3.5 feet	

The existing curb and gutter and sidewalk along the site perimeter will remain and need to be protected in-place. The pavement along Lake Center Drive to the intersection of Harbor Boulevard will be replaced/rehabilitated. Additional site improvements indicated currently include:

- Bioretention areas with 6-inch HDPE underdrain, 30-mil-thick geomembrane, soil media;
- Concrete tilt-up screen walls (up to 8 feet in height, portions along property line);
- Wrought iron fences; and vehicular gates;
- Fire and trash truck circulation road and trash enclosures;
- New asphalt pavements (drive and parking);
- Vehicular and pedestrian concrete pavements;
- Concrete driveways, truck ramps, and loading docks;
- Concrete curb ramps and ADA ramps;
- Concrete curbs and gutters and concrete ribbon gutters;
- New wet utility lines (sewer, storm drain, domestic water, fire water, reclaimed water, irrigation);
- New dry utility lines (electrical, gas, communications);
- Relocation and installation of existing and new light poles (Susan Street);
- Relocation and installation of existing fire hydrant, telephone vault (Susan Street);
- Landscape areas with trees and shrubs/ground cover;
- Pavers;
- Bollards;
- Decomposed granite (DG) paving; and
- Tables, seating, planters, and other landscape improvements.

1.5 Field Investigation

The subsurface exploration was conducted in late February and early March 2024. The CPT, boring, and backhoe trench locations were staked and cleared with DigAlert as required. Exploration consisted of seven CPT soundings, five backhoe trenches, and six hollow-stem-auger borings. The CPTs were advanced 50 to 100 feet in depth, the backhoe trenches were 15 to 16 feet deep, and the hollow-stem-auger borings were 26.5 to 51.5 feet deep. The borings and trenches were geotechnically logged and sampled. The CPT, trench, and boring logs are included in Appendix B, and the approximate locations are depicted on Plate 1.

The backhoe trenches (T-1 through T-5) were excavated on March 12, 2024, primarily for visual observation and evaluation of the near-surface soils.

Seven cone penetration tests (CPT-1 through CPT-7) were performed on February 26, 2024, by Kehoe Testing and Engineering, Inc. The CPTs use an integrated electronic cone system that measures and records tip resistance, sleeve friction, and friction ratio parameters at 5-cm depth intervals. These explorations were located across the site and encountered alluvial materials with soil behavior types consisting of heterogeneous layers of clays, silts, and silty sands to sands. The CPT sounding test data provided detailed subsurface profile information that was interpreted as part of our assessment of the potential liquefaction hazards. The CPT data was used with the adjacent boring information and laboratory test data to develop a consistent interpretation of the subsurface conditions. The CPT was also used to determine the shear wave velocity in the upper 100 feet of the existing soil (CPT-2).

The hollow-stem-auger borings (H-1 through H-6) were drilled on February 29 and March 1, 2024. Relatively undisturbed soil ring samples were collected using a 2.5-inch-inside-diameter modified California split-spoon sampler. Disturbed soils were collected using a standard penetration test split-spoon sampler. The samplers were driven with a 140-pound hammer, free-falling 30 inches. Representative bulk samples of onsite soil were collected from the hollow-stem cuttings and used for additional soil identification purposes and laboratory testing. The sampling was used to assess soil types beneath the site as well as to obtain a measure of resistance of the soil to penetration (recorded as blows-per-foot on the geotechnical boring logs). The borings were backfilled per County requirements with bentonite-cement grout using a tremie pipe. Borings within pavement were patched with concrete and dye to match existing.

1.6 Laboratory Testing

The type of laboratory tests performed as part of the investigation are indicated below. The laboratory tests were conducted on selected soil samples in general conformance with applicable ASTM test standards. The laboratory test results are presented in Appendix C. In-situ moisture and dry density results are included on the geotechnical boring logs (Appendix B).

- In-situ moisture content and dry density;
- Maximum density and optimum moisture content;
- Grain-size distribution (sieve and/or hydrometer);
- Atterberg Limits;
- Consolidation settlement and collapse;
- Direct shear (undisturbed and remolded);
- Expansion index;
- Maximum density;
- Corrosivity, and
- R-value.

The R-value testing was performed by LaBelle Marvin Professional Pavement Engineering and is discussed in Section 2.10. The corrosivity testing was performed by Project X Corrosion Engineering and is discussed in Section 2.11.

2.0 GEOTECHNICAL FINDINGS

2.1 Geologic Conditions and Earth Units

The alluvium consists of a heterogeneous mixture of silts, clays, and sands. The upper 15± feet of material at the site tends to be primarily composed of interlayered silts and silty sands. Between depths of 15 and 35 feet, there is a consistent layer of moderate to highly plastic clay. The soils in the upper 20 feet had minor organics and rootlets. At depths below 35 feet, there are layers of silty sands interbedded with silts. The alluvium is slightly porous, and generally becomes less porosity and higher density at depth. In general, the finer grained soils have relatively low in-situ dry densities and high water content.

The surface soils are mapped as Bolsa silt and clay loam (USDA, 1978). The site is classified as Hydrologic Group C, and the natural soil profile down from 0 to 29 inches is silt, followed by clay to 69 inches.

2.2 Geologic Structure

The site is located on the southern end of the Downey Plain and is underlain by approximately 1000± feet of Quaternary-age alluvial deposits (CDMG, 1980). The alluvium is composed of massive to crudely layered sediments that are generally flat lying, with a gentle dip toward the southwest.

2.3 Regional Faulting and Seismicity

Faulting: The site is not located within a fault-rupture hazard zone as defined by the Alquist-Priolo Special Studies Zones Act (California Geological Survey, 2018 and Hart and Bryant, 2007), and no evidence of active faulting was observed during this investigation. Also, based on mapping by the State (California Geological Survey, 2010), there are no active faults mapped at the site at depth. Using the USGS computer program (USGS, 2024) and the site coordinates of 33.6985 degrees north latitude and 117.9127 degrees west longitude, the controlling fault at the site is the San Joaquin Hills Blind Thrust Fault located 4.2 kilometers (2.6 miles) from the site. The maximum moment magnitude for the Controlling Fault is 7.15 M_W. The other faults noted that can produce strong ground shaking at the site include the Newport-Inglewood (Offshore), Whittier and Elsinore (Glen Ivy) Faults. Based on review of published maps, historic aerial photographs and topographic maps, the potential for primary ground rupture due to an earthquake is considered very low.

Seismicity: Properties in southern California are subject to seismic hazards of varying degrees depending upon the proximity, degree of activity, and capability of nearby faults. These hazards can be primary (i.e., directly related to the energy release of an earthquake, such as surface rupture and ground shaking) or secondary (i.e., related to the effect of earthquake energy on the physical world, which can cause phenomena such as liquefaction and ground lurching). The site is located in a seismic hazard zone for liquefaction potential (CDMG, 1997), as shown on Figure 2. Liquefaction potential is discussed in Section 2.6. Secondary seismic hazards, such as tsunami and seiche, need not be considered since the site is located over 5 miles from the ocean or any confined bodies of water and at elevations well above mean sea level.

As with the majority of sites in Southern California, the primary seismic hazard for this site is ground shaking due to a future earthquake on one of the major regional active faults, such as the San Joaquin Hills Blind Thrust, Newport-Inglewood, Whittier, or the Elsinore-Glen Ivy Faults. The site is designated as Class D for the seismicity analysis based on the Vs(30) shear wave velocity per ASCE 7-16 Table 20.3-1and collected field and laboratory test results from this site investigation. The seismic design parameters are presented in the Conclusions and Recommendations section of this report. Seismic design parameters were calculated based on a computer program by the Structural Engineers Association/Office of Statewide Health Planning and Development (2024). The results are tabulated in Section 3.5 and the data is included in Appendix D.

2.4 Groundwater

Groundwater was encountered in five of the six hollow-stem borings during our subsurface exploration. The present groundwater table is approximately 10 feet deep (elevation 26 feet msl). Based on historic well data and mapping by CDMG, the shallowest depth of free water, or high-water saturation below the ground surface, has been approximately 5 feet deep (CDMG, 1980 and 1997).

Arcadis (2019) monitored the groundwater levels at the nearby Chevron/Unocal station (3599 Harbor Boulevard) between 1992 and 2018. The historical well monitoring data from MW-1 indicates the shallowest water level to be 8 feet deep in March of 1995 and the deepest level to be 12 feet in March of 2018. The shallowest groundwater levels were recorded in the January to June months for each year. The seasonally high groundwater level is 10 feet based on this well-monitoring data.

2.5 Soil Conditions and Classification

Moisture content and dry densities were determined on the relatively undisturbed ring and disturbed SPT samples (moisture content only) collected from the borings. The existing soil moisture content varies from 2.41 to 43.8 percent and in-place dry density ranged from approximately 78.2 to 125.5 pcf. The near-surface soils (upper 5 feet) are damp to moist; however, the majority of the onsite soils are above optimum-moisture content. Maximum density tests were performed on two bulk samples of the alluvial soils collected from the upper 14 feet. The samples consisted of silty and clayey soils. The maximum density test results were a maximum dry density from 103 to 115 pcf with optimum moisture content from 12.0 to 15.5 percent.

A total of seven Atterberg Limit tests, three grain-size distribution were performed to aid in the classification of the collected soil samples. The Atterberg Limit test results indicate that the fine-grained soils encountered range from low to high plasticity silts and clays. The fines contents (passing No, 200 Sieve) ranged from 65 to 91 percent and the clay fraction (passing 2 Micron) ranged from 9 to 24 percent. The Liquid Limit of the fine-grained soils ranged from 10 to 38 percent and the Plasticity Index ranged from 10 to 38 percent. Most of the sandy material encountered have silty and clayey fines. Based on the USCS classification, the alluvium consists of crudely layered SM, SC, ML, CL, and CH soils. The soil sample descriptions, classification (USCS group symbol), in-situ soil dry density and moisture content are presented on the boring and trench logs (Appendix B).

2.6 Liquefaction Potential

Our field investigation and laboratory testing were performed in part to evaluate the subsurface soils for liquefaction potential. The California Geological Survey has developed seismic hazard maps as a part of the Seismic Hazards Mapping Act of 1991. The subject site is shown on Figure 2 as being mapped within a zone of liquefaction potential (CDMG, 1997a). Liquefaction is a phenomenon in which earthquake-induced cyclic stresses generate excess pore water pressure in low density (loose), saturated, sandy soils and soft silts below the water table. This causes a loss of shear strength and, in many cases, ground settlement. Liquefaction is generally thought to be a problem in earthquake-prone areas where conditions that promote liquefaction are present in the upper 50 feet of earth.

For liquefaction to occur, all the following four conditions must be present:

- There must be severe ground shaking, such as occurs during a strong earthquake.
- The soil material must be saturated or nearly saturated, generally below the water table.
- The corrected normalized standard penetration test (SPT) blow counts (N_1) or the CPT tip resistance (Q) must be relatively low.
- The soil material must be granular (usually sands or silts) with, at most, only low plasticity. Clayey soils and silts of relatively high plasticity are generally not subject to liquefaction.

Our liquefaction potential assessment was performed using the computer program CLiq version 2.2.0.18 developed by Geologismiki, which provides results and plots of the calculations. The liquefaction potential analysis is performed using the Robertson method (T.L. Youd, et al., 1996). The program provides the basic CPT data interpretation through to final plots of factor-of-safety, liquefaction potential index, and post-earthquake displacements, including vertical settlement. The site liquefaction potential was evaluated for a design earthquake magnitude of 7.15 (M_W) and a peak ground acceleration (PGA) of 0.62g, as determined in our site seismicity analysis discussed in Sections 2.3 and 3.5.

Based on our subsurface investigation, the site is underlain by a combination of clayey, silty, and sandy alluvium, and has shallow groundwater. A design groundwater depth of 5 feet (historic high) was utilized for this analysis (5 feet higher than the existing groundwater level).

The liquefiable layers varied from thin sandy lenses to local sandy and silty layers that were 5 to 10 feet thick. Some of the sandy layers appeared to be dense and/or had sufficient fines such that they were not deemed liquefiable.

Based on the collected CPTs, portions of the sandy and silty alluvium may liquefy during the design earthquake event and result in seismic settlement. The overall probability and risk for liquefaction at the site varies from low to moderate. The liquefaction analysis is included in Appendix E.

The calculated liquefaction settlement in the upper 50 feet ranged between 1.5 and 2.8 inches and was generally uniform for the CPTs located in the existing Lake Center office park site (CPT-2 through CPT-7). However, CPT-1 on the vacant field area (future Building 1) indicated 4.1 inches of seismic settlement and a higher probability of liquefaction. It should be noted that the seismic settlement in the upper 30 feet is 0.5 to 1 inch. Settlement of the deeper liquefaction layers would be diminished at the surface.

Potential surface manifestation damage caused by liquefaction is a function of the thickness of the non-liquefiable surface cover (consisting of the reworked onsite materials plus design fill) over the thickness of the underlying liquefiable layers. Based on the generalized subsurface profile and our overall liquefaction evaluation, the site indicates a low potential for surface manifestation and lateral spread.

2.7 Settlement Potential

Three laboratory consolidation tests were performed on the compressible clayey alluvium material at a depth of 15 feet near the new building pads. The materials above and below this zone are anticipated to be less compressible. The future compacted fill is also anticipated to be less compressible. A remolded sample of the silty soil was used to evaluate consolidation properties of compacted fill.

The settlement analysis was based on the interpreted geologic conditions, parameters developed from the consolidation test results, current site and building information, including the anticipated site remedial and future site grading. The typical foundation layout and loads for the proposed buildings were provided by the project structural engineer (HSA & Associates, Inc) as presented in Section 1.4. Our analysis indicates a maximum total static/consolidation settlement on the order of 2 to 2.5 inches. At this time, we anticipate the building areas will have minor design cuts and fills.

The settlement evaluation included the following:

- The settlement for the slabs with up to 600 psf uniformly distributed loading is a maximum of 1.8 inches toward the center of the slab. The settlement near the perimeter of the slab is 1.0 inch.
- The buildings columns (isolated point loads) will have a load on the order of 80 kips and will be spaced at approximately 60 feet on-center. Settlement at the columns (5.5-foot-square footing with a bearing pressure of 3,000 psf) is calculated to be a maximum of approximately 0.9 inch.
- The perimeter footings that support the concrete wall loads of 7.0 kpf (2.5 feet wide by 2 feet deep) have a calculated settlement of 1 inch at the center and 0.5 inch at the ends.

Based on our liquefaction analysis, we have calculated the potential settlement due to liquefaction to be on the order of 1.5 to 2.8 inches. Locally, higher liquefaction settlement up to 4 inches may occur. Differential settlement is difficult to predict; however, for structural design, we recommend a differential settlement of 1.5 inches over 60 feet.

2.8 Shear Strengths

A direct shear test was performed on a sample of alluvium that was remolded to 90 percent (based on ASTM Test Method D1557) to evaluate the strength of reworked onsite soils, to assess the strength of the future fill material derived from onsite soil. The selected sample was a silty material sampled in the upper 8 feet from Trench T-1. The test result for this sample indicated a ultimate cohesion of 100 psf and a soil friction angle of 25.5 degrees. The shear strength for the fill material derived from onsite materials can vary depending on the type of soils. Other direct shear test results for similar soil conditions from the prior projects indicated higher remolded strengths (average cohesion of 200 psf and a soil friction angle of 30 degrees).

The direct shear test was performed on an undisturbed soil sample, to evaluate the clayey alluvium strength at a depth of approximately 10 feet from existing grade. The selected sample tested was from Boring H-4 and the test results indicate an ultimate cohesion of 50 psf and a soil friction angle of 22.5 degrees.

2.9 Soil Expansion Characteristics

Two expansion index tests were performed on bulk samples of the near-surface alluvium to evaluate the expansion potential of onsite soils. Based on the laboratory test results and our visual classification of the onsite soils, the expansion potential varied from "Very Low" to "Medium." The clayey soils may have high expansion potential. Additional evaluation will be required to confirm the expansion potential once grading is completed.

2.10 R-Value

Two R-value tests were performed on onsite material that indicates values of 26 to 61. Based on the test results for nearby areas, similar clayey soils were found to have lower R-values. In general, the onsite soil is anticipated to be moderate to poor for street pavement subgrade. Additional soil sampling and testing of finish grade soil should be performed following the completion of grading to confirm the R-value of the street subgrade.

2.11 Corrosivity Testing

Corrosion testing was performed for a total of eight soil samples collected at depths ranging from 1 to 13 feet. The samples mainly consist of silty and clayey soil with sand. The corrosion evaluation was performed by Project X and included electrical resistivity, pH, soluble sulfate, and chloride. The specific soil analysis lab test results are presented in Appendix C and the table below summarizes the test results. The sandier soils should generally be less corrosive.

Soil Corrosion Test	***Test Results***
Minimum Resistivity (ohm-cm)	804 to 28,810
pH	7.9 – 8.6
Sulfate Content (ppm)	31 – 506
Chloride Content (ppm)	18 – 362

Electrical resistivities were in the mildly to moderately corrosive category with the in-situ moisture content. When saturated, the resistivities are in the moderately to severely corrosive categories for ferrous metals. The moisture content has a significant effect on the corrosivity of the site soils. Sulfate contents are negligible and indicate that onsite soils are not corrosive to concrete. The chloride contents are also negligible. Soil pH values indicate slight to medium alkalinity.

3.0 CONCLUSION AND PRELIMINARY RECOMMENDATIONS

3.1 General Conclusion

Based on the results from this study, redevelopment of the site is considered geotechnically acceptable, provided the recommendations herein are implemented during the design, grading, and future construction. The primary geotechnical impacts are the required remedial grading and the potential settlement due to the soft, saturated alluvial soils. The site grading will require removal and recompaction of the onsite soils that may have high moisture content. The foundations for the new buildings will need to be designed for anticipated seismic conditions, and expansive soil potential.

This subsurface investigation confirmed that the site has shallow groundwater, generally fine-grained soils, and liquefaction potential. The prior NMG report (NMG, 2023) provided more detailed historic groundwater information, which was confirmed during our subsurface investigation. The site is not considered suitable for onsite stormwater infiltration primarily due to the shallow groundwater. The current plan indicates alternative BMP measures for the site redevelopment will have bioretention systems.

Our recommendations are based on preliminary project development information and should be reviewed once design plans are available. The future as-graded site conditions should also be observed and confirmed prior to construction. The recommendations are also considered minimum and may be superseded by more stringent requirements of others.

3.2 Site Preparation and Earthwork

General earthwork and grading specifications are provided below and in Appendix F. Grading will also have to satisfy the requirements of the City of Santa Ana.

3.2.1 Site Preparation

Prior to grading, deleterious material (highly organic topsoil, vegetation, trash, construction debris), if any, should be cleared from the site and disposed of offsite. The existing structures to be demolished and the buried utilities within the site should be removed and the areas properly backfilled. The demolition operation should minimize disturbing/loosening existing soils and should protect existing improvements to remain.

Some of the existing utility lines may be locally deeper than the recommended remedial removals; therefore, special excavation for these lines may be necessary if encountered. It should be noted that asbestos cement pipes have been used in the past in the area.

3.2.2 Remedial Removals

The recommended remedial removals for the project are provided separately for the vacant field, office park and general items. Achieving the recommended removals is important to the settlement, foundation design assumptions and the overall site improvements.

Vacant Field (Building 1): The stockpile soil, undocumented fills, including possibly old in-filled drainage ditches or trenches, should be entirely removed and recompacted across the site. We anticipate these removals will be a minimum of 6.5 deep. In addition, we recommend that a minimum 5-foot-thick fill blanket be provided below the building pad (based on finish pad elevation 36 feet msl). Local variations in soil conditions may occur and result in the need for deeper removals. A minimum of 3-foot-deep remedial removals are recommended for the areas outside the buildings' adjacent areas (drives and parking). The lateral limits of the removals should extend a minimum of 5 feet beyond the building footprint to include any exterior architectural elements.

Office Park (Buildings 2 and 3): We recommend a minimum of 7-foot-deep remedial removals for the proposed building areas to provide a new, uniform compacted fill blanket. A minimum of 3-foot-deep remedial removals are recommended for the areas outside the buildings' adjacent areas (drives and parking). The lateral limits of the removals should extend a minimum of 5 feet beyond the building footprint to include any exterior architectural elements. The demolition operation and local variations in soil conditions may result in the need for deeper removals.

General (Overall site): The remedial removals should stay a minimum 2 feet above the current groundwater level (approximate elevation 26 feet msl) to reduce excess disturbance of the removal bottoms. Special stabilization measures may be required if the removal bottoms become disturbed. The bottoms of removals should be observed and accepted by the geotechnical consultant prior to placement of fill. Removals along the perimeter of the site will need to be performed in a manner that protects any existing improvements. If complete removals cannot be performed adjacent to the structural fill areas, a setback or special foundation design may be required.

3.2.3 Fill Placement

Onsite materials that are relatively free of deleterious material should be suitable for use as compacted fill. Our field investigation revealed the presence of very moist soil conditions below a depth of 3 to 4 feet from the surface. Prior to placement of fill, the removal bottoms should be scarified a minimum of 6 inches, moisture-conditioned as needed, and compacted to minimum 90 percent relative compaction. The relative compaction should be based upon ASTM Test Method D1557-91.

The moisture content of fill soil should be over optimum moisture content and consideration should be given to placing fill at higher moisture contents to facilitate the future presoaking process for slab-on-grade foundations. Much of the excavated soils may exceed compaction moisture-content requirements and some drying and/or mixing may be necessary. Fill material should be placed in loose lifts no greater than 8 inches in thickness and compacted prior to placement of the next lift. Ground sloping steeper than 5:1 (horizontal to vertical) should be prepared by benching into firm competent material as fill is placed.

Import soils are anticipated for Building 2 in order to achieve design grades. At this time, the import soil is anticipated to come from Buildings 1 and 2. If import material from a different site/source is deemed necessary, it should be evaluated and accepted by the geotechnical consultant prior to transport to the site to verify its suitability. Additional evaluation and recommendations may be required based on the import material. In general, we recommend that the import material be similar to the onsite soils (i.e., similar moisture content, expansion potential and soluble sulfate content). If the import material is a better material than the onsite soils, consideration should be given to capping the site or building areas with this material. It is important from a geotechnical standpoint to keep the foundation soils uniform.

3.2.4 Earthwork Factors

The loss or gain of volume (shrinkage or bulking, respectively) of excavated natural materials, upon recompaction as fill, varies according to earth material type and location. This volume change is represented as a percentage shrinkage (for volume loss) and as a percentage bulking (for volume gain) after recompaction of a unit volume of cut in this same material in its natural state. For example, a shrinkage value of 10 percent indicates that one cubic yard of cut will produce 0.9 cubic yard of compacted fill at 92 percent relative compaction.

Most of the shrinkage is anticipated for removals and overexcavation in the upper 5 to 7 feet. The estimated earthwork factors are:

- Stockpile and Undocumented Fill (Vacant Field) : 15 to 20 percent shrinkage.
- Existing Fill (Office Park): 1 to 3 percent shrinkage.
- Alluvium: 7 to 12 percent shrinkage.

3.3 Settlement Potential

Settlement analysis was performed to evaluate the general impact of future grading and site development. The settlement analysis utilized the collected subsurface data and the typical structural loading provided. The pad grade for Building 1 will require a cut of approximately 3 feet and the pads for Buildings 1 and 2 are not expected to change.

Overall, the estimated total consolidation (static) settlement for Building 1 is up to 2 inches. The presence of the existing stockpile soil has acted as a surcharge, reducing the anticipated settlement at Building 1 pad. The estimated total consolidation settlement for Buildings 2 and 3 is 2.5 inches. The actual settlement will depend upon the depth of removals performed during grading, the characteristics of materials left in place, soil properties of the compacted fill placed, and the structural loading.

The potential settlement due to liquefaction is expected to be on the order of 1.5 inches for the majority of the site. However, additional CPTs in the Building 1 area should be considered to further assess liquefaction settlement in this area.

We recommend 1.5 inches of differential settlement over a span of 60 feet for foundation design. Additional evaluation of the settlement should be performed once grading has been completed and foundation plans become available.

3.4 Foundation and Slab Design Guidelines

The following foundation recommendations are provided with the assumption that the recommendations included in Section 3.2 are implemented during grading of the site. The geotechnical parameters and recommendations provided are intended for the design of the footings, slab, and foundation system of the proposed structures. The design of shallow footings and slab-on-grade foundations will require collaboration between the geotechnical and structural engineers based on the anticipated structural loading conditions and considering the requirements of the 2022 CBC.

3.4.1 Allowable Bearing Capacity

The recommended net allowable bearing capacity for continuous and isolated footings, including retaining walls, may be calculated based on the following equation:

$q_{all} = 500\ D + 200\ B + 1000 \leq 3{,}000$ psf

where:

D = embedment depth of footing, in feet
B = width of footing, in feet

The following parameters may be used for design of foundation and slabs on grade:

- Soil unit weight = 120 pcf
- Soil internal friction angle = 26 degrees
- Coefficient of Friction = 0.33
- Subgrade modulus (k) of 75 pci

The dead load of concrete below adjacent grades (buried concrete foundations) may be neglected. The allowable bearing pressure and friction coefficient may be increased by one-third for wind and seismic loading.

We recommend that strip and isolated footings have a minimum embedment depth of 24 inches below the lowest adjacent grade. Continuous footings should be at least 12 inches wide and isolated column footings should be at least 24 inches wide. The footings of freestanding and isolated structures (including walls, pilasters, and other site amenities) should have a minimum embedment depth of 24 inches into approved soils.

3.4.2 Slab-on-Grade

We understand that the new buildings will have reinforced concrete slab-on-grade floor to support up to 600 psf uniformly distributed loading. The perimeter wall and column loads will be supported with separate footings. The floor slabs for the new buildings are anticipated to be 7- to 8-inch-thick and reinforced. Alternatively, the building could utilize a stiffer, thicker concrete mat slab designed to integrate the higher perimeter wall and column loads. The floor slab will also need to be designed for expansive soil in accordance with the 2022 CBC and Wire Reinforcement Institute (WRI) method. We recommended an Effective Plasticity Index of 22 for the upper 15 feet of soil materials.

In addition, we recommend the upper 18 inches of building pad subgrade soil should be pre-saturated to 130 percent of optimum moisture content prior to placement of moisture barrier and concrete.

3.4.3 Moisture Mitigation for Concrete Slab

In addition to geotechnical and structural considerations, the project owner should consider interior moisture mitigation when designing and constructing slabs-on-grade. The intended use of the interior space, types of flooring, and types of materials in contact with the floor may dictate the need for, and design of, measures to reduce potential effects of moisture emission from and/or moisture vapor transmission through the slab. Typically, for human-occupied structures, a vapor retarder or barrier is utilized under the slab to help mitigate moisture transmission through slabs.

Per Section 7.2 of ACI 302.2 R-06, the benefits of a granular layer include reducing the potential for the following:

- Concrete shrinkage cracks and slab curling during drying;
- Puncturing the vapor retarder;
- Surface blistering or delamination caused by an extended concrete bleeding period; and
- Settlement cracking over reinforcing steel.

The current guidelines by the American Concrete Institute (ACI 302.1R-04 and 302.2 R-06) allows the vapor retarder to be placed directly under the slab (with no granular fill layer). If the concrete slab is placed directly on the vapor retarder/barrier, a low-shrinkage mix design and other construction measures (i.e., better curing, reduced joint spacing, etc.) are required. Per Section 7.2 of ACI 302.2 R-06 the following are some benefits of placing the concrete directly on the vapor retarder:

- Reduced cost because of less excavation and no need for additional granular materials;
- Better curing of the slab bottom because the vapor retarder minimizes moisture loss; and
- Less chance of floor moisture problems caused by moisture being trapped in the granular layer.

Specifying the strength of the retarder to resist puncture and its permeance rating is important. These qualities are not necessarily a function of the retarder thickness. A minimum of 15-mil is typical but some materials, such as 15-mil polyethylene ("Visqueen"), may not meet the desired standards for toughness and permeance. The vapor retarder, when used, should be installed in accordance with standards such as ASTM E 1643 (and/or those specified by the manufacturer), including proper perimeter sealing. A 1 to 2 inch of sand (or granular fill material) can be used over and under the moisture retardant to protect the vapor retarder and to facilitate the concrete construction.

Concrete mix design and curing are also significant factors in mitigating slab moisture problems. Concrete with lower water/cement ratios (with higher compressive strength) results in denser, less permeable slabs. They also "dry" faster with regard to when flooring can be installed (reduced moisture emissions quantities and rates). Rewetting of the slab following curing should be avoided since this can result in additional drying time required prior to flooring installation. Proper concrete slab testing prior to flooring installation is also important.

3.5 Seismic Design Guidelines

The following table summarizes the seismic design criteria for the subject site. The seismic design parameters are developed in accordance with 2022 CBC and ASCE 7-16, including Supplement Nos. 1 through 3.

Selected Seismic Design Parameters from 2022 CBC/ASCE 7-16	***Seismic Design Values***	***Reference***
Latitude	33.6985 North	
Longitude	117.9127 West	
Controlling Seismic Source	San Joaquin Hills	USGS, 2024
Distance to Controlling Seismic Source	2.6 Miles (4.2 km)	USGS, 2024
Site Class per Table 20.3-1 of ASCE 7-16	D	SEA/OSHPD, 2024
S_S, Spectral Acceleration for Short Periods	1.31 g	SEA/OSHPD, 2024
S_1, Spectral Accelerations for 1-Second Periods	0.47 g	SEA/OSHPD, 2024
F_a, Site Coefficient, Table 11.4-1 of ASCE 7-16	1.0	SEA/OSHPD, 2024
F_v, Site Coefficient, Table 11.4-2 of ASCE 7-16	1.83	
S_{DS}, Design Spectral Response Acceleration at Short Periods from Equation 11.4-3 of ASCE 7-16	0.87 g	SEA/OSHPD, 2024
S_{D1}, Design Spectral Response Acceleration at 1-Second Period from Equation 11.4-4 of ASCE 7-16	0.86 g*	
T_S, S_{D1}/S_{DS}, Section 11.4.6 of ASCE 7-16	0.99 sec*	
T_L, Long-Period Transition Period	8 sec	SEA/OSHPD, 2024
PGA_M, Peak Ground Acceleration Corrected for Site Class Effects from Equation 11.8-1 of ASCE 7-16	0.62 g	SEA/OSHPD, 2024
Seismic Design Category, Section 11.6 of ASCE 7-16	D	

**These values have been increased by 50% as outlined in Supplement No. 3 of ASCE 7-16 Chapter 11.4.8.*

3.6 Lateral Earth Pressures for Retaining Structures

Recommendations for lateral earth pressures for retaining walls and structures (if any) with approved onsite drained soils are listed below. These parameters are based on a soil internal friction angle of 26 degrees and soil unit weight of 120 pcf.

Lateral Earth Pressures Equivalent Fluid Pressure (psf/ft.)	
Conditions	***Level***
Active	47
At Rest	67
Passive	310

The above parameters do not apply for backfill that is highly expansive. Retaining structures/walls may also need to be designed for additional lateral loads if other structures are planned within a 1H:1V projection.

Drainage behind retaining walls should also be provided in accordance with the attached figure (Figure 3). The waterproofing and drainage systems for the retaining walls that are located between the future residential lots may require additional measures to minimize the potential for nuisance seepage. Specific drainage connections, outlets and avoiding open joints should be considered for the retaining wall design.

To design an unrestrained retaining wall, such as a cantilever wall, the active earth pressure may be used. For a restrained retaining wall, the at-rest pressure should be used. Passive pressure is used to compute lateral soils resistance developed against lateral structural movement. The passive pressures provided above may be increased by one-third for wind and seismic loads. Passive resistance is taken into account only if it is ensured that the soil against embedded structure will remain intact with time. Future landscaping/planting and improvements adjacent to the retaining walls should also be taken into account in the design of the retaining walls. Excessive soil disturbance, trenches (excavation and backfill), future landscaping adjacent to footings and over-saturation can adversely impact retaining structures and result in reduced lateral resistance.

For sliding resistance, the friction coefficient of 0.33 may be used at the concrete and soil interface. The coefficient of friction may be increased by one-third for wind and seismic loads. The retaining walls may also need to be designed for additional lateral loads if other structures or walls are planned within a 1H:1V projection.

The seismic lateral earth pressure for walls retaining more than 6 feet of soil and level backfill conditions may be estimated to be an additional 19 pcf for active and at-rest conditions. The earthquake soil pressure has a triangular distribution and is added to the static pressures. For the active and at-rest conditions, the additional earthquake loading is zero at the top and maximum at the base. The seismic lateral earth pressure does not apply to walls retaining less than, or equal to, 6 feet of soil (2022 CBC Section 1803.5.12).

3.7 Pavement Design

Pavement design is based on the expected traffic index (TI) and the R-values of the subgrade soils. Onsite soils are considered to generally be poor to moderate subgrade soils for pavements. The pavement design is based on an R-value of 20. Final structural pavement sections should be based on R-value testing after the completion of grading. The following preliminary pavement sections are for the listed Traffic Indices (TIs):

Minimum Structural Pavement Section (Preliminary)			
Location	***TI***	***Composite Section***	***Full-Depth Section***
Auto parking areas	4.5	0.25' AC over 0.35' AB	0.50' AC
Auto circulation drives	5.5	0.35' AC over 0.60' AB	0.55' AC
Main Auto Drive	7.0	0.40' AC over 0.90' AB	0.75' AC
Main Truck Drive	8.0	0.45' AC over 1.20' AB	0.85' AC
Heavy Truck Drive	9.0	0.50' AC over 1.50' AB	1.00' AC
AC = Asphalt Concrete; AB = Aggregate Base			

Asphalt concrete should also be compacted to a minimum relative compaction of 95 percent. Please note that for two-stage paving operations, the initial based asphalt pavement layer should be a minimum of 0.25-foot AC and the final cap should be a minimum of 0.10 foot thick.

Prior to construction of pavement sections, the subgrade soils should be scarified to a minimum depth of 6 inches, moisture-conditioned as needed, and recompacted in place to a minimum of 90 percent relative compaction per ASTM D1557. The full-depth pavement area will require subgrade to have a minimum of 95 percent relative compaction. The geotechnical consultant should review the subgrade and provide additional recommendations if required based on actual conditions during construction. Subgrade for the proposed pavements should be uniform, firm, and unyielding.

AB materials can be crushed aggregate base or crushed miscellaneous base in accordance with the Greenbook (Section 200-2). The materials should be free of any deleterious materials. Aggregate base materials should be placed in 6- to 8-inch-thick loose lifts, moisture-conditioned as necessary, and compacted to a minimum of 95 percent relative compaction (per ASTM D1557).

Portland Cement Concrete (PCC) pavements: For the trash enclosure and drive areas, we recommend a minimum concrete pavement section of 6-inch PCC over 4 inches of AB compacted to 95 percent relative compaction. For heavy truck drive and loading areas, we recommend a minimum concrete pavement section of 7-inch PCC over 4 inches of aggregate base material compacted to 95 percent relative compaction. Where concrete is placed on native subgrade, the upper 6 inches of the subgrade should be moisture-conditioned and compacted to a minimum 90 percent relative compaction.

City standard plans/details specified for the project may also dictate the minimum concrete thickness. We recommend that concrete pavements should have a minimum compressive strength of 3,250 psi. Control joints should be carefully designed and constructed to minimize cracking of concrete pavements.

The pavement structural section was designed in accordance with the requirements of the City of Santa Ana the County of Orange Highway Design Manual. Street pavement should be placed in accordance with the City Standards (Number 1102 and 1102A) and requirements of Sections 301 and 302 of the Standard Specifications of Public Works Construction (the Greenbook). Prior to construction of pavement sections, the subgrade soils should be scarified to a minimum depth of 6 inches, moisture-conditioned as needed, and recompacted in-place to a minimum of 90 percent relative compaction (per ASTM D1557). Subgrade should be firm prior to AB placement.

3.8 Exterior Concrete (Non-Structural)

The recommendations provided below should be used for design and construction measures of the concrete pavements/hardscape. These recommendations are considered minimum and may be superseded by more stringent requirements/standards of the City of Santa Ana, the Standard Specifications for Public Work Construction "Greenbook" or other designers. The public pavements and other exterior concrete improvements (within the street right-of-way) should be constructed in accordance with City of Santa Ana Drawings (Plan No. 1104). The Greenbook provides concrete class and compressive strength requirements typically used for streets and surface improvements (i.e., pavements, curbs, gutters, ramps, sidewalks, driveways, walks, etc.) may be based on this specification provided more stringent requirements by governing agency or designer are not given.

Subgrade: The subgrade for the concrete pavement areas should be competent material that has been compacted and moisture-conditioned in accordance with the remedial grading recommendations for the site. The subgrade shall be compacted to a minimum of 90 percent relative compaction (as determined based on ASTM Test Method 1557).

Presaturation: For reducing the potential effects of expansive soils, we recommend presaturation of the subgrade prior to placement of the hardscape concrete. The recommended presaturation is 1.3 x optimum moisture to a depth of 18 inches. The subgrade of for concrete for pavements in drives/streets and the DG trail do not require presaturation.

Concrete Thickness: The nominal thickness for the concrete hardscape should be 4 inches. Pavements anticipated to have periodic vehicular traffic should be provided with the appropriate aggregate base, reinforcement and restraints, as discussed below. Note that City standards may govern the required minimum thicknesses for the public concrete pavements/sidewalks and exterior concrete elements in the right-of-way.

Aggregate Base: The approved subgrade soils should be adequate for support of the proposed pavements and should not require aggregate base sub-layer. For pavements that have vehicular loading, a minimum 4-inch-thick layer of aggregate base material should also be provided. Aggregate base should be crushed aggregate base (CAB) or crushed miscellaneous base (CMB) in accordance with Standard Specifications for Public Works Construction ("Greenbook"). The material should be free of any detrimental quantity of deleterious materials. The aggregate base should be observed and tested by the geotechnical consultant to verify that it is compacted to a minimum of 95 percent relative compaction based on ASTM Test Method D1557.

Reinforcement: Reinforcement should be considered for enhanced and/or decorative concrete pavements and pavements anticipated to have heavy loading. The minimum reinforcement should consist of No. 4 rebar placed 24 inches on-center (both directions) and placed near mid-height of the slab. Where used, reinforcing steel should be protected with the appropriate concrete cover.

The recommended subgrade preparation and presaturation, placement of aggregate base material, and sufficient control/expansion joints may be used in lieu of reinforcement for pavements that have typical soil conditions and pedestrian traffic only.

Joints: We recommend that longitudinal and transverse joint spacing for the concrete pavement be no more than 10 feet apart to control cracking. The depth of jointing must be at least ¼ of the slab thickness. Expansion joints need to be incorporated into the concrete pavements to allow for soil and thermal expansion (no more than 50 feet apart).

Restraints/Dowels: The use of dowels is considered optional for the pavements and other exterior concrete elements. In general, dowels are used most frequently for concrete pavements that carry traffic loads or require load transfer. The Portland Cement Association (PCA) and the American Concrete Institute (ACI) provide design documents for concrete pavements, proper joints and the use of dowels. Improper installation of dowels can increase the potential for concrete cracking at these locations.

Other Design Considerations:

- The design and construction should also be performed in adherence with the American Concrete Institute (ACI) and Portland Cement Association (PCA) guidelines for concrete improvements.
- Reducing cracking of concrete is also a function of proper concrete mix design, placement, and curing/finishing practices.
- The amount of post-construction watering, or lack thereof, can also have a significant impact on the adjacent concrete pavements, particularly when onsite soils are expansive.
- Additional measures, such as subdrains and/or moisture and root barriers, should be considered where planters or landscaping with irrigation are located adjacent to concrete improvements.
- Design and maintenance of proper surface drainage is important for reducing potential problems in hardscape/flatwork areas as a result of wetting and expansion of the subgrade soils.

3.9 Concrete Pavers

We recommend the following structural sections for concrete pavers:

- ***Pedestrian/Flatwork:*** 60-millimeter-thick pavers with a 1-inch sand leveling layer is typical. We also recommend placing a 4-inch AB layer with a geotextile over approved subgrade soil.
- ***Traffic / Drives:*** 80-millimeter-thick pavers with a 1-inch sand leveling layer is typical. We also recommend placing a 12-inch AB layer with a geotextile over approved subgrade soil.

A concrete band along the perimeter of the pavers should be provided for lateral restraint. The concrete band should, at minimum, be 12 inches deep and deepened to below the depth of disturbed soil, if adjacent to landscape/planter areas. The concrete band should include at least one No. 4 rebar placed at mid-height.

Prior to construction of the paver sections, the subgrade soils should be scarified to a minimum depth of 6 inches, and recompacted in-place to a minimum of 90 percent relative compaction. The subgrade for pedestrian flatwork should be presaturated to 130 percent of optimum moisture content to a depth of 18 inches. The subgrade of for concrete for pavements in drives/streets do not require presaturation.

3.10 Decomposed Granite Pavement Areas

Decomposed granite (DG) material should be placed in accordance with the product specifications and compacted to a minimum of 90 percent. The DG should be placed on approved subgrade material but does not require presaturation.

3.11 Soil Corrosivity and Cement Type

The soil soluble sulfates exposures at the site as found to be "negligible". However, we have encountered "moderate" sulfates in other nearby sites. The subject site may be classified as "S0" to "S1" per Table 19.3.2.1 of ACI-318-14. The chloride levels within the soils are classified as Class C1 and are moderately corrosive to metals.

Concrete mix requirements for structural concrete should be based on the "S1" exposure class of Table 19.3.2.1 in ACI-318-14 that lists the appropriate type of cement, maximum water-cement ratio, and minimum concrete compressive strength. The City of Santa Aana standards indicate a minimum compressive strength of 2,500 psi for the standard concrete hardscape/flatwork.

Structural concrete elements in contact with soil include footings and building slabs-on-grade. Concrete improvements for streets, sidewalk and hardscape typically are not considered structural elements. The onsite soils are severely corrosive to ferrous metals. The corrosivity evaluation report is included in Appendix C and provides specific corrosion control recommendations for differing materials.

3.12 Pipelines, Trench Excavations, Temporary Shoring, and Backfill

Excavations should conform to the latest edition of OSHA requirements (shoring or layback of trench or excavation walls). The near-surface soils across most of the site are anticipated to be classified as Type B for CalOSHA trenching and shoring excavation requirements. Excavations deeper than approximately 7 feet below existing ground surface and/or that encounter any seepage or shallow groundwater should be classified as Type C soils.

The published OSHA shoring design systems may be used for conventional shoring excavations less than 20 feet in depth. Soil loadings are provided below that do not include the effects of the additional loads from other surcharges. The geotechnical consultant should review the conditions during the deeper excavation and installation of shoring. Care should be taken at all times by personnel and/or equipment operators working adjacent to the excavations.

Native soils should generally be suitable for use as trench backfill. Backfill materials should be compacted to a minimum relative compaction of 90 percent (per ASTM D1557). We recommend that moisture content of native backfill to be over optimum moisture content. Native soils that are found to be wet will require reprocessing (e.g., mixing or drying) to achieve the uniform moisture content. Select backfill may be used in lieu of native soils.

Special bedding material may be required for trenches that encounter wet and soft soils at pipe depth. This will include overexcavating the trench 12 inches and providing the trench bottom with an additional layer compacted crushed rock to create a firm bottom. Specific bedding and shading requirements for the pipelines should be provided by the governing agency.

If a high-density, polyethylene (HDPE) pipe is proposed for the development, then excavation, installation, bedding, shading, and backfilling should be in strict accordance with the project and manufacturer's requirements. HDPE pipe has specific requirements for the width of the trench excavation. Also, HDPE pipe requires appropriate bedding and compaction of select granular backfill material in the pipe zone in order to provide uniform and adequate support. The initial backfill in the pipe zone (haunch) needs to be properly placed around the HDPE pipe, distributed by shovel to provide uniform support, "knifed-in" to remove voids and tamped for compaction.

3.13 Surface Drainage and Irrigation

Inadequate control of run-off water, heavy irrigation after development of the site, or regional groundwater level changes may result in shallow groundwater or seepage conditions where previously none existed. Maintaining adequate surface drainage, proper disposal of run-off water, and control of irrigation will help reduce the potential for future moisture-related problems and differential movements from soil heave/settlement.

Surface drainage should be carefully taken into consideration during grading, landscaping, and building construction. Positive surface drainage should be provided to direct surface water away from structures and slopes and toward the street or suitable drainage devices. Ponding of water adjacent to the structures should not be allowed. Buildings should have roof gutter systems and the run-off should be directed to parking lot/street gutters by area-drain pipes or by sheet flow over paved areas. Paved areas should be provided with adequate drainage devices, gradients, and curbing to prevent run-off flowing from paved areas onto adjacent unpaved areas.

Foundation performance is also dependent upon maintaining adequate surface drainage away from structures. The minimum gradient within 5 feet of the building will depend upon surface landscaping. Consideration should be given to the implementation of concrete flatwork adjacent to buildings. In general, we suggest that unpaved lawn and landscape areas have a minimum gradient of 2 percent away from structures, and drainage swales/bioswales should have a minimum gradient of 1 percent.

Construction of planter areas immediately adjacent to structures should be avoided if possible. If planter boxes are constructed adjacent to or near buildings, the planters should be provided with controls to prevent excessive penetration of the irrigation water into the foundation and flatwork subgrades. Provisions should be made to drain excess irrigation water from the planters without saturating the subgrade below or adjacent to the planters. Raised planter boxes may be drained with weepholes. Deep planters (such as palm tree planters) should be drained with below-ground, water-tight drainage lines connected to a suitable outlet. Moisture barriers should also be considered.

It is also important to maintain a consistent level of soil moisture, not allowing the subgrade soils to become overly dry or overly wet. Properly designed landscaping and irrigation systems can help in that regard.

3.14 Additional Geotechnical Review and Evaluation

The future grading and improvement plan, and the building foundation plan should be reviewed and accepted by the geotechnical consultant prior to site grading and construction. Additional soil testing and analysis may be required for more detailed recommendations or may result in updated/revised recommendations.

As discussed in Section 3.2, additional CPTs should be performed to better assess the liquefaction settlement potential within the vacant lot.

3.15 Observation and Testing During Grading and Construction

The findings, conclusions and recommendations in this report are based upon interpretation of data and data points having limited spatial extent. Verification and refinement of recommendations based on actual geotechnical conditions encountered during grading is very important. At minimum, geotechnical observation and testing should be conducted during site grading and construction at the following stages:

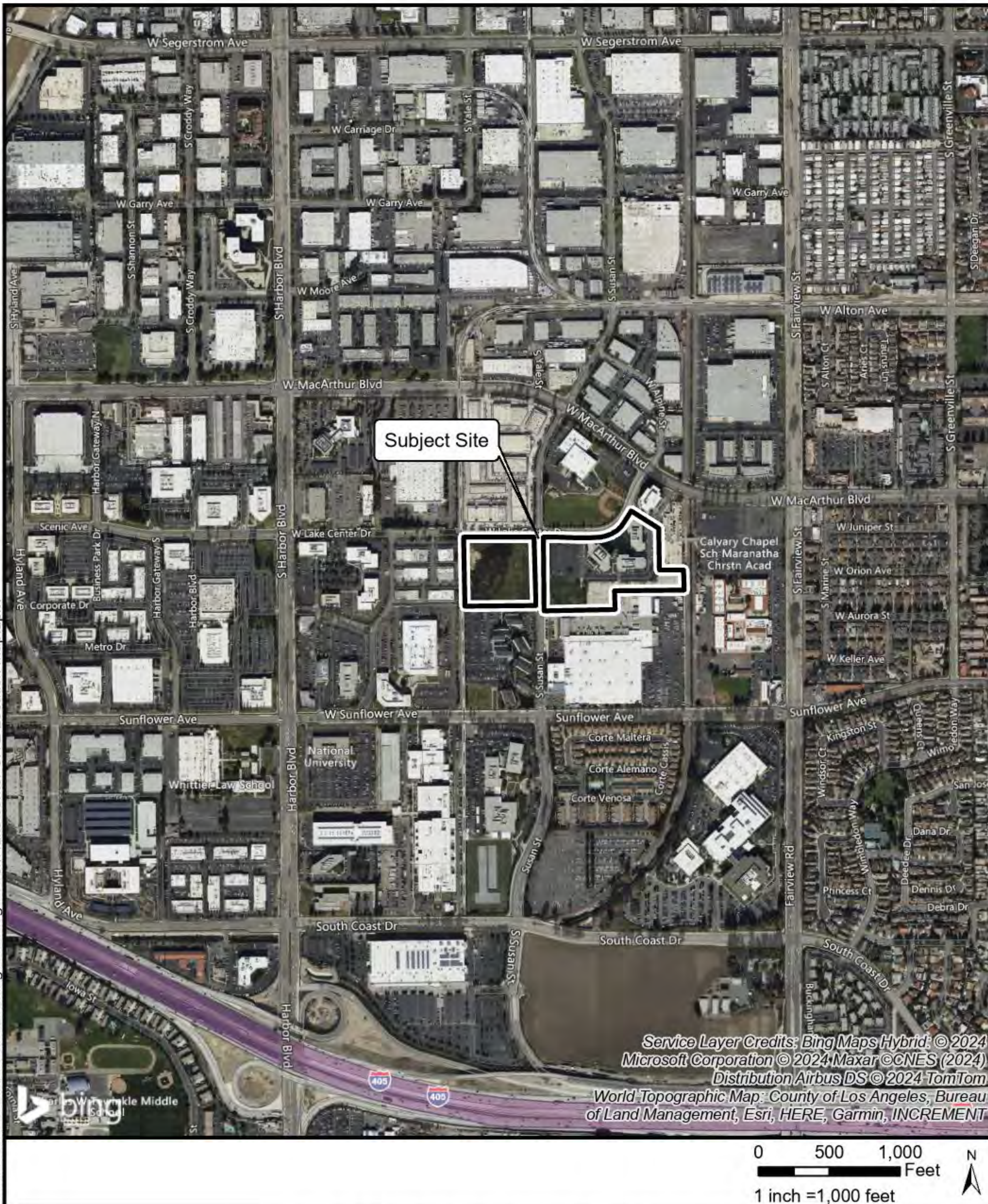
- During site preparation, clearing and demolition, prior to site processing;
- During backfill of excavations after removal of existing structures, improvements, and utility pipelines;
- During earthwork operations, including remedial removals and fill placement;
- Following the completion of rough grading, in order to verify soil properties for foundations, slab-on-grade and pavements;
- During excavation, backfilling and installation of utilities and storm water BMPs;
- Upon completion of any foundation excavations, prior to placement of pouring concrete;
- During slab and hardscape subgrade preparation and upon completion of presaturation;
- During construction of structural pavement sections;
- During construction of retaining walls, including subdrains (if any);
- During placement of backfill for utility trenches; and
- When any unusual soil conditions are encountered.

4.0 LIMITATIONS

This report has been prepared for the exclusive use of our client, C.J. Segerstrom & Sons, within the specific scope of services requested by them for the subject manufacturing/warehouse development in Santa Ana, California. This report or its contents should not be used or relied upon for other projects or purposes or by other parties without the written consent of NMG and the involvement of a geotechnical professional. The means and methods used by NMG for this study are based on local geotechnical standards of practice, care, and requirements of governing agencies. No warranty or guarantee, express or implied is given.

The findings, conclusions, and recommendations herein are professional opinions based on interpretations and inferences made from geologic and engineering data from specific locations and depths, observed or collected at a given time. By nature, geologic conditions can vary from point to point, can be very different in between points, and can also change over time. Our conclusions and recommendations are subject to verification and/or modification during excavation and construction when more subsurface conditions are exposed.

NMG's expertise and scope of services did not include assessment of potential subsurface environmental contaminants or environmental health hazards.



SITE LOCATION MAP

SOUTH COAST TECHNOLOGY CENTER
3100 LAKE CENTER DRIVE
CITY OF SANTA ANA, CALIFORNIA

Project Number: 23111-01 By: KGM/ZKH
Project Name: CJ Segerstrom/Lake Center
Date:4/12/24 Figure 1

NMG

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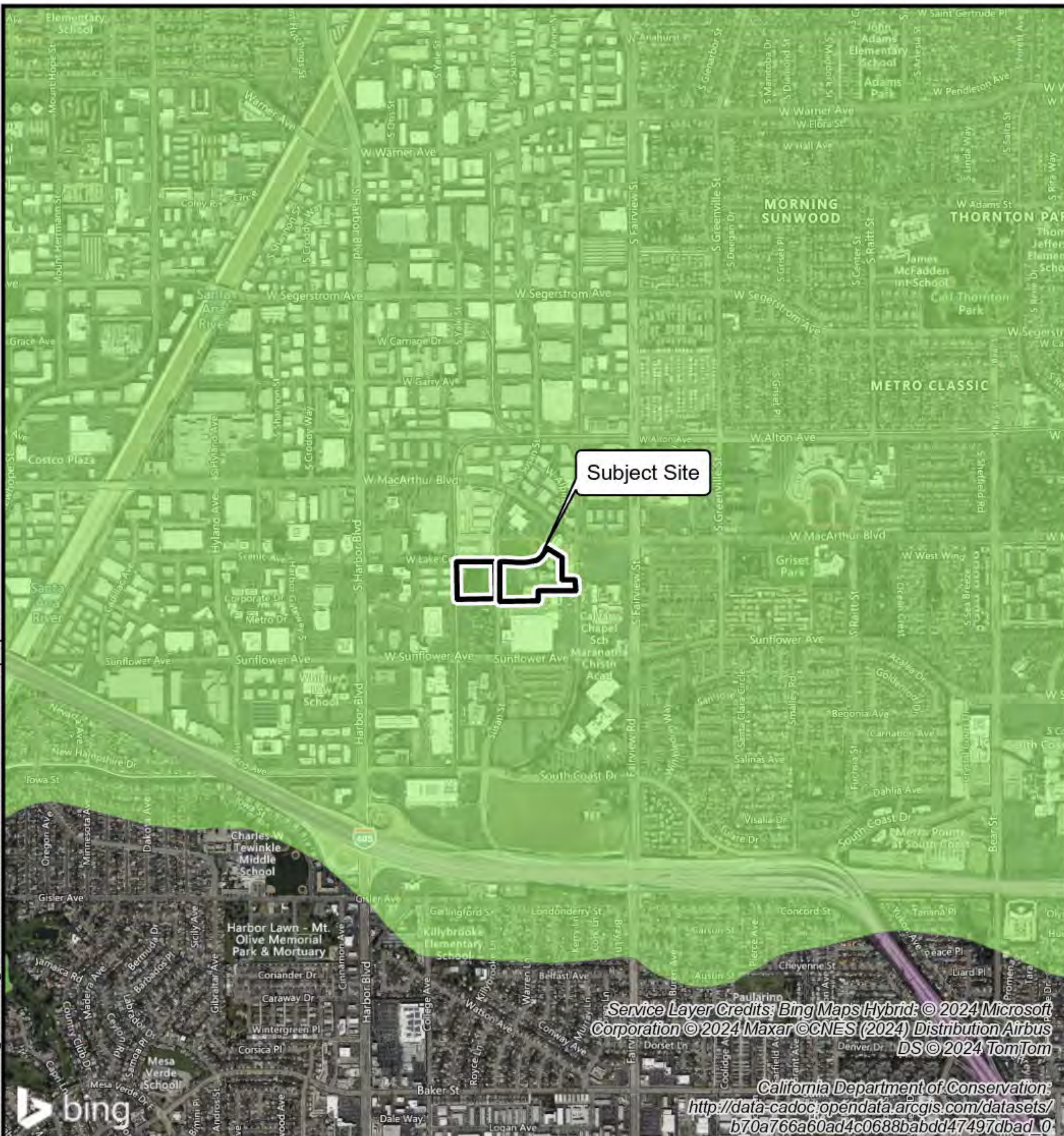

Legend

Liquefaction Zones

0 1,000 2,000 Feet

N

1 inch =2,000 feet

SEISMIC HAZARD ZONES MAP

Base: California Division of Mines and Geology, Seismic Hazard Zones Map, Newport Beach Quadrangle
Dated: April 17, 1997

SOUTH COAST TECHNOLOGY CENTER 3100 LAKE CENTER DRIVE CITY OF SANTA ANA, CALIFORNIA	Project Number: 23111-01 By: KGM/ZKH Project Name: CJ Segerstrom/Lake Center Date: 4/12/24 Figure 2	NMG

P:\2023\23111-01 Segerstrom - Lake Center Office Park\Drafting\GIS\Segerstrom lake center Seiscmic Hazard map.aprx

LEGEND

EARTH UNITS - CIRCLED WHERE BURIED

Afe	EXISTING COMPACTED FILL
Afu	UNDOCUMENTED FILL (IMPORT/STOCKPILE)
Qal	QUATERNARY ALLUVIUM

OTHER SYMBOLS - LOCATIONS ARE APPROXIMATE

H-6 T.D.26.5' Qal@7.5' No	HOLLOW-STEM AUGER BORING, SHOWING TOTAL DEPTH, DEPTH TO GEOLOGIC UNIT AND DEPTH TO GROUNDWATER (6 TOTAL)
CPT-7 T.D.60.1'	CONE PENETROMETER TEST, SHOWING TOTAL DEPTH (7 TOTAL)
T-5 T.D.15' Qal@1'	EXPLORATORY TRENCH, SHOWING TOTAL DEPTH, DEPTH TO GEOLOGIC UNIT, DEPTH TO SEEPAGE/GROUNDWATER (5 TOTAL)
	GROUNDWATER
	SEEPAGE

PLATE 1

GEOTECHNICAL MAP
BUILDINGS 1,2 AND 3, SOUTH COAST TECHNOLOGY CENTER
3100 LAKE CENTER DRIVE, CITY OF SANTA ANA, CALIFORNIA

Project No.: 23111-01
Project Name: Segerstrom/Lake Center
Date: 4/18/2024
By: KGM/ZKH
SCALE: 1" = 40'
NMG Geotechnical, Inc.

BASE MAP: DRA Architects

Drawing: P:\2023\23111-01 Segerstrom - Lake Center Office Park\Drafting_WP\23111-01 Geo Plate.dwg Layout: Layout1 Last Saved: Thu Apr 18, 2024 - 9:21am Last Plotted: Thu Apr 18, 2024 - 9:23am By: wrodriguez

APPENDIX A

APPENDIX A

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APPENDIX B

SOIL CLASSIFICATION CHART

MAJOR DIVISIONS			SYMBOLS		TYPICAL DESCRIPTIONS
COARSE GRAINED SOILS MORE THAN 50% OF MATERIAL IS LARGER THAN NO. 200 SIEVE SIZE	*GRAVEL AND GRAVELLY SOILS* MORE THAN 50% OF COARSE FRACTION RETAINED ON NO. 4 SIEVE	*CLEAN GRAVELS* (LITTLE OR NO FINES)		GW	WELL-GRADED GRAVELS, GRAVEL - SAND MIXTURES, LITTLE OR NO FINES
				GP	POORLY GRADED GRAVELS, GRAVEL - SAND MIXTURES, LITTLE OR NO FINES
		GRAVELS WITH FINES (APPRECIABLE AMOUNT OF FINES)		GM	SILTY GRAVELS, GRAVEL - SAND - SILT MIXTURES
				GC	CLAYEY GRAVELS, GRAVEL - SAND - CLAY MIXTURES
	SAND AND SANDY SOILS MORE THAN 50% OF COARSE FRACTION PASSING NO. 4 SIEVE	*CLEAN SANDS* (LITTLE OR NO FINES)		SW	WELL-GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES
				SP	POORLY GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES
		SANDS WITH FINES (APPRECIABLE AMOUNT OF FINES)		SM	SILTY SANDS, SAND - SILT MIXTURES
				SC	CLAYEY SANDS, SAND - CLAY MIXTURES
FINE GRAINED SOILS MORE THAN 50% OF MATERIAL IS SMALLER THAN NO. 200 SIEVE SIZE	*SILTS AND CLAYS*	LIQUID LIMIT LESS THAN 50		ML	INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOUR, SILTY OR CLAYEY FINE SANDS OR CLAYEY SILTS WITH SLIGHT PLASTICITY
				CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
				OL	ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY
	SILTS AND CLAYS	LIQUID LIMIT GREATER THAN 50		MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SANDY OR SILTY SOILS, ELASTIC SILTS
				CH	INORGANIC CLAYS OF HIGH PLASTICITY
				OH	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS
HIGHLY ORGANIC SOILS				PT	PEAT, HUMUS, SWAMP SOILS WITH HIGH ORGANIC CONTENTS

NOTE: Dual symbols are used to indicate gravels or sand with 5-12% fines and soils with fines classifying as CL-ML. Symbols separated by a slash indicate borderline soil classifications.

Sampler and Symbol Descriptions

- Modified California sample (D-#)
- Standard Penetration Test (S-#)
- Shelby tube sample (T-#)
- Large bulk sample (B-#)
- Small bulk sample (SB-#)
- Approximate depth of groundwater during drilling
- Approximate depth of static groundwater

Note: Number of blows required to advance driven sample 12 inches (or length noted).

Laboratory and Field Test Abbreviations

AL	Atterberg limits (plasticity)
CC	Chemical Testing incl. Soluble Sulfate
CN	Consolidation
DS	Direct Shear
EI	Expansion Index
GS	Grain Size Analysis (Sieve, Hydro. and/or -No. 200)
MD	Maximum Density and Optimum Moisture
RV	Resistance Value (R-Value)
SE	Sand Equivalent
UU	Unconsolidated Undrained Shear Strength

GENERAL NOTES

1. Soil classifications are based on the Unified Soil Classification System and include color, moisture, and relative density or consistency. Field descriptions have been modified to reflect results of laboratory tests where deemed appropriate. Bedrock descriptions are based on visual classification and include rock type, moisture, color, grain size, strength, and weathering.
2. Descriptions on these boring logs apply only at the specific boring locations and at the time the borings were drilled. They are not warranted to be representative of subsurface conditions at other locations or times.

KEY TO LOG OF BORING

Segerstrom / Lake Center Office Park
Santa Ana, CA
PROJECT NO. 23111-01

NMG Geotechnical, Inc.

Report: HOLLOW STEM; Project: 23111-01.GPJ; Data Template: NMG_GINT_2016.GDT; Printed: 4/11/24

Date(s) Drilled	2/29/24	Logged By	DDK
Drilling Company	2R Drilling, Inc	Drill Bit Size/Type	10"
Drill Rig Type	CME 75 Hollow Stem	Hammer Data	140 Lbs @ 30 Inch Drop
Sampling Method(s)	California Modified, SPT		
Approximate Groundwater Depth:	Groundwater Encountered at 25 Feet.		
Comments			

H-1

Sheet 1 of 2

Total Depth Drilled (ft)	51.5
Approximate Ground Surface Elevation (ft)	38.0 msl

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					ML-CL	Surface: Stockpile, locally ponded water. **Artificial Fill, Undocumented (Afu)**			
			D-1	5			@ 2.5' : Upper: Mottled dark yellowish brown to yellowish brown sandy SILT and silty CLAY, moist, soft.	17.8	105.5	
	5		D-2	22		ML	**Alluvium (Qal)** @ 5' : Dark yellowish brown sandy SILT, , moist, stiff, micaceous.	12.3	104.6	
						SM	Lower: Dark yellowish silty fine SAND, moist, medium dense, micaceous.			
30										
	10		D-3	15		ML	@ 10' : Yellowish brown to dark yellowish brown clayey SILT, wet, stiff.	30.9	86.2	
	15		D-4	10			@ 15' : Yellowish brown brown to olive brown clayey SILT, saturated, medium stiff, gray silt in tip.	32.2	87.5	
20										
	20		D-5	10		CL/CH	@ 20' : Very dark gray CLAY, saturated, medium stiff, decayed rootlets.	35.3	83.5	
	25						▽			

Elevation (ft)	Depth (ft)	SAMPLES Type	Number	Blows per foot	Graphic Log	USCS	MATERIAL DESCRIPTION	Moisture Content (%)	Dry Density (pcf)	OTHER TESTS and REMARKS
	25		D-6	9			@ 25' : Very dark gray CLAY, saturated, medium stiff, decayed rootlets, highly plastic.	38.6	81.5	
10										
	30		D-7	26		SC	@ 30' : Reddish brown clayey fine to medium SAND, saturated, medium dense.	13.2	122.0	
	35		SPT-1	22		SM	@ 35' : Reddish brown silty fine to medium SAND, saturated, medium dense.	18.5		
0										
	40		SPT-2	5		SM-ML	@ 40' : Yellowish brown silty fine SAND and yellowish brown SILT, saturated, medium stiff.	24.6		
	45		SPT-3	27		SM	@ 45' : Yellowish brown silty medium SAND, saturated, dense.	14.3		
-10										
	50		SPT-4	17			@ 50' : Yellowish brown silty medium to coarse SAND, saturated, medium dense.	16.6		Bottom 5" of sample missing.
							Notes: Total Depth 51.5 Feet. Groundwater Encountered at 25 Feet During Drilling. Backfilled with Tremie Pipe and Cement-Bentonite Grout.			
	55									

LOG OF BORING

Segerstrom / Lake Center Office Park
Santa Ana, CA

PROJECT NO. 23111-01

NMG

Date(s) Drilled	**2/29/24**	Logged By	**DDK**
Drilling Company	**2R Drilling, Inc**	Drill Bit Size/Type	**10"**
Drill Rig Type	**CME 75 Hollow Stem**	Hammer Data	**140 Lbs @ 30 Inch Drop**
Sampling Method(s)	**California Modified**		
Approximate Groundwater Depth:	**12.9 Feet After 24 Hours.**		
Comments			

H-2

Sheet 1 of 2

Total Depth Drilled (ft)	**26.5**
Approximate Ground Surface Elevation (ft)	**39.0 msl**

Elevation (ft)	Depth (ft)	SAMPLES Type	SAMPLES Number	SAMPLES Blows per foot	USCS	MATERIAL DESCRIPTION	Moisture Content (%)	Dry Density (pcf)	OTHER TESTS and REMARKS
	0				ML	Surface: Stockpile. **Artificial Fill, Undocumented (Afu)**			
		■	D-1	21		@ 2.5' : Mottled dark yellowish brown to yellowish brown SILT to sandy SILT, damp, stiff.	15.4	112.3	
	5	■	D-2	24	SM	**Alluvium (Qal)** @ 5' : Light gray silty fine SAND, damp, medium dense, friable.	2.4	95.6	
30									
	10	■	D-3	10	ML	@ 10' : Dark gray SILT with clay, wet, medium stiff, more clayey in upper rings.	36.4	84.6	
	15	■	D-4	9	CL/CH	@ 15' : Light olive brown silty CLAY and dark gray SILT, saturated, medium stiff.	35.8	82.7	
20									
	20	■	D-5	7		@ 20' : Dark gray silty CLAY, saturated, medium stiff, pores, decayed rootlets.	32.9	87.9	
	25								

LOG OF BORING

Segerstrom / Lake Center Office Park
Santa Ana, CA
PROJECT NO. 23111-01

NMG

Elevation (ft)	Depth (ft)	SAMPLES Type	Number	Blows per foot	Graphic Log	USCS	MATERIAL DESCRIPTION	Moisture Content (%)	Dry Density (pcf)	OTHER TESTS and REMARKS
	25		D-6	15		CL/CH	@ 25' : Gray to dark gray to light olive brown silty CLAY, saturated, stiff.	40.0	80.5	
10							Notes: Total Depth 26.5 Feet. Groundwater First Encountered at 24.3 Feet During Drilling. Groundwater Rose to 12.9 Feet Day After Drilling. Backfilled with Tremie Pipe and Cement-Bentonite Grout.			
	30									
	35									
0										
	40									
	45									
-10										
	50									
	55									

Date(s) Drilled	2/29/24	Logged By	DDK
Drilling Company	2R Drilling, Inc	Drill Bit Size/Type	10"
Drill Rig Type	CME 75 Hollow Stem	Hammer Data	140 Lbs @ 30 Inch Drop
Sampling Method(s)	California Modified, SPT		
Approximate Groundwater Depth:	Groundwater Encountered at 17 Feet.		
Comments			

H-3

Sheet 1 of 2

Total Depth Drilled (ft)	51.5
Approximate Ground Surface Elevation (ft)	35.2 msl

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: Asphalt parking lot. **Artificial Fill, Undocumented (Afu)**			
			D-1	25			**Alluvium (Qal)** @ 2.5': Yellowish brown to gray silty fine SAND, damp, medium dense, higher silt content in upper rings.	8.8	95.1	
30	5		D-2	9		SM/ML	@ 5' : Yellowish brown to olive brown silty fine SAND/fine sandy SILT, moist, medium stiff, micaceous.	18.2	93.5	
	10		D-3	8		ML	@ 10': Olive brown SILT, saturated, medium stiff, micaceous.	34.3	86.0	
20	15		D-4	9		CL/CH	@ 15' : Very dark gray CLAY, saturated, medium stiff, decayed rootlets. ∇	30.1	91.5	
	20		D-5	11			@ 20' : Very dark gray CLAY, saturated, medium stiff, decayed rootlets.	39.8	80.5	
	25									

Report: HOLLOW STEM; Project: 23111-01.GPJ; Data Template: NMG_GINT_2016.GDT; Printed: 4/11/24

Elevation (ft)	Depth (ft)	SAMPLES Type	Number	Blows per foot	Graphic Log	USCS	MATERIAL DESCRIPTION	Moisture Content (%)	Dry Density (pcf)	OTHER TESTS and REMARKS
10	25		D-6	18		SC	@ 25' : Mottled yellowish brown to reddish brown to dark brown clayey medium to coarse SAND, saturated, medium dense.	11.5	125.5	
	30		D-7	37		SM	@ 30' : Yellowish brown silty medium to coarse SAND, saturated, medium dense.	14.5	119.1	
0	35		SPT-1	41			@ 35' : Yellowish brown silty medium to coarse SAND, saturated, dense, little to no layering.	15.4		
	40		SPT-2	11		ML	@ 40' : Yellowish brown SILT, saturated, stiff.	31.3		
-10	45		SPT-3	10			@ 45' : Yellowish brown SILT, saturated, stiff, decayed rootlet.	21.2		
	50		SPT-4	13		SM	@ 50' : Yellowish brown silty medium to coarse SAND, saturated, medium dense, friable.	20.9		
	55						Notes: Totoal Depth 51.5 Feet. Groundwater Encountered at 17 Feet During Drilling. Backfilled with Tremie Pipe and Bentonite-Cement Grout. Patched with Concrete and Black Dye.			

LOG OF BORING

Segerstrom / Lake Center Office Park
Santa Ana, CA

PROJECT NO. 23111-01

NMG

Date(s) Drilled	**3/1/24**	Logged By	**DDK**
Drilling Company	**2R Drilling, Inc**	Drill Bit Size/Type	**10"**
Drill Rig Type	**CME 75 Hollow Stem**	Hammer Data	**140 Lbs @ 30 Inch Drop**
Sampling Method(s)	**California Modified**		
Approximate Groundwater Depth:	**Groundwater Encountered at 25 Feet.**		
Comments			

H-4

Sheet 1 of 2

Total Depth Drilled (ft)	**26.5**
Approximate Ground Surface Elevation (ft)	**38.0 msl**

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					ML	Surface : Grass in landscape area. **Artificial Fill, Uncocumented (Afu)**			
			D-1	13			@ 2.5' : Mottled dark yellowish brown to brown SILT, moist, stiff, rootlets.	22.5	98.8	
	5		D-2	10			@ 5' : Mottled dark brown to grayish brown SILT, medium stiff, moist.	22.5	97.6	
30						CH	**Alluvium (Qal)** @ 7.5': Contact based on surrounding borings.			
	10		D-3	10			@ 10': Upper: Dark brown to grayish brown CLAY, moist, medium stiff, pinhole pores	38.2	80.4	AL, DS, CN
	15		D-4	8			@ 15' : Grayish brown CLAY, medium dense, saturated, decayed roots, very dark gray clay with decayed roots in tip.	40.4	78.2	AL, CN
20										
	20		D-5	14		CL/CH	@ 20' : Very dark gray silty CLAY, saturated, medium stiff to stiff, decayed roots, gray caliche nodules.	35.6	84.9	
	25						▽			

LOG OF BORING

Segerstrom / Lake Center Office Park
Santa Ana, CA
PROJECT NO. 23111-01

NMG

Elevation (ft)	Depth (ft)	Type	Number	Blows per foot	Graphic Log	USCS	MATERIAL DESCRIPTION	Moisture Content (%)	Dry Density (pcf)	OTHER TESTS and REMARKS
	25		D-6	10		CL/CH	@ 25' : Very dark gray silty CLAY with trace sand grains, medium stiff, saturated, decayed roots.	31.4	91.2	
10							Notes: Total Depth 26.5 Feet. Water Encountered at 25 Feet During Drilling. Backfilled Tremie Pipe and Bentonite-Cement Grout.			
	30									
	35									
0										
	40									
	45									
-10										
	50									
	55									

Report: HOLLOW STEM; Project: 23111-01.GPJ; Data Template: NMG_GINT_2016.GDT; Printed: 4/11/24

LOG OF BORING

Segerstrom / Lake Center Office Park
Santa Ana, CA

PROJECT NO. 23111-01

NMG

Date(s) Drilled	3/1/24	Logged By	DDK
Drilling Company	2R Drilling, Inc	Drill Bit Size/Type	10"
Drill Rig Type	CME 75 Hollow Stem	Hammer Data	140 Lbs @ 30 Inch Drop
Sampling Method(s)	California Modified, SPT		
Approximate Groundwater Depth:	Groundwater Encountered at 15 Feet.		
Comments			

H-5

Sheet 1 of 2

Total Depth Drilled (ft)	51.5
Approximate Ground Surface Elevation (ft)	37.5 msl

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					ML	Surface : Asphalt parking lot. **Artificial Fill, Undocumented (Afu)**			
			D-1	20			@ 2.5' : Mottled very dark gray to very dark brown clayey SILT, stiff, moist.	20.8	103.7	
	5		D-2	13			@ 5' : Very dark gray clayey SILT, stiff moist., contact in tip.	24.7	96.9	
30						ML	**Alluvium (Qal)** Tip: Pale brown SILT medium dense, moist, micaceous.			
	10		D-3	10			@ 10' : Mottled grayish brown to light yellowish brown SILT, saturated, medium dense, micaceous.	33.5	87.0	
	15		D-4	6		CL	▽ @ 15' : Very dark gray CLAY, saturated, soft to medium stiff, decayed rootlets.	38.1	85.1	AL, CN
20										
	20		D-5	13		CL/CH	@ 20' : Very dark gray CLAY, saturated, stiff, decayed rootlets, light gray caliche nodules.	38.9	82.2	
	25									

LOG OF BORING

Segerstrom / Lake Center Office Park
Santa Ana, CA
PROJECT NO. 23111-01

NMG

Elevation (ft)	Depth (ft)	Type	Number	Blows per foot	Graphic Log	USCS	MATERIAL DESCRIPTION	Moisture Content (%)	Dry Density (pcf)	OTHER TESTS and REMARKS
	25		D-6	11			@ 25' : Very dark gray CLAY, saturated, stiff, decayed rootlets, light gray caliche nodules.	30.2	90.5	
10						SC	Tip: Yellowish brown clayey fine SAND, saturated, loose to medium dense.			
	30		D-7	18		CL	@ 30' : Strong brown sandy CLAY, saturated, stiff, laminated.	15.7	114.8	
	35		SPT-1	6		SM	@ 35' : Reddish yellow silty medium SAND, saturated, loose, little to no layering.	17.5		
0										
	40		SPT-2	20			@ 40' : Reddish yellow silty medium SAND, saturated, medium dense, little to no layering.	16.8		
	45		SPT-3	67			@ 45' : Yellowish brown medium to coarse silty SAND, saturated, very dense, 2" thick very dark brown clayey sand/silt layer within sampler.	16.5		
-10										
	50		SPT-4	25			@ 50' : No recovery.			
	55						Notes: Total Depth 51.5 Feet. Groundwater Encountered at 15 Feet During Drilling. Backfilled with Tremie Pipe and Bentonite-Cement Grout. Patched with Concrete and Black Dye.			

LOG OF BORING

Segerstrom / Lake Center Office Park
Santa Ana, CA

PROJECT NO. 23111-01

NMG

H-6

Date(s) Drilled	**3/1/24**	Logged By	**DDK**
Drilling Company	**2R Drilling, Inc**	Drill Bit Size/Type	**10"**
Drill Rig Type	**CME 75 Hollow Stem**	Hammer Data	**140 Lbs @ 30 Inch Drop**
Sampling Method(s)	**California Modified, Bulk**		
Approximate Groundwater Depth:	**Groundwater Not Encountered.**		
Comments			

Total Depth Drilled (ft)	**26.5**
Approximate Ground Surface Elevation (ft)	**37.5 msl**

Elevation (ft)	Depth (ft)	SAMPLES Type	SAMPLES Number	SAMPLES Blows per foot	USCS	MATERIAL DESCRIPTION	Moisture Content (%)	Dry Density (pcf)	OTHER TESTS and REMARKS
	0				CL	Surface : Asphalt Parking Lot, 4" AC over 4" AB. **Artificial Fill, Undocumented (Afu)** @ 1' - 5' : Brown to yellowish brown sandy CLAY, damp, becomes moist below 4', stiff.			GS, AL, RV, CC
		Bulk	B-1						
	5	Drive	D-1	35		@ 5' : Dark brown sandy CLAY, very stiff, damp, mica.	12.4	105.9	
30					ML/CL	**Alluvium (Qal)**			
	10	Drive	D-2	11		@ 10' : Grayish brown clayey SILT/silty CLAY, saturated, medium stiff, mica.	27.0	93.2	
	15	Drive	D-3	7	CL	@ 15' : Gray CLAY, saturated, medium stiff, decayed roots.	43.8	80.7	AL, CN
20									
	20	Drive	D-4	8	CL/CH	@ 20 : Yellowish brown CLAY, saturated, medium stiff, free water along decayed rootlets..	34.1	88.2	
	25								

LOG OF BORING

Segerstrom / Lake Center Office Park
Santa Ana, CA
PROJECT NO. 23111-01

NMG

Elevation (ft)	Depth (ft)	SAMPLES Type	Number	Blows per foot	Graphic Log	USCS	MATERIAL DESCRIPTION	Moisture Content (%)	Dry Density (pcf)	OTHER TESTS and REMARKS
	25		D-5	7		CL/CH	@ 25' : Grayish brown CLAY, saturated, medium stiff, no rootlets.	32.4	89.4	
10							Notes: Total Depth 26.5 Feet. No Groundwater Encountered During Drilling. Backfilled with Tremie Pipe and Bentonite-Cement Grout. Patched with Concrete and Black Dye.			
	30									
	35									
0										
	40									
	45									
-10										
	50									
	55									

LOG OF BORING

Segerstrom / Lake Center Office Park
Santa Ana, CA

PROJECT NO. 23111-01

NMG

NMG

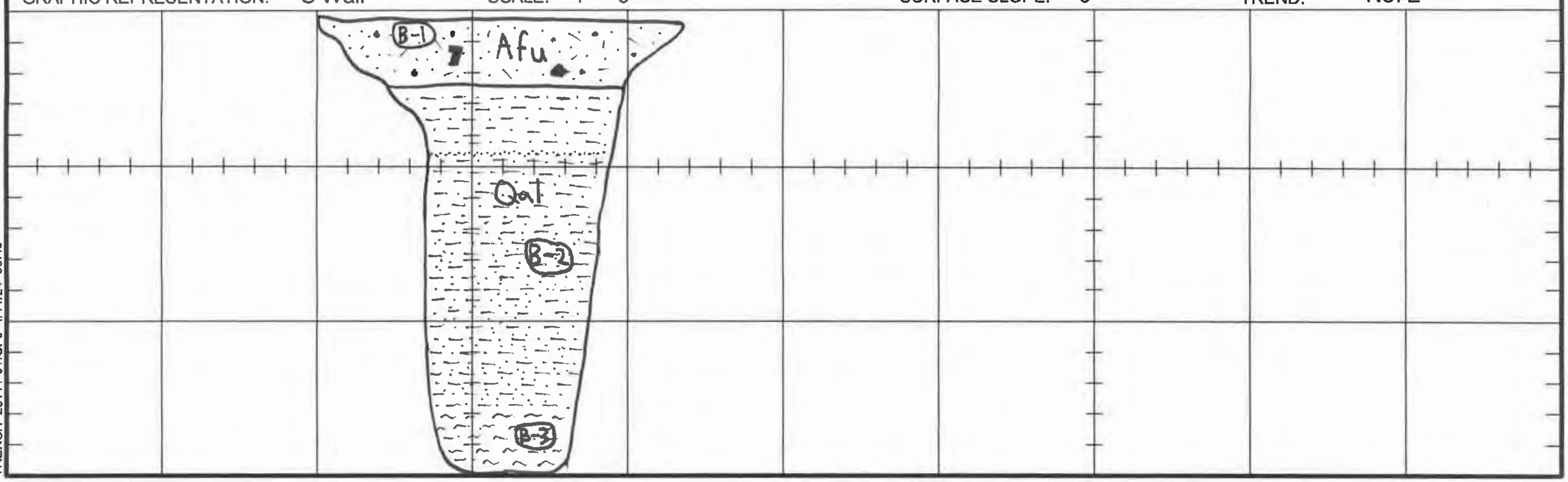
Project Name:	Segerstrom / Lake Center Office Park	Logged By:	DDK
Project Number:	23111-01	Elevation:	39' (msl)
Equipment:	John Deere 310SL HL	Location:	See Plate 1 for Trench Location.

TRENCH NO.: **T-1**

GEOLOGIC ATTITUDES	DESCRIPTION: DATE: 3/12/24	GEOLOGIC UNIT	U.S.C.S.	SAMPLE NO.	MOISTURE CONTENT (%)	DRY DENSITY (pcf)
	Artificial Fill, Undocumented (Afu) @ 0'-3': Mottled brown gravelly silty medium to coarse SAND and gray brown sandy CLAY with gravel, moist to wet, medium dense, scattered concrete fragments.	**Afu**	SM-CL	B-1 @ 0-1'	9.8	
	Alluvium (Qal) @ 3'-7': Mottled dark yellowish brown to dark brown SILT, trace sand, moist, medium stiff, pinhole pores, trace rootlets, micaceous.	**Qal**	ML			
	@ 4.5' : Yellowish brown silty fine sand layer 2" thick.					
	@ 7' : Gray SILT with fine sand, moist, medium stiff, pencil tip pores, iron staining.		ML	B-2 @ 8-9'	14.4	
	@ 13' : Dark gray silty CLAY, wet to saturated, medium stiff.		CL			
	Notes: Total Depth 15 Feet. No Groundwater Encountered. Backfilled with Cuttings and Tamped.			B-3 @ 14-15'	29.0	

ENGINEERING PROPERTIES

GRAPHIC REPRESENTATION: S Wall SCALE: 1" = 5' SURFACE SLOPE: 0° TREND: <--N87E--



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LOG OF TRENCH NO: **T-1**

NMG Geotechnical, Inc.

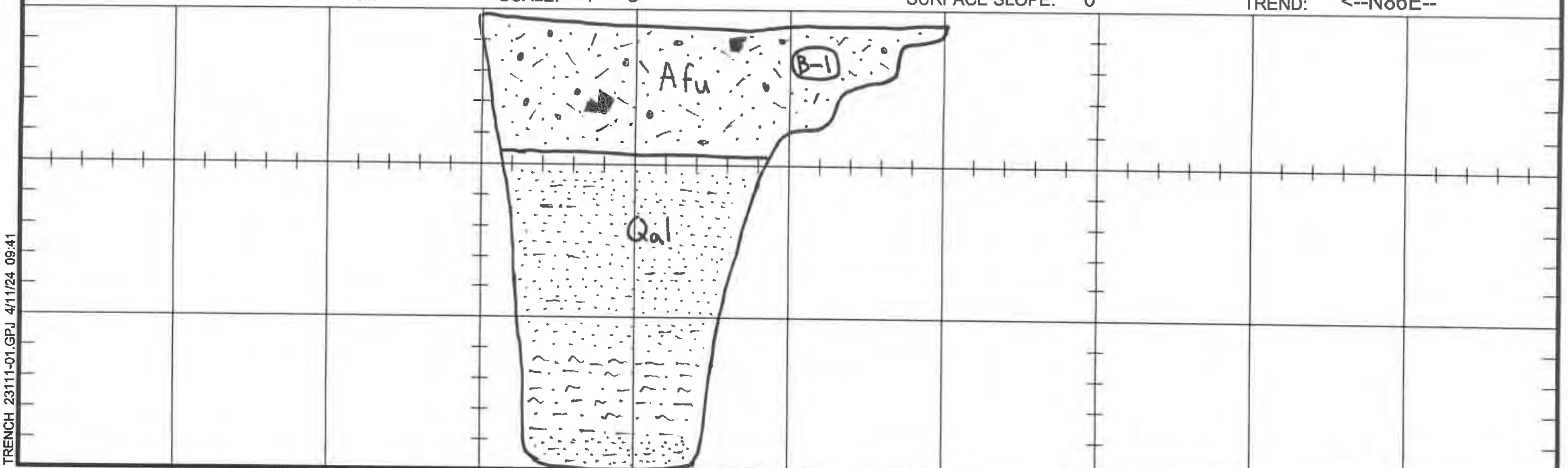
Project Name:	Segerstrom / Lake Center Office Park	Logged By:	DDK
Project Number:	23111-01	Elevation:	39' (msl)
Equipment:	John Deere 310SL HL	Location:	See Plate 1 for Trench Location.

TRENCH NO.: **T-2**

DATE: 3/12/24

GEOLOGIC ATTITUDES	DESCRIPTION	GEOLOGIC UNIT	U.S.C.S.	SAMPLE NO.	MOISTURE CONTENT (%)	DRY DENSITY (pcf)
	Artificial Fill, Undocumented (Afu) @ 0': Mottled dark brown sandy SILT with gravel, moist, stiff, rootlets, scattered concrete fragments.	**Afu**	ML	B-1 @ 2-3'	16.4	
	Alluvium (Qal) @ 4.5': Light grayish brown fine SAND with silt, moist, medium dense, friable, micaceous.	**Qal**	SM			
	@ 11' : Gray clayey SILT, moist to wet, medium stiff, pinhole pores.		ML			
	@ 14' : Grayish brown silty fine SAND, wet, medium dense, micaceous.		SM			
	Notes: Total Depth 15 Feet. No Groundwater Encountered. Backfilled with Cuttings and Tamped.					

GRAPHIC REPRESENTATION: S Wall SCALE: 1" = 5' SURFACE SLOPE: 0° TREND: <--N86E--



LOG OF TRENCH NO: T-2

Project Name:	Segerstrom / Lake Center Office Park	Logged By:	DDK	TRENCH NO.:
Project Number:	23111-01	Elevation:	39' (msl)	**T-3**
Equipment:	John Deere 310SL HL	Location:	See Plate 1 for Trench Location.	

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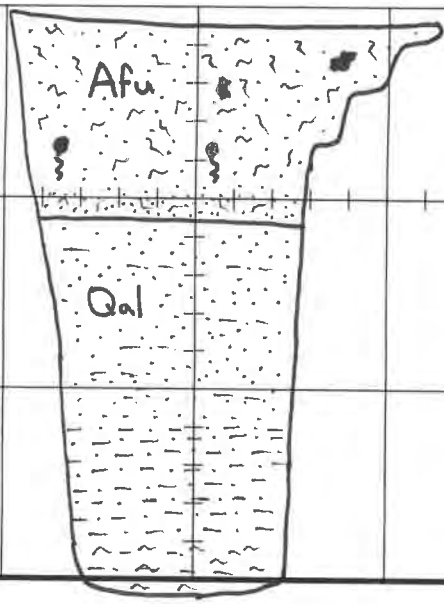
GEOLOGIC ATTITUDES	DESCRIPTION: DATE: 3/12/24	GEOLOGIC UNIT	U.S.C.S.	SAMPLE NO.	MOISTURE CONTENT (%)	DRY DENSITY (pcf)
	Artificial Fill, Undocumented (Afu) @ 0': Mottled dark brown to reddish brown sandy CLAY with gravel, medium stiff, wet, local seepage, scattered concrete fragments.	Afu	CL			
	@ 4.5' : Reddish brown clayey medium to coarse SAND, medium stiff, moist.		SC			
	Alluvium (Qal) @ 5.5' : Light grayish silty fine SAND, moist, medium dense, micaceous.	Qal	SM			
	@ 11' : Olive gray to gray SILT, medium stiff, moist.		ML			
	@ 14' : Gray SILT and yellowish brown CLAY, moist to wet, medium stiff.		ML-CL			
	Notes: Total Depth 16 Feet. Perched Seepage at 3.5 Feet. Backfilled with Cuttings and Tamped.					

ENGINEERING PROPERTIES

GRAPHIC REPRESENTATION: E Wall SCALE: 1" = 5' SURFACE SLOPE: 0° TREND: <--N5W--



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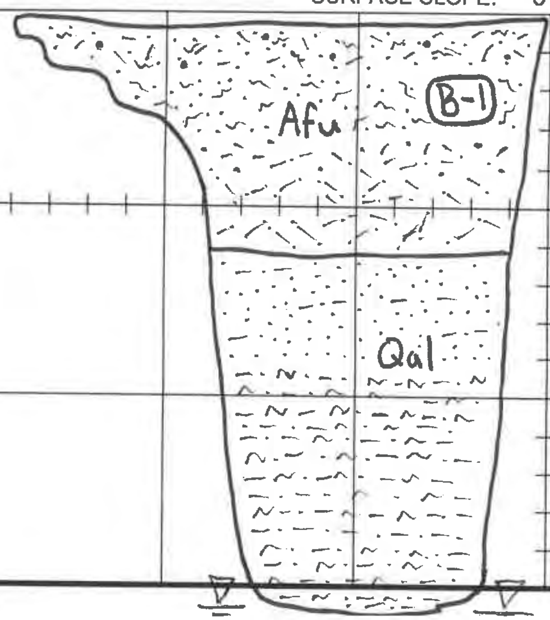
LOG OF TRENCH NO: T-3

TRENCH 23111-01.GPJ 4/11/24 09:41

Project Name:	Segerstrom / Lake Center Office Park	Logged By:	DDK	TRENCH NO.:
Project Number:	23111-01	Elevation:	38' (msl)	T-4
Equipment:	John Deere 310SL HL	Location:	See Plate 1 for Trench Location.	

GEOLOGIC ATTITUDES	DESCRIPTION: DATE: 3/12/24	GEOLOGIC UNIT	U.S.C.S.	SAMPLE NO.	MOISTURE CONTENT (%)	DRY DENSITY (pcf)
	Artificial Fill, Undocumented (Afu)	Afu				
	@ 0': Dark brown sandy SILT with gravel and clay, moist, medium stiff, rootlets.		ML			
	@ 1.5' : Reddish brown sandy CLAY, stiff, moist.		CL	B-1 @ 3-4'	16.8	
	@ 3' : Dark brown SILT with trace gravel, moist, medium stiff, micaceous.		ML			
	@ 4' : Mottled yellowish brown SILT, moist, medium stiff, caliche.					
	Alluvium (Qal)	Qal				
	@ 6': Gray fine SAND with silt, moist, medium dense.		SM			
	@ 9' : Very dark gray clayey SILT, moist, medium stiff, micaceous.		ML			
	@ 12' : Olive gray and yellowish brown clayey SILT, wet to saturated, medium stiff, micaceous.					
	Notes: Total Depth 16 Feet. Groundwater Encountered at 16 Feet. Backfilled with Cuttings and Tamped.					

GRAPHIC REPRESENTATION: E Wall SCALE: 1" = 5' SURFACE SLOPE: 0° TREND: <--N7W--



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LOG OF TRENCH NO.: T-4

Project Name: Segerstrom / Lake Center Office Park	Logged By: DDK	TRENCH NO.: T-5	
Project Number: 23111-01	Elevation: 37' (msl)		
Equipment: John Deere 310SL HL	Location: See Plate 1 for Trench Location.		

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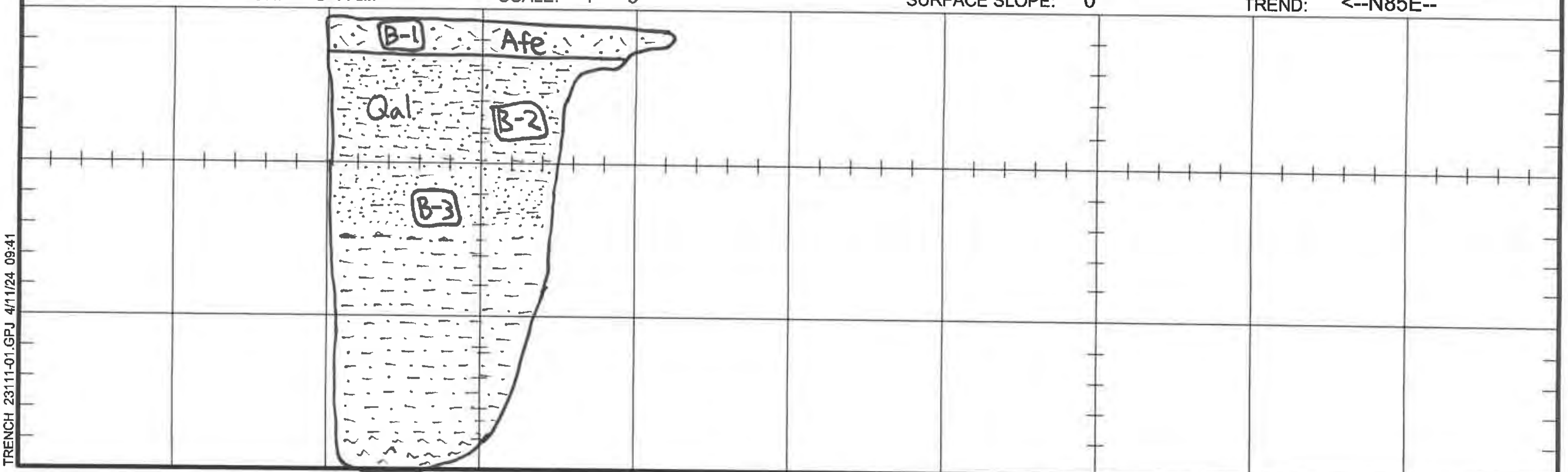
GEOLOGIC ATTITUDES	DESCRIPTION: DATE: 3/12/24	GEOLOGIC UNIT	U.S.C.S.	SAMPLE NO.	MOISTURE CONTENT (%)	DRY DENSITY (pcf)
	Existing Artificial Fill (Afe) @ 0': Dark brown SILT with trace gravel, moist, abundant grass roots.	Afe	ML			
	Alluvium (Qal) @ 1': Dark yellowish brown SILT with sand, moist, medium stiff, pinhole pores, rootlets, micaceous, sandier in upper 2', more silty between 2' and 4.5'.	Qal		B-1 @ 1-2'	18.4	
					18.1	
	@ 4.5' : Light yellowish brown silty fine SAND, moist, medium dense, micaceous.		SM	B-2 @ 3-4'	10.4	
				B-3 @ 6-7'		
	@ 7' : Grayish brown SILT, moist to wet, medium stiff, micaceous.		ML			
	@ 14' : Yellowish brown and gray CLAY, wet, medium stiff, decayed rootlets.		CL			
	Notes: Total Depth 15 Feet. No Groundwater Encountered. Backfilled with Cuttings and Tamped.					

GRAPHIC REPRESENTATION: S Wall SCALE: 1" = 5' SURFACE SLOPE: 0° TREND: <--N85E--



Kehoe Testing and Engineering
714-901-7270
steve@kehoetesting.com
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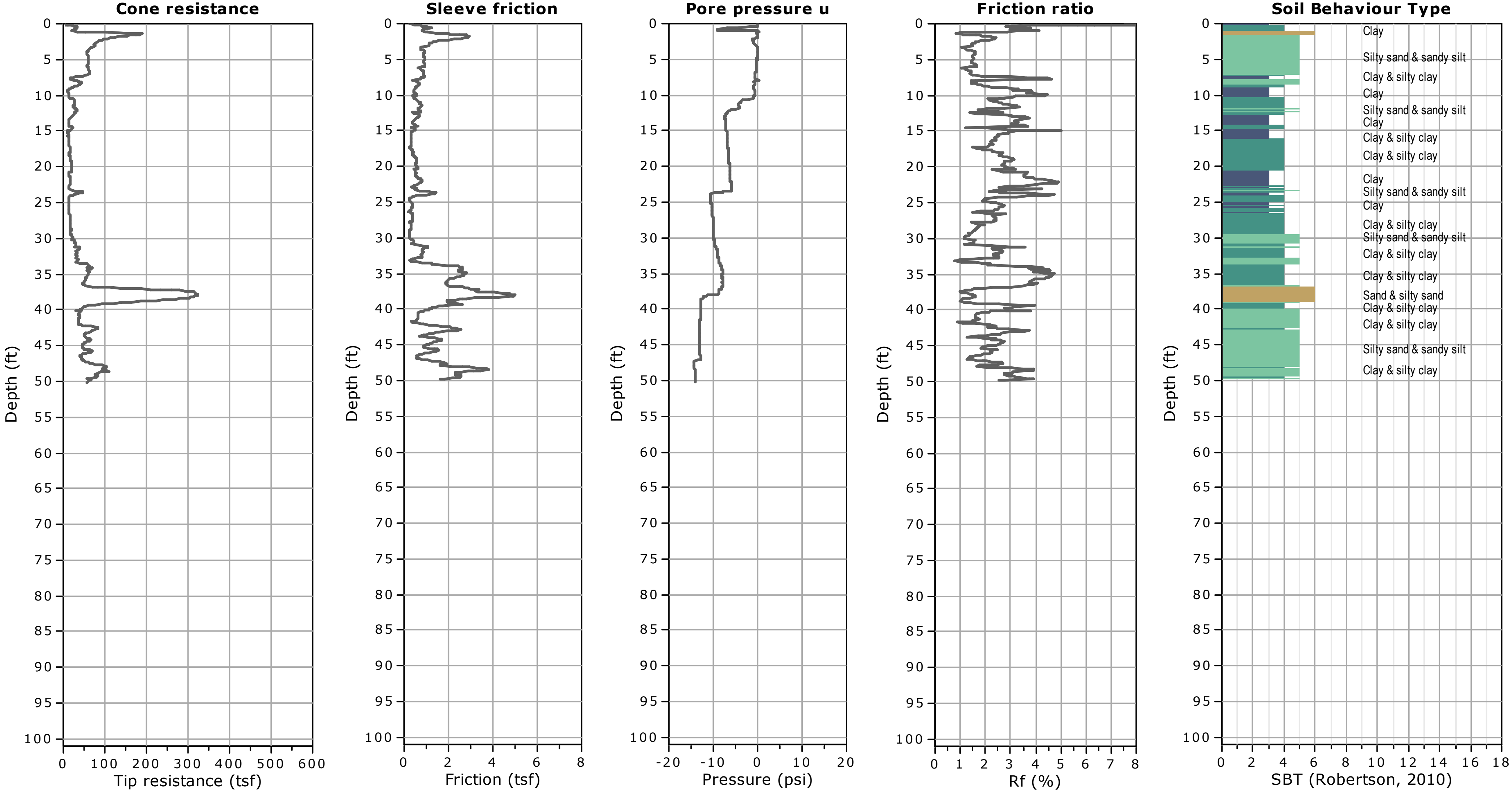
Project: **NMG Geotechnical - Segerstrom / Lake Center** **CPT-1**

Location: **3120 W. Lake Center Dr, Santa Ana, CA** Total depth: 50.30 ft, Date: 2/26/2024



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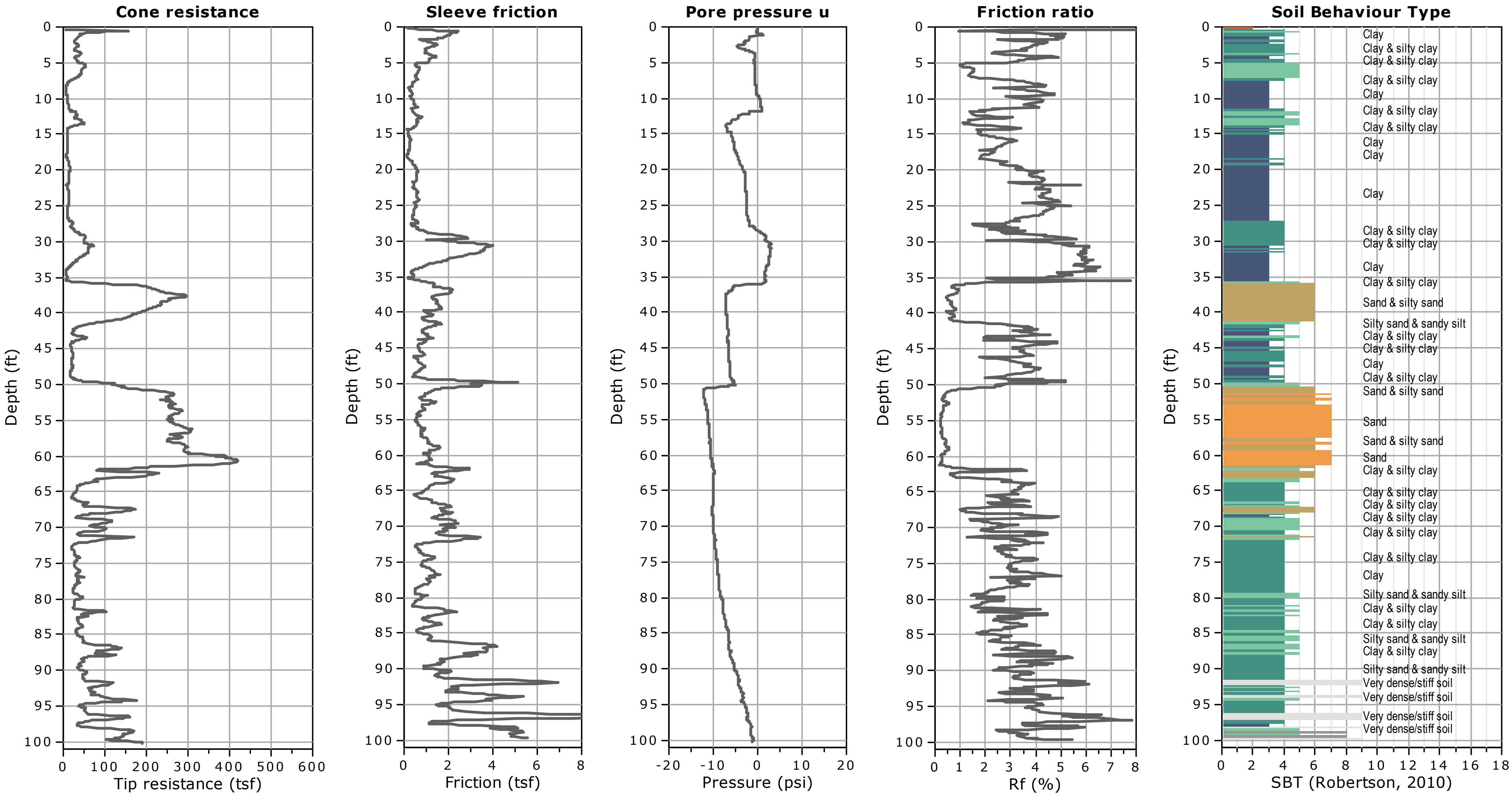
Project: **NMG Geotechnical - Segerstrom / Lake Center**
Location: **3120 W. Lake Center Dr, Santa Ana, CA**

CPT-2
Total depth: 100.14 ft, Date: 2/26/2024



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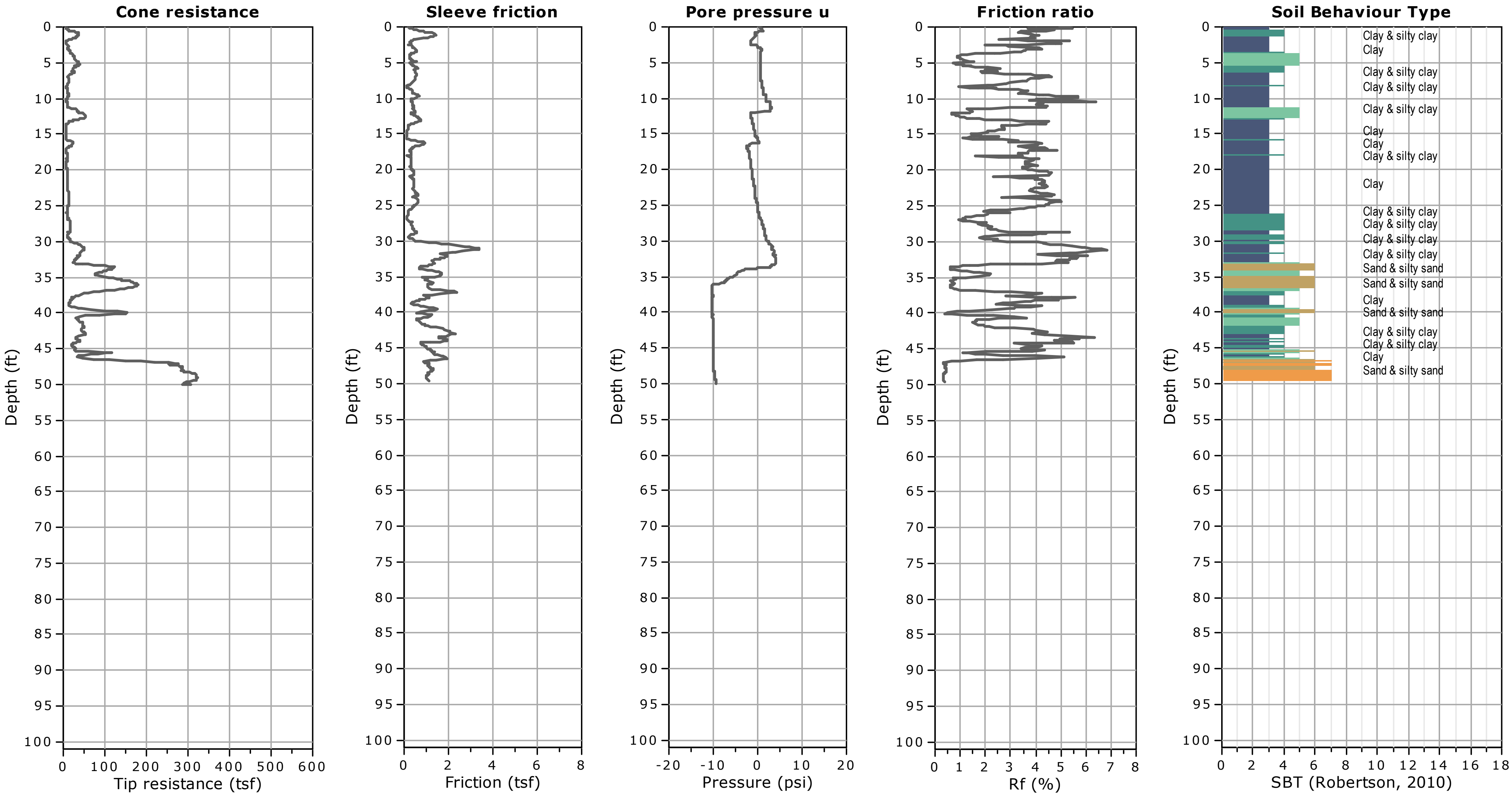
Project: NMG Geotechnical - Segerstrom / Lake Center
Location: 3120 W. Lake Center Dr, Santa Ana, CA

CPT-3
Total depth: 50.10 ft, Date: 2/26/2024



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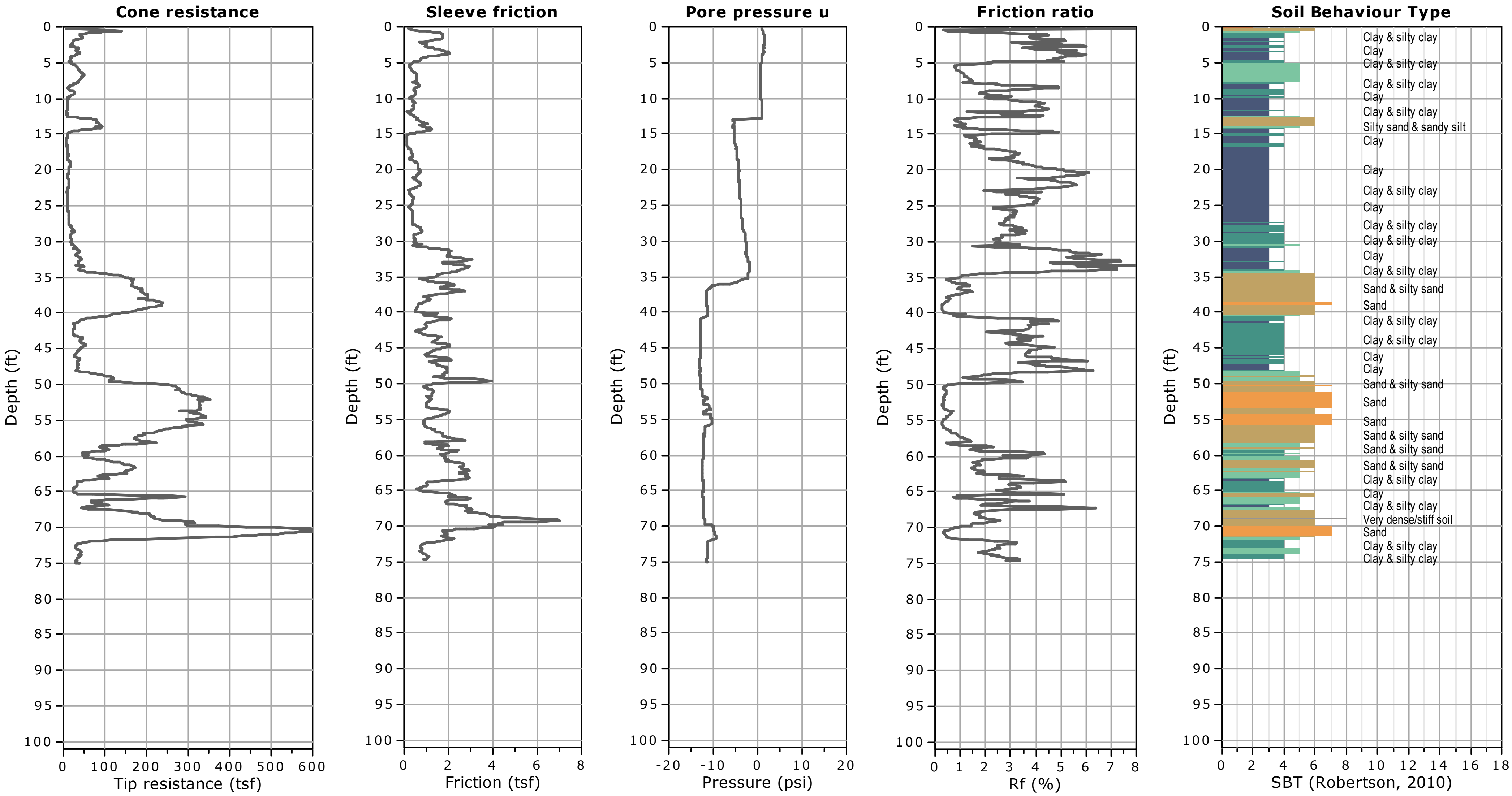
Project: NMG Geotechnical - Segerstrom / Lake Center
Location: 3120 W. Lake Center Dr, Santa Ana, CA

CPT-4
Total depth: 75.08 ft, Date: 2/26/2024



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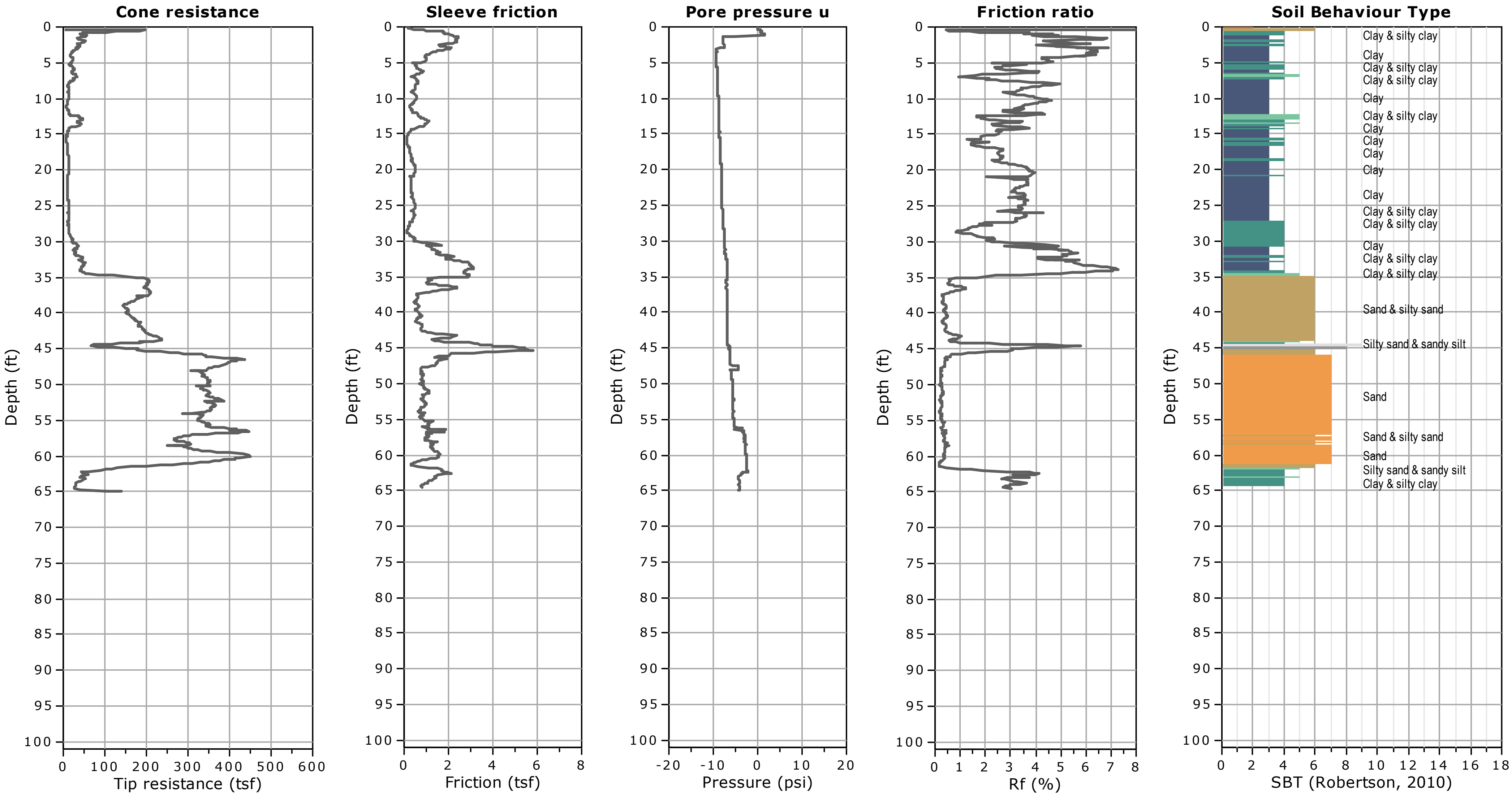
Project: **NMG Geotechnical - Segerstrom / Lake Center**
Location: **3120 W. Lake Center Dr, Santa Ana, CA**

CPT-5
Total depth: 64.93 ft, Date: 2/26/2024



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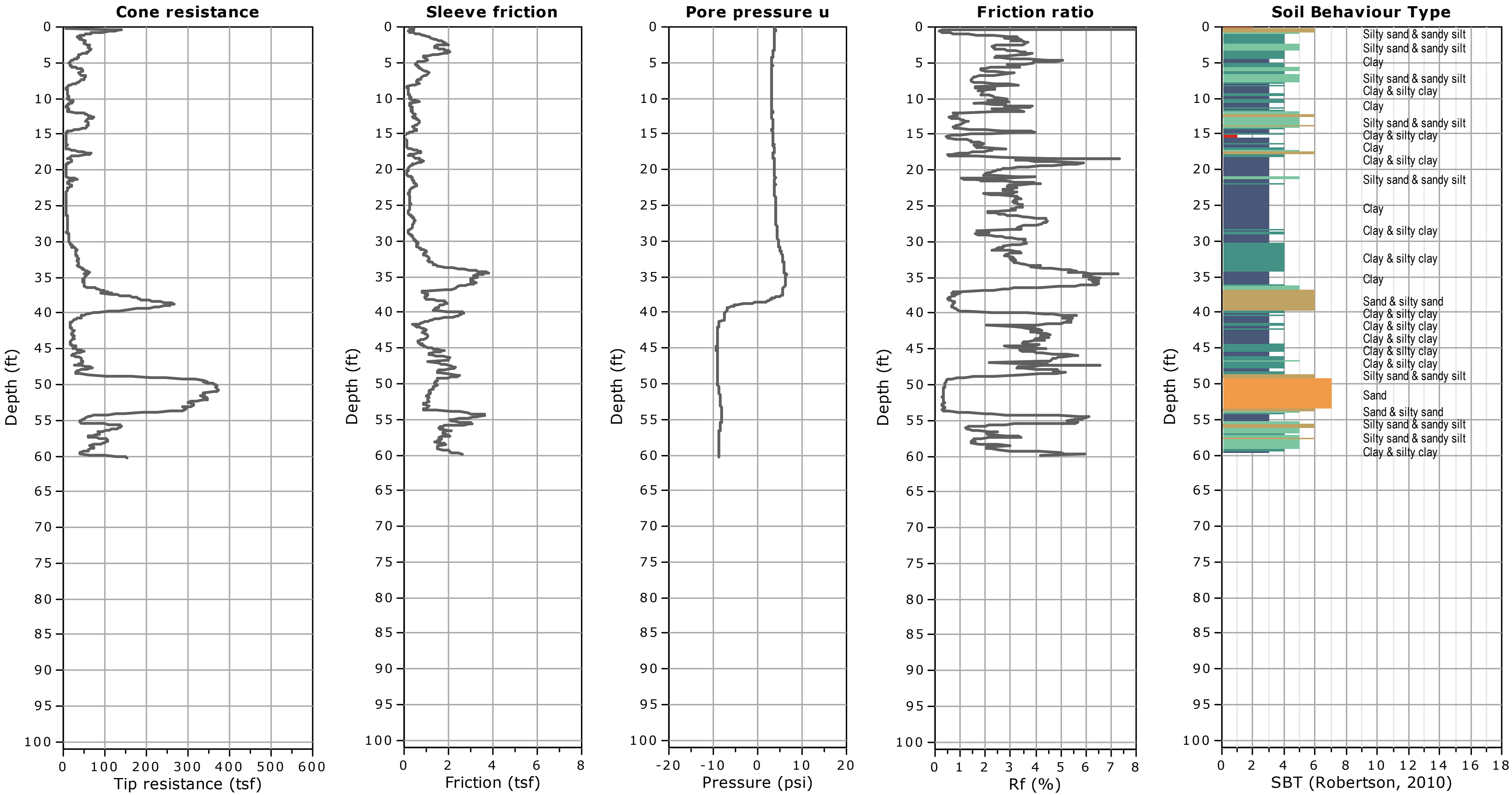
Project: NMG Geotechnical - Segerstrom / Lake Center
Location: 3120 W. Lake Center Dr, Santa Ana, CA

CPT-6
Total depth: 60.26 ft, Date: 2/26/2024



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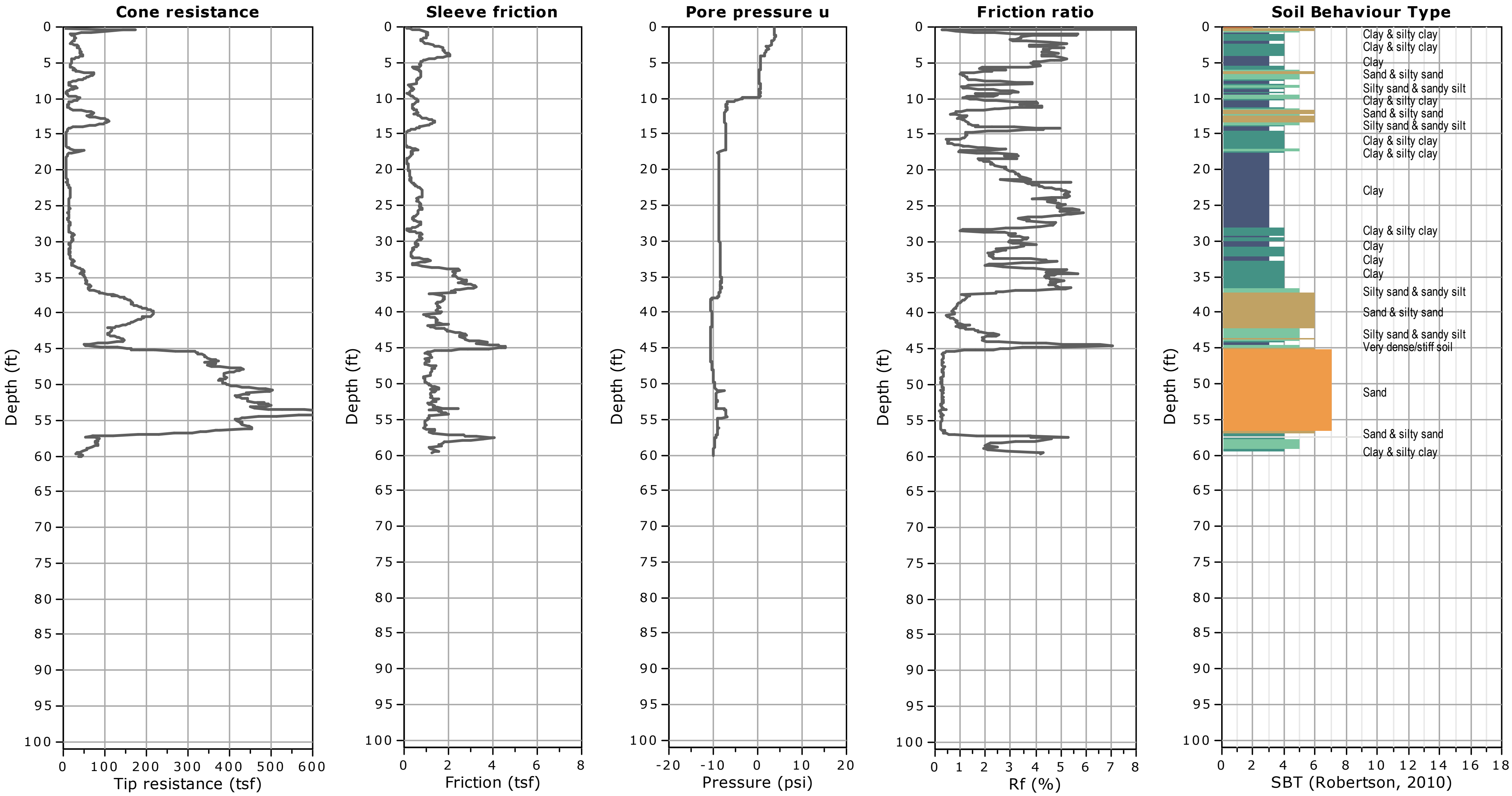
Project: NMG Geotechnical - Segerstrom / Lake Center

Location: 3120 W. Lake Center Dr, Santa Ana, CA

CPT-7

Total depth: 60.13 ft, Date: 2/26/2024



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3120 W. Lake Center Dr.
Santa Ana, CA

CPT Shear Wave Measurements

Location	Tip Depth (ft)	Geophone Depth (ft)	Travel Distance (ft)	S-Wave Arrival (msec)	S-Wave Velocity from Surface (ft/sec)	Interval S-Wave Velocity (ft/sec)
CPT-2	10.04	9.04	9.26	13.04	710	
	20.08	19.08	19.18	33.16	579	493
	30.12	29.12	29.19	51.36	568	550
	40.09	39.09	39.14	62.32	628	908
	50.07	49.07	49.11	73.44	669	897
	60.04	59.04	59.07	84.80	697	877
	70.05	69.05	69.08	94.64	730	1017
	80.09	79.09	79.12	103.50	764	1133
	90.16	89.16	89.18	113.08	789	1051
	100.13	99.13	99.15	122.14	812	1100

Shear Wave Source Offset - 2 ft

S-Wave Velocity from Surface = Travel Distance/S-Wave Arrival
Interval S-Wave Velocity = (Travel Dist2-Travel Dist1)/(Time2-Time1)

APPENDIX C

Segerstrom / Lake Center Office Park
Project Number: 23111-01

APPENDIX C
SUMMARY OF SOIL LABORATORY DATA

Santa Ana, CA

Boring/Sample Information: Boring No.	Sample No.	Depth (feet)	End Depth (feet)	Elevation (feet)	Blow Count (N)	Field Wet Density (pcf)	Field Dry Density (pcf)	Field Moisture Content (%)	Degree of Sat. (%)	Sieve/Hydrometer: Fines Content (% pass. #200)	Clay Content (% pass. 2µ)	Atterberg Limits: LL (%)	PI (%)	USCS Group Symbol	Direct Shear Ultimate: Cohesion (psf)	Friction Angle (°)	Direct Shear Peak: Cohesion (psf)	Friction Angle (°)	Compaction: Maximum Dry Density (pcf)	Optimum Moisture Content (%)	Expansion Index	R-Value	Soluble Sulfate Content (% by wt)	Remarks
H-1	D-1	2.5		35.5	5	124.4	105.5	17.8	80.7															
H-1	D-2	5.0		33.0	22	117.5	104.6	12.3	54.4															
H-1	D-3	10.0		28.0	15	112.8	86.2	30.9	87.3															
H-1	D-4	15.0		23.0	10	115.6	87.5	32.2	93.9															
H-1	D-5	20.0		18.0	10	113.0	83.5	35.3	93.6															
H-1	D-6	25.0		13.0	9	112.9	81.5	38.6	97.5															
H-1	D-7	30.0		8.0	26	138.0	122.0	13.2	93.2															
H-1	SPT-1	35.0		3.0	22			18.5																
H-1	SPT-2	40.0		-2.0	5			24.6																
H-1	SPT-3	45.0		-7.0	27			14.3																
H-1	SPT-4	50.0		-12.0	17			16.6																
H-2	D-1	2.5		36.5	21	129.5	112.3	15.4	82.8															
H-2	D-2	5.0		34.0	24	98.0	95.6	2.4	8.6															
H-2	D-3	10.0		29.0	10	115.4	84.6	36.4	99.0															
H-2	D-4	15.0		24.0	9	112.3	82.7	35.8	93.2															
H-2	D-5	20.0		19.0	7	116.9	87.9	32.9	97.0															
H-2	D-6	25.0		14.0	15	112.7	80.5	40.0	98.9															
H-3	D-1	2.5		32.7	25	103.4	95.1	8.8	30.7															
H-3	D-2	5.0		30.2	9	110.6	93.5	18.2	61.4															
H-3	D-3	10.0		25.2	8	115.5	86.0	34.3	96.5															
H-3	D-4	15.0		20.2	9	119.0	91.5	30.1	96.6															
H-3	D-5	20.0		15.2	11	112.6	80.5	39.8	98.5															
H-3	D-6	25.0		10.2	18	140.0	125.5	11.5	91.0															
H-3	D-7	30.0		5.2	37	136.4	119.1	14.5	94.6															
H-3	SPT-1	35.0		0.2	41			15.4																
H-3	SPT-2	40.0		-4.8	11			31.3																
H-3	SPT-3	45.0		-9.8	10			21.2																
H-3	SPT-4	50.0		-14.8	13			20.9																
H-4	D-1	2.5		35.5	13	121.0	98.8	22.5	86.1															
H-4	D-2	5.0		33.0	10	119.6	97.6	22.5	83.5															
H-4	D-3	10.0		28.0	10	111.1	80.4	38.2	94.2			52	24	CH	50	23	125	22.5						
H-4	D-4	15.0		23.0	8	109.8	78.2	40.4	94.6			69	38	CH										CN
H-4	D-5	20.0		18.0	14	115.0	84.9	35.6	97.5															
H-4	D-6	25.0		13.0	10	119.8	91.2	31.4	100.0															
H-5	D-1	2.5		35.0	20	125.2	103.7	20.8	89.7															

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Printed: 4/12/24; Template: SUM_SOIL_LAB_ALL; Proj ID: 23111-01.GPJ

Segerstrom / Lake Center Office Park
Project Number: 23111-01

APPENDIX C
SUMMARY OF SOIL LABORATORY DATA

Santa Ana, CA

Boring/Sample Information						Field Wet Density (pcf)	Field Dry Density (pcf)	Field Moisture Content (%)	Degree of Sat. (%)	Sieve/ Hydrometer		Atterberg Limits		USCS Group Symbol	Direct Shear				Compaction		Expansion Index	R-Value	Soluble Sulfate Content (% by wt)	Remarks
Boring No.	Sample No.	Depth (feet)	End Depth (feet)	Elevation (feet)	Blow Count (N)					Fines Content (% pass. #200)	Clay Content (% pass. 2μ)	LL (%)	PI (%)		Ultimate Cohesion (psf)	Ultimate Friction Angle (°)	Peak Cohesion (psf)	Peak Friction Angle (°)	Maximum Dry Density (pcf)	Optimum Moisture Content (%)				
H-5	D-2	5.0		32.5	13	120.8	96.9	24.7	90.1															
H-5	D-3	10.0		27.5	10	116.2	87.0	33.5	96.7															
H-5	D-4	15.0		22.5	6	117.6	85.1	38.1	100.0			46	25	CL										CN
H-5	D-5	20.0		17.5	13	114.2	82.2	38.9	100.0															
H-5	D-6	25.0		12.5	11	117.8	90.5	30.2	94.6															
H-5	D-7	30.0		7.5	18	132.9	114.8	15.7	91.0															
H-5	SPT-1	35.0		2.5	6			17.5																
H-5	SPT-2	40.0		-2.5	20			16.8																
H-5	SPT-3	45.0		-7.5	67			16.5																
H-5	SPT-4	50.0		-12.5	25																			NR
H-6	B-1	1.0	5.0	36.5						65	24	28	10	CL								26		CC
H-6	D-1	5.0		32.5	35	119.1	105.9	12.4	56.7															
H-6	D-2	10.0		27.5	11	118.3	93.2	27.0	90.1															
H-6	D-3	15.0		22.5	7	116.1	80.7	43.8	100.0			45	23	CL										CN
H-6	D-4	20.0		17.5	8	118.3	88.2	34.1	100.0															
H-6	D-5	25.0		12.5	7	118.3	89.4	32.4	98.8															
T-1	B-1	1.0	5.0					9.8													11			CC
T-1	B-2	7.0	8.0				92.8	14.4		75	9	NP	NP	ML	100	26	125	25.5	103.0	12.0				CN,CC
T-1	B-3	13.0	14.0					29.0		91		46	24	CL					115.0	15.5				CC
T-1A	sb-1	0.0	1.0																				0.05	CC
T-1B	sb-1	0.0	1.0																				0.05	CC
T-1C	sb-1	0.0	1.0																				0.05	CC
T-2	B-1	1.0	2.0					16.4																CC
T-4	B-1	2.0						16.8																
T-5	B-1	1.0						18.4																
T-5	B-2	3.0	4.0					18.1													59			
T-5		6.0	7.0					10.4														61		

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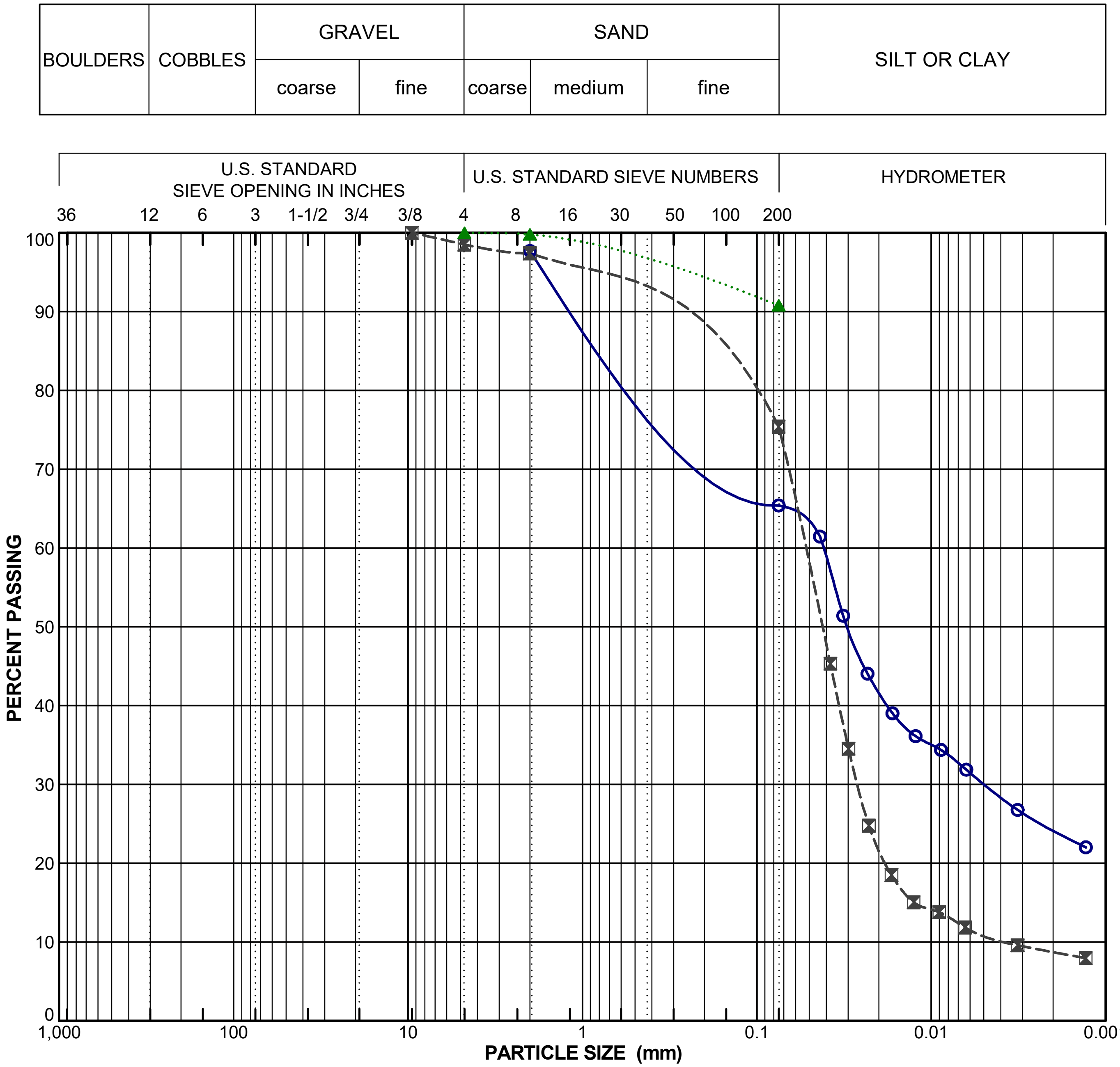

Symbol	Boring Number	Sample Number	Depth (feet)	Field Moisture (%)	LL	PI	Activity PI/-2µ	C_u	C_c	Passing No. 200 Sieve (%)	Passing 2µ (%)	USCS
○	H-6	B-1	1.0 - 5.0		28	10	0.42			65	24	CL
⊠	T-1	B-2	7.0 - 8.0	14	NP	NP				75	9	ML
▲	T-1	B-3	13.0 - 14.0	29	46	24				91		CL

PARTICLE SIZE DISTRIBUTION

Segerstrom / Lake Center Office Park
Santa Ana, CA
PROJECT NO. 23111-01

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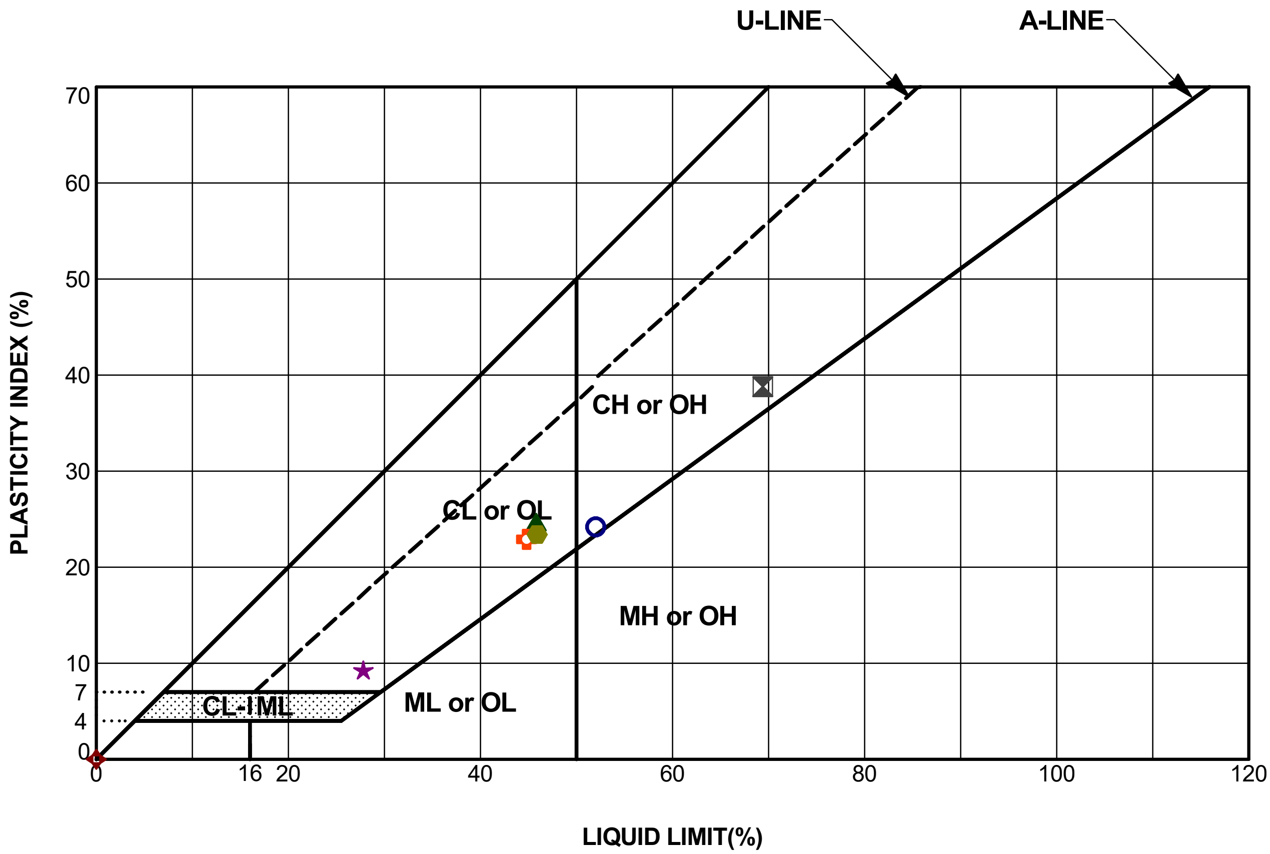
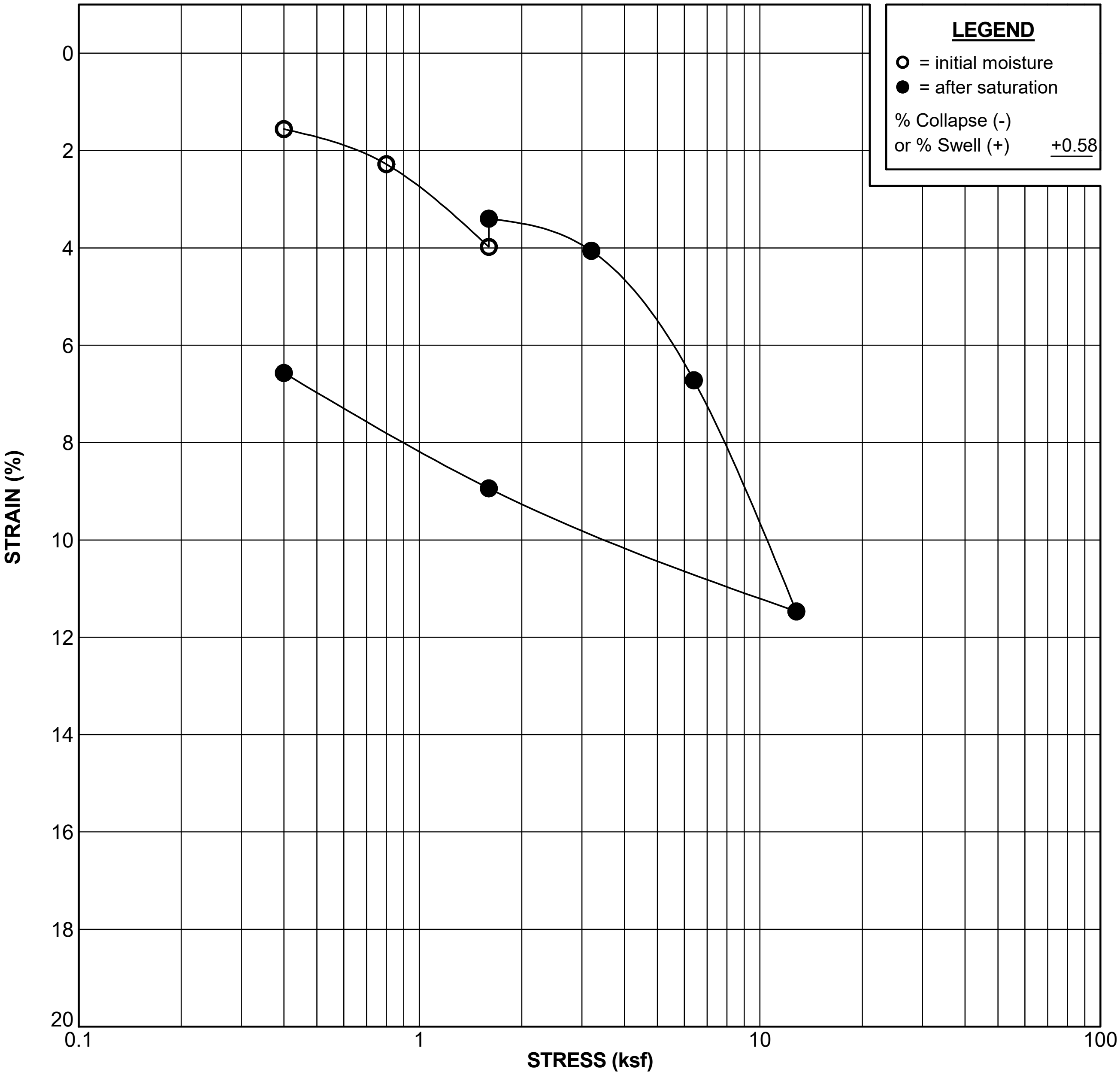


Symbol	Boring Number	Sample Number	Depth (feet)	Passing No. 200 Sieve (%)	LL	PI	USCS	Description
○	H-4	D-3	10.0		52	24	CH	(Qal) Very dark gray silty fat CLAY
⊠	H-4	D-4	15.0		69	38	CH	(Qal) Dark yellowish brown silty fat CLAY
▲	H-5	D-4	15.0		46	25	CL	(Qal) Dark grayish brown silty CLAY
★	H-6	B-1	1.0 - 5.0	65	28	10	CL	(Afu) Dark yellowish brown sandy CLAY
✪	H-6	D-3	15.0		45	23	CL	(Qal) Dark gray silty CLAY
◇	T-1	B-2	7.0 - 8.0	75	NP	NP	ML	(Qal) Pale gray sandy SILT
⬢	T-1	B-3	13.0 - 14.0	91	46	24	CL	(Qal) Gray silty CLAY

PLASTICITY CHART

Segerstrom / Lake Center Office Park
Santa Ana, CA
PROJECT NO. 23111-01

NMG Geotechnical, Inc.



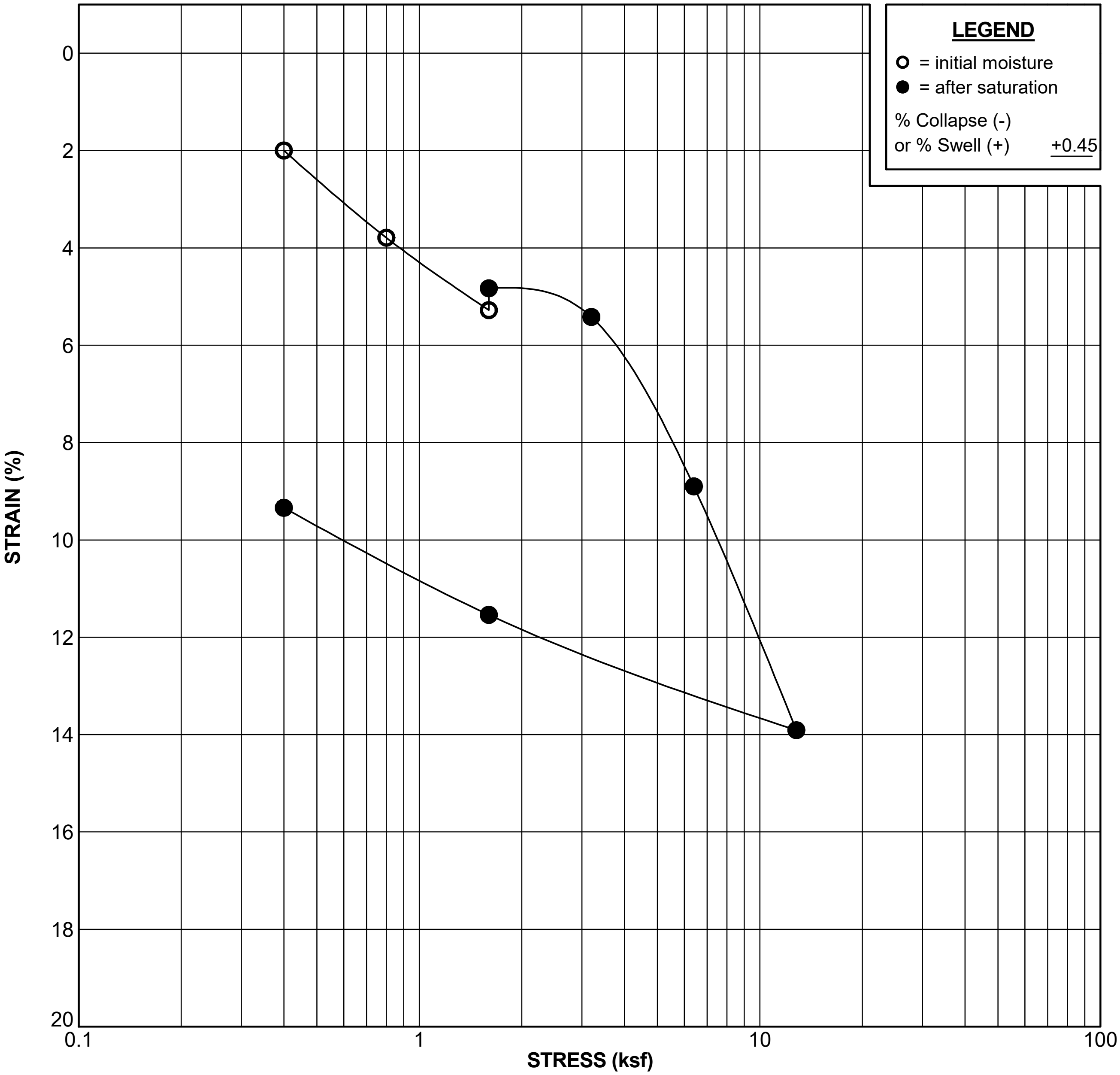
Boring No. H-4	Sample No. D-4	Depth: 15.0 ft
Sample Description: (Qal) Dark yellowish brown silty fat CLAY		**USCS:** CH
Liquid Limit: 69	**Plasticity Index:** 38	**Percent Passing No. 200 Sieve:**

Test Stage	Moisture Content (%)	Dry Density (pcf)	Degree of Saturation (%)	Void Ratio
Initial	42.5	77.1	92.5	1.315
Final	41.0	82.1	99.9	1.174

CONSOLIDATION TEST RESULTS

Segerstrom / Lake Center Office Park
Santa Ana, CA
PROJECT NO. 23111-01

NMG Geotechnical, Inc.



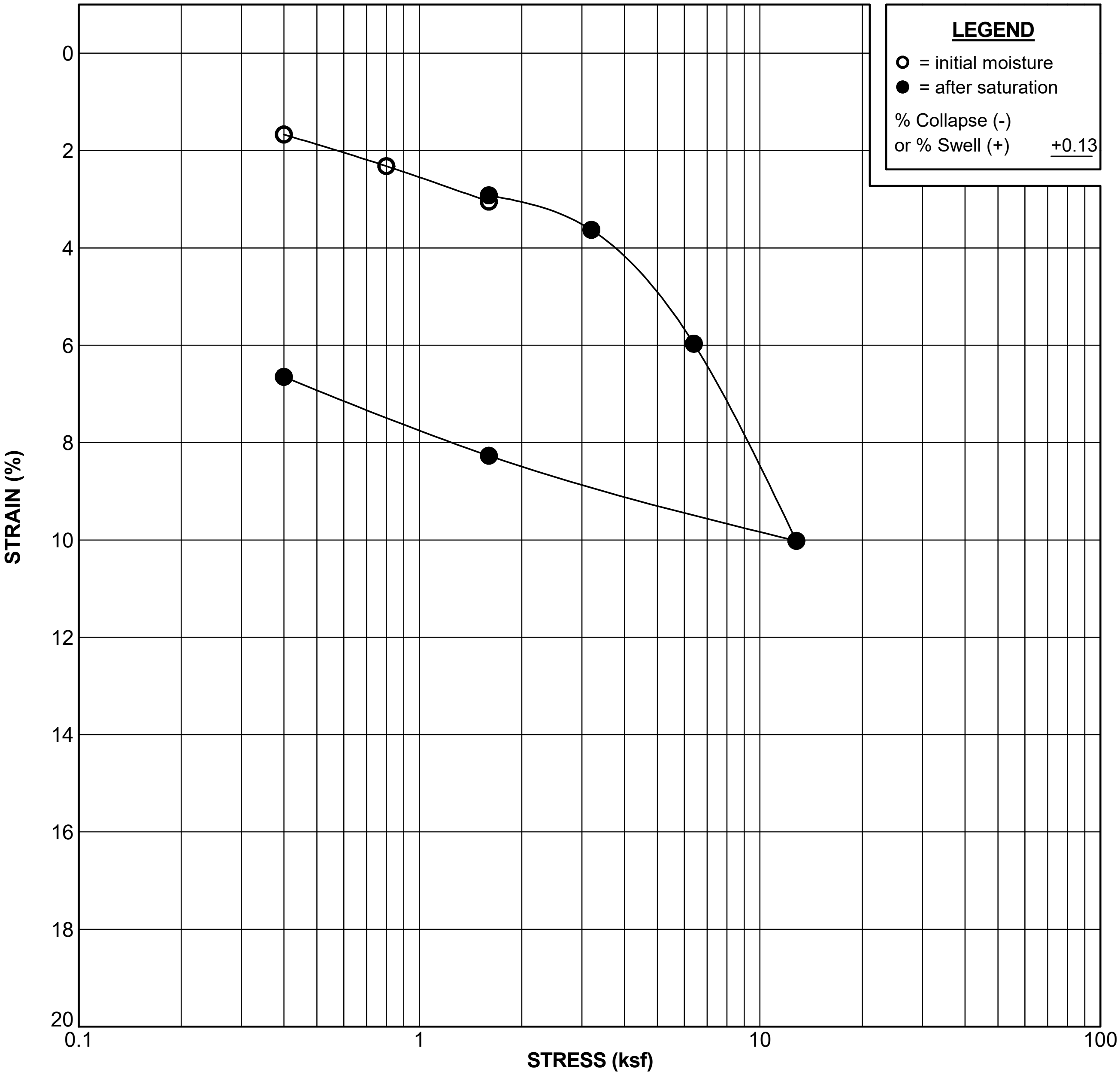
Boring No. H-5	Sample No. D-4	Depth: 15.0 ft
Sample Description: (Qal) Dark grayish brown silty CLAY		**USCS:** CL
Liquid Limit: 46	**Plasticity Index:** 25	**Percent Passing No. 200 Sieve:**

Test Stage	Moisture Content (%)	Dry Density (pcf)	Degree of Saturation (%)	Void Ratio
Initial	37.2	83.1	96.4	1.057
Final	32.0	90.9	99.5	0.881

CONSOLIDATION TEST RESULTS

Segerstrom / Lake Center Office Park
Santa Ana, CA
PROJECT NO. 23111-01

NMG Geotechnical, Inc.



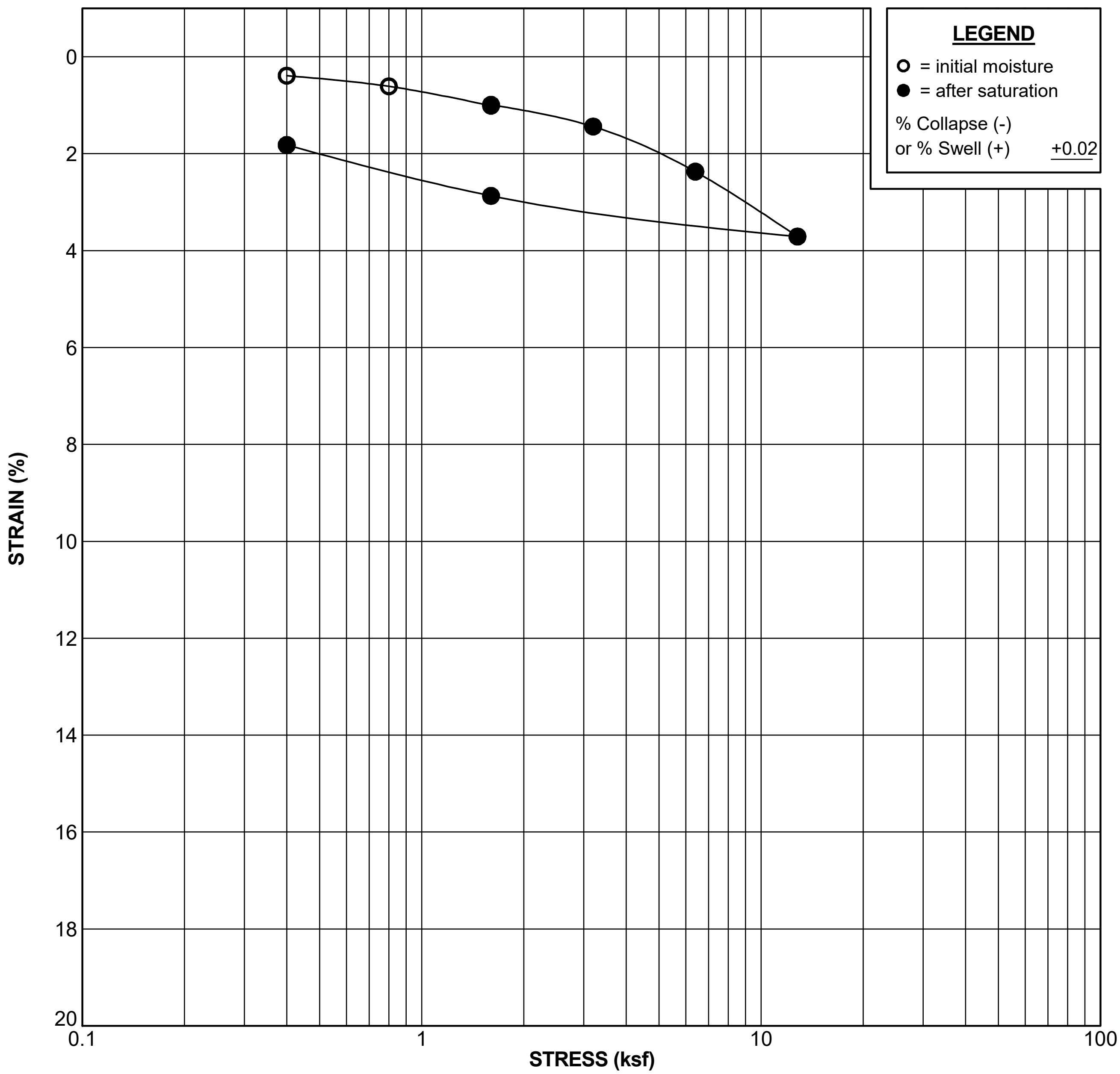
Boring No. H-6	Sample No. D-3	Depth: 15.0 ft
Sample Description: (Qal) Dark gray silty CLAY		**USCS:** CL
Liquid Limit: 45	**Plasticity Index:** 23	**Percent Passing No. 200 Sieve:**

Test Stage	Moisture Content (%)	Dry Density (pcf)	Degree of Saturation (%)	Void Ratio
Initial	35.4	85.7	98.2	0.980
Final	31.5	91.4	100.0	0.857

CONSOLIDATION TEST RESULTS

Segerstrom / Lake Center Office Park
Santa Ana, CA
PROJECT NO. 23111-01

NMG Geotechnical, Inc.



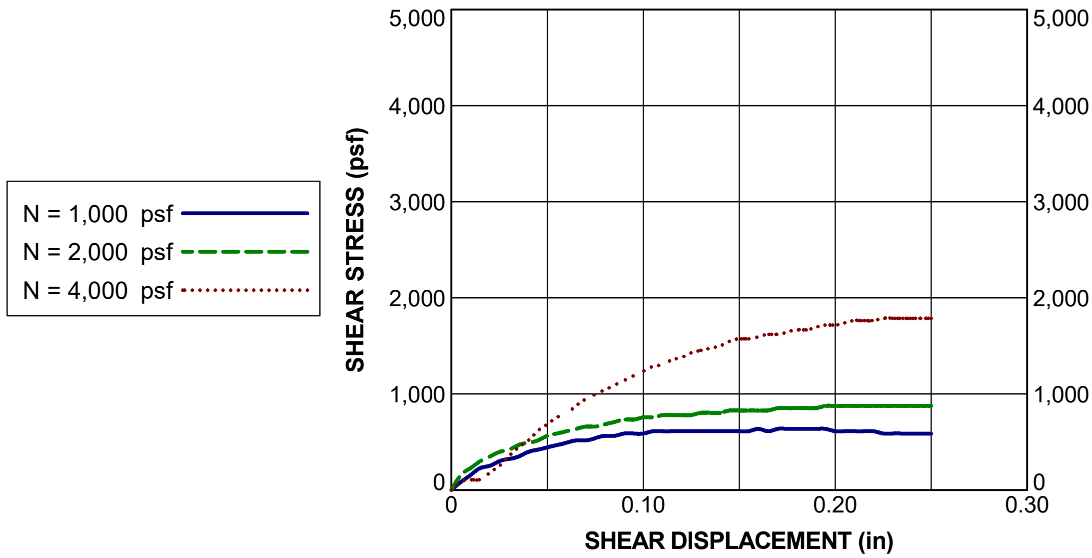
Boring No. T-1	**Sample No. B-2**	**Depth: 7.0 - 8.0 ft**	
Sample Description: (Qal) Pale gray sandy SILT		**USCS:** ML	
Liquid Limit: NP	**Plasticity Index:** NP	**Percent Passing No. 200 Sieve:**	75

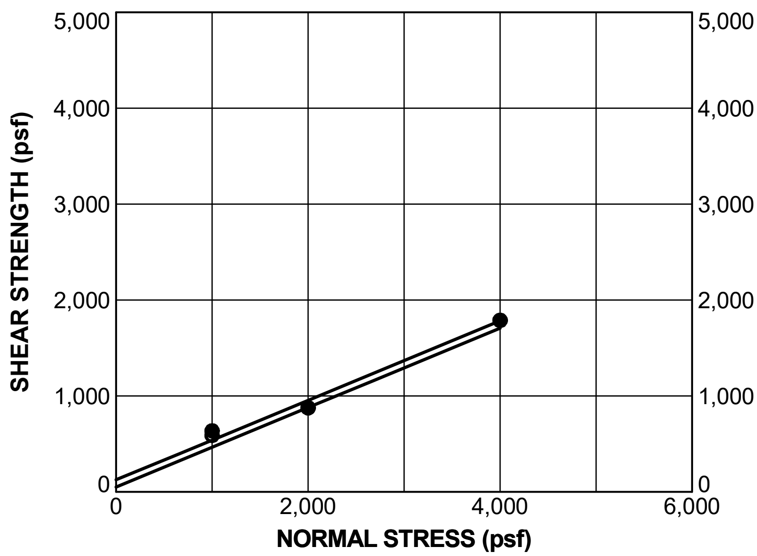
Test Stage	Moisture Content (%)	Dry Density (pcf)	Degree of Saturation (%)	Void Ratio
Initial	14.0	92.8	43.2	0.930
Final	29.9	94.5	95.9	0.895

CONSOLIDATION TEST RESULTS

Segerstrom / Lake Center Office Park
Santa Ana, CA
PROJECT NO. 23111-01

NMG Geotechnical, Inc.





Boring No. H-4	**Sample No. D-3**	**Depth: 10.0 ft**
Sample Description: (Qal) Very dark gray silty fat CLAY		**USCS:** CH
Liquid Limit: 52	**Plasticity Index:** 24	**Percent Passing No. 200 Sieve:**
Final Moisture Content (%): 46.5	**Final Dry Density (pcf):** 77.4	**Degree of Saturation (%):** 100
Sample Type: Undisturbed	**Rate of Shear (in./min.):** 0.001	

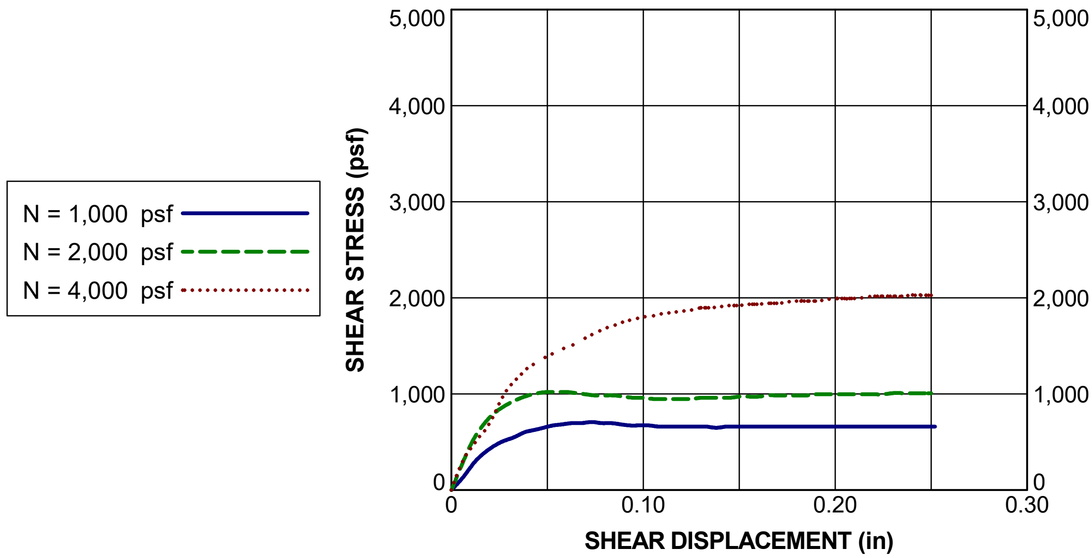
SHEAR STRENGTH PARAMETERS

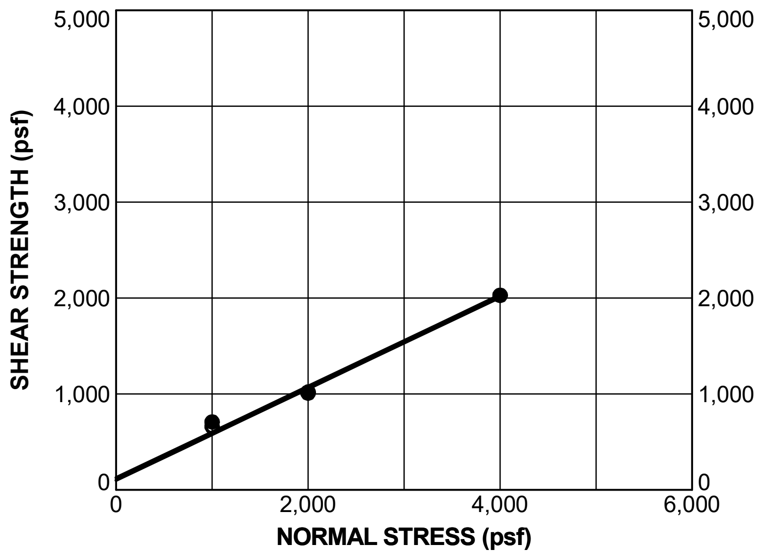
Parameter	Peak ●	Ultimate ○
Cohesion (psf)	125	50
Friction Angle (degrees)	22.5	22.5

DIRECT SHEAR TEST RESULTS

Segerstrom / Lake Center Office Park
Santa Ana, CA
PROJECT NO. 23111-01

NMG Geotechnical, Inc.





Boring No. T-1		**Sample No. B-2**		**Depth: 7.0 - 8.0 ft**	
Sample Description:	(Qal) Pale gray sandy SILT			**USCS:**	ML
Liquid Limit:	NP	**Plasticity Index:**	NP	**Percent Passing No. 200 Sieve:**	75
Final Moisture Content (%):	32.4	**Final Dry Density (pcf):**	92.8	**Degree of Saturation (%):**	100
Sample Type:	Remolded to 90% RC	**Rate of Shear (in./min.):**	0.005		

SHEAR STRENGTH PARAMETERS

Parameter	Peak ●	Ultimate ○
Cohesion (psf)	125	100
Friction Angle (degrees)	25.5	25.5

DIRECT SHEAR TEST RESULTS

Segerstrom / Lake Center Office Park
Santa Ana, CA
PROJECT NO. 23111-01

NMG Geotechnical, Inc.

Sample	Compacted Moisture (%)	Compacted Dry Density (pcf)	Final Moisture (%)	Volumetric Swell (%)	Expansion Index[1] Value/Method		Expansive Classification[2]	Soluble Sulfate (%)	Sulfate Exposure[3]
T-1 B-1 1-5'	9.0	114.1	15.2	1.09	11	A	Very Low	--	--
T-5 B-2 3-4'	11.0	166.7	21.9	5.86	59	A	Medium	--	--
T-1A sb-1 0-1'	--	--	--	--	--	--	--	0.05	S0
T-1B sb-1 0-1'	--	--	--	--	--	--	--	0.05	S0
T-1C sb-1 0-1'	--	--	--	--	--	--	--	0.05	S0

Test Method:

ASTM D4829

HACH SF-1 (Turbidimetric)

Notes:

1. Expansion Index (EI) method of determination*:*
   [A] E.I. determined by adjusting water content to achieve a 50 ±2% degree of saturation
   [B] E.I. calculated based on measured saturation within the range of 40% and 60%
2. ASTM D4829 *(Classification of Expansive Soil)*
3. ACI-318-14 Table 19.3.1.1 *(Requirement for Concrete Exposed to Sulfate-Containing Solutions)*

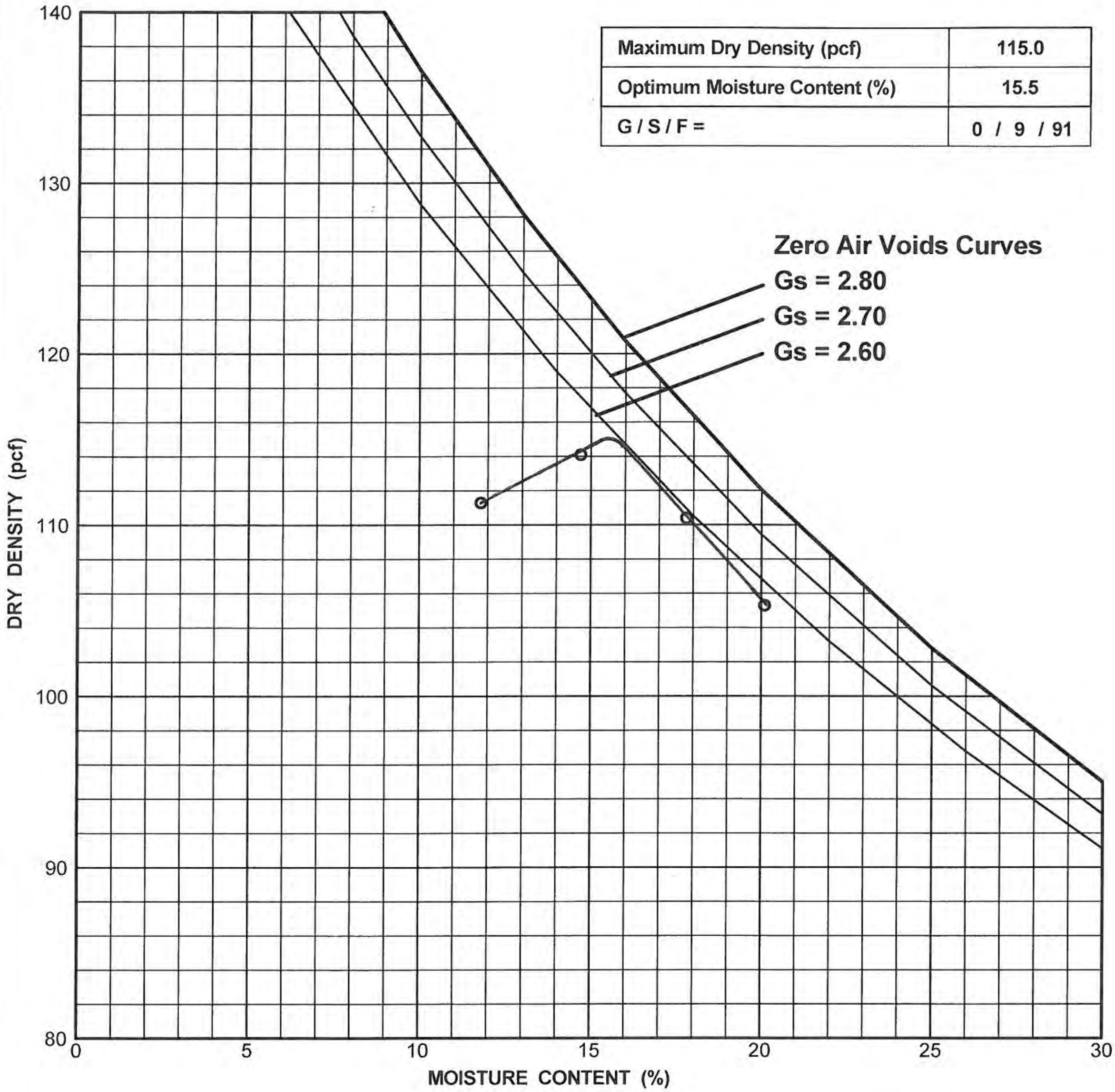
Expansion Index and Soluble Sulfate Test Results

(FRM001 Rev.5)

Project No. 23111-01

Project Name: Segerstrom / Lake Center Office Park

NMG



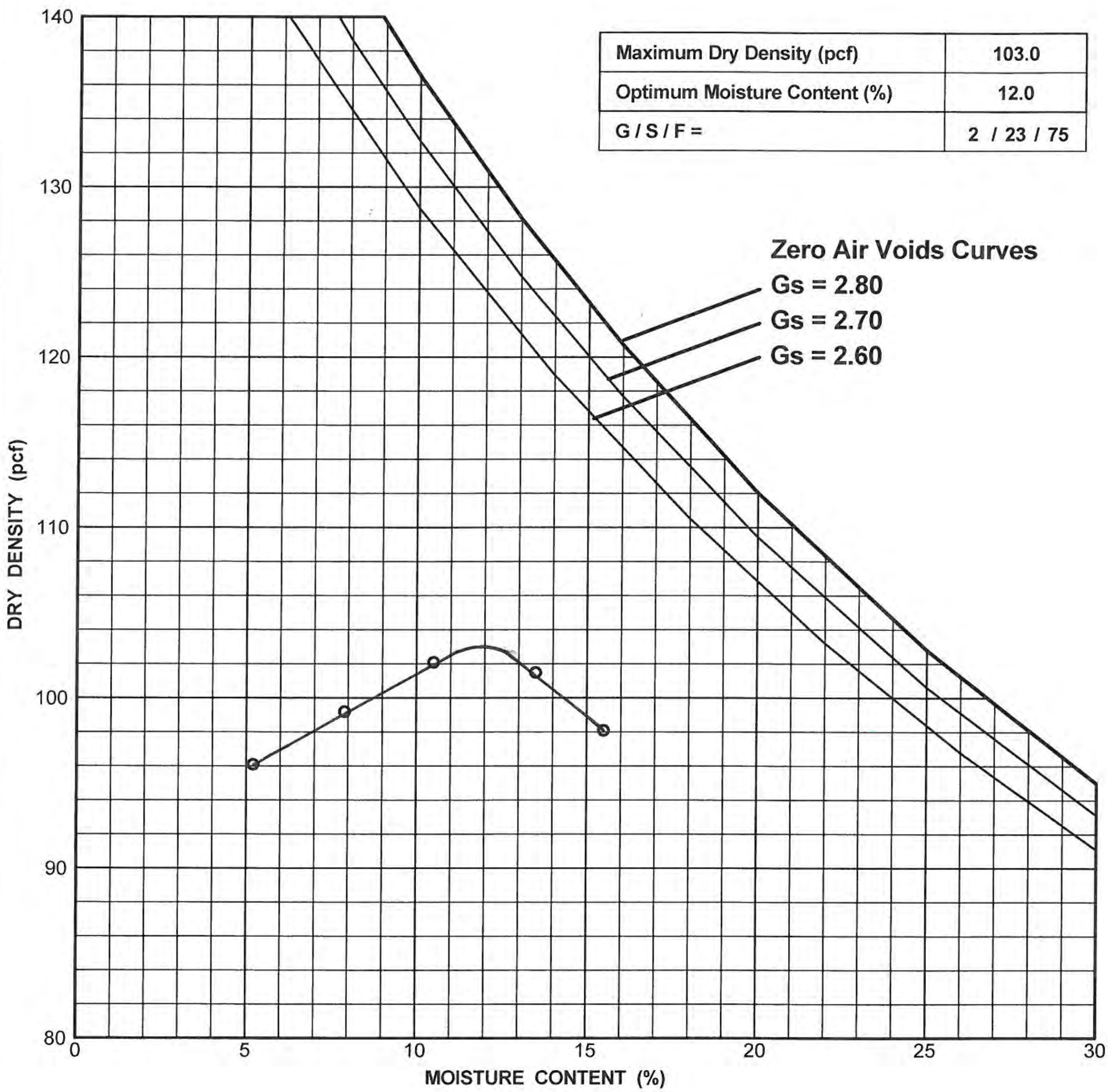
Maximum Dry Density (pcf)	115.0
Optimum Moisture Content (%)	15.5
G / S / F =	0 / 9 / 91

Boring No. T-1	**Sample No. B-3**	**Depth: 13.0 - 14.0 ft**
Sample Description: (Qal) Gray silty CLAY		**USCS:** CL
Liquid Limit: 46	**Plasticity Index:** 24	**Percent Passing No. 200 Sieve:** 91
Comments: 1557A		

COMPACTION TEST RESULTS

Segerstrom / Lake Center Office Park
Santa Ana, CA
PROJECT NO. 23111-01

NMG Geotechnical, Inc.



Maximum Dry Density (pcf)	103.0
Optimum Moisture Content (%)	12.0
G / S / F =	2 / 23 / 75

Boring No. T-1	**Sample No. B-2**		**Depth: 7.0 - 8.0 ft**
Sample Description: (Qal) Pale gray sandy SILT			**USCS:** ML
Liquid Limit: NP	**Plasticity Index:** NP		**Percent Passing No. 200 Sieve:** 75
Comments: 1557A			

COMPACTION TEST RESULTS

Segerstrom / Lake Center Office Park
Santa Ana, CA
PROJECT NO. 23111-01

NMG Geotechnical, Inc.

LaBelle Marvin

R - VALUE DATA SHEET

PROJECT No. 49883

DATE: 3/27/2024

BORING NO. T-5, B-3 @ 6'-7', Qal SM-ML)
Segerstrom Lake Center
P.N. 32111-01

SAMPLE DESCRIPTION: SM-ML Client Provided)

R-VALUE TESTING DATA | CA TEST 301			
	SPECIMEN ID		
	a	b	c
Mold ID Number	4	5	6
Water added, grams	66	71	77
Initial Test Water, %	17.8	18.4	19.1
Compact Gage Pressure,psi	350	350	350
Exudation Pressure, psi	455	310	161
Height Sample, Inches	2.59	2.54	2.58
Gross Weight Mold, grams	2933	2924	2945
Tare Weight Mold, grams	1949	1940	1951
Sample Wet Weight, grams	984	984	994
Expansion, Inches x 10exp-4	0	0	0
Stability 2,000 lbs (160psi)	20 / 36	21 / 38	22 / 41
Turns Displacement	5.00	5.05	5.10
R-Value Uncorrected	63	61	59
R-Value Corrected	65	61	61
Dry Density, pcf	97.7	99.2	98.0

DESIGN CALCULATION DATA

Traffic Index Assumed:	4.0	4.0	4.0
G.E. by Stability	0.36	0.40	0.40
G. E. by Expansion	0.00	0.00	0.00

Equilibrium R-Value	**61** by EXUDATION	Examined & Checked: 3 /27/ 24
REMARKS:	Gf = 1.25 0.0% Retained on the 3/4" Sieve. Baskets Used. Free Drainage.	REGISTERED PROFESSIONAL ENGINEER STEVEN R. MARVIN NO. 30659 CIVIL STATE OF CALIFORNIA DocuSigned by: D9A356504F9B448... Steven R. Marvin, RCE 30659

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.

LaBelle Marvin

R-VALUE GRAPHICAL PRESENTATION

PROJECT NO. 49883

DATE: 3 /27/ 24

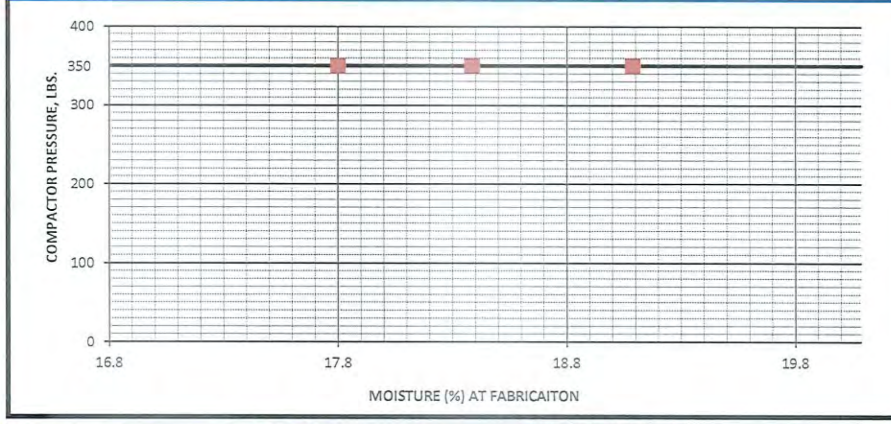
REMARKS:

BORING NO. T-5, B-3 @ 6'-7', Qal SM-ML)

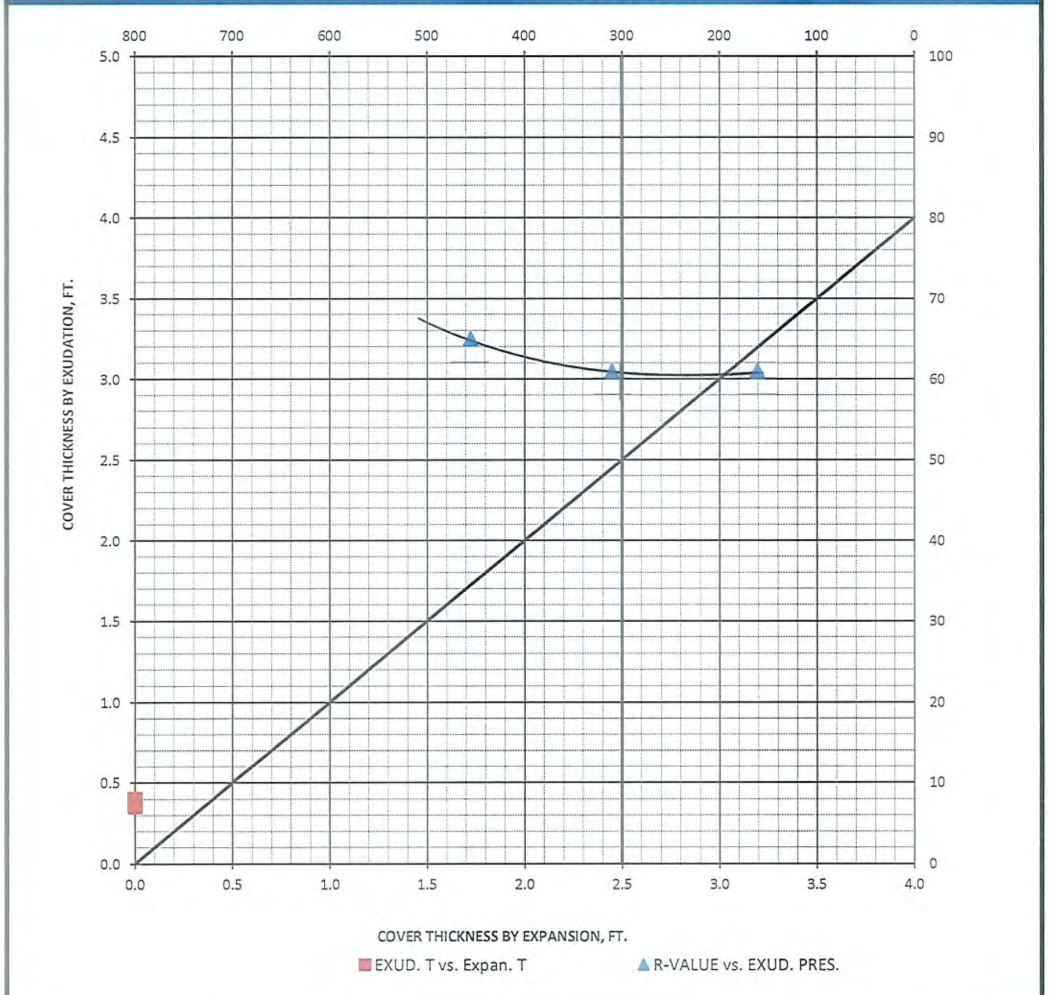
Segerstrom Lake Center

P.N. 32111-01

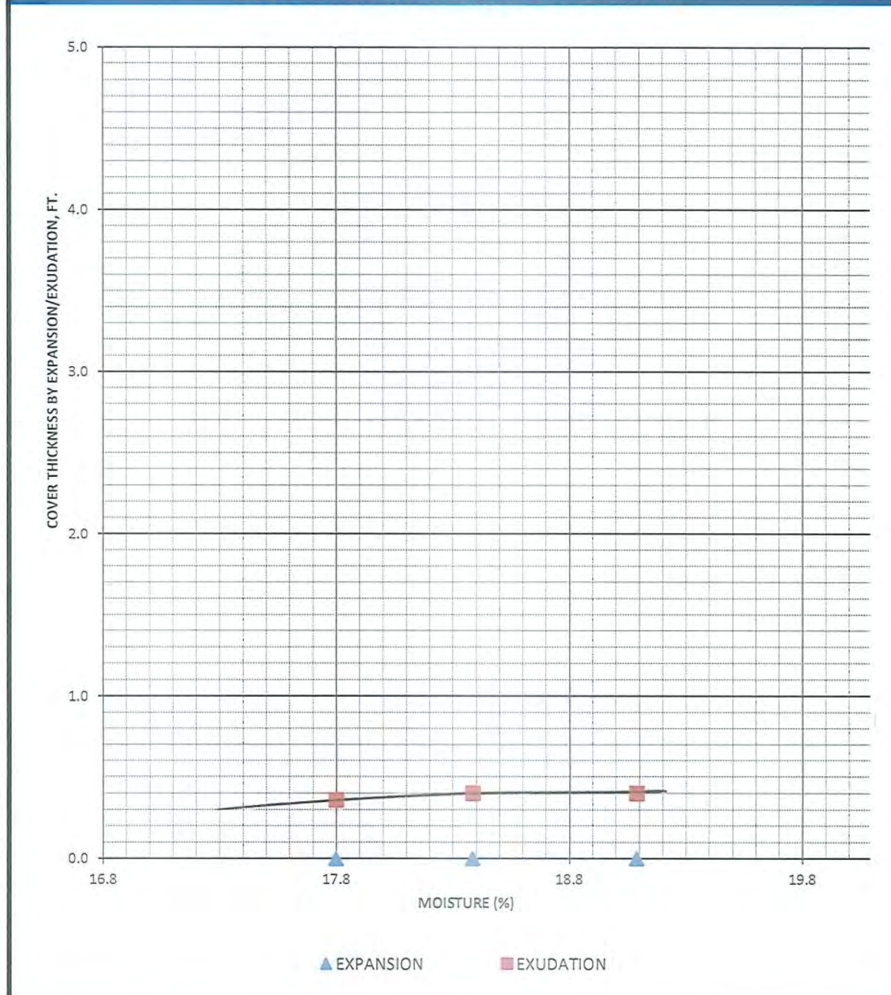
COMPACTOR PRESSURE vs MOISTURE %



COVER THICKNESS BY EXUDATION vs COVER THICKNESS BY EXPANSION



COVER THICKNESS vs MOISTURE %



LM
LaBelle Marvin

R - VALUE DATA SHEET

PROJECT No. 49883
DATE: 3/27/2024

BORING NO. H-6, B-1 @ 1'-5', Afe (CL)
Segerstrom Lake Center
P.N. 32111-01

SAMPLE DESCRIPTION: CL (Provided by Client)

R-VALUE TESTING DATA | CA TEST 301			
	SPECIMEN ID		
	a	b	c
Mold ID Number	1	2	3
Water added, grams	57	79	38
Initial Test Water, %	13.9	16.1	11.9
Compact Gage Pressure,psi	140	45	270
Exudation Pressure, psi	341	182	766
Height Sample, Inches	2.54	2.63	2.45
Gross Weight Mold, grams	3058	3070	3043
Tare Weight Mold, grams	1949	1942	1952
Sample Wet Weight, grams	1109	1128	1091
Expansion, Inches x 10exp-4	38	23	130
Stability 2,000 lbs (160psi)	30 / 66	40 / 93	23 / 50
Turns Displacement	4.89	5.55	4.24
R-Value Uncorrected	42	25	56
R-Value Corrected	42	27	56
Dry Density, pcf	116.2	111.9	120.6

DESIGN CALCULATION DATA

Traffic Index Assumed:	4.0	4.0	4.0
G.E. by Stability	0.59	0.75	0.45
G. E. by Expansion	1.27	0.77	4.33

Equilibrium R-Value	**26** by EXPANSION	Examined & Checked: 3 /27/ 24
REMARKS:	Gf = 1.25 0.4% Retained on the 3/4" Sieve.	REGISTERED PROFESSIONAL ENGINEER STEVEN R. MARVIN NO. 30659 CIVIL STATE OF CALIFORNIA DocuSigned by: D9A356504F9B448... Steven R. Marvin, RCE 30659

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.

LaBelle Marvin

R-VALUE GRAPHICAL PRESENTATION

PROJECT NO. 49883

DATE: 3 /27/ 24

REMARKS:

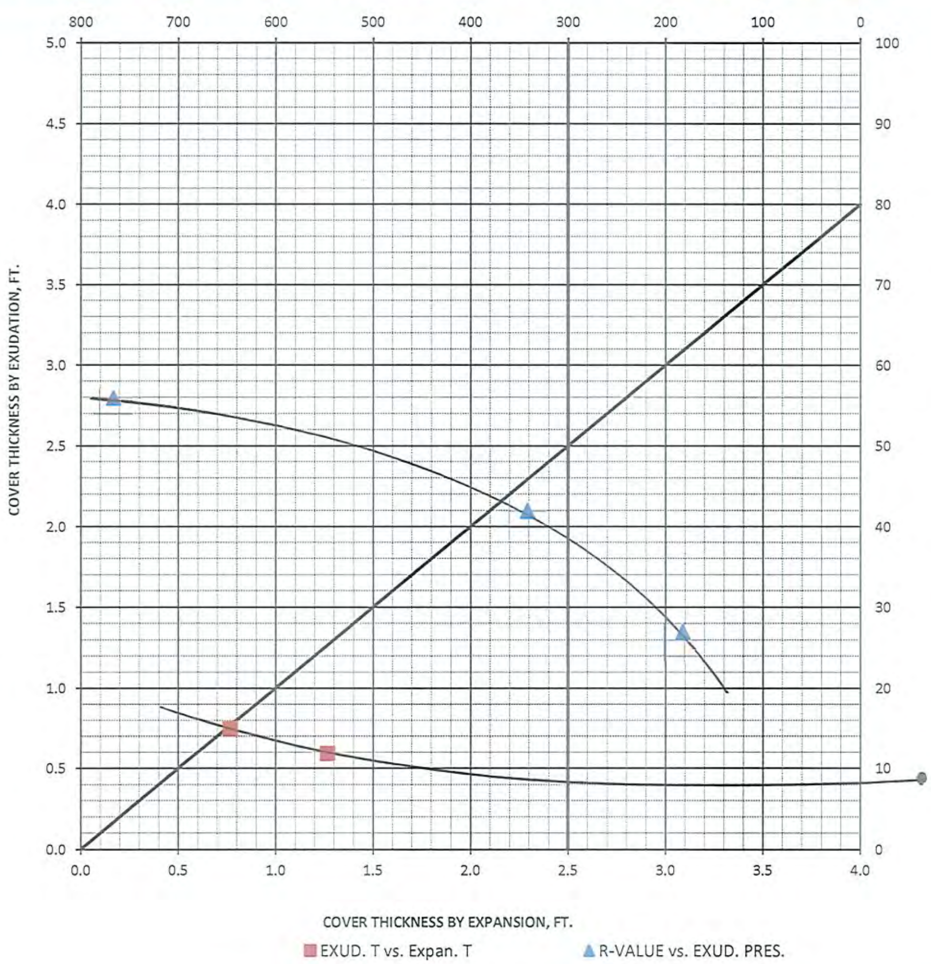
BORING NO. H-6, B-1 @ 1'-5', Afe (CL)

Segerstrom Lake Center

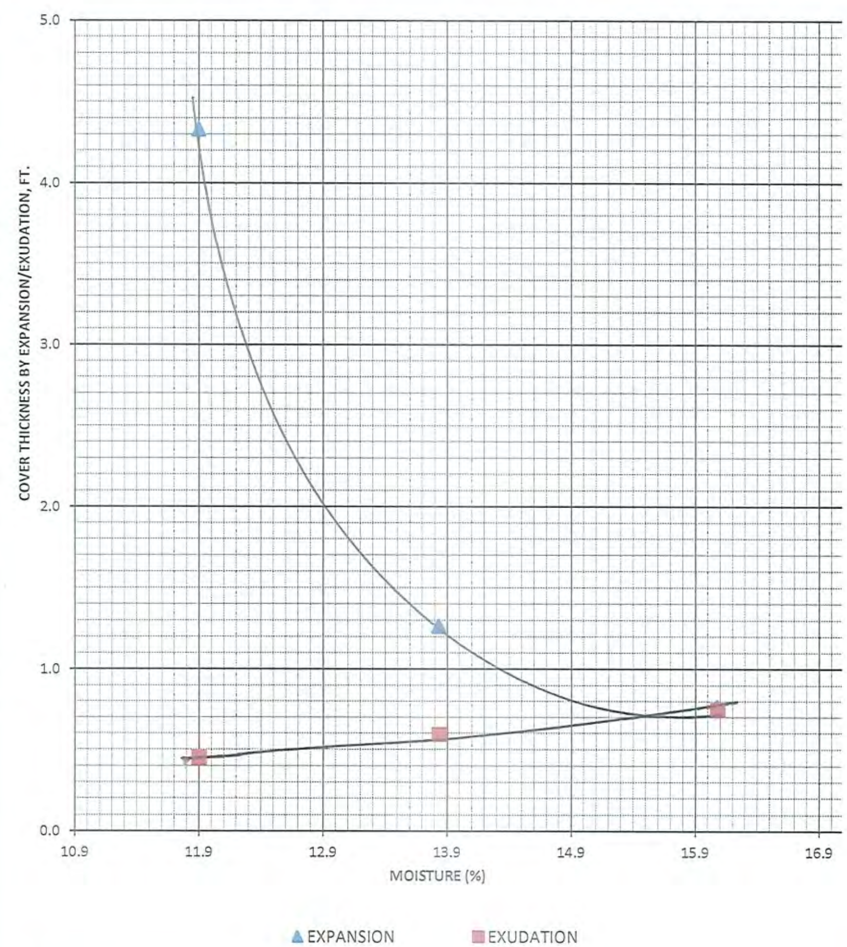
P.N. 32111-01

COMPACTOR PRESSURE vs MOISTURE %

COVER THICKNESS BY EXUDATION vs COVER THICKNESS BY EXPANSION



COVER THICKNESS vs MOISTURE %



Soil Corrosivity Evaluation Report for Segerstrom / Lake Center

April 11, 2024

Prepared for:

Bryan Jimenez
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17991 Fitch
Irvine, CA 92614
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Project X Job #: S240325J
Client Job or PO #: 23111-01

Contents

1 Executive Summary

A corrosion evaluation of the soils at Segerstrom / Lake Center was performed to provide corrosion control recommendations for general construction materials. The site is located at 300 Lake Center Drive, Santa Ana, CA . Eight (8) samples were tested to a depth of 13.0 ft. Site ground water and topography information was provided by NMG Geotechnical. Groundwater depth was determined to be 10 feet below finished grade.

Every material has its weakness. Aluminum alloys, galvanized/zinc coatings, and copper alloys do not survive well in very alkaline or very acidic pH environments. Copper and brasses do not survive well in high nitrate or ammonia environments. Steels and irons do not survive well in low soil resistivity and high chloride environments. High chloride environments can even overcome and attack steel encased in normally protective concrete. Concrete does not survive well in high sulfate environments. And nothing survives well in high sulfide and low redox potential environments with corrosive bacteria. This is why Project X tests for these 8 factors to determine a soil's corrosivity towards various construction materials. **Depending solely on soil resistivity or Caltrans corrosion guidelines (which concentrate on concrete/steel highways), will over-simplify descriptions as corrosive or non-corrosive. This approach will not detect these other factors attacking other metals because it is possible to have bad levels of corrosive ions and still have greater than 1,100 ohm-cm soil resistivity. We have observed this fact on thousands of soil samples tested in our laboratory.**

It should not be forgotten that import soil should also be tested for all factors to avoid making your site more corrosive than it was to begin with.

The recommendations outlined herein are not a substitute for any design documents previously prepared for the purpose of construction and apply only to the depth of samples collected.

Soil samples were tested for minimum resistivity, pH, chlorides, sulfates, ammonia, nitrates, sulfides and redox.

As-Received soil resistivities ranged between 1,474 ohm-cm and 28,810.0 ohm-cm. This data would be similar to a Wenner 4 pin test in the field and used in the design of a cathodic protection or grounding bed system. This resistivity can change seasonally depending on the weather and moisture in the ground. This is why minimum resistivity is more important for categorizing soil corrosivity. An as-received reading alone can be misleading because condensation or minor water leaks will occur underground along pipe surfaces creating a saturated soil environment in the trench on infrastructure surfaces. This is why minimum or saturated soil resistivity measurements are more important than as-received resistivities. This is also mentioned in AWWA C105 Appendix A "*The interpretation of the results of resistivity measurements is extremely important. A determination based on a four-pin reading with dry topsoil averaged with wetter subsoil would probably be inaccurate. Only by determining the resistivity in soil at pipe depth can an accurate interpretation be made. Also, the local situation should be determined concerning groundwater table, the presence of shallow groundwater, and the approximate percentage of time the soil is likely to be water saturated.*

In making field determinations of resistivity, temperature is important. Resistivity increases as the temperature decreases. As the water in the soil approaches freezing, resistivity increases greatly and, therefore, is not reliable. Field determinations under frozen soil conditions should be avoided. Reliable results under these conditions can be obtained only by the collection of suitable subsoil samples for analysis in laboratory conditions at a proper temperature.".

Saturated soil resistivities ranged between 804 ohm-cm to 2,010 ohm-cm. The worst of these values is considered to be severely corrosive to general metals.

PH levels ranged between 7.9 to 8.6 pH. PH levels were determined to be at levels not detrimental to copper or aluminum alloys. The pH of these samples can allow corrosion of steel and iron in moist environments.

Chlorides ranged between 18 mg/kg to 362 mg/kg. Chloride levels in these samples are enough to cause moderate corrosion in metals.

Sulfates ranged between 31 mg/kg to 506 mg/kg. Sulfate levels in these samples are negligible for corrosion of cement. Any type of cement can be used that does not contain encased metal.

Ammonia ranged between 2.4 mg/kg to 10.7 mg/kg. Nitrates ranged between 2.7 mg/kg to 115.2 mg/kg. Concentrations of these elements were high enough to cause accelerated corrosion of copper and copper alloys such as brass.

Sulfides presence was determined to be negative. REDOX ranged between + 133 mV to + 177 mV. The probability of corrosive bacteria was determined to be low due to the sulfide and positive REDOX levels determined in these samples.

Import soil should ideally have the following properties to avoid significant corrosion controls:

1. A minimum resistivity greater than 3,000 ohm-cm
2. Sulfates less than 1,000 mg/kg
3. Chlorides less than 300 mg/kg
4. pH between 6.5 and 8.5
5. Ammonia less than 10 mg/kg
6. Nitrates less than 50 mg/kg
7. Sulfides less than 1 mg/kg
8. REDOX potential greater than 100 mV.

2 Corrosion Control Recommendations

The following recommendations are based upon the results of soil testing.

2.1 Cement

The highest reading for sulfates was 506 mg/kg or 0.0506 percent by weight. One sample was found be higher but it was determined to be an anomaly after three soil samples surrounding that T-1 location were resampled and found to be low in sulfates.

Per ACI 318-14, Table 19.3.1.1, sulfate levels in these samples categorized as S0 and are negligible for corrosion of metals and cement. Per ACI 318-14 Table 19.3.2.1 any type of cement not containing steel or other metal can be used.

2.2 Steel Reinforced Cement/ Cement Mortar Lined & Coated (CML&C)

Chlorides in soil can overcome the corrosion inhibiting property of cement for steel, as it can also break through passivated surfaces of aluminum and stainless steels.[1,2] The highest concentration of chlorides was 362 mg/kg.

Chloride levels in these samples are enough to cause moderate corrosion of metals in soil or in cement. The following are the corrosion control options:

1) For any embedded steel/hardware extending or existing below 8 inches of finished floor (FF), 3 inches of Type II concrete cover or epoxy coating or powder coating or equivalent polymer coating is needed. #5 rebar or **Anchor bolts** with diameters of 5/8-inch or less require a minimum of 1.5 inches of cover per ACI 222.3R-5 Table 2.1, or
2) Prevent contact between cement and soil using impermeable waterproofing system assuring that water intrusion not occurs, or
3) Prevent contact between cement and soil using minimum 10 mil thick vapor barrier with joints overlapped at least 6 inches & taped, also sealing around plumbing & conduits. Barrier per ASTM E1745 installed in accordance with ASTM E1643. Class B barriers should be installed with capillary break layer, Class A barriers do not require capillary break layer. or,
4) Use 5,000 psi cement designed per ACI 318-14 Chapter 19 Table 19.3.1.1 per C2 category which exceeds the exposure class for this project, or
5) Since chlorides in these soil samples were not high enough to require DCI or special cement additives, additives such as DCI can be used in the slabs if 3 inches of cement cover cannot be achieved on steel items where coated hardware is not desired or possible.

As the cost of cement, epoxy or powder coated hardware, cement additives, and waterproofing systems seem to vary throughout the year and between contractors, we provide OPTIONS so that the most cost effect decision can be made.

Though soils at some locations are significantly corrosive to various metals, per ACI 318-14 Chapter 19 Table 19.3.1.1, all slabs on this site exposure categories and class for **Corrosion Protection of Reinforcement (C) would be considered C1** as Concrete exposed to moisture [mud/rain] (slab sides and bottom) but not to an external source of chlorides. Though there are chlorides in the soil, ACI 318's definition of "external source of chlorides" consists of deicing chemicals, salt, brackish water, seawater, or spray from these sources. The chloride levels in seawater are typically over 19,000 mg/L or 19,000 ppm.

When concrete is tested for water-soluble chloride ion content, the tests should be made at an age of 28 to 42 days. The limits in Per ACI 318-14 Table 5.3.2.1 are to be applied to chlorides

[1] Design Manual 303: Cement Cylinder Pipe. Ameron. p.65
[2] *Chapter 19, Table 1904.2.2(1), 2012 International Building Code*

contributed from the concrete ingredients, not those from the environment surrounding the concrete.[3]

2.3 Stainless Steel Pipe/Conduit/Fittings

Stainless steels derive their corrosion resistance from their chromium content and oxide layer which needs oxygen to regenerate if damaged. Thus stainless steel is not good for deep soil applications where oxygen levels are extremely low. Stainless steels should not be installed deeper than a plant root zone. Stainless steels typically have the same nobility as copper on the galvanic series and can be connected to copper. If stainless steel must be used, it must be backfilled with soil having greater than 10,000 ohm-cm resistivity and excellent drainage. 304 Stainless steel will also corrode if in contact with carbon materials such as activated carbon. Stainless steel welds should be pickled.

The soil at this site has low probability for anaerobic corrosive bacteria and moderate chloride levels. Per Nickel Institute guidelines, 316 Stainless steels should only be used in these soils.

2.4 Steel Post Tensioning Systems

The proper sealing of stressing holes is of utmost importance in PT Systems. Cut off excess strand 1/2" to 3/4" back in the hole. Coat or paint exposed anchorage, grippers, and stub of strands with "Rust-o-leum" or equal. After tendons have been coated, the cement contractor shall dry pack blockouts within ten (10) days. A non-shrink, non-metallic, non-porous moisture-insensitive grout (Master EMACO S 488 or equivalent), or epoxy grout shall be used for this purpose. If an encapsulated post-tension system is used, regular non-shrink grout can be used.

Due to the moderate chloride concentration measured on samples obtained from this site, post-tensioned slabs should be protected in accordance with soil considered normal (non-corrosive).[4,5] Additionally, add grease caps to the cut strand at live end anchors to provide protection against corrosion due to moderate chloride levels.

2.5 Steel Piles

Steel piles are most susceptible to corrosion in disturbed soil where oxygen is available. Further, a dissimilar environment corrosion cell would exist between the steel embedded in cement, such as pile caps and the steel in the soil. In the cell, the steel in the soil is the anode (corroding metal), and the steel in cement is the cathode (protected metal). This cell can be minimized by coating the part of the steel piles that will be embedded in cement to prevent contact with cement and reinforcing steel.

Piles driven into soils without disturbing soils will avoid oxygen introduction and low corrosion rates unless there is a probability for corrosive anaerobic bacteria. Galvanized steel's zinc

[3] ACI 381-14., BUILDING CODE REQUIREMENTS FOR STRUCTURAL CONCRETE (ACI 318-14) AND COMMENTARY (ACI 318R-14)

[4] *Standard Requirements for Design and Analysis of Shallow Post-Tensioned Concrete Foundations on Expansive Soils, PTI DC10.5-12,Table 4.1, pg 16*

[5] *Specification for Unbonded Single Strand Tendons. Post-tensioning Institute (PTI), Phoenix, AZ, 2000.*

coating can provide significant protection for driven piles. In corrosive soils in which normal zinc coatings are not enough, the life of piles can be extended by increasing zinc coating thickness, using sacrificial metal, or providing a combination of epoxy coatings and cathodic protection. Corrosion has been observed to be extremely localized even at and below underground water tables. Pit depths of this magnitude do not have an appreciable effect on the strength or useful life of piling structures because the reduction in pile cross section is not significant.[6] Pitting is of more importance to pipes transporting liquids or gases which should not be leaked into the ground.

The following recommendations are recommended to achieve desired life. We defer to structural engineers to use our estimated corrosion rates and to choose from the corrosion control options listed below.

1) Sacrificial metal by use of thicker piles per non-disturbed soil corrosion rates, or
2) Galvanized steel piles per non-disturbed soil corrosion rates, or
3) Combination of galvanized and sacrificial metal per non-disturbed soil corrosion rates, or
4) For no loss of metal, coat entire pile with abrasion resistant epoxy coating such as 3M Scotchkote 323, or PowercreteDD, or equivalent, or
5) Use high yield steel which will corrode at the same rate as mild steel but have greater yield strength and thus be able to suffer more material loss than mild steel.

2.5.1 *Expected Corrosion Rate of Steel and Zinc in disturbed soil*

In general, the corrosion rate of metals in soil depends on the electrical resistivity, the elemental composition, and the oxygen content of the soil. Soils can vary greatly from one acre to the next, especially at earthquake faults. The better a soil is for farming; the easier it will be for corrosion to take place. Expansive soils will also be considered disturbed simply because of their nature from dry to wet seasons.

In Melvin Romanoff's NBS Circular 579, the corrosion rates of carbon steels and various metals was studied over long term periods. Various metals were placed in various soil types to gather corrosion rate data of all metals in all soil types. Samples were collected and material loss measured over the course of 20 years in some sites. The following corrosion rates were estimated by comparing the worst results of soils tested with similar soils in Romanoff's studies and Highway Research Board's publications.[7] The corrosion rate of zinc in disturbed soils is determined per Romanoff studies and King Nomograph.[8]

Expected Corrosion Rate for Steel = 1.72 mils/year for one sided attack

Expected Corrosion Rate for Zinc = 0.67 mils/year for one sided attack.

Note: 1 mil = 0.001 inch

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[6] Melvin Romanoff, Corrosion of Steel Pilings in Soils, National Bureau of Standards Monograph 58, pg 20.

[7] Field test for Estimating Service Life of Corrugated Metal Culverts, J.L. Beaton, Proc. Highway Research Board, Vol 41, P. 255, 1962

[8] King, R.A. 1977, Corrosion Nomograph, TRRC Supplementary Report, British Corrosion Journal

In undisturbed soils, a corrosion rate of 1.00 mil/year for steel is expected with little change in the corrosion rate of zinc due to it's low nobility in the galvanic series.

Per CTM 643: Years to perforation of corrugated galvanized steel culverts

- 28.6 Years to Perforate 18 gage (0.052in) metal
- 37.1 Years to Perforation for a 16 gage metal culvert
- 45.7 Years to Perforation for a 14 gage metal culvert
- 62.9 Years to Perforation for a 12 gage metal culvert
- 80.0 Years to Perforation for a 10 gage metal culvert
- 97.2 Years to Perforation for a 8 gage metal culvert

2.5.2 Expected Corrosion Rate of Steel and Zinc in Undisturbed soil

Expected Corrosion Rate for Steel = 1.00 mils/year for one sided attack

Expected Corrosion Rate for Zinc = 0.67 mils/year for one sided attack.

Note: 1 mil = 0.001 inch

2.6 Steel Storage tanks

Underground fuel tanks must be constructed and protected in accordance with California Underground Storage Tank Regulations, CCR, Title 23, Division 3, Chapter 16. Metals should be protected with cathodic protection or isolated from backfill material with an epoxy coating.

2.7 Steel Pipelines

Though a site may not be corrosive in nature at the time of construction, **installation of corrosion test stations and electrical continuity joint bonding should be performed during construction** so that future corrosion inspections can be performed. If steel pipes with gasket joints or other possibly non-conductive type joints are installed, their joints should be bonded across by welding or pin brazing a #8 AWG copper strand bond cable. Electrical continuity is necessary for corrosion inspections and for cathodic protection.

Corrosion test stations should be installed every 1,000 feet of pipeline.

Test stations shall have two #8 HMWPE copper strand wire test leads welded or pin brazed to the underground pipe, brought up into the test station hand hole and marked CTS. Wires should be brought into test station hand hole at finished grade with 12 inches of wire coiled within test station.

At isolation joints and pipe casings, 4 wire test stations shall be installed using #8 HMWPE copper strand wire test leads. Use different color wires to distinguish which wires are bonded to one side of isolation joint or to casing. Wires should be brought into test station hand hole at finished grade with 12 inches of wire coiled within test station.

Prevent dissimilar metal corrosion cells per NACE SP0286:

1) Electrically isolate dissimilar metal connections
2) Electrically isolate dissimilar coatings (Epoxy vs CML&C) segments connections
3) Electrically isolate river crossing segments

4) Electrically isolate freeway crossing segments
5) Electrically isolate old existing pipelines from new pipelines
6) Electrically isolate aboveground and underground pipe segments with flange isolation joint kits per NACE SP0286 to avoid galvanic corrosion cells. **These are especially important for fire risers.**

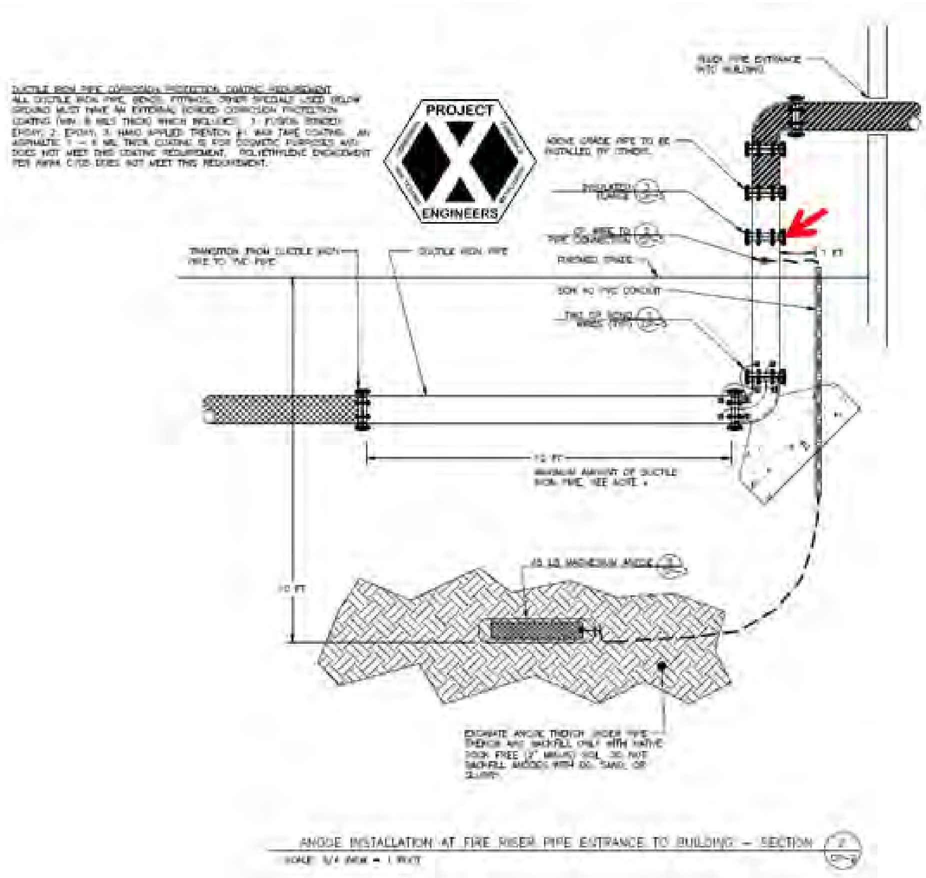

Figure 1- Fire Riser Detail: Install Isolation joint at red arrow

The corrosivity at this site is corrosive to steel. Any piping that must be jack-bored should use abrasion resistant epoxy coating such as 3M Scotchkote 323, or PowercreteDD, or equivalent. The corrosion control options for this site are as follows:

1) Apply impermeable dielectric coating such as minimum 10 mil thick polyethylene, and install cathodic protection system per NACE SP0169, or
2) Wax tape per AWWA C217, or
3) Coal tar enamel per AWWA C203, or
4) Fusion bonded epoxy per AWWA C213, or
5) For bare steel surfaces, such as welded pipe joints, apply 3 inch thick field coating of Type II cement or high pH slurry that will maintain pH higher than 12. Cement is both a corrosion inhibitor and a coating for ferrous metals. Cement naturally holds a pH of 12 or higher for many years if not exposed to high levels of carbon dioxide. (For CML&C

pipes, CML&C factory applied 3/4 inch thick coating is equivalent and needs no extra thickness added.)

It is critical for the life of the pipe that the protective wrap contains no openings or holes. Prevent damage to the protective sleeve during backfilling of the pipe trench. Penetrations of any kind within these or other protective materials generally leads to accelerated corrosion failure due to the fact that the corrosion attack is concentrated at the location of these penetrations. Cathodic protection will protect these defects. The better the coating, the less expensive a cathodic protection system will be in anode material and power requirement if needed.

2.8 Steel Fittings

The corrosivity at this site is corrosive to steel. The corrosion control options for this site are as follows:

1) Apply impermeable dielectric coating such as minimum 10 mil thick polyethylene, and install cathodic protection system per NACE SP0169, or
2) Tape coating system per AWWA C214, or
3) Wax tape all metallic surfaces per AWWA C217, or
4) Coal tar enamel per AWWA C203, or
5) Fusion bonded epoxy per AWWA C213
6) Apply 3 inch coating of Type II cement or high pH slurry that will maintain pH higher than 12. Cement is both a corrosion inhibitor and a coating for ferrous metals. Cement naturally holds a pH of 12 or higher for many years if not exposed to high levels of carbon dioxide.

It is critical for the life of the metal that the protective wrap contains no openings or holes. Prevent damage to the protective sleeve during backfilling of the pipe trench. Penetrations of any kind within these or other protective materials generally leads to accelerated corrosion failure due to the fact that the corrosion attack is concentrated at the location of these penetrations. Cathodic protection will protect these defects. The better the coating, the less expensive a cathodic protection system will be in anode material and power requirement if needed.

2.9 Ductile Iron (DI) & Cast Iron Fittings

AWWA C105 developed a 10 point system to classify sites as aggressive or non-aggressive to ductile iron materials. It is a tool to help in deciding whether or not to use polyethylene encasement [AWWA C105 Appendix A]. The 10-point system does not, and was never intended to; quantify the corrosivity of a soil. It is a tool used to distinguish nonaggressive from aggressive soils relative to iron pipe. Soils <10 points are considered nonaggressive to iron pipe, whereas soils ≥10 points are considered aggressive. A 15 and a 20 point soil are both considered aggressive to iron pipe, however, because of the nature of the soil parameters measured, the 20 point soil may not necessarily be more aggressive than the 15 point soil. The criterion is based upon soil resistivities, soil drainage, pH, sulfide presence, and reduction-oxidation (REDOX) potential. The soil samples tested for this site resulted in a score of 13 out of 25.5. A score greater or equal to 10 points classifies soils as aggressive to iron materials and would recommend the use of polyethylene encasement or other coating. The black coating on iron pipes is purely for aesthetic purposes and should not be relied upon for underground corrosion protection.[9]

The corrosivity at this site is corrosive to iron. The corrosion control options for this site are as follows:

1) Apply impermeable dielectric coating such as minimum 10 mil thick polyethylene, and install cathodic protection system per NACE SP0169, or
2) Wax tape all metallic surfaces per AWWA C217, or
3) Coal tar enamel per AWWA C203, or
4) Fusion bonded epoxy per AWWA C213
5) Apply standard concrete cover of Type II cement or high pH slurry that will maintain pH higher than 12. Cement is both a corrosion inhibitor and a coating for ferrous metals. Cement naturally holds a pH of 12 or higher for many years if not exposed to high levels of carbon dioxide.

It is critical for the life of the metal that the protective wrap contains no openings or holes. Prevent damage to the protective sleeve during backfilling of the pipe trench. Penetrations of any kind within these or other protective materials generally leads to accelerated corrosion failure due to the fact that the corrosion attack is concentrated at the location of these penetrations. Cathodic protection will protect these defects. The better the coating, the less expensive a cathodic protection system will be in anode material and power requirement if needed.

2.10 Ductile Iron & Cast Iron Pipe

AWWA C105 developed a 10 point system to classify sites as aggressive or non-aggressive to ductile iron materials. The 10-point system does not, and was never intended to, quantify the corrosivity of a soil. It is a tool used to distinguish nonaggressive from aggressive soils relative to iron pipe. Soils <10 points are considered nonaggressive to iron pipe, whereas soils ≥10 points are considered aggressive. A 15 and a 20 point soil are both considered aggressive to iron pipe, however, because of the nature of the soil parameters measured, the 20 point soil may not necessarily be more aggressive than the 15 point soil. The criterion is based upon soil resistivities, soil drainage, pH, sulfide presence, and reduction-oxidation (REDOX) potential. The soil samples tested for this site resulted in a score of 13 out of 25.5. A score greater or equal to 10 points classifies soils as aggressive to iron materials. The black coating on iron pipes is purely for aesthetic purposes and should not be relied upon for corrosion protection.[10]

Though a site may not be corrosive in nature at the time of construction, **<u>installation of corrosion test stations and electrical continuity joint bonding should be performed during construction</u>** so that future corrosion inspections can be performed. If steel pipes with gasket joints or other possibly non-conductive type joints are installed, their joints should be bonded

[9] https://www.dipra.org/ductile-iron-pipe-resources/frequently-asked-questions/corrosion-control

[10] https://www.dipra.org/ductile-iron-pipe-resources/frequently-asked-questions/corrosion-control

across by welding or pin brazing a #8 AWG copper strand bond cable. Electrical continuity is necessary for corrosion inspections and for cathodic protection. **If using thermite, perform one test bond using a half-charge then pressure test to confirm excess heat and pinholes were not created.**

Pea gravel is used by plumbers to lay pipes and establish slopes. If the gravel has more than 200 ppm chlorides or is not tested, a 25 mil plastic should be placed between the gravel and pipe to avoid corrosion.

Corrosion test stations should be installed every 1,000 feet of pipeline.

Test stations shall have two #8 HMWPE copper strand wire test leads welded or pin brazed to the underground pipe, brought up into the test station hand hole and marked CTS. Wires should be brought into test station hand hole at finished grade with 12 inches of wire coiled within test station.

At isolation joints and pipe casings, 4 wire test stations shall be installed using #8 HMWPE copper strand wire test leads. Use different color wires to distinguish which wires are bonded to one side of isolation joint or to casing. Wires should be brought into test station hand hole at finished grade with 12 inches of wire coiled within test station.

Prevent dissimilar metal corrosion cells per NACE SP0286:

1) Electrically isolate dissimilar metal connections
2) Electrically isolate dissimilar coatings (Epoxy vs CML&C) segments connections
3) Electrically isolate river crossing segments
4) Electrically isolate freeway crossing segments
5) Electrically isolate old existing pipelines from new pipelines
6) Electrically isolate aboveground and underground pipe segments with flange isolation joint kits per NACE SP0286**. These are especially important for fire risers.**

The corrosivity at this site is corrosive to iron. Any piping that must be jack-bored should use abrasion resistant epoxy coating such as 3M Scotchkote 323, or PowercreteDD, or equivalent. The corrosion control options for this site are as follows:

1) Apply impermeable dielectric coating such as minimum 10 mil thick polyethylene, and install cathodic protection system per NACE SP0169, or
2) Wax tape all metallic surfaces per AWWA C217, or
3) Coal tar enamel per AWWA C203, or
4) Fusion bonded epoxy per AWWA C213
5) Apply 3 inch coating of Type II cement or high pH slurry that will maintain pH higher than 12. Cement is both a corrosion inhibitor and a coating for ferrous metals. Cement naturally holds a pH of 12 or higher for many years if not exposed to high levels of carbon dioxide.

It is critical for the life of the metal that the protective wrap contains no openings or holes. Prevent damage to the protective sleeve during backfilling of the pipe trench. Penetrations of any kind within these or other protective materials generally leads to accelerated corrosion failure due to the fact that the corrosion attack is concentrated at the location of these penetrations. Cathodic protection will protect these defects. The better the coating, the less expensive a cathodic protection system will be in anode material and power requirement if needed.

2.11 Copper Materials

Copper is an amphoteric material which is susceptible to corrosion at very high and very low pH. It is one of the most noble metals used in construction thus typically making it a cathode when connected to dissimilar metals. Copper's nobility can change with temperature, similar to the phenomenon in zinc. When zinc is at room temperature, it is less noble than steel and can provide cathodic protection to steel. But when zinc is at a temperature above 140F such as in a water heater, it becomes more noble than the steel and the steel becomes the sacrificial anode. This is why zinc is not used in steel water heaters or boilers. Cold copper has one native potential, but when heated it develops a more electronegative electro-potential aka open circuit potential. Thus hot and cold copper pipes should be electrically isolated from each other to avoid creation of a thermo-galvanic corrosion cell.

2.11.1 __Copper Pipes__

The lowest pH for this area was measured to be 7.9. Copper is greatly affected by pH, ammonia and nitrate concentrations[11]. The highest nitrate concentration was 115.2 mg/kg and the highest ammonia concentration was 12.6 mg/kg at this site.

These soils were determined to be corrosive to copper and copper alloys such as brass.

Aboveground, underground, cold water and hot water pipes should be electrically isolated from each other by use of dielectric unions and plastic in-wall pipe supports per NACE SP0286. The following are corrosion control options for underground copper water pipes.

1) Run copper pipes within PVC pipes to prevent soil contact, or
2) Cover piping with a 20 mil epoxy coating, or 8-mil polyethylene sleeve, or encase in double 4-mil thick polyethylene sleeves free of scratches and defects then backfill with clean sand with 2 inch minimum cover above and below tubing. Backfill should have a pH between 6 and 8 with electrical resistivity greater than 2,000 ohm-cm
3) Cover copper pipes with minimum 8 mil polyethylene sleeve or incase in double 4-mil thick polyethylene sleeves over a suitable primer and apply cathodic protection per NACE SP0169

It is critical for the life of the metal that the protective wrap contains no openings or holes. Prevent damage to the protective sleeve during backfilling of the pipe trench. Penetrations of any kind within these or other protective materials generally leads to accelerated corrosion failure due to the fact that the corrosion attack is concentrated at the location of these penetrations. Cathodic protection will protect these defects. The better the coating, the less expensive a cathodic protection system will be in anode material and power requirement if needed.

[11] Corrosion Data Handbook, Table 6, Corrosion Resistance of copper alloys to various environments, 1995

2.11.2 *Brass Fittings*

Brass fittings should be electrically isolated from dissimilar metals by use of dielectric unions or isolation joint kits per NACE SP0286.

These soils were determined to be corrosive to copper and copper alloys such as brass.

The following are corrosion control options for underground brass.

1) Prevent soil contact by use of impermeable coating system such as wax tape, or
2) Prevent soil contact by use of a 20 mil epoxy coating free of scratches and defects and backfill with clean sand with 4 inch minimum cover above and below brass. Backfill should have a pH between 6 and 8 with electrical resistivity greater than 2,000 ohm-cm, or
3) Cover brass with minimum 10 mil polyethylene sleeve over a suitable primer and apply cathodic protection per NACE SP0169

It is critical for the life of the metal that the protective wrap contains no openings or holes. Prevent damage to the protective sleeve during backfilling of the pipe trench. Penetrations of any kind within these or other protective materials generally leads to accelerated corrosion failure due to the fact that the corrosion attack is concentrated at the location of these penetrations. Cathodic protection will protect these defects. The better the coating, the less expensive a cathodic protection system will be in anode material and power requirement if needed.

2.11.3 *Bare Copper Grounding Wire*

It is assumed that corrosion will occur at all sides of the bare wire, thus the corrosion rate is calculated as a two sided attack determining the time it takes for the corrosion from two sides to meet at the center of the wire. The estimated life of bare copper wire for this site is the following:[12]

Size (AWG)	**Diameter (mils)**	**Est. Time to penetration (Yrs)**
14	64.1	1068.3
13	72	1200.0
12	80.8	1346.7
11	90.7	1511.7
10	101.9	1698.3
9	114.4	1906.7
8	128.5	2141.7
7	144.3	2405.0
6	162	2700.0
5	181.9	3031.7
4	204.3	3405.0
3	229.4	3823.3
2	257.6	4293.3

[12] Soil-Corrosion studies 1946 and 1948: Copper Alloys, Lead, and Zinc, Melvin Romanoff, National Bureau of Standards, Research Paper RP2077, 1950

Size (AWG)	**Diameter (mils)**	**Est. Time to penetration (Yrs)**
1	289.3	4821.7

If the bare copper wire is being used as a grounding wire connected to less noble metals such as galvanized steel or carbon steel, the less noble metals will provide additional cathodic protection to the copper reducing the corrosion rate of the copper.

It is recommended that a corrosion inhibiting and water-repelling coating be applied to aboveground and belowground copper-to-dissimilar metal connections to reduce risk of dissimilar corrosion. This can be wax tape, or other epoxy coating.

Tinned copper wiring or laying copper wire in conductive concrete can protect against chemical attack in soils with high nitrates, ammonia, sulfide and severely low soil electrical resistivity.

2.12 Aluminum Pipe/Conduit/Fittings

Aluminum is an amphoteric material prone to pitting corrosion in environments that are very acidic or very alkaline or high in chlorides.

Conditions at this site are safe for aluminum.

Aluminum derives its corrosion resistance from its oxide layer which needs oxygen to regenerate if damaged, similar to stainless steels. Thus aluminum is not good for deep soil applications. Since aluminum corrodes at very alkaline environments, it cannot be encased or placed against cement or mortar such as brick wall mortar up against an aluminum window frame.

Aluminum is also very low on the galvanic series scale making it most likely to become a sacrificial anode when in contact with dissimilar metals in moist environments. Avoid electrical continuity with dissimilar metals by use of insulators, dielectric unions, or isolation joints per NACE SP0286. Pooling of water at post bottoms or surfaces should be avoided by integrating good drainage.

2.13 Carbon Fiber or Graphite Materials

Carbon fiber or other graphite materials are extremely noble on the galvanic series and should always be electrically isolated from dissimilar metals. They can conduct electricity and will create corrosion cells if placed in contact within a moist environment with any metal.

2.14 Plastic and Vitrified Clay Pipe

No special precautions are required for plastic and vitrified clay piping from a corrosion viewpoint.

Protect all metallic fittings and pipe restraining joints with wax tape per AWWA C217, cement if previously recommended, or epoxy.

3 CLOSURE

In addition to soils chemistry and resistivity, another contributing influence to the corrosion of buried metallic structures is stray electrical currents. These electrical currents flowing through the earth originate from buried electrical systems, grounding of electrical systems in residences, commercial buildings, and from high voltage overhead power grids. Therefore, it is imperative that the application of protective wraps and/or coatings and electrical isolation joints be properly applied and inspected.

It is the responsibility of the builder and/or contractor to closely monitor the installation of such materials requiring protection in order to assure that the protective wraps or coatings are not damaged.

The recommendations outlined herein are in conformance with current accepted standards of practice that meet or exceed the provisions of the Uniform Building Code (UBC), the International Building Code (IBC), California Building Code (CBC), the American Cement Institute (ACI), Nickel Institute, National Association of Corrosion Engineers (NACE International), Post-Tensioning Institute Guide Specifications and State of California Department of Transportation, Standard Specifications, American Water Works Association (AWWA) and the Ductile Iron Pipe Research Association (DIPRA).

Our services have been performed with the usual thoroughness and competence of the engineering profession. No other warranty or representation, either expressed or implied, is included or intended.

Please call if you have any questions.

Respectfully Submitted,

Ed Hernandez, M.Sc., P.E.
Sr. Corrosion Consultant
NACE Corrosion Technologist #16592
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4 SOIL ANALYSIS LAB RESULTS

Client: NMG Geotechnical
Job Name: Segerstrom / Lake Center
Client Job Number: 23111-01
Project X Job Number: S240325J
April 11, 2024

	Method	ASTM D4327		ASTM D4327		ASTM G187		ASTM G51	ASTM G200	SM 4500-D	ASTM D4327	ASTM D6919	ASTM D6919	ASTM D6919	ASTM D6919	ASTM D6919	ASTM D6919	ASTM D4327	ASTM D4327
Bore# / Description	Depth	Sulfates SO_4^{2-}		Chlorides Cl^-		Resistivity As Rec'd	Minimum	pH	Redox	Sulfide S^{2-}	Nitrate NO_3^-	Ammonium NH_4^+	Lithium Li^+	Sodium Na^+	Potassium K^+	Magnesium Mg^{2+}	Calcium Ca^{2+}	Fluoride F_2^-	Phosphate PO_4^{3-}
	(ft)	(mg/kg)	(wt%)	(mg/kg)	(wt%)	(Ω-cm)	(Ω-cm)		(mV)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)
T-1, B-1 SM-CL	1.0	2,459.4	0.2459	11.0	0.0011	1,005	871	7.7	167	ND	0.5	2.3	0.01	55.8	4.0	95.0	659.1	3.0	1.4
T-1, B-2 ML	7.0	173.5	0.0174	88.0	0.0088	5,628	1,943	8.0	141	0.1	115.2	2.4	ND	192.8	4.9	20.9	102.8	22.6	1.2
T-1, B-3 CH	13.0	110.5	0.0110	46.5	0.0047	2,345	1,005	8.5	164	0.2	101.5	3.6	ND	196.1	1.8	22.2	77.4	19.6	4.2
T-2, B-1 ML	1.0	76.2	0.0076	20.1	0.0020	7,370	2,010	8.6	162	1.4	2.7	6.3	ND	149.7	6.1	25.3	166.4	11.0	2.4
H-6, B-1 ML	0-5	505.7	0.0506	362.1	0.0362	28,810	804	8.0	177	0.1	4.8	10.7	0.02	266.4	6.2	31.8	120.6	8.8	10.7
T-1 A SB-1	0-1	59.2	0.0059	19.7	0.0020	2,077	1,407	8.3	140	1.7	7.8	3.8	0.04	95.7	7.1	25.4	140.1	10.9	8.5
T-1 C SB-1	0-1	30.7	0.0031	17.6	0.0018	1,474	1,273	8.1	133	1.2	6.1	10.7	0.02	61.8	4.7	20.9	136.1	11.6	7.3
T-1 B SB-1	0-1	118.6	0.0119	24.5	0.0024	2,111	1,340	7.9	139	0.2	10.4	3.2	0.02	63.9	6.6	20.9	123.6	8.1	7.7

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

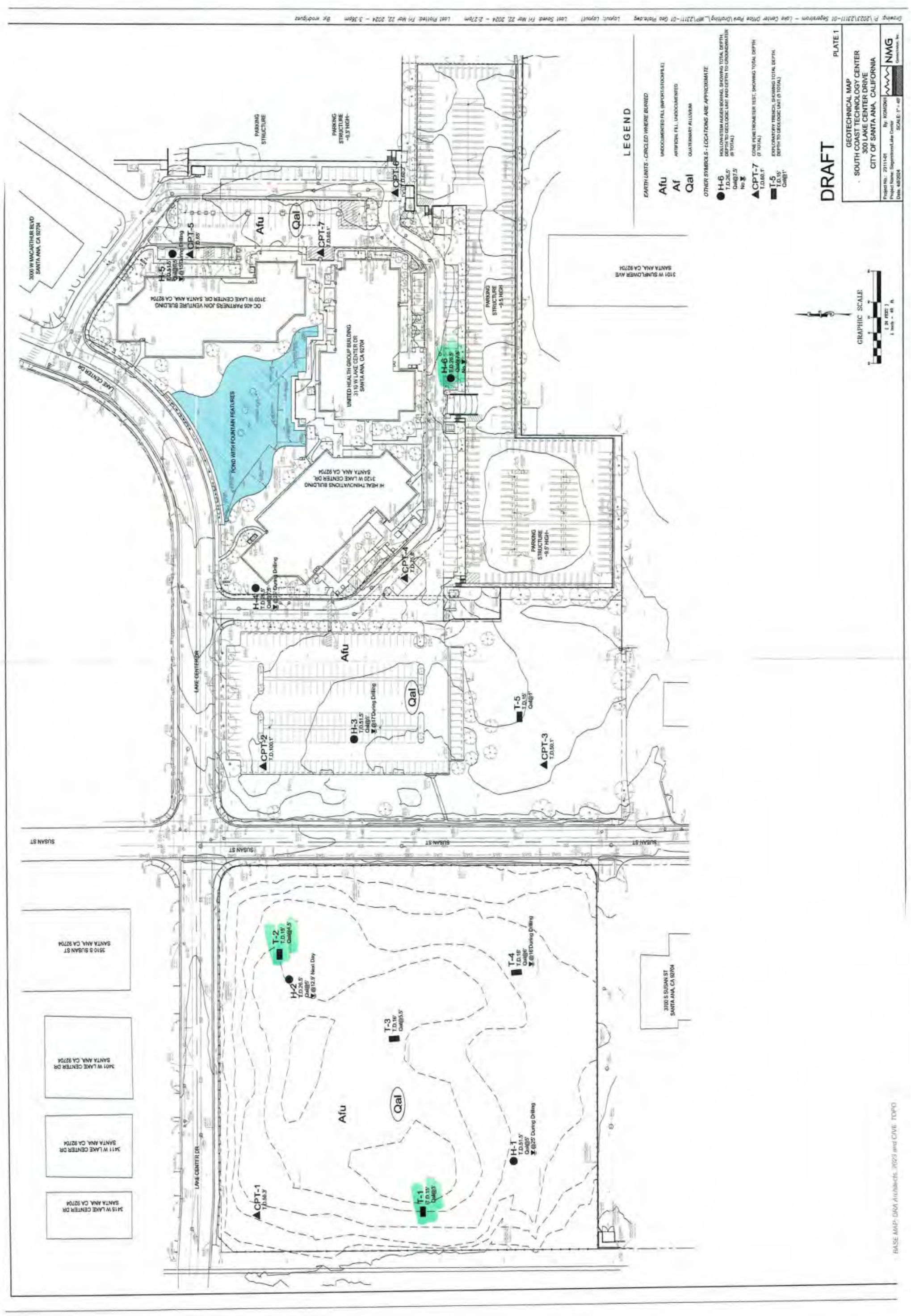


Figure 2- Soil Sample Locations, 300 Lake Center Drive, Santa Ana, CA

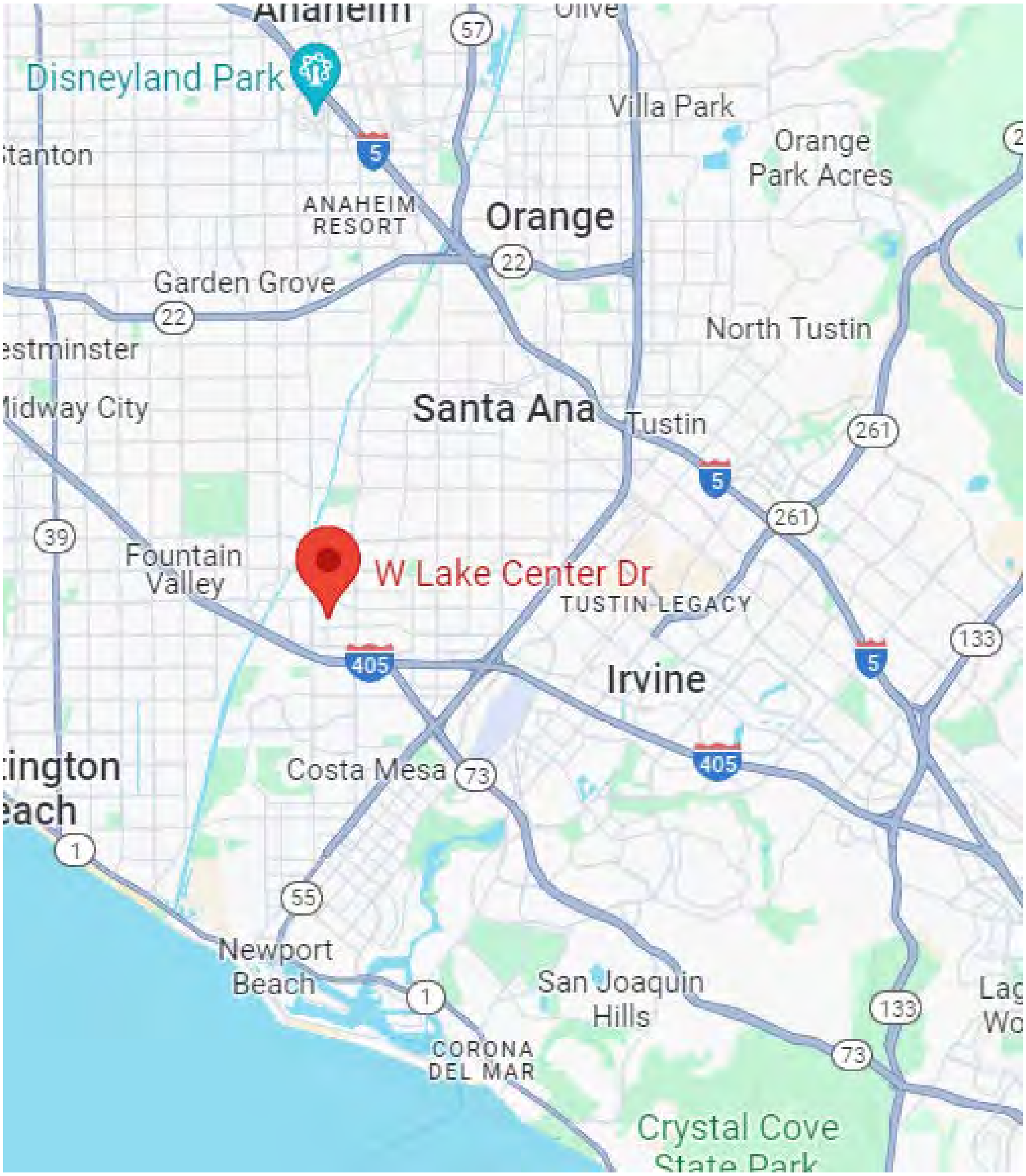


Figure 3- Vicinity Map, 300 Lake Center Drive, Santa Ana, CA

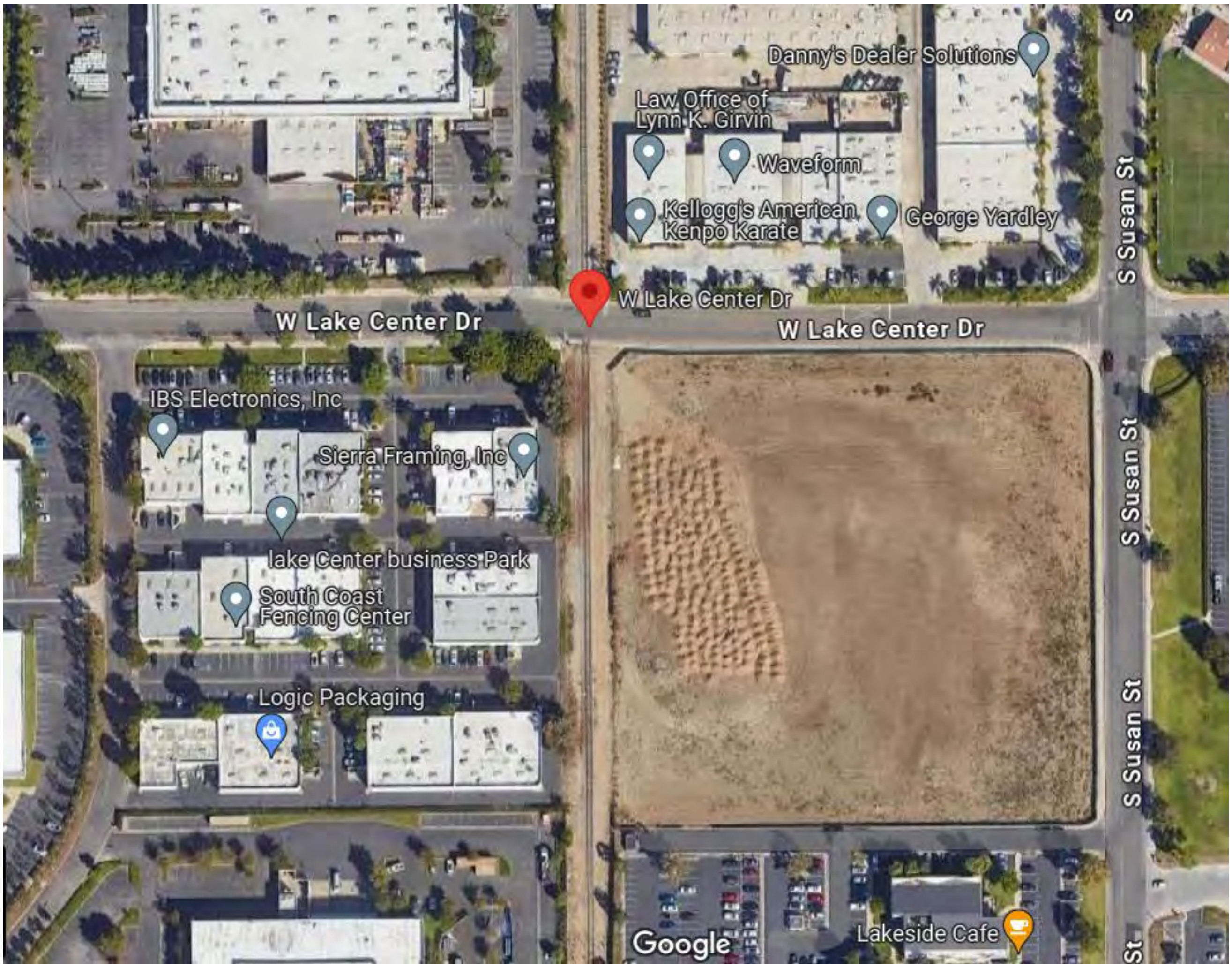


Figure 4- Satellite View

5 Corrosion Basics

In general, the corrosion rate of metals in soil depends on the electrical resistivity, the elemental composition, and the oxygen content of the soil. Soils can vary greatly from one acre to the next, especially at earthquake faults. The better a soil is for farming; the easier it will be for corrosion to take place. Expansive soils should be considered disturbed simply because of their nature from dry to wet seasons.

5.1 Pourbaix Diagram – In regards to a material's environment

All metals are unique and have a weakness. Some metals do not like acidic (low pH) environments. Some metals do not like alkaline (high pH) environments. Some metals don't like either high or low pH environments such as aluminum. These are called amphoteric materials. Some metals become passivated and do not corrode at high pH environments such as steel. These characteristics are documented in Marcel Pourbaix's book "Atlas of electrochemical equilibria in aqueous solutions"

In the mid 1900's, Marcel Pourbaix developed the Pourbaix diagram which describes a metal's reaction to an environment dependent on pH and voltage conditions. It describes when a metal remains passive (non-corroding) and in which conditions metals become soluble (corrode). Steels are passive in pH over 12 such as the condition when it is encased in cement. If the cement were to carbonate and its pH reduce to below 12, the cement would no longer be able to act as a corrosion inhibitor and the steel will begin to corrode when moist.

Some metals such as aluminum are amphoteric, meaning that they react with acids and bases. They can corrode in low pH and in high pH conditions. Aluminum alloys are generally passive within a pH of 4 and 8.5 but will corrode outside of those ranges. This is why aluminum cannot be embedded in cement and why brick mortar should not be laid against an aluminum window frame without a protective barrier between them.

5.2 Galvanic Series – In regards to dissimilar metal connections

All metals have a natural electrical potential. This electrical potential is measured using a high impedance voltmeter connected to the metal being tested and with the common lead connected to a copper copper-sulfate reference electrode (CSE) in water or soil. There are many types of reference electrodes. In laboratory measurements, a Standard Hydrogen Electrode (SHE) is commonly used. When different metal alloys are tested they can be ranked into an order from most noble (less corrosion), to least noble (more active corrosion). When a more noble metal is connected to a less noble metal, the less noble metal will become an anode and sacrifice itself through corrosion providing corrosion protection to the more noble metal. This hierarchy is known as the galvanic series named after Luigi Galvani whose experiments with electricity and muscles led Alessandro Volta to discover the reactions between dissimilar metals leading to the early battery. The greater the voltage difference between two metals, the faster the corrosion rate will be.

Table 1- Dissimilar Metal Corrosion Risk

	Zinc	Galvanized Steel	Aluminum	Cast Iron	Lead	Mild Steel	Tin	Copper	Stainless Steel
Zinc	None	Low	Medium	High	High	High	High	High	High
Galvanized Steel	Low	None	Medium	Medium	Medium	High	High	High	High
Aluminum	Medium	Medium	None	Medium	Medium	Medium	Medium	High	High
Cast Iron	High	Medium	Medium	None	Low	Low	Low	Medium	Medium
Lead	High	Medium	Medium	Low	None	Low	Low	Medium	Medium
Mild Steel	High	High	Medium	Low	Low	None	Low	Medium	Medium
Tin	High	High	Medium	Low	Low	Low	None	Medium	Medium
Copper	High	High	High	Medium	Medium	Medium	Medium	None	Low
Stainless Steel	High	High	High	Medium	Medium	Medium	Medium	Low	None

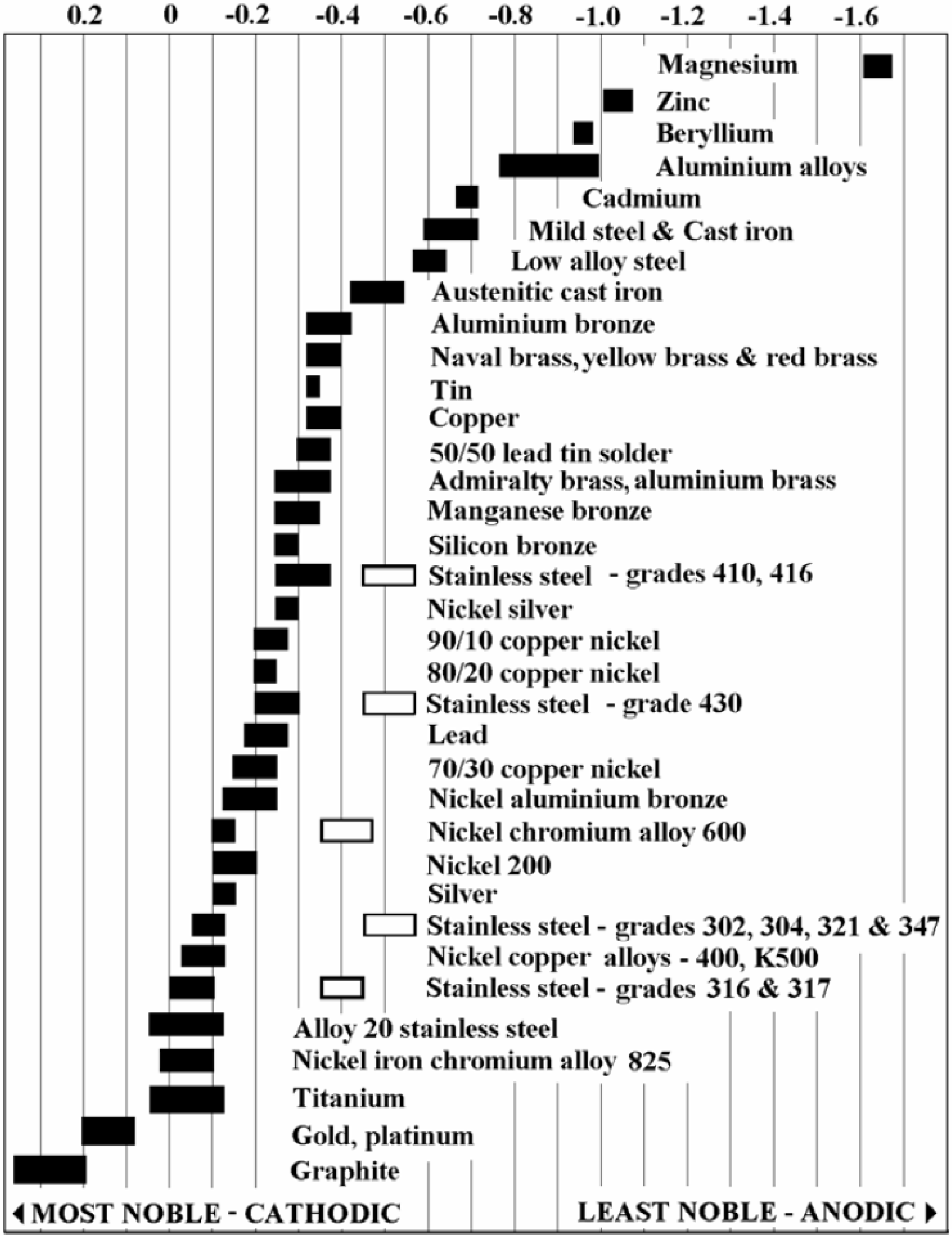
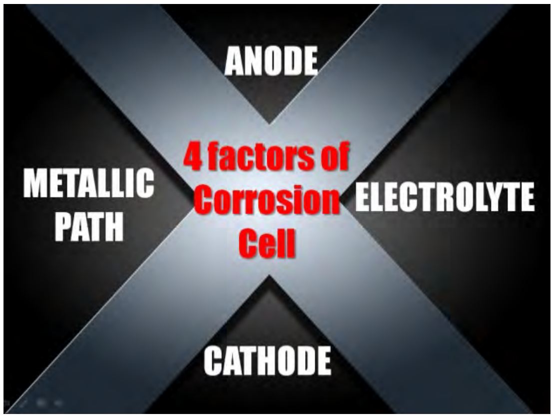


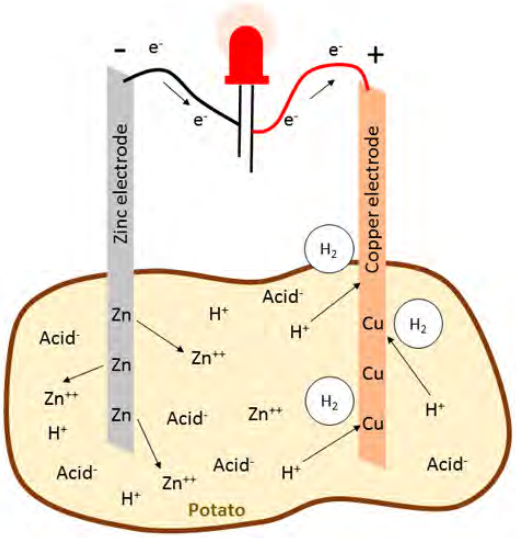
Figure 5 - Galvanic series of metals relative to CSE half cell.

5.3 Corrosion Cell

In order for corrosion to occur, four factors must be present. (1) The anode (2) the cathode (3) the electrolyte and (4) the metallic or conductive path joining the anode and the cathode. If any one of these is removed, corrosion activity will stop. This is how a simple battery produces electricity. An example of a non-metallic yet conductive material is graphite. Graphite is similar in nobility to gold. Do not connect graphite to anything in moist environments.



The anode is where the corrosion occurs, and the cathode is the corrosion free material. Sometimes the anode and cathode are different materials connected by a wire or union. Sometimes the anode and cathode are on the same pipe with one area of the pipe in a low oxygen zone while the other part of the pipe is in a high oxygen zone. A good example of this is a post in the ocean that is repeatedly splashed. Deep underwater, corrosion is minimal, but at the splash zone, the corrosion rate is greatest.



Low oxygen zones and crevices can also harbor corrosive bacteria which in moist environments will lead to corrosion. This is why pipes are laid on backfill instead of directly on native cut soil in a trench. Filling a trench slightly with backfill before installing pipe then finishing the backfill creates a uniform environment around the entire surface of the pipe.

The electrolyte is generally water, seawater, or moist soil which allows for the transfer of ions and electrical current. Pure water itself is not very conductive. It is when salts and minerals dissolve into pure water that it becomes a good conductor of electricity and chemical reactions. Metal ores are turned into metal alloys which we use in construction. They naturally want to return to their natural metal ore state but it requires energy to return to it. The corrosion cell, creates the energy needed to return a metal to its natural ore state.

The metallic or conductive path can be a wire or coupling. Examples are steel threaded into a copper joint, or an electrician grounding equipment to steel pipes inadvertently connecting electrical grid copper grounding systems to steel or iron underground pipes.

The ratio of surface area between the anode and the cathode is very important. If the anode is very large, and the cathode is very small, then the corrosion rate will be very small and the anode may live a long life. An example of this is when short copper laterals were connected to a large and long steel pipeline. The steel had plenty of surface area to spread the copper's attack, thus corrosion was not noticeable. But if the copper was the large pipe and the steel the short laterals, the steel would corrode at an amazing rate.

5.4 Design Considerations to Avoid Corrosion

The following recommendations are based upon typical observations and conclusions made by forensic engineers in construction defect lawsuits and NACE International (Corrosion Society) recommendations.

5.4.1 Testing Soil Factors (Resistivity, pH, REDOX, SO, CL, NO3, NH3)

As previously mentioned, different factors can cause corrosion. The most useful and common test for categorizing a soil's corrosivity has been the measure of soil resistivity which is typically measured in units of (ohm-cm) by corrosion engineers and geologists. Soil resistivity is the ability of soil to conduct or resist electrical currents and ion transfer. The lower the soil resistivity, the more conductive and corrosive it is. The following are "generally" accepted categories but keep in mind, the question is not "Is my soil corrosive?", the question should be, "What is my soil corrosive to?" and to answer that question, soil resistivity and chemistry must be tested. Though **soil resistivity is a good corrosivity indicator for steel materials, high chlorides or other corrosive elements do not always lower soil resistivity, thus if you don't test for chlorides and other water soluble salts, you can get an unpleasant surprise.** The largest contributing factor to a soil's electrical resistivity is its clay, mineral, metal, or sand make-up.

Table 2 - Corrosion Basics- An Introduction, NACE, 1984, pg 191

(Ohm-cm)	Corrosivity Description
0-500	Very Corrosive
500-1,000	Corrosive
1,000-2,000	Moderately Corrosive
2,000-10,000	Mildly Corrosive
Above 10,000	Progressively less corrosive

Testing a soil's pH provides information to reference the Pourbaix diagram of specific metals. Some elements such as ammonia and nitrates can create localized alkaline conditions which will greatly affect amphoteric materials such as aluminum and copper alloys.

Excess sulfates can break-down the structural integrity of cement and high concentrations of chlorides can overcome cement's corrosion inhibiting effect on encased ferrous metals and break down protective passivated surface layers on stainless steels and aluminum.

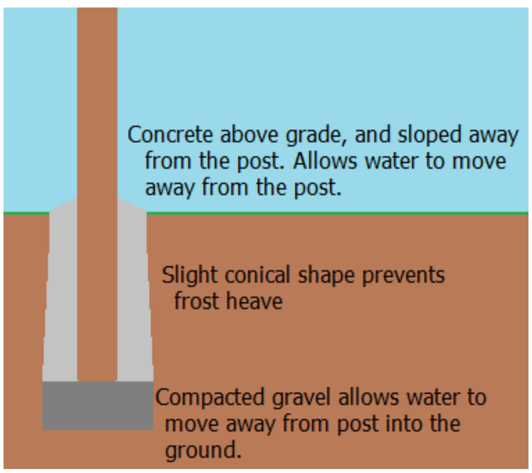
Corrosive bacteria are everywhere but can multiply significantly in anaerobic conditions with plentiful sulfates. The bacteria themselves do not eat the metal but their by-products can form corrosive sulfuric acids. The probability of corrosive bacteria is tested by measuring a soil's oxidation-reduction (REDOX) electro-potential and by testing for the presence of sulfides.

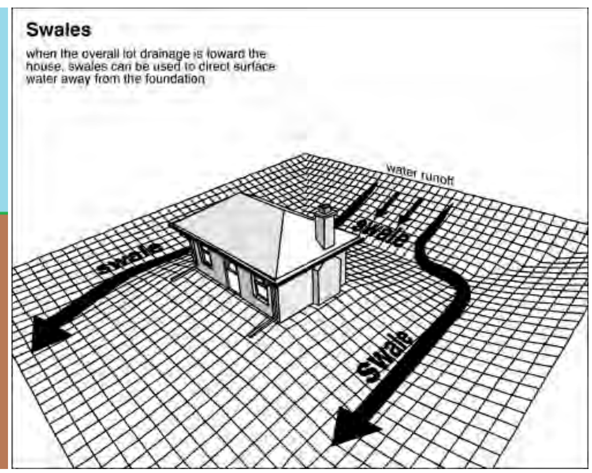
Only by testing a soil's chemistry for minimum resistivity, pH, chlorides, sulfates, sulfides, ammonia, nitrate, and redox potential can one have the information to evaluate the corrosion risk to construction materials such as steel, stainless steel, galvanized steel, iron, copper, brass, aluminum, and concrete.

5.4.2 ***Proper Drainage***

It cannot be emphasized enough that pooled stagnant water on metals will eventually lead to corrosion. This stands for internal corrosion and external corrosion situations. In soils, providing good drainage will lower soil moisture content reducing corrosion rates. Attention to properly sealing polyethylene wraps around valves and piping will avoid water intrusion which would allow water to pool against metals. Above ground structures should not have cupped or flat surfaces that will pond water after rain or irrigation events.

Buildings typically are built on pads and have swales when constructed to drain water away from buildings directing it towards an acceptable exit point such as a driveway where it continues draining to a local storm drain. Many homeowners, landscapers and flatwork contractors appear to not be aware of this and destroy swales during remodeling. The majority of garage floor and finished grade elevations are governed by drainage during design. [13,14]



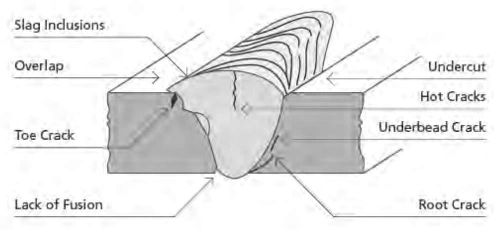


5.4.3 ***Avoiding Crevices***

Crevices are excellent locations for oxygen differential induced corrosion cells to begin. Crevices can also harbor corrosive bacteria even in the most chemically treated waters. Crevices will also gather salts. If water's total alkalinity is low, its ability to maintain a stable pH can also become more difficult within a crevice allowing the pH to drop to acidic levels continuing a pitting process. Welds in extremely corrosive environments should be complete and well filleted without sharp edges to avoid crevices. Sharp edges should be avoided to allow uniform coating of protective epoxy. Detection of crevices in welds should be treated immediately. If pressures and loads are low, sanding and rewelding or epoxy patching can be suitable repairs. Damaged coatings can usually be repaired with Direct to Metal paints. **Scratches and crevice corrosion are like infections, they should not be left to fester or the infection will spread making things worse.**

[13] https://www.fencedaddy.com/blogs/tips-and-tricks/132606467-how-to-repair-a-broken-fence-post

[14] http://southdownstudio.co.uk/problme-drainage-maison.html



BAD **GOOD**

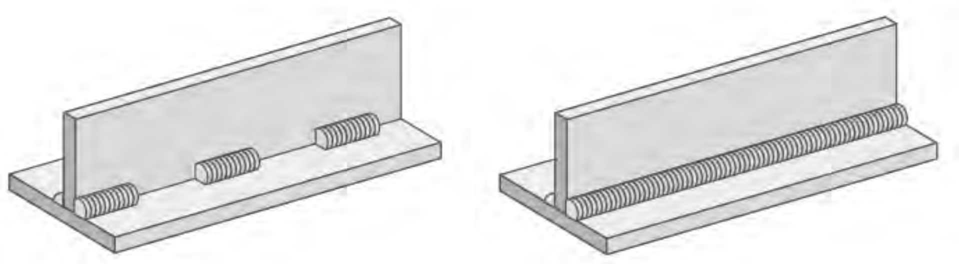


Figure 6- Defects which form weld crevices[15]

5.4.4 Coatings and Cathodic Protection

When faced with a corrosive environment, the best defense against corrosion is removing the electrolyte from the corrosion cell by applying coatings to separate the metal from the soil. During construction and installation, there is always some scratch or damage made to a coating. NACE training recommends that coatings be used as a first line of defense and that sacrificial or impressed current cathodic protection is used as a 2nd line of defense to protect the scratched areas. Use of a good coating dramatically reduces the amount of anodes a CP system would need. If CP is not installed as a 2nd line of defense in an extremely corrosive environment, the small scratched zones will suffer accelerated corrosion. CP details such as anode installation instructions must be designed by corrosion engineers or vessel manufacturers on a per project basis because it depends on electrolyte resistivity, surface area of infrastructure to be protected, and system geometry.

There are two types of cathodic protection systems, a Galvanic Anode Cathodic Protection (GACP) system and an Impressed Current Cathodic Protection (ICCP) system. A Galvanic Anode Cathodic Protection (GACP) system is simpler to install and maintain than an Impressed Current Cathodic Protection (ICCP) system. To protect the metals, they must all be electrically continuous to each other. In a GACP system, sacrificial zinc or magnesium anodes are then buried at locations per the CP design and connected by wire to a structure at various points in system. At the connection points, a wire connecting to the structure and the wire from the anode are joined in a Cathodic Protection Test Station hand hole which looks similar in size and shape to an irrigation valve pull box. By coating the underground structures, one can reduce the number of anodes needed to provide cathodic protection by 80% in many instances.

An ICCP system requires a power source, a rectifier, significantly more trenching, and more expensive type anodes. These systems are typically specified when bare metal is requiring protection

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[15] http://www.daroproducts.co.uk/makes-good-weld/

in severely corrosive environments in which galvanic anodes do not provide enough power to polarize infrastructure to -850 mV structure-to-soil potential or be able to create a 100 mV potential shift as required by NACE SP169 to control corrosion. In severely corrosive environments, a GACP system simply may not last a required lifetime due to the high rate of consumption of the sacrificial anodes. ICCP system rectifiers must be inspected and adjusted quarterly or at a minimum bi-annually per NACE recommendations. Different anode installations may be possible but for large sites, anodes are placed evenly throughout the site and all anode wires must be trenched to the rectifier. For a large site, it may be beneficial to use two or more rectifiers to reduce wire lengths or trenching.

To simplify, a GACP system can be installed and practically forgotten with minor trenching because the anodes can be installed very close to the structures. An ICCP system must be inspected annually and anode wires run back to the rectifier which itself connects to the pile system. If any type of trenching or development is expected to occur at the site during the life of the site, it is a good idea to inspect the anode connections once a year to make sure wires are not cut and that the infrastructure is still being provided adequate protection. A common situation that occurs with ICCP systems is that a contractor accidently cuts the wires during construction then reconnects them incorrectly, turning the once cathode, into a sacrificing anode.

Design of a cathodic protection system protecting against soil side corrosion requires that Wenner Four Pin ground resistance measurements per ASTM G57 be performed by corrosion engineers at various locations of the site to determine the best depths and locations for anode installations. Ideally, a sample pile is installed and experiments determining current requirement are conducted. Using this data, the decision is made whether a GACP system is feasible or if an ICCP must be used.

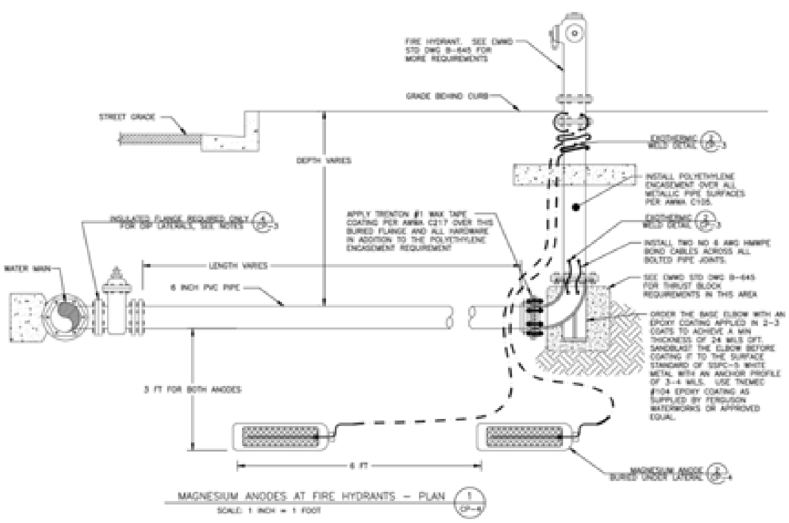


Figure 7- Sample anode design for fire hydrant underground piping

Vessels such as water tanks will have protective interior coatings and anodes to protect the interior surfaces. Anodes can also be buried on site and connected to system skid supports to protect the metal in contact with soil. A good example of a vessel cathodic protection system exists in all home water heaters which contain sacrificial aluminum or magnesium anodes. In environments that exceed 140F, zinc anodes cannot be used with carbon steel because they become the aggressor (Cathodic) to the steel instead of sacrificial (anodic). Anodes in vessels containing extremely brackish water with chloride levels over 2,000 ppm should inspect or change out their anodes every 6 months.

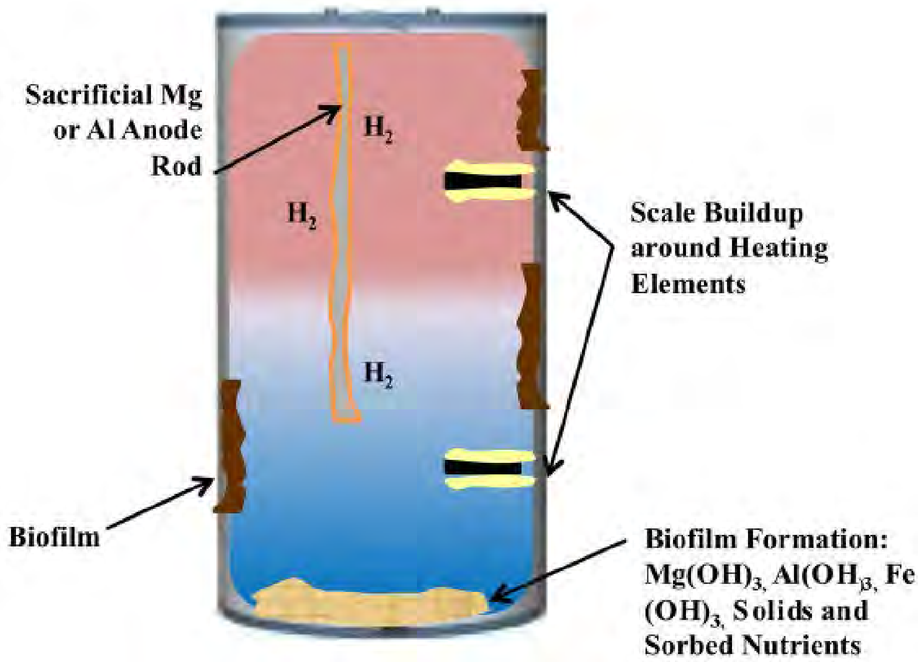


Figure 8- Cross section of boiler with anode

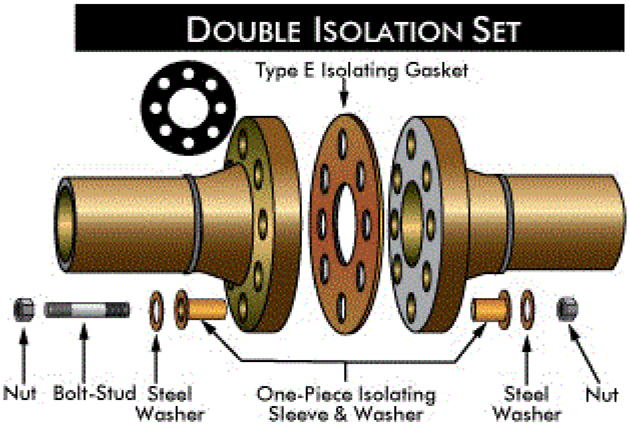
Cathodic protection can only protect a few diameters within a pipeline thus it is not recommended for small diameter pipelines and tubing internal corrosion protection. Anodes are like a lamp shining light in a room. They can only protect along their line of sight.

5.4.5 Good Electrical Continuity

In order for cathodic protection to protect a long pipeline or system of pipes from external soil side corrosion, they must all be electrically continuous to each other so that the electric current from the anode can travel along the pipes, then return through the earth to the anode. Electrical continuity is achieved by welding or pin brazing #8 AWG copper strand bond cable to the end of pipe sticks which have rubber gaskets at bell and spigots. If steel pipes are joined by full weld, bonding wires are not needed.

Electrical continuity between dissimilar metals is not desirable. Isolation joints or di-electric unions should be installed between dissimilar metals, such as steel pipes connecting to a brass valve per NACE SP0286. Bonding wires should then be welded onto the steel pipes by-passing the brass valve so that the cathodic protection system's current can continue to travel along the steel piping but isolate the brass valve from the steel pipeline. Another option would be to provide a separate cathodic protection system for steel pipes on both sides of the brass valve.

Typically, water heater inlets and outlets, gas meters and water meters have dielectric unions installed in them to separate utility property from homeowner property. This also protects them in the case that a home owner somehow electrically connects water pipes or gas pipes to a neighborhood electrical grounding system which can potentially have less noble steel in soil now connected to much more noble copper in soil which will then create a corrosion cell. This is exactly how a lemon powered clock works when a galvanized zinc nail and a steel nail are inserted into a lemon then connected to a clock. The clock is powered by the corrosion cell created.



5.4.6 <u>Bad Electrical Continuity</u>

Bad electrical continuity is when two different materials or systems are made electrically continuous (aka shorted) when they were not designed to be electrically continuous. Examples of this would be when gas lines are shorted to water lines or to electrical grounding beds. Very often, fire risers are shorted to electrical grounding systems, and water pipes at business parks. Since fire risers usually have a very short ductile iron pipe in the ground which connects to PVC pipe systems, they tend to experience leaks after 7 to 10 years of being attacked by underground copper systems.

It is absolutely imperative that any copper water piping or other metal conduits penetrating cement slab or footings, not come in contact with the reinforcing steel or post-tensioning tendons to avoid creation of galvanic corrosion cells.

5.4.7 <u>Corrosion Test Stations</u>

Corrosion test stations should be installed every 1,000 feet along pipelines in order to measure corrosion activity in the future. For a simple pipeline, two #8 AWG copper strand bond cable welded or pin brazed onto the pipeline are run up to finished grade and left in a hand hole. Corrosion test stations are used to measure pipe-to-soil electro potential relative to a copper copper-sulfate reference electrode to determine if the pipe is experiencing significant corrosion activity. By measuring test stations along a pipeline, hot spots can be determined, if any. The wires also allow for electrical continuity testing, condition assessment, and a multitude of other types of tests.

At isolation joints and pipe casings, two wires should be welded to either side of the isolation joint for a total of 4 wires to be brought up to the hand hole. This allows for future tests of the isolation joint, casing separation confirmation, and pipe-to-soil potential readings during corrosion surveys.

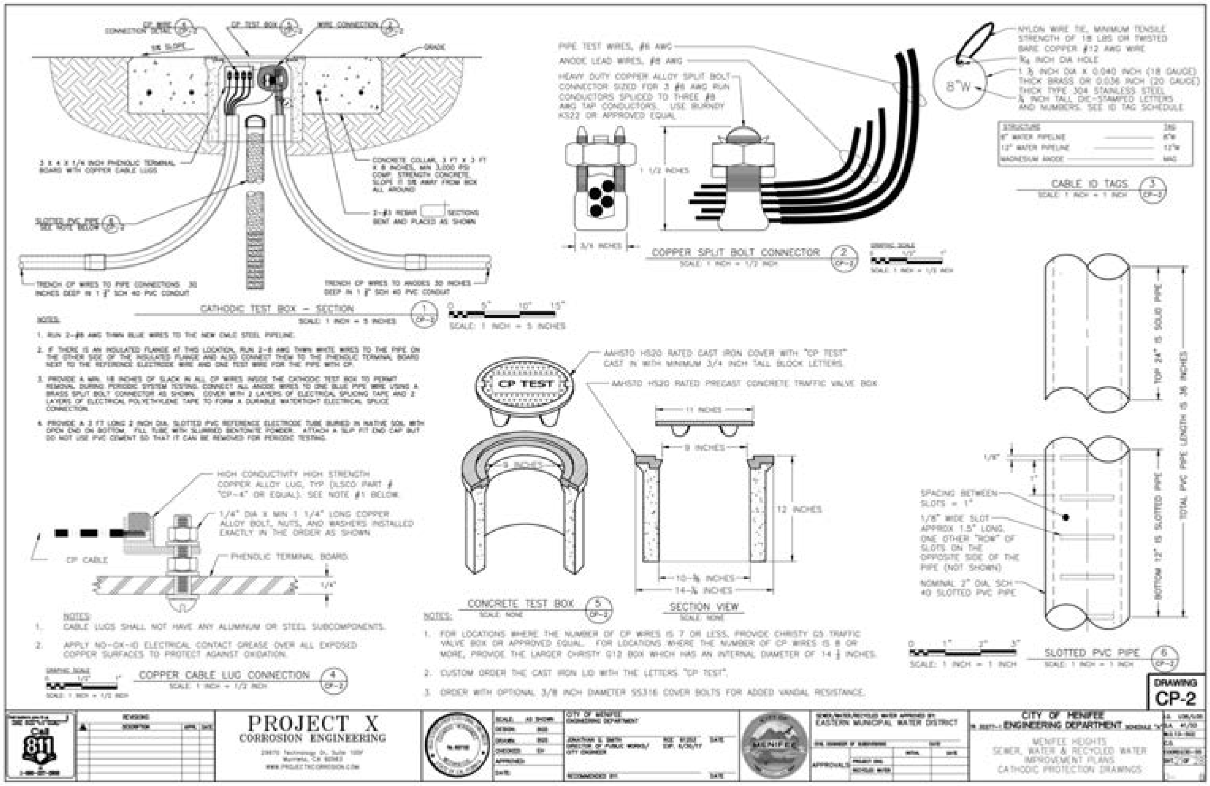


Figure 9- Sample of corrosion test station specification drawing

5.4.8 *Excess Flux in Plumbing*

Investigations of internal corrosion of domestic water plumbing systems almost always finds excess flux to be the cause of internal pitting of copper pipes. Some people believe that there is no such thing as too much flux. Flux runs have been observed to travel up to 20 feet with pitting occurring along the flux run. Flushing a soldered plumbing system with hot water for 15 minutes can remove significant amounts of excess flux left in the pipes. If a plumbing system is expected to be stagnant for some time, it should be drained to avoid stagnant water conditions that can lead to pitting and dezincification of yellow brasses.

5.4.9 *Landscapers and Irrigation Sprinkler Systems*

A significant amount of corrosion of fences is due to landscaper tools scratching fence coatings and irrigation sprinklers spraying these damaged fences. Recycled water typically has a higher salt content than potable drinking water, meaning that it is more corrosive than regular tap water. The same risk from damage and water spray exists for above ground pipe valves and backflow preventers. Fiber glass covers, cages, and cement footings have worked well to keep tools at an arm's length.

5.4.10 *Roof Drainage splash zones*

Unbelievably, even the location where your roof drain splashes down can matter. We have seen drainage from a home's roof valley fall directly down onto a gas meter causing it's piping to corrode at an accelerated rate reaching 50% wall thickness within 4 years. It is the same effect as a splash zone in the ocean or in a pool which has a lot of oxygen and agitation that can remove material as it corrodes.

5.4.11 Stray Current Sources

Stray currents which cause material loss when jumping off of metals may originate from direct-current distribution lines, substations, or street railway systems, etc., and flow into a pipe system or other steel structure. Alternating currents may occasionally cause corrosion. The corrosion resulting from stray currents (external sources) is similar to that from galvanic cells (which generate their own current) but different remedial measures may be indicated. In the electrolyte and at the metal-electrolyte interfaces, chemical and electrical reactions occur and are the same as those in the galvanic cell; specifically, the corroding metal is again considered to be the anode from which current leaves to flow to the cathode. Soil and water characteristics affect the corrosion rate in the same manner as with galvanic-type corrosion.

However, stray current strengths may be much higher than those produced by galvanic cells and, as a consequence, corrosion may be much more rapid. Another difference between galvanic-type currents and stray currents is that the latter are more likely to operate over long distances since the anode and cathode are more likely to be remotely separated from one another. Seeking the path of least resistance, the stray current from a foreign installation may travel along a pipeline causing severe corrosion where it leaves the line. Knowing when stray currents are present becomes highly important when remedial measures are undertaken since a simple sacrificial anode system is likely to be ineffectual in preventing corrosion under such circumstances.[16] Stray currents can be avoided by installing proper electrical shielding, installation of isolation joints, or installation of sacrificial jump off anodes at crossings near protected structures such as metal gas pipelines or electrical feeders.

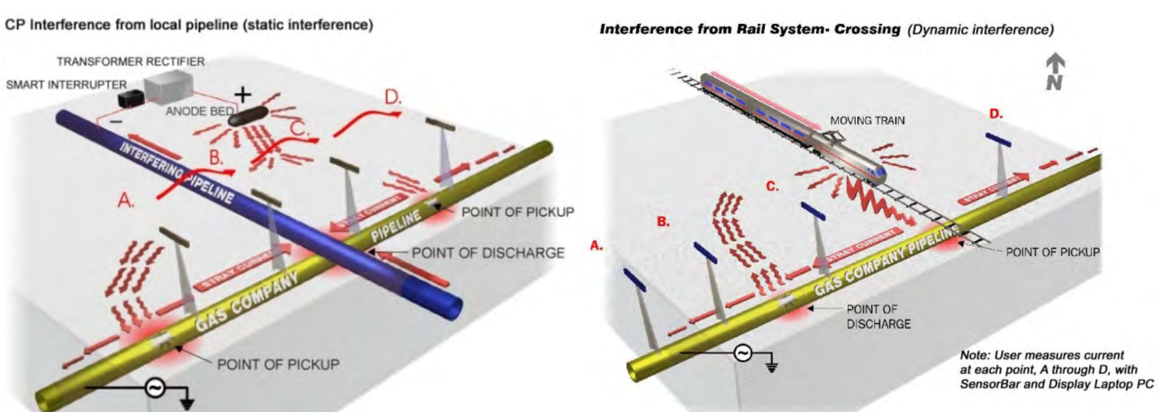


Figure 10- Examples of Stray Current[17]

[16] http://corrosion-doctors.org/StrayCurrent/Introduction.htm

[17] http://www.eastcomassoc.com/

Project X Corrosion Engineering

Lab Request Sheet Chain of Custody

Phone: (213) 928-7213 · Fax (951) 226-1720 · www.projectxcorrosion.com

Ship Samples To: 29990 Technology Dr, Suite 13, Murrieta, CA 92563

Project X Job Number: S240325J NMG-GEO 23111-01 SEGERSTROM 8 Full + RPT

IMPORTANT: Please complete Project and Sample Identification Data as you would like it to appear in report & include this form with samples.

Company Name:	NMG Geotechnical, Inc.	Contact Name:	Bryan Jimenez — Phone No: (949)442-2442
Mailing Address:	17991 Fitch, Irvine, CA 92614	Contact Email:	BJimenez@nmggeo.com
Accounting Contact:	Tina Kami	Invoice Email:	TKami@nmggeo.com
Client Project No:	23111-01	Project Name:	Segerstrom/Lake Center

P.O. #:	3-5 Day Standard	3 Day Guarantee 50% mark-up	24 Hour RUSH 100% mark-up
(Business Days) Turn Around Time:	●		

METHOD ANALYSIS REQUESTED (Please circle)

For Corrosion Control Recommendations (350g soil sample):
NEED (1) Groundwater depth and
(2) Soil Sample Locations Map

10' to 13' deep below the ground surface

Default Method

FOR THERMAL RESISTIVITY PROVIDE (1,500g soil sample):
(1) Optimal Moisture %
(2) Dry Density{PCF}
(3) Desired Compaction
Date & Received By:

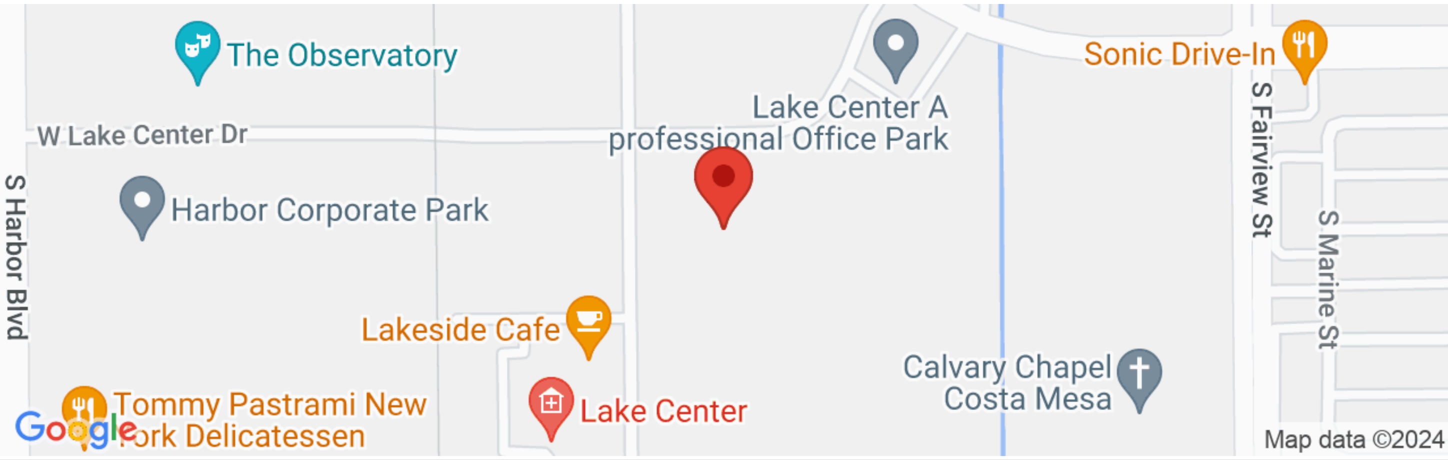
	#	SAMPLE ID - BORE # - Description	DEPTH (ft)	DATE COLLECTED	Soil Resistivity	pH	Sulfate	Chloride	Redox Potential	Sulfide	Ammonia	Nitrate	Flouride	Phosphate	Lithium	Sodium	Potassium	Magnesium	Calcium	Full Corrosion Series	Soil Corrosivity Evaluation Report	Water Corrosivity Mini Report	Moisture Content	Total Alkalinity	Soil Remolding	Thermal Resistivity	(ANC) Alk Neutr Cap	(SRB) Sulfate Reducing	Puckorius Index	XRF Elemental Analysis	Water Hardness
349	1	T-1, B-1 SM-CL	1'	2-29/3-12																●	●										
350	2	T-1,B-2 ML	7'	2-29/3-12																●	●										
351	3	T-1, B-3 CH	13'	2-29/3-12																●	●										
352	4	T-2, B-1 ML	1'	2-29/3-12																●	●										
353	5	H-6, B-1 ML	0-5'	2-29/3-12																●	●										
197	6	T-1 A SB-1	0-1																	●	●										
198	7	T-1 C SB-1	0-1																	●	●										
199	8	T-16 SB-1	0-1																	●	●										
	9																														
	10																														
	11																														
	12																														
	13																														
	14																														
	15																														

APPENDIX D

USGS web services were down for some period of time and as a result this tool wasn't operational, resulting in *timeout* error.
USGS web services are now operational so this tool should work as expected.

OSHPD

Latitude, Longitude: 33.698479, -117.912727



Date	2/26/2024, 4:21:27 PM
Design Code Reference Document	ASCE7-16
Risk Category	II
Site Class	D - Stiff Soil

Type	Value	Description
S_S	1.307	MCE_R ground motion. (for 0.2 second period)
S_1	0.47	MCE_R ground motion. (for 1.0s period)
S_{MS}	1.307	Site-modified spectral acceleration value
S_{M1}	null -See Section 11.4.8	Site-modified spectral acceleration value
S_{DS}	0.872	Numeric seismic design value at 0.2 second SA
S_{D1}	null -See Section 11.4.8	Numeric seismic design value at 1.0 second SA

Type	Value	Description
SDC	null -See Section 11.4.8	Seismic design category
F_a	1	Site amplification factor at 0.2 second
F_v	null -See Section 11.4.8	Site amplification factor at 1.0 second
PGA	0.562	MCE_G peak ground acceleration
F_{PGA}	1.1	Site amplification factor at PGA
PGA_M	0.618	Site modified peak ground acceleration
T_L	8	Long-period transition period in seconds
SsRT	1.307	Probabilistic risk-targeted ground motion. (0.2 second)
SsUH	1.421	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration
SsD	1.995	Factored deterministic acceleration value. (0.2 second)
S1RT	0.47	Probabilistic risk-targeted ground motion. (1.0 second)
S1UH	0.509	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration.
S1D	0.687	Factored deterministic acceleration value. (1.0 second)
PGAd	0.819	Factored deterministic acceleration value. (Peak Ground Acceleration)
PGA_{UH}	0.562	Uniform-hazard (2% probability of exceedance in 50 years) Peak Ground Acceleration
C_{RS}	0.92	Mapped value of the risk coefficient at short periods

Type	Value	Description
C_{R1}	0.923	Mapped value of the risk coefficient at a period of 1 s
C_V	1.361	Vertical coefficient

DISCLAIMER

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Unified Hazard Tool

Please do not use this tool to obtain ground motion parameter values for the design code reference documents covered by the U.S. Seismic Design Maps web tools (e.g., the International Building Code and the ASCE 7 or 41 Standard). The values returned by the two applications are not identical.

Please also see the new USGS Earthquake Hazard Toolbox for access to the most recent NSHMs for the conterminous U.S. and Hawaii.

Input

Edition

Dynamic: Conterminous U.S. 2014 (u…

Spectral Period

Peak Ground Acceleration

Latitude

Decimal degrees

33.698479

Time Horizon

Return period in years

2475

Longitude

Decimal degrees, negative values for western longitudes

-117.912727

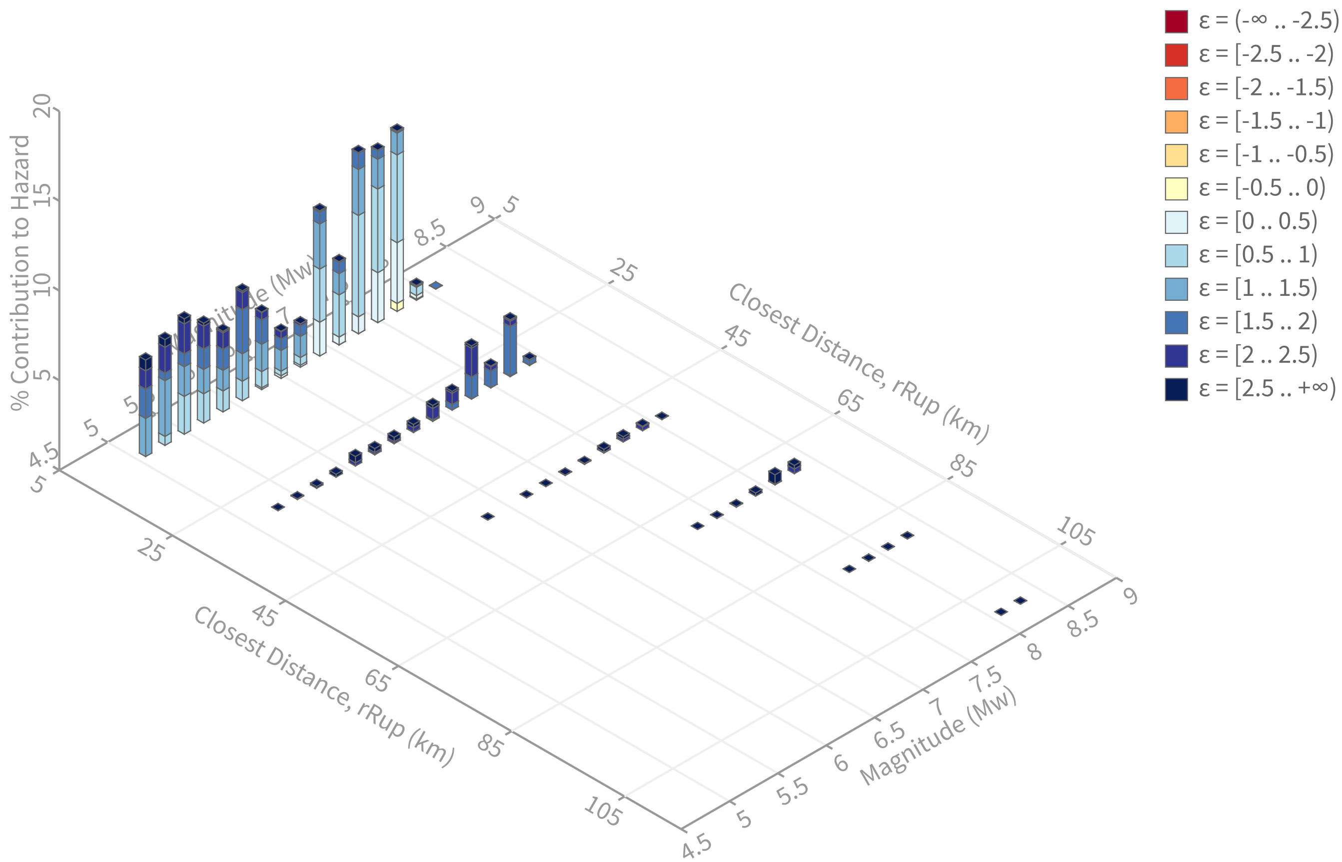
Site Class

259 m/s (Site class D)

Deaggregation

Component

Total



Summary statistics for, Deaggregation: Total

Deaggregation targets

Return period: 2475 yrs
Exceedance rate: 0.0004040404 yr^{-1}
PGA ground motion: 0.6537669 g

Recovered targets

Return period: 2943.8203 yrs
Exceedance rate: 0.00033969465 yr^{-1}

Totals

Binned: 100 %
Residual: 0 %
Trace: 0.04 %

Mean (over all sources)

m: 6.67
r: 11.48 km
ε_0: 1.29 σ

Mode (largest m-r bin)

m: 7.3
r: 10.18 km
ε_0: 0.93 σ
Contribution: 10.09 %

Mode (largest m-r-ε_0 bin)

m: 7.31
r: 10.39 km
ε_0: 0.8 σ
Contribution: 5.65 %

Discretization

r: min = 0.0, max = 1000.0, Δ = 20.0 km
m: min = 4.4, max = 9.4, Δ = 0.2
ε: min = -3.0, max = 3.0, Δ = 0.5 σ

Epsilon keys

ε0: [-∞ .. -2.5)
ε1: [-2.5 .. -2.0)
ε2: [-2.0 .. -1.5)
ε3: [-1.5 .. -1.0)
ε4: [-1.0 .. -0.5)
ε5: [-0.5 .. 0.0)
ε6: [0.0 .. 0.5)
ε7: [0.5 .. 1.0)
ε8: [1.0 .. 1.5)
ε9: [1.5 .. 2.0)
ε10: [2.0 .. 2.5)
ε11: [2.5 .. +∞]

Deaggregation Contributors

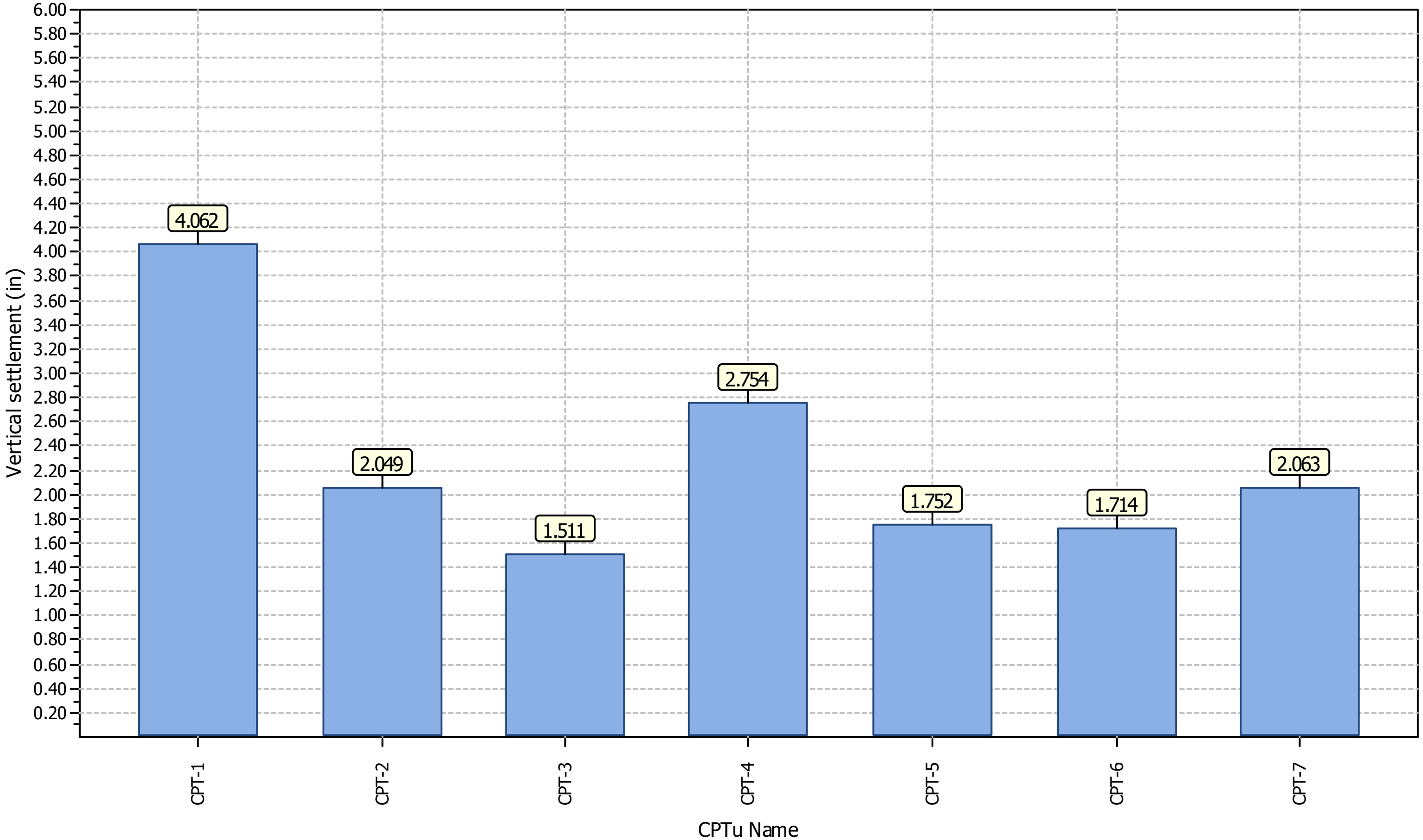
Source Set ↳ Source	Type	r	m	ε_0	lon	lat	az	%
UC33brAvg_FM32	System							30.61
San Joaquin Hills [0]		4.15	7.15	0.56	117.914°W	33.673°N	183.02	9.39
Newport-Inglewood alt 2 [1]		7.49	7.48	0.83	117.974°W	33.657°N	231.07	6.07
Compton [0]		13.01	7.33	0.96	118.043°W	33.702°N	272.14	4.64
Palos Verdes [7]		25.14	7.45	1.98	118.145°W	33.581°N	238.81	1.89
Anaheim [0]		10.66	6.87	1.24	117.943°W	33.780°N	343.04	1.37
Whittier alt 2 [2]		26.15	7.64	1.87	117.773°W	33.903°N	29.53	1.10
UC33brAvg_FM31	System							27.64
San Joaquin Hills [0]		4.15	7.53	0.45	117.914°W	33.673°N	183.02	6.69
Newport-Inglewood alt 1 [0]		7.57	7.45	0.82	117.976°W	33.658°N	232.44	6.57
Compton [0]		13.01	7.27	0.99	118.043°W	33.702°N	272.14	4.46
Palos Verdes [7]		25.14	7.29	2.08	118.145°W	33.581°N	238.81	1.76
Whittier alt 1 [3]		26.21	7.59	1.90	117.777°W	33.905°N	28.66	1.39
Anaheim [0]		10.66	6.83	1.26	117.943°W	33.780°N	343.04	1.36
UC33brAvg_FM31 (opt)	Grid							21.06
PointSourceFinite: -117.913, 33.712		5.30	5.60	1.12	117.913°W	33.712°N	0.00	5.25
PointSourceFinite: -117.913, 33.712		5.30	5.60	1.12	117.913°W	33.712°N	0.00	5.25
PointSourceFinite: -117.913, 33.802		11.31	5.93	1.83	117.913°W	33.802°N	0.00	1.38
PointSourceFinite: -117.913, 33.802		11.31	5.93	1.83	117.913°W	33.802°N	0.00	1.38
UC33brAvg_FM32 (opt)	Grid							20.69
PointSourceFinite: -117.913, 33.712		5.31	5.58	1.12	117.913°W	33.712°N	0.00	5.07
PointSourceFinite: -117.913, 33.712		5.31	5.58	1.12	117.913°W	33.712°N	0.00	5.07
PointSourceFinite: -117.913, 33.802		11.34	5.92	1.83	117.913°W	33.802°N	0.00	1.38
PointSourceFinite: -117.913, 33.802		11.34	5.92	1.83	117.913°W	33.802°N	0.00	1.38

APPENDIX E

Project title : Segerstrom/ Lake Center Office Park

Location : Santa Ana, California

Overall vertical settlements report



NMG Geotechnical, Inc.
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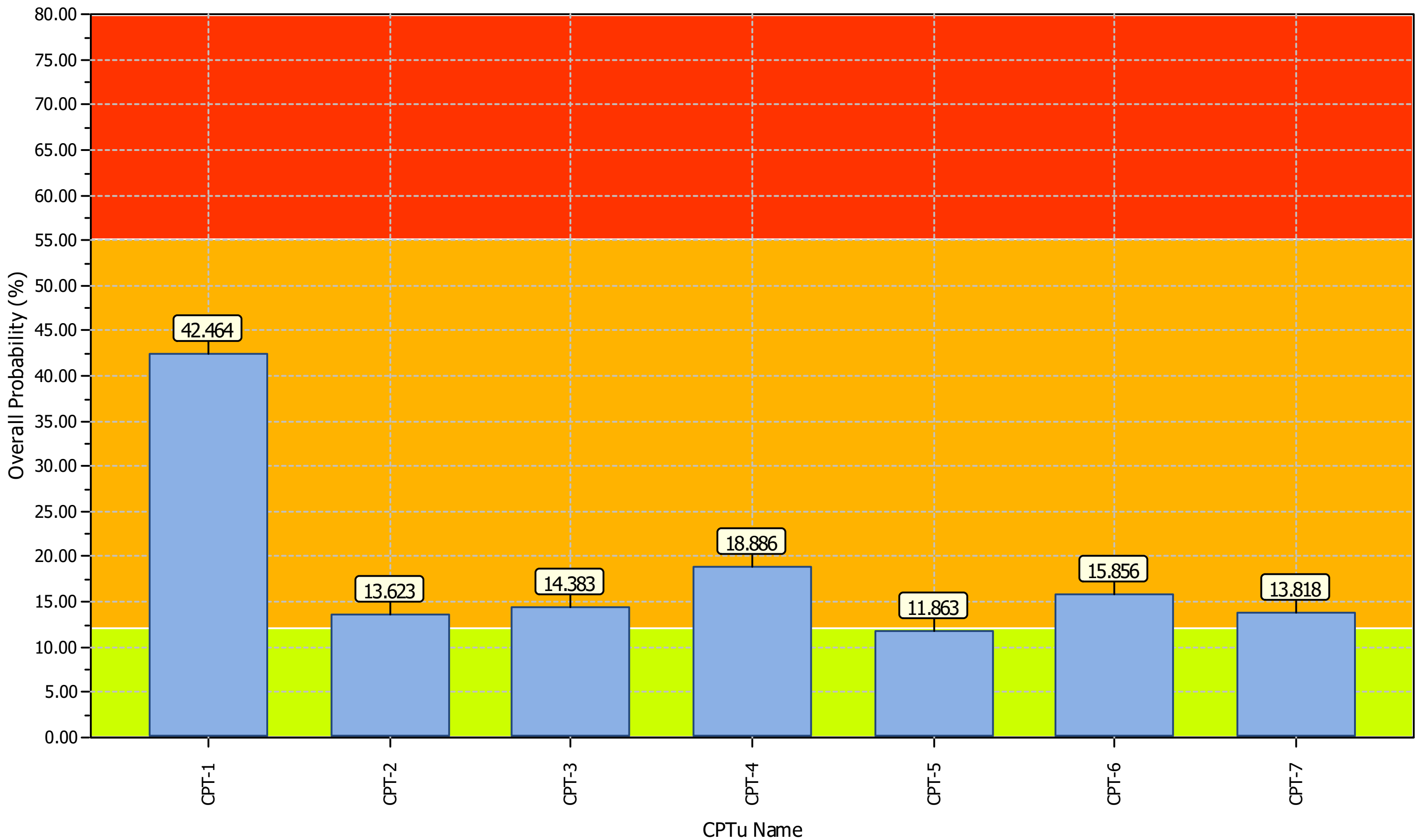
Irvine, CA 92614

Project title : Segerstrom/ Lake Center Office Park

Location : Santa Ana, California

Overall Probability for Liquefaction report



Probability color scheme

- Very High Probability
- High Probability
- Low Probability

Basic statistics

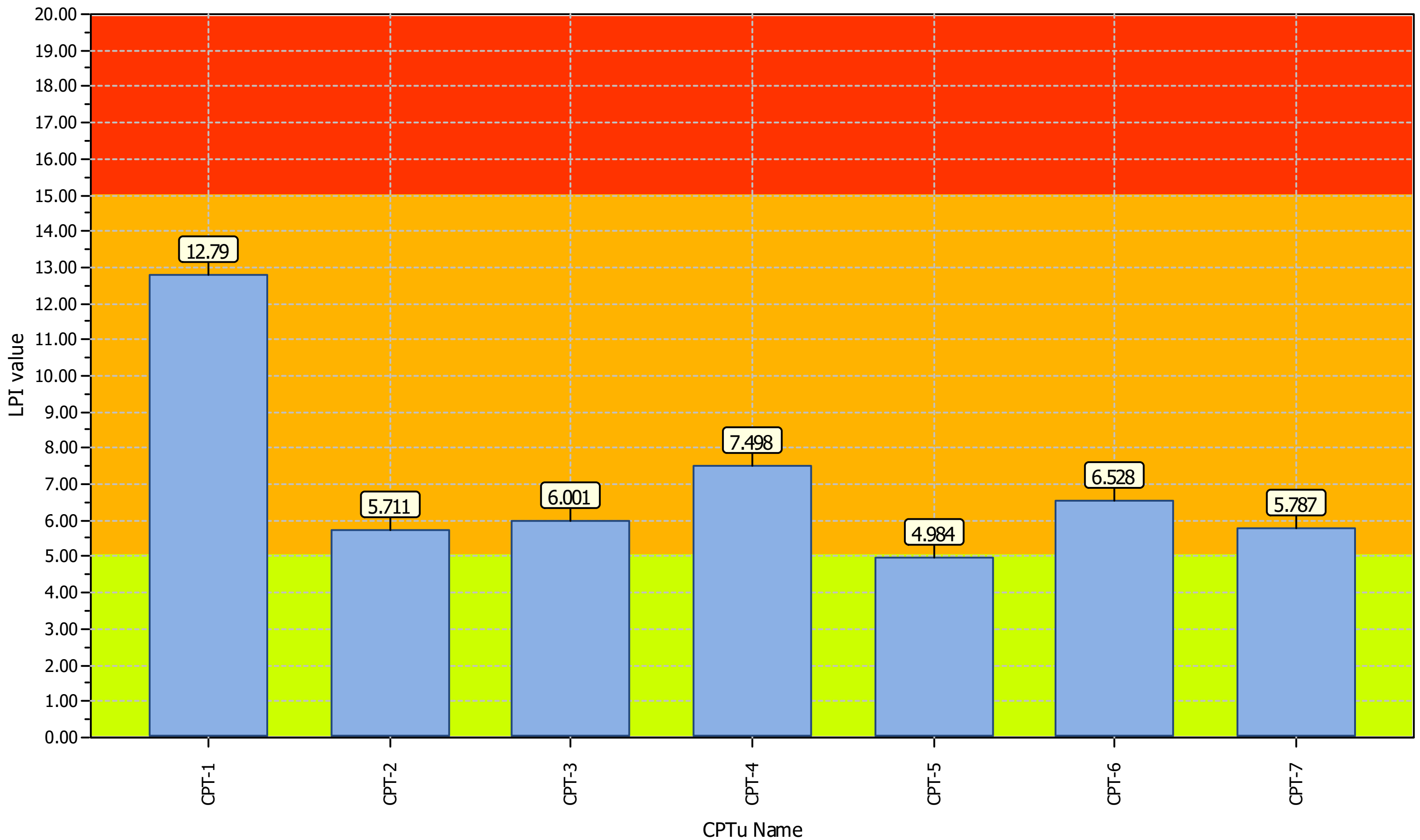
Total CPT number: 7
14% low probability
86% high probability
0% very high probability

NMG Geotechnical, Inc.
17991 Fitch
Irvine, CA 92614

Project title : Segerstrom/ Lake Center Office Park

Location : Santa Ana, California

Overall Liquefaction Potential Index report



LPI color scheme

- Very high risk
- High risk
- Low risk

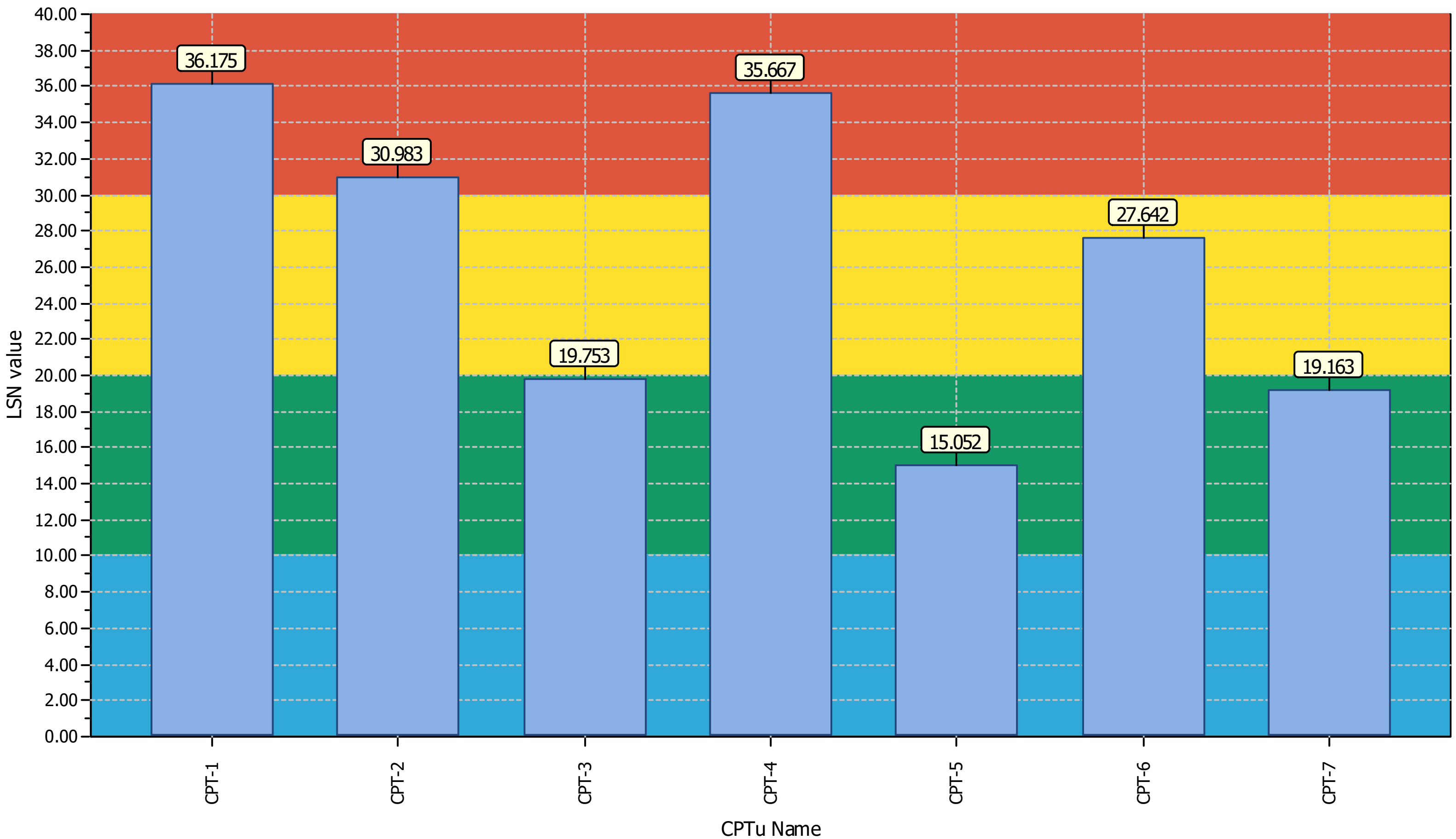
Basic statistics

Total CPT number: 7
14% low risk
86% high risk
0% very high risk

Project title : Segerstrom/ Lake Center Office Park

Location : Santa Ana, California

Overall Liquefaction Severity Number report



LSN color scheme

- Severe damage
- Major expression of liquefaction
- Moderate to severe exp. of liquefaction
- Moderate expression of liquefaction
- Minor expression of liquefaction
- Little to no expression of liquefaction

Basic statistics

Total CPT number: 7

0% little liquefaction

43% minor liquefaction

14% moderate liquefaction

43% moderate to major liquefaction

0% major liquefaction

0% severe liquefaction

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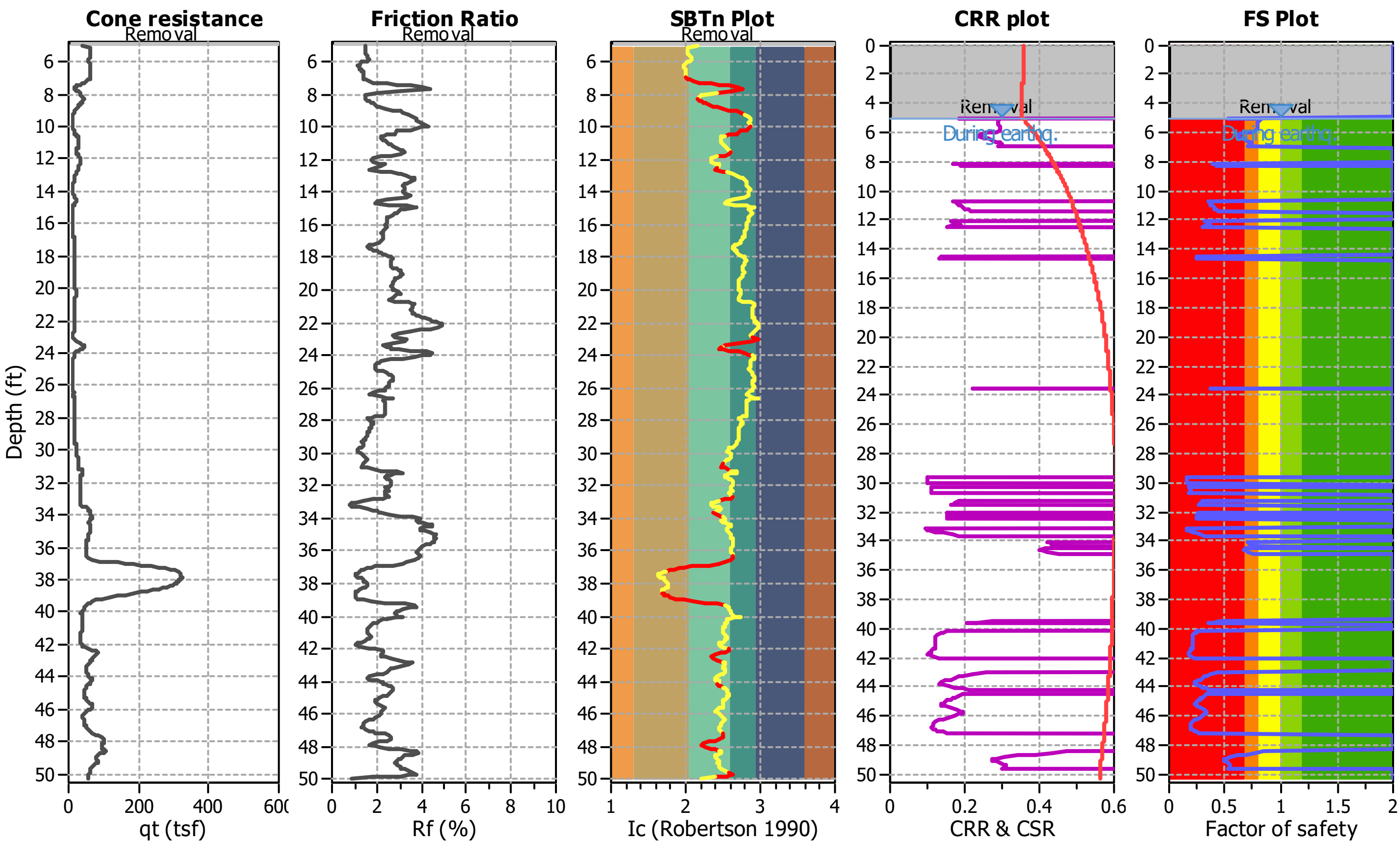
LIQUEFACTION ANALYSIS REPORT

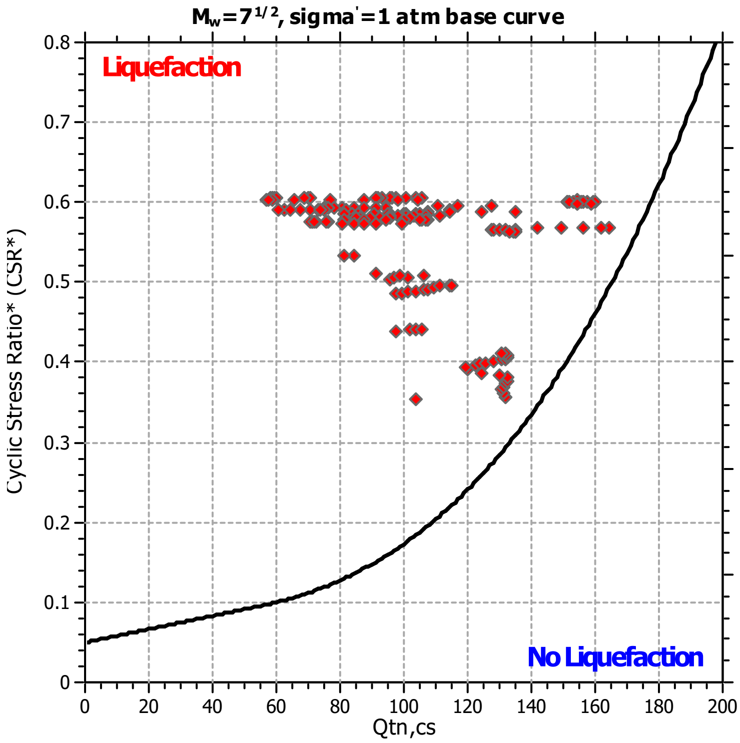
Project title : Segerstrom/ Lake Center Office Park **Location : Santa Ana, California**

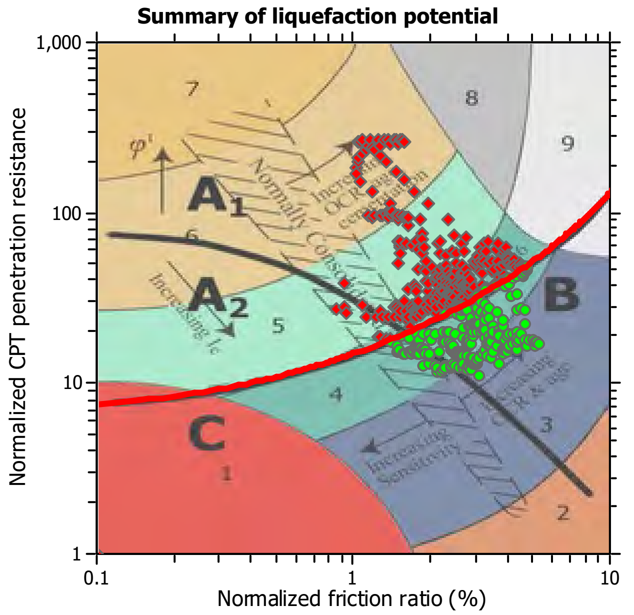
CPT file : CPT-1

Input parameters and analysis data

Analysis method:	NCEER (1998)	G.W.T. (in-situ):	10.00 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	5.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	7.15	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.62	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

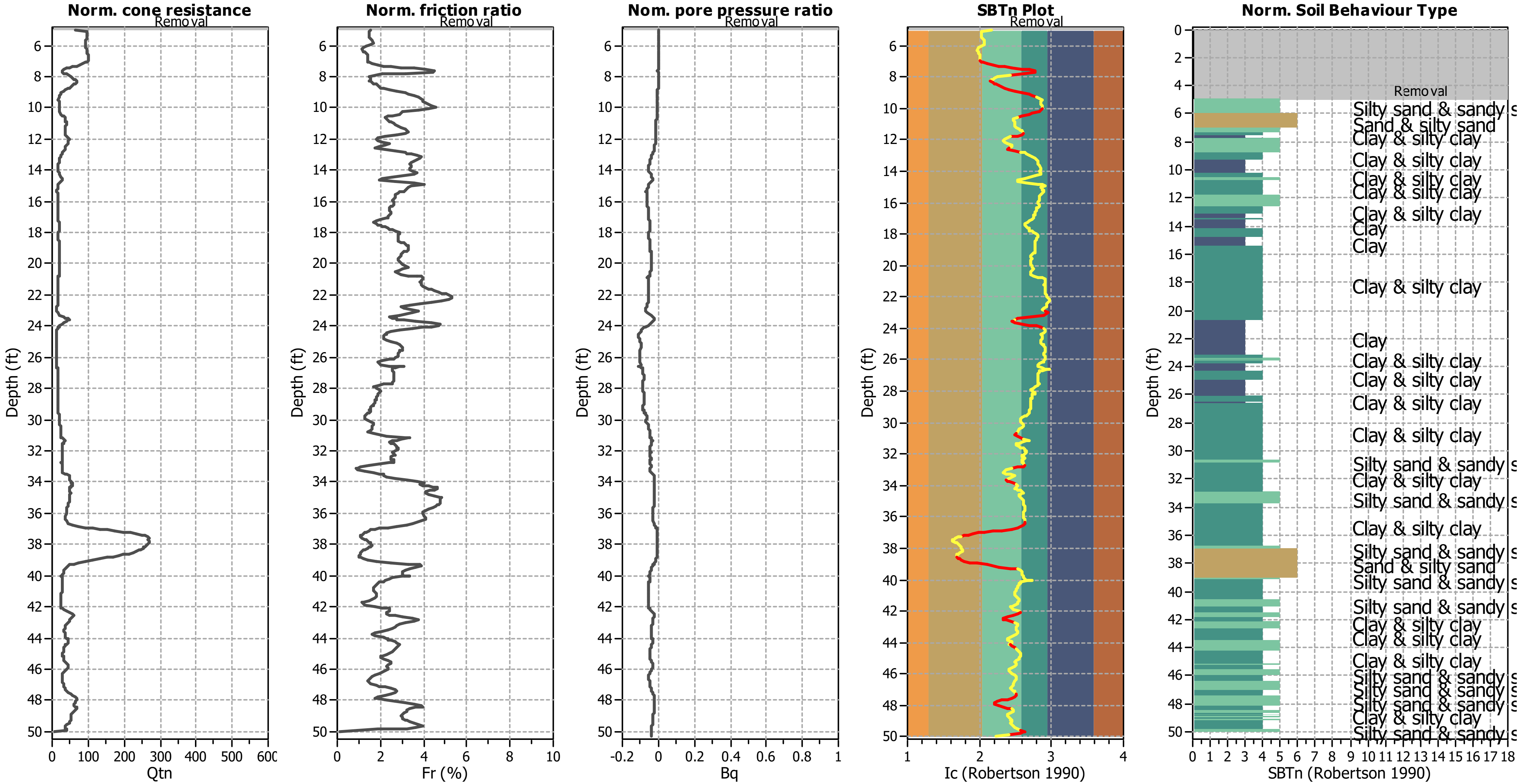






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and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

CPT basic interpretation plots (normalized)



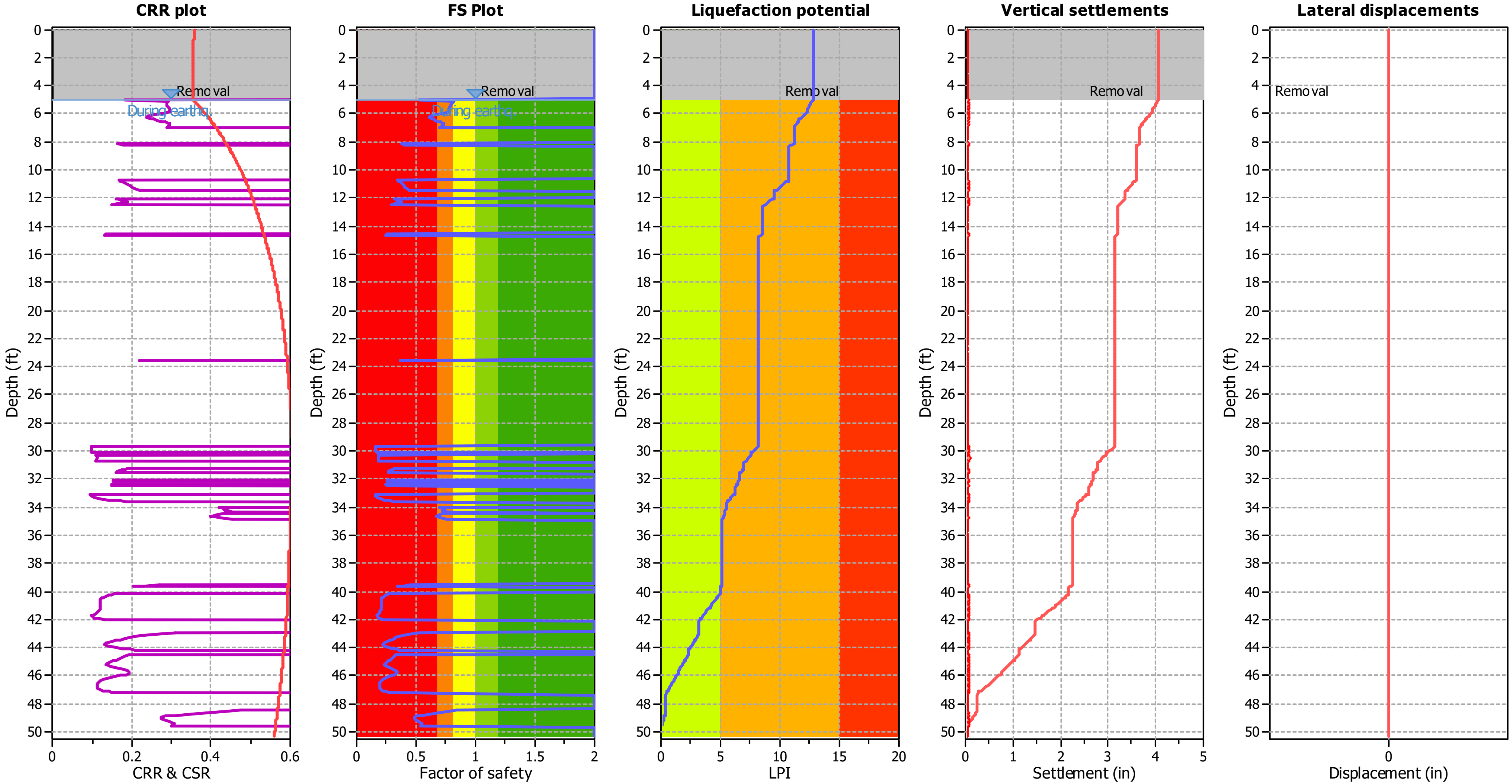
Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	5.00 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:	7.15	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.62	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	10.00 ft	Fill height:	N/A	Limit depth:	N/A

SBTn legend

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

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	5.00 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:	7.15	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.62	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	10.00 ft	Fill height:	N/A	Limit depth:	N/A

F.S. color scheme

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

LPI color scheme

- Very high risk
- High risk
- Low risk

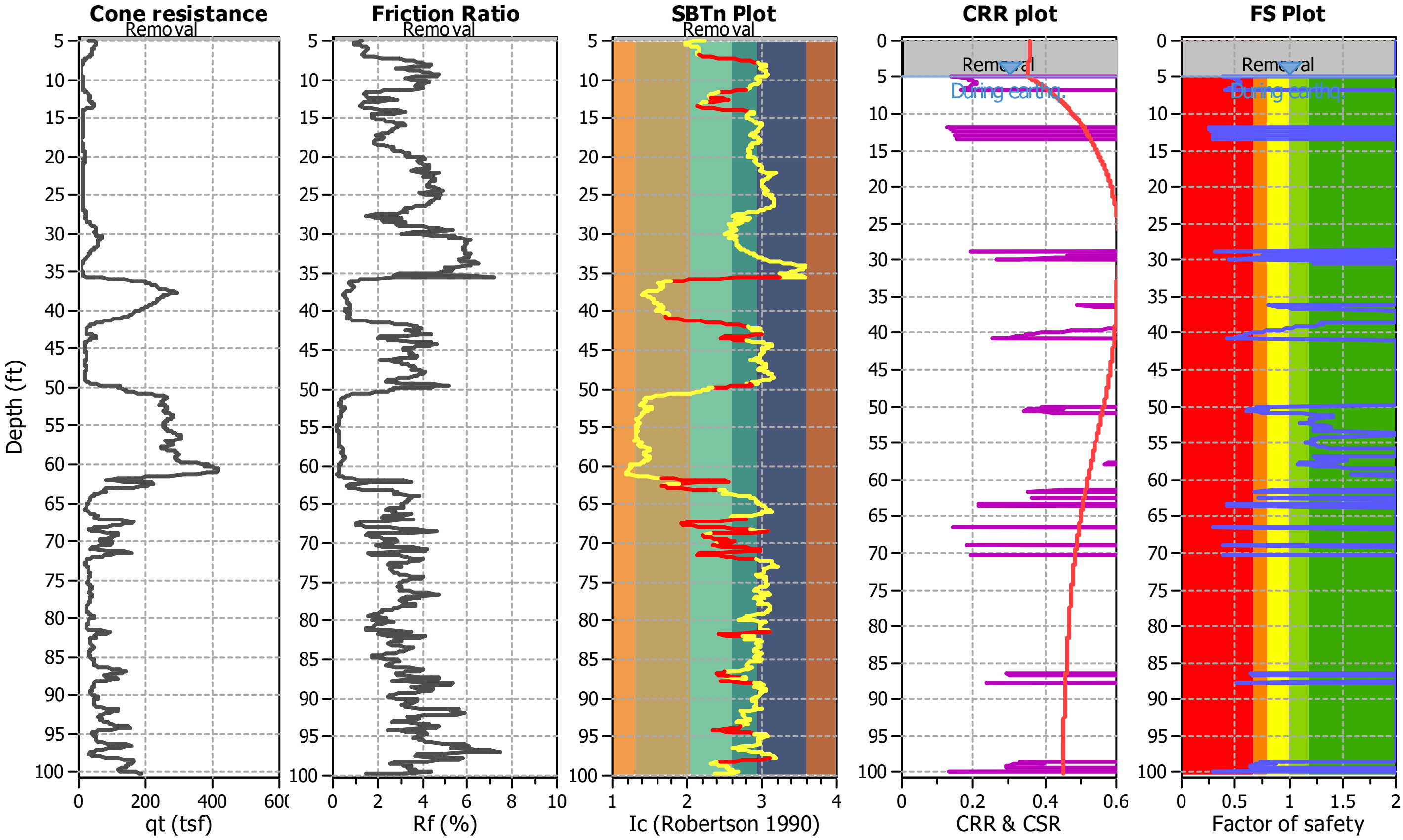
LIQUEFACTION ANALYSIS REPORT

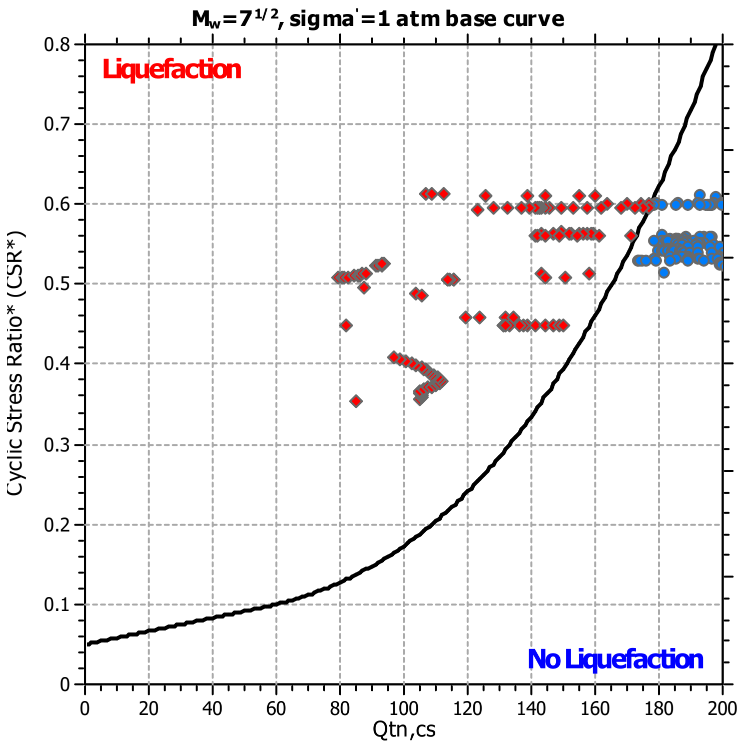
Project title : Segerstrom/ Lake Center Office Park **Location : Santa Ana, California**

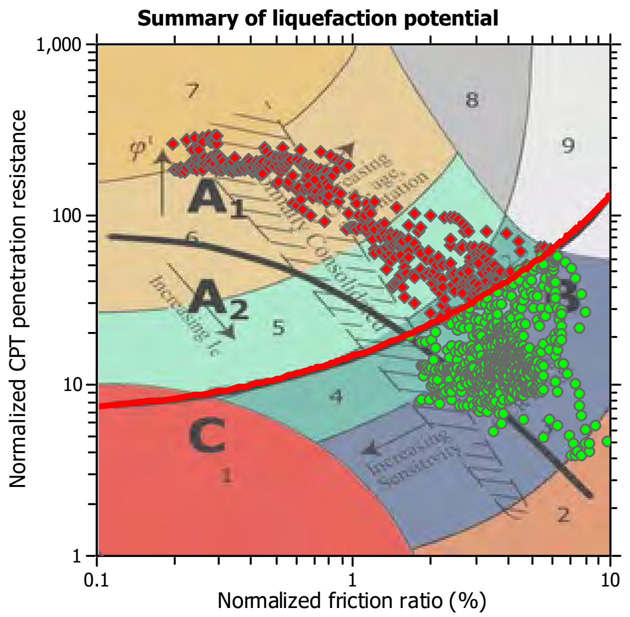
CPT file : CPT-2

Input parameters and analysis data

Analysis method:	NCEER (1998)	G.W.T. (in-situ):	10.00 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	5.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	7.15	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.62	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

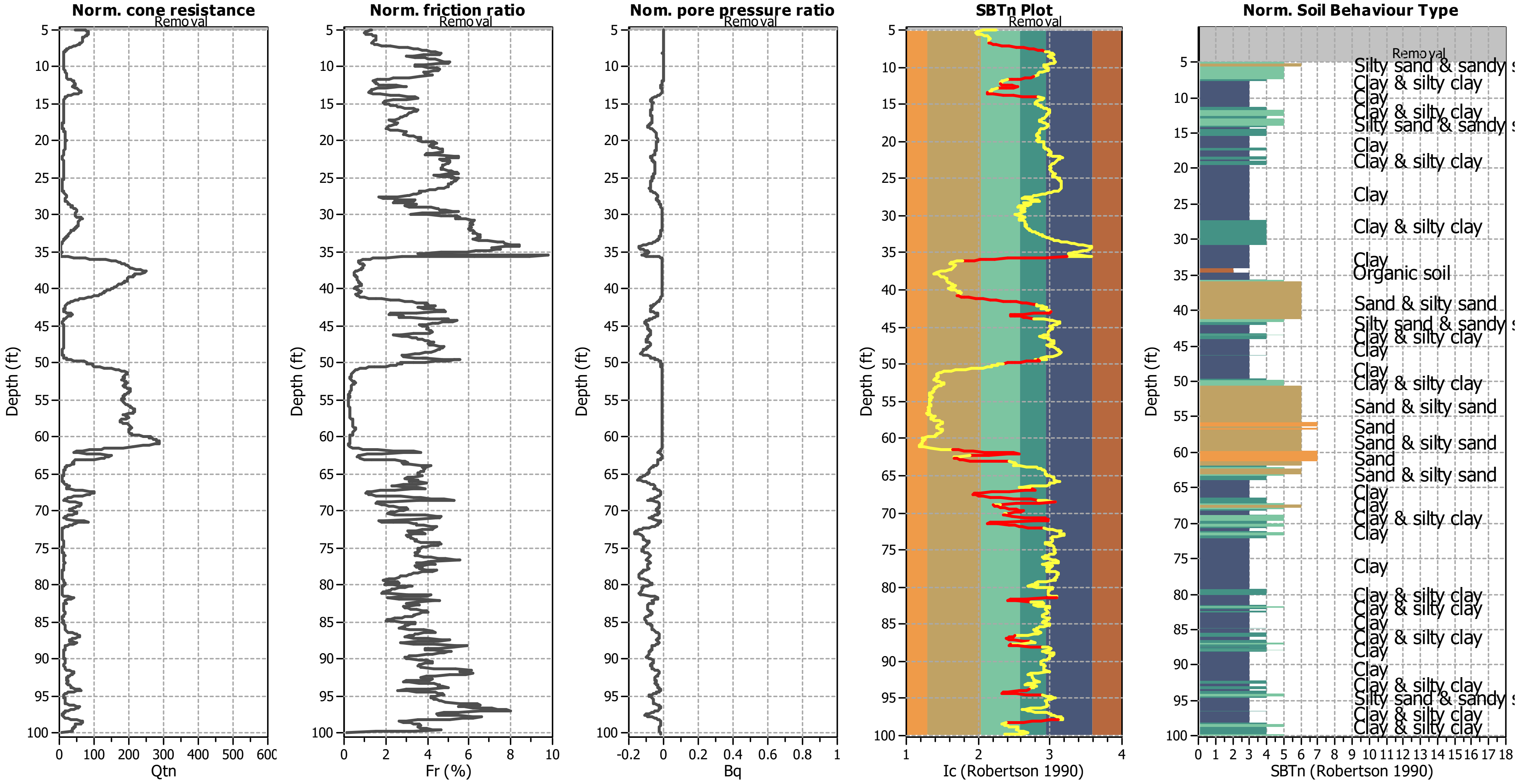






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and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

CPT basic interpretation plots (normalized)



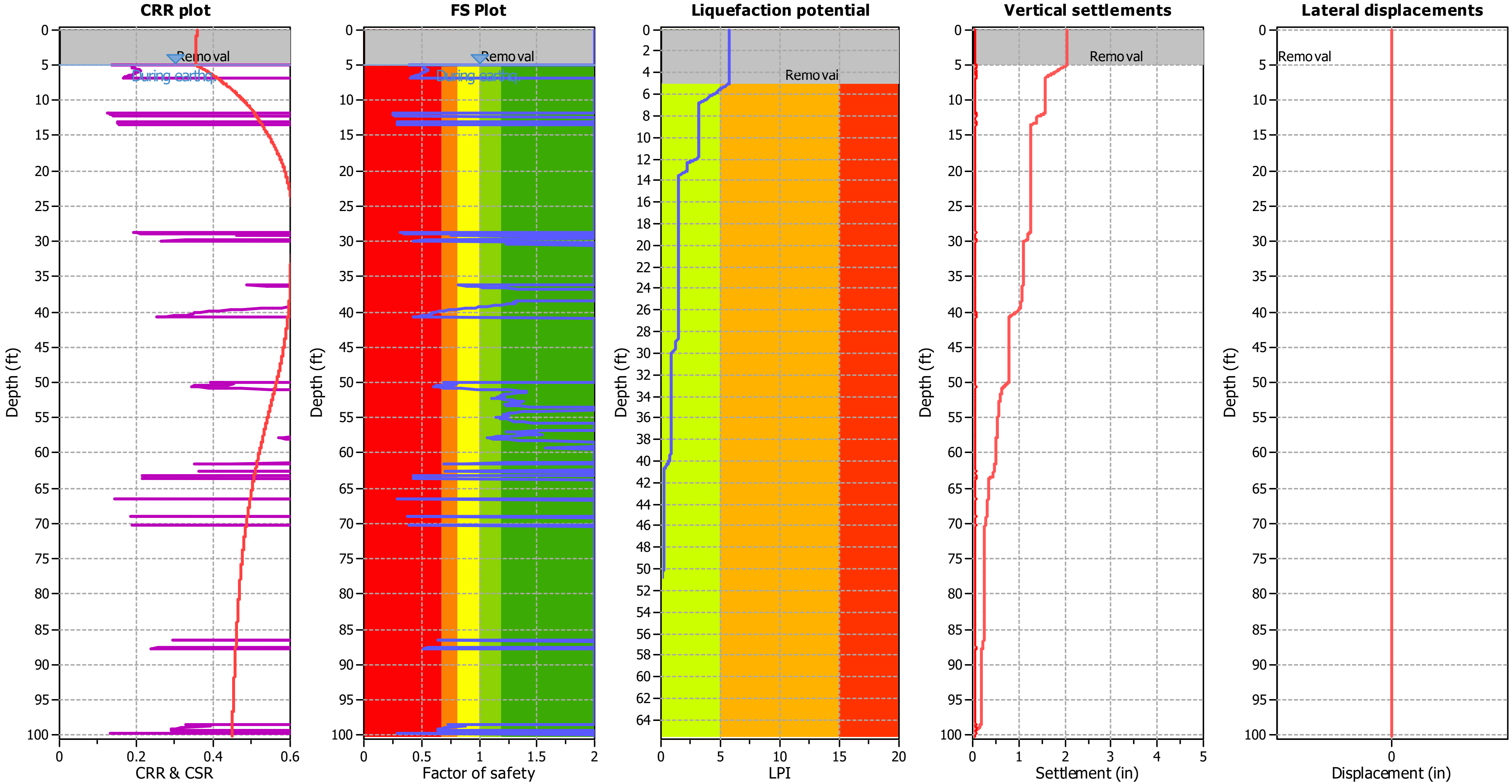
Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	5.00 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:	7.15	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.62	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	10.00 ft	Fill height:	N/A	Limit depth:	N/A

SBTn legend

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

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	5.00 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:	7.15	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.62	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	10.00 ft	Fill height:	N/A	Limit depth:	N/A

F.S. color scheme

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

LPI color scheme

- Very high risk
- High risk
- Low risk

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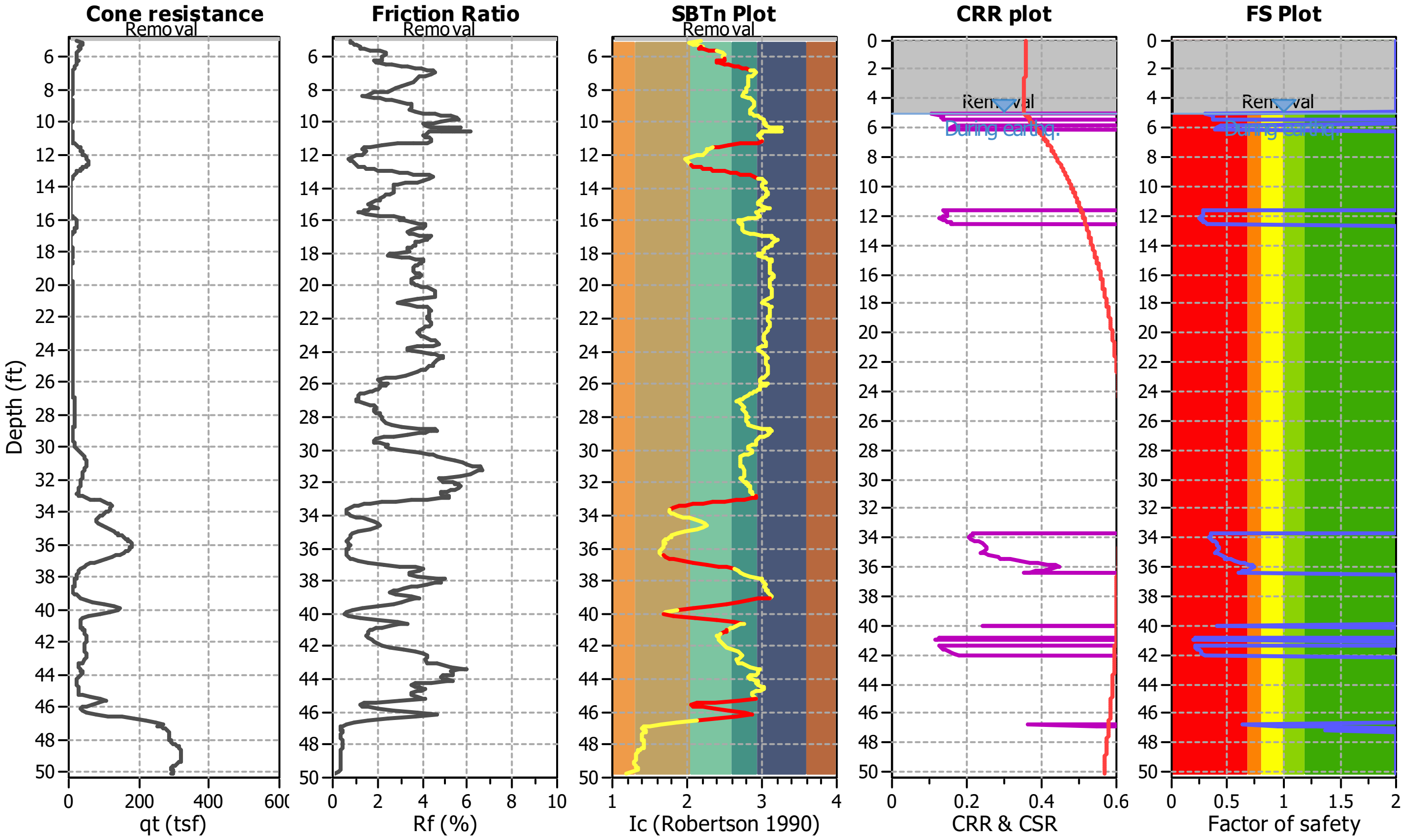
LIQUEFACTION ANALYSIS REPORT

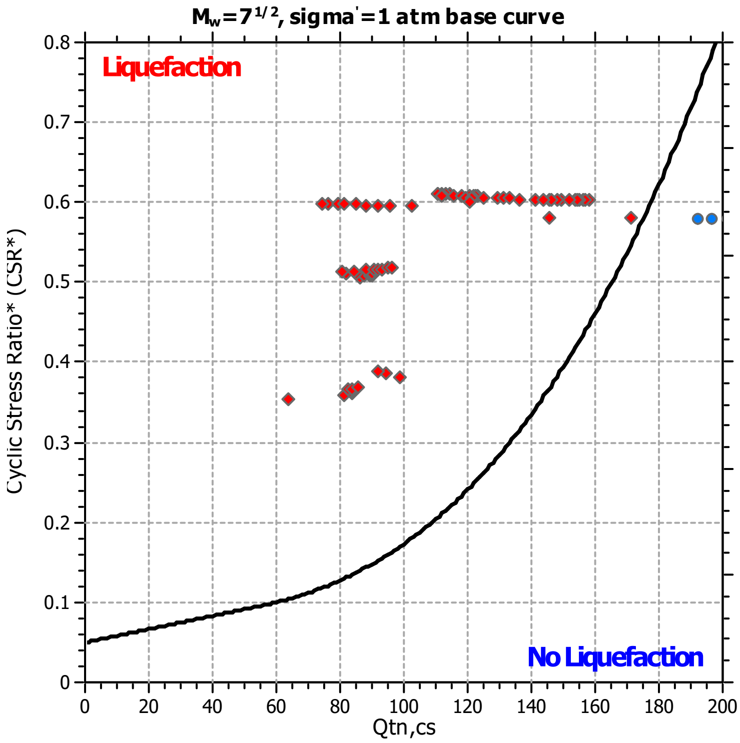
Project title : Segerstrom/ Lake Center Office Park **Location : Santa Ana, California**

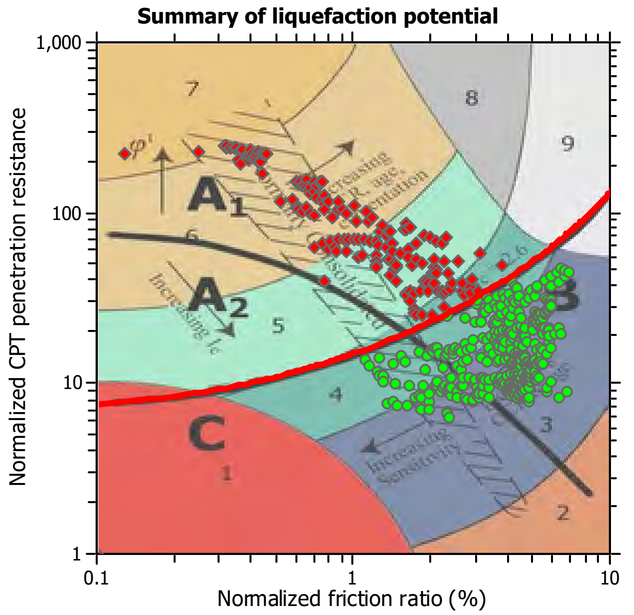
CPT file : CPT-3

Input parameters and analysis data

Analysis method:	NCEER (1998)	G.W.T. (in-situ):	10.00 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	5.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	7.15	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.62	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

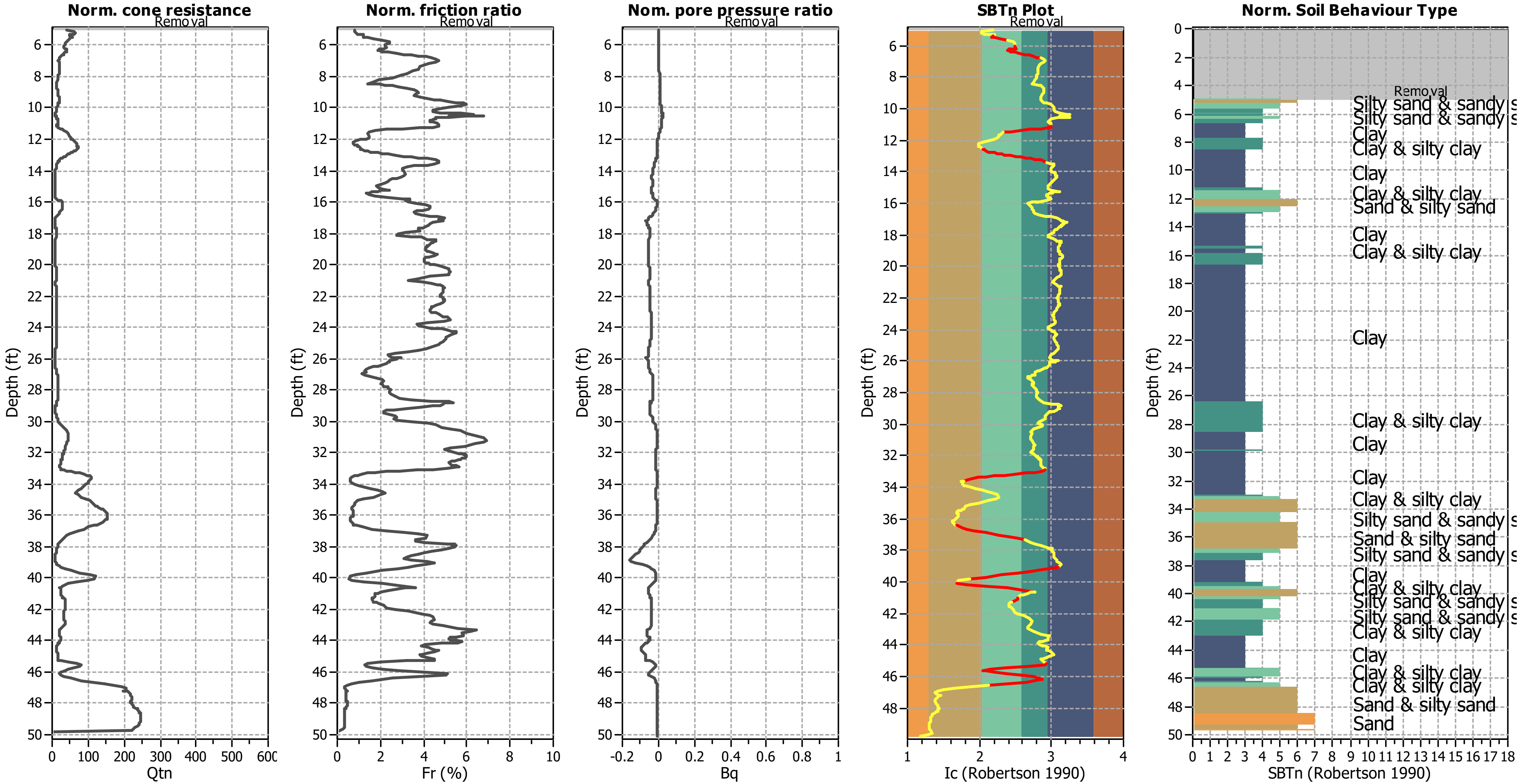






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and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

CPT basic interpretation plots (normalized)



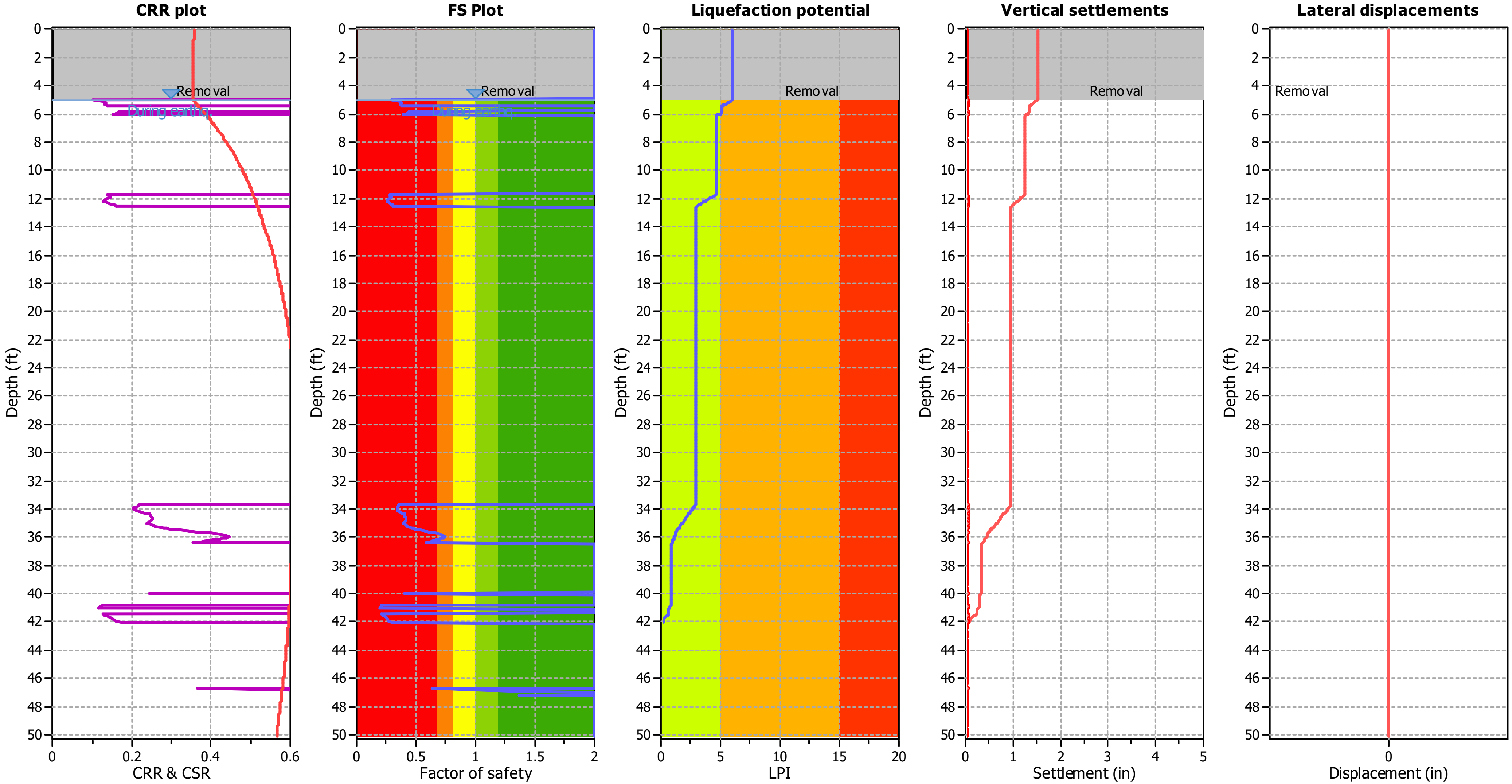
Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	5.00 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:	7.15	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.62	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	10.00 ft	Fill height:	N/A	Limit depth:	N/A

SBTn legend

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

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	5.00 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:	7.15	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.62	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	10.00 ft	Fill height:	N/A	Limit depth:	N/A

F.S. color scheme

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

LPI color scheme

- Very high risk
- High risk
- Low risk

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Irvine, CA 92614

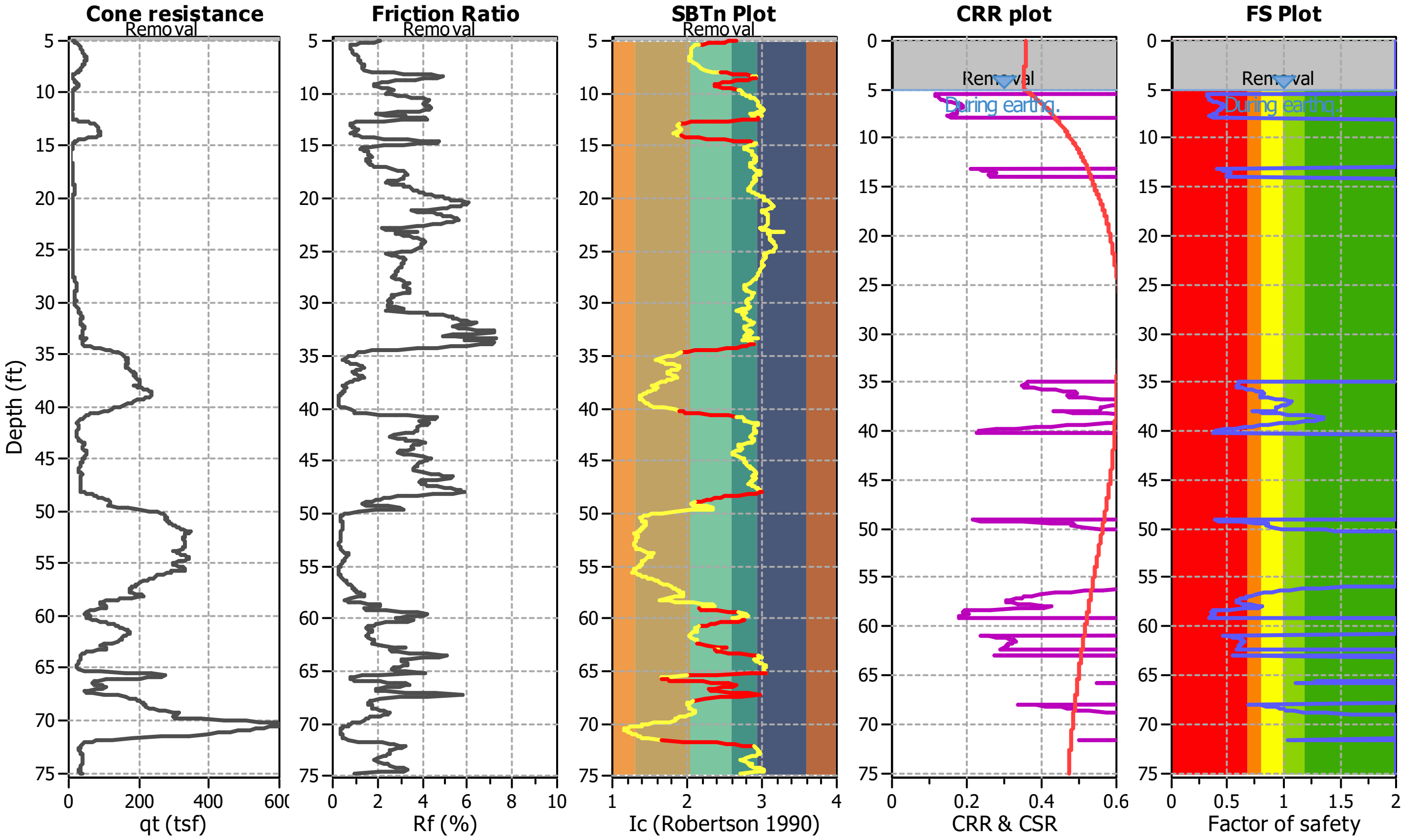
LIQUEFACTION ANALYSIS REPORT

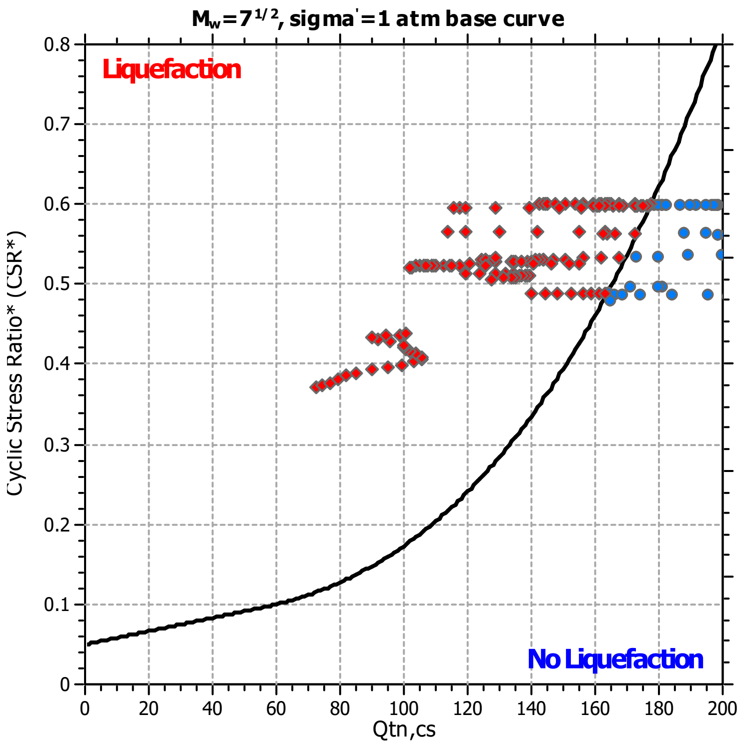
Project title : Segerstrom/ Lake Center Office Park **Location : Santa Ana, California**

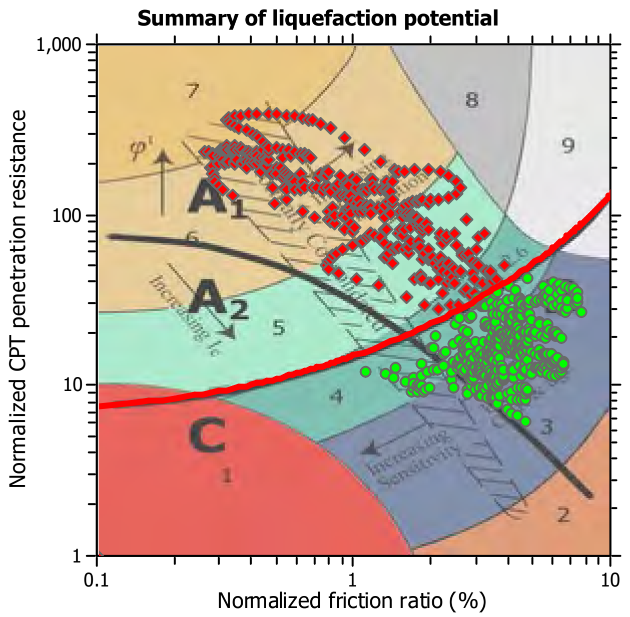
CPT file : CPT-4

Input parameters and analysis data

Analysis method:	NCEER (1998)	G.W.T. (in-situ):	10.00 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	5.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	7.15	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.62	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

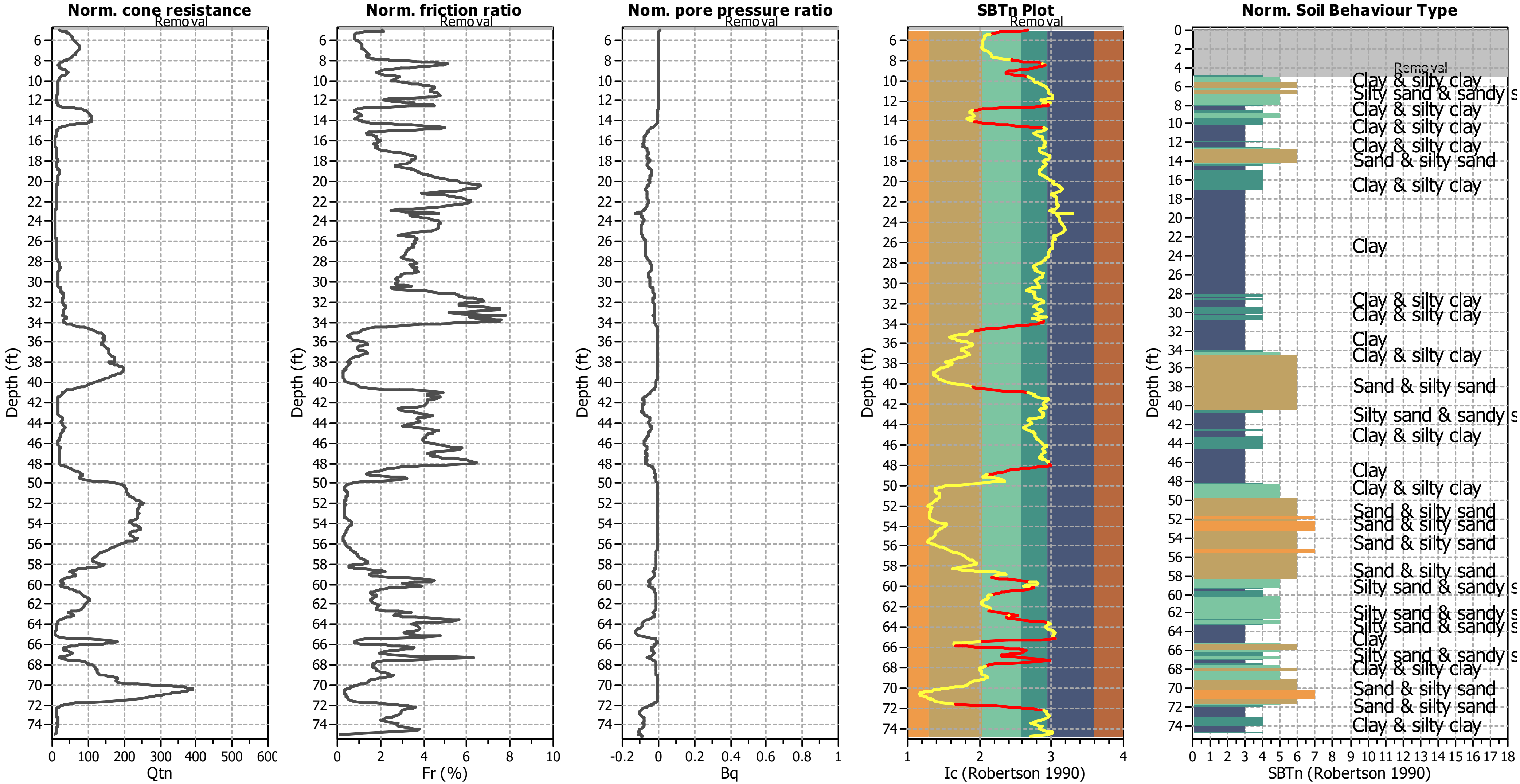






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and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

CPT basic interpretation plots (normalized)



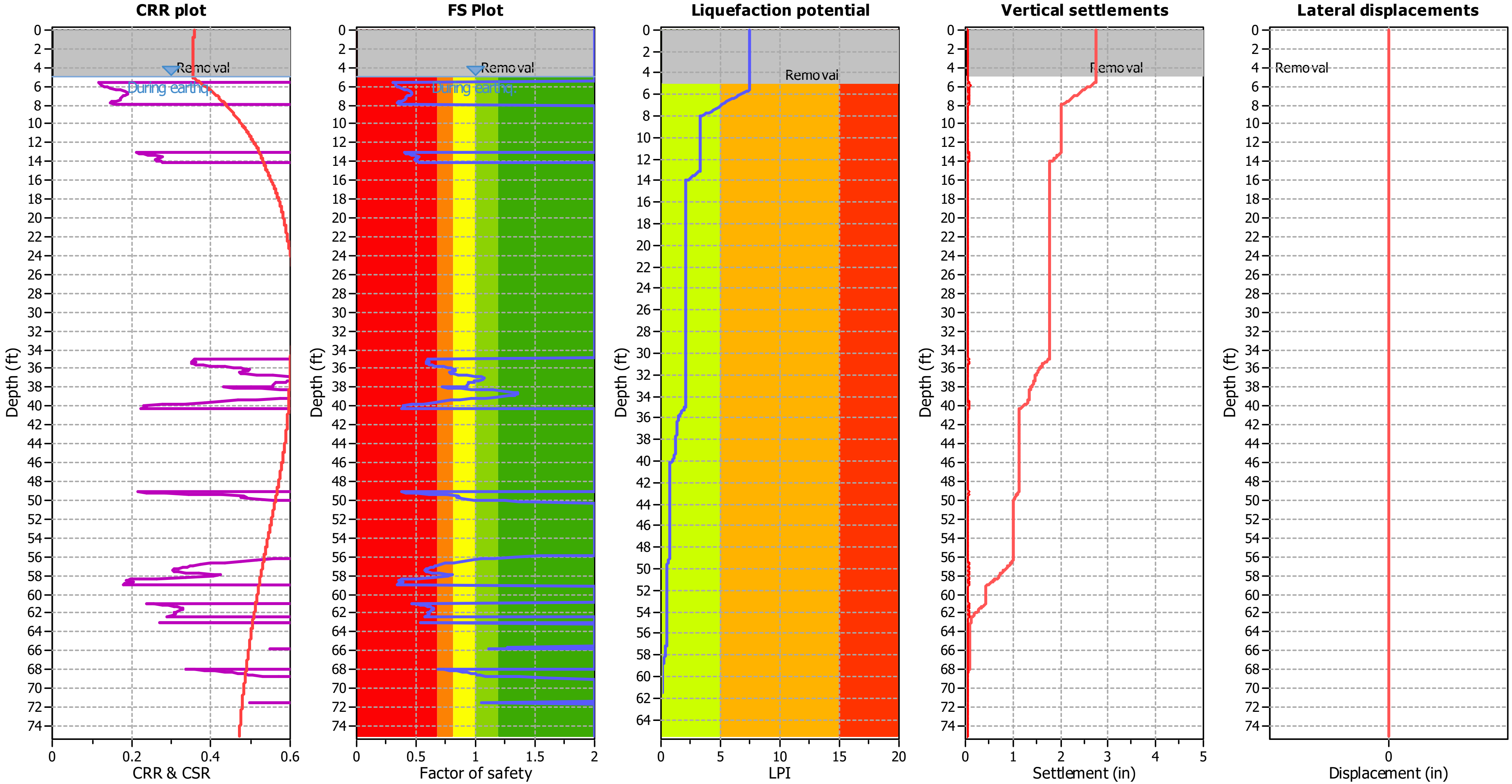
Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	5.00 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:	7.15	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.62	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	10.00 ft	Fill height:	N/A	Limit depth:	N/A

SBTn legend

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

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	5.00 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:	7.15	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.62	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	10.00 ft	Fill height:	N/A	Limit depth:	N/A

F.S. color scheme

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

LPI color scheme

- Very high risk
- High risk
- Low risk

NMG Geotechnical, Inc.
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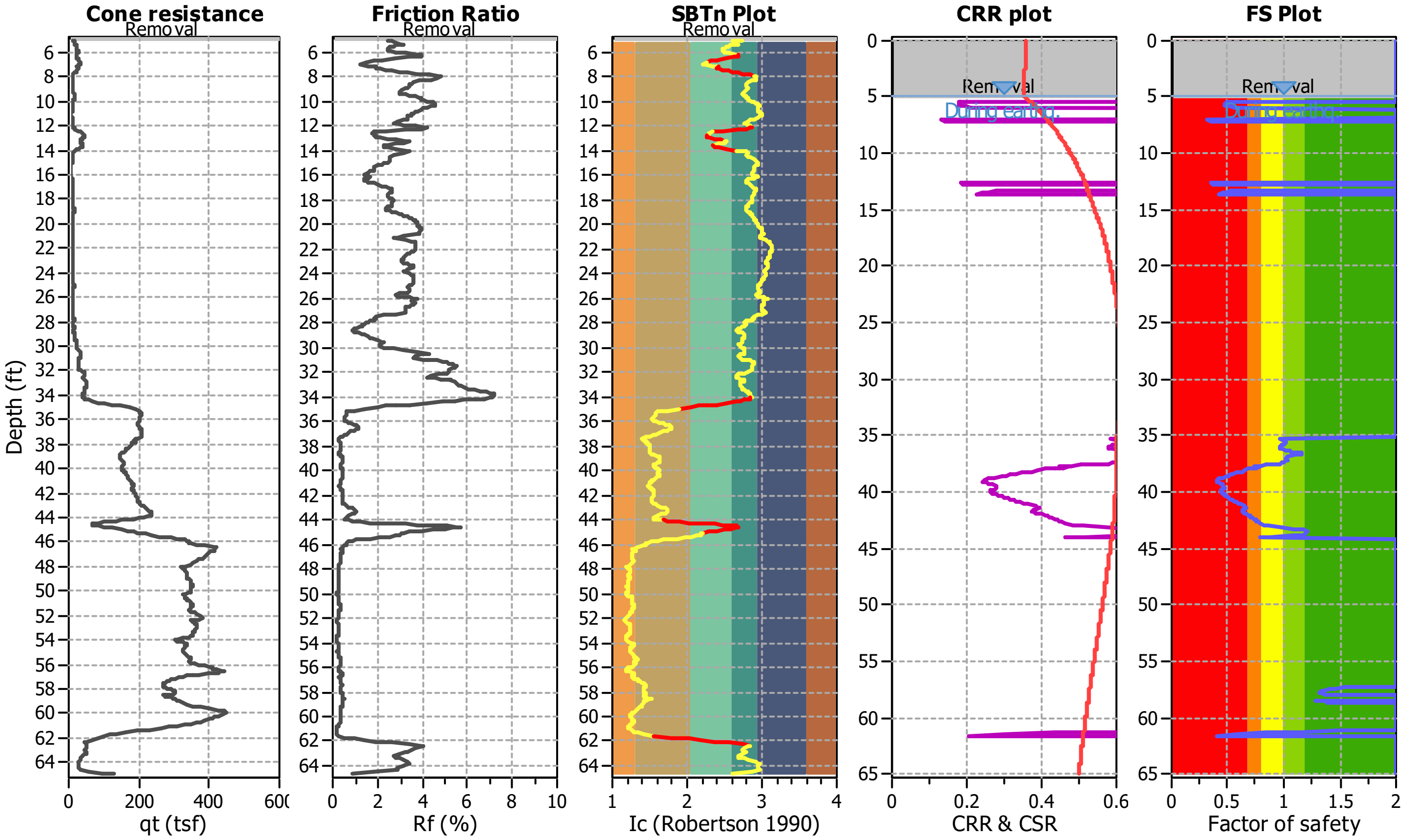
LIQUEFACTION ANALYSIS REPORT

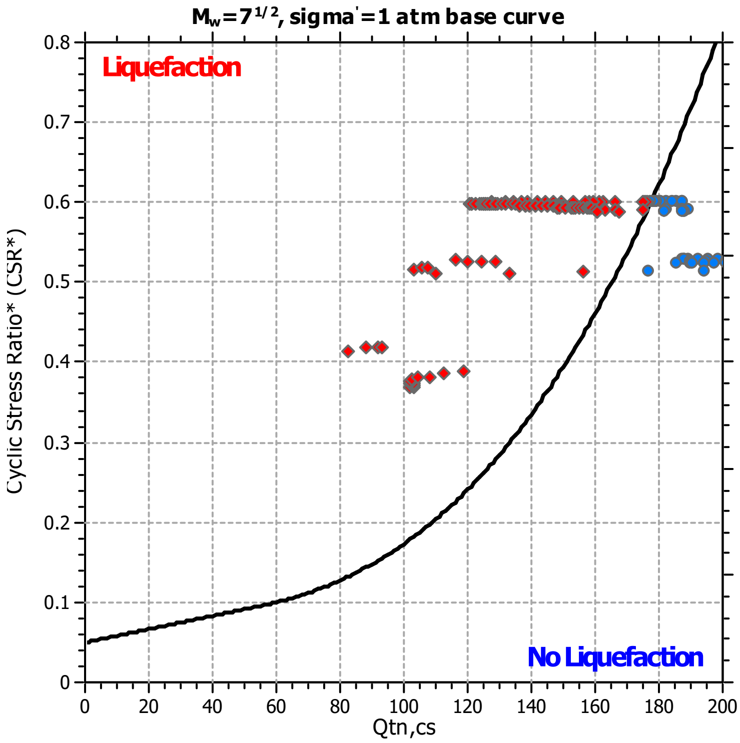
Project title : Segerstrom/ Lake Center Office Park **Location : Santa Ana, California**

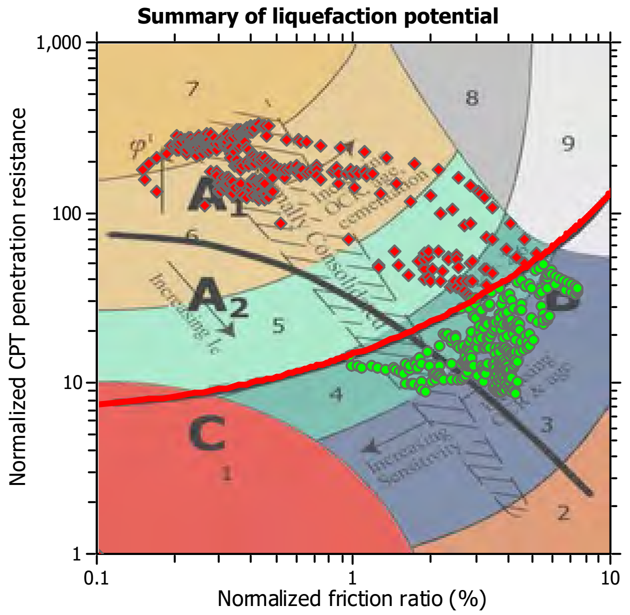
CPT file : CPT-5

Input parameters and analysis data

Analysis method:	NCEER (1998)	G.W.T. (in-situ):	10.00 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	5.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	7.15	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.62	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

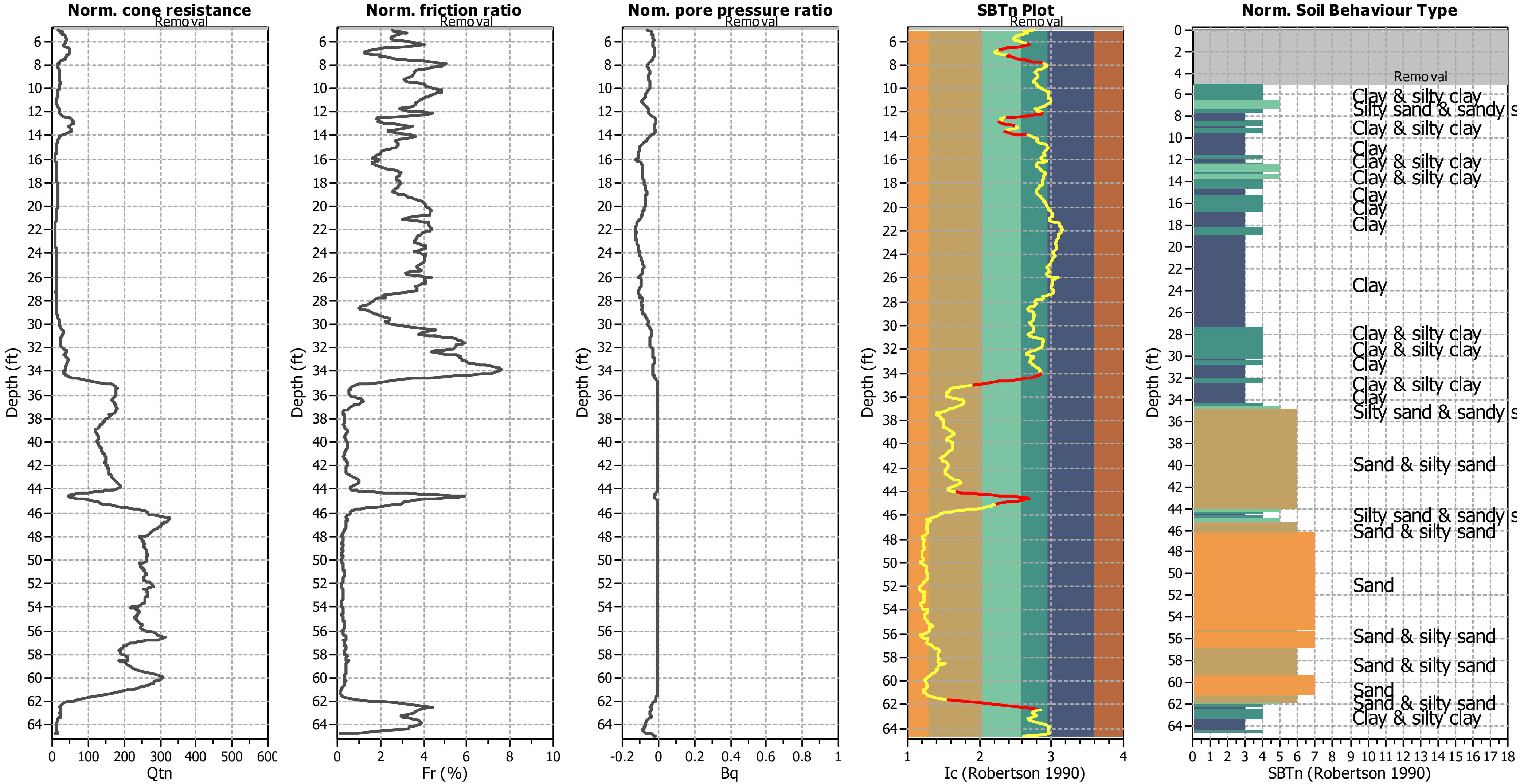






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and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

CPT basic interpretation plots (normalized)



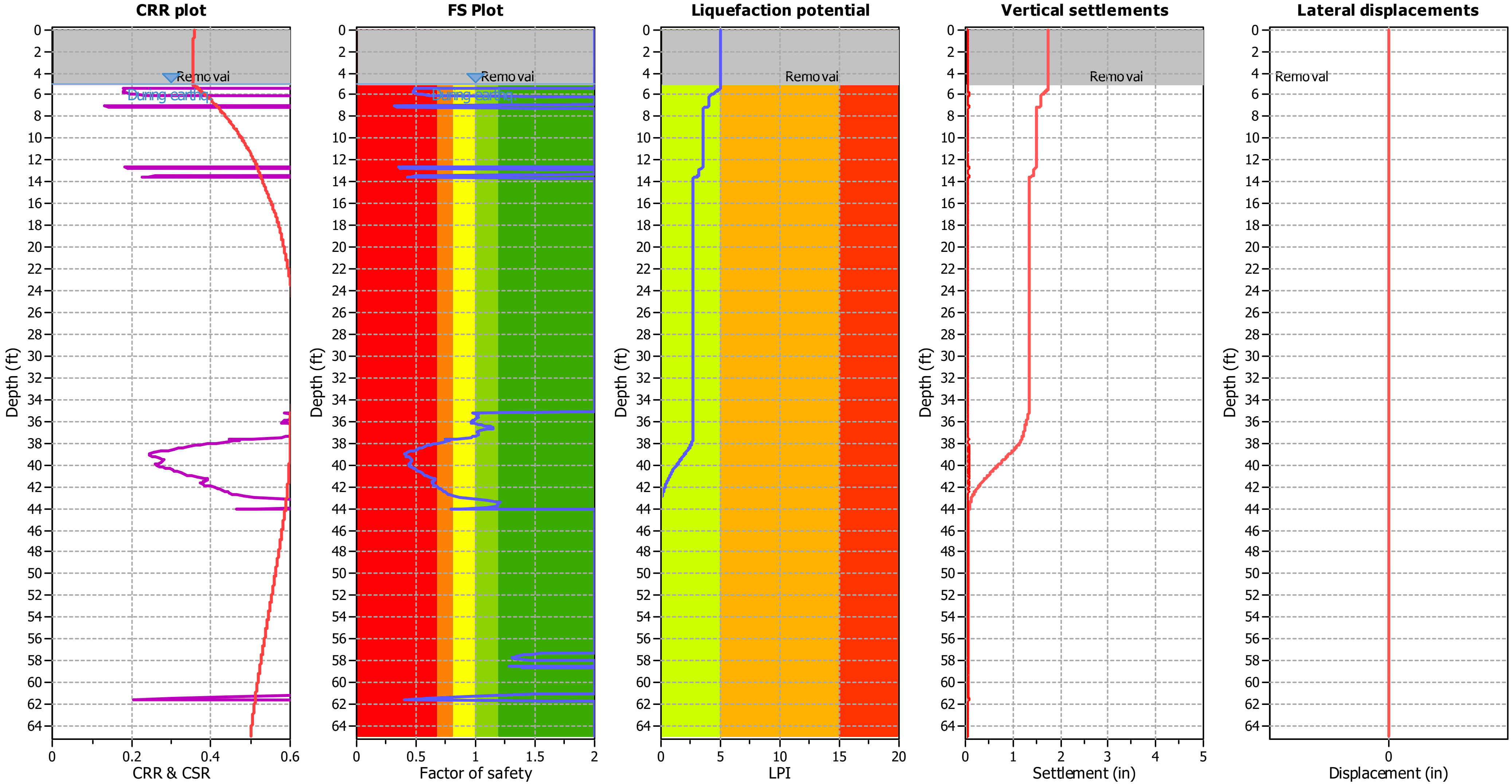
Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	5.00 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:	7.15	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.62	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	10.00 ft	Fill height:	N/A	Limit depth:	N/A

SBTn legend

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

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	5.00 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:	7.15	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.62	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	10.00 ft	Fill height:	N/A	Limit depth:	N/A

F.S. color scheme

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

LPI color scheme

- Very high risk
- High risk
- Low risk

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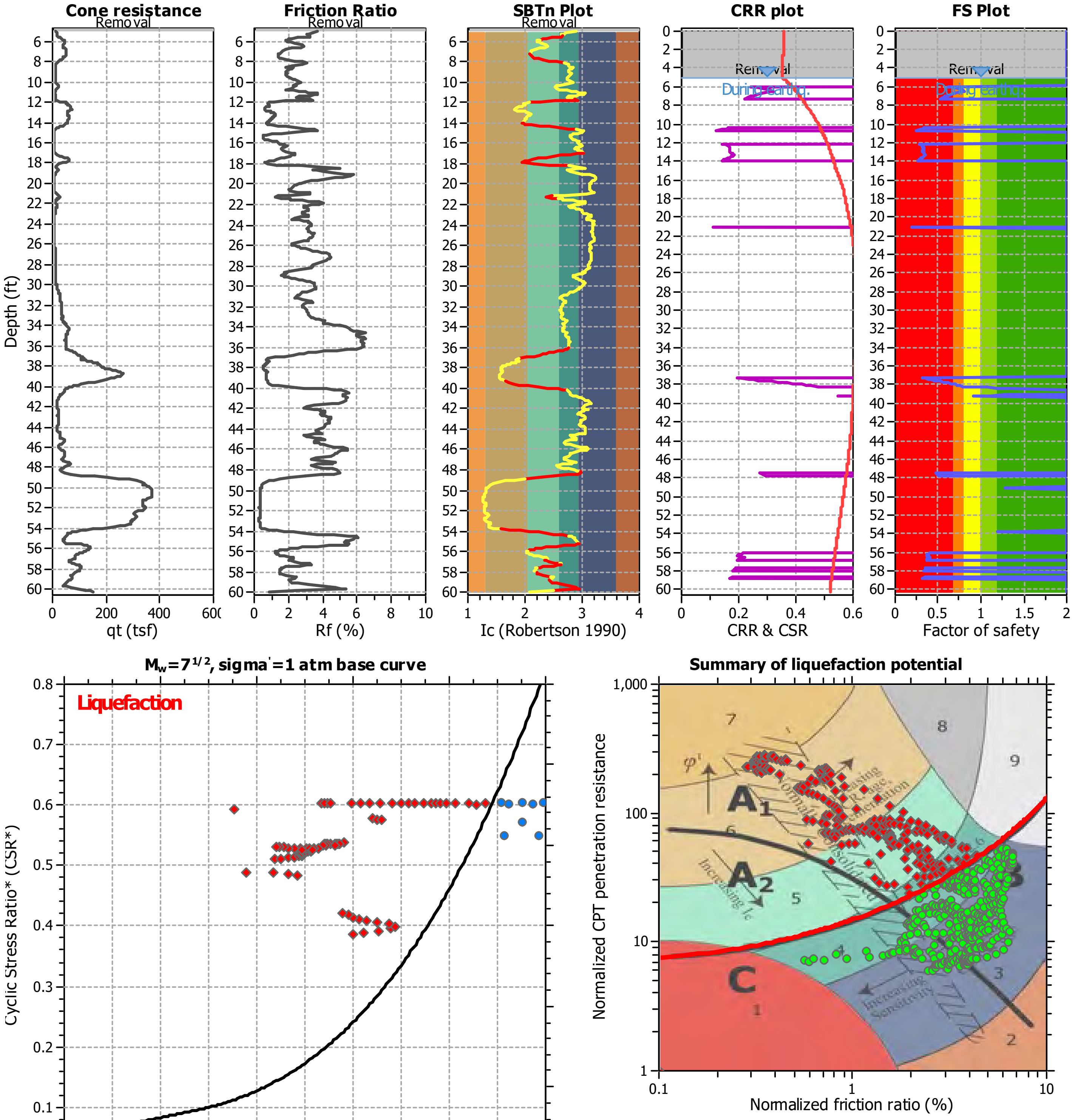
LIQUEFACTION ANALYSIS REPORT

Project title : Segerstrom/ Lake Center Office Park **Location : Santa Ana, California**

CPT file : CPT-6

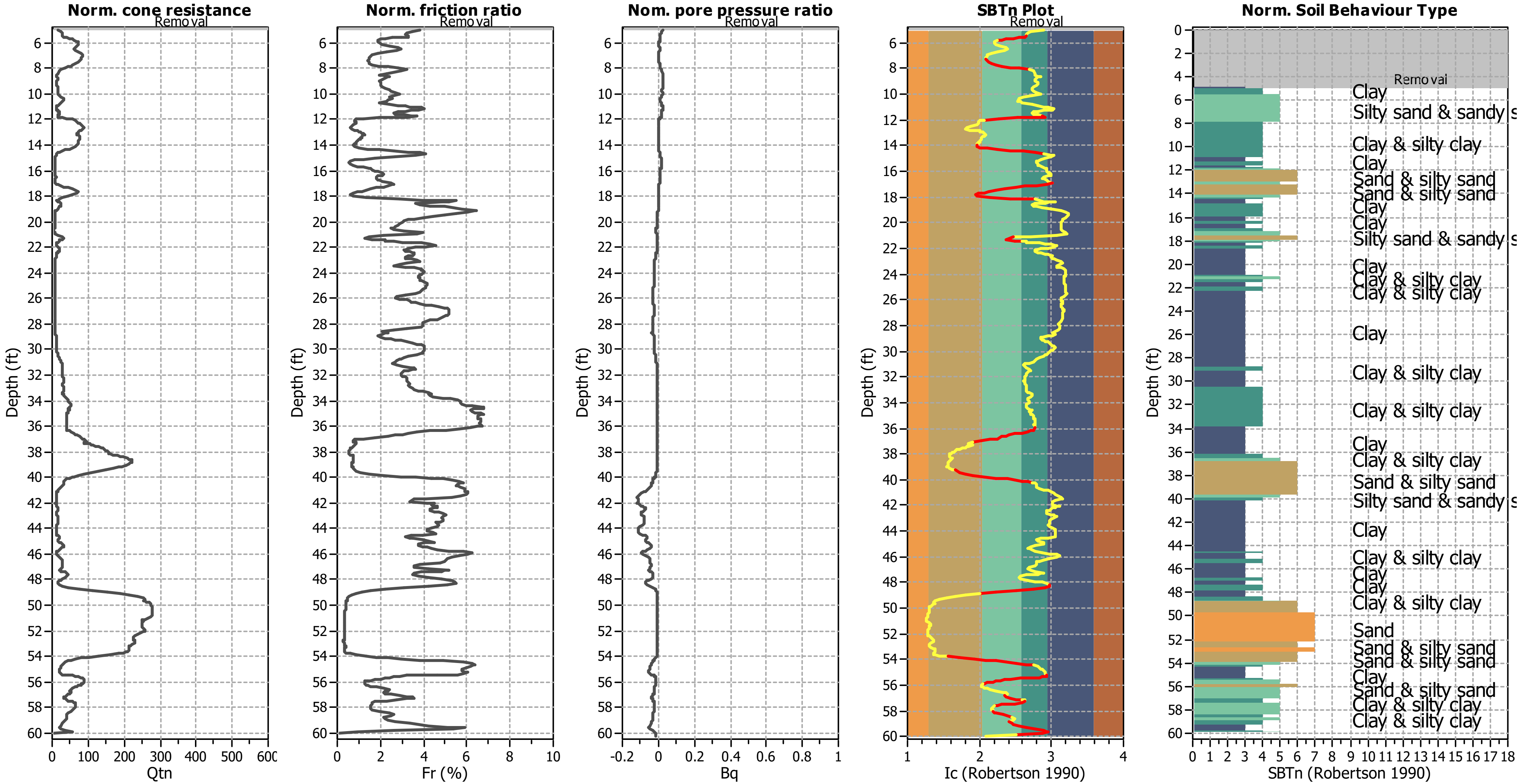
Input parameters and analysis data

Analysis method:	NCEER (1998)	G.W.T. (in-situ):	10.00 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	5.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	7.15	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.62	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		



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

CPT basic interpretation plots (normalized)



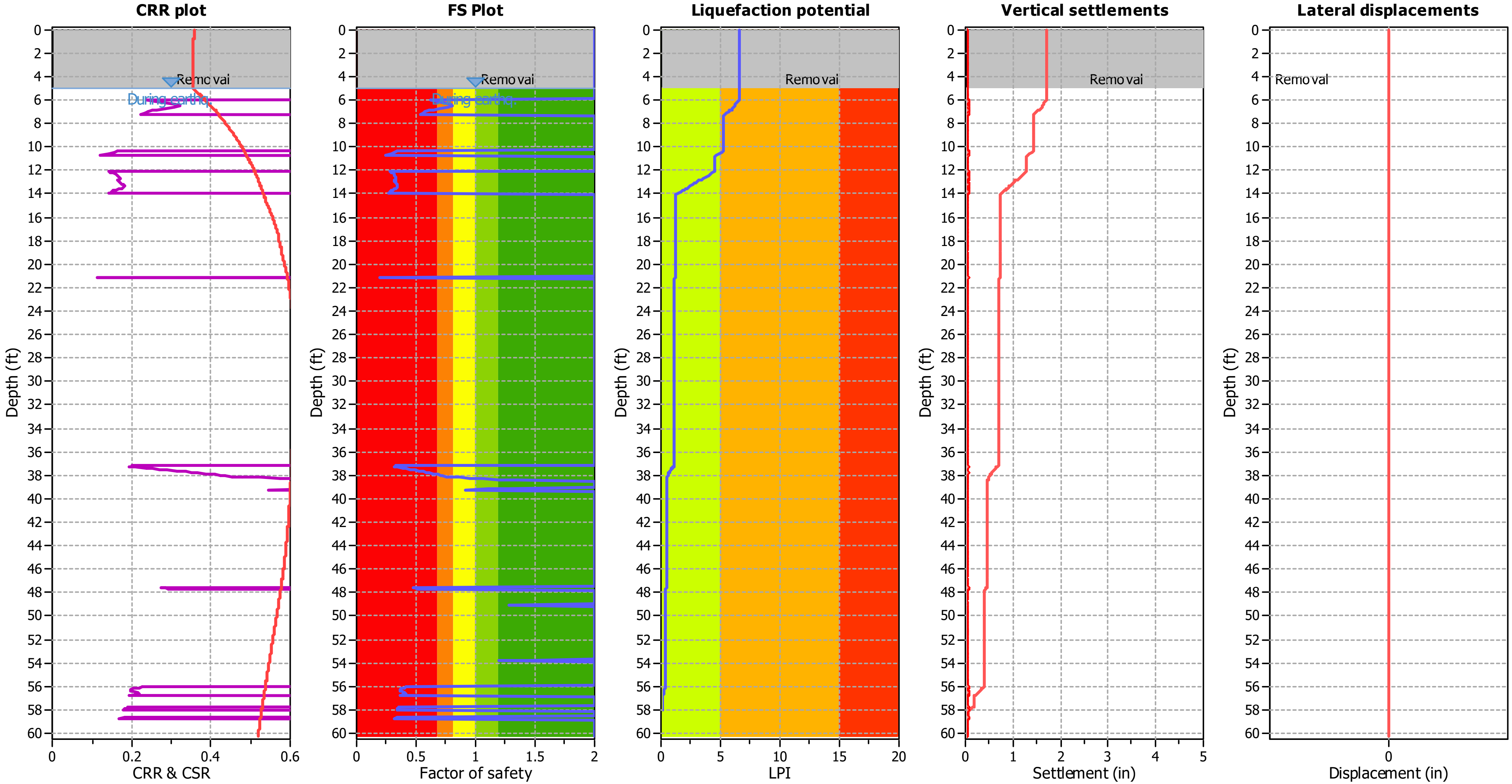
Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	5.00 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:	7.15	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.62	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	10.00 ft	Fill height:	N/A	Limit depth:	N/A

SBTn legend

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

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	5.00 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:	7.15	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.62	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	10.00 ft	Fill height:	N/A	Limit depth:	N/A

F.S. color scheme

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

LPI color scheme

- Very high risk
- High risk
- Low risk

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Irvine, CA 92614

LIQUEFACTION ANALYSIS REPORT

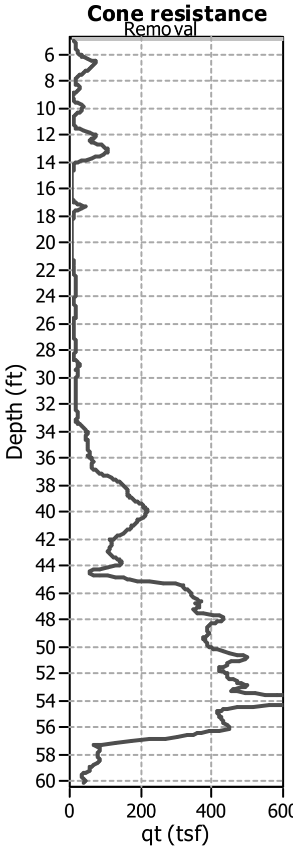
Project title : Segerstrom/ Lake Center Office Park **Location : Santa Ana, California**

CPT file : CPT-7

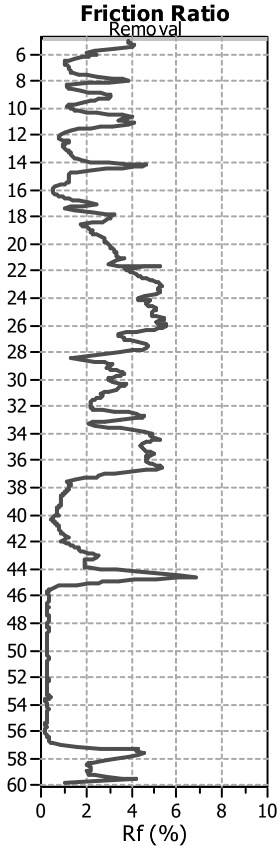
Input parameters and analysis data

Analysis method:	NCEER (1998)	G.W.T. (in-situ):	10.00 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	5.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	7.15	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.62	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

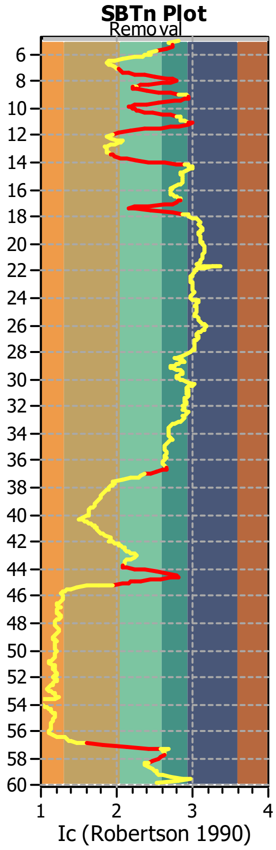
Cone resistance



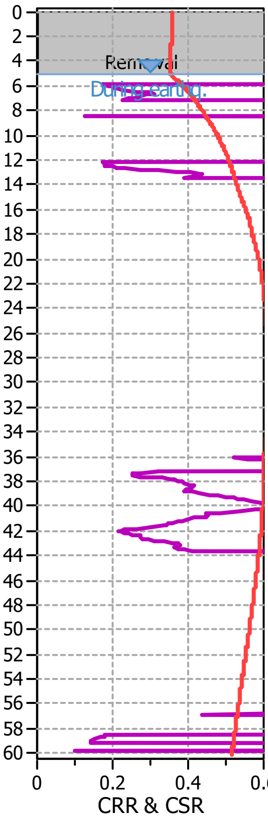
Friction Ratio



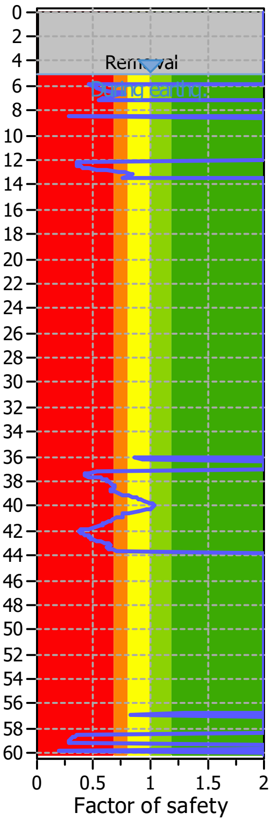
SBTn Plot



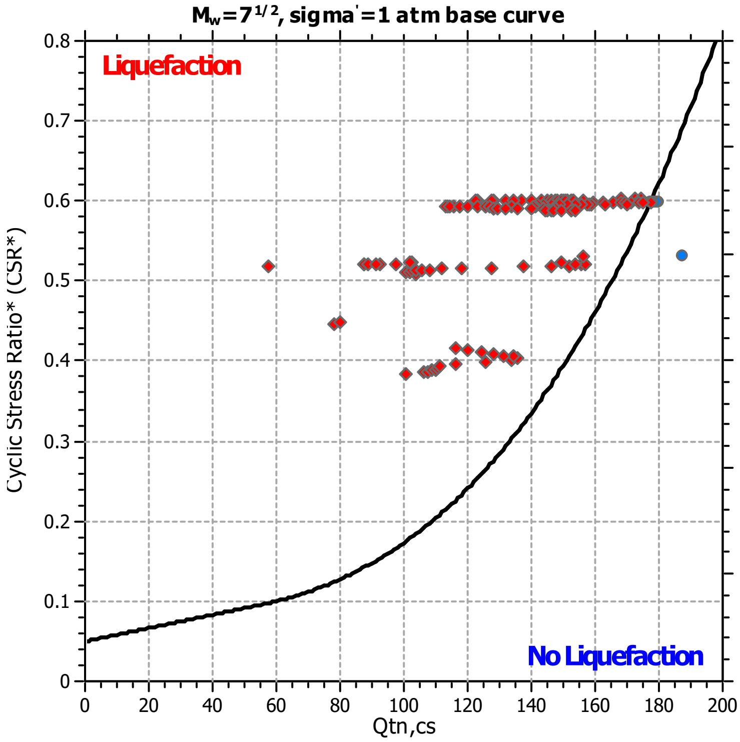
CRR plot



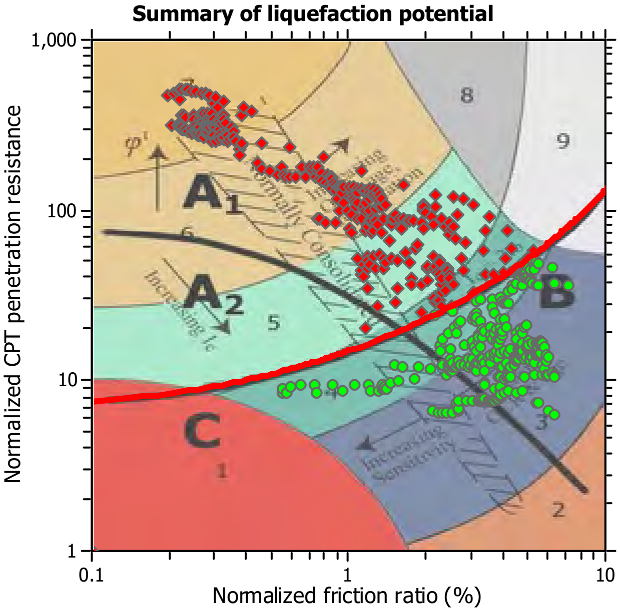
FS Plot



$M_w=7^{1/2}$, sigma'=1 atm base curve

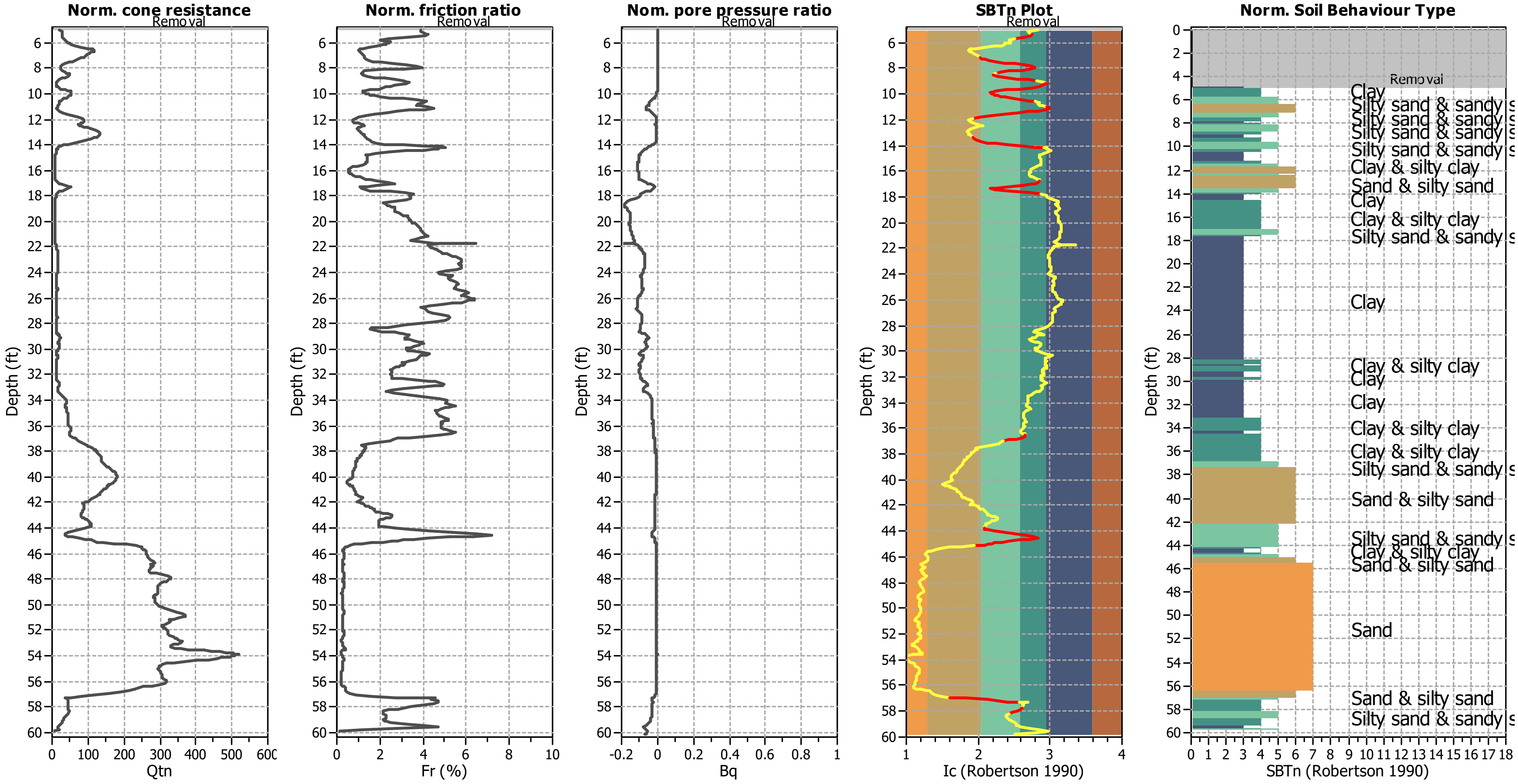


Summary of liquefaction potential



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

CPT basic interpretation plots (normalized)



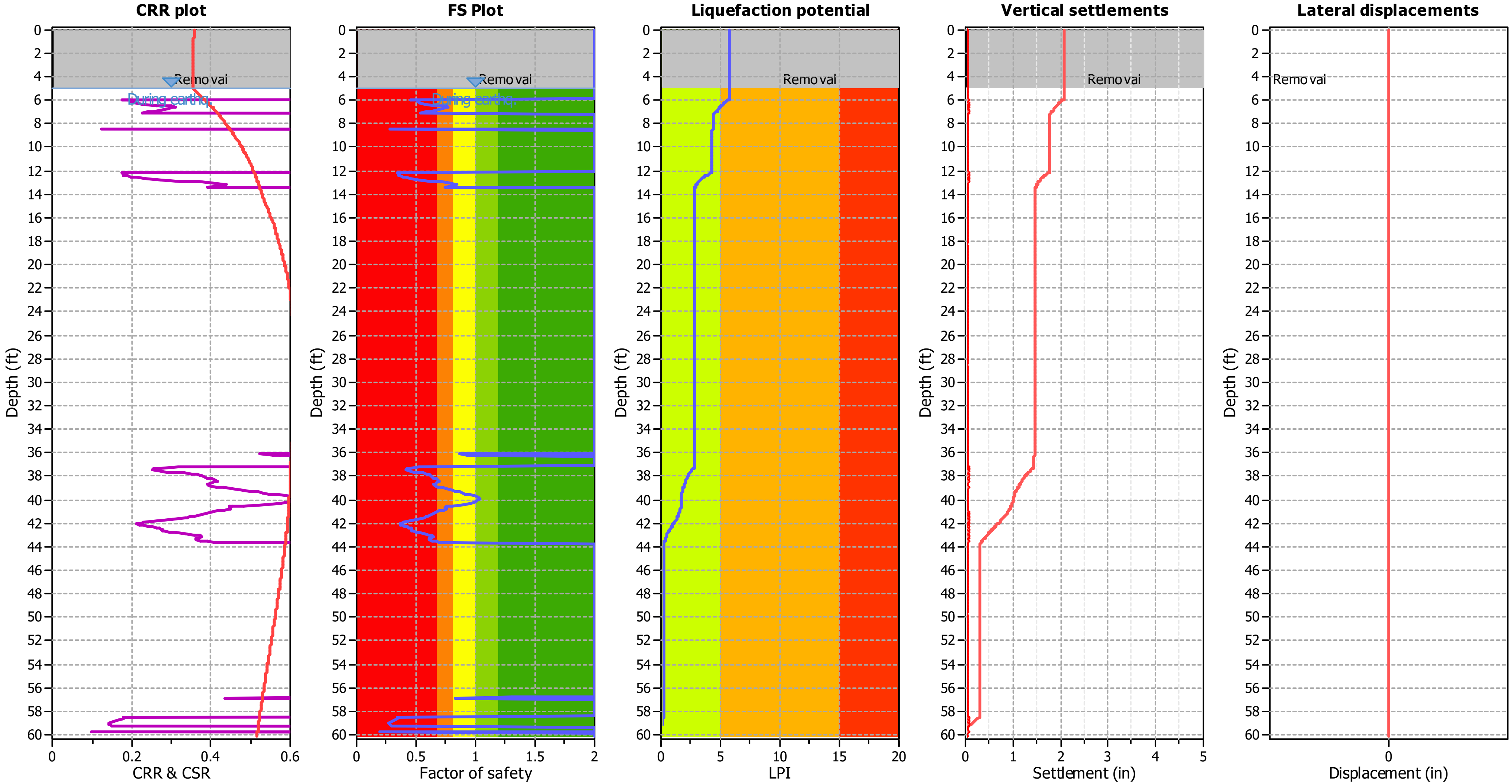
Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	5.00 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:	7.15	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.62	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	10.00 ft	Fill height:	N/A	Limit depth:	N/A

SBTn legend

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

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	5.00 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:	7.15	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.62	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	10.00 ft	Fill height:	N/A	Limit depth:	N/A

F.S. color scheme

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

LPI color scheme

- Very high risk
- High risk
- Low risk

APPENDIX F

APPENDIX F

GENERAL EARTHWORK AND GRADING SPECIFICATIONS

1.0 General

1.1 Intent: These General Earthwork and Grading Specifications are for the grading and earthwork shown on the approved grading plan(s) and/or indicated in the geotechnical report(s). These Specifications are a part of the recommendations contained in the geotechnical report(s). In case of conflict, the specific recommendations in the geotechnical report shall supersede these more general Specifications. Observations of the earthwork by the project Geotechnical Consultant during the course of grading may result in new or revised recommendations that could supersede these specifications or the recommendations in the geotechnical report(s).

1.2 Geotechnical Consultant: Prior to commencement of work, the owner shall employ a geotechnical consultant. The geotechnical consultant shall be responsible for reviewing the approved geotechnical report(s) and accepting the adequacy of the preliminary geotechnical findings, conclusions, and recommendations prior to the commencement of the grading.

Prior to commencement of grading, the Geotechnical Consultant shall review the "work plan" prepared by the Earthwork Contractor (Contractor) and schedule sufficient personnel to perform the appropriate level of observation, mapping, and compaction testing.

During the grading and earthwork operations, the Geotechnical Consultant shall observe, map, and document the subsurface exposures to verify the geotechnical design assumptions. If the observed conditions are found to be significantly different than the interpreted assumptions during the design phase, the Geotechnical Consultant shall inform the owner, recommend appropriate changes in design to accommodate the observed conditions, and notify the review agency where required. Subsurface areas to be geotechnically observed, mapped, elevations recorded, and/or tested include natural ground after it has been cleared for receiving fill but before fill is placed, bottoms of all "remedial removal" areas, all key bottoms, and benches made on sloping ground to receive fill.

The Geotechnical Consultant shall observe the moisture-conditioning and processing of the subgrade and fill materials and perform relative compaction testing of fill to determine the attained level of compaction. The Geotechnical Consultant shall provide the test results to the owner and the Contractor on a routine and frequent basis.

1.3 The Earthwork Contractor: The Earthwork Contractor (Contractor) shall be qualified, experienced, and knowledgeable in earthwork logistics, preparation and processing of ground to receive fill, moisture-conditioning and processing of fill, and compacting fill. The Contractor shall review and accept the plans, geotechnical report(s), and these Specifications prior to commencement of grading. The Contractor shall be solely responsible for performing the grading in accordance with the plans and specifications.

The Contractor shall prepare and submit to the owner and the Geotechnical Consultant a work plan that indicates the sequence of earthwork grading, the number of "spreads" of work and the estimated quantities of daily earthwork contemplated for the site prior to commencement of grading. The Contractor shall inform the owner and the Geotechnical Consultant of changes in work schedules and updates to the work plan at least 24 hours in advance of such changes so that appropriate observations and tests can be planned and accomplished. The Contractor shall not assume that the Geotechnical Consultant is aware of all grading operations.

The Contractor shall have the sole responsibility to provide adequate equipment and methods to accomplish the earthwork in accordance with the applicable grading codes and agency ordinances, these Specifications, and the recommendations in the approved geotechnical report(s) and grading plan(s). If, in the opinion of the Geotechnical Consultant, unsatisfactory conditions, such as unsuitable soil, improper moisture condition, inadequate compaction, insufficient buttress key size, adverse weather, etc., are resulting in a quality of work less than required in these specifications, the Geotechnical Consultant shall reject the work and may recommend to the owner that construction be stopped until the conditions are rectified.

2.0 Preparation of Areas to be Filled

2.1 Clearing and Grubbing: Vegetation, such as brush, grass, roots, and other deleterious material shall be sufficiently removed and properly disposed of in a method acceptable to the owner, governing agencies, and the Geotechnical Consultant.

The Geotechnical Consultant shall evaluate the extent of these removals depending on specific site conditions. Earth fill material shall not contain more than 1 percent of organic materials (by volume). No fill lift shall contain more than 5 percent of organic matter. Nesting of the organic materials shall not be allowed.

If potentially hazardous materials are encountered, the Contractor shall stop work in the affected area, and a hazardous material specialist shall be informed immediately for proper evaluation and handling of these materials prior to continuing to work in that area.

As presently defined by the State of California, most refined petroleum products (gasoline, diesel fuel, motor oil, grease, coolant, etc.) have chemical constituents that are considered to be hazardous waste. As such, the indiscriminate dumping or spillage of these fluids onto the ground may constitute a misdemeanor, punishable by fines and/or imprisonment, and shall not be allowed.

2.2 Processing: Existing ground that has been declared satisfactory for support of fill by the Geotechnical Consultant shall be scarified to a minimum depth of 6 inches. Existing ground that is not satisfactory shall be overexcavated as specified in the following section. Scarification shall continue until soils are broken down and free of large clay lumps or clods and the working surface is reasonably uniform, flat, and free of uneven features that would inhibit uniform compaction.

2.3 Overexcavation: In addition to removals and overexcavations recommended in the approved geotechnical report(s) and the grading plan, soft, loose, dry, saturated, spongy, organic-rich, highly fractured or otherwise unsuitable ground shall be overexcavated to competent ground as evaluated by the Geotechnical Consultant during grading.

2.4 Benching: Where fills are to be placed on ground with slopes steeper than 5:1 (horizontal to vertical units), the ground shall be stepped or benched. Please see the Standard Details for a graphic illustration. The lowest bench or key shall be a minimum of 15 feet wide and at least 2 feet deep, into competent material as evaluated by the Geotechnical Consultant. Other benches shall be excavated a minimum height of 4 feet into competent material or as otherwise recommended by the Geotechnical Consultant. Fill placed on ground sloping flatter than 5:1 shall also be benched or otherwise overexcavated to provide a flat subgrade for the fill.

2.5 Evaluation/Acceptance of Fill Areas: All areas to receive fill, including removal and processed areas, key bottoms, and benches, shall be observed, mapped, elevations recorded, and/or tested prior to being accepted by the Geotechnical Consultant as suitable to receive fill. The Contractor shall obtain a written acceptance from the Geotechnical Consultant prior to fill placement. A licensed surveyor shall provide the survey control for determining elevations of processed areas, keys, and benches.

3.0 Fill Material

3.1 General: Material to be used as fill shall be essentially free of organic matter and other deleterious substances evaluated and accepted by the Geotechnical Consultant prior to placement. Soils of poor quality, such as those with unacceptable gradation, high expansion potential, or low strength shall be placed in areas acceptable to the Geotechnical Consultant or mixed with other soils to achieve satisfactory fill material.

3.2 Oversize: Oversize material defined as rock, or other irreducible material with a maximum dimension greater than 12 inches, shall not be buried or placed in fill unless location, materials, and placement methods are specifically accepted by the Geotechnical Consultant. Placement operations shall be such that nesting of oversized material does not occur and such that oversize material is completely surrounded by compacted or densified fill. Oversize material shall not be placed within 10 vertical feet of finish grade or within 2 feet of future utilities or underground construction.

3.3 Import: If importing of fill material is required for grading, proposed import material shall meet the requirements of Section 3.1. The potential import source shall be given to the Geotechnical Consultant at least 48 hours (2 working days) before importing begins so that its suitability can be determined and appropriate tests performed.

4.0 Fill Placement and Compaction

4.1 Fill Layers: Approved fill material shall be placed in areas prepared to receive fill (per Section 3.0) in near-horizontal layers not exceeding 8 inches in loose thickness. The Geotechnical Consultant may accept thicker layers if testing indicates the grading procedures can adequately compact the thicker layers. Each layer shall be spread evenly and mixed thoroughly to attain relative uniformity of material and moisture throughout.

4.2 Fill Moisture Conditioning: Fill soils shall be watered, dried back, blended, and/or mixed, as necessary to attain a relatively uniform moisture content at or slightly over optimum. Maximum density and optimum soil moisture content tests shall be performed in accordance with the American Society of Testing and Materials (ASTM Test Method D1557-91).

4.3 Compaction of Fill: After each layer has been moisture-conditioned, mixed, and evenly spread, it shall be uniformly compacted to not less than 90 percent of maximum dry density (ASTM Test Method D1557-91). Compaction equipment shall be adequately sized and be either specifically designed for soil compaction or of proven reliability to efficiently achieve the specified level of compaction with uniformity.

4.4 Compaction of Fill Slopes: In addition to normal compaction procedures specified above, compaction of slopes shall be accomplished by backrolling of slopes with sheepsfoot rollers at increments of 3 to 4 feet in fill elevation, or by other methods producing satisfactory results acceptable to the Geotechnical Consultant. Upon completion of grading, relative compaction of the fill, out to the slope face, shall be at least 90 percent of maximum density per ASTM Test Method D1557-91.

4.5 Compaction Testing: Field tests for moisture content and relative compaction of the fill soils shall be performed by the Geotechnical Consultant. Location and frequency of tests shall be at the Consultant's discretion based on field conditions encountered. Compaction test locations will not necessarily be selected on a random basis. Test locations shall be selected to verify adequacy of compaction levels in areas that are judged to be prone to inadequate compaction (such as close to slope faces and at the fill/bedrock benches).

4.6 Frequency of Compaction Testing: Tests shall be taken at intervals not exceeding 2 feet in vertical rise and/or 1,000 cubic yards of compacted fill soils embankment. In addition, as a guideline, at least one test shall be taken on slope faces for each 5,000 square feet of slope face and/or each 10 feet of vertical height of slope. The Contractor shall assure that fill construction is such that the testing schedule can be accomplished by the Geotechnical Consultant. The Contractor shall stop or slow down the earthwork construction if these minimum standards are not met.

4.7 Compaction Test Locations: The Geotechnical Consultant shall document the approximate elevation and horizontal coordinates of each test location. The Contractor shall coordinate with the project surveyor to assure that sufficient grade stakes are established so that the Geotechnical Consultant can determine the test locations with sufficient accuracy. At a minimum, two grade stakes within a horizontal distance of 100 feet and vertically less than 5 feet apart from potential test locations shall be provided.

5.0 Subdrain Installation

Subdrain systems shall be installed in accordance with the approved geotechnical report(s), the grading plan, and the Standard Details. The Geotechnical Consultant may recommend additional subdrains and/or changes in subdrain extent, location, grade, or material depending on conditions encountered during grading. All subdrains shall be surveyed by a land surveyor/civil engineer for line and grade after installation and prior to burial. Sufficient time should be allowed by the Contractor for these surveys.

6.0 Excavation

Excavations, as well as over-excavation for remedial purposes, shall be evaluated by the Geotechnical Consultant during grading. Remedial removal depths shown on geotechnical plans are estimates only. The actual extent of removal shall be determined by the Geotechnical Consultant based on the field evaluation of exposed conditions during grading. Where fill-over-cut slopes are to be graded, the cut portion of the slope shall be made, evaluated, and accepted by the Geotechnical Consultant prior to placement of materials for construction of the fill portion of the slope, unless otherwise recommended by the Geotechnical Consultant.

7.0 Trench Backfills

7.1 Contractor shall follow all OHSA and Cal/OSHA requirements for safety of trench excavations.

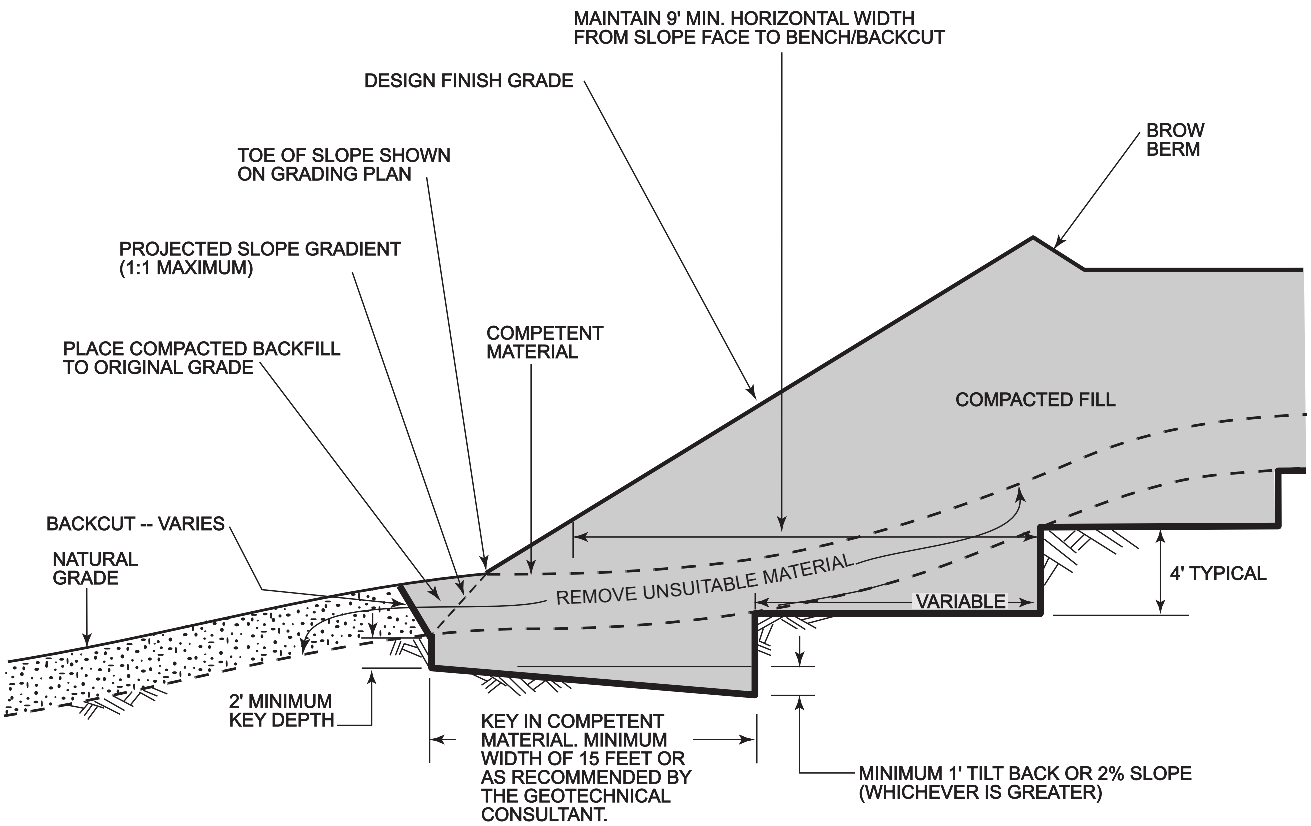
7.2 Bedding and backfill of utility trenches shall be done in accordance with the applicable provisions of Standard Specifications of Public Works Construction. Bedding material shall have a Sand Equivalent greater than 30 (SE>30). The bedding shall be placed to 1 foot over the top of the conduit and densified by jetting. Backfill shall be placed and densified to a minimum 90 percent of maximum from 1 foot above the top of the conduit to the surface, except in traveled ways (see Section 7.6 below).

7.3 Jetting of the bedding around the conduits shall be observed by the Geotechnical Consultant.

7.4 Geotechnical Consultant shall test the trench backfill for relative compaction. At least one test should be made for every 300 feet of trench and 2 feet of fill.

7.5 Lift thickness of trench backfill shall not exceed those allowed in the Standard Specifications of Public Works Construction unless the Contractor can demonstrate to the Geotechnical Consultant that the fill lift can be compacted to the minimum relative compaction by his alternative equipment and method.

7.6 Trench backfill in the upper foot measured from finish grade/subgrade within existing or future traveled way, shoulder, and other paved areas (or areas to receive pavement) should be placed to a minimum 95 percent relative compaction unless specified differently by the governing agency.



NOTE: BENCHING SHALL BE REQUIRED WHEN NATURAL SLOPES ARE EQUAL TO OR STEEPER THAN 5:1 OR WHEN RECOMMENDED BY THE SOIL ENGINEER. WHERE THE NATURAL SLOPE APPROACHES OR EXCEEDS THE DESIGN SLOPE RATIO, SPECIAL RECOMMENDATIONS WILL BE PROVIDED BY THE GEOTECHNICAL ENGINEER.

FIGURE 1

TYPICAL FILL KEY ABOVE NATURAL SLOPE
MINIMUM STANDARD GRADING DETAILS

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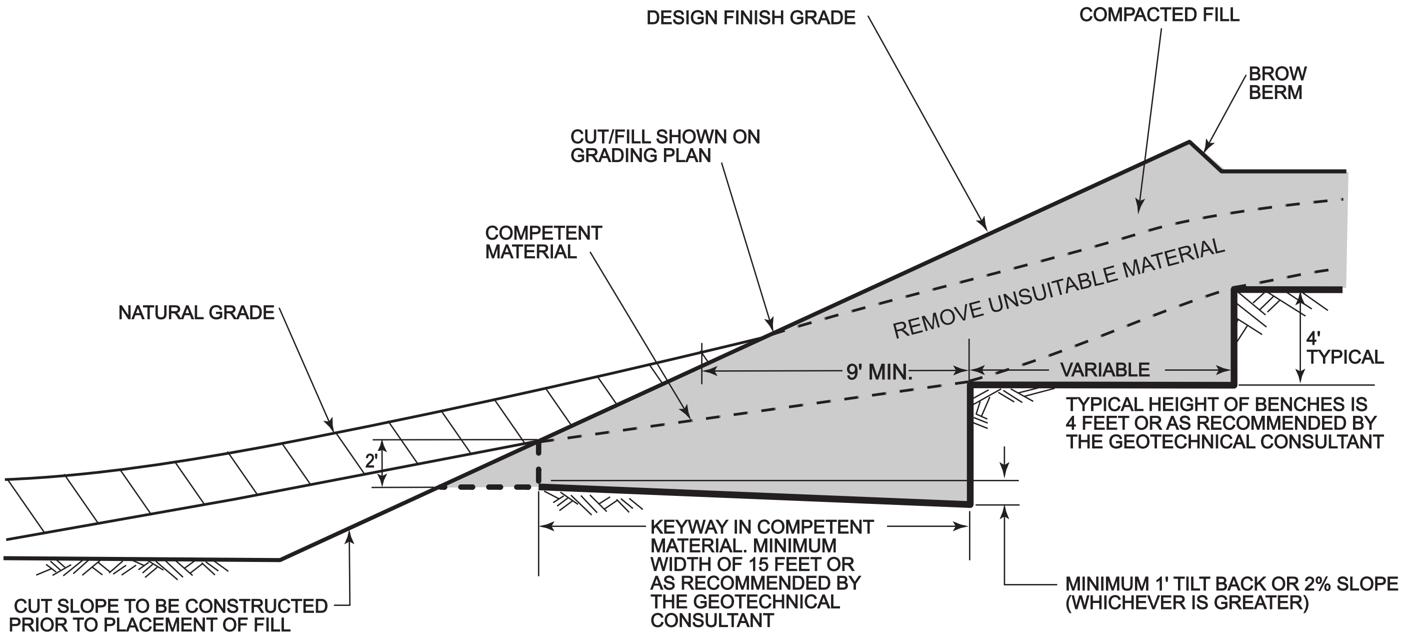

NOTE: THE FILL PORTION OF THE SLOPE SHALL BE COMPACTED AS STATED IN THE PROJECT SPECIFICATIONS.

FIGURE 2

TYPICAL FILL ABOVE CUT SLOPE
MINIMUM STANDARD GRADING DETAILS

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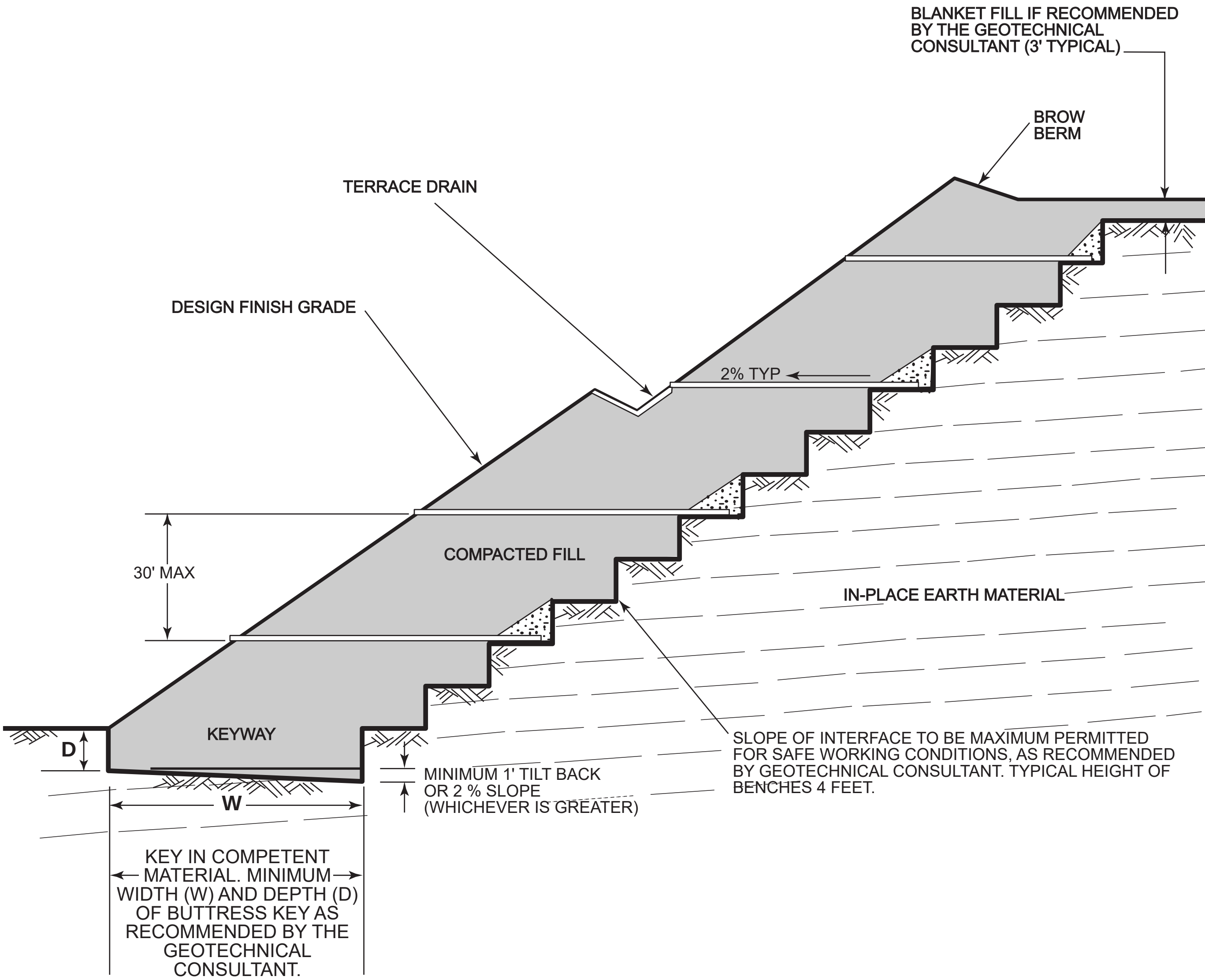
8/03 TYP FILL ABOVE CUT SLOPE.ai



NOTE: SUBDRAIN DETAILS, SEE FIGURE 5.

FIGURE 3

TYPICAL BUTTRESS FILL
MINIMUM STANDARD GRADING DETAILS

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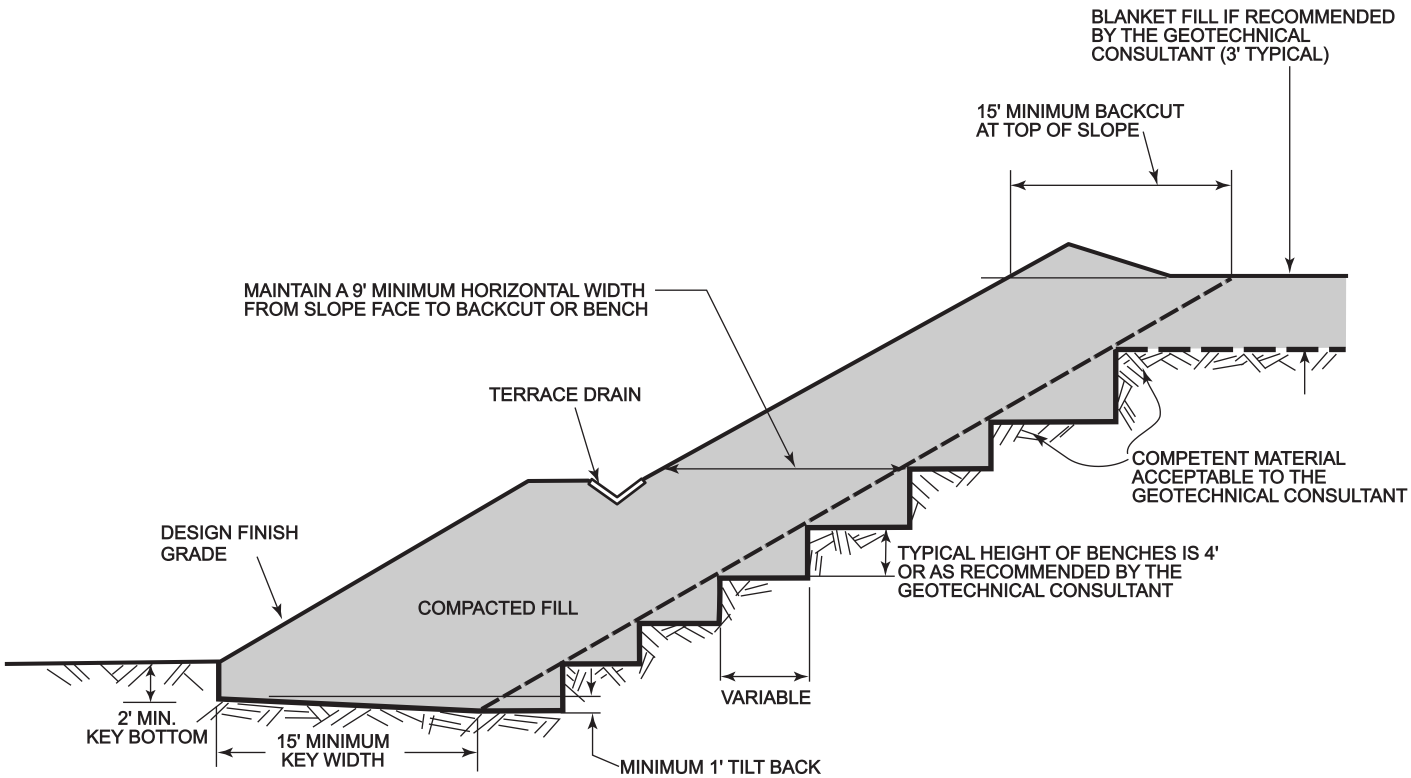
1/04 TYP BUTTRESS FILL.ai



NOTE:
SEE FIGURE 5 FOR TYPICAL SUBDRAIN DETAILS FOR STABILIZATION FILLS

FIGURE 4

TYPICAL STABILIZATION FILL
MINIMUM STANDARD GRADING DETAILS

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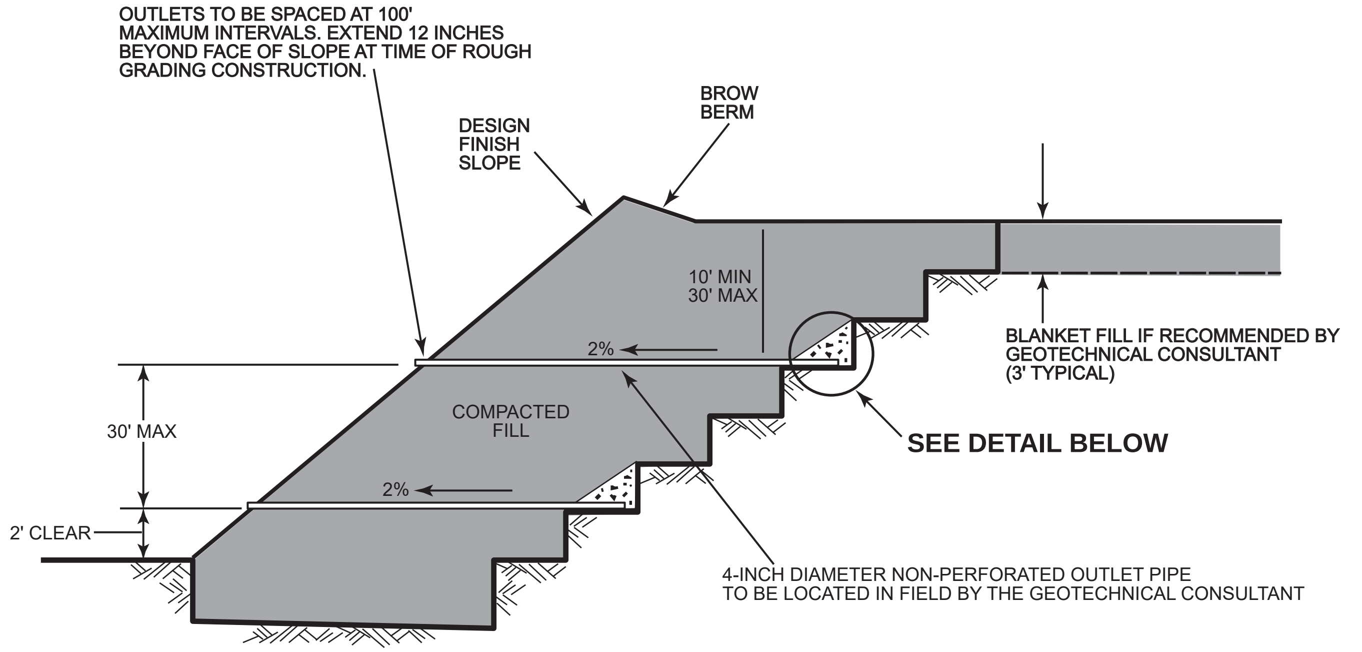


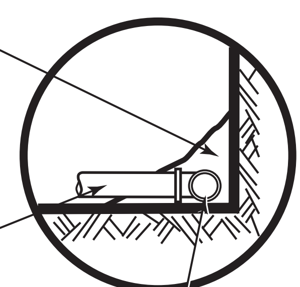
FILTER MATERIAL - MINIMUM OF THREE CUBIC FEET PER FOOT OF PIPE. SEE FILTER MATERIAL SPECIFICATION.

ALTERNATE: IN LIEU OF FILTER MATERIAL, THREE CUBIC FEET OF GRAVEL PER FOOT OF SUBDRAIN (WITHOUT PIPE) MAY BE ENCASED IN FILTER FABRIC. SEE GRAVEL SPECIFICATION, AND FIGURE 6 FOR FILTER FABRIC SPECIFICATION

"GRAVEL" TO CONSIST OF 1/2" TO 1" CRUSHED ROCK PER STANDARD SPECIFICATIONS FOR PUBLIC WORKS CONSTRUCTION.

FILTER FABRIC SHALL BE LAPPED A MINIMUM OF 12 INCHES ON ALL JOINTS.

DETAIL



OUTLET PIPE TO BE CONNECTED TO SUBDRAIN PIPE WITH TEE OR ELBOW

MINIMUM 4-INCH DIAMETER SCHEDULE 40 ASTM D1527 OR D1785 OR SDR 35 ASTM D2751 OR D 3034. FOR FILL DEPTH OF 90 FEET OR GREATER, USE ONLY SCHEDULE 40 OR EQUIVALENT. THERE SHALL BE A MINIMUM OF 8 UNIFORMLY SPACED PERFORATIONS PER FOOT OF PIPE INSTALLED WITH PERFORATIONS ON BOTTOM OF PIPE. PROVIDE CAP AT UPSTREAM END OF PIPE. SLOPE AT 2 PERCENT TO OUTLET PIPE.

"FILTER MATERIAL" TO MEET FOLLOWING SPECIFICATION OR APPROVED EQUIVALENT.

SIEVE SIZE	PERCENTAGE PASSING
1"	100
3/4"	90-100
3/8"	40-100
NO. 4	25-40
NO. 8	18-33
NO. 30	5-15
NO. 50	0-7
NO. 200	0-3

NOTE:
TRENCH FOR OUTLET PIPES TO BE BACKFILLED WITH ON-SITE SOIL.

FIGURE 5

TYPICAL STABILIZATION AND BUTTRESS FILL SUBDRAINS
MINIMUM STANDARD GRADING DETAILS

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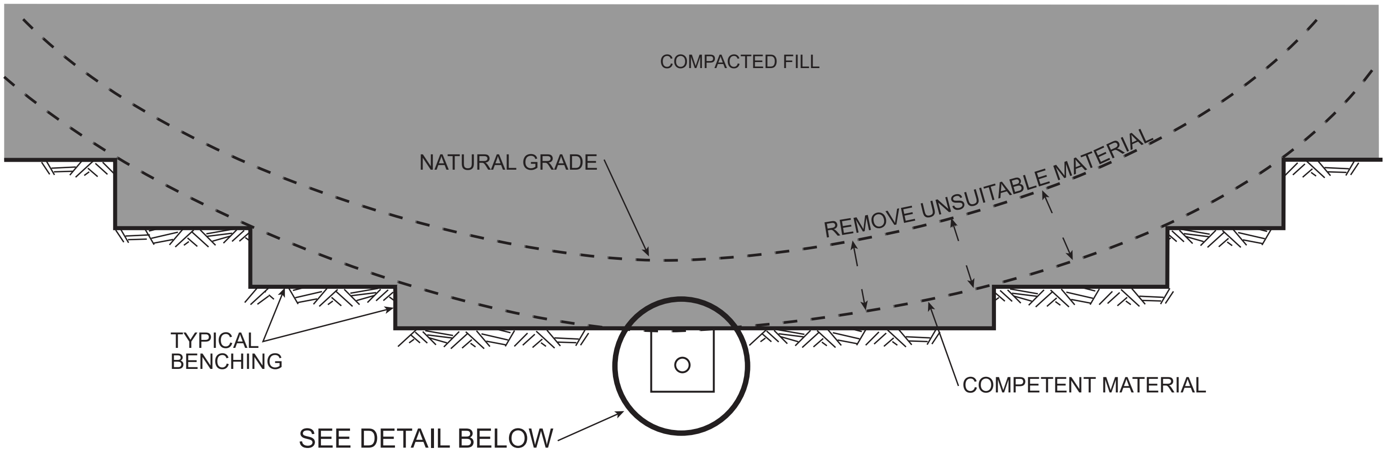
8/96 STAB. BUTTRESS FILL SUBDRAINS.ai



FILTER FABRICS SHALL BE PERMEABLE NON-WOVEN POLYESTER, NYLON, OR POLYPROPYLENE MATERIAL CONFORMING TO THE FOLLOWING:

1) GRAB TENSILE STRENGTH. POUNDS, MIN. ASTM D 4632...90
2) ELONGATION, AT PEAK LOAD, PERCENT, MIN. ASTM D 4632..50
3) PUNCTURE STRENGTH, LBS., MIN. ASTM D 3787..45
4) COEFFICIENT OF WATER PERMITTIVITY, 1/SEC. ASTM D 4491...>0.7
5) BURST STRENGTH, P.S.I., MIN. ASTM D 3786...180

NOTES: DOWNSTREAM 20' OF PIPE AT OUTLET SHALL BE NON-PERFORATED AND BACKFILLED WITH FINE-GRAINED MATERIAL

PIPE SHALL BE A MINIMUM OF 4-INCH DIAMETER. FOR RUNS OF 500 FEET OR MORE, USE 6-INCH DIAMETER PIPE, OR AS RECOMMENDED BY THE GEOTECHNICAL CONSULTANT

DETAIL

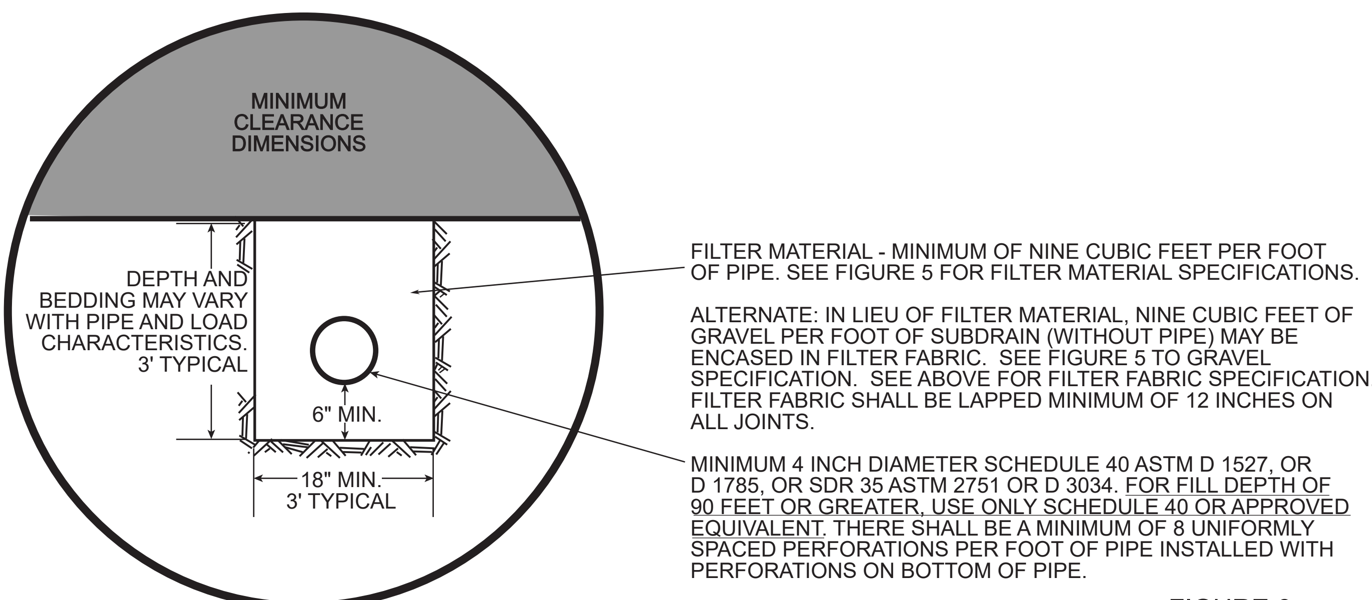


FIGURE 6

TYPICAL CANYON SUBDRAIN
MINIMUM STANDARD GRADING DETAILS

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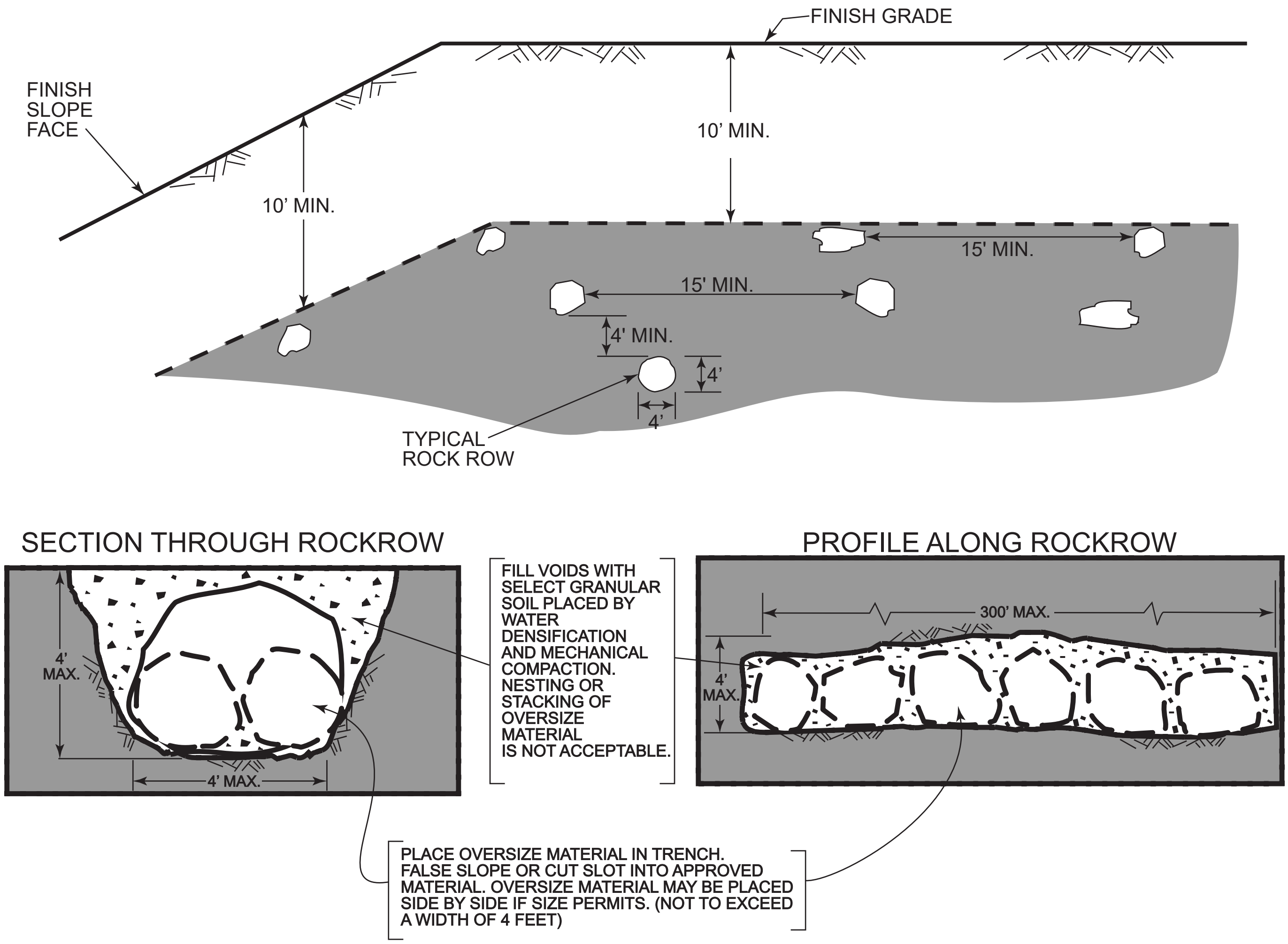
Rev. 8/96 CANYON SUBDRAIN.ai



NOTES:

A) OVERSIZED ROCK IS DEFINED AS LARGER THAN 12" IN SIZE (IN GREATEST DIMENSION).

B) SPACE BETWEEN ROCKROWS SHOULD BE ONE EQUIPMENT WIDTH OR A MINIMUM OF 15 FEET.

C) THE WIDTH AND HEIGHT OF THE ROCKROW SHALL BE LIMITED TO FOUR FEET AND THE LENGTH LIMITED TO 300 FEET UNLESS APPROVED OTHERWISE BY THE GEOTECHNICAL CONSULTANT. OVERSIZE SHOULD BE PLACED WITH FLATEST SIDE ON THE BOTTOM.

D) OVERSIZE MATERIAL EXCEEDING FOUR FEET MAY BE PLACED ON AN INDIVIDUAL BASIS IF APPROVED BY THE GEOTECHNICAL CONSULTANT.

E) FILLING OF VOIDS WILL REQUIRE SELECT GRANULAR SOIL (SE > 20, OR LESS THAN 20 PERCENT FINES) AS APPROVED BY THE GEOTECHNICAL CONSULTANT. VOIDS IN THE ROCKROW TO BE FILLED BY WATER DENSIFYING GRANULAR SOIL INTO PLACE ALONG WITH MECHANICAL COMPACTION EFFORT.

F) IF APPROVED BY THE GEOTECHNICAL CONSULTANT, ROCKROWS MAY BE PLACED DIRECTLY ON COMPETENT MATERIALS OR BEDROCK, PROVIDED ADEQUATE SPACE IS AVAILABLE FOR COMPACTION.

G) THE FIRST LIFT OF MATERIAL ABOVE THE ROCKROW SHALL CONSIST OF GRANULAR MATERIAL AND SHALL BE PROOF-ROLLED WITH A D-8 OR LARGER DOZER OR EQUIVALENT.

H) ROCKROWS NEAR SLOPES SHOULD BE ORIENTED PARALLEL TO SLOPE FACE.

I) NESTING OR STACKING OF ROCKS IS NOT ACCEPTABLE.

FIGURE 7

TYPICAL OVERSIZE ROCK PLACEMENT METHOD MINIMUM STANDARD GRADING DETAIL FOR STRUCTURAL FILL

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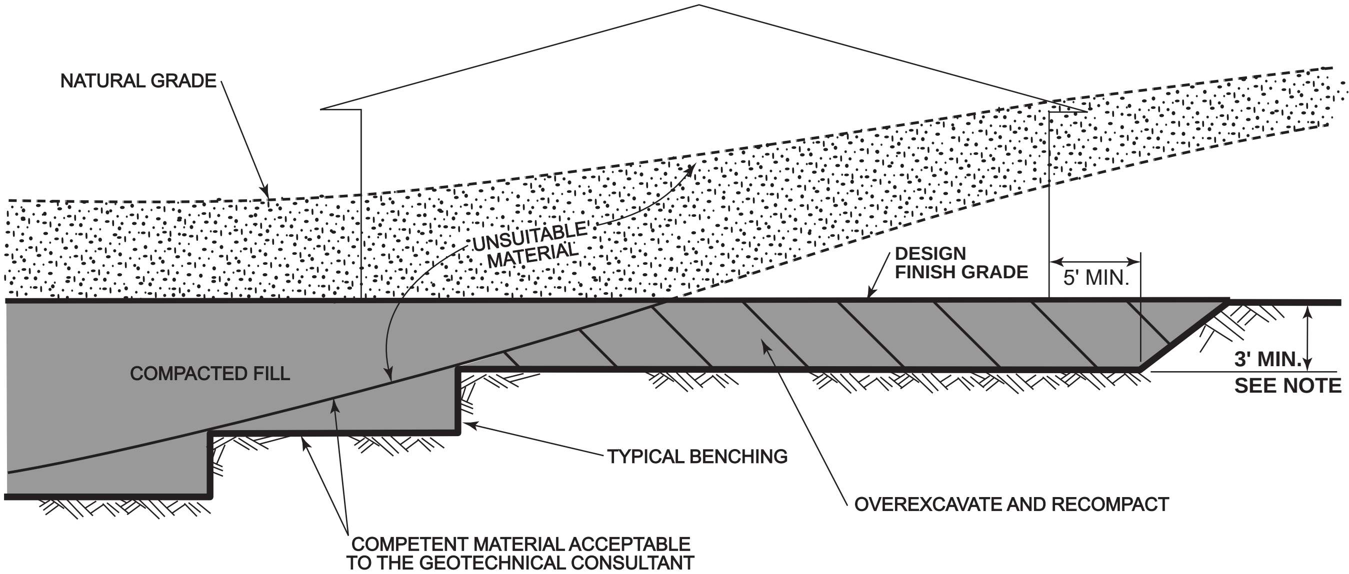
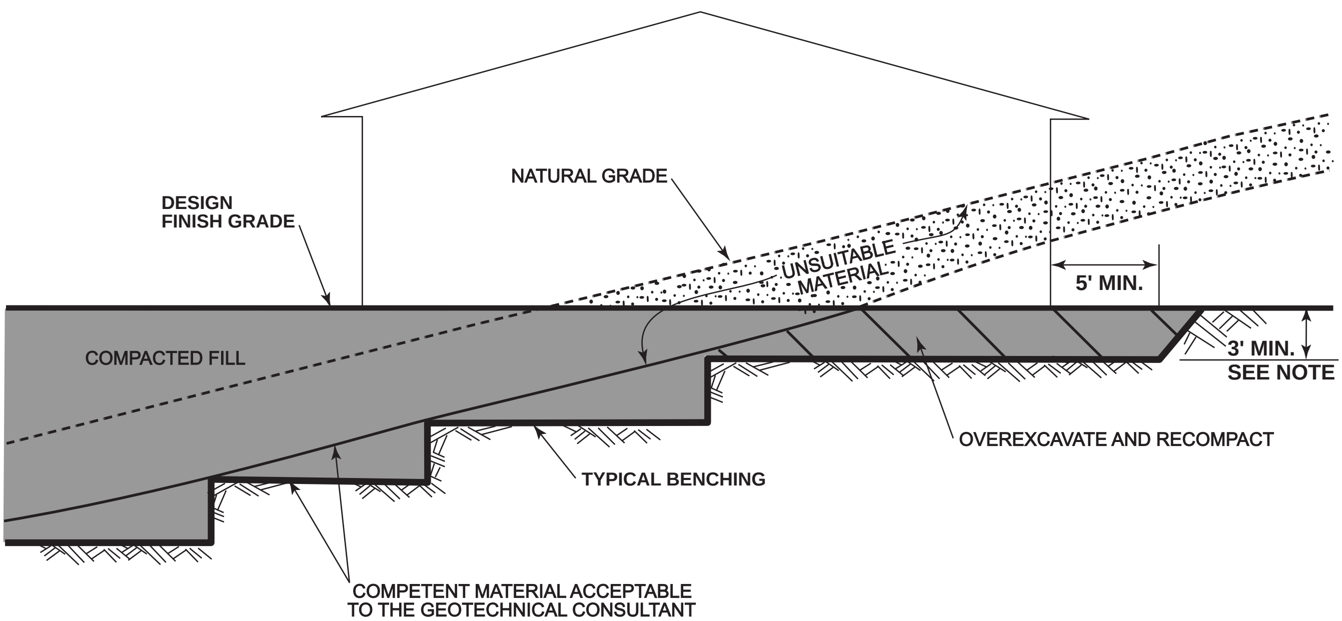
CUT LOT



CUT FILL LOT (TRANSITION)



NOTE: DEEPER THAN THE 3-FOOT OVEREXCAVATION MAY BE RECOMMENDED BY THE GEOTECHNICAL CONSULTANT IN STEEP TRANSITIONS.

FIGURE 8

TYPICAL OVEREXCAVATION OF DAYLIGHT LINE MINIMUM STANDARD GRADING DETAILS

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