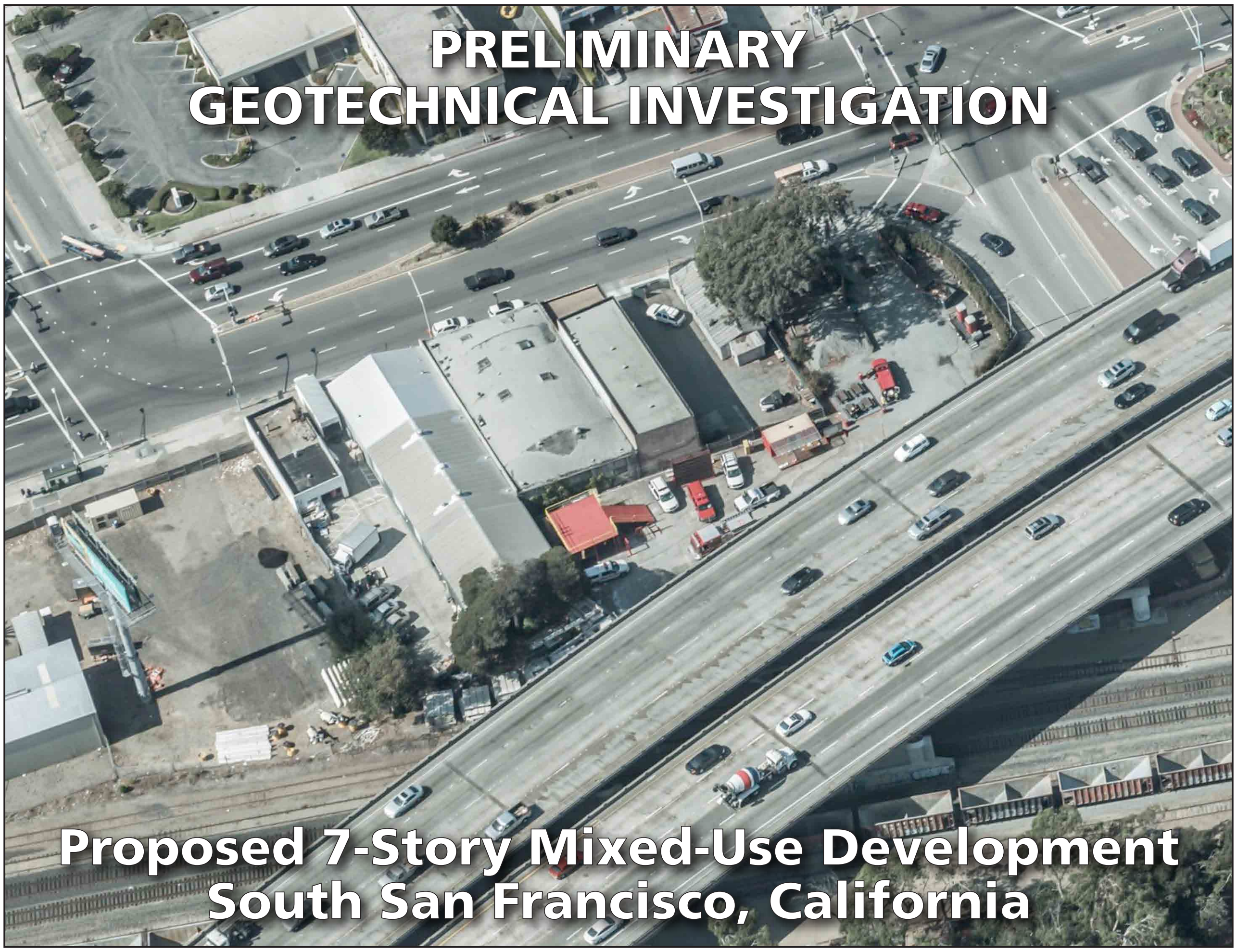

PREPARED FOR:

HANOVER R.S. LIMITED PARTNERSHIP
5847 SAN FELIPE, SUITE 3600
HOUSTON, TEXAS 77057

PREPARED BY:

GEOCON CONSULTANTS, INC.
6671 BRISA STREET
LIVERMORE, CALIFORNIA 94550

GEOCON

GEOCON PROJECT NO. E9065-04-01 **JUNE 2018**

Project No. E9065-04-01
June 18, 2018

Hanover R.S. Limited Partnership
5847 San Felipe, Suite 3600
Houston, Texas 77057

Attention: Ms. Kathy Binford

Subject: PROPOSED 7-STORY MIXED-USE DEVELOPMENT
200-214 AIRPORT BOULEVARD
SOUTH SAN FRANCISCO, CALIFORNIA
PRELIMINARY GEOTECHNICAL INVESTIGATION

Dear Ms. Binford:

In accordance with your authorization of our proposal dated April 16, 2018, we have performed a preliminary geotechnical investigation for the subject mixed-use development in South San Francisco, California. Our investigation was performed to identify major geotechnical constraints at the site as they pertain to the feasibility of your proposed project and develop preliminary design recommendations. The accompanying report presents the results of our study, which indicates the proposed development is feasible from a geotechnical standpoint. This preliminary investigation is a due-diligence level study intended to evaluate the geotechnical feasibility of the site for the proposed project. A design-level geotechnical study will be necessary to incorporate updated project details, corroborate the findings, opinions and recommendations provided herein, and provide geotechnical parameters for foundation design, site grading, etc.

If you have any questions regarding this report, or if we may be of further service, please contact the undersigned at your convenience.

Sincerely,

GEOCON CONSULTANTS, INC.

DRAFT

Shane Rodacker, PE, GE
Senior Engineer

DRAFT

Jacob Bishop-Moser, EIT
Senior Staff Engineer

(1/e-mail) Addressee

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PRELIMINARY GEOTECHNICAL INVESTIGATION

1. PURPOSE AND SCOPE

This report presents the results of a preliminary geotechnical investigation for a proposed mixed-use development in South San Francisco, California (see Vicinity Map, Figure 1). The purpose of this investigation was to evaluate the subsurface soil and geologic conditions for the planned development and, based on the encountered conditions, identify major geotechnical constraints relative to the feasibility of the proposed project and provide preliminary recommendations for project planning.

The scope of this investigation included field exploration, engineering analysis and the preparation of this report. Our field exploration was performed on May 10, 2018 and included the advancement of 4 Cone Penetrometer Test (CPT) soundings to maximum depths of approximately 42 feet below existing grade at the site. The locations of the CPT soundings are depicted on the Site Plan, Figure 2. A more-detailed discussion of our field investigation and CPT profiles are presented in Appendix A.

The opinions expressed herein are based on analysis of the data obtained during the investigation and our experience with similar soil and geologic conditions. References reviewed to prepare this report are provided in the *List of References* section.

If project details vary significantly from those described herein, Geocon should be contacted to determine the necessity for review and possible revision of this report.

2. SITE CONDITIONS AND PROJECT DESCRIPTION

The approximately 0.55-acre site is comprised of six adjacent parcels fronting the eastern side of Airport Boulevard in South San Francisco, California. WGS 84 (Google Earth) coordinates for the site are N 37.6540°, W 122.4077°. One- to two-story retail and shop buildings and areas of pavement are present throughout the site. Site topography is generally flat with approximate ground surface elevation of 16 to 18 feet MSL according to web-based mapping. A city-owned parcel exists to the north and east of the site. An elevated viaduct for US-101 is just east of the city-owned parcel.

Based on the conceptual project plans by TCA Architecture dated April 2018, the proposed development will include a 7-story mixed use structure generally constructed at/near existing grade with no subterranean levels. We understand the lower two levels will be concrete podium with leasing, commercial and retail spaces and parking for residents. The overlying five levels will be wood-framed residential units with an interior courtyard that opens to the west (toward Airport Boulevard). Vehicular access to the development will be from a new alleyway at the southern side. Ancillary site improvements such as underground utilities, driveways, and exterior flatwork and landscaping are also anticipated. The conceptual plans indicate at-grade commercial parking and a plaza are planned offsite to the east and north, respectively. We have assumed these offsite improvements are not part of the subject project.

3. GEOLOGIC SETTING

South San Francisco is located within the Coast Ranges Geomorphic Province of California, which is characterized by a series of northwest trending mountains and valleys along the north and central coast of California. Topography is controlled by the predominant geological structural trends within the Coast Range that generally consist of northwest trending synclines, anticlines and faulted blocks. The dominant structure is a result of both active northwest trending strike-slip faulting, associated with the San Andreas Fault system, and east-west compression within the province.

The San Andreas Fault (SAF) is a major right-lateral strike-slip fault that extends from the Gulf of California in Mexico to Cape Mendocino in northern California. The SAF forms a portion of the boundary between two tectonic plates on the surface of the earth. To the west of the SAF is the Pacific Plate, which moves north relative to the North American Plate, located east of the fault. In the San Francisco Bay Area, movement across this plate boundary is concentrated on the SAF but also distributed, to a lesser extent, across several other faults including the Hayward, Calaveras and Rodgers Creek faults, among others. Together, these faults are referred to as the SAF system.

Basement rock west of the SAF is generally granitic, while to the east it consists of a chaotic mixture of highly deformed marine sedimentary, submarine volcanic and metamorphic rocks of the Franciscan Complex. Both are typically Jurassic to Cretaceous in age (205 to 65 million years old). Overlying the basement rocks are Cretaceous (about 140 to 65 million years old) marine, as well as Tertiary (about 65 to 1.6 million years old) marine and non-marine sedimentary rocks with some continental volcanic rock. These Cretaceous and Tertiary rocks have typically been extensively folded and faulted largely because of movement along the SAF system, which has been ongoing for about the last 25 million years, and regional compression during the last about 4 million years. The inland valleys, as well as the structural depression within which San Francisco Bay is located, are filled with unconsolidated to semi-consolidated deposits of Quaternary age (about the last 1.6 million years). Continental deposits (alluvium) consist of unconsolidated to semi-consolidated sand, silt, clay and gravel, while the bay deposits typically consist of soft organic-rich silt and clay (bay mud) or sand.

Available geologic mapping by the United States Geological Survey (USGS) and other sources indicates the site is underlain by artificial fills over tidal flat deposits. Pleistocene age Colma Formation is mapped to the north, west and southwest of the site. Comparatively older Franciscan Formation is present at/near existing grade approximately 400 feet to the south of the site based on our experience at the adjacent project site.

4. GEOLOGIC HAZARDS

4.1 Faulting and Seismicity

Geologists and seismologists recognize the San Francisco Bay Area as one of the most seismically-active regions in the United States. The significant earthquakes that occur in the Bay Area are associated with crustal movements along well-defined active fault zones that generally trend in a northwesterly direction.

The site and greater Bay Area are seismically dominated by the presence of the active San Andreas Fault System. In the theory of plate tectonics, the San Andreas Fault System is a transform fault that forms the boundary between the northward moving Pacific Plate (west of the fault) and the southward moving North American Plate (east of the fault). Locally, the movement is distributed across a complex system of strike-slip, right lateral parallel and subparallel faults, which include the San Andreas, Hayward and Calaveras faults, among others.

The table below presents approximate distances to active faults in the site vicinity based on web-based mapping by the California Geological Survey (CGS), as presented in an online fault database maintained by Caltrans. Site latitude is 37.6540° N, 122.4077° W.

TABLE 4
REGIONAL FAULT SUMMARY

Fault Name	Approximate Distance to Site (miles)	Maximum Earthquake Magnitude, M_w
San Andreas	3	8.0
San Gregorio	8 ½	7.4
Hayward	16	7.3
Monte Vista - Shannon	21 ½	6.4
Silver Creek	23 ¼	6.9
Calaveras	24	6.9
Contra Costa Shear Zone	24 ¼	6.5
Pleasanton	25 ½	6.6

Faults tabulated above and many others in the Bay Area are sources of potential ground motion. However, earthquakes that might occur on other faults within the northern California area are also potential generators of significant ground motion and could cause ground shaking at the site.

4.2 Surface Fault Rupture

The site is not within a currently established State of California Earthquake Fault Zone for surface fault rupture hazards. No active or potentially-active faults are known to pass directly beneath the site. Therefore, the potential for surface rupture due to faulting occurring beneath the site during the design life of the proposed development is considered low. By CGS definition, an active fault is one with surface displacement within the last 11,000 years. A potentially-active fault has demonstrated evidence of surface displacement with the past 1.6 million years. Faults that have not moved in the last 1.6 million years are typically considered inactive.

4.3 Ground Shaking

We used the USGS web-based *Unified Hazard Tool* to estimate the peak ground acceleration (PGA) and modal (most probable) magnitude associated with a 2,475-year return period. This return period corresponds to an event with 2% chance of exceedance in a 50-year period. The USGS estimated PGA is 1.6g and the modal magnitude is 7.9 for Borderline Seismic Site Class D/E (V_s30 = 180 m/sec).

While listing PGA is useful for comparison of potential effects of fault activity in a region, other considerations are important in seismic design, including frequency and duration of motion and soil conditions underlying the site.

4.4 Liquefaction

The site is not located within a State of California Seismic Hazard Zone Hazard Zone for liquefaction. However, interactive web-based mapping by USGS and CGS indicates the northern portion of the site possesses a “very high” susceptibility to liquefaction. Liquefaction is a phenomenon in which saturated cohesionless soils are subject to a temporary loss of shear strength due to pore pressure buildup under the cyclic shear stresses associated with intense earthquakes. Primary factors that trigger liquefaction are: moderate to strong ground shaking (seismic source), relatively clean, loose granular soils (primarily poorly graded sands and silty sands), and saturated soil conditions (shallow groundwater). Due to the increasing overburden pressure with depth, liquefaction of granular soils is generally limited to the upper 50 feet of a soil profile.

We assessed the potential for liquefaction using the computer software program *CLiq* (Version 2.0, Geologismiki) and the in-situ soil parameters measured in the CPT soundings. The software applied the methodology of Boulanger and Idriss (2014) to the CPT data to evaluate liquefaction potential and estimate resultant settlements. Our analysis also considered the potential for cyclic softening in clayey soil. Our evaluation incorporated an earthquake moment magnitude (M_w) of 7.9 and a groundwater depth of 5 feet. We used a ground motion/Peak Ground Acceleration (PGA) of 0.80g for our analysis based on seismic design criteria from the USGS *US Seismic Design Maps* application.

Our liquefaction analysis identified potentially liquefiable layers at each CPT location. In general, these layers are less than 3 feet in thickness and located more than 10 feet below existing grade at the site. Thicker layers of liquefiable soils are located at greater depths. Consequences of liquefaction can include ground surface settlement, ground loss (sand boils) and lateral slope displacements (lateral spreading). For liquefaction-induced sand boils or fissures to occur, pore water pressure induced within liquefied strata must exert enough force to break through overlying, non-liquefiable layers. Based on methodology recommended by Youd and Garris (1995), which modified and advanced original research by Ishihara (1985), a capping layer of non-liquefiable soil can prevent the occurrence of sand boils and fissures. In our opinion, the presence of the non-liquefiable layer that mantles the site (which is at least approximately 10 feet thick) and the depth to significant liquefiable layers, the potential for ground loss due to sand boils or fissures in a seismic event is considered low.

The likely consequence of potential liquefaction at the site is settlement. Our analysis indicates that, if liquefaction and cyclic softening were to occur, total ground surface settlements on the order of 1 inch or less may result. We anticipate that deep foundations or ground improvement for the project will mitigate the potential for liquefaction-induced settlements to impact the proposed development. Selected output from our liquefaction analysis is presented in Appendix C.

4.5 Landslides

There are no known landslides near the site, nor is the site in the path of any known or potential landslides. We do not consider the potential for a landslide to be a hazard to this project.

4.6 Tsunamis and Seiches

Based on mapping published by the California Emergency Management Agency and CGS, the site is outside of a tsunami (seismic sea wave) inundation area.

Seiches are large waves generated in enclosed bodies of water in response to ground shaking. No major water-retaining structures are located immediately up gradient from the project site. Flooding from a seismically-induced seiche is considered unlikely.

5. SOIL AND GROUNDWATER CONDITIONS

Based on our review of geologic mapping and other information in our files, the site is mantled by a layer of artificial fill materials. The source and composition of the fill materials are generally unknown and should be studied further in a design-level geotechnical investigation for the project. The thickness of the fill materials was not directly observed in our CPTs. As a point of reference, in our study for the adjacent site, we encountered 13 to 14 feet of artificial fill in soil borings located approximately 100 feet south of the site. Topographic and geologic mapping suggests the fill thickness may be deeper on the subject site.

USGS maps the site within an area of artificial fill over tidal flat deposits. The USGS mapping provides no description of the tidal deposits. However, in our experience, tidal deposits are typically poorly consolidated, weak, fine-grained soils with limited support for additional fills or foundation loads.

Two formational units are mapped in close proximity to the site. Pleistocene-age Colma Formation is mapped immediately to the north, west and southwest of the site and described by the USGS as mostly yellowish-orange to gray, sandy clay and silty sand in the site vicinity. We observed Jurassic- to Cretaceous-age Franciscan Formation at grade in a soil boring for the adjacent site, approximately 400 feet south of the site. USGS describes the formational sub-unit in this area as interbedded sandstone and shale that is medium dark gray where fresh.

Based on the conditions encountered in our CPT soundings, the artificial fills and tidal deposits at the site are underlain by either or possibly both of the formational units discussed herein. In addition, layers of residual soils may be present atop the Franciscan Formation. The soil types within the formational unit(s) and the potential presence of residual soils were not directly observed due to our CPT-based field exploration and should be further evaluated and sampled with conventional soil borings during a future design-level geotechnical investigation for the project. The table below presents the depth to formational materials at our CPT locations. The tabulated depths should be considered approximate and should be confirmed in a future study for project design.

TABLE 5
INTERPRETED DEPTH TO FORMATIONAL MATERIALS

Exploration Location	Interpreted Depth to Formation (feet)
CPT-1	27
CPT-2	23
CPT-3	35
CPT-4	35

Note: the depths indicated above do not reflect residual soils that may be present atop the formational materials.

Based on dissipation testing in our CPTs, groundwater was interpreted at approximately 4 to 6 feet below grade at the time of our field exploration. Actual groundwater levels will fluctuate seasonally and with variations in rainfall, temperature and other factors and may be higher or lower than observed during our study.

6. CONCLUSIONS AND RECOMMENDATIONS

6.1 General

6.1.1 No overriding geotechnical constraints were encountered during our investigation that would preclude the project as presently proposed. Primary geotechnical considerations are the presence of undocumented artificial fills and tidal flat deposits to depths of at least 20 feet below existing grade and potential settlements due to liquefaction and/or cyclic softening. Deep foundations or a ground improvement system will be needed to support the development.

6.1.2 This report identifies foundation types and ground improvement systems that are considered feasible for the proposed structure and the site soil and geologic conditions. General discussion on each is presented herein. Other foundation types or ground improvement systems may also be feasible and the selected foundation type or ground improvement may depend on non-geotechnical aspects such as the acceptability of vibration and noise from pile driving operations and/or the environmental characteristics of the soils that underlie the site. Given the history of land use in the area, as well as the unknown source and composition of the fill materials that underlie the site, the costs for disposal of spoils generated by drilled shafts or similar foundation types should be considered in project planning. The project team should review the information provided herein when selecting foundation type for the project. The design of specialty foundation types should be reviewed by Geocon.

6.1.3 The proposed project redevelops a site with past episodes of grading and construction. As such, unknown underground improvements and areas of undocumented fill materials (not discussed herein) may be present. If encountered, supplemental recommendations will be provided during site development.

6.1.4 As discussed in Section 4.3, the site is susceptible to liquefaction. Our analysis indicates that, if liquefaction and cyclic softening were to occur, total ground surface settlements on the order of 1 inch and differential settlements up to approximately ¾ inch may result. We anticipate that deep foundations or ground improvement for the project will mitigate the potential effects of liquefaction-induced settlement at the building locations. Potential liquefaction settlements should be considered in the design of exterior improvements and/or improvements not supported by deep foundations.

6.1.5 Any changes in the design, location or elevation of the proposed improvements, as outlined in this report, should be reviewed by this office. Geocon should be contacted to determine the necessity for review and possible revision of this report.

6.2 Seismic Design Criteria

6.2.1 The following table summarizes site-specific design criteria for the project. We derived the following seismic design parameters using the web-based USGS application *U.S. Seismic Design Maps*. The information presented herein is based on the 2016 CBC. The building structure and improvements should be designed using a Site Class D based on the information in Section 1613.3.2 of the 2016 CBC and Table 20.3-1 of ASCE 7-10. The values presented in Table 6.2.1 are for the risk-targeted maximum considered earthquake (MCE_R).

TABLE 6.2.1
2016 CBC SEISMIC DESIGN PARAMETERS

Parameter	Value	CBC Reference
Site Class	D	Section 1613.3.2/ Table 20.3-1
MCE_R Ground Motion Spectral Response Acceleration – Class B (0.2 sec), S_S	2.041 g	Figure 1613.3.1(1) / Figure 22-1
MCE_R Ground Motion Spectral Response Acceleration – Class B (1 sec), S_1	0.963 g	Figure 1613.3.1(2) / Figure 22-2
Site Coefficient, F_A	1.0	Table 1613.3.3(1) / Table 11.4-1
Site Coefficient, F_V	1.5	Table 1613.3.3(2) / Table 11.4-2
Site Class Modified MCE_R Spectral Response Acceleration (0.2 sec), S_{MS}	2.041 g	Eq. 16-37 / Eq. 11.4-1
Site Class Modified MCE_R Spectral Response Acceleration (1 sec), S_{M1}	1.445 g	Eq. 16-38 / Eq. 11.4-2
5% Damped Design Spectral Response Acceleration (0.2 sec), S_{DS}	1.361 g	Eq. 16-39 / Eq. 11.4-3
5% Damped Design Spectral Response Acceleration (1 sec), S_{D1}	0.963 g	Eq. 16-40 / Eq. 11.4-4

6.2.2 Table 6.2.2 presents additional seismic design parameters for projects located in Seismic Design Categories of D through F in accordance with ASCE 7-10 for the mapped maximum considered geometric mean (MCE_G).

TABLE 6.2.2
2016 CBC SITE ACCELERATION DESIGN PARAMETERS

Parameter	Value	ASCE 7-10 Reference
Mapped MCE_G Peak Ground Acceleration, PGA	0.798g	Figure 22-7
Site Coefficient, F_{PGA}	1.0	Table 11.8-1
Site Class Modified MCE_G Peak Ground Acceleration, PGA_M	0.798g	Section 11.8.3 (Eq. 11.8-1)

6.2.3 Conformance to the criteria in Tables 6.2.1 and 6.2.2 for seismic design does not constitute a guarantee or assurance that significant structural damage or ground failure will not occur if a large earthquake occurs. The primary goal of seismic design is to protect life, not to avoid all damage, since such design may be economically prohibitive.

6.3 Soil and Excavation Characteristics

6.3.1 Based on the soils conditions encountered in our CPTs, the onsite soils can be excavated with moderate effort using conventional excavation equipment or drilling rigs. We do not anticipate excavations in the native Colma Formation at the site will generate oversize material (greater than 6 inches in nominal dimension). Excavations or drilling in Franciscan Formation will require additional effort and possibly special equipment or techniques. The artificial fills at the site are undocumented and may contain constituents not reported herein. Below-grade improvements associated with prior site development may also be present.

6.3.2 It is the responsibility of the contractor to ensure that all excavations and trenches are properly shored and maintained in accordance with applicable Occupational Safety and Health Administration (OSHA) rules and regulations to maintain safety and maintain the stability of adjacent existing improvements.

6.4 Deep Foundations and Ground Improvement

6.4.1 Deep foundations or a ground improvement system will be required to support the proposed mixed-use development. Presented below are one deep foundation type and one ground improvement system that are considered feasible based the site soil and geologic conditions discussed herein. Deep foundations and ground improvement are not meant to be used together – proper implementation of one or the other can effectively mitigate the identified geotechnical constraints. Other foundation types and ground improvement may also be feasible.

Augercast Piles

Augercast piles (ACPs) are installed by advancing a large hollow-stem auger to design depth and filling the remnant shaft with structural grout. The grout is placed through the hollow-stem as the auger is withdrawn. After the auger is removed, the required steel reinforcement is then wet-set into the pile to complete the installation.

ACPs are typically 14 to 24 inches in diameter and can be installed to depths of 80 feet or more. ACPs are typically designed and installed by specialty geotechnical contractors because constructability, installation production, performance and capacity will vary depending on the contractor's equipment, experience, skill, materials and installation procedures. We strongly recommend performing a comprehensive pile installation and load testing program to evaluate constructability as well as capacity. The program should include environmental sampling and analysis of soil cuttings generated during test pile installation (if any) to evaluate offsite disposal options.

The specialty foundation contractor should prepare a complete design-build submittal with design details, calculations, estimated capacities, installation procedures, proposed load testing procedures, acceptance criteria and quality control procedures. Geocon should perform a geotechnical review of the design-build submittal and observed ACP construction.

Compacted Aggregate Piers

The development may be supported by spread footings if used in conjunction with Compacted Aggregate Piers (CAPs) or similar ground improvement systems. CAPs are designed and installed by specialty ground improvement contractors. The CAP system is based on soil improvement that consists of installing densified, aggregate columns within drilled shafts. Drilling depths depend on specific site soil conditions and shaft diameters are commonly 18 to 30 inches. The system increases density and lateral stress in the surrounding soil and claims improvement in bearing capacity and settlement potential; thus, allowing the use of conventional shallow foundations over the CAP elements. CAPs typically allow the use of increased allowable bearing pressures for foundation design and result in estimated post-construction total and differential settlements of less than 1 inch and ½ inch, respectively.

CAP elements are constructed by drilling shafts that are subsequently backfilled with Class 2 aggregate base in approximate 1-foot lifts. An excavator equipped with a special ramming attachment is used to compact each lift of aggregate. Drill spoils are commonly reused as fill material or exported for offsite disposal. Soil and groundwater conditions at the site may require the use of temporary casing during CAP construction.

If the CAP system is selected for structural support, the CAP specialty contractor would provide a complete design-build submittal with design recommendations, engineered plans and specifications. Geocon will need to perform a geotechnical review of the RAP design.

Geocon should monitor CAP construction. Our Quality Assurance (QA) services will supplement the contractor internal Quality Control (QC) program. Together the QA/QC program will monitor drill depths, shaft length, average lift thicknesses, installation procedures, aggregate quality, and densification of lifts. The allowable vertical capacities should be verified by full-scale modulus and uplift load tests performed on RAP elements. The contractor QC program should document each RAP element installed, which will be reviewed by Geocon.

7. FURTHER GEOTECHNICAL SERVICES

7.1 Design-Level Geotechnical Investigation

7.1.1 As discussed herein, we should conduct a design-level geotechnical investigation to address updated project details, confirm the opinions and preliminary recommendations provided herein, and develop specific recommendations for the selected foundation type and/or ground improvement.

7.2 Testing and Observation Services

7.2.1 The recommendations provided in this report are based on the assumption that we will continue as Geotechnical Engineer of Record throughout the construction phase. It is important to maintain continuity of geotechnical interpretation and confirm that field conditions encountered are similar to those anticipated during design. If we are not retained for these services, we cannot assume any responsibility for others interpretation of our recommendations, and therefore the future performance of the project.

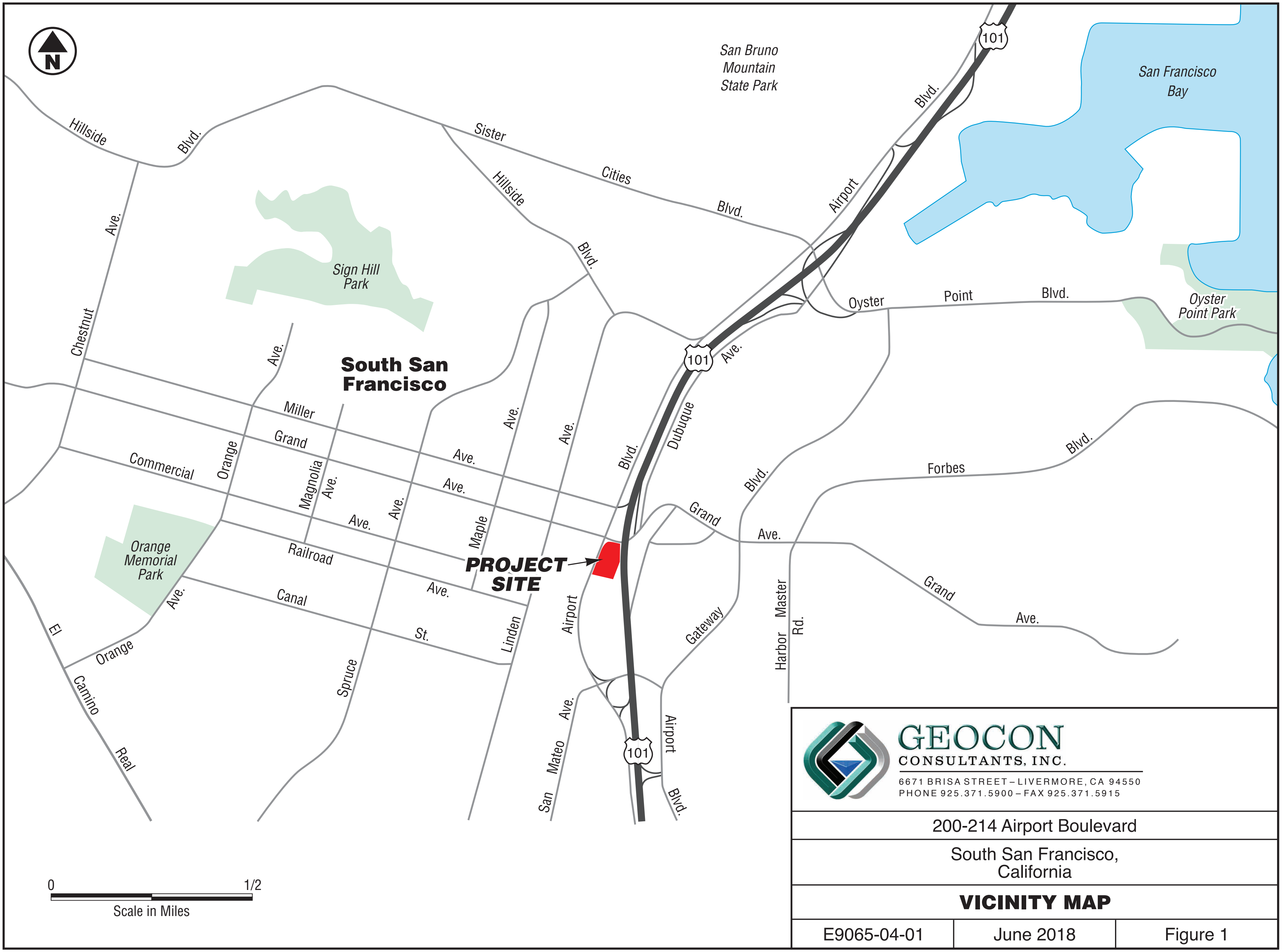
LIMITATIONS AND UNIFORMITY OF CONDITIONS

The recommendations of this report pertain only to the site investigated and are based upon the assumption that the soil conditions do not deviate from those disclosed in the investigation. If any variations or undesirable conditions are encountered during construction, or if the proposed construction will differ from that anticipated herein, Geocon Consultants, Inc. should be notified so that supplemental recommendations can be given. The evaluation or identification of the potential presence of hazardous or corrosive materials was not part of the geotechnical scope of services provided by Geocon Consultants, Inc.

This report is issued with the understanding that it is the responsibility of the owner, or of his representative, to ensure that the information and recommendations contained herein are brought to the attention of the architect and engineer for the project and incorporated into the plans, and the necessary steps are taken to see that the contractor and subcontractors carry out such recommendations in the field.

The findings of this report are valid as of the present date. However, changes in the conditions of a property can occur with the passage of time, whether they are due to natural processes or the works of man on this or adjacent properties. In addition, changes in applicable or appropriate standards may occur, whether they result from legislation or the broadening of knowledge. Accordingly, the findings of this report may be invalidated wholly or partially by changes outside our control. Therefore, this report is subject to review and should not be relied upon after a period of three years.

Our professional services were performed, our findings obtained, and our recommendations prepared in accordance with generally accepted geotechnical engineering principles and practices used in the site area at this time. No warranty is provided, express or implied.



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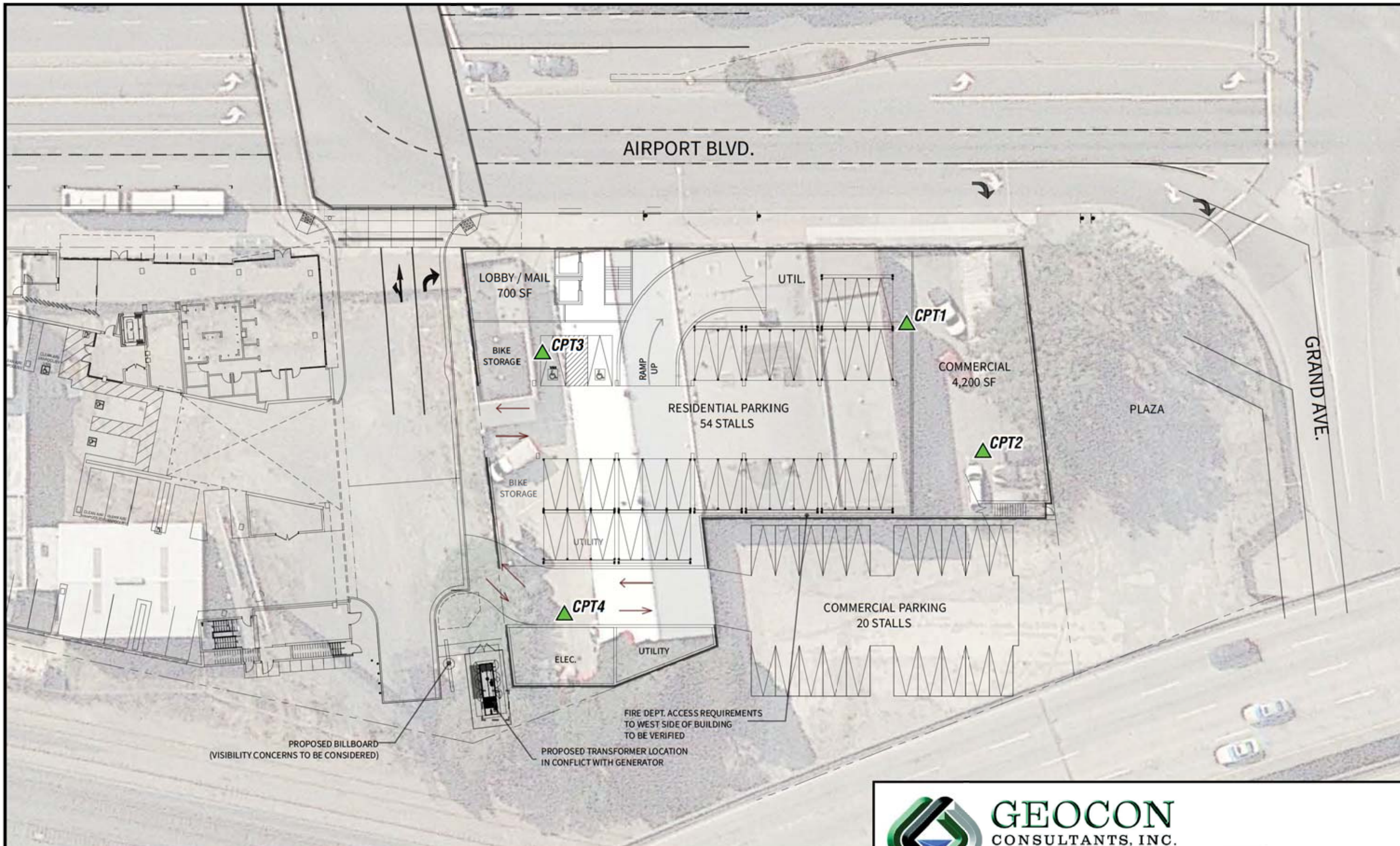
200-214 Airport Boulevard

South San Francisco, California

VICINITY MAP

E9065-04-01	June 2018	Figure 1



LEGEND:

CPT4 ▲ Approximate Boring Location

0 50

Scale in Feet

GEOCON CONSULTANTS, INC.

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200-214 Airport Boulevard

South San Francisco, California

SITE PLAN

E9065-04-01	June 2018	Figure 2

APPENDIX A

APPENDIX A
FIELD EXPLORATION

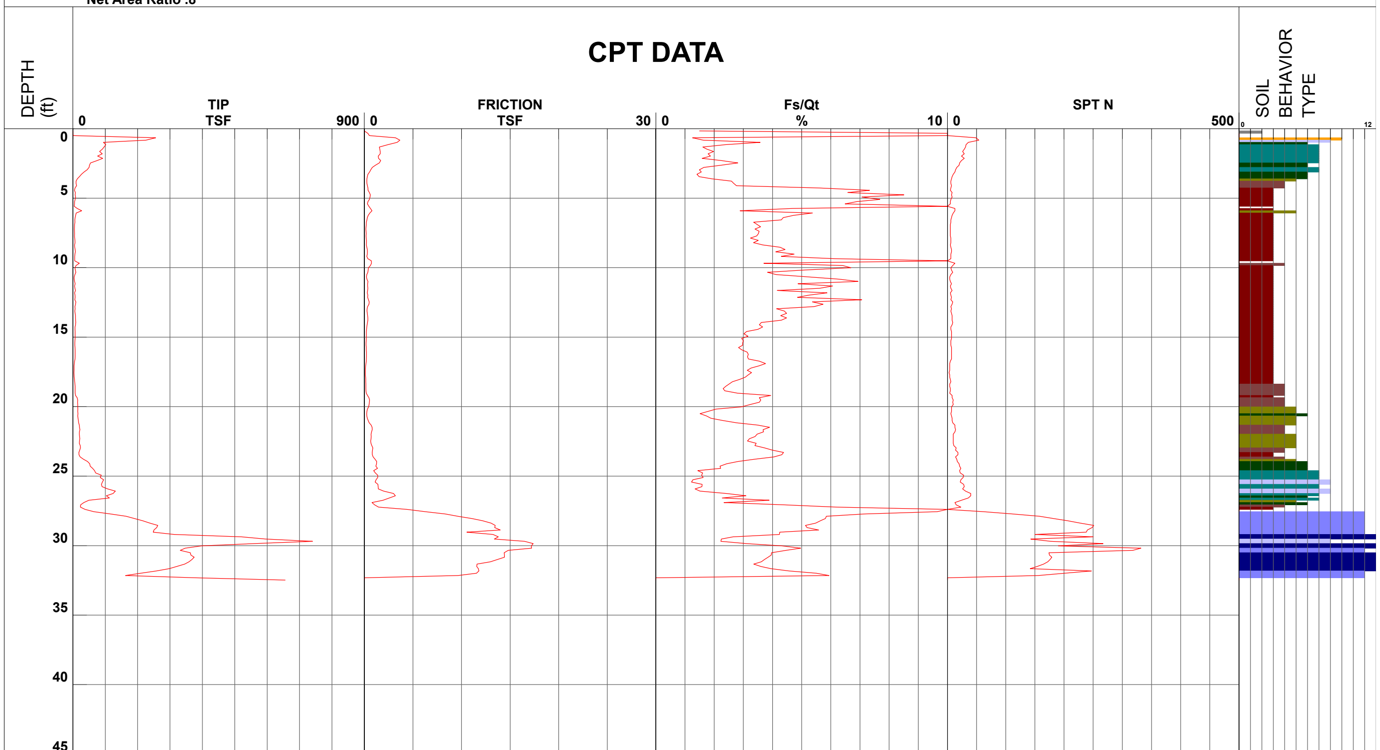
Fieldwork for our investigation included a site visit and subsurface exploration. Subsurface exploration included four Cone Penetrometer Test (CPT) soundings to maximum depths of approximately 42 feet. The locations of the CPTs are shown on the Site Plan, Figure 2. CPT data from our exploration are presented in figures following the text in this appendix. We located exploration locations in the field by pacing from existing reference points. Therefore, actual locations may deviate slightly from those shown on Figure 2.

The CPTs were performed on May 10, 2018 by Middle Earth Geo Testing, Inc. using a 20-ton truck mounted rig equipped with an integrated electronic cone system. The cone has a tip area of 10 cm^2, a friction sleeve area of 150 cm^2, and a ratio of friction sleeve area to tip end area equal to 0.80. The cone bearing (Q_c) and sleeve friction (F_s) were measured and recorded during tests at approximately 2-inch depth intervals. The CPT data consisting of cone bearing, sleeve friction, friction ratio and equivalent standard penetration blow counts (N) versus penetration depth below the existing ground surface for each location has been recorded and is presented at the end of this appendix.

Upon completion, the CPT boreholes were backfilled with grout per San Mateo County Environmental Health Services Division permit requirements.

Project	200-214 Airport Boulevard	Operator	RB-BH	Filename	SDF(069).cpt
Job Number	E9065-04-01	Cone Number	DDG1350	GPS	
Hole Number	CPT-01	Date and Time	5/10/2018 11:07:27 AM	Maximum Depth	32.48 ft
EST GW Depth During Test			4.00 ft		

Net Area Ratio .8



1 - sensitive fine grained
2 - organic material
3 - clay
4 - silty clay to clay
5 - clayey silt to silty clay
6 - sandy silt to clayey silt
7 - silty sand to sandy silt
8 - sand to silty sand
9 - sand
10 - gravelly sand to sand
11 - very stiff fine grained (*)
12 - sand to clayey sand (*)

Cone Size 10cm squared

S*Soil behavior type and SPT based on data from UBC-1983

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CONE PENETROMETER TEST CPT-1

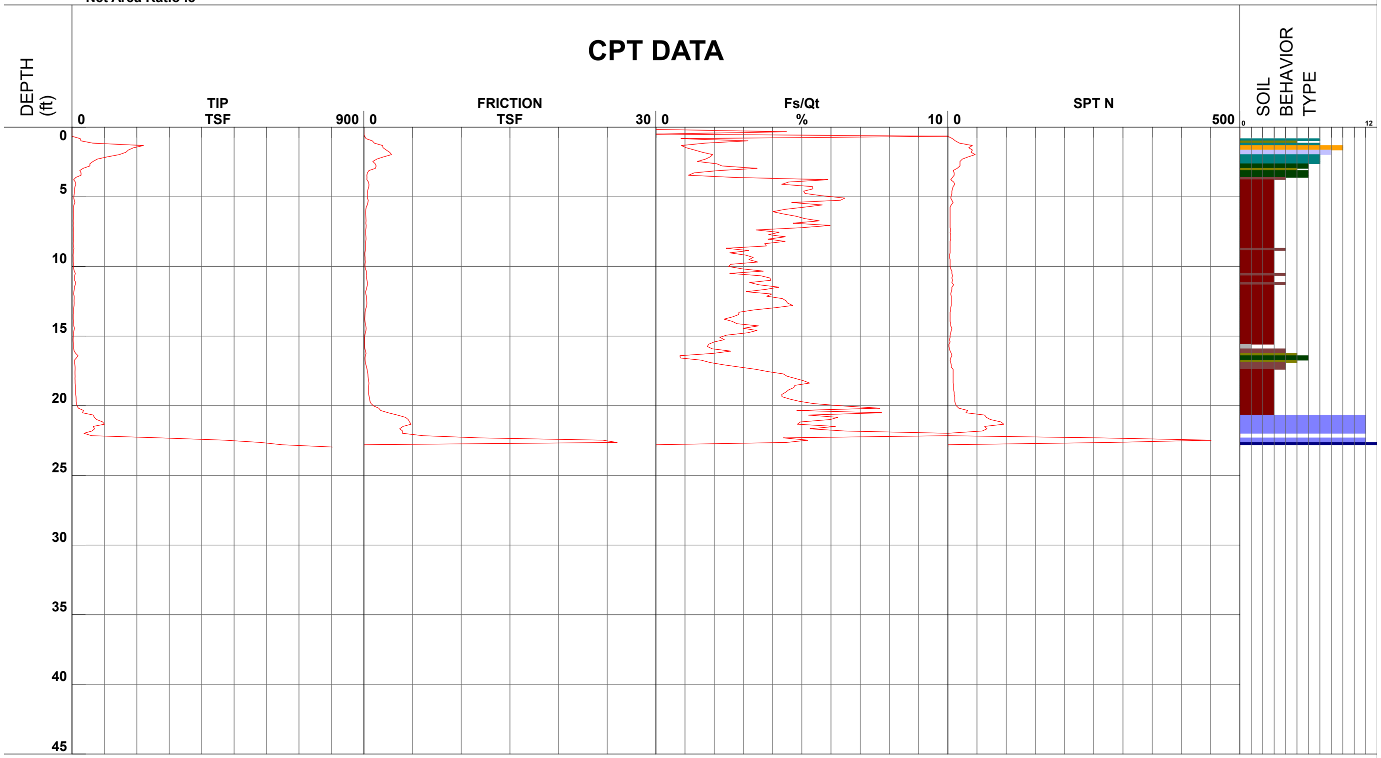
Project: 200-214 Airport Blvd. Mixed Use
Location: South San Francisco
Project No. E9065-04-01
Date: June 2018

FIGURE A1

Project	200-214 Airport Boulevard	Operator	RB-BH	Filename	SDF(068).cpt
Job Number	E9065-04-01	Cone Number	DDG1350	GPS	
Hole Number	CPT-02	Date and Time	5/10/2018 10:36:09 AM	Maximum Depth	22.97 ft
EST GW Depth During Test		4.00 ft			

Net Area Ratio .8

CPT DATA



1 - sensitive fine grained	4 - silty clay to clay	7 - silty sand to sandy silt	10 - gravelly sand to sand
2 - organic material	5 - clayey silt to silty clay	8 - sand to silty sand	11 - very stiff fine grained (*)
3 - clay	6 - sandy silt to clayey silt	9 - sand	12 - sand to clayey sand (*)

Cone Size 10cm squared

S*Soil behavior type and SPT based on data from UBC-1983

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CONE PENETROMETER TEST CPT-2

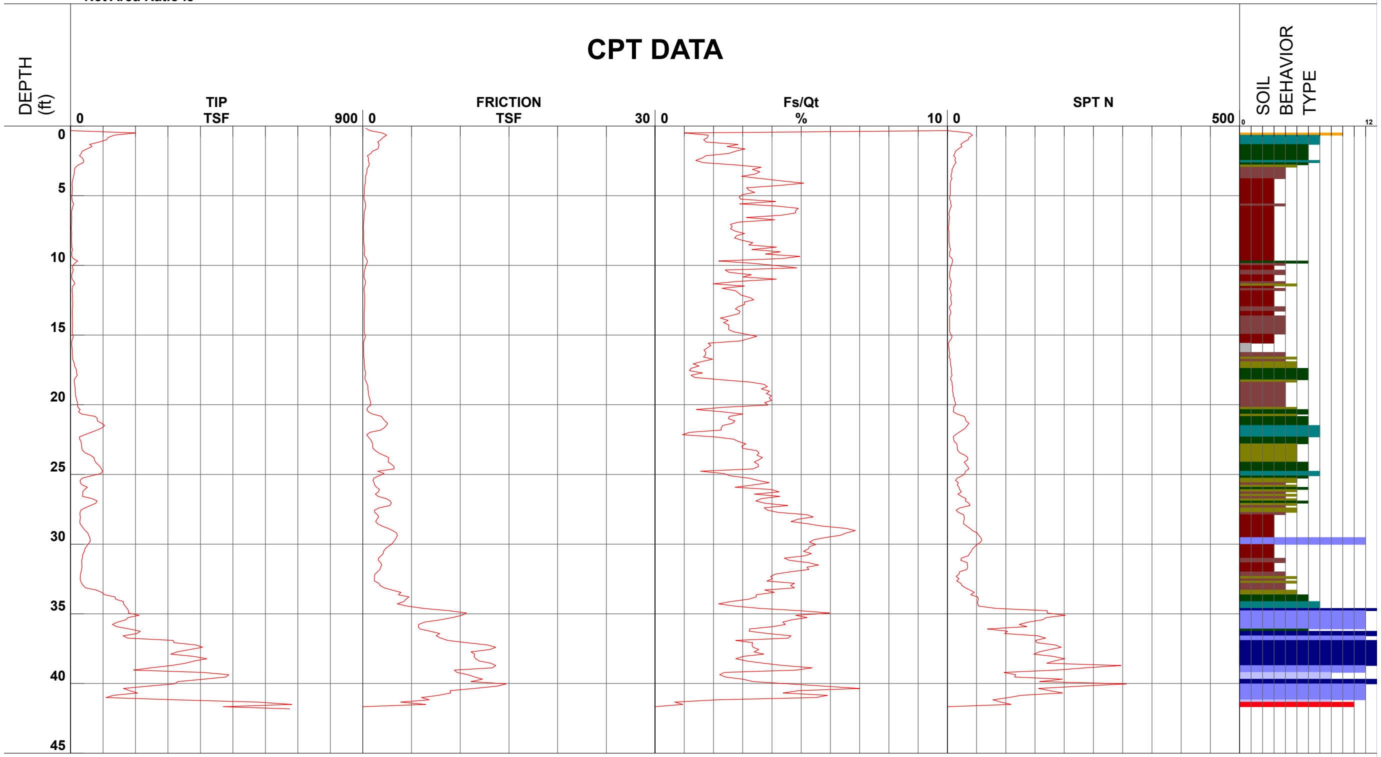
Project: 200-214 Airport Blvd. Mixed Use
Location: South San Francisco
Project No. E9065-04-01
Date: June 2018

FIGURE A2

Project	200-214 Airport Boulevard	Operator	RB-BH	Filename	SDF(066).cpt
Job Number	E9065-04-01	Cone Number	DDG1350	GPS	
Hole Number	CPT-03	Date and Time	5/10/2018 7:27:15 AM	Maximum Depth	41.83 ft
EST GW Depth During Test		5.00 ft			

Net Area Ratio .8

CPT DATA



1 - sensitive fine grained
2 - organic material
3 - clay
4 - silty clay to clay
5 - clayey silt to silty clay
6 - sandy silt to clayey silt
7 - silty sand to sandy silt
8 - sand to silty sand
9 - sand
10 - gravelly sand to sand
11 - very stiff fine grained (*)
12 - sand to clayey sand (*)

Cone Size 10cm squared

S*Soil behavior type and SPT based on data from UBC-1983

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CONE PENETROMETER TEST CPT-3

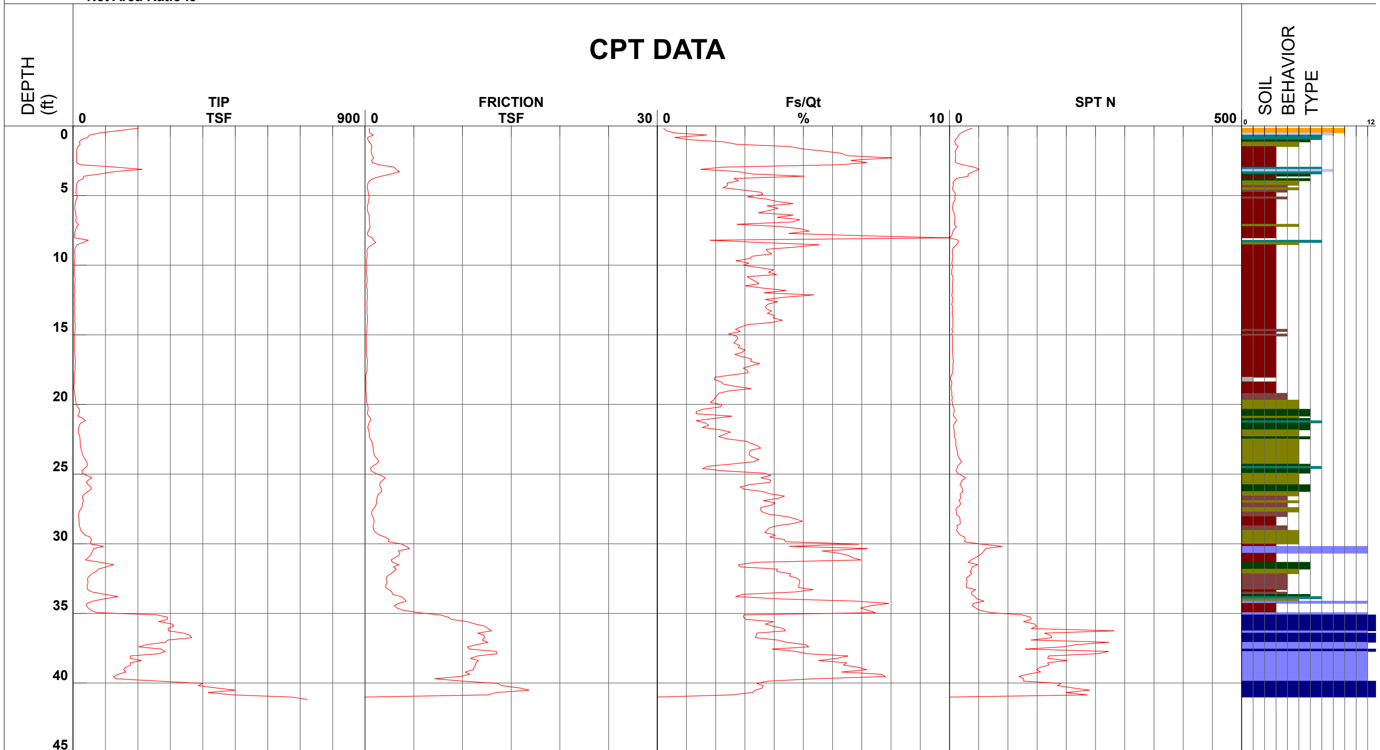
Project: 200-214 Airport Blvd. Mixed Use
Location: South San Francisco
Project No. E9065-04-01
Date: June 2018

FIGURE A3

Project	200-214 Airport Boulevard	Operator	RB-BH	Filename	SDF(067).cpt
Job Number	E9065-04-01	Cone Number	DDG1350	GPS	
Hole Number	CPT-04	Date and Time	5/10/2018 8:02:33 AM	Maximum Depth	41.17 ft
EST GW Depth During Test		6.00 ft			

Net Area Ratio .8

CPT DATA



■ 1 - sensitive fine grained
■ 2 - organic material
■ 3 - clay
■ 4 - silty clay to clay
■ 5 - clayey silt to silty clay
■ 6 - sandy silt to clayey silt
■ 7 - silty sand to sandy silt
■ 8 - sand to silty sand
■ 9 - sand
■ 10 - gravelly sand to sand
■ 11 - very stiff fine grained (*)
■ 12 - sand to clayey sand (*)

Cone Size 10cm squared

S*Soil behavior type and SPT based on data from UBC-1983

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CONE PENETROMETER TEST CPT-4

Project: 200-214 Airport Blvd. Mixed Use
Location: South San Francisco
Project No. E9065-04-01
Date: June 2018

FIGURE A4

APPENDIX B

APPENDIX B
LIQUEFACTION ANALYSIS

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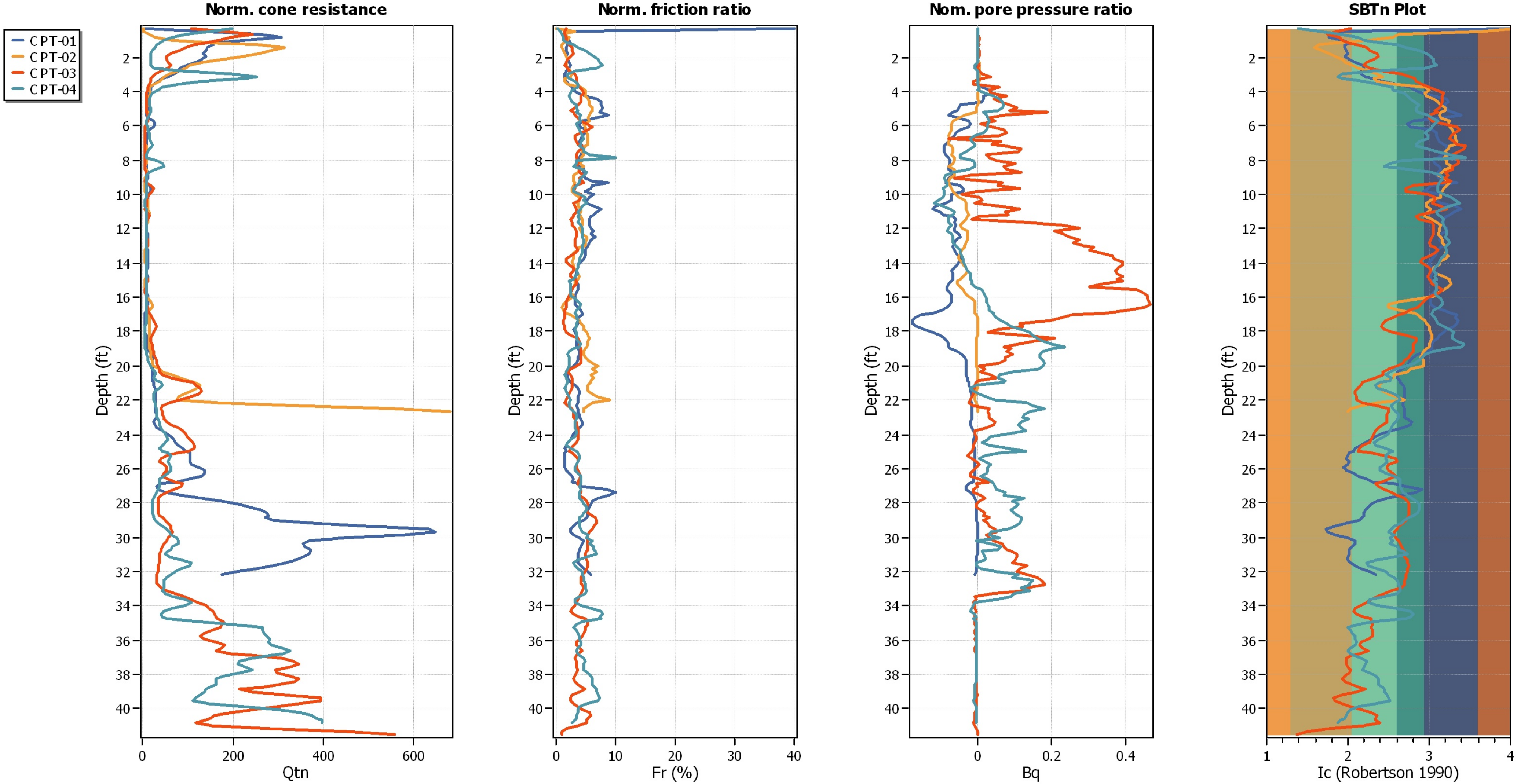
Project: 200-214 Airport Mixed Use

Overlay Normalized Plots



i

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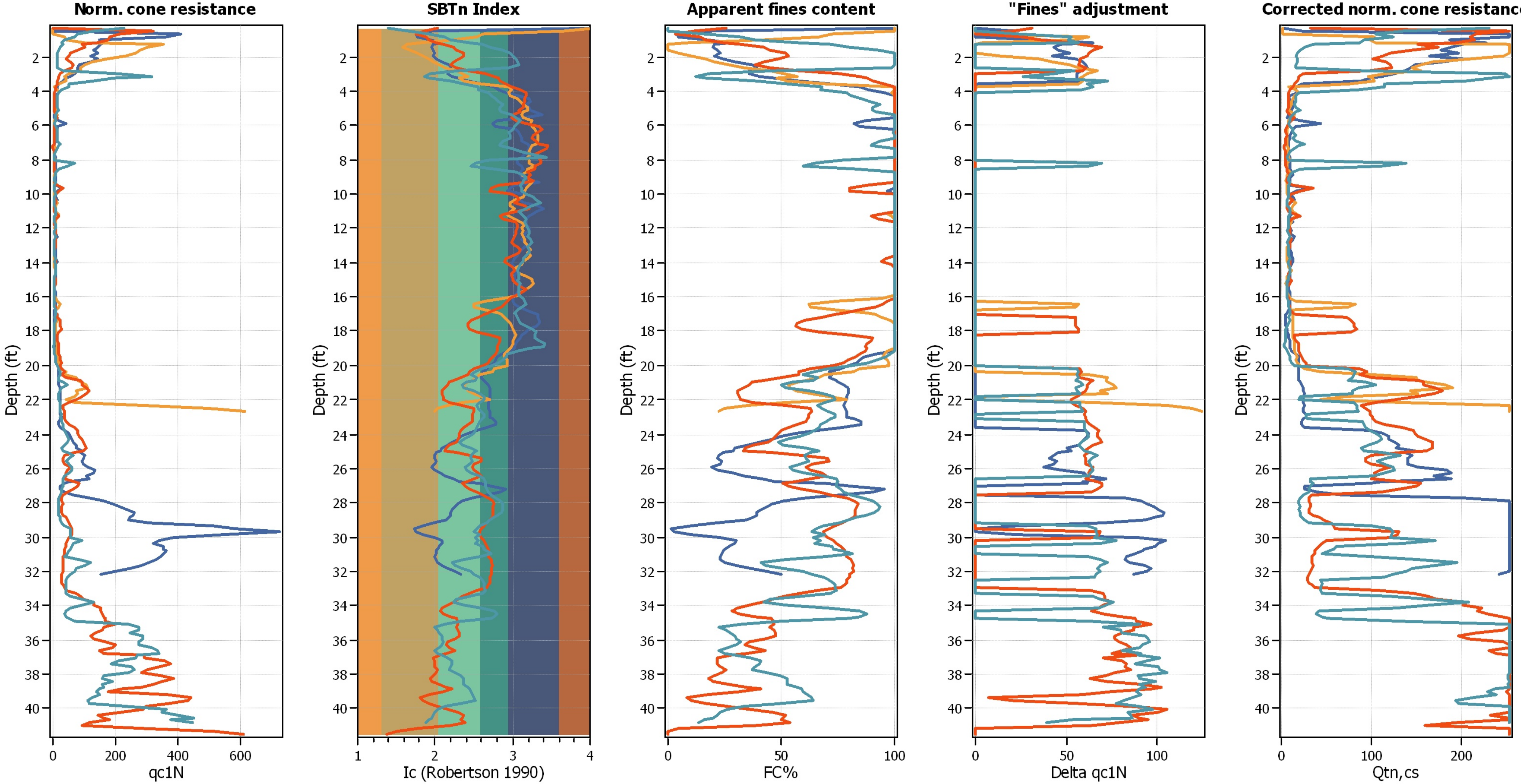
Project: 200-214 Airport Mixed Use

Overlay Intermediate Results



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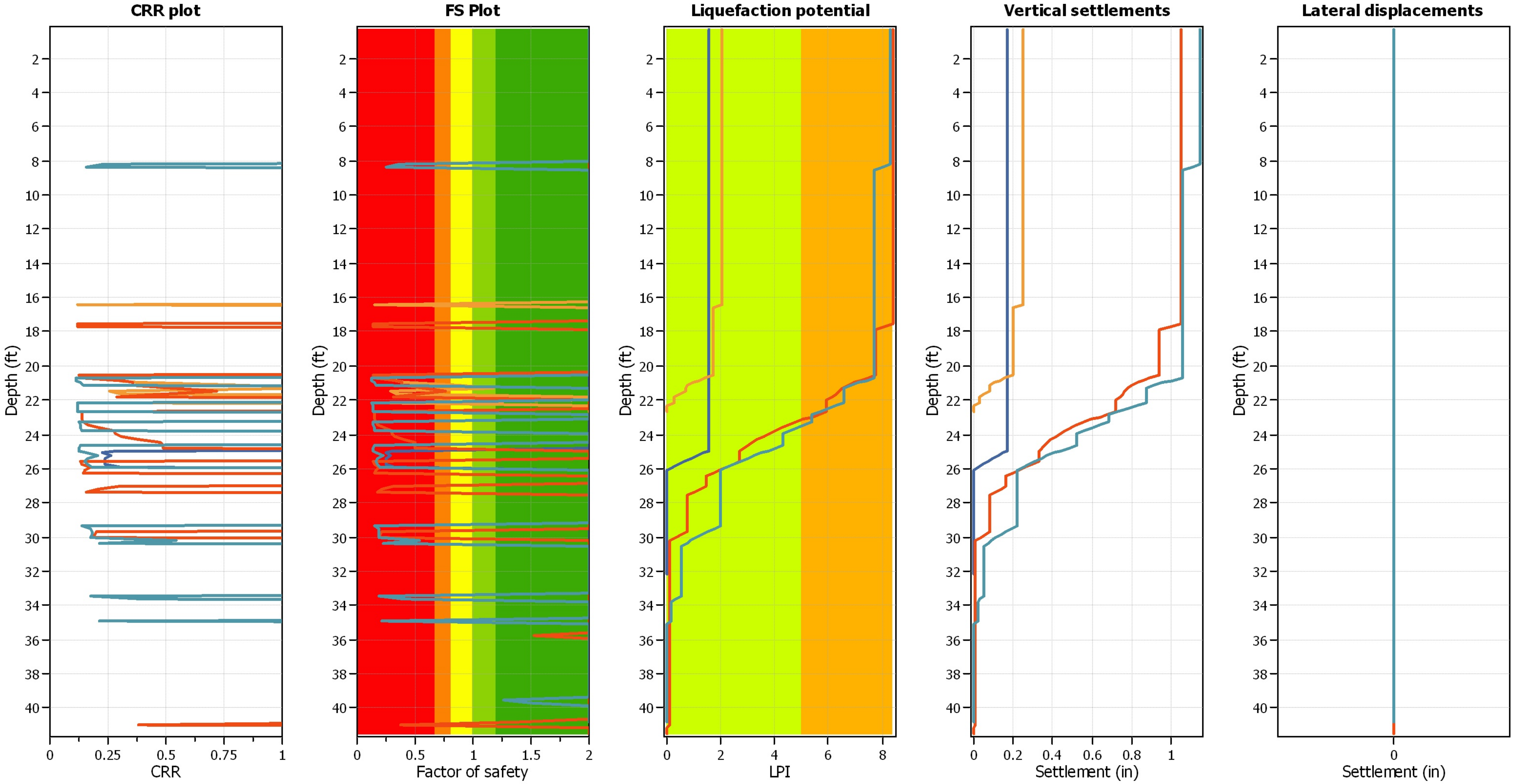
i

Project: 200-214 Airport Mixed Use

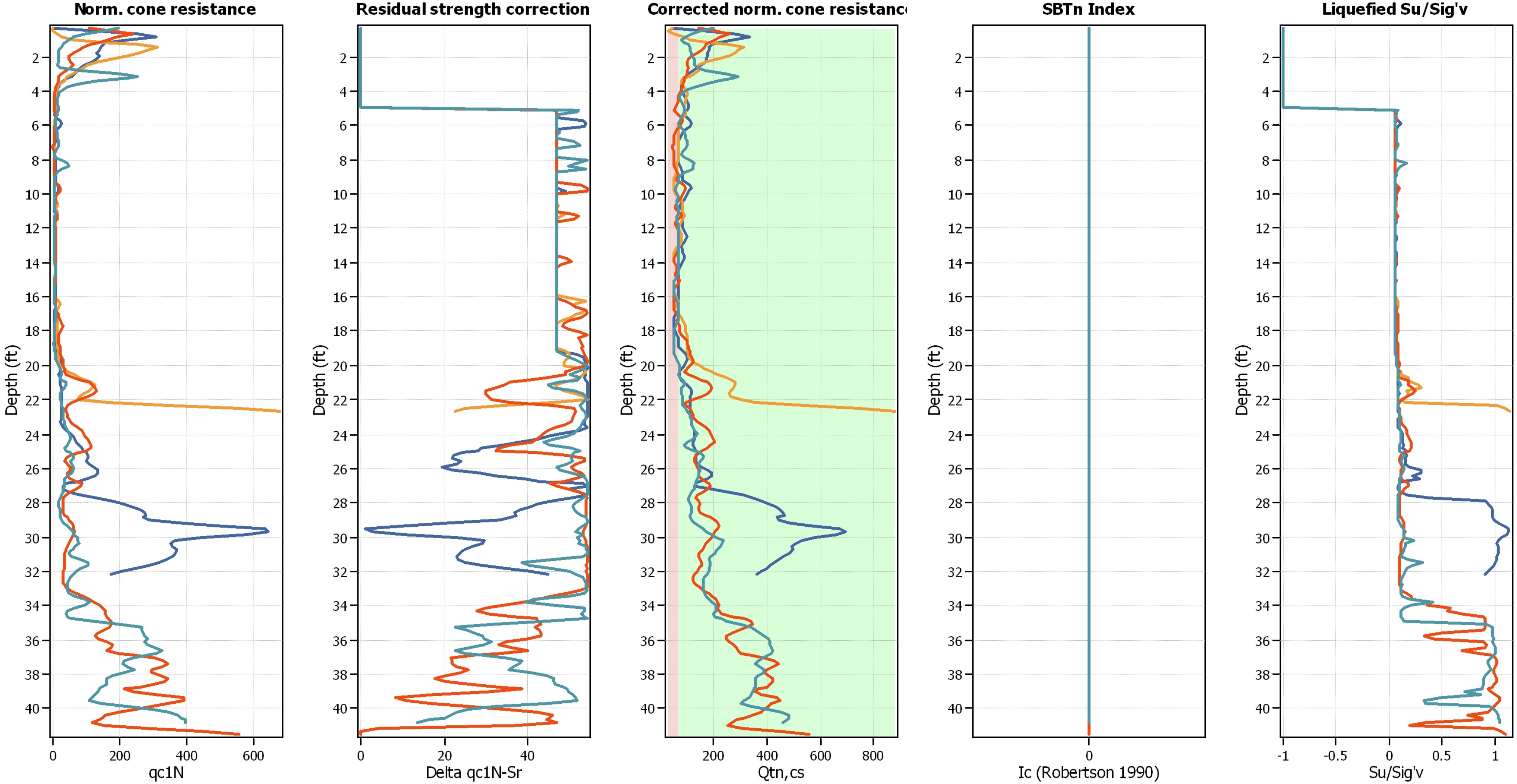
Overlay Cyclic Liquefaction Plots



i

Project: 200-214 Airport Mixed Use

Overlay Strength Loss Plots



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