APPENDIX E

GEOTECHNICAL REPORT

PREPARED FOR:

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PREPARED BY:

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GEOCON

GEOCON PROJECT NO. S2673-05-01 **DECEMBER 2023**

Project No. S2673-05-01
December 21, 2023

VIA ELECTRONIC MAIL

Terri McCracken
Associate Principal
Placeworks
tmccracken@placeworks.com

Subject: PRELIMINARY GEOTECHNICAL EVALAUTION
201 GOLDEN GATE AVENUE MIXED-USE BUILDING
201 GOLDEN GATE AVENUE
SAN FRANCISCO, CALIFORNIA

Ms. McKraken:

In accordance with your authorization of our proposal (Geocon Proposal No. LS-23-307, dated September 18, 2023), we have performed a preliminary geotechnical evaluation report for the proposed new mixed-use development located at 201 Golden Gate Avenue in San Francisco, California.

The purpose of our study was to generally evaluate site soil and geologic conditions, identify potential geotechnical constraints that may impact the proposed development, and provide preliminary geotechnical recommendations to aid in preparing the project Environmental Impact Report (EIR). Based on the results of our study, the proposed project is feasible from a geotechnical standpoint. The accompanying report presents the results of our study. An additional design-level geotechnical investigation will be required for project design and will likely include additional targeted subsurface investigation, in-situ testing, laboratory testing, engineering analysis and final report preparation.

Please contact us if you have any questions regarding this preliminary study or if we may be of further service.

Sincerely,

GEOCON CONSULTANTS, INC.

REGISTERED PROFESSIONAL ENGINEER
JEREMY J. ZORNE
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TABLE OF CONTENTS

PRELIMINARY GEOTECHNICAL INVESTIGATION PAGE

PRELIMINARY GEOTECHNICAL INVESTIGATION

1.0 PURPOSE AND SCOPE

This report presents results of our preliminary geotechnical evaluation for the proposed new mixed-use development located at 201 Golden Gate Avenue in San Francisco, California. The approximate site location is depicted on the Vicinity Map, Figure 1.

The purposes of our study were to generally evaluate subsurface conditions at the site, identify geotechnical constraints that may impact the proposed development, and provide preliminary geotechnical recommendations and design parameters to aid in preparing the project Environmental Impact Report (EIR). An additional design-level geotechnical investigation will be required for project design and will likely include additional targeted subsurface investigation, in-situ testing, laboratory testing, engineering analysis and final report preparation.

To prepare this preliminary report, we:

- Performed a literature review to aid in evaluating the geotechnical and geologic conditions present at the site. A list of referenced material is included in Section 8.0 of this report.
- Performed a site reconnaissance to evaluate exploration equipment access and mark out exploratory excavation locations for subsequent utility clearance.
- Notified subscribing utility companies via Underground Service Alert (USA) a minimum of 48 hours (as required by law) prior to performing exploratory excavations at the site.
- Retained the services of a private utility locator to clear exploration locations for existing utilities that may not be located by USA subscribers.
- Performed one (1) cone penetration test (CPT) sounding with shear wave velocity measurements within the existing private alley located in the southern portion of the site to a depth of approximately 50 feet.
- Upon completion, backfilled the sounding with neat cement grout.
- Prepared this report summarizing our findings, identifying geotechnical constraints that may impact the proposed development, and providing preliminary geotechnical recommendations and design parameters to assist in forward project planning.

Details of our field investigation program including logs of the CPT sounding and shear wave velocity measurements are presented in Appendix A. The approximate location of the CPT sounding is shown on the Site Plan, Figure 2. Details of our preliminary liquefaction analysis are presented in Appendix B.

2.0 SITE AND PROJECT DESCRIPTION

The approximately ½-acre site is currently developed with five adjoining one- to two-story commercial buildings located at 210, 209, 215, 243 and 247 Golden Gate Avenue occupying the majority of the site, a private alley along the southern portion of the site, and a Portland cement concrete (PCC) paved parking area in the southwest corner of the site. The site is bounded by Golden Gate Avenue to the north, Leavenworth Street to the east, Continuum Alley to the west and mixed-use buildings to the south. Site topography is relatively flat at approximate elevations ranging between 60 and 65 feet, referenced to the San Francisco City Datum (SFCD).

The project consists of redeveloping the site with a new mixed-use building consisting of a daylight basement level with 13 stories above. The new building will likely be of steel-frame construction and will be supported on deep foundations. The basement excavation will likely require excavations on the order of 10 feet deep, with some areas slightly deeper. Temporary excavation shoring will be required during construction. Associated improvements will likely include underground utility infrastructure, concrete flatwork and landscaping.

Our firm performed a preliminary geotechnical investigation for a Hastings College of Law building located at 333 Golden Gate Avenue (approximately 400 feet west of the site) and issued our report in March 2016. We performed an exploratory boring at the Hastings College of Law building (B1, Geocon 2015) to a depth of approximately 51½ feet.

Treadwell & Rollo, Inc. (TRI), performed a geotechnical investigation in 2001 for a parking structure located adjacent to and west of the Hastings College of Law building. Included in the TRI geotechnical investigation were six borings (B1 through B6) drilled to approximate depths of 51½ feet and two groundwater probes (P1 and P2) advanced to depths of 30 feet and 25 feet, respectively, for the purpose of measuring groundwater levels.

Information from our 2015 Boring B1 and TRI 2001 Borings B1 and B6 and Probe P2 has been incorporated into our geotechnical characterization of the site for the proposed new building and these previous boring and probe locations are shown on the Site Plan, Figure 2.

3.0 SOIL AND GEOLOGIC CONDITIONS

We identified soil and geologic conditions by reviewing previous exploratory excavations in the site vicinity, performing a CPT sounding at the site (CPT1) reviewing referenced geologic/geotechnical literature (Section 8.0). Soil and geologic conditions at the site generally consist of existing fill overlying dune sand. Descriptions provided below include the USCS symbol where applicable.

3.1 Site and Regional Geology

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 and also distributed, to a lesser extent, across a number of 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 as a result 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.

Based on the *Preliminary Geologic Map of the San Francisco South 7.5" Quadrangle and Part of the Hunters Point 7.5" Quadrangle*, United States Geological Survey (USGS, 1998), the site is located near the boundary of artificial fill (af) and Holocene age Dune Sand deposits (Map Symbol Qhs). A portion of the Geologic Map containing the site is presented as Figure 3.

3.2 Artificial Fill

Artificial fill was encountered in TRI Borings B1 and B6 (2001), in our Boring B1 (December 22, 2015) as well as in our recent CPT sounding (CPT1) to approximate depths of 15, 6, 11 and 8 feet, respectively. This artificial fill consists of medium dense sand with varying amounts of gravel, concrete and brick fragments (SP), and medium stiff sandy silt (ML).

3.3 Alluvium

Underlying the artificial fill in CPT1, we encountered soft, compressible clayey silt (ML) and silty clay (CL) to a depth of approximately 25 feet.

3.4 Dune Sands

Holocene-age dune sands were encountered below the artificial fill in TRI Borings B1 and B6 (2001) in our Boring B1 (2015) and below the alluvium in CPT1. The dune sands extended to the maximum depth explored of approximately 51½ feet and consisted of medium dense to dense sand with varying amounts of silt (SP, SM, SP-SM). In each boring, an approximately five to ten foot-thick, medium stiff to very stiff silt and clay (ML/CL) layer was encountered within the dune sand at depths varying between 20 and 30 feet.

4.0 GROUNDWATER

We encountered groundwater in our CPT1 at a depth of approximately 21 feet (approximately Elevation 36 feet SFCD) and at a depth of approximately 15 feet (approximately Elevation 37 feet SFCD) in Boring B1 on December 22, 2015. TRI encountered groundwater at an approximate depth of 23 feet (Elevation 28 feet SFCD) in Probe P2 on September 21, 2001. Based on historic groundwater information available from the California Department of Conservation, historic high groundwater in the site vicinity ranges between 10 and 30 feet below grade. A *Historically High Groundwater Contour Map* containing the site is presented as Figure 4. For the parking structure, TRI recommended using a design groundwater elevation of 27 feet SFCD. Given that we observed groundwater at a slightly higher elevation (approximately 36 to 37 feet SFCD) in CPT1 and Boring B1 on December 22, 2015, we recommend that the design-level geotechnical investigation include further evaluation of groundwater elevations at the site, especially if the proposed project includes below-grade levels.

It should be noted that fluctuations in the depth to groundwater may vary significantly due to changes in rainfall, temperature, localized pumping, irrigation practices, and seasonal fluctuations. Therefore, it is possible that groundwater may be higher or lower than the levels stated above.

5.0 SEISMICITY AND GEOLOGIC HAZARDS

5.1 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 the entire San Francisco 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). In the Bay Area, the movement is distributed across a complex system of strike-slip, right lateral parallel and subparallel faults, which include the San Andreas, Hayward and San Gregorio faults, among others.

To determine the distance of known active faults within 50 miles of the site, we used the 2013 *Caltrans Fault Database* KML overlay file for Google Earth. Principal references used within the 2013 Caltrans Fault Database are the *Fault Activity Map of California* (Jennings and Bryant 2010), *Working Group on California Earthquake Predictions* (WGCEP), and *Uniform California Earthquake Rupture Forecast Version 3*. The 12 closest faults are summarized in Table 5.1.

TABLE 5.1
REGIONAL ACTIVE FAULTS

Fault Name	Approximate Distance from Site (miles)	Maximum Moment Magnitude (M_W)
San Andreas (Peninsula) 2011	7.8	8.0
Hayward (North)	10.8	7.3
San Gregorio	11.0	7.4
Hayward (South)	11.7	7.3
Contra Costa Shear Zone 2011	18.8	6.5
Calaveras (North) 2011	20.6	6.9
Mount Diablo Thrust	21.3	6.6
Pleasanton	23.6	6.6
Concord 2011	24.2	6.6
Green Valley 2011	25.9	6.8
Rodgers Creek	26.3	7.3
Los Medanos – Roe Island	27.4	6.8

The faults tabulated above and numerous other faults 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 subject the site to intense ground shaking.

5.2 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 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.

5.3 Ground Shaking

We used the United States Geological Survey (USGS) *Unified Hazard Tool* (https://earthquake.usgs.gov/hazards/interactive/) to determine the deaggregated seismic source parameters, including controlling magnitude and fault distance. The USGS estimated modal magnitude is 7.8 and the estimated Peak Ground Acceleration (PGA) for the Maximum Considered Earthquake (MCE) with a 2,475-year return period is 0.81g.

5.4 Liquefaction

The site is located within a State of California Seismic Hazard Zone for liquefaction. A Seismic Hazard Zone Map containing the site is included as Figure 5. 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.

As a preliminary screening measure, we used the computer software program CLiq (Version 1.7, Geologmiski) and the in-situ soil parameters measured in the CPT sounding to perform this analysis. The software utilizes the 1998 National Center for Earthquake Engineering Research (NCEER) method of analysis which was developed with the broad consensus of national geotechnical and earthquake engineering experts. We assumed a typical seasonal high groundwater depth of 15 feet below existing grade, an earthquake magnitude of 7.8 (modal USGS deaggregation), and a maximum considered earthquake (MCE) event (2,475-year return interval event) peak ground acceleration (PGA) of 0.812g.

Based on the results of our analyses, there is the potential for liquefaction at the site within sandy soil layers generally present between depths of approximately 15 and 26 feet, and 42 to 46 feet. Results of our liquefaction analysis are presented in Appendix B.

Consequences of liquefaction may include ground surface settlement, ground loss (sand boils) and lateral slope displacements (lateral spreading). Due to the lack of a free-face geometry in the vicinity of the site, the potential for lateral spreading is considered low. 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 artificial fills that mantle the potentially liquefiable materials, 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 ground surface settlement. Our analysis indicates that, if liquefaction were to occur, total ground surface settlements on the order of 1 to 2 inches may result. It is typical geotechnical practice to assume that differential settlement would be approximately one-half of the total settlement. Therefore, differential settlement due to liquefaction could range from ½ to 1 inch over a distance of approximately 30 feet.

5.5 Tsunamis and Seiches

The site is not located within a coastal area and ground surface elevations are on the order of 60 to 65 feet SFCD. Therefore, tsunamis (seismic sea waves) are not considered a significant hazard at the site.

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.

6.0 CONCLUSIONS AND PRELIMINARY RECOMMENDATIONS

6.1 General

6.1.1 No soil or geologic conditions were encountered during our preliminary geotechnical evaluation that would preclude development of the project as presently proposed, provided the recommendations contained in this report and subsequent design-level geotechnical report are incorporated into the design and construction of the project.

6.1.2 Based on our findings, evaluation, and analyses to date, we have identified the following key geotechnical constraints:

- **Undocumented Fill:** An approximately 8-foot-thick layer of undocumented artificial fill was encountered in CPT1 within the proposed new building footprint. Due to the unknown placement history of the fill, removal, screening, and re-compaction may be required during site grading.
- **Existing Structures and Utilities:** Given that the site is currently developed with several existing structures, complete removal of any such structures and associated features will be required as part of site development.
- **Potentially Liquefiable Soil:** An approximately four- to eleven-foot-thick layer of potentially liquefiable soil is present at depths ranging between 15 to 26 and 42 to 46 feet below grade within the proposed new building footprint. Total settlement due to liquefaction could range from 1 to 2 inches and differential settlement due to liquefaction could range from ½ to 1 inch over a distance of approximately 30 feet. Deep foundation design should consider the impacts of liquefaction and be designed accordingly.
- **Adjacent Existing Structures:** The project may include mass excavation to achieve up to one level below grade for the proposed building. Temporary excavation shoring will be required and adjacent structures may require underpinning, shoring, or similar protection.

Discussion of geotechnical constraints and preliminary mitigation recommendations are provided herein.

6.1.3 Due to the presence of undocumented fill, soft, compressible soil and potentially liquefiable soils at the site, we recommend using deep foundations for support. Deep foundations would penetrate the existing undocumented fill, soft compressible soil and potentially liquefiable soil and bear within the underlying dense native dune sand deposits. In addition, the use of deep foundations would reduce potential adverse surcharge loading on adjacent structures. Preliminary foundation recommendations are provided herein.

6.1.4 Preliminary conclusions and recommendations presented herein are based on our review of the referenced literature, analysis of data obtained from our field investigation program, laboratory testing program, and our understanding of the project at this time. A design-level geotechnical investigation with additional targeted subsurface exploration, laboratory testing, and engineering analysis should be performed to provide detailed, design-level recommendations for the project.

6.2 Code-Based Seismic Design Values

6.2.1 We understand that seismic design of the proposed structures will be performed in accordance with the provisions of the 2022 *California Building Code* (CBC), the seismic provisions of which are based on the American Society of Civil Engineers (ASCE)/Structural Engineering Institute (SEI) publication: *ASCE/SEI 7-16, Minimum Design Loads and Associated Criteria for Buildings and Other Structures* (ASCE/SEI, 2017). We used the Structural Engineers Association of California (SEAOC) and Office of Statewide Health Planning and Development (OSHPD) web application *Seismic Design Maps* (https://seismicmaps.org/) to evaluate code-based seismic design parameters in accordance with ASCE 7-16.

6.2.2 For seismic design purposes, sites are classified as Site Class "A" through "F" as follows:

- Site Class A – Hard Rock;
- Site Class B – Rock;
- Site Class C – Very Dense Soil and Soft Rock;
- Site Class D – Stiff Soil;
- Site Class E – Soft Clay Soil; and
- Site Class F – Soils Requiring Site Response Analysis.

6.2.3 Based on the subsurface conditions at the site and the results of shear wave velocity measurements performed in CPT1, the Site Classification is Site Class "D" per Table 20.3-1 of ASCE/SEI 7-16. For the purposes of evaluating code-based seismic parameters for design, we assumed the building will have a seismic Risk Category II or III (per the CBC) for the project. Results are summarized in Table 6.2.3.

TABLE 6.2.3
ASCE 7-16 (CODE-BASED) SEISMIC DESIGN PARAMETERS
SITE CLASS “D” – STIFF SOIL

Parameter	Value	ASCE 7-16 Reference
MCE_R Ground Motion Spectral Response Acceleration – Class B (short), S_S	1.5g	Figure 22-1
MCE_R Ground Motion Spectral Response Acceleration – Class B (1 sec), S_1	0.6g	Figure 22-2
Site Coefficient, F_A	1.2	Table 11.4-1
Site Coefficient, F_V	1.7	Table 11.4-2
Site Class Modified MCE_R Spectral Response Acceleration (short), S_{MS}	1.8g	Eq. 11.4-1
Site Class Modified MCE_R Spectral Response Acceleration (1 sec), S_{M1}	1.53g*	Eq. 11.4-2
5% Damped Design Spectral Response Acceleration (short), S_{DS}	1.2g	Eq. 11.4-3
5% Damped Design Spectral Response Acceleration (1 sec), S_{D1}	1.02g*	Eq. 11.4-4
* Per Supplement 3 of ASCE7-16 (effective November 5, 2021), a ground motion hazard analysis (GMHA) shall be performed for projects on Site Class “D” sites with 1-second spectral acceleration (S_1) greater than or equal to 0.2g, which is true for this site. However, Supplement 3 of ASCE 7-16 provides an exception stating that that the GMHA may be waived provided that the parameter S_{M1} is increased by 50% for all applications of S_{M1}. The values for parameters S_{M1} and S_{D1} presented above have been increased in accordance with Supplement 3 of ASCE 7-16.		

6.2.4 Table 6.2.4 presents the mapped maximum considered geometric mean (MCE_G) seismic design parameters for projects located in Seismic Design Categories of D through F, in accordance with ASCE 7-16.

TABLE 6.2.4
ASCE 7-16 PEAK GROUND ACCELERATION PARAMETERS

Parameter	Value	ASCE 7-16 Reference
Mapped MCE_G Peak Ground Acceleration, PGA	0.554g	Figure 22-7
Site Coefficient, F_{PGA}	1.2	Table 11.8-1
Site Class Modified MCE_G Peak Ground Acceleration, PGA_M	0.665g	Section 11.8.3 (Eq. 11.8-1)

6.2.5 Conformance to the criteria presented in Tables 6.2.3 and 6.2.4 for seismic design does not constitute any kind of guarantee or assurance that significant structural damage or ground failure will not occur if a maximum level earthquake occurs. The primary goal of seismic design is to protect life and not to avoid structural damage, as such design may be economically prohibitive.

6.3 Anticipated Soil and Excavation Characteristics

6.3.1 In our opinion, grading and excavations at the site may be accomplished with standard effort using heavy-duty grading/excavation equipment. We do not anticipate project excavations to generate oversized rock material (greater than 6 inches in dimension) or boulders, although some debris (such as brick, wood, and concrete chunks) may be encountered in the existing fill.

6.3.2 Excavated soils generated from cut operations at the site are suitable for use as fill in structural areas provided they do not contain deleterious matter, organic material, or cementations larger than 6 inches in maximum dimension.

6.3.3 Import soil for general use (if needed) should be similar to onsite, native soils (i.e. similar plasticity and grain size distribution characteristics). Import soil should be free of organic material and construction debris, and not contain rock/cementations larger than 6 inches in greatest dimension.

6.3.4 Import fill material should be primarily granular with a “very low” expansion potential (Expansion Index less than 20), a Plasticity Index less than 15, be free of organic material and construction debris, and not contain rock/cementations larger than 6 inches in greatest dimension. Low-expansive fill may also consist of Caltrans Class 2 AB or lime-treated native soils.

6.3.5 Environmental characteristics and corrosion potential of import soil materials should also be considered. Proposed import materials should be sampled, tested, and approved by Geocon prior to its transportation to the site.

6.3.6 Temporary excavation slopes must meet Cal-OSHA requirements as appropriate. We anticipate that the majority of excavations in existing fill and native dune sand deposits soils will be classified as Cal-OSHA “Type C” soil. Excavation sloping, benching, the use of trench shields, and the placement of trench spoils should conform to the latest applicable Cal-OSHA standards. The contractor should have a Cal-OSHA-approved “competent person” onsite during excavation to evaluate trench conditions and to make appropriate recommendations where necessary. It is the contractor’s responsibility to provide sufficient and safe excavation support as well as protecting nearby utilities, structures, and other improvements which may be damaged by earth movements.

6.3.7 Permanent cut and fill slopes should be constructed no steeper than 2H:1V. To mitigate potential erosion, slopes should be vegetated as soon as possible, and surface drainage should be directed away from the tops of slopes.

6.4 Preliminary Grading Recommendations

6.4.1 The following grading recommendations are preliminary and intended for planning purposes only. Specific grading recommendations should be provided in the design-level geotechnical report.

6.4.2 References to relative compaction and optimum moisture content in this report are based on the American Society for Testing and Materials (ASTM) D1557 Test Procedure, latest edition. As used in this report, the structural building pad area is defined as the area extending a minimum of 5 feet horizontally beyond the outside dimensions of the structure, including footings.

6.4.3 Prior to commencing grading, a pre-construction conference with representatives of the client, grading contractor, and Geocon should be held at the site. Site preparation, soil handling and/or the grading plans should be discussed at the pre-construction conference.

6.4.4 To prepare the site, remove surface/subsurface structures, underground utilities and associated backfill/pipe embedment materials, and debris. Restore excavations or depressions resulting from site clearing operations, or other existing excavations or depressions, with engineered fill.

6.4.5 Within the building pad area, the top 2 feet of existing fill should be removed and re-compacted as engineered fill in order to provide suitable slab-on-grade support.

6.3.6 Over-excavated areas, areas to receive fill, or areas left at-grade should be thoroughly scarified to a minimum depth of 12 inches, uniformly moisture-conditioned at or near optimum moisture content, and compacted to at least 90% relative compaction. A representative of the geotechnical engineer should observe scarification and re-compaction operations to evaluate performance of the subgrade under compaction equipment loading and to identify any areas that may require additional removals or stabilization.

6.4.7 Engineered fill should be compacted in horizontal lifts not exceeding 8 inches (loose thickness) and brought to final subgrade elevations. Each lift should be moisture-conditioned at or near optimum moisture content and compacted to at least 90% relative compaction.

6.4.8 The upper 12 inches of final flatwork subgrade, whether completed at-grade, by excavation, or by filling, should be uniformly moisture-conditioned at or near optimum moisture content and compacted to at least 90% relative compaction.

6.4.9 Underground utility trenches within structural areas should be backfilled with properly compacted material. Pipe bedding, shading, and backfill should conform to the requirements of the appropriate utility authority. Material excavated from trenches should be adequate for use as general backfill above shading provided it does not contain deleterious matter, vegetation or cementations larger than 3 inches in maximum dimension. Trench backfill should be placed in loose lifts not exceeding 8 inches. Lifts should be mechanically compacted to a minimum of 90% relative compaction and at least 2% above optimum moisture content.

6.5 Deep Foundation Preliminary Recommendations

6.5.1 The following preliminary foundation recommendations are provided for planning purposes. Specific foundation recommendations should be provided as part of the design-level geotechnical investigation once the building details and structural foundation loading conditions are known.

6.5.2 Deep foundations would penetrate the existing undocumented fill, soft, compressible soil and potentially liquefiable soil and bear within the underlying dense native dune sand deposits. In addition, the use of deep foundations would reduce potential adverse surcharge loading on adjacent structures. There is a wide variety of deep foundation "pile" types available and each pile type behaves differently depending on installation and construction methods. Each pile type has specific advantages and disadvantages with respect to structural capacity, constructability, installation rates, cost, and a host of other factors. The two major pile types include (1) manufactured fixed-length piles, such as pre-cast concrete, steel, or timber piles, and (2) drilled, cast-in-place piles. Each of these pile types includes "displacement" and "non-displacement" versions. Displacement piles move the soil laterally during installation (i.e. does not excavate or remove the soil) while non-displacement piles either cut through the soil (in the case of driven piles) or removes the soil (in the case of drilled piles). Displacement piles typically develop higher capacity due to the densification achieved as a result of soil displacement; however, they typically induce higher vibrations during installation.

6.5.3 The use of fixed-length, driven piles can be problematic due to early refusal and/or deeper penetration than designed; both of which may require post-installation modifications to the pile such as cutting or splicing, which can add significant cost. For this site, driven displacement piles may encounter early refusal in the dense dune sand deposits. In addition, pile driving noise and potential vibrations may be undesirable for the project due to adjacent structures and improvements. Therefore, we do not recommend the use of fixed-length, driven displacement piles for the project. However, driven non-displacement piles, such as steel H-Piles may be used. For preliminary planning purposes, driven steel HP 10x42 or 12x53 piles on the order of 40 to 50 feet long will likely provide an allowable axial compression capacity of 140 kips (70 tons). Actual pile lengths should be evaluated as part of the future design-level geotechnical investigation for the project.

6.5.4 Alternatively, if it is desired to reduce vibrations and noise during construction, conventional non-displacement, auger cast pressure grout (APG) piles may be used. APG piles are installed using a plugged continuous flight auger that is advanced into ground. Once the desired depth is reached, the plug is removed and high-strength grout is pumped under pressure as the auger is withdrawn. As the auger is withdrawn, the soil retained on the auger is removed from the hole and replaced with grout placed under pressure. After the auger is removed, the required steel reinforcement is then "wet-set" into the pile to complete the installation. This pile type produces approximately 100% to 120% of the theoretical hole volume of spoils. APG piles 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. For preliminary planning purposes, 16-inch diameter APG piles on the order of 40 to 50 feet long will likely provide an allowable axial compression capacity of 140 kips (70 tons) or higher. Actual pile lengths should be evaluated by the design-build contractor by performing a comprehensive pile installation and load testing program to evaluate constructability as well as capacity.

6.5.5 Deep foundations generate vertical load-carrying capacity from a combination of side friction and end-bearing. However, seismic-induced liquefaction may impart negative skin friction resulting in down drag forces on the piles. Pile foundation capacity should be evaluated in accordance with Section 12.13.9.3.1 of ASCE 7-16 in consideration of liquefaction-induced downdrag.

6.6 Retaining Walls

6.6.1 At the time of this report, the structure edge limits, details, and types of permanent retaining walls are not defined. New retaining walls will likely consist of the walls associated with the potential subterranean portion of the building (basement walls). Preliminary design of retaining walls and buried structures may be based on the lateral earth pressures (equivalent fluid pressure) summarized in Table 6.6.

TABLE 6.6
PRELIMINARY LATERAL EARTH PRESSURES

Condition	Equivalent Fluid Density
Active (Above Groundwater)	40 pcf
Active (Below Groundwater)	85 pcf
At-Rest (Above Groundwater)	60 pcf
At-Rest (Below Groundwater)	95 pcf
Passive (Above Groundwater)	250 pcf
Passive (Below Groundwater)	125 pcf
Seismic Earth Pressure[1]	15 pcf

1. Applicable for walls that support more than 6 feet of backfill in accordance with Section 1803.5.12 of the 2013 CBC. Conventional triangular distribution. Should be combined with ACTIVE lateral earth pressure for seismic case analysis.

6.6.2 Unrestrained walls should be designed using the active case. Unrestrained walls are those that are allowed to rotate more than 0.001H (where H is the height of the wall). Walls restrained from movement (such as basement walls) should be designed using the at-rest case (or active + seismic if higher). The soil pressures above assume drained conditions and that the backfill material within an area bounded by the wall and a 1:1 plane extending upward from the base of the wall will be composed of the existing onsite soils.

6.6.3 Additional pressures should be added for surcharge conditions due to sloping ground, vehicular traffic or adjacent structures and should be designed for each condition as part of the future design-level geotechnical investigation for the project. If not designed for hydrostatic conditions, retaining walls should be provided with drainage systems and waterproofed as required by the project architect.

6.7 Concrete Sidewalks and Flatwork

6.7.1 Sidewalk, curb, and gutter within City right-of-way should be designed and constructed in accordance with the latest City of San Francisco standards and details as applicable. Onsite Exterior concrete flatwork, not subject to traffic loads, should be at least 4 inches thick and underlain by at least 4 inches of compacted Class 2 aggregate base (AB).

6.7.2 Construction joints and control joints should be provided in accordance with ACI and/or PCA guidelines and should be constructed as soon as practical following concrete placement. Crack control joints should extend a minimum depth of one-fourth the slab thickness. The project structural engineer and/or architect should design construction joints as necessary.

6.8 Rigid Concrete Pavement

6.8.1 Rigid concrete pavement may be used in vehicular traffic areas, such as loading and parking. Based on the soil conditions encountered at the site, concrete pavement should consist of at least 6 inches Portland Cement Concrete (PCC) overlying at least 6 inches of Class 2 AB meeting the requirements of Section 26 of the Caltrans Standard Specifications.

6.8.2 Subgrade soils should be prepared in accordance with the recommendations of the geotechnical report. Class 2 AB and subgrade should be compacted to at least 95% relative compaction near optimum moisture content. Subgrade should be proof-rolled with a loaded water truck to verify stability.

6.8.3 Concrete should have a minimum 28-day compressive strength of 4,000 psi. Adequate construction and crack control joints should be used to control cracking inherent in concrete construction. It would be advantageous to provide minimal reinforcement, such as No. 3 steel bars placed 18 inches on center in both horizontal directions to help control cracking. Consideration should be given to providing maximum control joint spacing of 12 feet in both directions for a 6-inch-thick slab. Adequate dowels should also be used at joints to facilitate load transfer and reduce vertical offset. In addition, the recommendations above pertaining to depended curbs, moisture cut-offs, and subsurface drainage applies to concrete pavements, sidewalks and flatwork, as well as asphalt pavements.

6.8.4 In general, we recommend that concrete pavements be designed, constructed and maintained in accordance with industry standards such as those provided by the American Concrete Pavement Association.

6.9 Site Drainage and Moisture Protection

6.9.1 Proper site drainage is critical to reduce the potential for differential soil movement, soil expansion, erosion and subsurface seepage. Under no circumstances should water be allowed to pond adjacent to building foundations. The site should be graded and maintained such that surface drainage is directed away from structures in accordance with the 2013 CBC or other applicable standards. In addition, surface drainage should be directed away from the top of slopes into swales or other controlled drainage devices.

6.9.2 Underground utilities should be leak free. Utility and irrigation lines should be checked periodically for leaks, and detected leaks should be repaired promptly. Detrimental soil movement could occur if water is allowed to infiltrate the soil for prolonged periods of time.

6.9.3 Landscaping planters adjacent to paved areas are not recommended due to the potential for surface or irrigation water to infiltrate the pavement's subgrade and base course. We recommend that area drains to collect excess irrigation water and transmit it to drainage structures or impervious above-grade planter boxes be used. In addition, where landscaping is planned adjacent to the pavement, we recommend construction of a cutoff wall (deepened concrete curb, plastic root barrier, or similar cutoff) along the edge of the pavement that extends at least 4 inches into the soil subgrade below the bottom of the base material.

6.9.4 We recommend that roof drains be connected to water-tight drainage piping connected to the storm drain system. However, we understand that Leadership in Engineering and Environmental Design (LEED) requests disconnecting the roof drains to help obtain certification. At a minimum, the water from the roof drains should be directed away from buildings. Consideration should be given to draining roofs to lined planter boxes or placing liners below the proposed landscape areas to prevent infiltration of the water. Geocon can be contacted for additional recommendations.

6.9.5 Experience has shown that even with these provisions, subsurface seepage may develop in areas where no such water conditions existed prior to site development. This is particularly true where a substantial increase in surface water infiltration has resulted from an increase in landscape irrigation.

6.10 Design-Level Geotechnical Investigation

6.10.1 An additional design-level geotechnical investigation will be required after schematic design is complete and building layout and anticipated structural foundation loading is available.

6.10.2 Based on the results of our preliminary geotechnical investigation, we recommend the design-level geotechnical investigation include the following additional investigative activities:

- Additional exploratory borings or in-situ testing (e.g. cone penetration or dilatometer testing) to further evaluate foundation support conditions.
- Additional evaluation of seasonal high groundwater elevation at the site.
- Additional soil sampling and laboratory analysis.

6.10.3 We can prepare a proposal for the design-level geotechnical investigation once schematic design is complete.

7.0 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, we should be notified so that supplemental recommendations can be given. The evaluation or identification of the potential presence of hazardous materials or environmental contamination was not part of our scope of services.

The conclusions and recommendations contained in this report are preliminary and intended for forward project planning purposes only. Additional design-level geotechnical investigation(s) will be necessary prior to final design.

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. Additionally, 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 partially or wholly 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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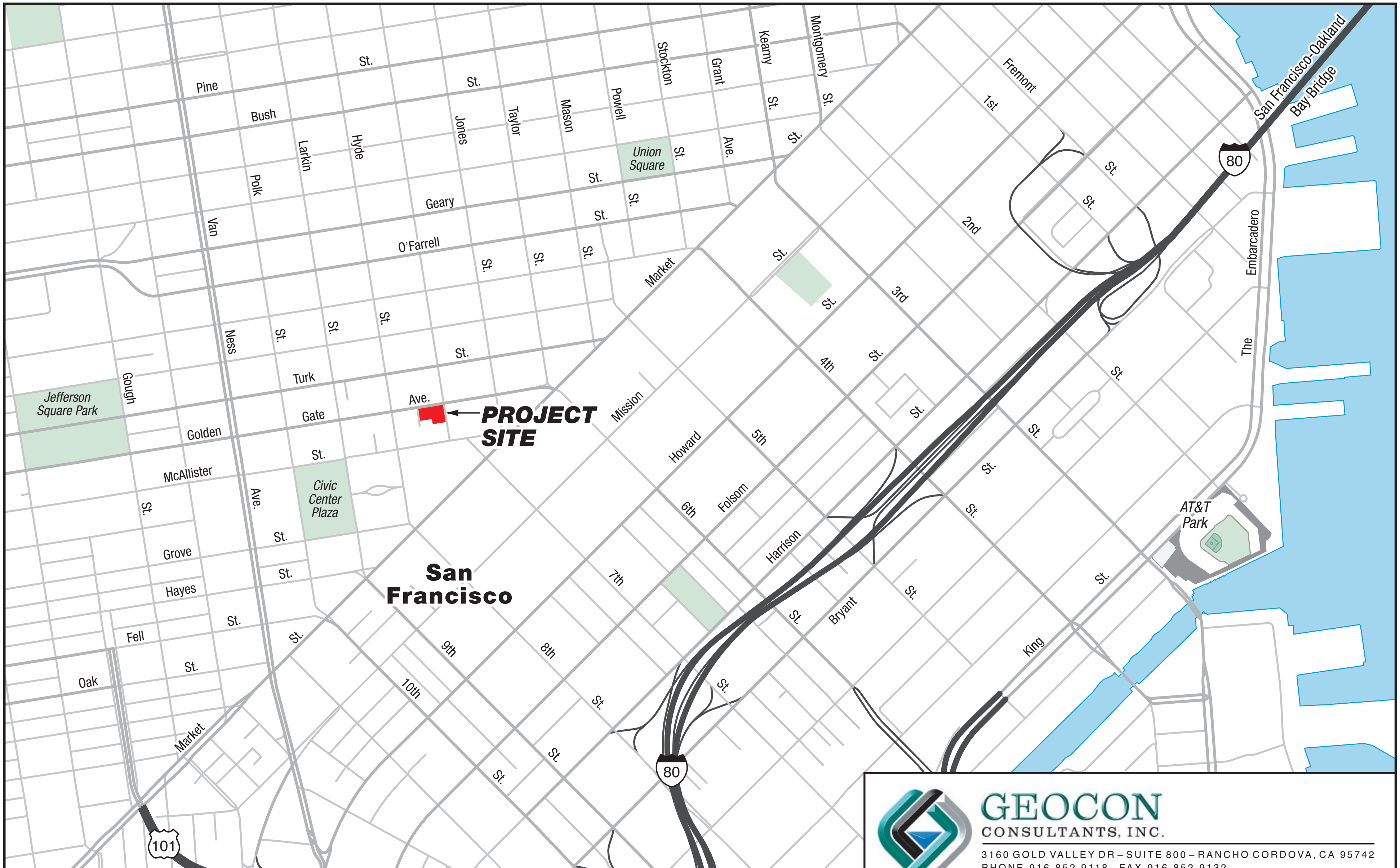

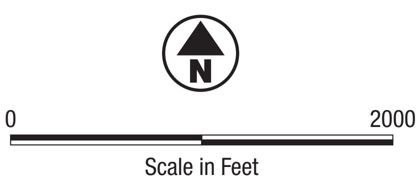

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201 Golden Gate Avenue
San Francisco, California

VICINITY MAP

S2673-05-01	December 2023	Figure 1

Legend

- CPT1 Approximate Cone Penetration Test (CPT) Location (Geocon, 2023)
- B1 Approximate Previous Boring Location (Geocon, 2015)
- B6 Approximate Previous Boring Location (Treadwell & Rollo, 2001)
- Approximate Site Boundary

N

0 60
Scale in Feet

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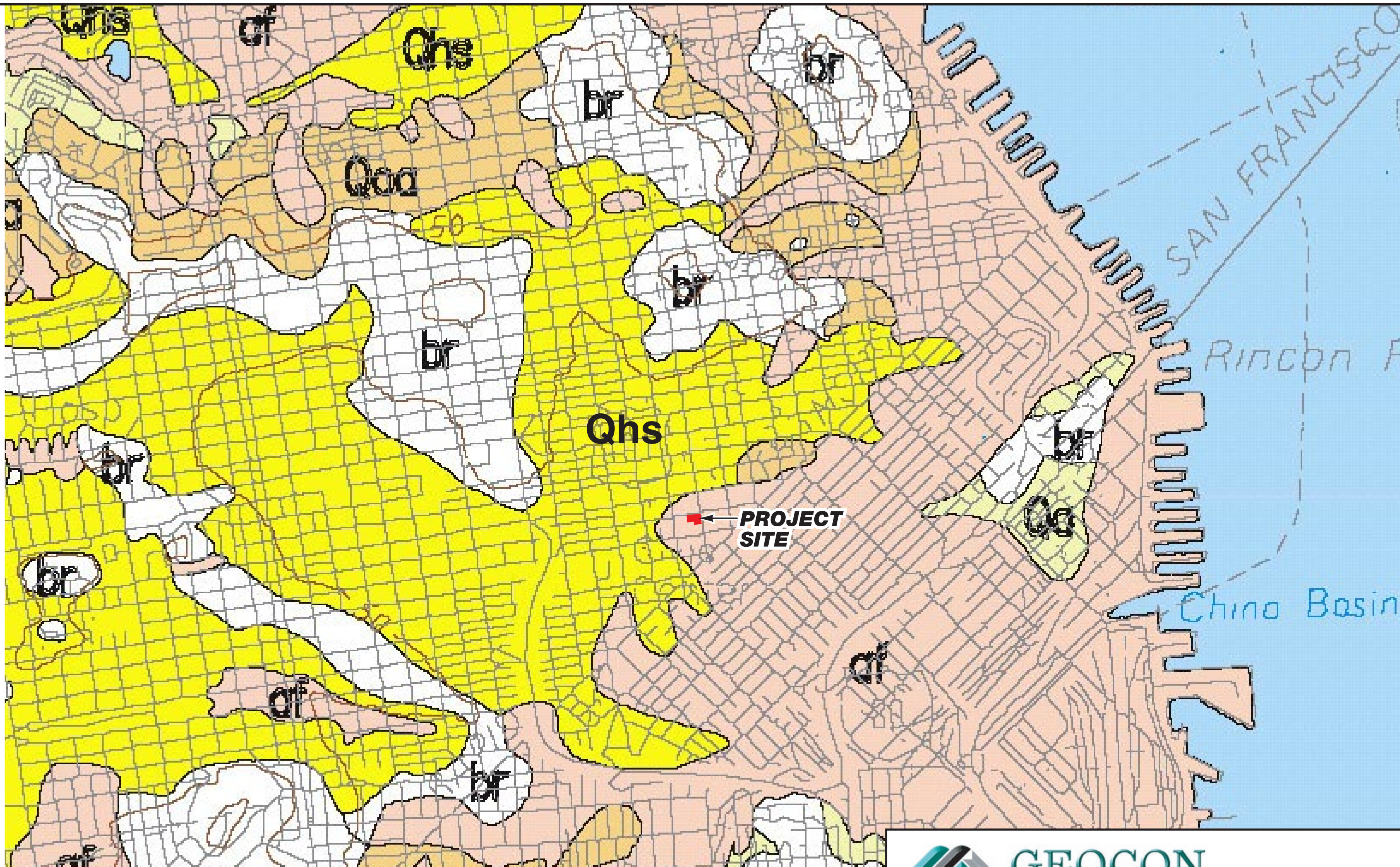
SITE PLAN

S2673-05-01	December 2023	Figure 2



Ref:

af	Artificial fill
Qhs	Holocene dune and beach sand
Qa	Late Pleistocene to Holocene alluvium, undifferentiated
Qoa	Early of middle Pleistocene alluvial deposits
br	Pre-Quaternary deposits and bedrock, undifferentiated

N

0 2000

Scale in Feet

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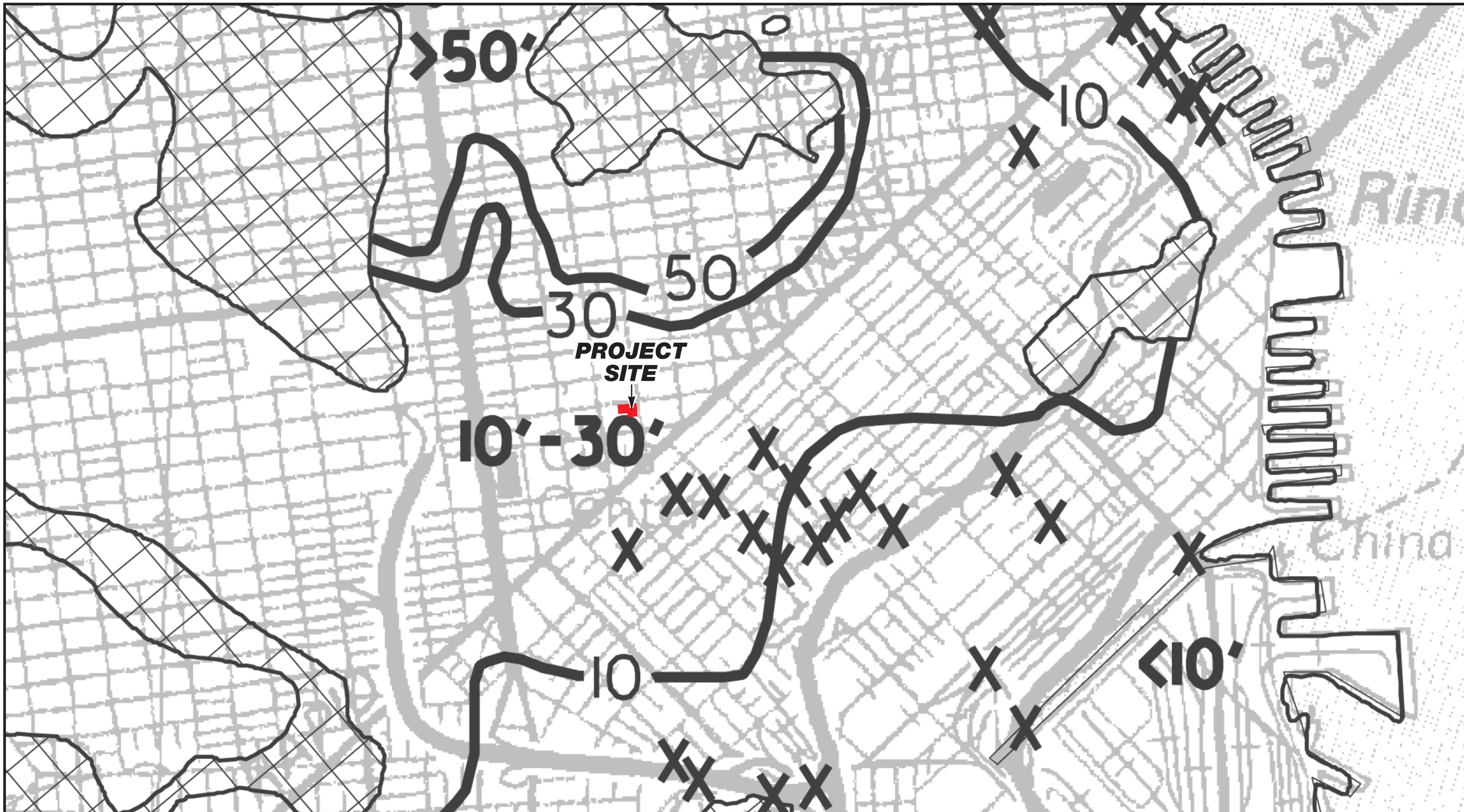
GEOLOGIC MAP

S2673-05-01	December 2023	Figure 3



Map Ref: *Seismic Hazard Zone Report of the City and County of San Francisco, California, Plate 1.2*
Historically highest ground water contours, city and county of San Francisco,
Department of Conservation, Division of Mines and Geology, 2000

X Site of Historical Earthquake-Generated Liquefaction

30 Depth to Groundwater in Feet

10'-30' Generalized Groundwater Levels

Bedrock

N

0 2000
Scale in Feet

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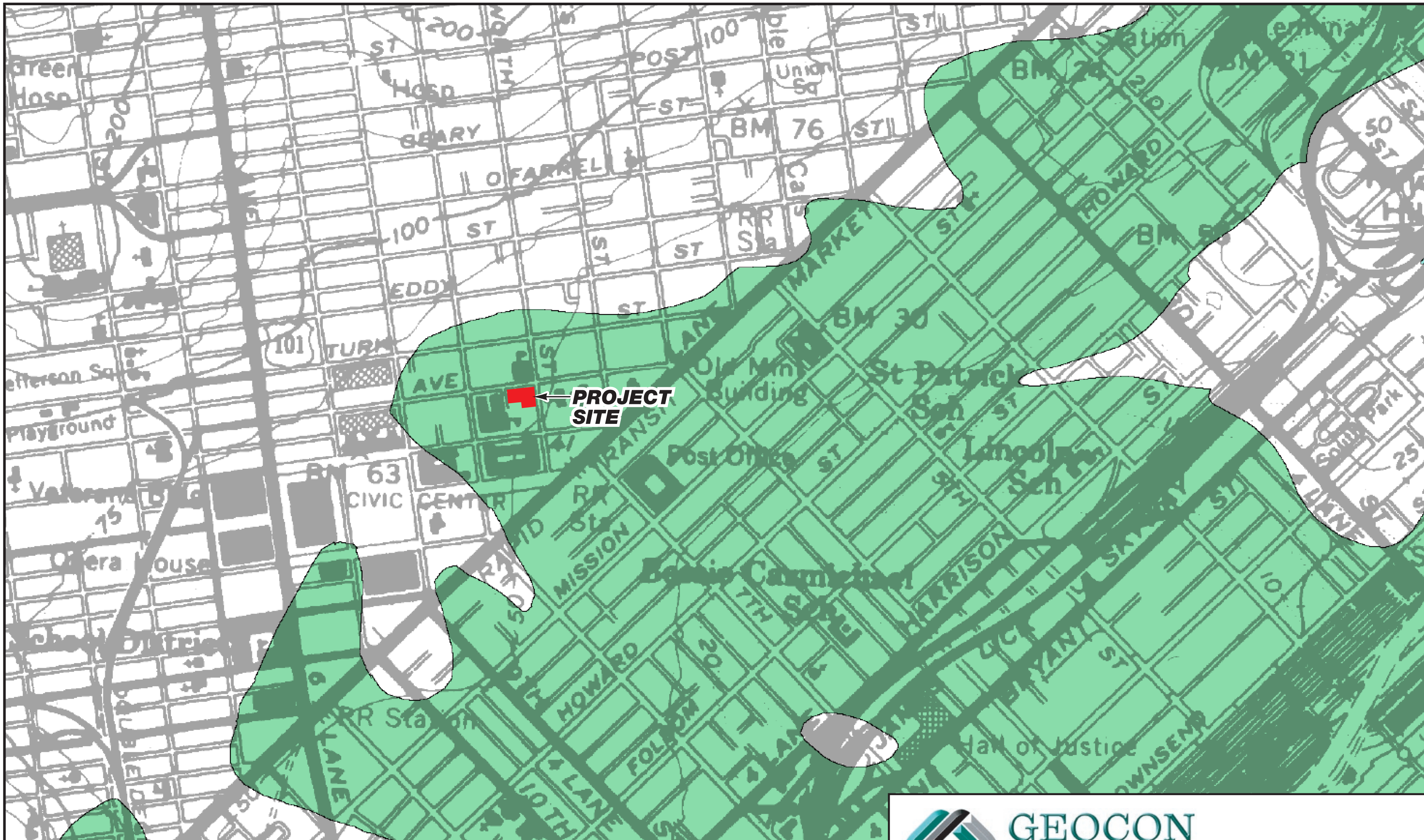
201 Golden Gate Avenue San Francisco, California		
Historically High Groundwater Contour Map		
S2673-05-01	December 2023	Figure 4



Map Ref: State of California - Seismic Hazard Zones, City and County of San Francisco Offical Map, Released Nov. 17, 2000

Liquefaction

Areas where historic occurrence of liquefaction, or local geological, geotechnical and groundwater conditions indicate a potential for permanent ground displacements such that mitigation as defined in Public Resources Code Section 2693(c) would be required.

N

0 1000

Scale in Feet

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SEISMIC HAZARD ZONE MAP

S2673-05-01	December 2023	Figure 5

APPENDIX A

APPENDIX A

FIELD EXPLORATION

Our field exploration program was performed on November 22, 2023, and of advancing g one cone penetration test (CPT) sounding (CPT1). The approximate CPT locations is shown on the Site Plan, Figure 2.

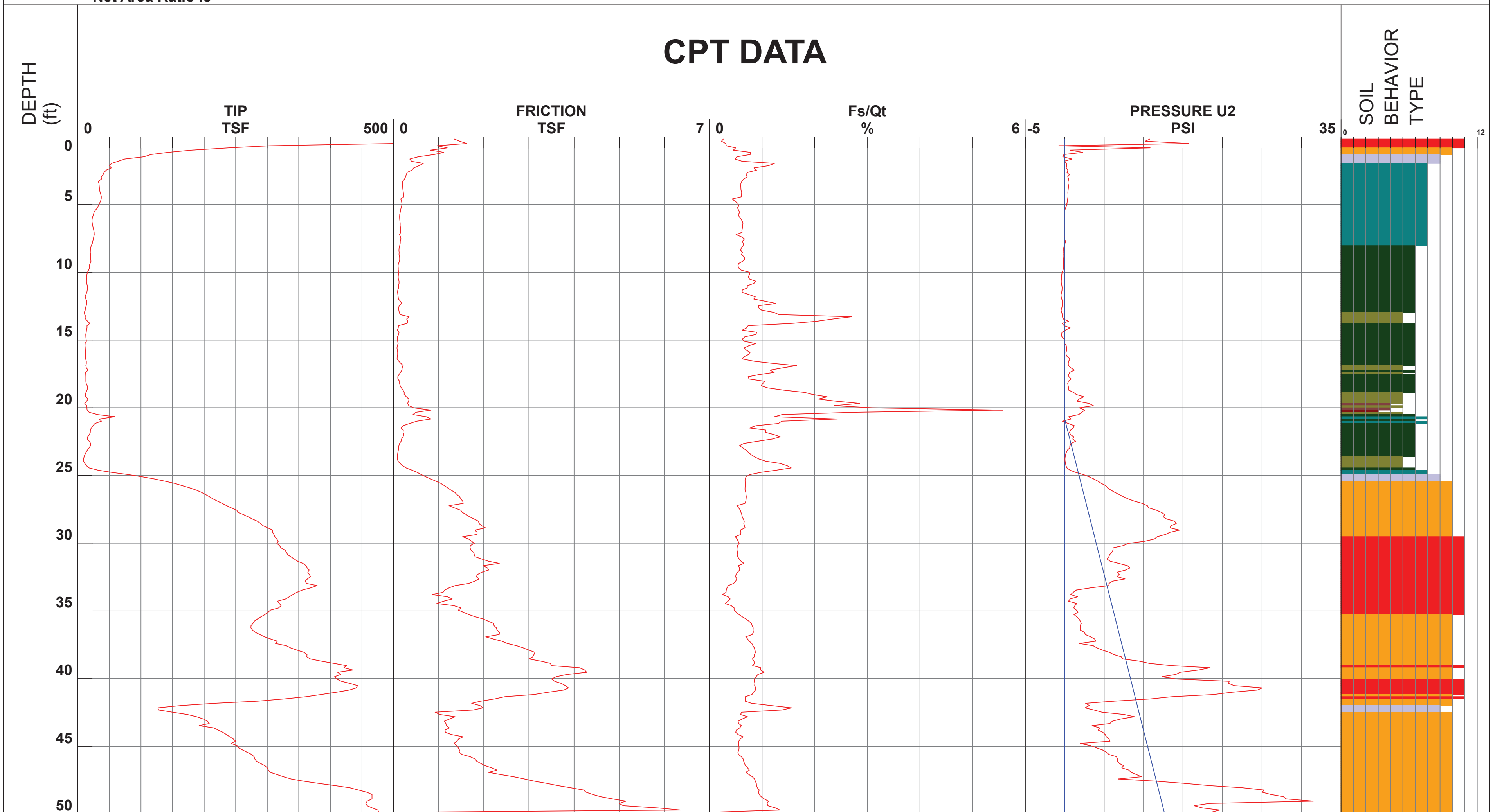
The CPT sounding was performed using a 20-ton truck-mounted CPT rig. CPT parameters, including tip resistance (q_c), sleeve friction (f_s) and dynamic pore pressure (U), were measured at approximate 2-inch intervals as the cone advanced. Soil behavior types were determined using correlations by Lunne, Robertson and Powell (1997). Seismic shear wave velocity measurements were performed at approximate 5-foot intervals in the sounding. After completion, the sounding was backfilled with neat cement grout.

Logs of the CPT sounding and shear wave velocity measurements are included as Figures A1 and A2. A Key to Logs, Boring Log B1 (Geocon 2015), and copies of the TRI Boring B1 and B6 boring logs are attached.

Subsurface conditions encountered in the borings were visually examined, classified and logged in general accordance with the American Society for Testing and Materials (ASTM) Practice for Description and Identification of Soils (Visual-Manual Procedure D2488-90). This system uses the Unified Soil Classification System (USCS) for soil designations. The logs depict the soil and geologic conditions encountered and the depths at which samples were obtained. The logs also include our interpretation of the conditions between sampling intervals. Therefore, the logs contain both observed and interpreted data. We determined the lines designating the interface between soil materials on the logs using visual observations, excavation characteristics and other factors. The transition between the materials may be abrupt or gradual. Where applicable, the field logs were revised based on subsequent laboratory testing.

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 15cm²

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

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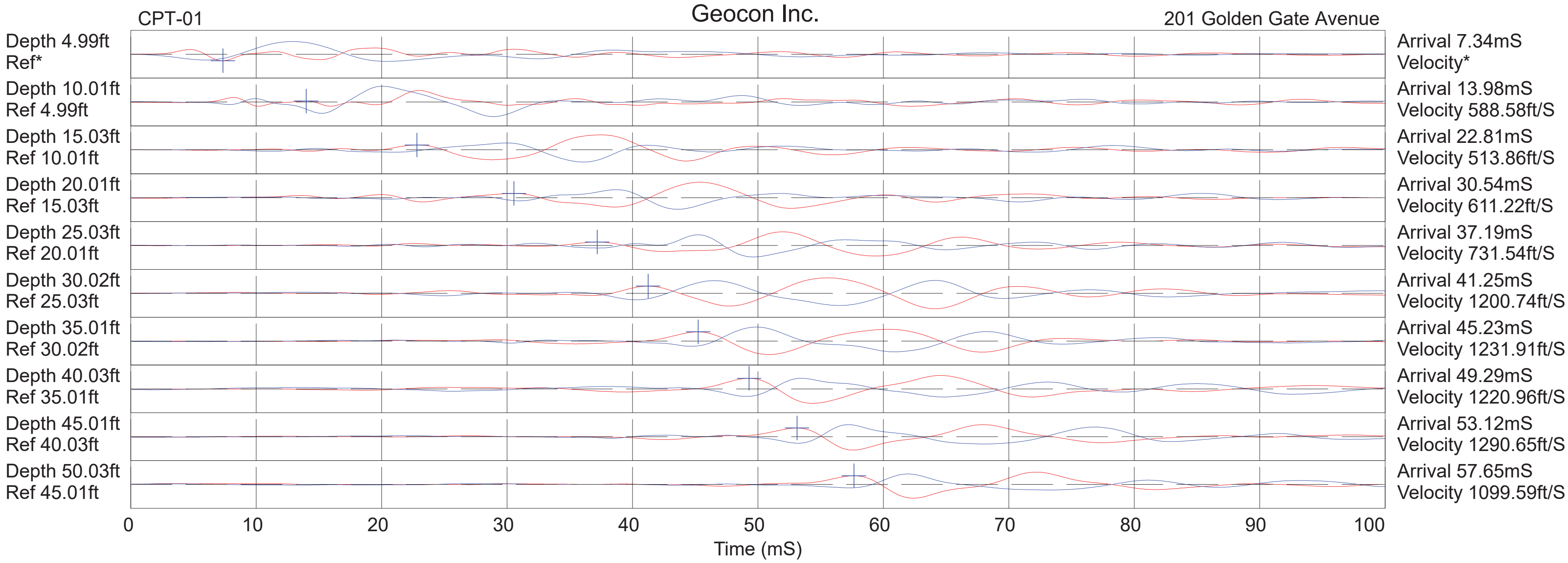
CPT1

201 Golden Gate Avenue Mixed-Use Building		
201 Golden Gate Avenue San Francisco, California		
S2673-05-01	December 2023	Figure A1



Hammer to Rod String Distance (ft): 5.83
* = Not Determined

COMMENT:

GEOCON CONSULTANTS, INC. 3160 GOLD VALLEY DR – SUITE 800 – RANCHO CORDOVA, CA 95742 PHONE 916.852.9118 – FAX 916.852.9132	**CPT1 SHEAR WAVE VELOCITY PLOT**	201 Golden Gate Avenue Mixed-Use Building 201 Golden Gate Avenue San Francisco, California S2673-05-01 · December 2023 · Figure A2

UNIFIED SOIL CLASSIFICATION

MAJOR DIVISIONS				TYPICAL NAMES
COARSE-GRAINED SOILS MORE THAN HALF IS COARSER THAN NO. 200 SIEVE	**GRAVELS** MORE THAN HALF COARSE FRACTION IS LARGER THAN NO.4 SIEVE SIZE	CLEAN GRAVELS WITH LITTLE OR NO FINES	GW	WELL GRADED GRAVELS WITH OR WITHOUT SAND, LITTLE OR NO FINES
			GP	POORLY GRADED GRAVELS WITH OR WITHOUT SAND, LITTLE OR NO FINES
		GRAVELS WITH OVER 12% FINES	GM	SILTY GRAVELS, SILTY GRAVELS WITH SAND
			GC	CLAYEY GRAVELS, CLAYEY GRAVELS WITH SAND
	SANDS MORE THAN HALF COARSE FRACTION IS SMALLER THAN NO.4 SIEVE SIZE	CLEAN SANDS WITH LITTLE OR NO FINES	SW	WELL GRADED SANDS WITH OR WITHOUT GRAVEL, LITTLE OR NO FINES
			SP	POORLY GRADED SANDS WITH OR WITHOUT GRAVEL, LITTLE OR NO FINES
		SANDS WITH OVER 12% FINES	SM	SILTY SANDS WITH OR WITHOUT GRAVEL
			SC	CLAYEY SANDS WITH OR WITHOUT GRAVEL
FINE-GRAINED SOILS MORE THAN HALF IS FINER THAN NO. 200 SIEVE	**SILTS AND CLAYS** LIQUID LIMIT 50% OR LESS		ML	INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOUR, SILTS WITH SANDS AND GRAVELS
			CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, CLAYS WITH SANDS AND GRAVELS, LEAN CLAYS
			OL	ORGANIC SILTS OR 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, FAT CLAYS
			OH	ORGANIC CLAYS OR CLAYS OF MEDIUM TO HIGH PLASTICITY
	HIGHLY ORGANIC SOILS		PT	PEAT AND OTHER HIGHLY ORGANIC SOILS

BORING/TRENCH LOG LEGEND

- No Recovery
- Shelby Tube Sample
- Bulk Sample
- SPT Sample
- Modified California Sample
- Groundwater Level (At Completion)
- Groundwater Level (Seepage)

PENETRATION RESISTANCE

SAND AND GRAVEL			SILT AND CLAY			
RELATIVE DENSITY	BLOWS PER FOOT (SPT)*	BLOWS PER FOOT (MOD-CAL)*	CONSISTENCY	BLOWS PER FOOT (SPT)*	BLOWS PER FOOT (MOD-CAL)*	COMPRESSIVE STRENGTH (tsf)
VERY LOOSE	0 - 4	0 - 6	VERY SOFT	0 - 2	0 - 3	0 - 0.25
LOOSE	5 - 10	7 - 16	SOFT	3 - 4	4 - 6	0.25 - 0.50
MEDIUM DENSE	11 - 30	17 - 48	MEDIUM STIFF	5 - 8	7 - 13	0.50 - 1.0
DENSE	31 - 50	49 - 79	STIFF	9 - 15	14 - 24	1.0 - 2.0
VERY DENSE	OVER 50	OVER 79	VERY STIFF	16 - 30	25 - 48	2.0 - 4.0
			HARD	OVER 30	OVER 48	OVER 4.0

*NUMBER OF BLOWS OF 140 LB HAMMER FALLING 30 INCHES TO DRIVE LAST 12 INCHES OF AN 18-INCH DRIVE

MOISTURE DESCRIPTIONS

FIELD TEST	APPROX. DEGREE OF SATURATION, S (%)	DESCRIPTION
NO INDICATION OF MOISTURE; DRY TO THE TOUCH	$S<25$	DRY
SLIGHT INDICATION OF MOISTURE	$25 \le S<50$	DAMP
INDICATION OF MOISTURE; NO VISIBLE WATER	$50 \le S<75$	MOIST
MINOR VISIBLE FREE WATER	$75 \le S<100$	WET
VISIBLE FREE WATER	100	SATURATED

QUANTITY DESCRIPTIONS

APPROX. ESTIMATED PERCENT	DESCRIPTION
<5%	TRACE
5 - 10%	FEW
11 - 25%	LITTLE
26 - 50%	SOME
>50%	MOSTLY

GRAVEL/COBBLE/BOULDER DESCRIPTIONS

CRITERIA	DESCRIPTION
PASS THROUGH A 3-INCH SIEVE AND BE RETAINED ON A NO. 4 SIEVE (#4 TO 3")	GRAVEL
PASS A 12-INCH SQUARE OPENING AND BE RETAINED ON A 3-INCH SIEVE (3"-12")	COBBLE
WILL NOT PASS A 12-INCH SQUARE OPENING (>12")	BOULDER

BEDDING SPACING DESCRIPTIONS

THICKNESS/SPACING	DESCRIPTOR
GREATER THAN 10 FEET	MASSIVE
3 TO 10 FEET	VERY THICKLY BEDDED
1 TO 3 FEET	THICKLY BEDDED
3 5/8-INCH TO 1 FOOT	MODERATELY BEDDED
1 1/4-INCH TO 3 5/8-INCH	THINLY BEDDED
3/8-INCH TO 1 1/4-INCH	VERY THINLY BEDDED
LESS THAN 3/8-INCH	LAMINATED

STRUCTURE DESCRIPTIONS

CRITERIA	DESCRIPTION
ALTERNATING LAYERS OF VARYING MATERIAL OR COLOR WITH LAYERS AT LEAST 1/4-INCH THICK	STRATIFIED
ALTERNATING LAYERS OF VARYING MATERIAL OR COLOR WITH LAYERS LESS THAN 1/4-INCH THICK	LAMINATED
BREAKS ALONG DEFINITE PLANES OF FRACTURE WITH LITTLE RESISTANCE TO FRACTURING	FISSURED
FRACTURE PLANES APPEAR POLISHED OR GLOSSY, SOMETIMES STRIATED	SLICKENSIDED
COHESIVE SOIL THAT CAN BE BROKEN DOWN INTO SMALLER ANGULAR LUMPS WHICH RESIST FURTHER BREAKDOWN	BLOCKY
INCLUSION OF SMALL POCKETS OF DIFFERENT SOIL, SUCH AS SMALL LENSES OF SAND SCATTERED THROUGH A MASS OF CLAY	LENSED
SAME COLOR AND MATERIAL THROUGHOUT	HOMOGENOUS

CEMENTATION/INDURATION DESCRIPTIONS

FIELD TEST	DESCRIPTION
CRUMBLES OR BREAKS WITH HANDLING OR LITTLE FINGER PRESSURE	WEAKLY CEMENTED/INDURATED
CRUMBLES OR BREAKS WITH CONSIDERABLE FINGER PRESSURE	MODERATELY CEMENTED/INDURATED
WILL NOT CRUMBLE OR BREAK WITH FINGER PRESSURE	STRONGLY CEMENTED/INDURATED

IGNEOUS/METAMORPHIC ROCK STRENGTH DESCRIPTIONS

FIELD TEST	DESCRIPTION
MATERIAL CRUMBLES WITH BARE HAND	WEAK
MATERIAL CRUMBLES UNDER BLOWS FROM GEOLOGY HAMMER	MODERATELY WEAK
1/8-INCH INDENTATIONS WITH SHARP END FROM GEOLOGY HAMMER	MODERATELY STRONG
HAND-HELD SPECIMEN CAN BE BROKEN WITH **ONE** BLOW FROM GEOLOGY HAMMER	STRONG
HAND-HELD SPECIMEN CAN BE BROKEN WITH **COUPLE** BLOWS FROM GEOLOGY HAMMER	VERY STRONG
HAND-HELD SPECIMEN CAN BE BROKEN WITH **MANY** BLOWS FROM GEOLOGY HAMMER	EXTREMELY STRONG

IGNEOUS/METAMORPHIC ROCK WEATHERING DESCRIPTIONS

DEGREE OF DECOMPOSITION	FIELD RECOGNITION	ENGINEERING PROPERTIES
SOIL	DISCOLORED, CHANGED TO SOIL, FABRIC DESTROYED	EASY TO DIG
COMPLETELY WEATHERED	DISCOLORED, CHANGED TO SOIL, FABRIC MAINLY PRESERVED	EXCAVATED BY HAND OR RIPPING (Saprolite)
HIGHLY WEATHERED	DISCOLORED, HIGHLY FRACTURED, FABRIC ALTERED AROUND FRACTURES	EXCAVATED BY HAND OR RIPPING, WITH SLIGHT DIFFICULTY
MODERATELY WEATHERED	DISCOLORED, FRACTURES, INTACT ROCK-NOTICEABLY WEAKER THAN FRESH ROCK	EXCAVATED WITH DIFFICULTY WITHOUT EXPLOSIVES
SLIGHTLY WEATHERED	MAY BE DISCOLORED, SOME FRACTURES, INTACT ROCK-NOT NOTICEABLY WEAKER THAN FRESH ROCK	REQUIRES EXPLOSIVES FOR EXCAVATION, WITH PERMEABLE JOINTS AND FRACTURES
FRESH	NO DISCOLORATION, OR LOSS OF STRENGTH	REQUIRES EXPLOSIVES

IGNEOUS/METAMORPHIC ROCK JOINT/FRACTURE DESCRIPTIONS

FIELD TEST	DESCRIPTION
NO OBSERVED FRACTURES	UNFRACTURED/UNJOINTED
MAJORITY OF JOINTS/FRACTURES SPACED AT 1 TO 3 FOOT INTERVALS	SLIGHTLY FRACTURED/JOINTED
MAJORITY OF JOINTS/FRACTURES SPACED AT 4-INCH TO 1 FOOT INTERVALS	MODERATELY FRACTURED/JOINTED
MAJORITY OF JOINTS/FRACTURES SPACED AT 1-INCH TO 4-INCH INTERVALS WITH SCATTERED FRAGMENTED INTERVALS	INTENSELY FRACTURED/JOINTED
MAJORITY OF JOINTS/FRACTURES SPACED AT LESS THAN 1-INCH INTERVALS; MOSTLY RECOVERED AS CHIPS AND FRAGMENTS	VERY INTENSELY FRACTURED/JOINTED

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PHONE 916.852.9118 – FAX 916.852.9132

KEY TO LOGS

Figure A1

BORING B1

ELEV. (MSL.) **52** DATE COMPLETED **12/22/2015**

ENG./GEO. **Sean Dixon** DRILLER **V & W**

EQUIPMENT **CME 55 with Mud Rotary** HAMMER TYPE **Automatic**

DEPTH IN FEET	SAMPLE NO.	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0				**ASPHALT (AC)** 5 inches			
				AGGREGATE BASE (AB) 12 inches			
1			SP	**FILL** Medium dense, moist, gray, Poorly graded SAND with gravel, concrete and brick			
2	B1-Bulk						
3							
4							
5	B1-5.5			- fine sand, little silt			
6	B1-6.0				42	108.1	5.2
7							
8							
9							
10	B1-10.5						
11	B1-11.0		SP	**DUNE SAND** Medium dense, moist, reddish brown, fine SAND, some silt	35	104.4	4.3
12							
13							
14							
15	B1-15.5	▼		- becomes wet			
16	B1-16.0				41	106.0	18.7
17							
18							
19							

Figure A2, Log of Boring, page 1 of 3

GEOCON

SAMPLE SYMBOLS

- ... SAMPLING UNSUCCESSFUL
- ... DISTURBED OR BAG SAMPLE
- ... STANDARD PENETRATION TEST
- ... CHUNK SAMPLE
- ... DRIVE SAMPLE (UNDISTURBED)
- ▼ ... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

BORING B1

ELEV. (MSL.) **52** DATE COMPLETED **12/22/2015**

ENG./GEO. **Sean Dixon** DRILLER **V & W**

EQUIPMENT **CME 55 with Mud Rotary** HAMMER TYPE **Automatic**

DEPTH IN FEET	SAMPLE NO.	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
20	B1-20.5					
21	B1-21.0			46	108.4	20.6
22						
23						
24						
25	B1-25.5	CL	Very stiff, moist to wet, brown, Sandy CLAY, with some roots/grass/organics			
26	B1-26.0			47	126.6	12.4
27						
28						
29		ML	Medium stiff, wet, SILT with sand			
30	B1-30.5					
31	B1-31.0			16	114.0	17.9
32						
33		SM	Dense, wet, gray, Poorly graded SAND with silt			
34						
35	B1-35.5					
36	B1-36.0			85	114.5	18.6
37						
38		SP-SM	- less sand			
39						

Figure A3, Log of Boring, page 2 of 3

GEOCON

SAMPLE SYMBOLS ... SAMPLING UNSUCCESSFUL ... STANDARD PENETRATION TEST ... DRIVE SAMPLE (UNDISTURBED) ... DISTURBED OR BAG SAMPLE ... CHUNK SAMPLE ... WATER TABLE OR SEEPAGE

PROJECT NO. **S9961-05-04** PROJECT NAME **Proposed Hastings College of the Law**

BORING B1

ELEV. (MSL.) **52** DATE COMPLETED **12/22/2015**

ENG./GEO. **Sean Dixon** DRILLER **V & W**

EQUIPMENT **CME 55 with Mud Rotary** HAMMER TYPE **Automatic**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
40	B1-40.0				- less silt	30		
41								
42								
43								
44								
45	B1-45.0					43		
46								
47								
48								
49								
50	B1-50.0				- becomes reddish brown	39		
51								

BORING TERMINATED AT 51.5 FEET
GROUNDWATER ENCOUNTERED AT 15 FEET
BACKFILLED WITH NEAT CEMENT GROUT

Figure A4, Log of Boring, page 3 of 3

IN PROGRESS S9961-05-04 HASTINGS COLLEGE OF THE LAW.GPJ 01/22/16

GEOCON

SAMPLE SYMBOLS
- ... SAMPLING UNSUCCESSFUL
- ... DISTURBED OR BAG SAMPLE
- ... STANDARD PENETRATION TEST
- ... CHUNK SAMPLE
- ... DRIVE SAMPLE (UNDISTURBED)
- ... WATER TABLE OR SEEPAGE

PROJECT:	HASTINGS C[illegible]LEGE OF LAW PARKING STRUCTURE San Francisco, California	Log of Boring B-1	PAGE 1 OF 2

Boring location: See Site Plan, Figure 2

Date started: 9/5/01 | Date finished: 9/5/01

Drilling method: Rotary Wash

Hammer weight/drop: 140lbs./30-inches | Hammer type: Safety with Rope & Pulley

Sampler: Sprague & Henwood (S&H), Standard Penetration Test (SPT)

Logged by: A. M. Oillarburu

LABORATORY TEST DATA

Surface Elevation: 51 feet[2]

DEPTH (feet)	Sampler Type	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
0				4-inches Asphalt Concrete						
1–2				Concrete slab, about 5-inches thick						
3–5			CL	SANDY CLAY with GRAVEL (CL) orange brown, stiff, moist						
6	S&H	14							13.0	115
6–15			SP	SAND (SP) yellow brown, medium dense, moist, with some concrete brick and mortar fragments (FILL, 0–15 ft)						
11	SPT	26	SP							
16	S&H	31	SP	SAND (SP) yellow brown, dense, moist				3.2	10.8	102
21	SPT	38								
22			SP							
26	S&H	22		medium dense, wet				2.7	21.3	103
29			CL-							
30										

TEST GEOTECH LOG 280403.GPJ T&R.GDT 11/29/01

Treadwell&Rollo

Project No.: 2804.03 | Figure: A-1a

PROJECT: HASTINGS COLLEGE OF LAW
PARKING STRUCTURE
San Francisco, California

Log of Boring B-1

DEPTH (feet)	SAMPLES: Sampler Type	SAMPLES: Sample	SAMPLES: SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	LABORATORY TEST DATA: Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
31	S&H		12	ML	SANDY CLAY/SANDY SILT (CL/ML) brown, medium stiff to stiff, wet				59.2	17.4	114
32				CL-ML	SANDY CLAY/SANDY SILT (CL/ML) orange-brown, stiff, wet						
33					SAND (SP) orange-brown, very dense, wet, with silt						
34											
35											
36	SPT		55								
37											
38											
39											
40											
41	S&H		30/ 6"								
42				SP							
43											
44											
45											
46	SPT		62		brown						
47											
48											
49											
50											
51	SPT		76								
52											
53											
54											
55											
56											
57											
58											
59											
60											

Boring terminated at 51.5 feet below ground surface.
Boring backfilled with cement grout.
Groundwater obscured due to drilling method.

[1] S&H blow counts converted to SPT N-Values using a factor of 0.6.
[2] Elevations based on San Francisco City datum.

Treadwell&Rollo

Project No.: 2804.03

Figure: A-1b

TEST GEOTECH LOG 280403.GPJ T&R.GDT 11/29/01

PROJECT:	HASTINGS COLLEGE OF LAW PARKING STRUCTURE San Francisco, California	Log of Boring B-6
		PAGE 1 OF 2

Boring location: See Site Plan, Figure 2	Logged by: A. M. Oillarburu
Date started: 9/5/01	Date finished: 9/5/01
Drilling method: Rotary Wash	
Hammer weight/drop: 140lbs./30-inches	Hammer type: Safety with Rope & Pulley
Sampler: Sprague & Henwood (S&H), Standard Penetration Test (SPT)	

DEPTH (feet)	Sampler Type	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION (Surface Elevation: 51 feet[2])	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
0				3.5-inches Asphalt Concrete						
0–2			SC	FILL: CLAYEY SAND with GRAVEL (SC) brown and orange brown, moist with brick rubble						
2–6			SM	FILL: SILTY SAND with GRAVEL (SM) brown, moist, with brick and concrete rubble; 5 to 6-inch thick concrete rubble fragments						
6			SP	SAND (SP) yellow brown, medium dense, moist						
7	SPT	24								
11	SPT	24								
16	SPT	30		orange brown, medium dense to dense, wet						
20								2.4	23.2	
20.5			SM	SILTY SAND (SM) dark brown, medium dense, wet						
21	SPT	10								
22.5			SC-SM	CLAYEY SAND/SILTY SAND (SC/SM) brown, medium dense, wet				42.7	14.2	122
23	S&H	16								
26	S&H	8		brown to orange brown, loose, increased clay content LL = 19, PI = 5; see Figure B-4	TxUU	2,500	560	45.2	17.3	122
28			SP	SAND (SP) orange brown, dense, wet						
30										

TEST GEOTECH LOG 280403.GPJ T&R.GDT 11/29/01

Treadwell&Rollo

Project No.: 2804.03 | Figure: A-6a

PROJECT:	HASTINGS C[illegible]LEGE OF LAW PARKING STRUCTURE San Francisco, California	Log of Boring B-6	PAGE 2 OF 2

DEPTH (feet)	Sampler Type	Sample	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
					SAND (SP) (continued)						
31	S&H		46								
32											
33											
34											
35											
36	SPT		64		very dense						
37											
38											
39											
40											
41	SPT		41	SP	olive-brown, dense						
42											
43											
44											
45	S&H		30/ 5"								
46					very dense						
47											
48											
49											
50											
51	SPT		73								
52											
53											
54											
55											
56											
57											
58											
59											
60											

Boring terminated at 51.5 feet below ground surface.
Boring backfilled with cement grout.
Groundwater obscured due to drilling method.

[1] S&H blow counts converted to SPT N-Values using a factor of 0.6.
[2] Elevations based on San Francisco City datum.

Treadwell&Rollo

Project No.: 2804.03 | Figure: A-6b

TEST GEOTECH LOG 280403.GPJ T&R.GDT 10/9/01

APPENDIX B

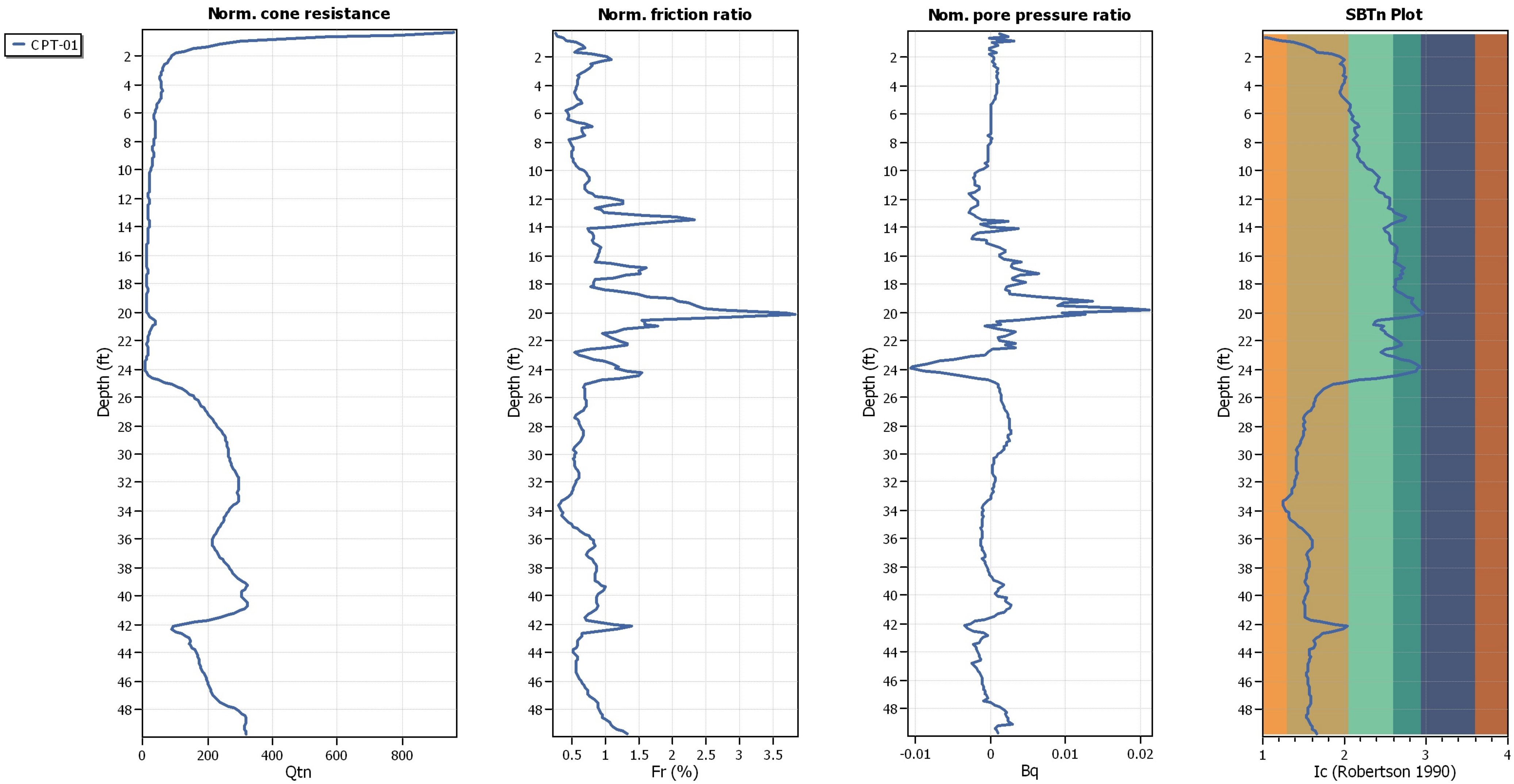
APPENDIX B

LIQUEFACTION ANALYSIS

GEOCON

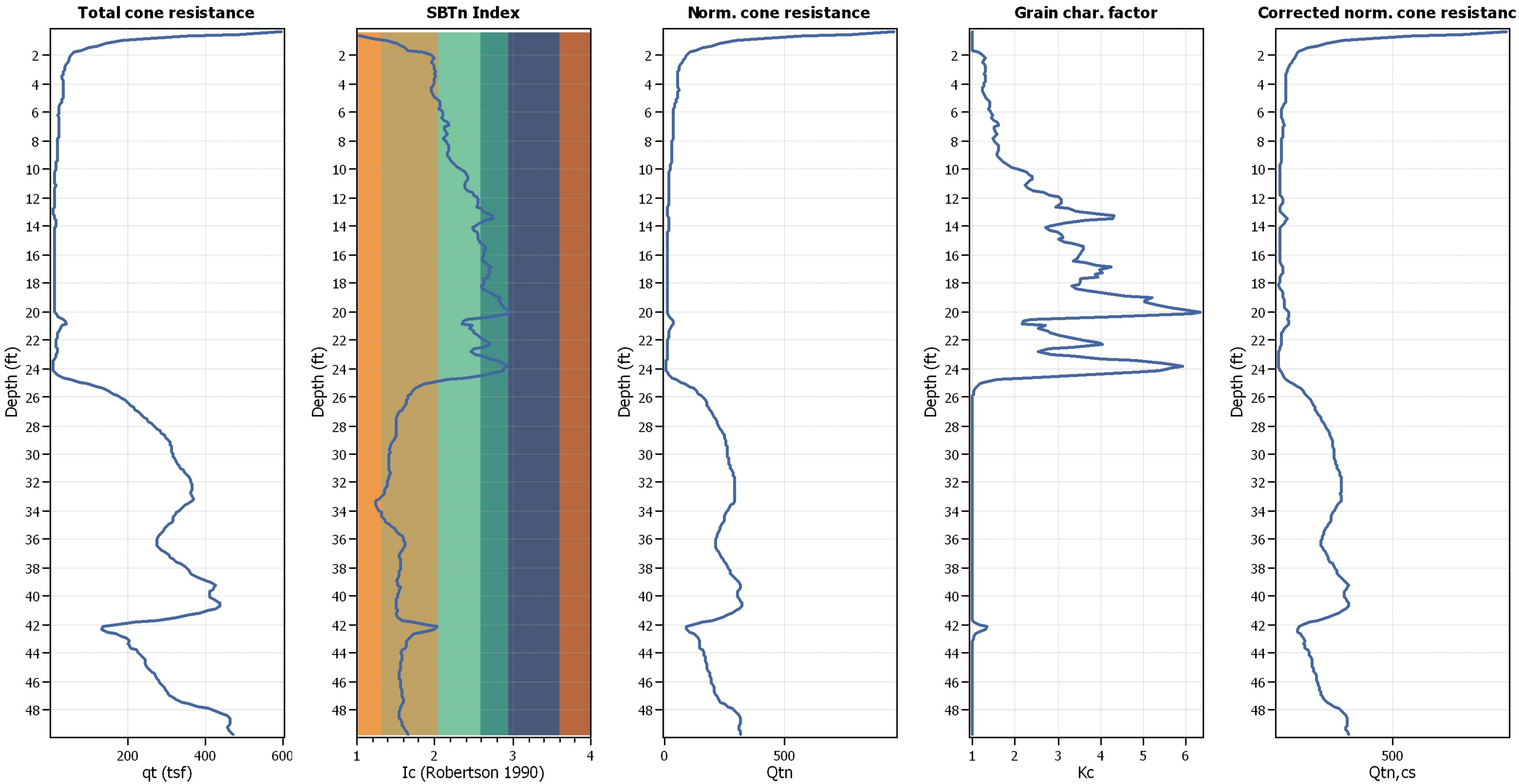
Project: UCCL Golden Gate

Overlay Normalized Plots



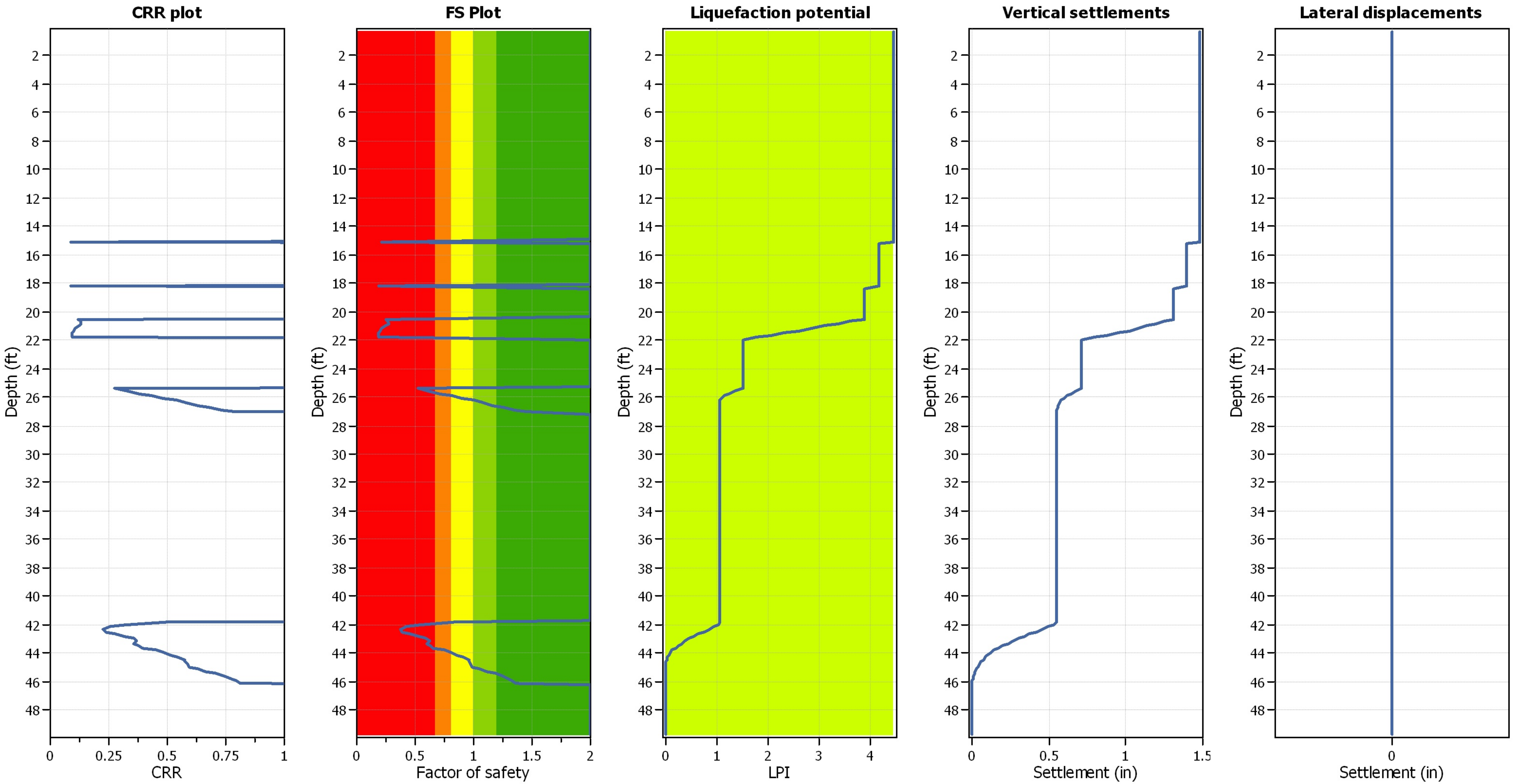
Project: UCCL Golden Gate

Overlay Intermediate Results



Project: UCCL Golden Gate

Overlay Cyclic Liquefaction Plots



Project: UCCL Golden Gate

Overlay Strength Loss Plots