Appendix IS-2

Geotechnical Report

Final Report

GEOTECHNICAL INVESTIGATION REPORT PROPOSED MIXED-USE DEVELOPMENT 708 S CLOVERDALE AVENUE

LOS ANGELES, CALIFORNIA

Prepared for

Onni Group
1031 S Broadway, Suite 400
Los Angeles, CA 90003

October 17, 2022

Prepared by

GeoPentech
101 Academy, Suite 100
Irvine, CA 92617
(714) 796-9100
Fax (714) 796-9191
Web Site: www.geopentech.com

October 17, 2022
Project No.: 22010A

Ms. Brigid Williams
Development Manager
Onni Group
1031 S Broadway, Suite 400
Los Angeles, CA 90015

Subject: **Geotechnical Investigation Report**
Proposed Mixed-Use Development
708 S Cloverdale Avenue
Los Angeles, California 90036

Dear Ms. Williams:

This report presents the results of GeoPentech's geotechnical investigation for the proposed mixed-use development to be located at 708 South Cloverdale Avenue in Los Angeles, California. This investigation was performed in general accordance with our agreement dated February 9, 2022.

This report provides geotechnical recommendations for the design and construction of the project in accordance with the plans provided to us. Current field and laboratory test results, as well as geologic hazard evaluation and details of the ground-motion evaluation, are also included in the report.

Thank you for providing GeoPentech with the opportunity to participate in this project. If you have any questions or require additional information, please call.

Very truly yours,

GeoPentech, Inc.

Mandro M. Eslami, Ph.D., PE
Project Engineer

James Heins, EIT
Staff Engineer

Rambod Hadidi, Ph.D., PE, GE
Associate

Steve Duke, PG, PGp, CEG, CHg
Associate

REPORT REVISION HISTORY

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1	Draft for Review and Comment	7/20/2022
2	Final Draft for Review and Comment	8/30/2022
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TABLE OF CONTENTS

List of Tables

List of Figures

Appendices

1.0 INTRODUCTION

This report presents the results of GeoPentech's geotechnical investigation for the proposed mixed-use development that includes a high-rise tower (Tower) and the associated low-rise Podium (Podium) to be located at 708 S Cloverdale Avenue in Los Angeles, California (34.061605° N, -118.345947° W). The general location of the project site is shown on Figure 1, and the extent of the projects site is shown on Figure 2. The Project site spans the following addresses and parcel numbers:

Street Address, Los Angeles, CA	APN
5366, 5368, 5370, 5374, and 5376 W. Wilshire Blvd. 706, 708, 710, 712, and 714 S. Coverdale Ave.	5089002025
5358, 5360, 5364, 5362 Wilshire Blvd.	5089002026
5354, 5356 Wilshire Blvd.	5089002002
5350, 5352 Wilshire Blvd.	5089002003
716 S Coverdale Ave.	5089002019
721 S Detroit St.	5089002004
725 S Detroit St.	5089002005

This report was prepared in accordance with the agreement between GeoPentech and Onni Group dated February 9, 2022.

2.0 PROJECT DESCRIPTION

Our understanding of the project is based on information provided by Onni Group as well as architectural drawings provided by MVE+ Partners dated September 22, 2022. Architectural views of the proposed development are shown on Figure 3.

We understand that the proposed development includes the design and construction of 46 story Highrise Tower with an approximate total height of 530 ft above ground surface, including a five (5) story Podium with a total of nine (9) parking levels; five (5) above ground and four (4) below ground parking. The Project also includes existing commercial buildings along Wilshire Boulevard. These commercial buildings would be retained for use without changes to the structure or foundation of these buildings. As such, these commercial buildings are not further discussed in this report. The extents of these buildings are shown on Figure 2, and architectural views of the proposed construction are show on Figure 3. The lowest grade at the site is currently about elevation +192 ft (NAVD88). We understand that the height of the Tower will be about 530 feet above the lowest current grade (i.e., elevation +722 ft), and the height of the Podium will be about 61 ft above grade (i.e., elevation +253 ft). The lowest subterranean floor level will be at elevation +130 ft, about 62 ft below the ground surface, and the excavation of the subterranean levels to the anticipated bottom of the foundation level will extend as deep as 76 feet below the lowest current grade or to elevation +116 ft. The approximate extents of the proposed buildings are shown on Figure 2. The subterranean and the Podium section of the proposed structure will nearly cover the entire site footprint except for the existing commercial buildings along Wilshire Blvd. Level 6 will include an amenity deck spanning roughly the eastern half of the remaining site footprint. Levels 7 through 43 will comprise the high-rise portion of the Tower covering the western half of the site. We understand that the average bearing pressure under the Tower is estimated to be on the order of 12,000 psf. The average bearing pressure under the Podium is estimated to be on the order of 3 ksf.

We understand that the project, including geotechnical aspects of the design, will be submitted for review and approval to Los Angeles Department of Building Safety (LADBS) in conformance with the 2019 California Building Code (CBC 2019), ASCE 7-16 requirements. Furthermore, the seismic design of the Tower will be subject to LADBS's Peer Review Process. Accordingly, we have assumed that the design for this structure will be carried out in conformance with the 2019 California Building Code (CBC 2019), ASCE 7-16 requirements, and the performance-based design procedure as specified by the Los Angeles Tall Buildings Design Council, 2020 edition (LATBDC 2020).

This report presents the results of GeoPentech's geotechnical investigation (including field exploration) as well as design recommendations for the Tower and the associated Podium portion of the proposed mixed-use development.

3.0 SCOPE OF WORK

GeoPentech's scope of work for this report included the following:

- ***Review of Existing Information*** – GeoPentech reviewed existing geotechnical, geologic, and seismic information for the site as well as the currently proposed development plans.
- ***Field Investigation and Laboratory Testing*** – to investigate the nature and stratigraphy of the subsurface materials and to obtain soil samples for laboratory testing, we drilled three (3) borings, two within the Tower footprint and one within the Podium footprint to depths between 91 and 131.5 feet below the existing ground surface. Furthermore, we performed three (3) Cone Penetration tests (CPT), two within the Tower footprint and one within the Podium footprint to depths between approximately 36 and 71 feet below the existing ground surface. Additionally, geophysical surveys were performed to measure shear-wave (S-wave) velocity profile for the project site.

The borings were drilled using hollow-stem auger drilling equipment with the addition of drilling mud when groundwater was encountered. The approximate locations GeoPentech's borings and CPTs are shown on Figure 2. Select soil samples were taken to a geotechnical laboratory for testing, including soil index testing and strength testing.

- ***Geologic-Seismic Hazards Evaluation*** – Evaluated site subsurface conditions, geologic setting, and assessed seismic conditions and geologic-seismic hazards and their potential impact on the subject project.
- ***Ground-Motion Evaluation*** – Completed a site-specific ground-motion hazard analysis in accordance with the requirements of the 2019 CBC and, 7-16, and LATBC 2020 guidelines.
- ***Engineering Analysis*** – Performed engineering evaluation of the geotechnical data to develop recommendations for design of foundations, walls below grade, shoring, excavation, earthwork criteria, and paving.
- ***Preparation of this report.***

4.0 EXISTING SITE CONDITIONS

As shown on Figure 2, the site is bounded by S. Cloverdale Avenue to the west, Wilshire Boulevard to the north, existing 2-story residential development to the south, and S Detroit Street to the east. The Metro Purple Line is also located north of the project along Wilshire Boulevard. The Project site is predominately occupied by a parking lot and 1-story commercial buildings on the north portion of the site that will be retained as part of the Project. The existing ground surface elevation is approximately 194 feet (NAVD88) and varies by about 3 to 4 feet across the project site.

5.0 FIELD EXPLORATION AND LABORATORY TESTING

5.1 Boring Exploration

Three (3) borings (GP-1, GP-2, and GP-3) were completed by GeoPentech to investigate subsurface conditions at the project site. Laboratory tests were also performed on selected samples from the borings to evaluate the index and engineering properties of the encountered material. The results of the current field borings and laboratory tests are presented in Appendices A and B, respectively.

GP-1, GP-2, and GP-3 were advanced to depths of 131.5, 126.5, and 91 feet below ground surface, respectively, at the locations shown on Figure 2. The borings were drilled using 8-inch diameter hollow-stem auger drilling equipment, and drilling mud was added when groundwater was encountered. Standard Penetration Test (SPT) samples, modified California (MC) samples, bulk samples, and grab samples were collected during drilling. The work was performed under the supervision of a registered civil engineer who monitored the drilling operations and prepared a field record of soils observed and drilling conditions. The drilling was subcontracted to Martini Drilling, who provided all drilling equipment, crew, and supplies. Details of the current explorations and the logs of the borings are presented in Appendix A.

5.2 Laboratory Tests

Laboratory tests were performed on selected samples obtained from the borings to aid in the classification of the soils and to evaluate the pertinent engineering properties of the soils. The following tests were performed:

- Moisture content and dry density
- Passing No. 200 sieve (wash) and sieve analysis
- Atterberg Limits
- Corrosion suite
- Direct shear
- Consolidation

The geotechnical testing was conducted at the laboratory facilities of AP Engineering & Testing, Inc. in Pomona, California. The tests were performed in general accordance with applicable procedures of the American Society for Testing and Materials (ASTM). The in-place dry density and moisture content values, results of wash and sieve analyses as well as plasticity index values of the samples tested are presented in the boring logs, Appendix A. The complete results of laboratory tests along with the test results are presented in Appendix B.

5.3 Cone Penetration Testing (CPT)

Three (3) CPTs (CPT-1, CPT-2, and CPT-3) were performed by GeoPentech at the locations shown in Figure 2. CPT-1, CPT-2, and CPT-3 were advanced to refusal depths of about 70.9, 69.0, and 36.3 feet below existing ground surface, respectively.

The CPT work was subcontracted to ConeTec, who provided all CPT equipment, crew, and supplies. Details and results of the CPTs are presented in Appendix C.

5.4 Geophysical Surveys

Geophysical surveys were performed to measure shear-wave (S-wave) velocity profile for the project site. The geophysical investigation consisted of surface wave surveys using Multichannel Analysis of Surface Waves (MASW) and Refraction Microtremor (ReMi) methods. The geophysical measurements were performed along three survey lines (SW22-1 through SW22-3) on March 10, 2022. The locations of all survey lines are shown on Figure 2.

The geophysical data were collected and processed under the supervision of a California-licensed Professional Geophysicist. Details and results of the geophysical survey can be found in Appendix D.

6.0 GEOLOGIC AND SEISMIC CONDITIONS

6.1 Regional Geology and Seismicity

Regionally, the site is located in the northern end of the Peninsular Ranges physiographic province near the southern boundary of the Transverse Ranges physiographic province. Northwest trending mountains and faults characterize the Peninsular Ranges, while east-west trending mountains and faults characterize the Transverse Ranges. Figure 4a shows a geologic map of the site area, compiled by the California Geological Survey (CGS, 2012), and Figure 4b shows the map legend with the geologic unit descriptions. As indicated on Figure 4a, the site is within the northern edge of the Los Angeles Basin, about 2 ½ miles south of the Santa Monica Mountains range front. The site is located on old alluvial fan deposits (Qof) of late to middle Pleistocene-age. The underlying sediments are generally composed of clays, silts, sands, and gravels associated with fluvial and alluvial fan depositional environments.

The site is located within a seismically active region of southern California. Recent examples of the seismic activity in the region include the **M**6 1987 Whittier Narrows earthquake and the **M**6.7 1994 Northridge earthquake. Figure 5a shows the site location relative to mapped active faults in the region, as identified by the US Geological Survey (USGS, 2021). The site is not crossed by any known active faults with late Quaternary surface displacement. Significant faults near the site mapped with late Quaternary surface displacement include the Hollywood fault (located about 4½ km northwest), Newport-Inglewood fault (located about 4½ km to the southwest); the Santa Monica fault (located about 7 km to the west); and the Overland Avenue fault (located about 7 ½ km to the southwest). The San Andreas Fault is located approximately 58 km to the northeast.

Potentially active blind thrust faults are also believed to exist in the region, as shown on Figure 5b. These blind thrust faults are not expressed at the surface, but are inferred to exist based on indirect information, such as seismicity and folded stratigraphy. Recognition of the existence of blind thrust faults in the region was largely triggered by the occurrence of the 1987 Whittier Narrows earthquake. As shown on Figure 5b, the site is located on the hanging wall of the Compton blind thrust fault.

6.2 Site Geology and Subsurface Conditions

Based on the published geologic maps and the field investigation boring data, the geologic units underlying the site were interpreted to be Quaternary-age alluvial fan deposits (alluvium) and early Pliocene-age Fernando formation sedimentary bedrock. Figure 6a and 6b show geologic Cross Sections A-A' and B-B', and the location of the geologic cross section is shown on Figure 2. Descriptions of the geologic units are discussed below, and a summary of the geotechnical properties of the geologic units based on the laboratory test results performed during this investigation is presented in Table 1.

Artificial Fill

A thin layer of artificial fill soils (less than 1 ft) was encountered generally associated with the pavement section.

Quaternary Alluvium

Quaternary age alluvium was encountered in Borings GP-1, GP-2, and GP-3 to depths of about 90 to 95 feet below the ground surface. The alluvium generally consisted of stiff to very stiff clay (CH and CL) and medium dense to very dense sand (SC, SM, SP, SC-SM, SP-SC, SP-SM, and SW-SC). The upper portion of the alluvium from the ground surface to depths of about 20 feet predominantly consisted of clays and clayey sands, and from 20 feet to a depth of about 60 feet below the ground surface was dominantly consisted of clays and some clayey sands. The lower portion of the alluvium, from approximately 60 to 90-95 feet below the ground surface predominantly consists of sands with varying percentages of silt. Additionally, the alluvium generally increases in density/stiffness with greater depth.

Fernando Formation

The alluvium at the site is underlain by Fernando Formation bedrock to the total depth drilled to about 132 feet). The bedrock encountered consisted of hard interbedded claystone. We classify the upper portions of the bedrock as weathered to a depth about 107 ft, and consider the formational material below this depth as less weathered material due to very high blowcounts and drilling refusal.

6.3 Groundwater

Groundwater was observed during drilling initially at a depth of 60 feet below the ground surface in Boring GP-1 and rose to approximately a depth of 53.4 feet after about two hours. Groundwater was also initially encountered in Boring GP-2 at a depth of 65 feet below the ground surface and rose to about 36 feet after being allowed to reach equilibrium overnight (about 22 hours).

Based on a review of the Seismic Hazard Zone Report for the Hollywood Quadrangle (CGS, 1998), the historic high groundwater level beneath the site is estimated to be about 10 feet below the ground surface. Based on this information, we recommend a design ground water level of 10 feet bgs.

It should be recognized that groundwater levels can fluctuate over time, depending on seasonal rainfall and other influences (i.e., irrigation). Furthermore, there may be a potential for perched water to occur locally in sandy zones of the alluvial deposits above the static groundwater level. In addition, recent changes in policies for the use of stormwater infiltration could result in changing seepage conditions at shallow depths across the region.

6.4 Geotechnical Properties for Engineering Analysis

A summary of engineering properties for the geologic units present within the project sites are summarized in Table 2. These properties were developed for developing design recommendations based on the results of field and laboratory testing (see Table 1).

7.0 POTENTIAL GEOLOGIC AND SEISMIC HAZARDS

An evaluation of the potential geologic hazards is presented in the following sections.

7.1 Surface Fault Rupture

The site is not located within a currently established Alquist-Priolo (AP) Zone based on a review of the Earthquake Zones of Required Investigation for the Hollywood Quadrangle (CGS, 2018); however, the Project is located as close as about 11,000 feet east of the Earthquake Fault Zone for the Newport-Inglewood Fault. Additionally, the site is not located within 1,000 feet of a mapped Holocene-active fault based on a review of mapping by (USGS, 2018), as shown on Figure 5a. Therefore, the site is not considered susceptible to surface fault rupture hazards.

7.2 Seismic Shaking

The site is located within a seismically active region of southern California and should be designed in accordance with the seismic design requirements of governing codes and guidelines. We understand that the design for the Tower is being carried out in conformance with the 2022 CBC and ASCE 7-16 requirements using the performance-based design procedure specified by LATBDC, 2020. A site-specific hazard evaluation that included both Probabilistic Seismic Hazard Analysis (PSHA) and Deterministic Seismic Hazard Analysis (DSHA) has been carried out for the site. This analysis and its detailed results are presented in Appendix E of the report. To fulfill the seismic design requirements, the following site-specific response spectra are developed:

- A “Maximum Considered Event” uniform hazard spectrum with risk-targeted, maximum rotated ordinates at 5% damping; also known as a site-specific MCE_R response spectrum (corresponding to a 1% probability of collapse in a 50-year period; i.e., a modified 2,475-year return period spectrum).
- A “Service-Level Earthquake” uniform hazard spectrum with average horizontal spectral ordinates at 1.6% damping (corresponding to a 50% probability of exceedance in a 30-year period; i.e., a 43-year return period)

For completeness, the code-compliant, site-specific “Design Level” or DRS uniform hazard spectrum with risk-targeted, maximum-rotated ordinates at 5% damping has also been provided.

Based on the definitions per ASCE 7-16, Section 11.4.1, this site is classified as “near-fault” due to significant hazard contribution from sources located within 10 km for $M_W \geq 6$, or within 15 km for $M_W \geq 7$.

7.3 Liquefaction Potential

Liquefaction potential is greatest where the groundwater level is shallow and submerged loose to medium-dense sand occur within a depth of about 50 feet or less below the ground surface. Liquefaction potential generally decreases as fines and gravel content increase. As ground acceleration and shaking duration increase during an earthquake, liquefaction potential increases.

According to the CGS map of Earthquake Zones of Required Investigation for the Hollywood Quadrangle (CGS, 2018), and the County of Los Angeles Seismic Safety Element (1990), the site is not located within an area identified as having a potential for liquefaction. Furthermore, excavation for the basement levels will remove the upper approximately 70 to 80 feet of the alluvial soils at the site and the remaining soils below this depth are dense to very dense. As such, liquefaction is not considered to be a hazard at this site.

7.4 Seismically-Induced Settlement

Seismically-induced settlement may also be caused by unsaturated loose to medium-dense granular soils densifying during ground shaking. Uniform settlement beneath a given structure would cause minimal damage; however, because of variations in distribution, density, and confining conditions of the soils, seismically-induced settlement is generally non-uniform and can cause serious structural damage.

As part of the site development, the upper approximately 70 to 80 feet of the site will be excavated and the soils removed for the new basement level which will extend to below the groundwater, thereby removing all the unsaturated soils that are potentially susceptible seismically-induced settlement. Accordingly, seismically-induced settlement at the site for this project configuration is considered to be negligible.

7.5 Subsidence

Ground surface subsidence generally results from the extraction of fluids or gas from the subsurface that can result in the gradual lowering of the overlying ground surface. Subsidence can also occur when subsurface peat deposits oxidize and undergo volume loss. Although the site is located over the southern edge of the Salt Lake Oil Field, subsidence of the area above this oil field has not been reported. Additionally, the subsurface soils are not known to contain significant quantities of peat that would create a potential for subsidence. As there are no known ongoing extractions of oil or water that would lead to subsidence at the site, and the subsurface soils are not known to contain significant quantities of peat. Therefore, the potential for subsidence at the site is considered low.

7.6 Flooding

According to FEMA (2008), the site is not located within a defined floodplain or floodway boundary. The site has been assigned a FEMA Flood Zone X, which indicates "areas determined to be outside the 0.2% annual chance floodplain". As such, flooding is not considered a hazard at the site.

7.7 Seiches and Inundation (Water Storage Facilities)

This potential hazard is associated with seiches (water waves created when a body of water is shaken that have the potential to overtop a water storage facility) and inundation due to water storage facility failure. The site is located within the potential inundation area associated with Hollywood Reservoir according to the California Department of Water Resources (DWR). According to DWR, the level of potential inundation at the project site is indicated to be between about 8 and 17 feet. Hollywood Reservoir is regulated by the DWR Department of Safety of Dams (DSOD) which oversees design and construction of significant dams in California and conducts annual inspections. Therefore, the hazard of inundation due to dam failure affecting the project site is considered low.

7.8 Tsunami

A tsunami is a sea wave generated by a large submarine landslide or an earthquake-related ground deformation beneath the ocean. Historic tsunamis have been observed to produce a run-up on shore of several tens of feet in extreme cases. The site is located at an elevation of about 194 feet above mean sea level and is 9 miles from the shoreline. As such, the site is not considered susceptible to tsunami hazards.

7.9 Landslide and Lateral Spreading

A potential for landsliding and lateral spreading is often indicated in areas of moderate to steep terrain that are underlain by unfavorably oriented geologic discontinuities. The site is located on relatively level terrain and no landslides are mapped in the vicinity of the site (CGS, 1998). In addition, the site is not in a designated earthquake-induced landslide hazard or liquefaction hazard zones (CGS, 2018). Therefore, the potential for landsliding and lateral spreading is considered negligible.

7.10 Volcanic Eruption

Potential hazards from volcanic eruptions include both lava flows and ash falls from relatively nearby volcanoes. No active volcanic sources are present in the Los Angeles basin. Therefore, the potential for damage at the site due to volcanic eruption is negligible.

7.11 Erosion

The majority of the ground surface at the site is relatively level and is or will be covered with asphalt or concrete pavements. As such, erosion is not considered a hazard at the site.

7.12 Methane Gas

The site is located within the boundaries of a Methane Zone, as defined by the City of Los Angeles and subject to the City's methane code. We recommend that a methane specialist should perform a methane study to provide specific methane mitigation recommendations for the design and construction of the project.

7.13 Expansive and Collapsible Soils

Based on our observations during the field investigation, the soils encountered have predominantly low to no plasticity with isolated zones that had medium to high plasticity. The soils observed during our field investigation also did not show a reaction to hydrochloric acid (HCl) indicating the presence of cementation in the soil structure. Furthermore, all the soils within the footprint of the proposed building to a depth of about 70 to 80 ft will be removed. Given these considerations, expansive and collapsible soils are not considered a geologic hazard at the site.

8.0 GEOTECHNICAL RECOMMENDATIONS

Based on our understanding of the project and the results of our investigation, the proposed development is feasible from a geotechnical point of view. Key geotechnical considerations are discussed below:

Temporary Excavation: The construction of the below-grade levels of the building will require temporary excavation on the order of 70 to 80 ft, about 60 to 70 ft below the design groundwater level, for the tower and podium, respectively, and will require an excavation support system (i.e., shoring). Given the presence of adjacent existing buildings, the system should be designed to account for the loads from these buildings. Furthermore, the excavation will require dewatering and groundwater control measures to create a dry working area. However, to protect the adjacent buildings from the potential settlement due to changes in groundwater levels beyond the project site, the changes to the groundwater level outside of the project site should be limited. Given these constraints, the design of the excavation support system, dewatering, and groundwater control measures will be a key consideration for the project and is described in this section.

Foundation Systems: Due to relatively high building loads (average bearing pressures of 12,000 psf under the tower), controlling the settlement of the foundations under the proposed loads is a key geotechnical consideration. Based on the investigation results and our understanding of the structural loads, the proposed building is recommended to be supported on a continuous mat foundation to control settlements. Furthermore, foundation design and below-grade levels have to account for the presence of shallow groundwater. If needed, ground anchors should be used to resist hydrostatic uplift pressures.

Detailed recommendations for the project are provided in the following sections.

8.1 Seismic Design Parameters

In developing the preliminary seismic design parameters in accordance with the 2022 CBC and ASCE 7-16 Standard, a seismic site class C was selected based on a review of the shear-wave velocity data recently collected at the site (see Appendix D). S_S = 2.025g and S_1 = 0.721g are the mapped seismic values provided by USGS. Using ASCE 7-16, Section 21.4, the site-specific seismic design parameters for new structures at the project site are developed in Appendix E and are defined below. These parameters were developed in accordance with ASCE 7-16, Section 21.3.

S_{DS} = 1.532 g, based on 90% of the spectral acceleration at a period of 0.3-seconds
S_{D1} = 0.875 g, based on the spectral acceleration at a period of 1.0-second
S_{MS} = 2.298 g, based on 1.5 times S_{DS}
S_{M1} = 1.312 g, based on 1.5 times S_{D1}

Further details of the development of the seismic hazard analysis and the site-specific design response spectra, for the project are included in Appendix E.

8.2 Foundation Recommendations

Preliminary loading conditions provided to us by the Structural Engineer indicate an average bearing pressure of about 12,000 psf under the footprint of the Tower and about 3,000 psf under the footprint of the Podium. Considering uplift pressures with a design groundwater level of 10 ft bgs, net pressures beneath the tower and the Podium are expected to be approximately up to 7,600, with potential for hydrostatic uplift pressures under the Podium footprint. If the net pressures are calculated based on the deepest observed groundwater level during our field investigation (i.e., approximately 53 ft bgs), net pressures beneath the tower and podium are expected to be about 10,300 and 1,950 psf, respectively. In summary, net pressures beneath the Tower are expected to be between 7,600 to 10,300 psf with potential for hydrostatic uplift under the Podium under design groundwater level of 10 ft bgs.

Due to relatively high building loads under the Tower and potential for hydrostatic uplift, we recommend a continuous mat foundation under the footprint of both the Tower and Podium. As needed, tiedown ground anchors should be used to resist uplift pressures, in areas that the weight of the building is not adequate to resist uplift pressures. Considering four (4) below-grade parking levels and assuming a slab thickness of 14 ft at the core of the Tower, excavation of the upper 80 feet of soils in anticipated under the Tower. Under the Podium, considering a slab thickness of 4 feet, excavation of the upper 70 feet of soils is anticipated.

Mat Foundation

The proposed development is recommended to be supported on a mat foundation bearing on native alluvial deposits. A mat foundation founded on native alluvium at a depth of approximately 70 to 80 ft from the existing grade may be designed using a net allowable bearing capacity of up to 10,300 psf. This value is for dead plus live loads and may be increased by one-third to accommodate transient loads that include wind or seismic loads. Based on our evaluation (see Appendix F), we estimate the settlement of the proposed building on a mat foundation in the manner recommended could be up to 3 inches for net average mat bearing pressure of 10,300 psf. Differential settlement is estimated to be about half of the total settlement across the mat in either direction.

Settlement

For structural analyses of the mat foundation supported on undisturbed natural soils at the planned excavation level, a modulus of subgrade reaction, k, of 250 pounds per cubic inch (pci) may be used. This value is a unit value for use with a 1-foot-square area. The modulus should be reduced in accordance with the following equation when used with larger foundations:

$$K_R = K\left[\frac{B+1}{2B}\right]^2$$

Where:

K = unit subgrade modulus

K_R = reduced subgrade modulus

B = foundation width

We request that the final distribution of the pressures under the mat and estimated settlements be provided to us for review to confirm consistency with geotechnical recommendations.

Lateral loads may be resisted by soil friction and by the passive resistance of the soils. A coefficient of friction of 0.35 may be used between the mat foundation and the underlying native soils. The allowable passive resistance of undisturbed natural soils is recommended to be equal to the pressure developed by a fluid with a density of 300 pcf. The allowable passive resistance should be limited to a maximum value of 3,000 psf. The upper foot of the material should be ignored for calculating this value. A one-third increase in the passive value may be used for wind or seismic loads. The frictional resistance and passive resistance of the soils may be combined without reduction in evaluating the total lateral resistance.

The recommended bearing and lateral load design values are for use with loadings determined by a conventional working stress design. When considering an ultimate design approach, the recommended design values shall be multiplied by the following factors:

Design Item	Ultimate Design Factor
Bearing Value	3.0
Passive Pressure	2.0
Coefficient of Friction	2.0

8.3 Uplift and Waterproofing Considerations

As previously discussed, we recommend a design groundwater level of 10 feet below existing grades. For portions of the foundation extending more than 10 feet below existing ground surface, hydrostatic uplift pressure should be incorporated into the design. The uplift pressure can be calculated based on a fluid weight of 62.4 pounds per cubic foot (pcf) and can be resisted by self-weight of the building and/or tiedown ground anchors as needed.

Note that the foundations, basement walls, and interior slabs should be waterproofed to prevent seepage of water or moisture due to cracks or water migration. Waterproofing should extend at least 5 feet above the design groundwater level (i.e., to 5 feet below existing ground surface) and that a qualified waterproofing consultant should be retained for recommendations of suitable waterproofing applications behind all walls below grade, foundations, and slabs if necessary.

8.4 Tiedown Ground Anchors

Vertical tiedown ground anchors are recommended to resist hydrostatic uplift pressures, as needed. Tiedown anchors should be designed to conform to the requirements of Chapter 18 of the California Building Code. A contractor with demonstrated successful experience with the design and construction of permanent tiedown ground anchors with qualified personnel in similar conditions should be chosen to perform the design and construction of the anchors.

We recommend that the tiedown anchors be extended and grouted within the Fernando Formation bedrock, i.e., to or below elevation +100 ft msl, with a minimum unbonded length of 20 ft and a minimum bond length of 20 ft. Typical drill holes for tiedown anchors range from about 6 to 12 inches in diameter. Vertical ground anchors bonded in Fernando bedrock formation may be designed using an allowable transfer load of 0.75, 1, and 2 kips/ft for 6-8-, and 12-inch diameter anchors, respectively. These values are for gravity-grouted anchors and can be increased by 100% for pressure-grouted anchors. Ultimate anchor capacities can be estimated by applying a factor of safety of 3. Vertical tiedown anchors should be spaced at ten (10) times the diameter of the hole within the bonded length, or 5 ft minimum.

The design criteria given herein must be verified by a minimum of two sacrificial verification tests conducted at the site in advance of production installation. The verification tests should include extended creep tests. The allowable capacities could be adjusted based on the results of testing and structural considerations. Non-sacrificial verification tests should be completed for at least 10% of production anchors, and no less than two (2) anchors. Furthermore, all other anchors shall be proof tested. After testing, the anchors should lock off at loads specified by the structural engineer. All lock-off loads should be confirmed with lift-off tests. The test and acceptance criteria should follow recommendations presented by Post Tensioning Institute (PTI) guidelines publication titled “Recommendations for Prestressed Rock and Soil Anchors”.

Anchors should be protected against corrosion meeting minimum requirements for Class I protection in accordance with Post Tensioning Institute guidelines.

Other details involving the vertical tiedown anchors design such as its sufficient connection to the mat foundation and details of corrosion protection are commonly determined by the structural engineer and specialty anchor contractor, subject to the review and approval of the project structural and geotechnical engineers. Specialty contractors may be able to achieve higher capacities than indicated. The higher capacities should be verified by tests, and should be subject to review and approval of the geotechnical and structural engineers.

8.5 Walls Below Grade

<u>Lateral Earth Pressure</u>

Subterranean parking and basement walls should be designed to resist lateral earth pressures plus any surcharges from adjacent loads. Given the presence of shallow water level, it is anticipated that the basement walls will be designed without drainage and have to resist hydrostatic pressures based on the groundwater level at the ground surface. The walls without a drainage system have to be designed to resist hydrostatic pressures assuming groundwater at the ground surface. For submerged conditions (i.e., groundwater at the surface), retaining walls that are free to move and rotate at the top, such as cantilever walls, may be designed for an active pressure imposed by an equivalent fluid weighing 15 pcf. Permanent basement walls that are restrained at the top of the wall should be designed to resist an at-rest lateral earth pressure imposed by an equivalent fluid weighing 25 pcf. Hydrostatic pressures should be added to these values.

For walls with a drainage system to relieve hydrostatic pressure buildup behind the walls, hydrostatic pressure can be ignored in the wall design. Retaining walls with drainage that are free to move and rotate at the top, such as cantilever walls, may be designed for an active pressure imposed by an equivalent fluid weighing 35 pcf. Permanent basement walls that are restrained at the top of the wall should be designed to resist an at-rest lateral earth pressure imposed by an equivalent fluid weighing 50 pcf.

In addition to the recommended earth pressure, the upper 10 feet of walls below grade and retaining walls adjacent to areas subject to vehicular traffic should be designed to resist a uniform lateral pressure of 100 psf, acting as a result of an assumed 300 psf surcharge behind the walls due to normal vehicular traffic. If the traffic is kept back at least 10 feet from the top of walls, the traffic surcharge can be neglected. For the basement walls adjacent to the at-grade structures, surcharge pressures can be provided on a case-by-case basis once the estimated loading conditions from these structures and the details of the foundations are provided to us.

Loads from equipment surcharge imposed on adjacent ground may be computed using a coefficient of 0.4 times the uniform load applied.

In addition to the above-mentioned lateral earth pressures, the walls below grade should be designed to support a seismic lateral pressure of 22H (psf) applied uniformly along the wall height H (in feet). This seismic load is a directly calculated value and can be used as is. When designing for seismic loads, the seismic lateral earth pressure should be combined with the active earth pressure mentioned previously. If designing for static loading condition only, the at-rest lateral earth pressure should be used.

Drainage

Given the shallow groundwater, we anticipate that the building walls below grade will be designed to resist hydrostatic pressures. As such, building walls below grade and retaining walls should be designed to resist hydrostatic pressures (equivalent fluid pressure of 62.4 pcf).

For other walls that may require a drainage system, a drainage system be provided by either a 1-ft wide zone of crushed rock protected by filter fabric, or a 4-foot wide strips of Miradrain 6000 (or equivalent) placed at 8 to 10 feet on center. The crushed rock zone or Miradrain (or equivalent) strips may be placed at a depth starting at about 3 feet below the grade and should be connected to a perforated discharge pipe at the base of the wall. The drain pipe should consist of a minimum 4-inch-diameter perforated pipe placed with perforations down along the base of the wall.

The pipe should be sloped at least 2 inches in 100 feet and surrounded by filter gravel separated from the on-site soils by an appropriate filter fabric. The filter gravel should meet the requirements of Class 2 Permeable Material as defined in the current State of California, Department of Transportation, Standard Specifications. If Class 2 Permeable Material is not available, ¾ inch crushed rock or gravel separated from the on-site soils by an appropriate filter fabric should be used. The crushed rock or gravel should have less than 5% passing a No. 200 sieve.

The installed drainage system should be observed by personnel from our firm prior to being backfilled. Inspection of the drainage system may also be required by the reviewing governmental agencies.

Waterproofing

We recommend that all retaining walls and walls below grade be waterproofed. See Section 8.3 (Uplift and Waterproofing Considerations) for further detail.

8.6 Sulfate Attack and Corrosion Potential of Soils

One (1) sample from the field investigation was tested for minimum resistivity, sulfates, chlorides, and pH during the current investigation (results of the current testing are presented in Appendix B). The corrosion tests from the current investigation were performed in accordance with guidelines of Caltrans Test 417, 422, and 643. Based on the results of these tests, the tested soil is not considered corrosive for structures based on guidelines from California Department of Transportation (2021). However, based on the results of the resistivity test and Caltrans guidelines, there is potential for presence of high quantities of soluble salts and higher propensity for corrosion.

We recommend that a corrosion consultant or project civil engineer review results of corrosion tests and provide detailed recommendations for underground metallic pipes and below-grade structures if needed.

8.7 Excavations and Temporary Shoring

General

Earthwork operations at the site will include removals of undocumented fill soils and rubble, excavations for the subterranean parking level, excavations for foundations, and trenching for utility lines.

To provide support for the foundations, any exterior pavements, and exterior concrete walks, all existing undocumented fill soils and upper loose/soft natural soils should be excavated and replaced as engineered fill if required. Based on the understanding that the upper 70-80 feet of the site will be excavated for the proposed basement level, we expect that all existing fill soils will likely be removed from the site.

Temporary excavations up to a height of 4 feet can be cut vertically. Unshored excavations should not extend below a plane drawn at 1½:1 extending downward from adjacent existing footings.

Where space is available, excavations can be made with slopes of 1:1 (horizontal to vertical). Where space is unavailable, shoring is recommended for the proposed excavations adjacent to existing streets and/or buildings.

Where sloped embankments are used, the tops of the slopes should be barricaded to prevent vehicles and storage loads within 5 feet of the tops of the slopes. A greater setback may be necessary when considering heavy vehicles, such as concrete trucks and cranes; we should be advised of such heavy vehicle loadings or heavy construction equipment, stockpile material etc. so that specific setback requirements can be established. If the temporary construction embankments are to be maintained during the rainy season, berms are suggested along the tops of the slopes where necessary to prevent runoff water from entering the excavation and eroding the slope faces. The soils exposed in the cut slopes should be inspected during excavation by our personnel so that modifications of the slopes can be made if variations in the soil conditions occur.

We recommend that a qualified geotechnical firm observe the excavations and shoring installation, so that necessary modifications based on variations in the soil conditions can be made. Applicable safety requirements and regulations, including OSHA regulations, should be met.

Temporary Shoring System

Cantilever (for excavation below 20 ft) or braced or tied-back shoring system (for deeper excavation) can be used to support the sides of the proposed excavations. Given the shallow groundwater level, groundwater dewatering and control measures will be required. Furthermore, there is a potential of settlement of existing buildings adjacent to the project site, if the groundwater level outside of the site is changed due to dewatering within the site. As such, excavation support systems that may cause a significant change in groundwater level outside of the project site is not considered not be feasible. A sealed shoring system with adequate embedment to mitigate changes in groundwater level outside of the excavation should be considered. The shoring piles should be extended deep enough to resist lateral loads as well as the potential for instability of the base of excavation (heave and sand boils).

The design of the shoring and dewatering system should be coordinated and performed by a qualified engineer familiar with deep excavation below the water table.

Temporary Shoring Lateral Pressures

For the design of the shoring system, we recommend the following lateral earth pressures for drained and submerged conditions, respectively. As indicated above, the excavation for the proposed tower and podium should be supported by a sealed system, and should be designed to resist hydrostatic pressures (i.e. submerged conditions). For cantilever piles we recommend using the triangular lateral earth pressure with a maximum pressure equal to 40H and 24H psf, for drained and submerged conditions, respectively. For the design of braced or tied-back shoring, we recommend using a trapezoidal pressure distribution with a maximum lateral earth pressure equal to 24H and 15H psf, for drained and submerged conditions, respectively, where H is the retained height in feet. For submerged conditions, hydrostatic pressures due to groundwater should also be added to the earth pressures indicated. We recommend a groundwater level of 10 ft below surface for temporary shoring design. These recommendations are shown on Figures 7 and 8. All of these pressures are for level ground behind the wall (i.e., no backslope).

In addition to the recommended earth pressure, the upper 10 feet of the shoring adjacent to traffic area should be designed to resist a uniform lateral pressure of 100 psf, acting as a result of an assumed 300 psf surcharge behind the shoring due to normal street traffic. If the traffic is kept back at least 10 feet from the face of the shoring, the traffic surcharge may be omitted. In addition, any surcharge (live or dead load) located within a 1:1 (horizontal to vertical) plane drawn upward from the base of the shored excavation should be added to the lateral earth pressures. The details of the adjacent structures (elevation of foundation, loads, configuration, etc.) should be provided to us to estimate the pressure on the shoring walls due to surcharge, if applicable.

Tie-Back Anchor Design

Tieback friction anchors may be used to resist lateral loads. For design purposes, it may be assumed that the active wedge adjacent to the shoring is defined by a plane drawn at 35 degrees from the vertical through the bottom of the excavation. These anchors should extend to a minimum of 15 feet beyond the potential active wedge and to a greater length if necessary to develop the desired capacities.

For design purposes, it may be estimated that drilled and grouted friction anchors would develop a soil friction of 750 psf along the anchors in the bonded zone. This value is provided for gravity grouted anchors. For pressure grouted anchors, a soil friction of 2,500 psf may be used along the anchors in the bonded zone. The capacities of the anchors should be determined by testing of the initial anchors as outlined below under the Tie-back Anchor Testing section.

Only the frictional resistance developed beyond the active wedge would be effective in resisting lateral loads. If the anchors are spaced at least 6-feet on center, then no reduction in capacity is necessary. Closer spacing would require evaluation of an appropriate reduction factor.

Tie-Back Anchor Installation

The anchors may be installed at angles of 15 to 40 degrees below the horizontal. The anchors should be filled with concrete, placed by pumping from the tip out. The concrete should extend from the tip of the anchor to the active wedge. To minimize caving, we suggest that the portion of the anchor shaft within the active wedge be backfilled with sand. A small amount of cement may be used to allow the sand to be placed by pumping. The sand-cement mixture should fill the portion of the tieback anchor tightly and should be flush with the face of the shoring when finished.

Tie-Back Anchor Testing

The installation of the anchors and the testing of the completed anchors should be observed by a representative of a qualified geotechnical firm. The geotechnical engineer or his representative should select at least four of the initial anchors for 24-hour 200% tests and six additional anchors for "quick" 200% tests to verify in the field the friction value assumed in this report. Also, we recommend that the 200% tests be performed at representative locations around the site and not concentrated in a single area.

The total deflection during the 24-hour 200% tests should not exceed 12 inches during loading; the anchor deflection should not exceed ¾ inch during the 24-hour period, measured after the 200% test load is applied. If the anchor movement after the 200% load has been applied for 12 hours is less than ½ inch, and the movement over the previous 4 hours has been less than 0.1 inch, the test may be terminated.

For the quick 200% tests, the test load should be maintained for 30 minutes. The total deflection of the anchor during the 200% quick test should not exceed 12 inches; the deflection after the 200% test load has been applied should not exceed ¼ inch during the 30-minute period.

All of the production anchors should be pre-tested to at least 150% of the design load; the total deflection during the test should not exceed 12 inches. The rate of creep under the 150% test should not exceed 0.1 inch over a 15-minute period in order for the anchor to be approved for the design loading.

After a satisfactory test, each production anchor should be locked off at the design load. The locked-off load should be verified by rechecking the load on the anchor. If the locked-off load varies by more than 10% from the design load, the load should be reset until the anchor is locked off within 10% of the design load.

The installation of the anchors and the testing of the completed anchors should be observed by a qualified geotechnical firm.

Raker Bracing

Raker bracing, if used, should be supported by temporary concrete footings (deadmen). For design of such temporary footings, poured with the bearing surface normal to the rakers inclined at 45 to 60 degrees with the vertical, a bearing value of 2,000 pounds per square foot may be used, provided the shallowest point of the footing is at least 2 foot below the lowest adjacent grade and is founded in the native alluvium. To reduce the deflection of the shoring, the rakers should be preloaded to the design load.

Deflection

Predicting actual deflections of a shored embankment is difficult. It should, however, be realized that some deflection would occur. We estimate that deflections could be about 1 inch at the top of the shored embankment. If greater deflection occurs during construction, additional bracing may be necessary to prevent settlement and loss of support from beneath and adjacent to the shored excavation.

Monitoring

Monitoring of the performance of the shoring system is recommended. The monitoring should consist of periodic surveying of the lateral and vertical locations of the tops of all soldier piles. Initial survey should be taken prior to the first level of excavation so that an accurate baseline may be established.

We recommend that the initial survey and monitoring program also include any adjacent existing structures. Photographs and videos of the existing structures are recommended as part of the documentation process.

Monitoring considerations should be discussed further with the design consultants and the contractor when the design of the shoring system has been finalized.

8.8 Earthwork

General

Earthwork should be performed in accordance with the applicable sections of the grading code for the City of Los Angeles and the State of California, as well as the recommendations in this report.

Subgrade Preparation and Moisture Conditioning

Areas excavated to receive fill should be cleared and stripped of all debris, deleterious matter, organics and vegetation, and remnants resulting from demolition of existing foundations. Cleared and grubbed material should be disposed of offsite.

After clearing the site of existing debris, the exposed subgrade should be observed for debris, organic material, or other undesirable materials. The exposed subgrade should then be proof-rolled so as to allow placement of any required fill. Compacted fill should be placed immediately upon approval of the prepared subgrade by the geotechnical engineer of record.

Mat/Foundation Excavations

The exposed excavated surface should be observed by the geotechnical engineer to confirm that satisfactory subgrade soils have been encountered. If loose, soft or clayey native soils, or undocumented fill soils are encountered at the bottom of excavation, additional removals may be required. The bottom of the excavations should be proof-rolled so as to allow placement of any required fill at 95% relative compaction in accordance with ASTM D1557, or the placement of concrete or concrete slurry mix as backfill. Compacted fill should be placed immediately upon approval of the prepared subgrade by the geotechnical engineer of record.

Where foundation excavations are deeper than about 4 feet, the sides of the excavations should be sloped back at ¾:1 (horizontal to vertical) or shored for safety. Unshored excavations should not extend below a plane drawn at 1½:1 (horizontal to vertical) extending downward from adjacent existing foundations.

Fill Materials and Placement of Fill

The on-site excavated granular materials such as sands and silty sands can be used as engineered fill. However, the on-site clayey soils are anticipated to be moderately expansive and should not be used within 3 feet of the lightly-loaded foundation, slabs or pavements. The existing fill materials, once debris and vegetation are removed, may be re-used as compacted fill. Oversized material (greater than 6 inches in longest dimension) should be removed from excavated material prior to reuse as engineered fill.

Imported fill material should be granular, non-corrosive, free of organic matter or other deleterious material. The Expansion Index of the fill material should be less than 35 and fill material should have a fines content (passing #200 sieve) less than 40 percent. Oversize material (larger than 6 inches in diameter) should not be used in the fill. All imported fill material should be approved by the geotechnical engineer prior to placement. A sample of proposed fill material(s) should be submitted to the geotechnical engineer for testing at least three business days prior to use at the site.

Fill material should be compacted to at least 95% of the maximum dry density obtainable by the ASTM Designation D1557 method of compaction.

Backfill

All required backfill should be mechanically compacted in layers; flooding should not be permitted. Proper compaction of backfill will be necessary to reduce settlement of the backfill and to reduce settlement of overlying elements such as slabs and paving. Backfill should be compacted to at least 95% of the maximum dry density obtainable by the ASTM Designation D1557 method of compaction. The on-site soils excluding clayey soils may be used in the compacted backfill.

Some settlement of the backfill should be expected, and any utilities supported therein should be designed to accept differential settlement, particularly at the points of entry to the building. Also, provisions should be made for some settlement of concrete walks supported on backfill.

The exterior grades should be sloped to drain away from the foundation to prevent ponding of water.

Compaction

The preparation of the subgrade, excavations for the mat foundation and reworking of on-site soils and compaction of any required fills or backfill should be observed and tested by a representative of a qualified geotechnical firm.

The bottom of the excavations should be proof-rolled so as to allow placement of any required fill at 95% relative compaction in accordance with ASTM D1557. Compacted fill should be placed immediately upon approval of the prepared subgrade by the geotechnical engineer of record.

Any required fill below the foundations should be compacted to a minimum of 95 percent maximum dry density as determined in accordance with ASTM D 1557. The field density of fill should be determined in accordance with the Sand Cone Method (ASTM D1556) or the Nuclear Method (ASTM D2922 and D3017).

Fill material should be placed in loose lifts generally no greater than 8 inches thick. The moisture content of the on-site sandy soils at the time of compaction should vary no more than 2% below or above optimum moisture content. The moisture content of the on-site clayey soils at the time of compaction should be between 2% and 4% above optimum moisture content.

8.9 Geotechnical Observation

We recommend that a qualified geotechnical engineer or his representative observe the condition of the final subgrade soils immediately prior to foundation construction, and if necessary, perform further density and moisture content tests to determine the suitability of the final prepared subgrade. This representative should perform at least the following duties:

- Observe the clearing and grubbing operations for proper removal of all unsuitable materials.
- Observe the installation of excavation support system and groundwater control measures.
- Observe the exposed subgrade in areas to receive fill and in areas where excavation has resulted in the desired finished subgrade. The representative should also observe proof-rolling and delineation of areas requiring over-excavation.
- Evaluate the suitability of on-site and import soils for fill placement; collect and submit soil samples for required or recommended laboratory testing where necessary.
- Observe the fill and backfill for uniformity during placement.
- Test backfill for field density and compaction to determine the percentage of compaction achieved during backfill placement.
- Observe and probe foundation materials to confirm that suitable bearing materials are present at the design foundation depths.
- Observation of the installation of tiedown anchors, including performance and proof testing. A log should be maintained detailing the depth and grout pressure and volume of each anchor installation.

The governmental agencies having jurisdiction over the project should be notified prior to commencement of grading so that the necessary grading permits can be obtained, and arrangements can be made for required inspection(s). The contractor should be familiar with the inspection requirements of the reviewing agencies.

9.0 GENERAL CONDITIONS

In view of the general geology of the project area, the possibility of different subsurface conditions cannot be discounted. Conclusions and recommendations presented in this report are based upon GeoPentech's understanding of the project and the assumption that the subsurface conditions do not deviate appreciably from those disclosed by the field explorations performed. In the event that the locations, configurations, layout, or features of the proposed tower and associated podium are changed, the recommendations presented in this report may not be applicable. It is the responsibility of the Owner to bring any such changes of the proposed structures and any deviations of the subsurface conditions to the attention of GeoPentech. In this way, supplemental recommendations, if required, can be made without delay to the project.

Professional judgments presented in this report are based on an evaluation of the technical information gathered and GeoPentech's general experience in the field of geotechnical engineering. GeoPentech does not guarantee the performance of the project in any respect, only that the engineering work and judgment rendered meet the standard of care of the geotechnical profession at this time.

10.0 REFERENCES

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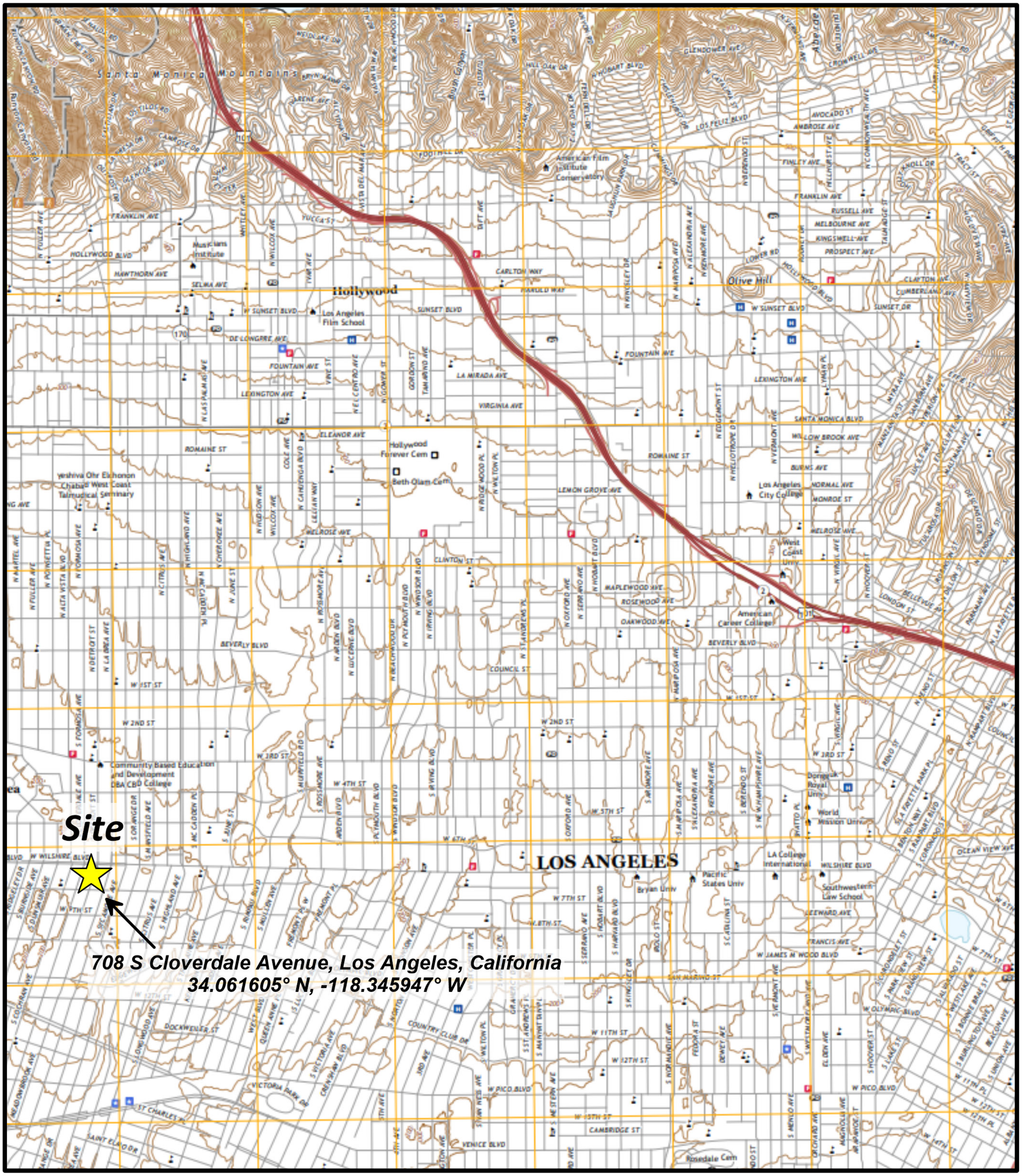
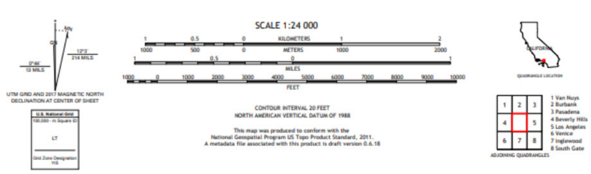
Table 1 – Summary of Geologic Unit Properties

Geologic Unit	*Material Description*	*Approx. Depth Range (ft)*	*Key*	*Moisture Content (%)*	*Unit Weight (pcf)*	*Particle Size Distribution (%)*			*Atterberg Limits*		*Shear Strength*		*Consolidation*		
						Gravel	*Sand*	*Fines*	*PI*	*LL*	*Friction Angle (deg)*	*Cohesion (psf)*	*OCR*	C_{ce}	C_{re}
Artificial Fill	Asphalt and Base	<1	-	-	-	-	-	-	-	-	-	-	-	-	-
Quaternary Alluvium	Sands (SP-SC, SC, and SP-SM) and Clays (CL)	0 to 20	Range Median (# Tests)	6 to 22 13 (5)	116 to 134 125 (4)	-	45 45 (1)	9 to 73 38 (5)	6 to 29 28 (4)	23 to 41 41 (4)	36 36 (1)	250 250 (1)	-	-	-
	Low to High Plasticity Clays (CL and CH)	20 to 60	Range Median (# Tests)	16 to 33 22 (11)	119 to 132 121 (9)	-	-	35 to 81 62 (10)	16 to 50 38 (13)	38 to 71 52 (13)	23 to 31 31 (3)	700 to 1200 750 (3)	1.0 to 2.3 1.4 (5)	0.070 to 0.155 0.094 (5)	0.009 to 0.033 0.017 (8)
	Sands (SM and SP-SM)	60 to 90	Range Median (# Tests)	12 to 25 19 (8)	119 to 137 130 (8)	-	92 92 (1)	8 to 15 11 (8)	-	-	36 to 37 37 (2)	200 to 350 275 (2)	-	-	-
Fernando Formation	More weathered	90 to 107	Range Median (# Tests)	30 to 46 38 (3)	113 to 118 115 (2)	-	-	70 to 99 99 (3)	24 to 27 25 (3)	46 to 62 53 (3)	29 29 (1)	1450 1450 (1)	-	-	0.011 to 0.022 0.021 (4)
	Less weathered	>107	Range Median (# Tests)	30 to 35 33 (2)	120 120 (2)	-	-	90 90 (1)	26 to 27 27 (2)	50 to 51 51 (2)	-	-	-	-	-

Table 2 – Summary of Engineering Properties for Design

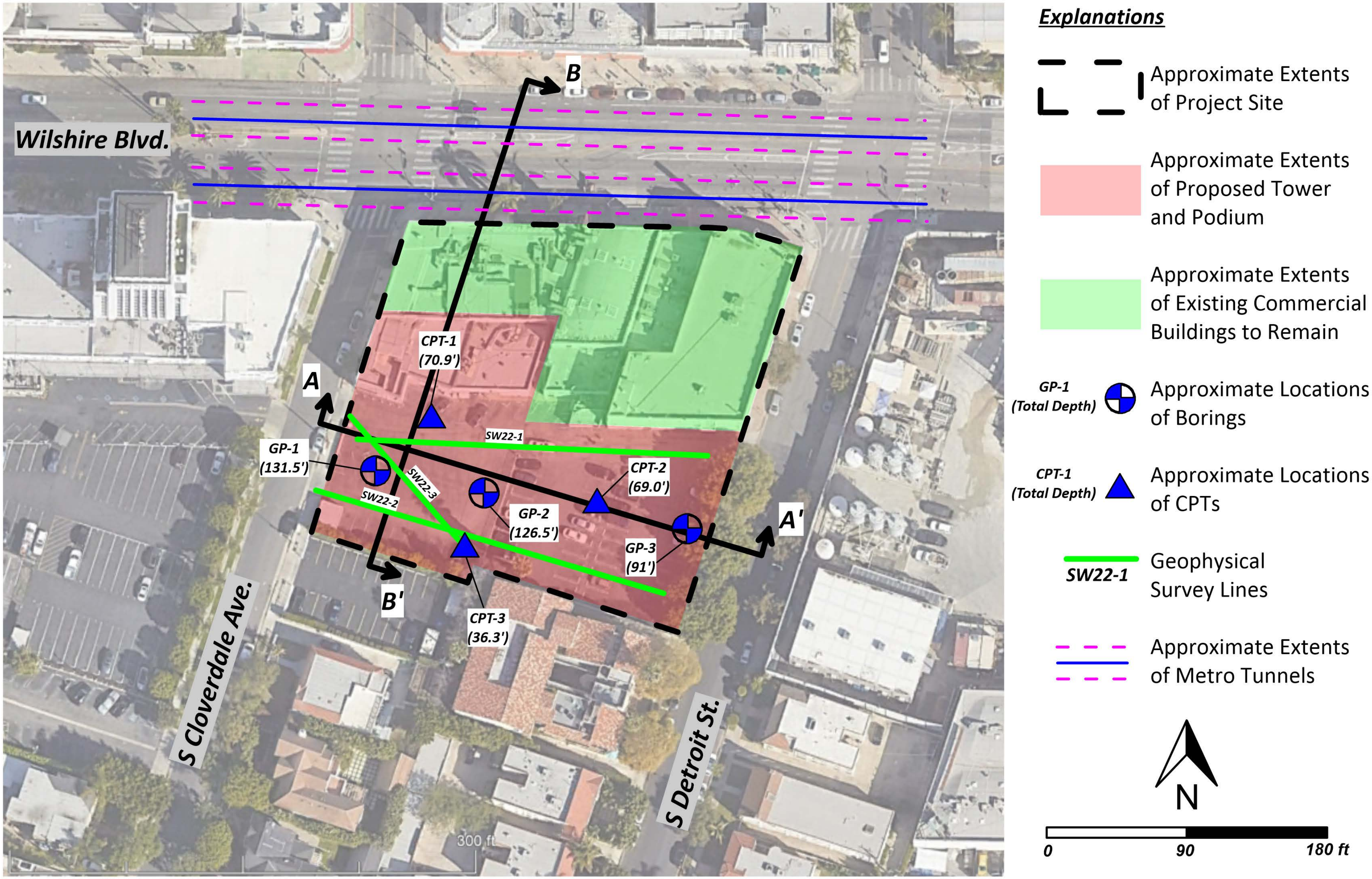
Geologic Unit	*Material Description*	*Approx. Depth Range (ft)*	*Unit Weight (pcf)*	*Consolidation*		*Drained Shear Strength*		*Undrained Shear Strength Ratio* S_u/σ'_v
				C_{ce}	C_{re}	*Friction Angle (deg)*	*Cohesion (psf)*	
Artificial Fill	Asphalt and Base	<1 ft	130	-	-	30	-	-
Quaternary Alluvium	Sands (SP-SC, SC, and SP-SM) and Clays (CL)	1 to 20	125	-	-	36	250	-
	Low to High Plasticity Clays (CL and CH)	20 to 60	121	0.102	0.019	31	750	0.29
	Sands (SM and SP-SM)	60 to 90	130	-	-	37	275	-
Fernando Formation	More weathered	90 to 107	115	-	0.011	-	-	Very Weak to Weak (ISRM, 1978)
	Less weathered	>107	120	-	-	-	-	-





SITE LOCATION MAP			
Project No.: 22010A	Project: 708 S Cloverdale Ave. Development	Date: APR 2022	Figure 1

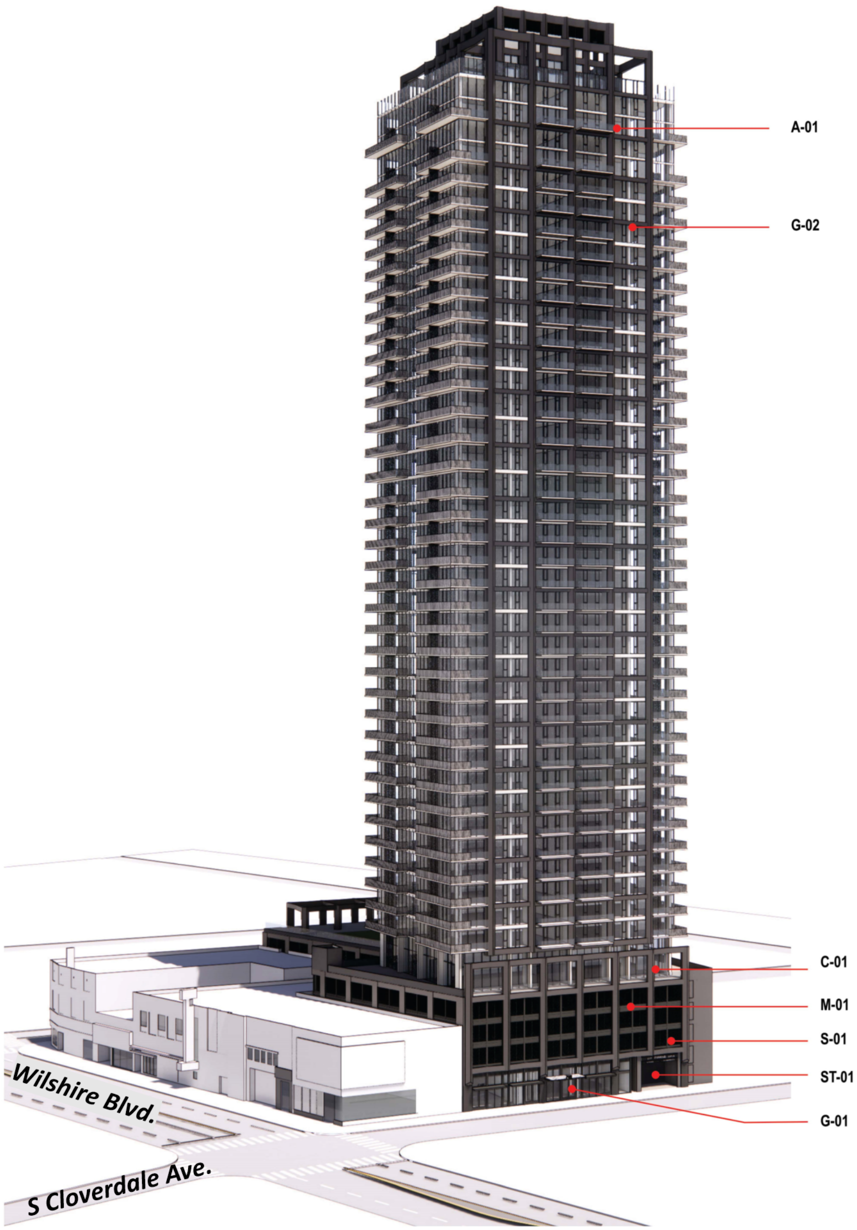
GeoPentech

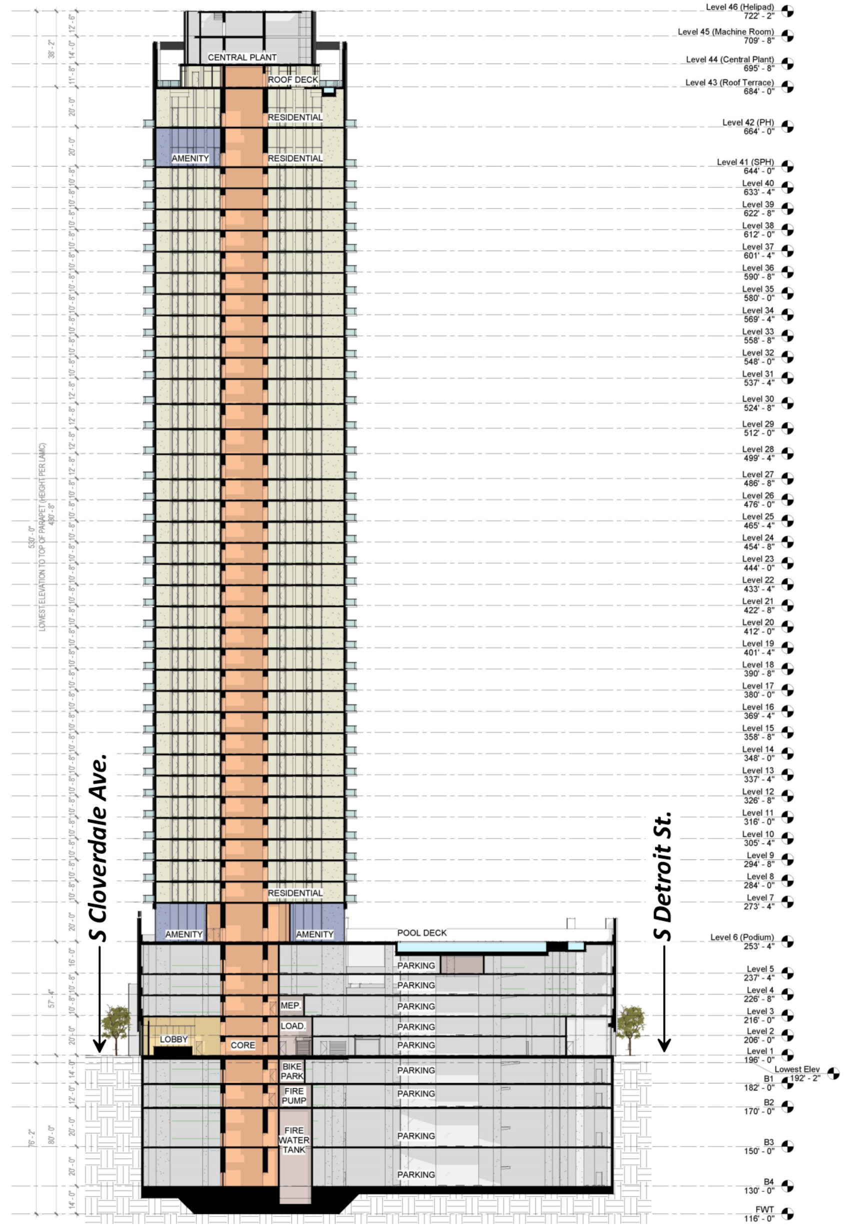


Site Plan

Date: APR 2022	Project No.: 22010A	Project: 708 S Cloverdale Ave. Development	Figure 2

GeoPentech





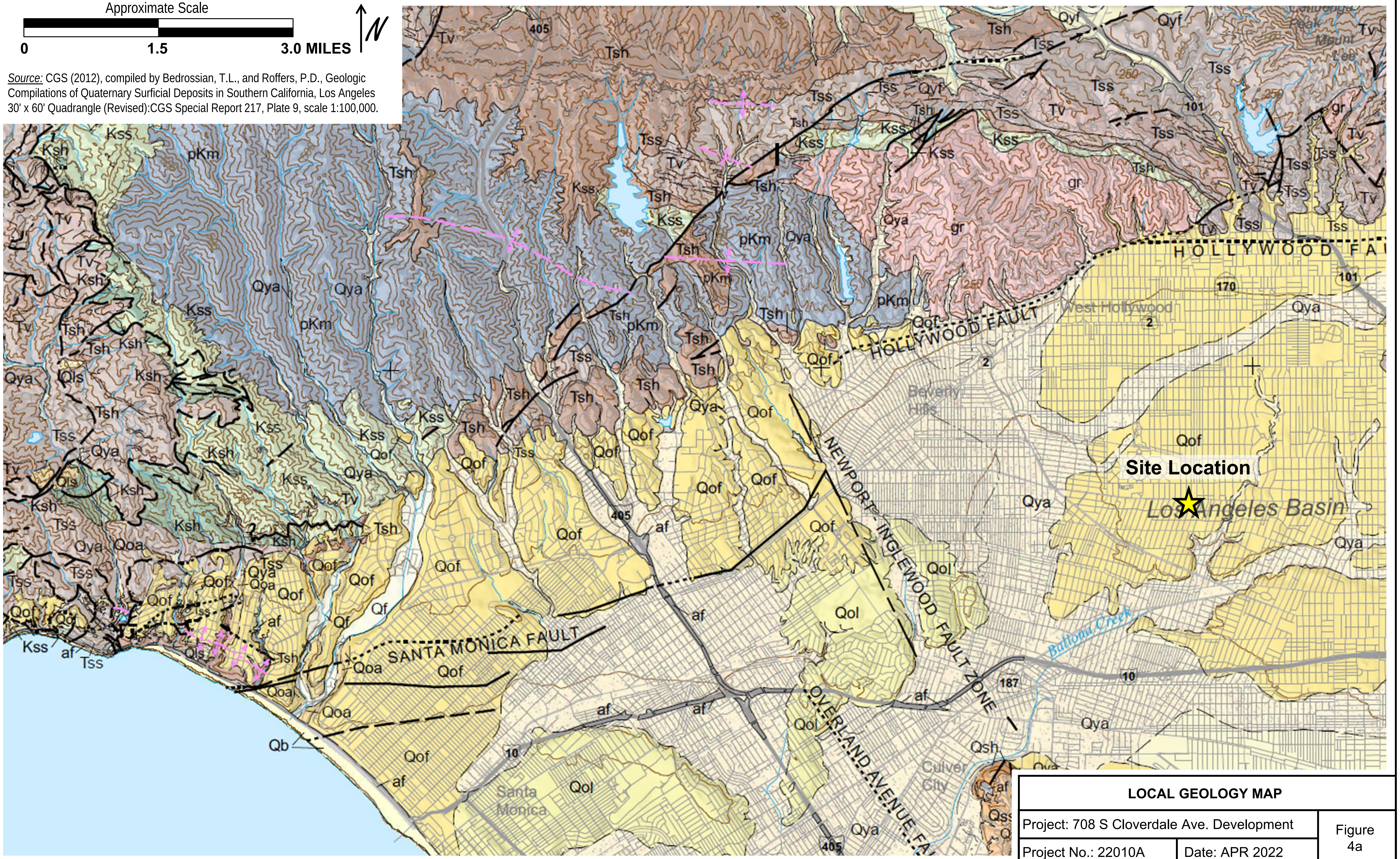
SECTION DIAGRAM 1 1/32" = 1'-0"

Source: *MVE+ Partners Architectural Package (dated 09/22/2022)*

Architectural Views

Date: OCT 2022	Project No.: 22010A	Project: 708 S Cloverdale Ave. Development	Figure 3

GeoPentech



Source: CGS (2012), compiled by Bedrossian, T.L., and Roffers, P.D., Geologic Compilations of Quaternary Surficial Deposits in Southern California, Los Angeles 30' x 60' Quadrangle (Revised):CGS Special Report 217, Plate 9, scale 1:100,000.

LOCAL GEOLOGY MAP		
Project: 708 S Cloverdale Ave. Development		Figure 4a
Project No.: 22010A	Date: APR 2022	

GeoPentech

Source: CGS (2012), compiled by Bedrossian, T.L., and Roffers, P.D., Geologic Compilations of Quaternary Surficial Deposits in Southern California, Los Angeles 30' x 60' Quadrangle (Revised):CGS Special Report 217, Plate 9, scale 1:100,000.

MAP UNITS

Late Holocene (Surficial Deposits)

af	**Artificial Fill** - deposits of fill resulting from human construction, mining, or quarrying activities; includes engineered fill for buildings, roads, dams, airport runways, harbor facilities, and waste landfills
Qsu	**Undifferentiated Surficial Deposits** - includes colluvium, slope wash, talus deposits, and other surface deposits of all ages; generally unconsolidated but locally may contain consolidated layers
Qls	**Landslide Deposits** - may include debris flows and older landslides of various earth material and movement types; unconsolidated to moderately well-consolidated
Qb	**Beach Deposits** - unconsolidated marine beach sediments consisting mostly of fine- and medium-grained, well-sorted sand
Qw	**Alluvial Wash Deposits** - unconsolidated sandy and gravelly sediment deposited in recently active channels of streams and rivers; may contain loose to moderately loose sand and silty sand
Qf	**Alluvial Fan Deposits** - unconsolidated boulders, cobbles, gravel, sand, and silt recently deposited where a river or stream issues from a confined valley or canyon; sediment typically deposited in a fan-shaped cone; gravelly sediment generally more dominant than sandy sediment
Qa	**Alluvial Valley Deposits** - unconsolidated clay, silt, sand, and gravel recently deposited parallel to localized stream valleys and/or spread more regionally onto alluvial flats of larger river valleys; sandy sediment generally more dominant than gravelly sediment
Qt	**Terrace Deposits** - includes marine and stream terrace deposits; marine deposits include slightly to moderately consolidated and bedded gravel and conglomerate, sand and sandstone, and silt and siltstone; river terrace deposits consist of unconsolidated thin- to thick-bedded gravel
Ql	**Lacustrine, Playa, and Estuarine (Paralic) Deposits** - mostly unconsolidated fine-grained sand, silt, mud, and clay from fresh water (lacustrine) lakes, saline (playa) dry lakes that are periodically flooded, and estuaries; deposits may contain salt and other evaporites
Qe	**Eolian and Dune Deposits** - unconsolidated, generally well-sorted wind-blown sand; may occur as dune forms or sheet sand

Holocene to Late Pleistocene (Surficial Deposits)

Qyf	**Young Alluvial Fan Deposits** - unconsolidated to slightly consolidated, undissected to slightly dissected boulder, cobble, gravel, sand, and silt deposits issued from a confined valley or canyon
Qya	**Young Alluvial Valley Deposits** - unconsolidated to slightly consolidated, undissected to slightly dissected clay, silt, sand, and gravel along stream valleys and alluvial flats of larger rivers

Late to Middle Pleistocene (Surficial Deposits)

Qof	**Old Alluvial Fan Deposits** - slightly to moderately consolidated, moderately dissected boulder, cobble, gravel, sand, and silt deposits issued from a confined valley or canyon
Qoa	**Old Alluvial Valley Deposits** - slightly to moderately consolidated, moderately dissected clay, silt, sand, and gravel along stream valleys and alluvial flats of larger rivers
Qot	**Old Terrace Deposits** - slightly to moderately consolidated, moderately dissected marine and stream terrace deposits
Qol	**Old Lacustrine, Playa, and Estuarine (Paralic) Deposits** - slightly to moderately consolidated, moderately dissected fine-grained sand, silt, mud, and clay from lake, playa, and estuarine deposits of various types

Middle to Early Pleistocene (Surficial Deposits)

Qvof	**Very Old Alluvial Fan Deposits** - moderately to well-consolidated, highly dissected boulder, cobble, gravel, sand, and silt deposits issued from a confined valley or canyon
Qvoa	**Very Old Alluvial Valley Deposits** - moderately to well-consolidated, highly dissected clay, silt, sand, and gravel along stream valleys and alluvial flats of larger rivers; generally uplifted and deformed

Quaternary (Bedrock)

Qss	**Coarse-grained formations of Pleistocene age and younger** - primarily sandstone and conglomerate
Qsh	**Fine-grained formations of Pleistocene age and younger** - includes fine-grained sandstone, siltstone, mudstone, shale, siliceous and calcareous sediments

Tertiary (Bedrock)

Tss	**Coarse-grained Tertiary age formations** - primarily sandstone and conglomerate
Tsh	**Fine-grained Tertiary age formations** - includes fine-grained sandstone, siltstone, mudstone, shale, siliceous and calcareous sediments
Tv	**Tertiary age formations of volcanic origin**

Mesozoic and Older (Bedrock)

Kss	**Coarse-grained Cretaceous age formations of sedimentary origin**
Ksh	**Fine-grained Cretaceous age formations of sedimentary origin**
pKm	**Cretaceous and pre-Cretaceous metamorphic formations of sedimentary and volcanic origin**
sp	**Serpentinite of all ages**
gr	**Granitic and other intrusive crystalline rocks of all ages**

SYMBOL EXPLANATION

[For geologic line symbols: lines are solid where location is accurate, long-dashed where location is approximate, short-dashed where location is inferred, dotted where location is concealed. Queries added where identity or existence may be questionable.]

Contacts

- Contact
- Gradational contact
- Reference contact -- Used to delineate geologic units that were mapped as separate units on the original source map, but are consolidated on this map.

Fault -- Includes strike-slip, normal, reverse, oblique, and unspecified slip

Lineament

Folds -- Showing direction of plunge where appropriate

- Anticline
- Overturned anticline
- Syncline

Dike

Stream

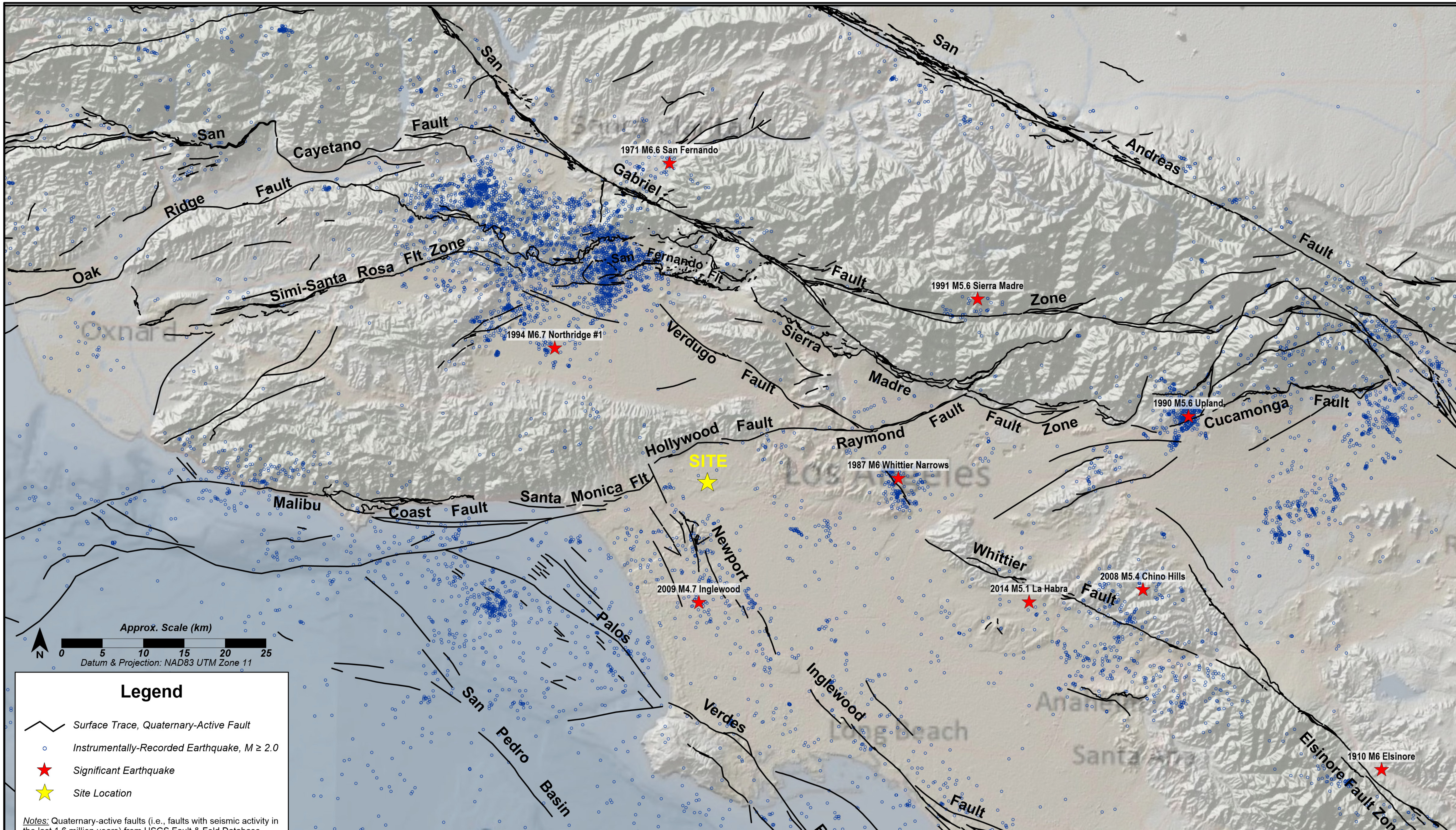
Spring

Road

County boundary

LOCAL GEOLOGY MAP LEGEND		
Project: 708 S Cloverdale Ave. Development		Figure 4b
Project No.: 22010A	Date: APR 2022	

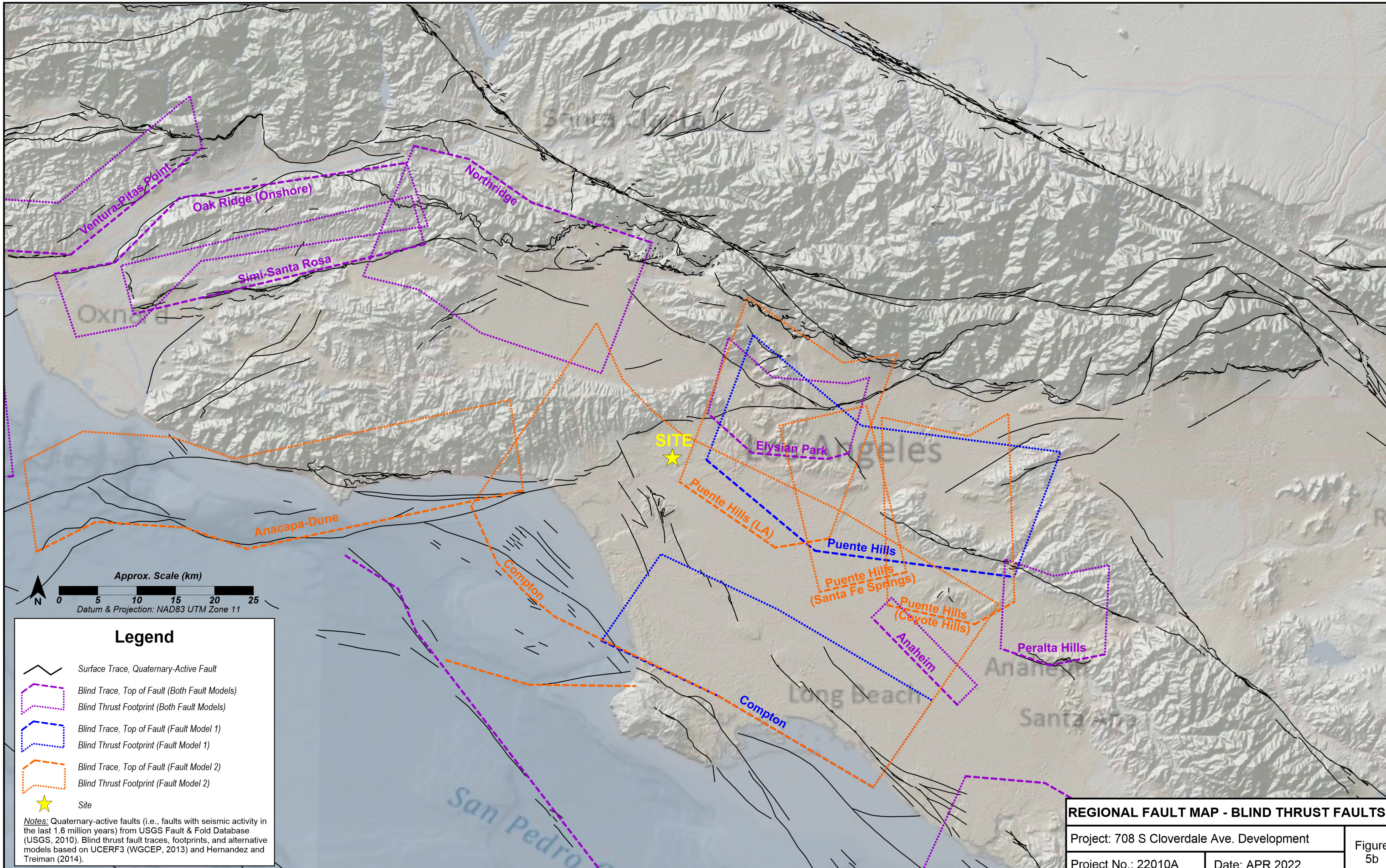
GeoPentech



REGIONAL FAULT & SEISMICITY MAP

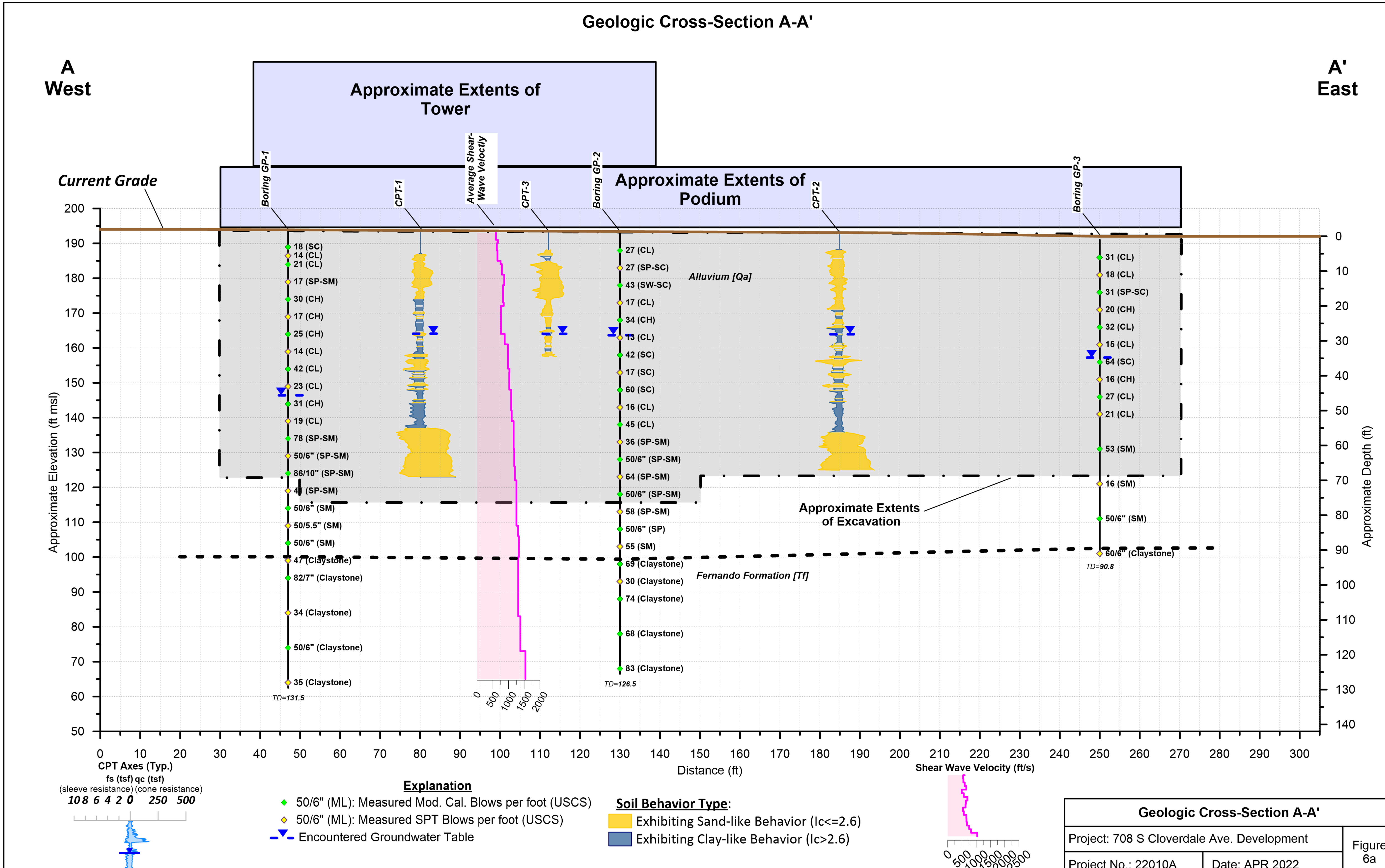
Project: 708 S Cloverdale Ave. Development		Figure 5a
Project No.: 22010A	Date: APR 2022	

GeoPentech



Ventura-Pitas Point
Oak Ridge (Onshore)
Northridge
Simi-Santa Rosa
Oxnard
Santa Clarita
Anacapa-Dune
SITE
Elysian Park
Los Angeles
Puente Hills (LA)
Puente Hills
Puente Hills (Santa Fe Springs)
Puente Hills (Coyote Hills)
Anaheim
Peralta Hills
Compton
Compton
Long Beach
Anaheim
Santa Ana
San Pedro
Approx. Scale (km)
0 5 10 15 20 25
N
Datum & Projection: NAD83 UTM Zone 11
Legend
Surface Trace, Quaternary-Active Fault
Blind Trace, Top of Fault (Both Fault Models)
Blind Thrust Footprint (Both Fault Models)
Blind Trace, Top of Fault (Fault Model 1)
Blind Thrust Footprint (Fault Model 1)
Blind Trace, Top of Fault (Fault Model 2)
Blind Thrust Footprint (Fault Model 2)
Site
Notes: Quaternary-active faults (i.e., faults with seismic activity in the last 1.6 million years) from USGS Fault & Fold Database (USGS, 2010). Blind thrust fault traces, footprints, and alternative models based on UCERF3 (WGCEP, 2013) and Hernandez and Treiman (2014).
REGIONAL FAULT MAP - BLIND THRUST FAULTS
Project: 708 S Cloverdale Ave. Development
Project No.: 22010A
Date: APR 2022
Figure 5b

GeoPentech



Geologic Cross-Section A-A'
A
West
A'
East
Approximate Extents of Tower
Approximate Extents of Podium
Current Grade
Boring GP-1
CPT-1
Average Shear-Wave Velocity
CPT-3
Boring GP-2
CPT-2
Boring GP-3
Alluvium [Qa]
Fernando Formation [Tf]
Approximate Extents of Excavation
18 (SC)
14 (CL)
21 (CL)
17 (SP-SM)
30 (CH)
17 (CH)
25 (CH)
14 (CL)
42 (CL)
23 (CL)
31 (CH)
19 (CL)
78 (SP-SM)
50/6" (SP-SM)
86/10" (SP-SM)
44 (SP-SM)
50/6" (SM)
50/5.5" (SM)
50/6" (SM)
47 (Claystone)
82/7" (Claystone)
34 (Claystone)
50/6" (Claystone)
35 (Claystone)
TD=131.5
27 (CL)
27 (SP-SC)
43 (SW-SC)
17 (CL)
34 (CH)
13 (CL)
42 (SC)
17 (SC)
60 (SC)
16 (CL)
45 (CL)
36 (SP-SM)
50/6" (SP-SM)
64 (SP-SM)
50/6" (SP-SM)
58 (SP-SM)
50/6" (SP)
55 (SM)
69 (Claystone)
30 (Claystone)
74 (Claystone)
68 (Claystone)
83 (Claystone)
TD=126.5
31 (CL)
18 (CL)
31 (SP-SC)
20 (CH)
32 (CL)
15 (CL)
64 (SC)
16 (CH)
27 (CL)
21 (CL)
53 (SM)
16 (SM)
50/6" (SM)
50/6" (Claystone)
TD=90.8
Approximate Elevation (ft msl)
Approximate Depth (ft)
Distance (ft)
0 500 1000 1500 2000
CPT Axes (Typ.)
fs (tsf) qc (tsf)
(sleeve resistance) (cone resistance)
10 8 6 4 2 0 250 500
Explanation
50/6" (ML): Measured Mod. Cal. Blows per foot (USCS)
50/6" (ML): Measured SPT Blows per foot (USCS)
Encountered Groundwater Table
Soil Behavior Type:
Exhibiting Sand-like Behavior (Ic<=2.6)
Exhibiting Clay-like Behavior (Ic>2.6)
Shear Wave Velocity (ft/s)
0 500 1000 1500 2000 2500
Geologic Cross-Section A-A'
Project: 708 S Cloverdale Ave. Development
Figure 6a
Project No.: 22010A
Date: APR 2022

GeoPentech

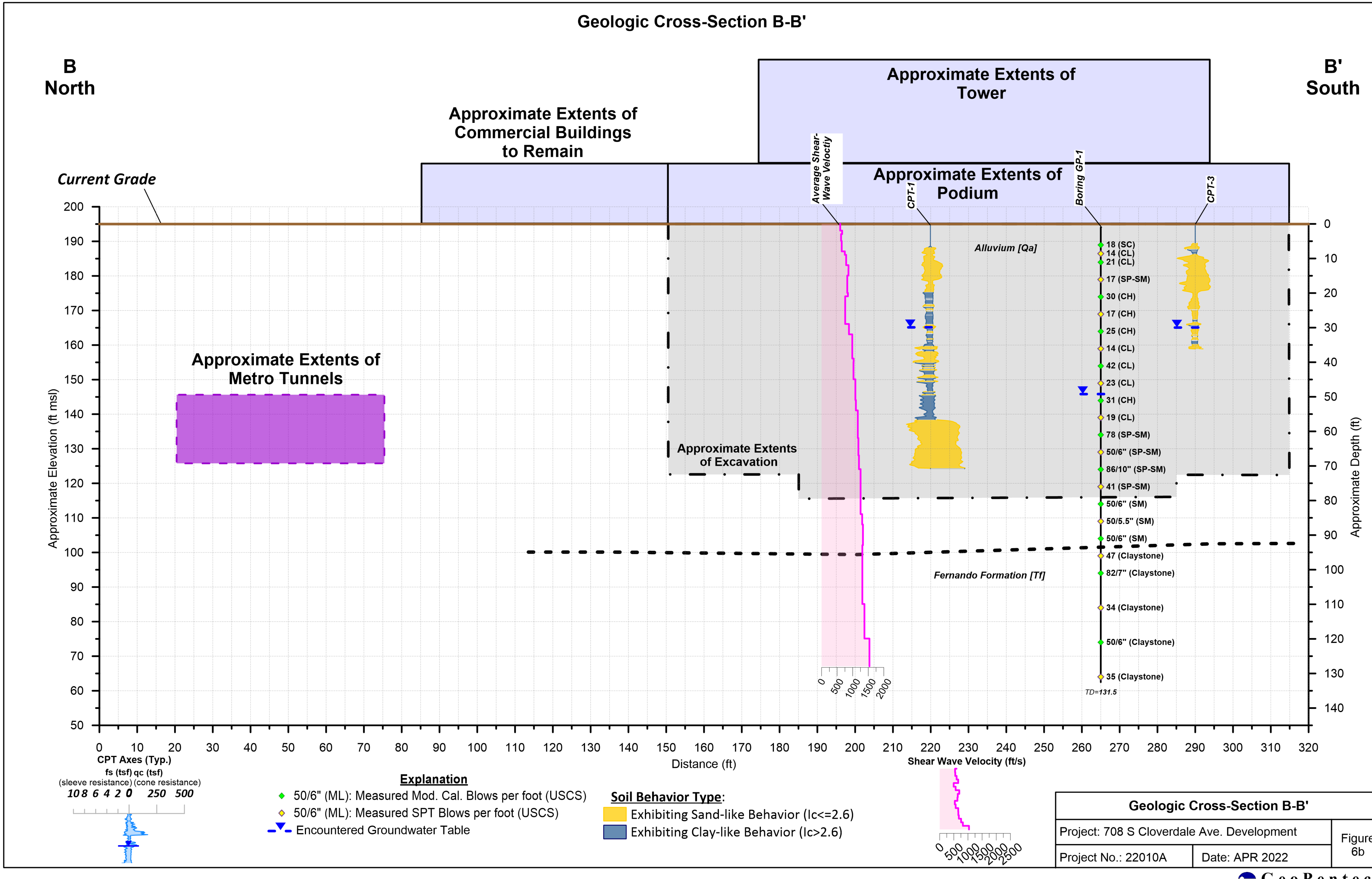
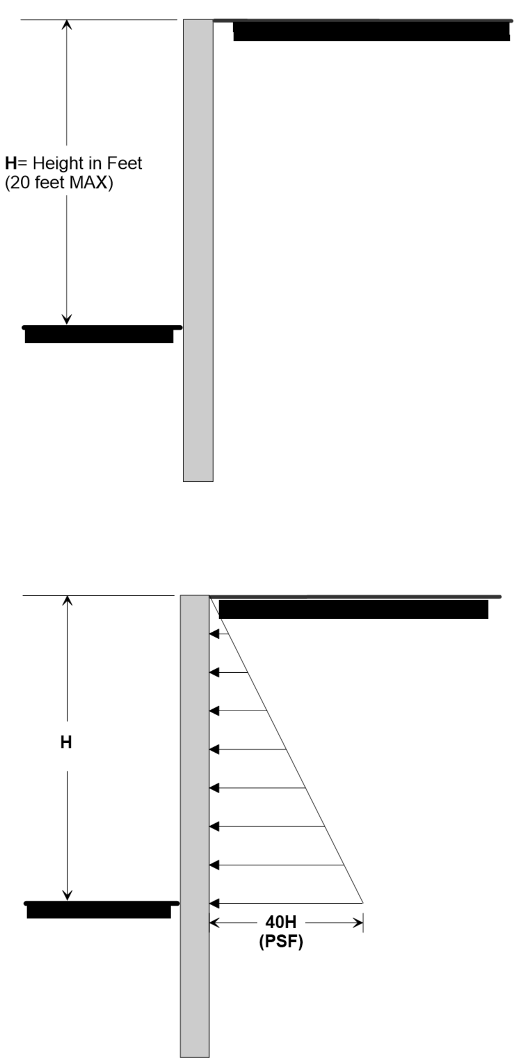
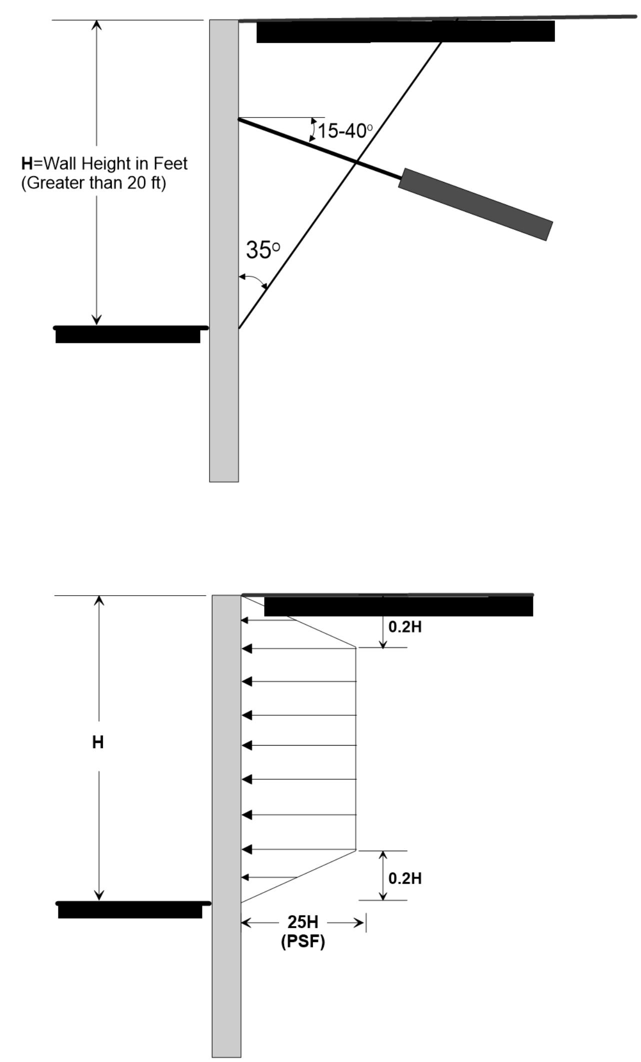

Geologic Cross-Section B-B'
B
North
B'
South
Approximate Extents of Tower
Approximate Extents of Commercial Buildings to Remain
Approximate Extents of Podium
Current Grade
Average Shear-Wave Velocity
CPT-1
Boring GP-1
CPT-3
Alluvium [Qa]
Approximate Extents of Metro Tunnels
Approximate Extents of Excavation
Fernando Formation [Tf]
18 (SC)
14 (CL)
21 (CL)
17 (SP-SM)
30 (CH)
17 (CH)
25 (CH)
14 (CL)
42 (CL)
23 (CL)
31 (CH)
19 (CL)
78 (SP-SM)
50/6" (SP-SM)
86/10" (SP-SM)
41 (SP-SM)
50/6" (SM)
50/5.5" (SM)
50/6" (SM)
47 (Claystone)
82/7" (Claystone)
34 (Claystone)
50/6" (Claystone)
35 (Claystone)
TD=131.5
Approximate Elevation (ft msl)
Approximate Depth (ft)
Distance (ft)
CPT Axes (Typ.)
fs (tsf) qc (tsf)
(sleeve resistance) (cone resistance)
10 8 6 4 2 0 250 500
Shear Wave Velocity (ft/s)
0 500 1000 1500 2000 2500
Explanation
50/6" (ML): Measured Mod. Cal. Blows per foot (USCS)
50/6" (ML): Measured SPT Blows per foot (USCS)
Encountered Groundwater Table
Soil Behavior Type:
Exhibiting Sand-like Behavior (Ic<=2.6)
Exhibiting Clay-like Behavior (Ic>2.6)
Geologic Cross-Section B-B'
Project: 708 S Cloverdale Ave. Development
Project No.: 22010A
Date: APR 2022
Figure 6b
GeoPentech

Figure 7a
Cantilever Shoring - Drained



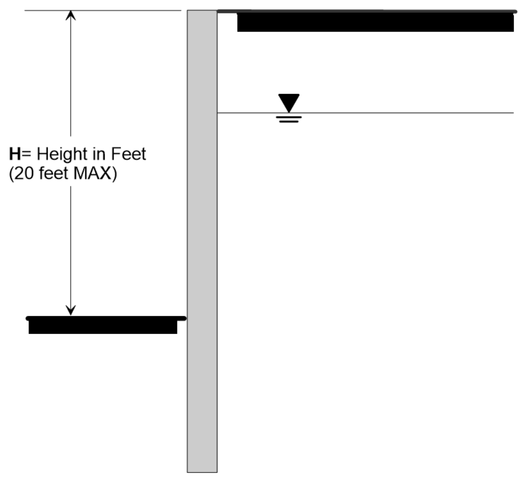
1. Lateral Earth Pressure is 40H with a triangular distribution.
2. The lateral pressure above assumes no build up of hydrostatic pressure behind the shoring.
3. The shoring should be designed to resist applicable surcharge loads from adjacent structures, stockpiled material, or traffic loads.

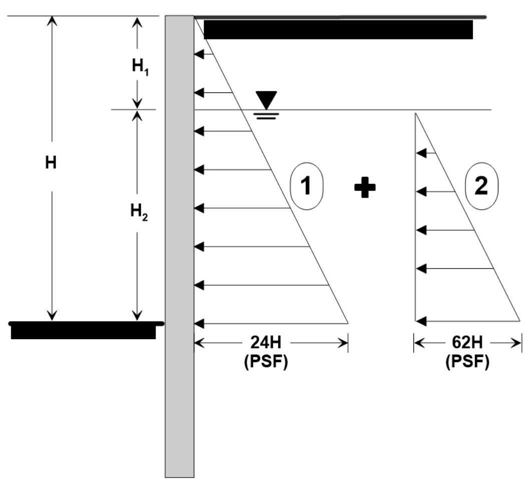
Figure 7b
Tiedback Shoring - Drained



1. Maximum Lateral Earth Pressure is 25H with a trapezoidal distribution.
2. The shoring should be designed to resist applicable surcharge loads from adjacent structures, stockpiled material, or traffic loads.

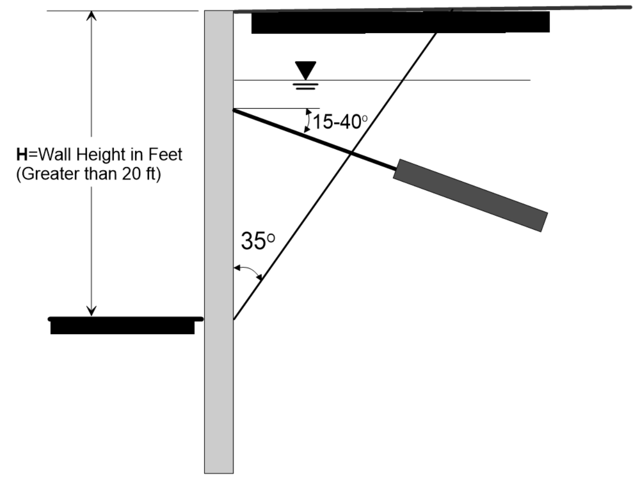
Figure 8a
Cantilever Shoring - Undrained

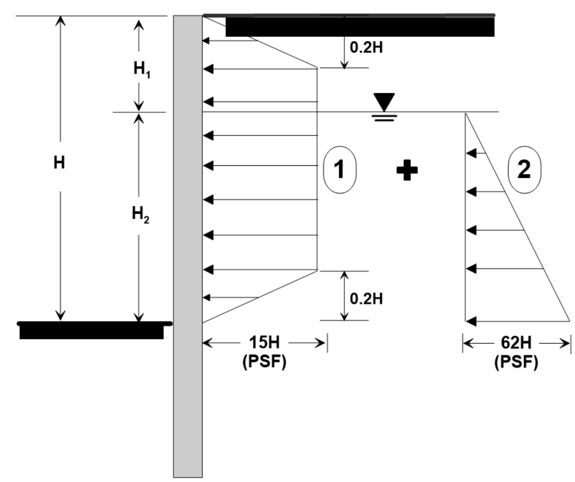




1. Recommended values are based on Groundwater depth of 10 ft bgs (H_1=10') and a total wall height of 75 ft (H=75').
2. Lateral Earth Pressure is 24H with a triangular distribution.
3. Hydrostatic pressure is 62H with a triangular distribution below the water table.
4. The shoring should be designed to resist applicable surcharge loads from adjacent structures, stockpiled material, or traffic loads.

Figure 8b
Tiedback Shoring - Undrained





1. Recommended values are based on a Groundwater depth of 10 ft bgs (H_1=10') and a total wall height of 75 ft (H=75').
2. Maximum Lateral Earth Pressure is 15H with a trapezoidal distribution.
3. Hydrostatic pressure is 62H with a triangular distribution below the water table.
4. The shoring should be designed to resist applicable surcharge loads from adjacent structures, stockpiled material, or traffic loads.

TEMPORARY SHORING DETAILS - SUBMERGED			
Project No.: 22010A	Project: 708 S Cloverdale Ave. Development	Date: APR 2022	Figure 8

GeoPentech

APPENDIX A

BORING LOGS

A.1 BORING LOGS

The current drilling was performed by GeoPentech over the course of three days on March 7-9, 2022 (Borings GP-1 , GP-2, and GP-3). The explorations consisted of advancing three borings: GP-1 to a depth of approximately 131.5 ft, GP-2 to approximately 126.5 ft, and GP-3 to approximately 90.8 ft below the ground surface. The approximate locations of the borings are indicated on Figure 2 in the main report. The borings were drilled using an 8-inch diameter hollow stem auger, and drilling mud was poured into the auger when the boring initially encountered groundwater. The work was performed under the supervision of an engineer or a geologist who monitored the drilling operations and prepared a field record of soils observed and drilling conditions. The drilling was subcontracted to Martini Drilling, who provided all drilling equipment, crew, and supplies.

During drilling, soil samples were obtained at approximate intervals ranging between 2.5 and 5-foot using a Standard Penetration Test (SPT) sampler, or a Modified California (MC) sampler SPT and MC samples were taken by driving a sampler approximately 18 inches into the soil at the bottom of the boring using a 140-pound hammer falling approximately 30 inches. The truck mounted CME 75 rig used by Martini Drilling utilized an automatic-trip hammer.

The SPT sampler cutting shoe and barrel have nominal inside diameters of 1.375 and 1.50 inches, respectively, and a nominal outside diameter of 2.00 inches. The barrel had no space for internal liners which were not used. The SPT samples were placed in plastic bags, labeled, and sealed. The MC sampler cutting shoe and barrel have nominal inside diameters of 2.38 and 2.50 inches, respectively, and a nominal outside diameter of 3 inches. Nominal 6-inch long, 2.4-inch diameter brass tubes or alternatively assemblies of 1-inch long, 2.4-inch diameter brass rings combined to fill the sampler were used to line the barrel. Plastic end caps were placed on the MC tubes to help preserve the moisture content of the samples. Bulk soil samples were also obtained at certain depths in selected boreholes. Upon completion of drilling, logging, and sampling, all borings were backfilled with neat cement slurry and patched at the surface with concrete.

After recovering the sample, the engineer or geologist noted the depth interval, recorded a description of the recovered material onto a field log, and sealed and labeled the sample for transport to the laboratory. The soil descriptions noted on the field logs were visually classified in accordance with the Unified Soil Classification System. The results of the borehole drilling and logging effort are provided on the borehole logs and on a key to the logs of boreholes.

Project: 708 S Cloverdale Ave. Development Project Location: 708 S Cloverdale Ave. Project Number: 22010A	Key to Log of Boring Sheet 1 of 1

Elevation, feet	Depth, feet	SAMPLES Type	Number	Blows Per 6"	Recovery	Graphic Log	MATERIAL DESCRIPTION	Dry Unit Weight, pcf	Water Content, %	REMARKS
1	2	3	4	5	6	7	8	9	10	11

COLUMN DESCRIPTIONS

1 **Elevation:** Elevation in feet referenced to mean sea level (MSL).

2 **Depth:** Depth in feet below the ground surface.

3 **Sample Type:** Type of soil sample collected at depth interval shown; sampler symbols are explained below.

4 **Sample Number:** Sample identification number.

5 **Sampling Resistance:** Number of blows required to advance driven sampler 6 inches, or distance noted, using the drive weight listed in hammer data. Hydraulic down-pressure may be recorded for pushed samplers.

6 **Sample Recovery:** Amount of sample recovered from sampling interval; given as inches of sample recovered or ratio of sample length to drive length (expressed as a percentage, %)

7 **Graphic Log:** Graphic depiction of subsurface material encountered; typical symbols are explained below.

8 **Material Description:** Description of material encountered; may include density/consistency (from field assessments), moisture, color (Munsell code), and grain size.

9 **Water Content:** Water content of sample, as percentage of dry weight of soil, measured in lab according to ASTM D2216.

10 **Dry Unit Weight:** The weight of soil solids per cubic foot of total volume of soil mass, measured according to ASTM D2937.

11 **Remarks and Other Tests:** Comments and observations regarding drilling or sampling made by driller or field personnel. Other lab tests are indicated using abbreviations explained below.

TYPICAL MATERIAL GRAPHIC SYMBOLS

POORLY GRADED SAND W/ Clay (SP-SC)

POORLY GRADED SAND W/ Silt (SP-SM)

WELL GRADED SAND W/ Clay (SW-SC)

Silty SAND (SM)

Clayey SAND (SC)

LEAN CLAY (CL)

FAT CLAY (CH)

CLAYSTONE

TYPICAL SAMPLER GRAPHIC SYMBOLS

Bulk Sample

Modified Calfiornia

Standard Penetration Test

Grab Sample

OTHER LABORATORY TEST ABBREVIATIONS

CONSOL	1-D Consolidation testing (ASTM D2435)
CORR	Corrosion testing (DOT CA test methods 643, 417, 422)
DS	Consolidated drained direct shear test (ASTM D3080)
FC	Fines Content wash on #200 sieve (ASTM D1140)
LL	Liquid Limit from Atterberg Limits test (ASTM D4318)
PI	Plasticity Index; NP indicates non-plastic determination
G=XX%	Percentage (%) Gravel
S=XX%	Percentage (%) Sand
F=XX%	Percentage (%) Fines

OTHER GRAPHIC SYMBOLS

____ Contact between strata

_ _ _ Inferred contact between strata or gradational change

Change within material properties within a stratum

Soil classifications are based on the Unified Soil Classification System. Descriptions and stratum lines are interpretive; field descriptions have been modified to reflect lab test results. Descriptions on these logs apply only at the specific boring locations and at the time the borings were advanced; they are not warranted to be representative of subsurface conditions at other locations or times. Datum used is WGS84.

Project: 708 S Cloverdale Ave. Development
Project Location: 708 S Cloverdale Ave.
Project Number: 22010A

Log of GP-1

Date(s) Drilled	**3/7/2022**	Logged By	**R. Wakefield**	Checked By	**M. Eslami**
Drilling Method	**Hollow Stem Auger & Mud Rotary**	Drill Bit Size/Type	**8" bullet-type bit**	Total Depth of Borehole	**131.5 feet**
Drill Rig Type	**CME 75**	Drilling Contractor	**Martini Drilling**	Approximate Surface Elevation	**~193 ft NAVD88**
Groundwater Level(s)	**53.4'**	Sampling Method	**Bulk, Grab, SPT, MC**	Hammer Data	**Automatic Trip Hammer 140-lbs/30" drop**
Borehole Location	**34.061721°, -118.346265°**	Borehole Completion	**Borehole backfilled with neat cement slurry from bottom of hole to surface**		

Elevation, feet	Depth, feet	SAMPLES Type	Number	Blows Per 6"	Recovery	Graphic Log	MATERIAL DESCRIPTION	Dry Unit Weight, pcf	Water Content, %	REMARKS
	0		Bulk-1				**[ASPHALT]** 5" Asphalt 5" Base			Hand Auger 0 - 5ft B-1: CORR
							[Alluvium (Qa)] **Clayey SAND (SC)**, moist, very dark gray, medium plasticity, no HCl reaction, abundant mica			
190			1							
	5		2	4 7 11			becomes medium dense	112.3	13.4	FC = 38.1% LL=41 PI=29
185			3	5 6 8			**Sandy Lean CLAY (CL)**, moist, stiff, light yellowish brown, fine sand, slight HCl reaction, occasional strong HCl reaction, localized calcite nodules			
	10		4	4 7 14			same as above		21.7	G=0% S=45% F=55% LL=40 PI=27
180										
	15		5	5 8 9			**Poorly Graded SAND with Silt (SP-SM)**, moist, medium dense, pale yellow, fine sand, no HCl reaction, slight mica			
175										
	20		6	6 11 19			**Fat CLAY with Sand (CH)**, moist, very stiff, olive gray, very plastic, moderate HCl reaction, some strong HCl reaction with calcite rich nodule	93.7	29.3	FC = 80.5% LL=71 PI=50 CONSOL
170										
	25		7	6 7 10			becomes light yellowish brown, spotty iron oxide staining			
165										
	30									

Elevation, feet	Depth, feet	Type	Number	Blows Per 6"	Recovery	Graphic Log	MATERIAL DESCRIPTION	Dry Unit Weight, pcf	Water Content, %	REMARKS
	30		8	6 11 14			becomes dark greenish gray	89.5	33.0	LL=61 PI=44
160										
	35		9	5 6 8			**Sandy Lean CLAY (CL)**, moist, stiff, dark greenish gray gray, high plasticity, moderate HCl reaction, some strong HCl reaction with calcite rich nodule			
155										
	40		10	17 19 23			becomes very stiff	110.5	19.8	FC = 55.3% LL=47 PI=32 DS CONSOL
150										
	45		11	4 10 13			occasional sandier zones, low plasticity			
145										
	50		12	6 8 23			**Fat CLAY with Sand (CH)**, moist, very stiff, dark greenish gray, high plasticity, moderate HCl reaction, some strong HCl reaction with calcite rich nodule	93.7	29.5	FC = 76.5% LL=65 PI=48 CONSOL
140							∇			Groundwater measured after 2 hours, rose from 60' bgs.
	55		13	4 7 12			**Lean CLAY (CL)**, moist, very stiff, dark greenish gray, high plasticity, moderate HCl reaction, some strong HCl reaction with calcite rich nodule			LL=38 PI=16
135										
	60		14	17 28 50/6"			**Poorly-graded SAND with Silt (SP-SM)**, moist, very dense, dark greenish gray, fine sand, no HCl reaction, odor	112.5	18.2	Groundwater initially encountered, rose to 53.4' bgs after 2 hours. Begin adding drilling mud. FC = 11.5%
130										
	65									

Elevation, feet	Depth, feet	Type	Number	Blows Per 6"	Recovery	Graphic Log	MATERIAL DESCRIPTION	Dry Unit Weight, pcf	Water Content, %	REMARKS
	65		15	10 50/6"			becomes gray, finer sand, strong odor			
125										
	70		16	11 36 50/4"			becomes moderate odor	144.4	17.3	FC = 10.9%
120										
	75		17	6 16 25			becomes dense, strong odor			G=0% S=92% F=8%
115										
	80		18	18 50/6"			**Silty SAND (SM)**, moist, very dense, gray, fine sand, no HCl reaction, strong odor	99.1	25.2	FC = 14%
110										
	85		19	24 50/5.5"						
105										
	90		20	32 50/6"			becomes moderate mica	95.8	24.5	
100										
	95		21	11 19 28			**[Fernando Formation]** **CLAYSTONE**, moist, hard, dark olive grey, high plasticity, no HCl reaction, abundant mica, possible laminations could be from sampling			
95										
	100									

Elevation, feet	Depth, feet	SAMPLES Number	Blows Per 6"	Recovery	MATERIAL DESCRIPTION	Dry Unit Weight, pcf	Water Content, %	REMARKS
	100	22	7 32 50/1"		some slight laminations visible	81.8	37.7	FC = 98.5% LL=53 PI=25 DS CONSOL
90								
	105							
85								
	110	23	25 13 21		Same as above			
80								
	115							
75								
	120	24	21 50/6"		Same as above		34.8	LL=51 PI=26
70								
	125							
65								
	130	25	8 14 21		becomes some darker brown angled laminations, slight petrol odor			
					TD: 131.5'			
60					Groundwater initially encountered at 60' bgs, rose to 53.4' bgs after 2 hours.			
	135				Borehole backfilled with neat cement slurry from bottom of hole to surface using a tremie pipe.			

Project: 708 S Cloverdale Ave. Development **Project Location: 708 S Cloverdale Ave.** **Project Number: 22010A**	# Log of GP-2 Sheet 1 of 4

Date(s) Drilled	**3/8-3/9/2022**	Logged By	**R. Wakefield**	Checked By	**M. Eslami**
Drilling Method	**Hollow Stem Auger & Mud Rotary**	Drill Bit Size/Type	**8" bullet-type bit**	Total Depth of Borehole	**126.5 feet**
Drill Rig Type	**CME 75**	Drilling Contractor	**Martini Drilling**	Approximate Surface Elevation	**~194 ft NAVD88**
Groundwater Level(s)	**36.1'**	Sampling Method	**Bulk, SPT, MC**	Hammer Data	**Automatic Trip Hammer 140-lbs/30" drop**
Borehole Location	**34.061656°, -118.345978°**	Borehole Completion	**Borehole backfilled with neat cement slurry from bottom of hole to surface**		

Elevation, feet	Depth, feet	SAMPLES: Type	SAMPLES: Number	SAMPLES: Blows Per 6"	SAMPLES: Recovery	Graphic Log	MATERIAL DESCRIPTION	Dry Unit Weight, pcf	Water Content, %	REMARKS
	0		Bulk-1				**[ASPHALT]** 4" Asphalt 3" Base **[Alluvium (Qa)]** **Lean CLAY with Sand (CL)**, stiff, moist, very dark gray, high plasticy; no HCl reaction except occasional CaCO3 specks			Hand Auger 0 - 5ft
190										
	5		1	5 10 17				115.7	16.1	FC = 72.6% LL=41 PI=29
							Color change in cuttings to light yellowish brown			
185										
	10		2a 2b	9 13 14			**Poorly Graded SAND with Silty Clay (SP-SC)**, medium dense, moist, very dark grey, fine SAND; no HCl reaction except occasional CaCO3 specks **Silty, Clayey SAND (SC-SM)**, medium dense, moist, light yellowish brown, fine SAND; no HCl reaction, slight mica			FC = 12.4% LL=23 PI=6
180										
	15		3	10 17 26			**Well-Graded SAND with Clay (SW-SC)**, medium dense, moist, pale brown, fine to medium SAND, trace fine gravel, no HCl reaction, some mica	109.4	5.9	FC = 9.3%
175										
	20		4	4 7 10			**Sandy Lean CLAY (CL)**, very stiff, moist, light olive grey, medium plasticity, strong HCl reaction on lighter colored CaCO3, slight mica			
170										
	25		5	8 13 21			**Sandy Fat CLAY (CH)**, very stiff, moist, dark greenish gray, very plastic, strong HCl reaction on lighter colored CaCO3, slight mica	95.0	27.8	FC = 65.2% LL=68 PI=48
165										
	30									

Elevation, feet	Depth, feet	Type	Number	Blows Per 6"	Recovery	Graphic Log	MATERIAL DESCRIPTION	Dry Unit Weight, pcf	Water Content, %	REMARKS
	30		6	3 5 8			**Lean CLAY (CL)**, stiff, moist, dark greenish gray, high plasticity, strong HCl reaction on lighter colored CaCO3, slight mica			
160										
	35		7	11 17 25			**Clayey SAND (SC)**, medium dense, moist, dark greenish gray, fine SAND; strong HCl reaction from lighter colored CaCO3 stringers	111.1	17.6	FC = 35.3% LL=53 PI=41 Groundwater measured after 22 hours, rose from 65' bgs.
155										
	40		8	7 7 10			same as above			
150										
	45		9	8 24 36			becomes dense, moderate HCl reaction	110.7	16.1	FC = 35.2% LL=46 PI=32 DS CONSOL
145										
	50		10	4 7 9			**Sandy Lean CLAY (CL)**, very stiff, moist, dark greenish gray, high plasticity; no HCl reaction from normal clay, strong HCl reaction on lighter colored CaCO3 veins			
140										
	55		11	6 16 24			abundant mica			LL=44 PI=21
135										
	60		12	10 17 19			**Poorly-graded SAND with Silt (SP-SM)**, dense, moist, greenish grey, fine sand, no HCl reaction, strong odor			
130										
	65									

Elevation, feet	Depth, feet	SAMPLES Type	Number	Blows Per 6"	Recovery	Graphic Log	MATERIAL DESCRIPTION	Dry Unit Weight, pcf	Water Content, %	REMARKS
	65		13	8 50/6"			becomes very dense	107.1	20.0	Groundwater initially encountered, rose to 36.1 ft bgs after 22 hours. Begin adding drilling mud. FC = 10.7% DS
125										
	70		14	7 27 37			same as above			
120										
	75		15	29 50/6"			becomes dark grey	103.9	21.2	FC = 9.4%
115										
	80		16	16 26 32			becomes more silty			LL=NP PI=NP
110										
	85		17	50/6"			**Poorly Graded SAND (SP)**, very dense, moist, dark grey, fine to medium sand, occasional fine to large well rounded gravel, no HCl reaction, moderate odor	122.2	11.9	
105										
	90		18	15 25 30			**Silty SAND (SM)**, very dense, moist, dark grey, fine sand, no HCl reaction, strong odor			
100										
	95		19	12 22 47			**[Fernando Formation]** **CLAYSTONE**, hard, moist, dark olive grey, can scrape and indent with thumbnail, no HCl reaction, abundant mica	90.2	30.4	FC = 98.5% LL=46 PI=24 CONSOL
95										
	100									

Project: 708 S Cloverdale Ave. Development
Project Location: 708 S Cloverdale Ave.
Project Number: 22010A

Log of GP-2

Elevation, feet	Depth, feet	SAMPLES Type	Number	Blows Per 6"	Recovery	Graphic Log	MATERIAL DESCRIPTION	Dry Unit Weight, pcf	Water Content, %	REMARKS
	100		20	7 12 18			slight HCl reaction, some CaCO3 and brown laminate within, dipping fairly steeply			
90										
	105		21	10 24 50/6"			no HCl reaction, spotty speckled white inclusions			
85										
	110									
80										
	115		22	14 26 42			slight HCl reaction, trace lighter colored CaCO3 rich strings	91.7	30.4	FC = 89.5% LL=50 PI=27 CONSOL
75										
	120									
70										
	125		23	18 33 50/6"			some darker brown layers, trace white specks			
65										
	130									
60										
	135									

Total depth: 126.5' bgs

Groundwater initially encountered at 65' bgs, rose to 36.1' bgs after 22 hours.

Borehole backfilled with neat cement slurry from bottom of hole to surface using a tremie pipe.

GeoPentech
Geotechnical & Geoscience Consultants

Project: 708 S Cloverdale Ave. Development Project Location: 708 S Cloverdale Ave. Project Number: 22010A	Log of GP-3 Sheet 1 of 3

Date(s) Drilled	3/8/2022	Logged By	R. Wakefield	Checked By	M. Eslami
Drilling Method	Hollow Stem Auger & Mud Rotary	Drill Bit Size/Type	8" bullet-type bit	Total Depth of Borehole	91.0 feet
Drill Rig Type	CME 75	Drilling Contractor	Martini Drilling	Approximate Surface Elevation	~191 ft NAVD88
Groundwater Level(s)	43.9'	Sampling Method	SPT, MC	Hammer Data	Automatic Trip Hammer 140-lbs/30" drop
Borehole Location	34.061660°, -118.345543°	Borehole Completion	Borehole backfilled with neat cement slurry from bottom of hole to surface		

Elevation, feet	Depth, feet	SAMPLES Type	Number	Blows Per 6"	Recovery	Graphic Log	MATERIAL DESCRIPTION	Dry Unit Weight, pcf	Water Content, %	REMARKS
	0						[ASPHALT] 5" Asphalt 5" Base			Hand Auger 0 - 5ft
190							[Alluvium (Qa)] **Sandy Lean CLAY (CL)**, moist, light yellowish brown, medium plasticity, no HCl reaction except from occasional mottled CaCO3 nodules			
185	5		1	7 10 21			becomes very stiff			
180	10		2	3 8 10			becomes very highly mottled with CaCO3 veins			
175	15		3	7 16 15			**Poorly Graded SAND with Clay (SP-SC)**, medium dense, moist, light yellowish brown	111.2	9.4	
							Sandy Lean CLAY (CL), very stiff, moist, light yellowish brown			
170	20		4	4 8 12			**Fat CLAY with sand (CH)**, very stiff, moist, pale olive, very plastic; very strong HCl reaction, abundant CaCO3 throughout, less sandy		22.1	FC = 73.5% LL=58 PI=43
165	25		5	4 11 21			**Sandy Lean CLAY (CL)**, very stiff, moist, pale olive, high plasticity; very strong HCl reaction, abundant CaCO3 throughout, less sandy	99.0	25.2	
	30									

Project: 708 S Cloverdale Ave. Development
Project Location: 708 S Cloverdale Ave.
Project Number: 22010A

Log of GP-3

Elevation, feet	Depth, feet	SAMPLES Type	Number	Blows Per 6"	Recovery	Graphic Log	MATERIAL DESCRIPTION	Dry Unit Weight, pcf	Water Content, %	REMARKS
160	30		6	2 5 10			becomes stiff, dark greenish grey, medium plasticity			
155	35		7	21 27 43			**Clayey SAND (SC)**, dense, moist, dark greenish gray, fine SAND; no HCl reaction	102.1	18.1	FC = 42.3% LL=48 PI=35 CONSOL
150	40		8	4 6 10			**Sandy Fat CLAY (CH)**, very stiff, moist, dark greenish gray, high plasticity; no HCl reaction, no CaCO3 veins			FC = 67.9% LL=52 PI=38
							∇			Groundwater initially encountered. Begin adding drilling mud.
145	45		9	4 8 19			**Sandy Lean CLAY (CL)**, very stiff, moist, dark greenish gray, high plasticity; slight HCl reaction, occasional CaCO3 veins			
140	50		10	4 7 14			same as above		22.2	FC = 57.8% LL=41 PI=25
135	55									
130	60		11	6 17 36			**Silty SAND (SM)**, dense, moist, greenish grey, fine sand, no HCl reaction, strong odor	112.3	17.8	FC = 12.2%
	65									

Report: GP SOIL BA LOG; File: 21010A 5350 WILSHIRE.GPJ; 5/5/2022

Project: 708 S Cloverdale Ave. Development
Project Location: 708 S Cloverdale Ave.
Project Number: 22010A

Log of GP-3

Elevation, feet	Depth, feet	Type	Number	Blows Per 6"	Recovery	Graphic Log	MATERIAL DESCRIPTION	Dry Unit Weight, pcf	Water Content, %	REMARKS
125	65									
120	70		12	4 6 10			becomes medium dense			
115	75									
110	80		13	22 50/6"			becomes very dense			FC = 15.2% DS
105	85									
100	90		14	4 10 50/0"			**[Fernando Formation]** **CLAYSTONE**, hard, freshly weathered, moist, dark olive grey, medium plasticity, abundant mica		46.3	FC = 70.4% LL=62 PI=27
95	95									
	100									

Total depth: 91' bgs

Groundwater initially encountered at 43.9' bgs.

Borehole backfilled with neat cement slurry from bottom of hole to surface using tremie pipe

APPENDIX B

LABORATORY TESTING

B.1 LABORATORY TESTING

The laboratory testing program performed by GeoPentech for the proposed project site included the following tests: moisture content, dry density, sieve analysis, wash analysis, direct shear, consolidation, and corrosion. The geotechnical testing was conducted at the laboratory facilities of AP Engineers in Pomona, California. The tests were performed in general accordance with applicable procedures of ASTM and the State of California Department of Transportation, Standard Test Methods (DOT CA). The results of the laboratory testing are included in this Appendix and are summarized in Table B-1 and on the boring logs in Appendix A. GeoPentech has reviewed the results of the laboratory testing and finds them acceptable. Brief descriptions of each test are presented in the following sections.

B.1.1 Moisture Content and Dry Density

For selected Modified California samples, the dry unit weight (in units of pounds-per-cubic-foot) and field moisture content (%) were measured in general accordance with ASTM D2937 and ASTM D2216, respectively, or with ASTM D7263.

B.1.2 Sieve Analysis and Wash Analysis

For selected samples, the particle-size distribution was determined by sieve analysis in general accordance with ASTM D6913. Sieve sizes ranged from ¾ in to 75 µm (No. 200).

For other selected samples, the percentage of fines (material passing the No. 200 sieve) was measured by wash analysis in accordance with ASTM D1140.

B.1.3 Atterberg Limits

The Atterberg limits test is a classification test that is performed on cohesive soils (i.e., silty and clayey soils) to measure the soil plastic limit (PL) and liquid limit (LL), from which the plasticity index (PI) is calculated. The measured values can be plotted on a plasticity chart, which is used as an aid in classifying the soil material and behavior. These tests were performed in accordance with ASTM D4318.

B.1.4 Corrosion Tests

Soil samples were tested for electrical resistivity, pH, sulfate content, and chloride content. These tests were performed in general accordance with DOT CA test methods 643 (electrical resistivity and pH), 417 (sulfate content), and 422 (chloride content). The test results were used to evaluate the corrosivity potential of the soil on underground improvements associated with the proposed structure.

B.1.5 Direct Shear

Direct shear tests were performed on selected Modified California samples in accordance with ASTM D3080 to measure peak and ultimate strength parameters. Shear stress and sample deformation were monitored throughout the tests.

B.1.6 Consolidation

Tests for one-dimensional consolidation properties of soils using incremental loading were performed on relatively undisturbed soil samples according to ASTM D2435. The test determines the magnitude and rate of consolidation of soil when it is restrained laterally and drained axially while subjected to incrementally applied controlled-stress loading. The test results provide clayey soil settlement parameters under different loading conditions.

Table B-1
Summary of Laboratory Testing

	Location		Classification	Initial Condition		Atterberg		Gradation			Corrosion				Peak Strength (DS)		Other Tests
Boring Number	Sample No.	Depth (ft)	USCS Symbol	Moisture (%)	Dry Density (pcf)	Liquid Limit	Plasticity Index	Gravel (%)	Sand (%)	Fines (%)	Min. Resistivity (Ohm-cm)	Sulfate Cont. (ppm)	Chloride Cont. (ppm)	pH	Friction Angle (Degrees)	Cohesion (psf)	TEST TYPE
GP-1	B-1	0-5'	SC								1037	153	23	8.7			
GP-1	1	2.5	SC														
GP-1	2	5	SC	13.4	112.3	41	29			38.1							
GP-1	3	7.5	CL														
GP-1	4	10	CL	21.7		40	27	0	45	55							
GP-1	5	15	SP-SM														
GP-1	6	20	CH	29.3	93.7	71	50			80.5							CONSOL
GP-1	7	25	CH														
GP-1	8	30	CH	33.0	89.5	61	44										
GP-1	9	35	CL														
GP-1	10	40	CL	19.8	110.5	47	32			55.3					31	700	CONSOL
GP-1	11	45	CL														
GP-1	12	50	CH	29.5	93.7	65	48			76.5					23	1200	
GP-1	13	55	CL			38	16										
GP-1	14	60	SP-SM	18.2	112.5					11.5							
GP-1	15	65	SP-SM														
GP-1	16	70	SP-SM	17.3	114.4					10.9							
GP-1	17	75	SP-SM					0	92	8							
GP-1	18	80	SM	25.2	99.1					14							
GP-1	19	85	SM														
GP-1	20	90	SM	24.5	95.8												
GP-1	21	95	Claystone														
GP-1	22	100	Claystone	37.7	81.8	53	25			98.5					29	1450	CONSOL
GP-1	23	110	Claystone														
GP-1	24	120	Claystone	34.8		51	26										
GP-1	25	130	Claystone														
GP-2	B-1	0-5'	CL														
GP-2	1	5	CL	16.1	115.7	41	29			72.6							
GP-2	2a	10	SP-SC			23	6			12.4							
GP-2	2b	11	SC-SM														
GP-2	3	15	SW-SC	5.9	109.4					9.3					36	250	
GP-2	4	20	CL														
GP-2	5	25	CH	27.8	95.0	68	48			65.2							CONSOL

Table B-1
Summary of Laboratory Testing

	Location		Classification	Initial Condition		Atterberg		Gradation			Corrosion				Peak Strength (DS)		Other Tests
Boring Number	Sample No.	Depth (ft)	USCS Symbol	Moisture (%)	Dry Density (pcf)	Liquid Limit	Plasticity Index	Gravel (%)	Sand (%)	Fines (%)	Min. Resistivity (Ohm-cm)	Sulfate Cont. (ppm)	Chloride Cont. (ppm)	pH	Friction Angle (Degrees)	Cohesion (psf)	TEST TYPE
GP-2	6	30	CL														
GP-2	7	35	SC	17.6	111.1	53	41			35.3							
GP-2	8	40	SC														
GP-2	9	45	SC	16.1	110.7	46	32			35.2					31	750	CONSOL
GP-2	10	50	CL														
GP-2	11	55	CL			44	21										
GP-2	12	60	SP-SM														
GP-2	13	65	SP-SM	20.0	107.1					10.7					37	200	
GP-2	14	70	SP-SM														
GP-2	15	75	SP-SM	21.2	103.9					9.4							
GP-2	16	80	SP-SM			NP	NP										
GP-2	17	85	SP	11.9	122.2												
GP-2	18	90	SM														
GP-2	19	95	Claystone	30.4	90.2	46	24			98.5							CONSOL
GP-2	20	100	Claystone														
GP-2	21	105	Claystone														
GP-2	22	115	Claystone	30.4	91.7	50	27			89.5							CONSOL
GP-2	23	125	Claystone														
GP-3	1	5	CL														
GP-3	2	10	CL														
GP-3	3	15	SP-SC	9.4	111.2												
GP-3	4	20	CH	22.1		58	43			73.5							
GP-3	5	25	CL	25.2	99.0												
GP-3	6	30	CL														
GP-3	7	35	SC	18.1	102.1	48	35			42.3							CONSOL
GP-3	8	40	CH			52	38			67.9							
GP-3	9	45	CL														
GP-3	10	50	CL	22.2		41	25			57.8							
GP-3	11	60	SM	17.8	112.3					12.2							
GP-3	12	70	SM														
GP-3	13	80	SM							15.2					36	350	
GP-3	14	90	Claystone	46.3		62	27			70.4							

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CORROSION TEST RESULTS

Client Name: GeoPentech
Project Name: 5350 Wilshire Development
Project No.: 22010A

AP Job No.: 22-0347
Date: 03/25/22

Boring No.	Sample No.	Depth (feet)	Soil Description	Minimum Resistivity (ohm-cm)	pH	Sulfate Content (ppm)	Chloride Content (ppm)
GP-1	B-1	0-5	Clay	1,037	8.7	153	23

NOTES:
Resistivity Test and pH: California Test Method 643
Sulfate Content : California Test Method 417
Chloride Content : California Test Method 422
ND = Not Detectable
NA = Not Sufficient Sample
NR = Not Requested

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MOISTURE AND DENSITY TEST RESULTS

ASTM D2216 and ASTM D7263 (Method B)

Client: GeoPentech
Project Name: 5350 Wilshire Development
Project No.: 22010A

AP Lab No.: 22-0347
Test Date: 03/22/22

Boring No.	Sample No.	Sample Depth (ft.)	Moisture Content (%)	Dry Density (pcf)
GP-1	2	5	13.4	112.3
GP-1	4	10	21.7	NA
GP-1	6	20	29.3	93.7
GP-1	8	30	33.0	89.5
GP-1	10	40	19.8	110.5
GP-1	12	50	29.5	93.7
GP-1	14	60	18.2	112.5
GP-1	16	70	17.3	114.4
GP-1	18	80	25.2	99.1
GP-1	20	90	24.5	95.8
GP-1	22	100	37.7	81.8
GP-1	24	120	34.8	NA

PERCENT PASSING NO. 200 SIEVE

ASTM D1140

Client: GeoPentech

Project Name: 5350 Wilshire Development

Project Number: 22010A

AP Lab No.: 22-0347

Test Date: 03/23/22

Boring No.	Sample No.	Depth (ft)	Percent Fines (%)
GP-1	2	5	38.1
GP-1	6	20	80.5
GP-1	10	40	55.3
GP-1	12	50	76.5
GP-1	14	60	11.5
GP-1	16	70	10.9
GP-1	18	80	14.0
GP-1	22	100	98.5

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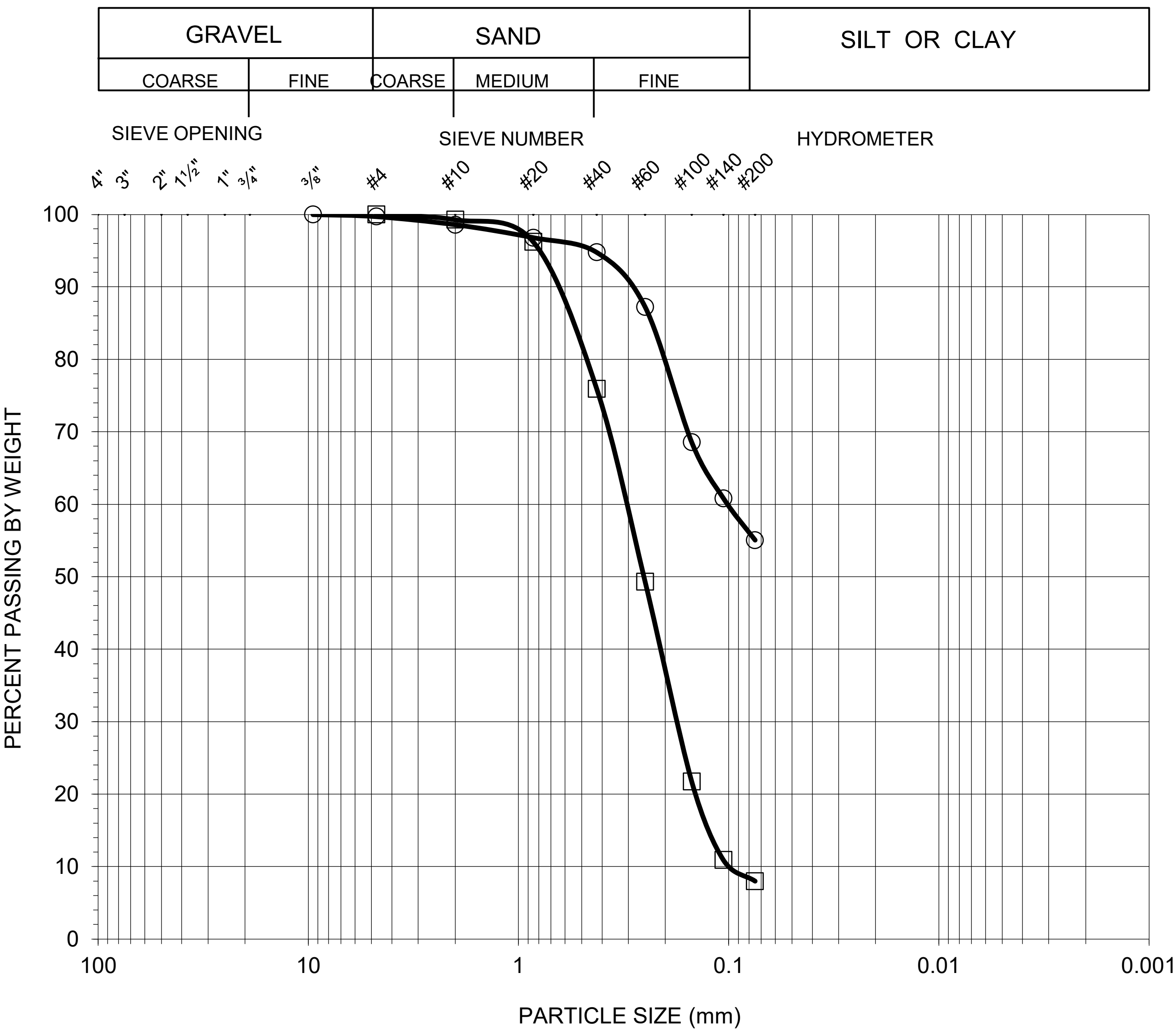
GRAIN SIZE DISTRIBUTION CURVE
ASTM D 6913

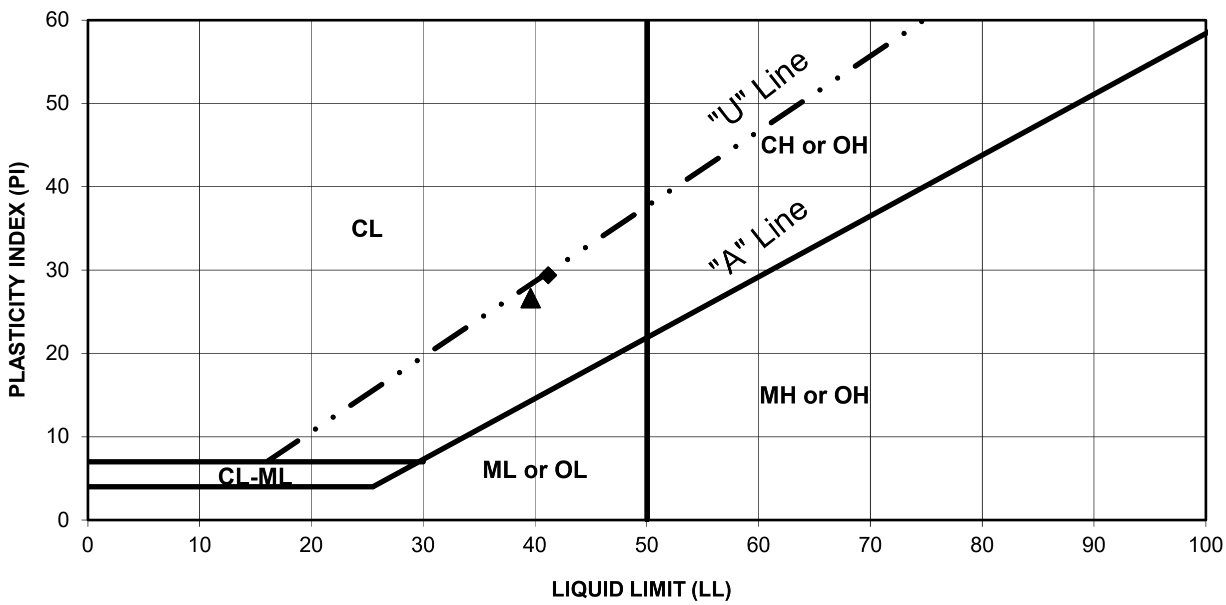
Client Name:	GeoPentech	Tested by:	JT	Date:	03/23/22
Project Name:	5350 Wilshire Development	Computed by:	NR	Date:	03/25/22
Project No.:	22010A	Checked by:	AP	Date:	03/30/22



Symbol	Boring No.	Sample No.	Sample Depth (feet)	Percent			Atterberg Limits LL:PL:PI	Soil Type U.S.C.S
				Gravel	Sand	Silt & Clay		
○	GP-1	4	10	0	45	55	40:13:27	CL
□	GP-1	17	75	0	92	8	N/A	SP-SM

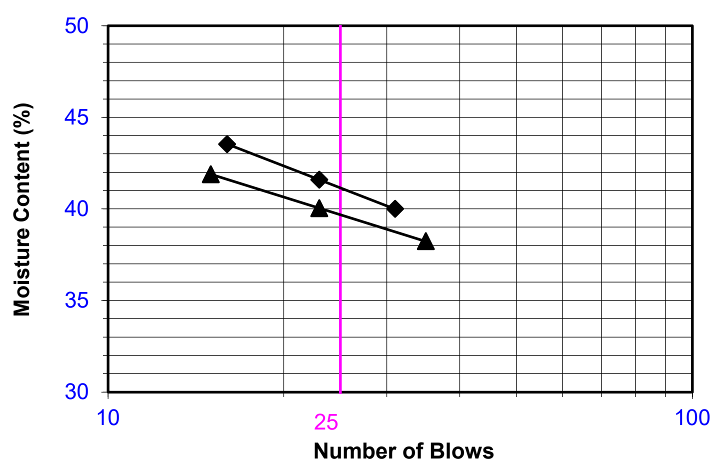
ATTERBERG LIMITS
ASTM D 4318

Client Name: GeoPentech　Tested By: LS　Date: 03/21/22
Project Name: 5350 Wilshire Development　Computed By: NR　Date: 03/30/22
Project No.: 22010A　Checked By: AP　Date: 03/30/22



PROCEDURE USED

- [ ] Wet Preparation
- [X] Dry Preparation
- [X] Procedure A Multipoint Test
- [ ] Procedure B One-point Test



Symbol	Boring Number	Sample Number	Depth (feet)	LL	PL	PI	Plasticity Chart Symbol
♦	GP-1	2	5	41	12	29	CL
▲	GP-1	4	10	40	13	27	CL

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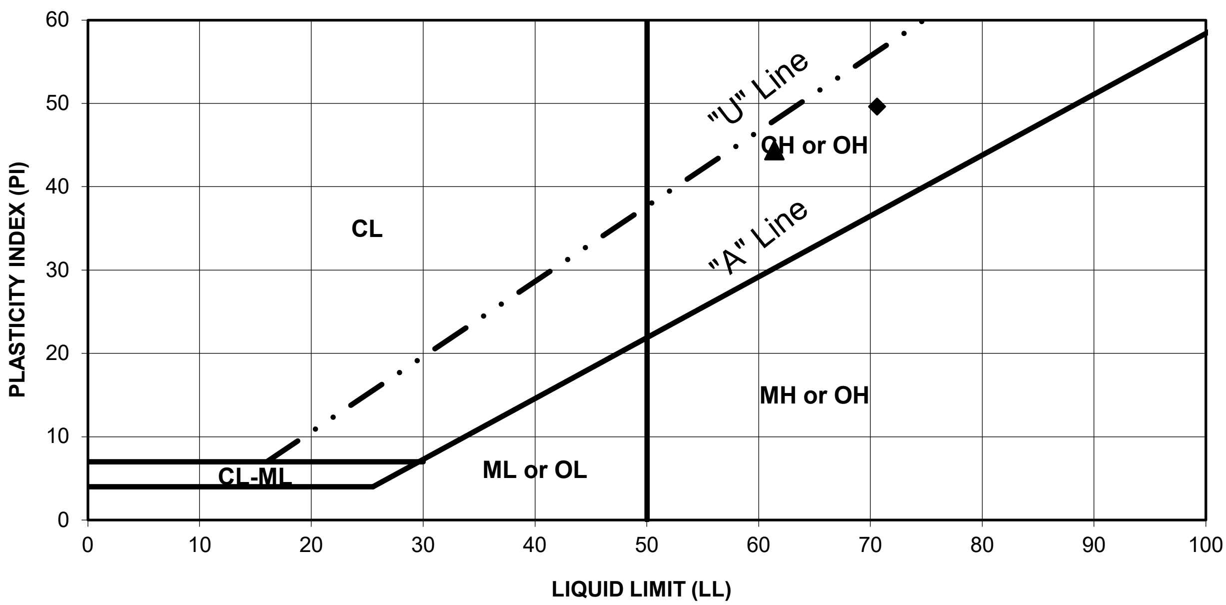
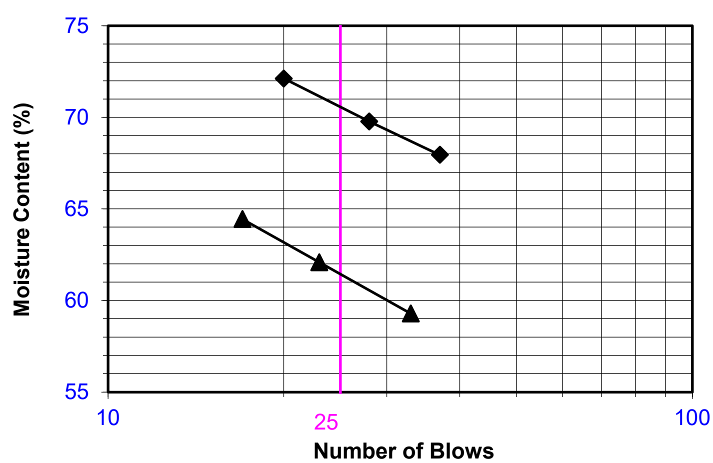
ATTERBERG LIMITS
ASTM D 4318

Client Name: GeoPentech | Tested By: LS | Date: 03/21/22
Project Name: 5350 Wilshire Development | Computed By: NR | Date: 03/30/22
Project No.: 22010A | Checked By: AP | Date: 03/30/22



PROCEDURE USED

- [ ] Wet Preparation
- [X] Dry Preparation
- [X] Procedure A Multipoint Test
- [ ] Procedure B One-point Test



Symbol	Boring Number	Sample Number	Depth (feet)	LL	PL	PI	Plasticity Chart Symbol
♦	GP-1	6	20	71	21	50	CH
▲	GP-1	8	30	61	17	44	CH

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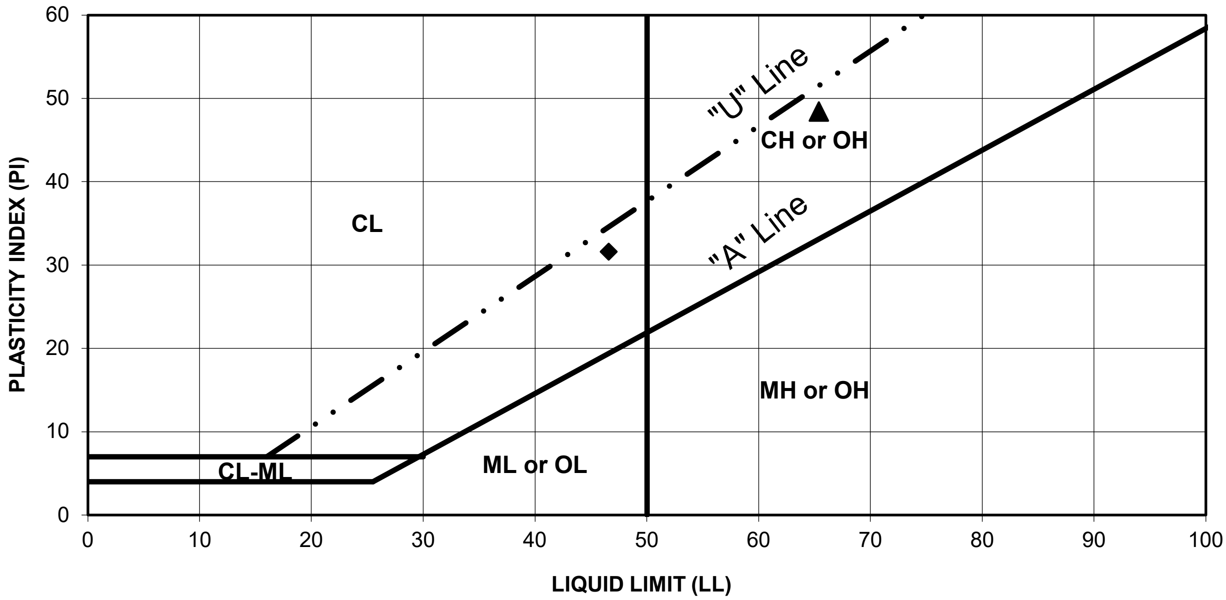
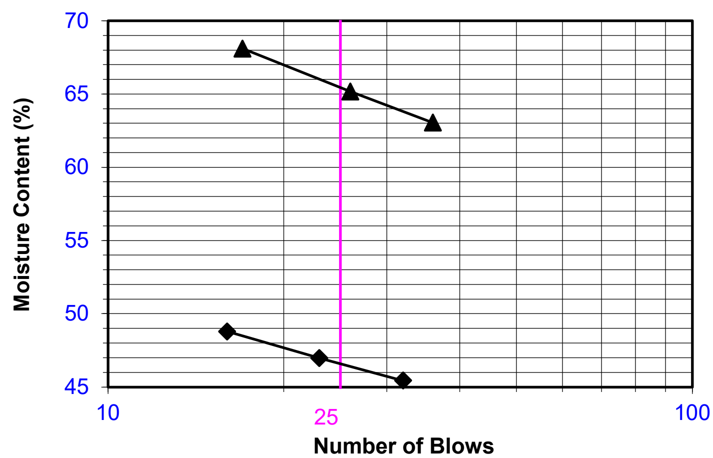
ATTERBERG LIMITS
ASTM D 4318

Client Name:	GeoPentech	Tested By:	LS	Date:	03/21/22
Project Name:	5350 Wilshire Development	Computed By:	NR	Date:	03/30/22
Project No.:	22010A	Checked By:	AP	Date:	03/30/22



PROCEDURE USED

- [ ] Wet Preparation
- [x] Dry Preparation
- [x] Procedure A Multipoint Test
- [ ] Procedure B One-point Test



Symbol	Boring Number	Sample Number	Depth (feet)	LL	PL	PI	Plasticity Chart Symbol
♦	GP-1	10	40	47	15	32	CL
▲	GP-1	12	50	65	17	48	CH

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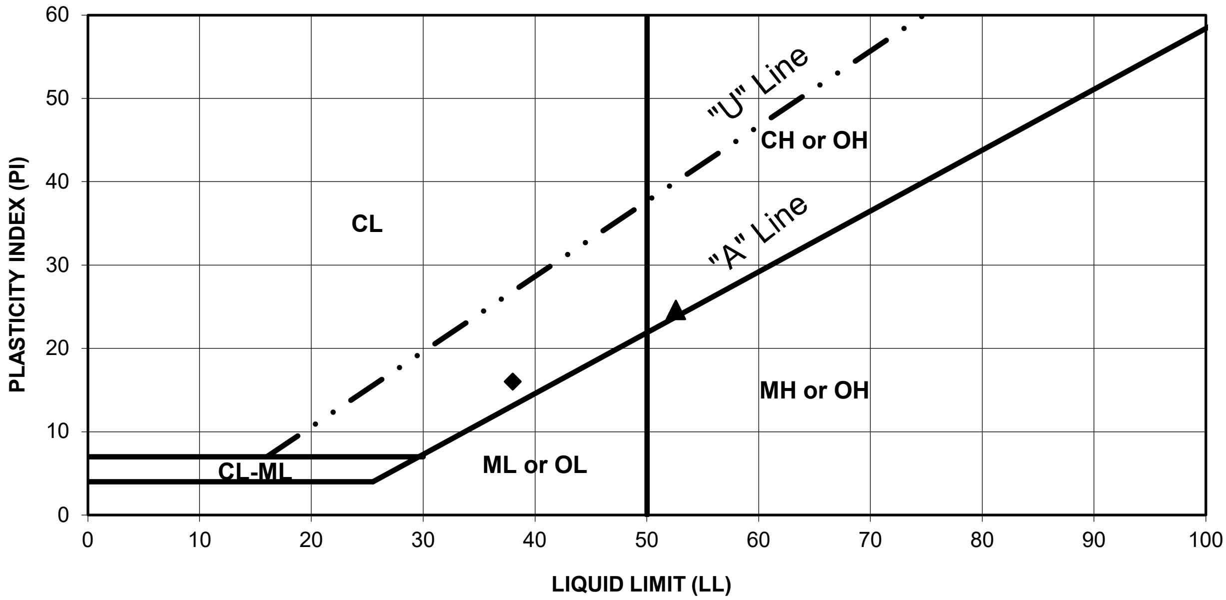
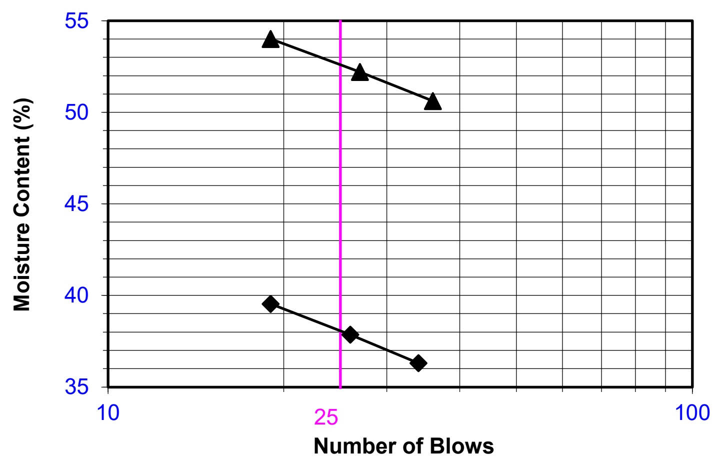
ATTERBERG LIMITS
ASTM D 4318

Client Name: GeoPentech — Tested By: LS — Date: 03/21/22
Project Name: 5350 Wilshire Development — Computed By: NR — Date: 03/30/22
Project No.: 22010A — Checked By: AP — Date: 03/30/22



PROCEDURE USED

- [ ] Wet Preparation
- [X] Dry Preparation
- [X] Procedure A Multipoint Test
- [ ] Procedure B One-point Test



Symbol	Boring Number	Sample Number	Depth (feet)	LL	PL	PI	Plasticity Chart Symbol
♦	GP-1	13	55	38	22	16	CL
▲	GP-1	22	100	53	28	25	CH

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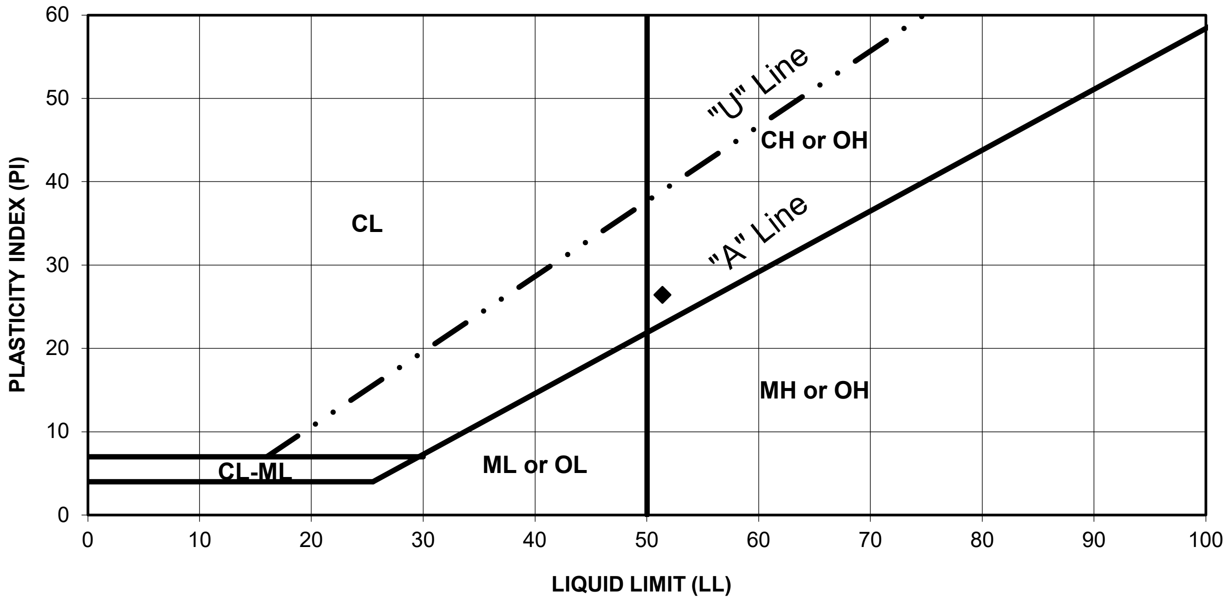
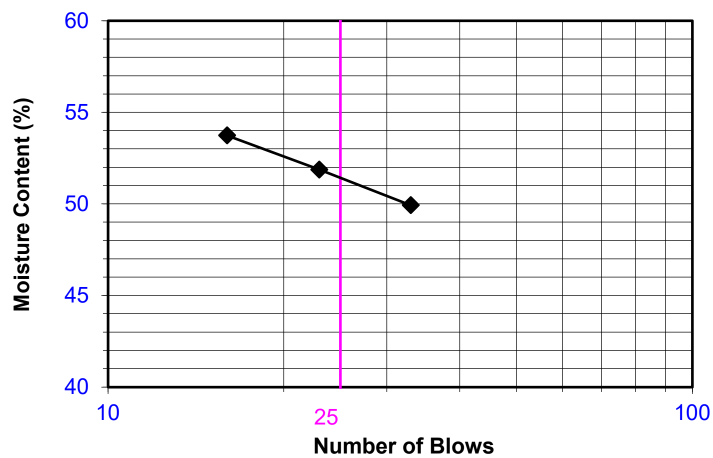
ATTERBERG LIMITS
ASTM D 4318

Client Name:	GeoPentech	Tested By:	LS	Date:	03/21/22
Project Name:	5350 Wilshire Development	Computed By:	NR	Date:	03/30/22
Project No.:	22010A	Checked By:	AP	Date:	03/30/22



PROCEDURE USED

- [ ] Wet Preparation
- [X] Dry Preparation
- [X] Procedure A Multipoint Test
- [ ] Procedure B One-point Test



Symbol	Boring Number	Sample Number	Depth (feet)	LL	PL	PI	Plasticity Chart Symbol
♦	GP-1	24	120	51	25	26	CH

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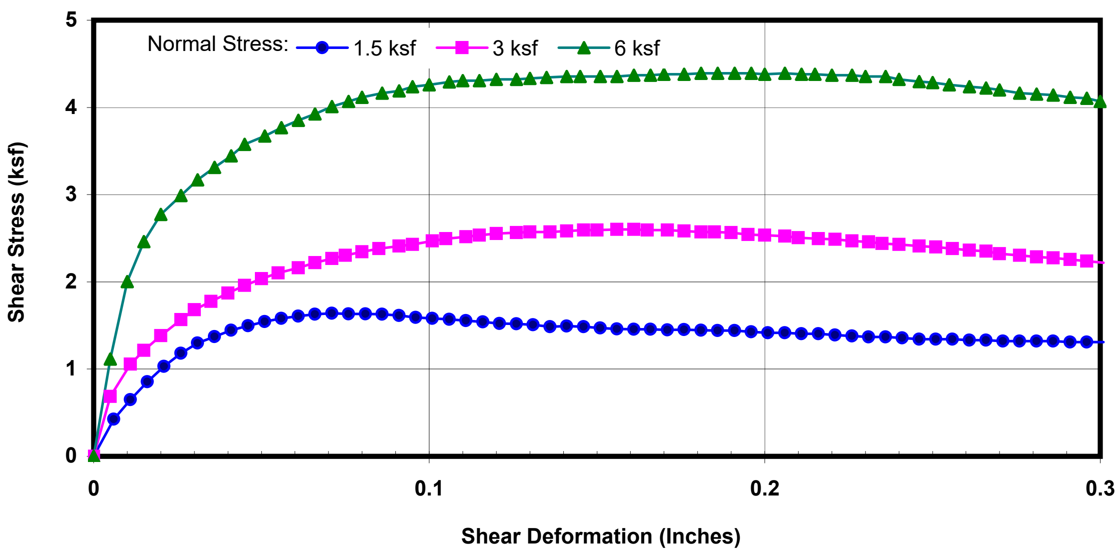
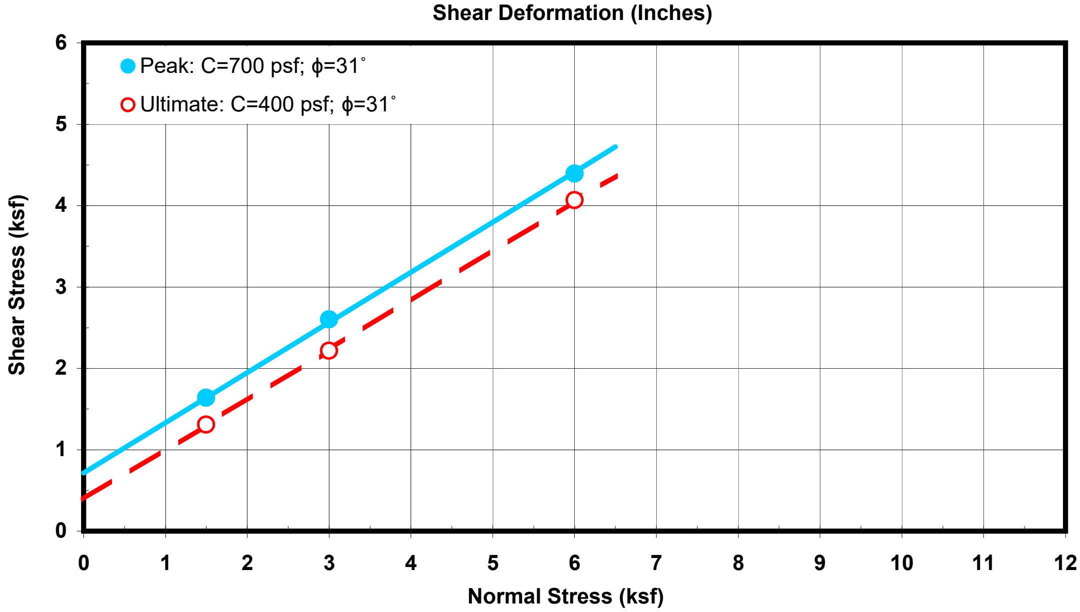
DIRECT SHEAR TEST RESULTS
ASTM D 3080

Client: GeoPentech
Project Name: 5350 Wilshire Development
Project No.: 22010A
Boring No.: GP-1
Sample No.: 10 **Depth (ft):** 40
Sample Type: Mod. Cal.
Soil Description: Sandy Lean Clay
Test Condition: Inundated **Shear Type:** Regular

Tested By: ST **Date:** 03/23/22
Computed By: NR **Date:** 03/24/22
Checked by: AP **Date:** 03/30/22

Wet Unit Weight (pcf)	Dry Unit Weight (pcf)	Initial Moisture Content (%)	Final Moisture Content (%)	Initial Degree Saturation (%)	Final Degree Saturation (%)	Normal Stress (ksf)	Peak Shear Stress (ksf)	Ultimate Shear Stress (ksf)
130.3	108.8	19.8	20.4	97	100	1.5	1.639	1.308
						3	2.602	2.218
						6	4.392	4.068





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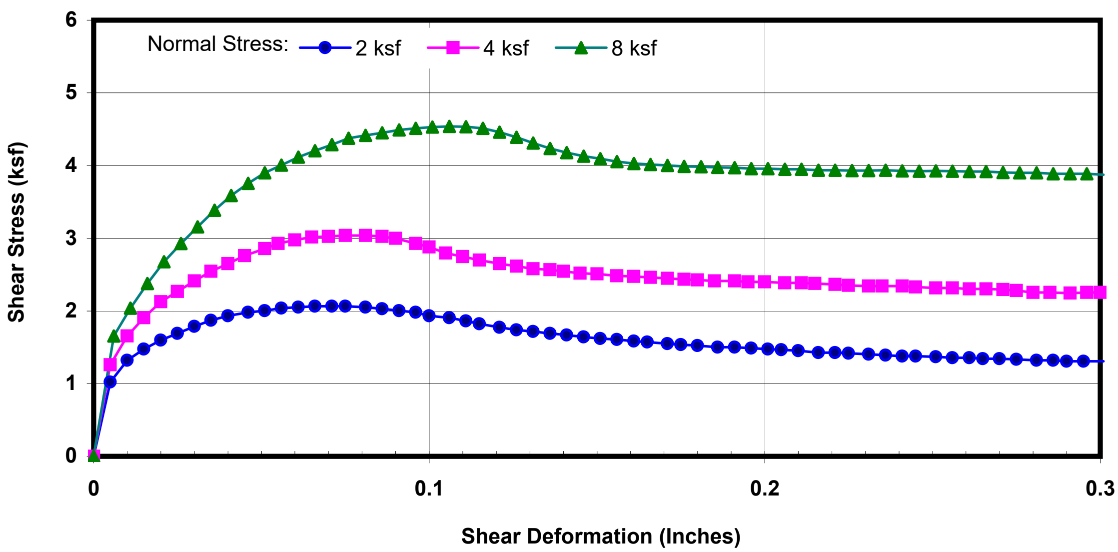
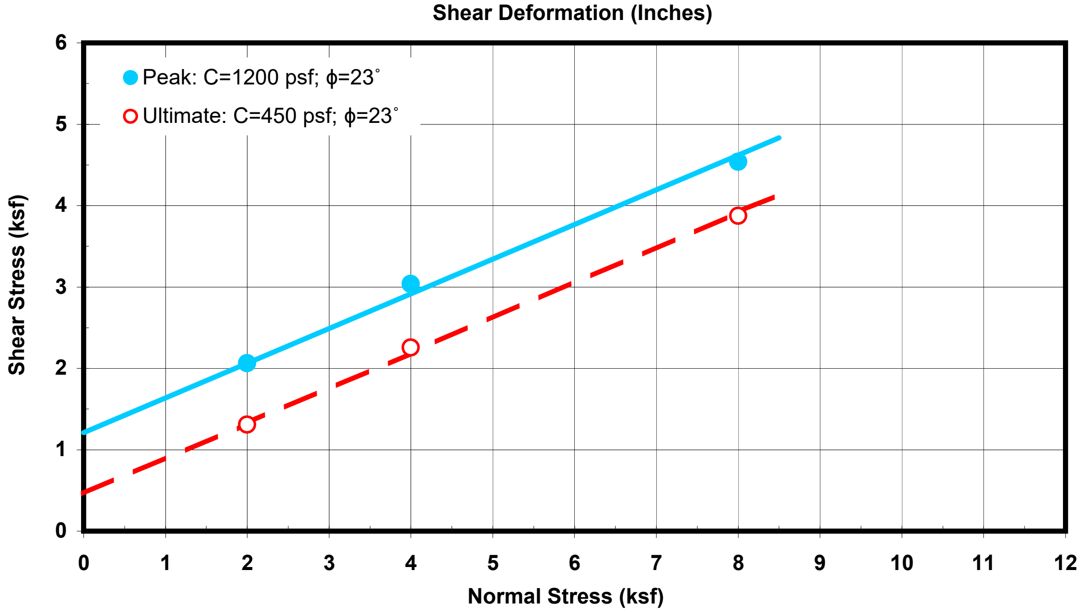
DIRECT SHEAR TEST RESULTS
ASTM D 3080

Client:	GeoPentech	**Tested By:** KM	**Date:** 03/24/22
Project Name:	5350 Wilshire Development	**Computed By:** NR	**Date:** 03/30/22
Project No.:	22010A	**Checked by:** AP	**Date:** 03/30/22
Boring No.:	GP-1		
Sample No.:	12	**Depth (ft):** 50	
Sample Type:	Mod. Cal.		
Soil Description:	Fat Clay w/sand		
Test Condition:	Inundated	**Shear Type:** Regular	

Wet Unit Weight (pcf)	Dry Unit Weight (pcf)	Initial Moisture Content (%)	Final Moisture Content (%)	Initial Degree Saturation (%)	Final Degree Saturation (%)	Normal Stress (ksf)	Peak Shear Stress (ksf)	Ultimate Shear Stress (ksf)
121.1	93.5	29.5	29.7	99	100	2	2.064	1.308
						4	3.037	2.256
						8	4.539	3.876





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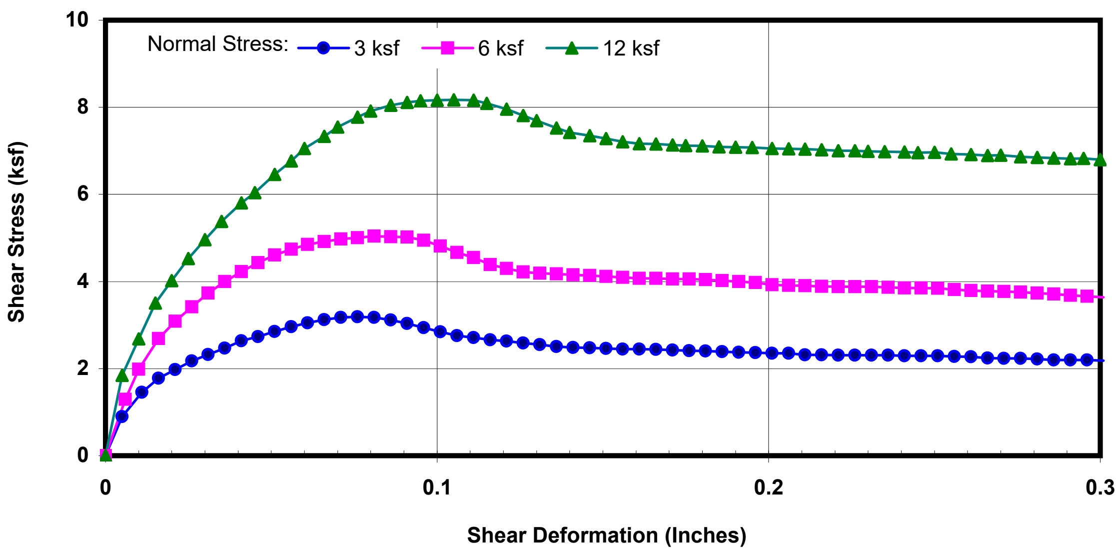
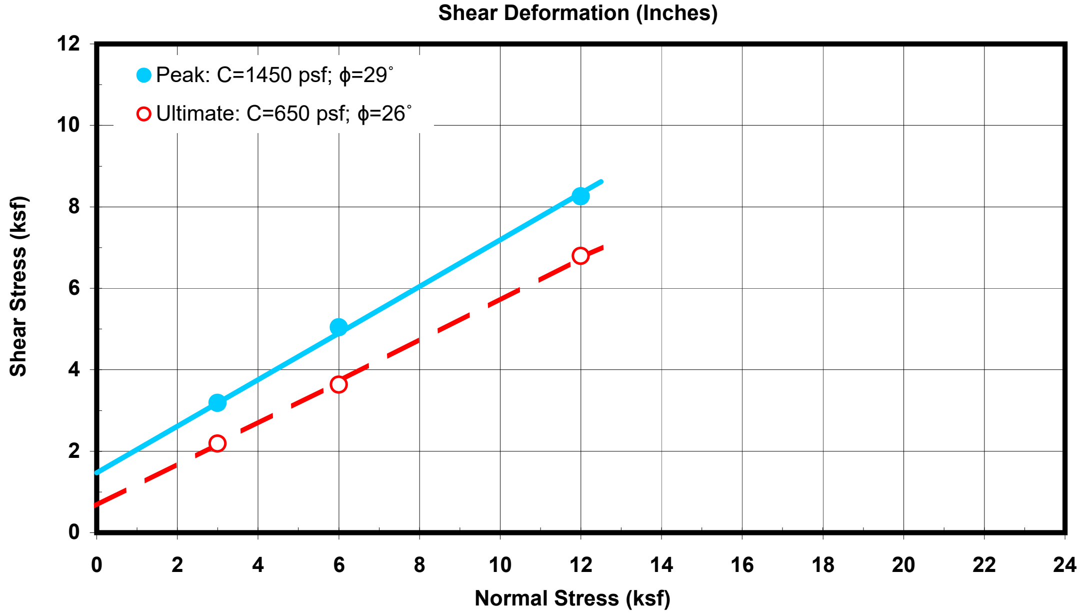
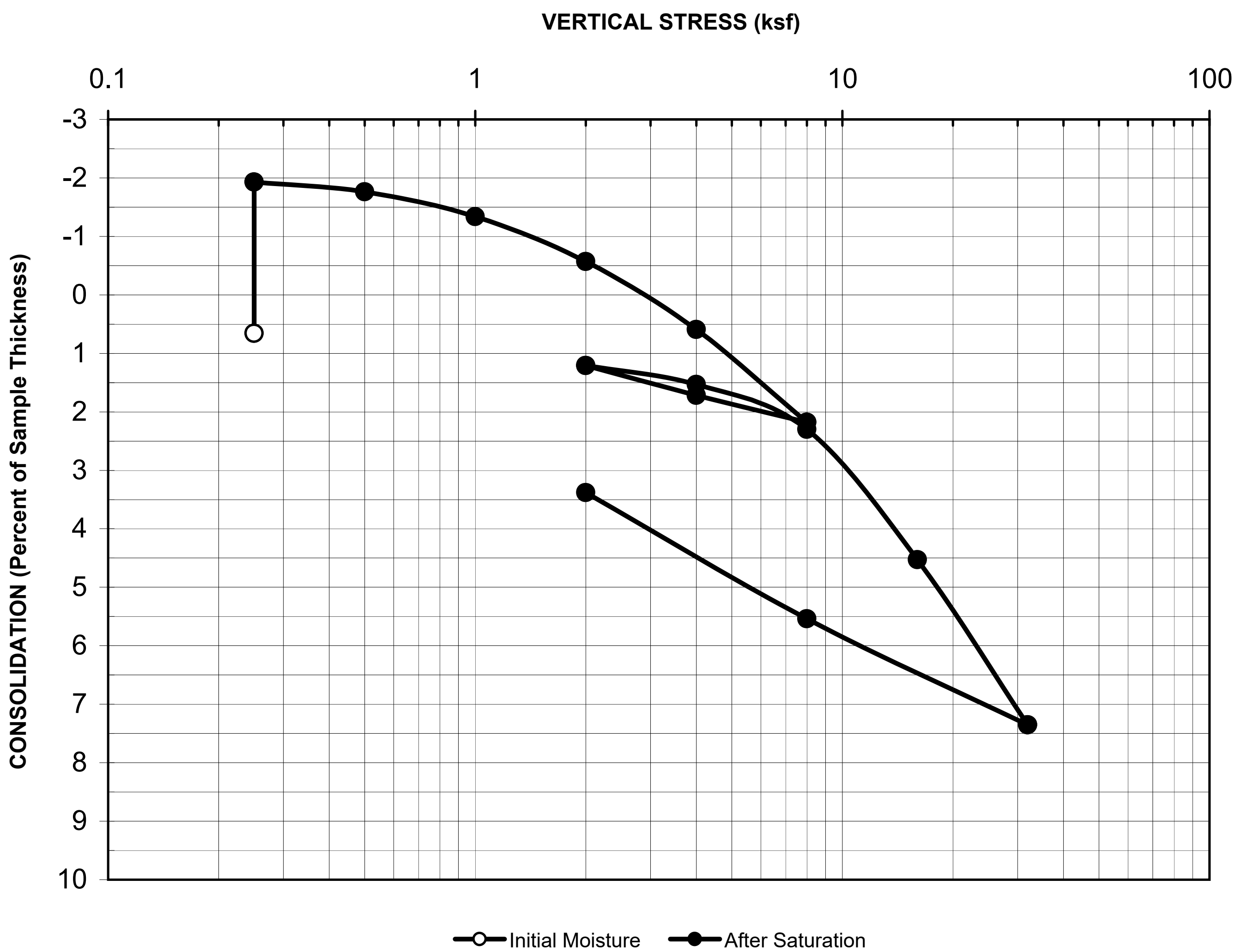
DIRECT SHEAR TEST RESULTS
ASTM D 3080

Client:	GeoPentech	**Tested By:**	KM	**Date:**	03/24/22
Project Name:	5350 Wilshire Development	**Computed By:**	NR	**Date:**	03/30/22
Project No.:	22010A	**Checked by:**	AP	**Date:**	03/30/22
Boring No.:	GP-1				
Sample No.:	22 **Depth (ft):** 100				
Sample Type:	Mod. Cal.				
Soil Description:	Claystone				
Test Condition:	Inundated **Shear Type:** Regular				

Wet Unit Weight (pcf)	Dry Unit Weight (pcf)	Initial Moisture Content (%)	Final Moisture Content (%)	Initial Degree Saturation (%)	Final Degree Saturation (%)	Normal Stress (ksf)	Peak Shear Stress (ksf)	Ultimate Shear Stress (ksf)
112.0	81.3	37.7	39.6	95	100	3	3.188	2.184
						6	5.040	3.636
						12	8.255	6.793







Boring No. :	GP-1	Initial Dry Unit Weight (pcf):	94.5
Sample No.:	6	Initial Moisture Content (%):	29.3
Depth (feet):	20	Final Moisture Content (%):	29.2
Sample Type:	Mod Cal	Assumed Specific Gravity:	2.9
Soil Description:	Fat Clay w/sand	Initial Void Ratio:	0.92
Remarks:	Swell= 2.59% upon inundation		

CONSOLIDATION CURVE
ASTM D 2435

Project Name:	5350 Wilshire Development
Project No.:	22010A
Date:	03/18/22
AP No:	22-0347
Sheet No:	1

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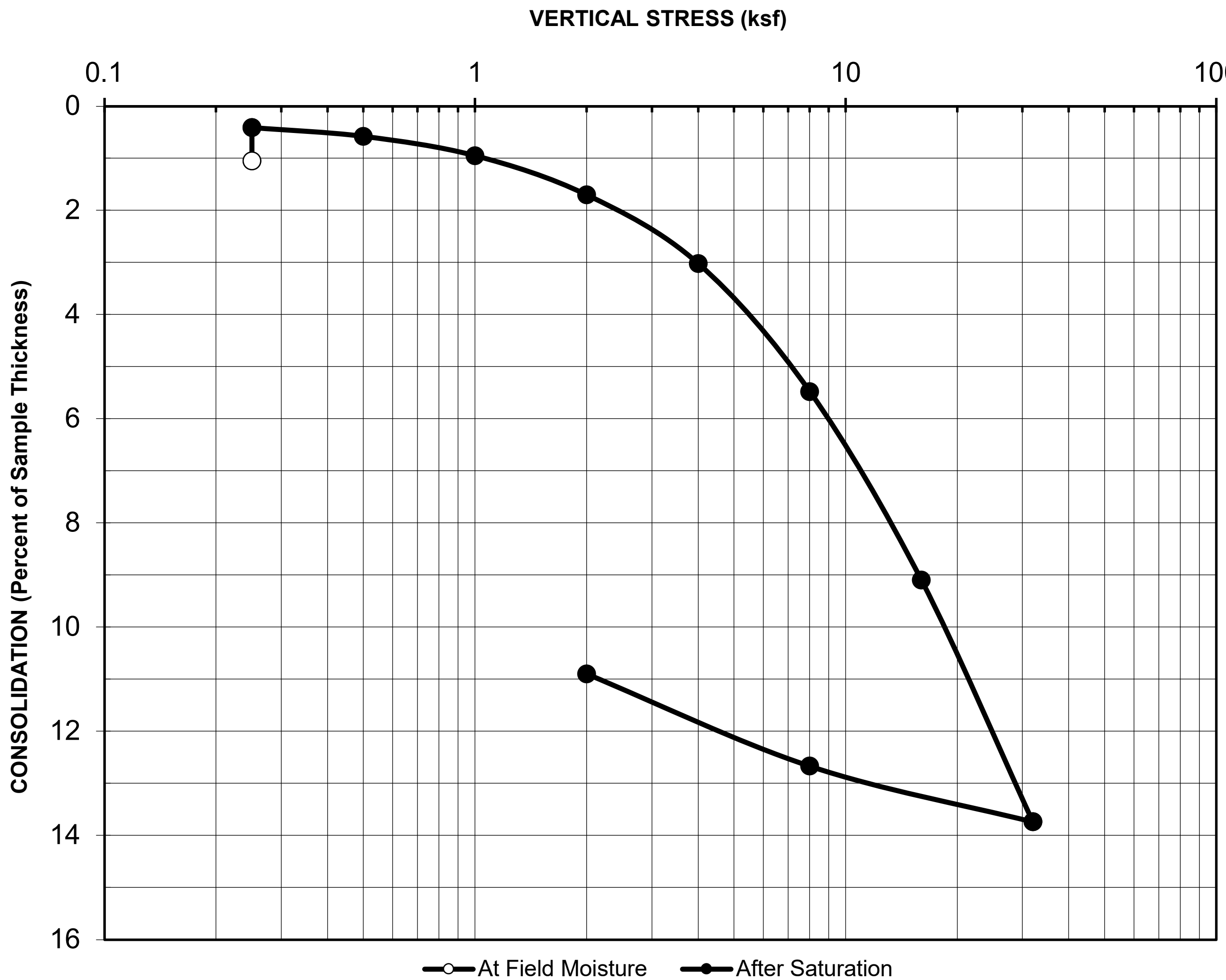
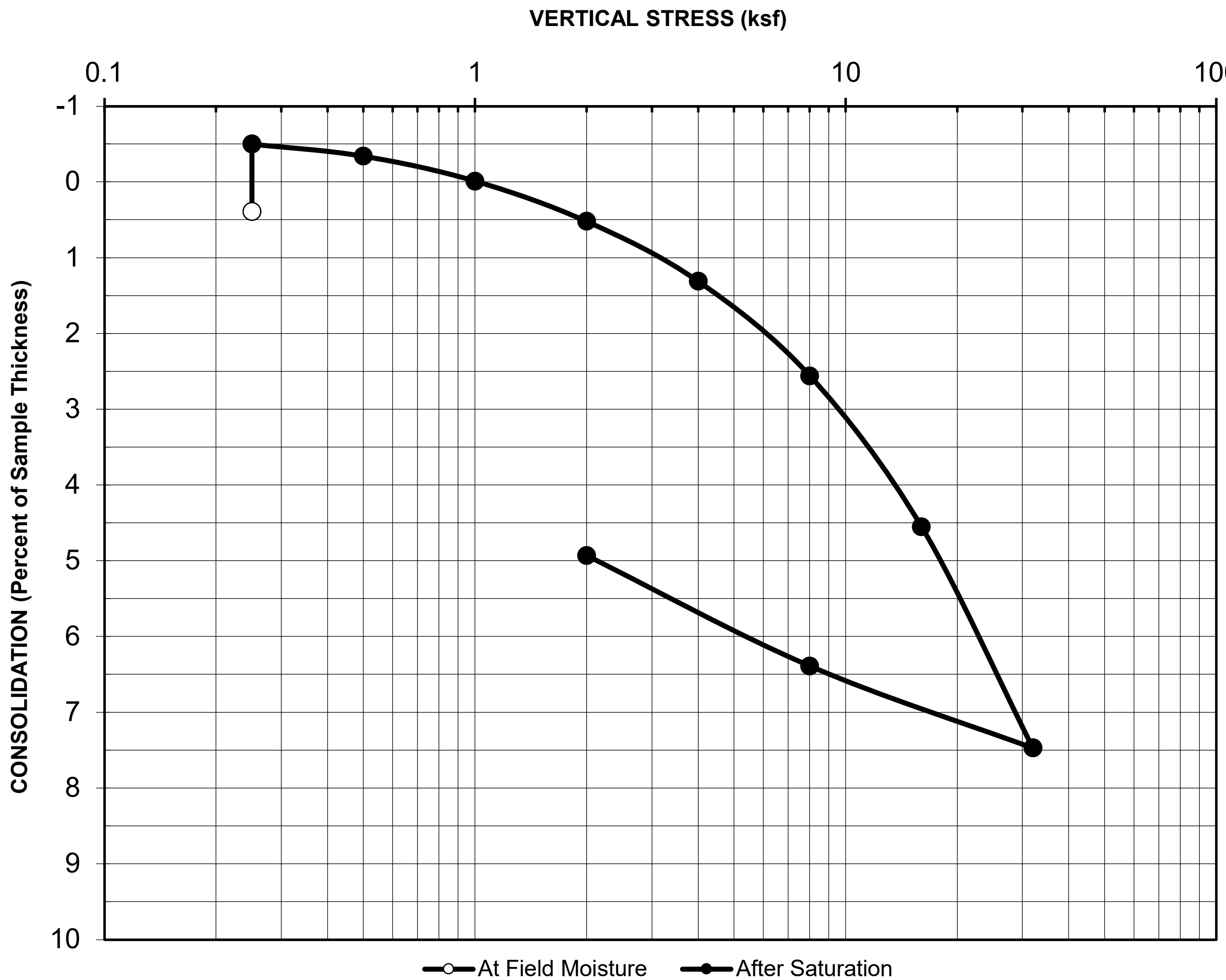


Boring No. :	GP-1	Initial Dry Unit Weight (pcf):	106.2
Sample No.:	10	Initial Moisture Content (%):	19.8
Depth (feet):	40	Final Moisture Content (%):	21.6
Sample Type:	Mod Cal	Assumed Specific Gravity:	2.7
Soil Description:	Sandy Lean Clay	Initial Void Ratio:	0.59
Remarks:	Swell= 0.64% upon inundation		

CONSOLIDATION CURVE
ASTM D 2435

Project Name: 5350 Wilshire Development
Project No.: 22010A
Date: 3/18/2022
AP No: 22-0347 **Sheet No:** 1



Boring No. :	GP-1	Initial Dry Unit Weight (pcf):	82.6
Sample No.:	22	Initial Moisture Content (%):	37.7
Depth (feet):	100	Final Moisture Content (%):	37.8
Sample Type:	Mod Cal	Assumed Specific Gravity:	2.7
Soil Description:	Claystone	Initial Void Ratio:	1.04
Remarks:	Swell= 0.89% upon inundation		

CONSOLIDATION CURVE
ASTM D 2435

Project Name: 5350 Wilshire Development
Project No.: 22010A
Date: 3/18/2022
AP No: 22-0347 **Sheet No:** 1

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MOISTURE AND DENSITY TEST RESULTS

ASTM D2216 and ASTM D7263 (Method B)

Client: GeoPentech
Project Name: 5350 Wilshire Development
Project No.: 22010A

AP Lab No.: 22-0347
Test Date: 03/22/22

Boring No.	Sample No.	Sample Depth (ft.)	Moisture Content (%)	Dry Density (pcf)
GP-2	1	5	16.1	115.7
GP-2	3	15	5.9	109.4
GP-2	5	25	27.8	95.0
GP-2	7	35	17.6	111.1
GP-2	9	45	16.1	110.7
GP-2	13	65	20.0	107.1
GP-2	15	75	21.2	103.9
GP-2	17	85	11.9	122.2
GP-2	19	95	30.4	90.2
GP-2	22	115	30.4	91.7

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PERCENT PASSING NO. 200 SIEVE

ASTM D1140

Client: GeoPentech
Project Name: 5350 Wilshire Development
Project Number: 22010A
AP Lab No.: 22-0347
Test Date: 03/23/22

Boring No.	Sample No.	Depth (ft)	Percent Fines (%)
GP-2	1	5	72.6
GP-2	2a	10	12.4
GP-2	3	15	9.3
GP-2	5	25	65.2
GP-2	7	35	35.3
GP-2	9	45	35.2
GP-2	13	65	10.7
GP-2	15	75	9.4
GP-2	19	95	98.5
GP-2	22	115	89.5

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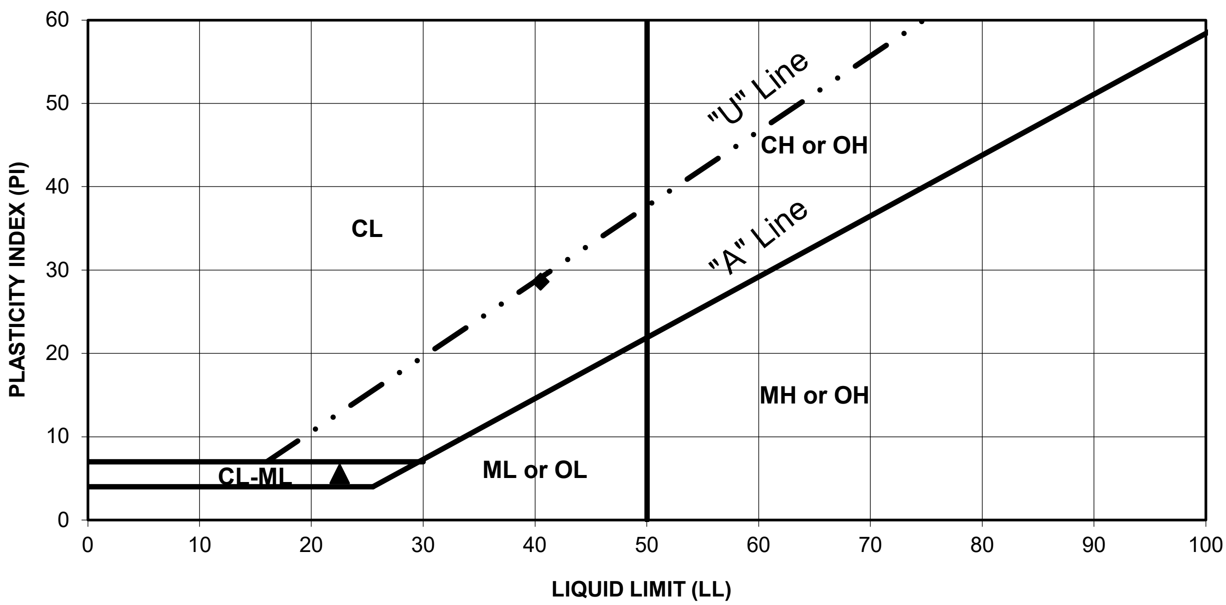
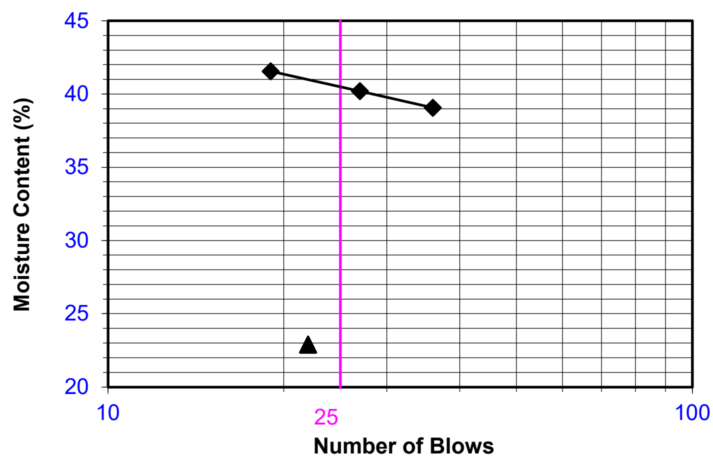
ATTERBERG LIMITS
ASTM D 4318

Client Name:	GeoPentech	Tested By:	DK	Date:	03/28/22
Project Name:	5350 Wilshire Development	Computed By:	NR	Date:	03/30/22
Project No.:	22010A	Checked By:	AP	Date:	03/30/22



PROCEDURE USED

- [ ] Wet Preparation
- [X] Dry Preparation
- [X] Procedure A
  Multipoint Test
- [X] Procedure B
  One-point Test



Symbol	Boring Number	Sample Number	Depth (feet)	LL	PL	PI	Plasticity Chart Symbol
♦	GP-2	1	5	41	12	29	CL
▲	GP-2	2a	10	23	17	6	CL-ML

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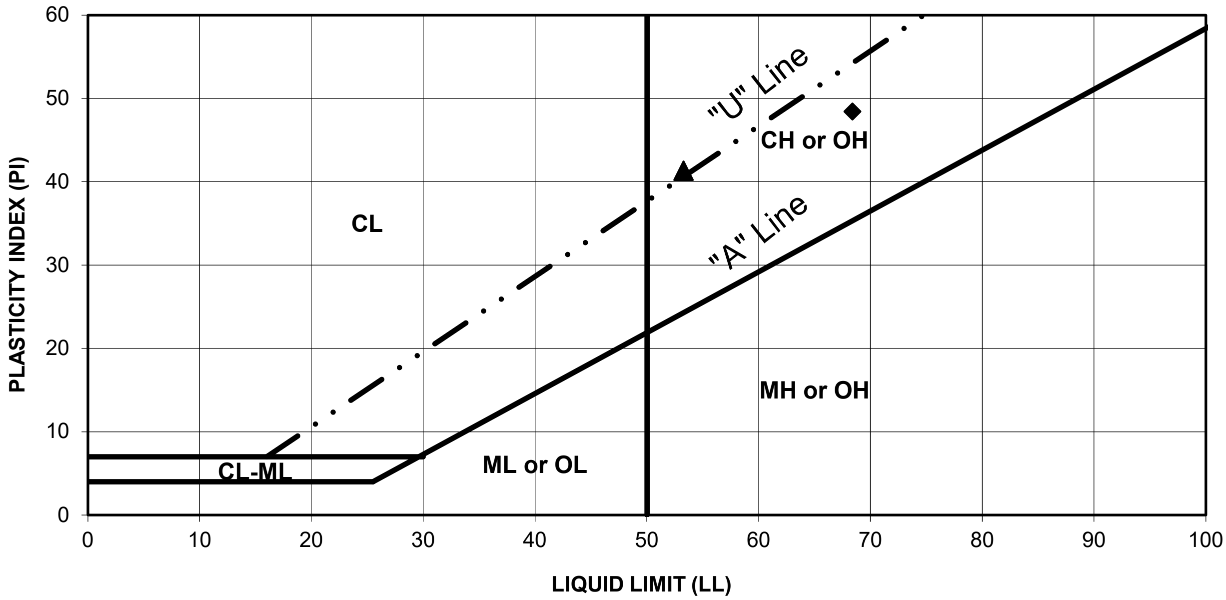
ATTERBERG LIMITS
ASTM D 4318

Client Name: GeoPentech Tested By: DK Date: 03/28/22
Project Name: 5350 Wilshire Development Computed By: NR Date: 03/30/22
Project No.: 22010A Checked By: AP Date: 03/30/22



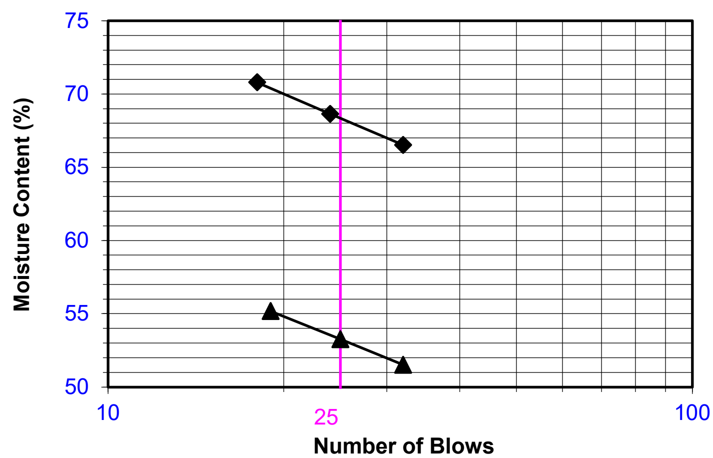
PROCEDURE USED

☐ Wet Preparation

☒ Dry Preparation

☒ Procedure A
Multipoint Test

☐ Procedure B
One-point Test



Symbol	Boring Number	Sample Number	Depth (feet)	LL	PL	PI	Plasticity Chart Symbol
♦	GP-2	5	25	68	20	48	CH
▲	GP-2	7	35	53	12	41	CH

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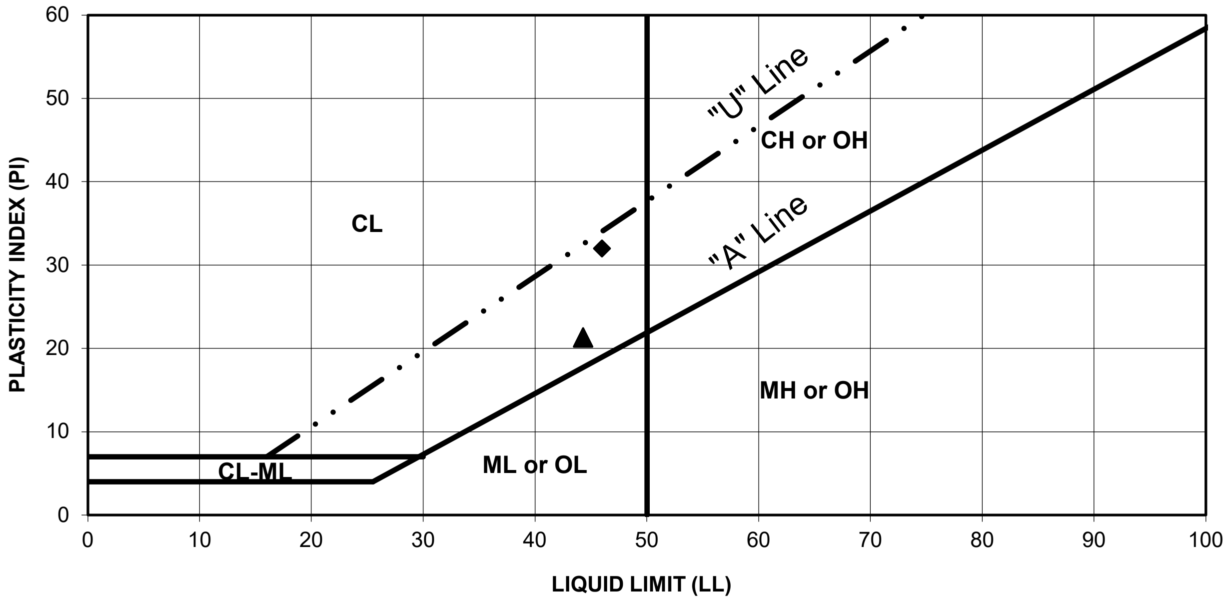
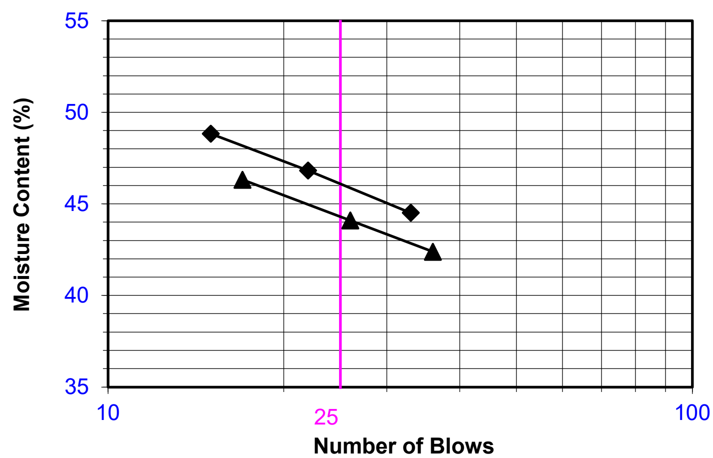
ATTERBERG LIMITS
ASTM D 4318

Client Name: GeoPentech — Tested By: DK — Date: 03/28/22
Project Name: 5350 Wilshire Development — Computed By: NR — Date: 03/30/22
Project No.: 22010A — Checked By: AP — Date: 03/30/22



PROCEDURE USED

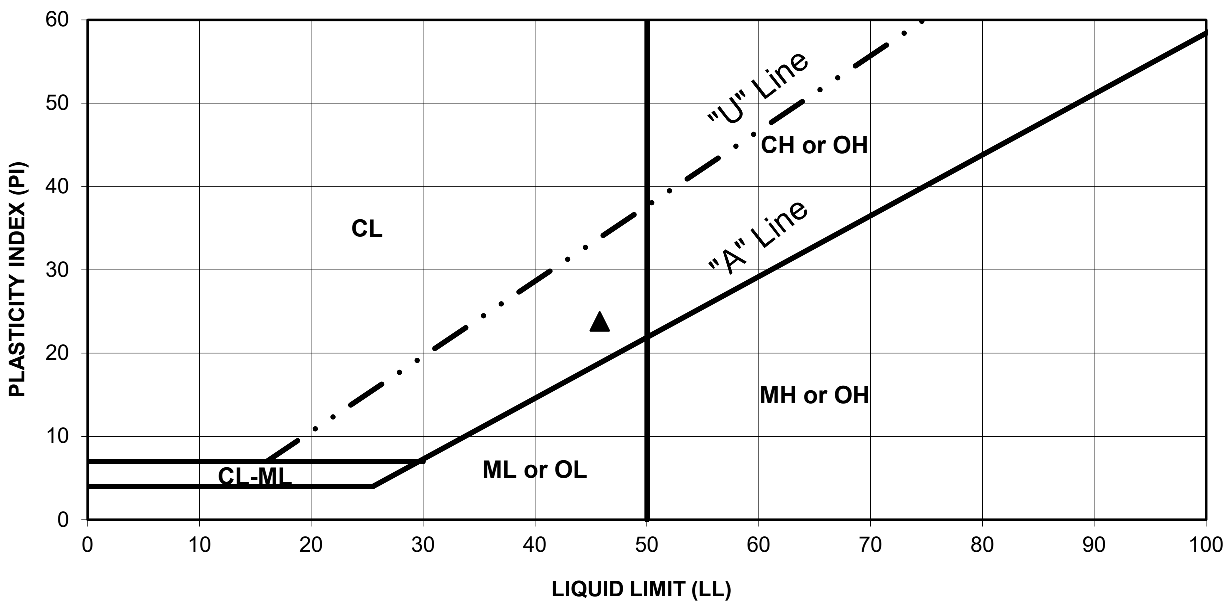
- [ ] Wet Preparation
- [X] Dry Preparation
- [X] Procedure A Multipoint Test
- [ ] Procedure B One-point Test



Symbol	Boring Number	Sample Number	Depth (feet)	LL	PL	PI	Plasticity Chart Symbol
♦	GP-2	9	45	46	14	32	CL
▲	GP-2	11	55	44	23	21	CL

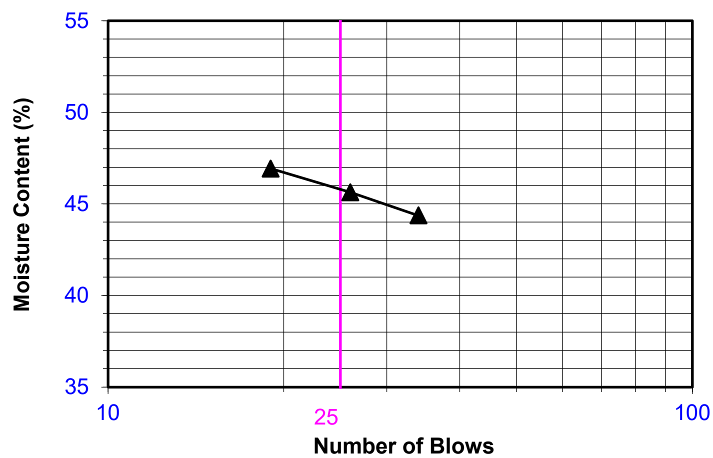
ATTERBERG LIMITS
ASTM D 4318

Client Name: GeoPentech — Tested By: DK — Date: 03/28/22
Project Name: 5350 Wilshire Development — Computed By: NR — Date: 03/30/22
Project No.: 22010A — Checked By: AP — Date: 03/30/22



PROCEDURE USED

- [ ] Wet Preparation
- [X] Dry Preparation
- [X] Procedure A Multipoint Test
- [ ] Procedure B One-point Test



Symbol	Boring Number	Sample Number	Depth (feet)	LL	PL	PI	Plasticity Chart Symbol
	GP-2	16	80	NP	NP	NP	
▲	GP-2	19	95	46	22	24	CL

* NP denotes "non-plastic"

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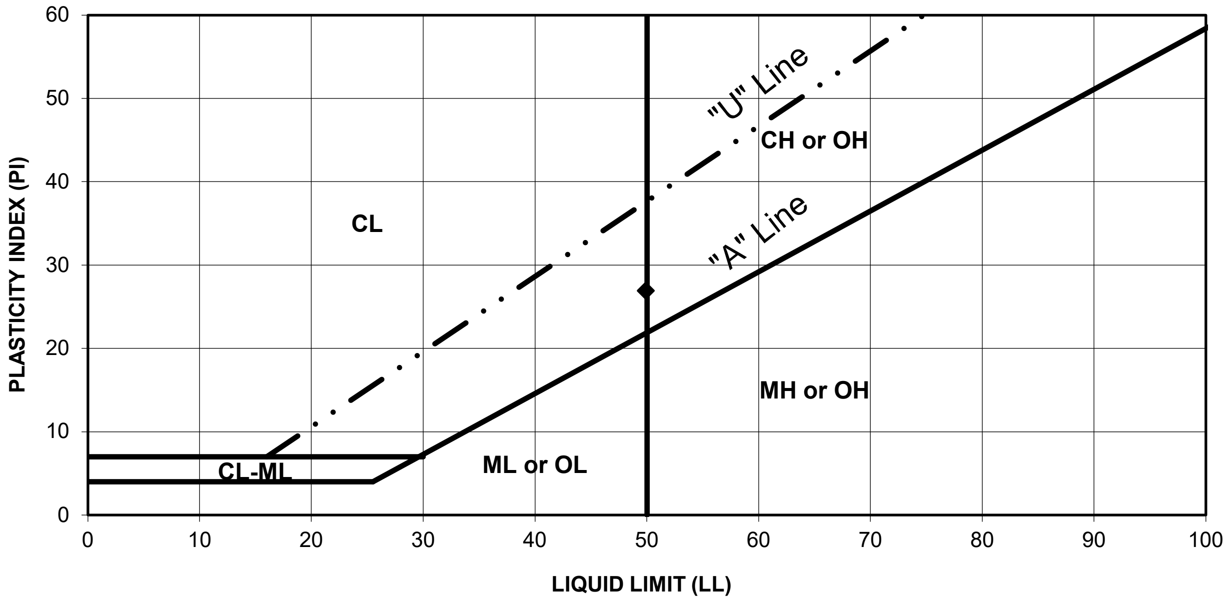
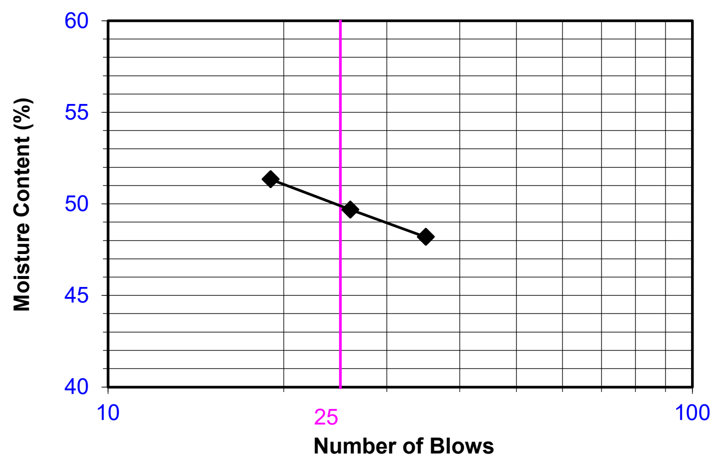
ATTERBERG LIMITS
ASTM D 4318

Client Name:	GeoPentech	Tested By:	DK	Date:	03/28/22
Project Name:	5350 Wilshire Development	Computed By:	NR	Date:	03/30/22
Project No.:	22010A	Checked By:	AP	Date:	03/30/22



PROCEDURE USED

- [ ] Wet Preparation
- [X] Dry Preparation
- [X] Procedure A Multipoint Test
- [ ] Procedure B One-point Test



Symbol	Boring Number	Sample Number	Depth (feet)	LL	PL	PI	Plasticity Chart Symbol
♦	GP-2	22	115	50	23	27	CH

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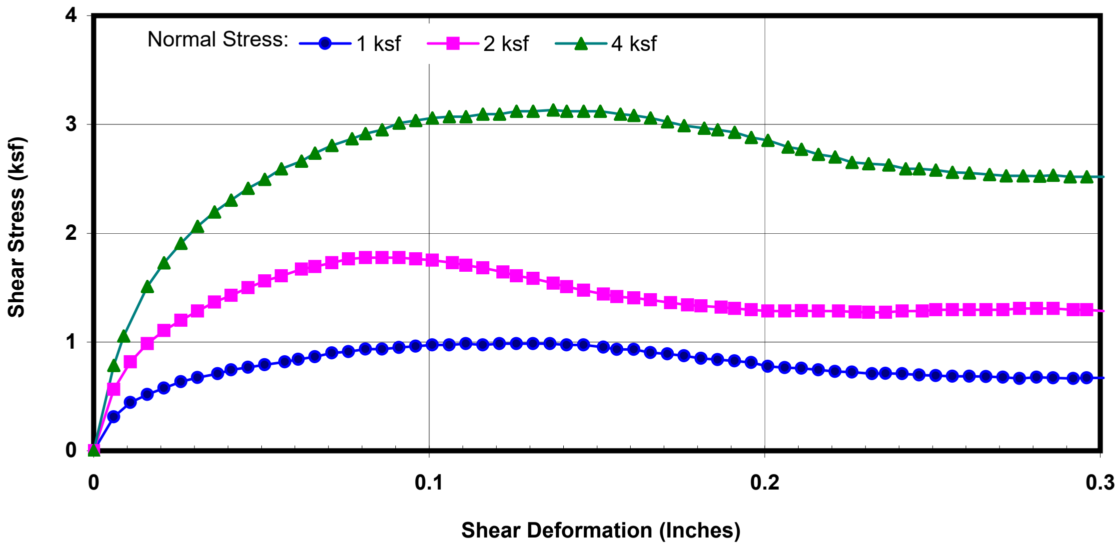
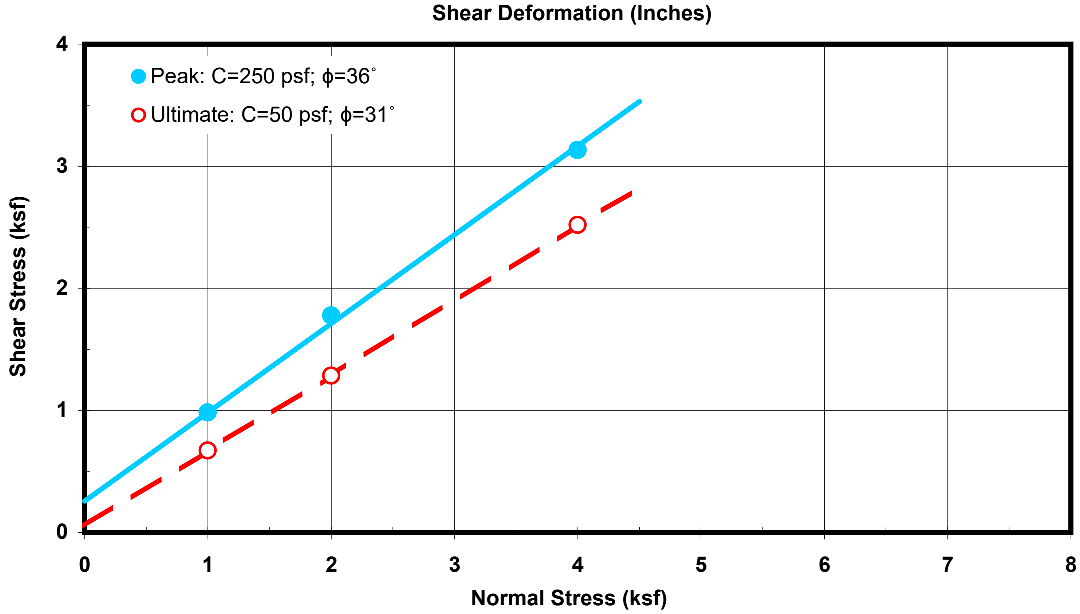
DIRECT SHEAR TEST RESULTS
ASTM D 3080

Client: GeoPentech
Project Name: 5350 Wilshire Development
Project No.: 22010A
Boring No.: GP-2
Sample No.: 3 **Depth (ft):** 15
Sample Type: Mod. Cal.
Soil Description: Sand w/silt
Test Condition: Inundated **Shear Type:** Regular

Tested By: KM **Date:** 03/24/22
Computed By: NR **Date:** 03/30/22
Checked by: AP **Date:** 03/30/22

Wet Unit Weight (pcf)	Dry Unit Weight (pcf)	Initial Moisture Content (%)	Final Moisture Content (%)	Initial Degree Saturation (%)	Final Degree Saturation (%)	Normal Stress (ksf)	Peak Shear Stress (ksf)	Ultimate Shear Stress (ksf)
114.5	108.1	5.9	18.5	28	89	1	0.984	0.670
						2	1.776	1.284
						4	3.132	2.520





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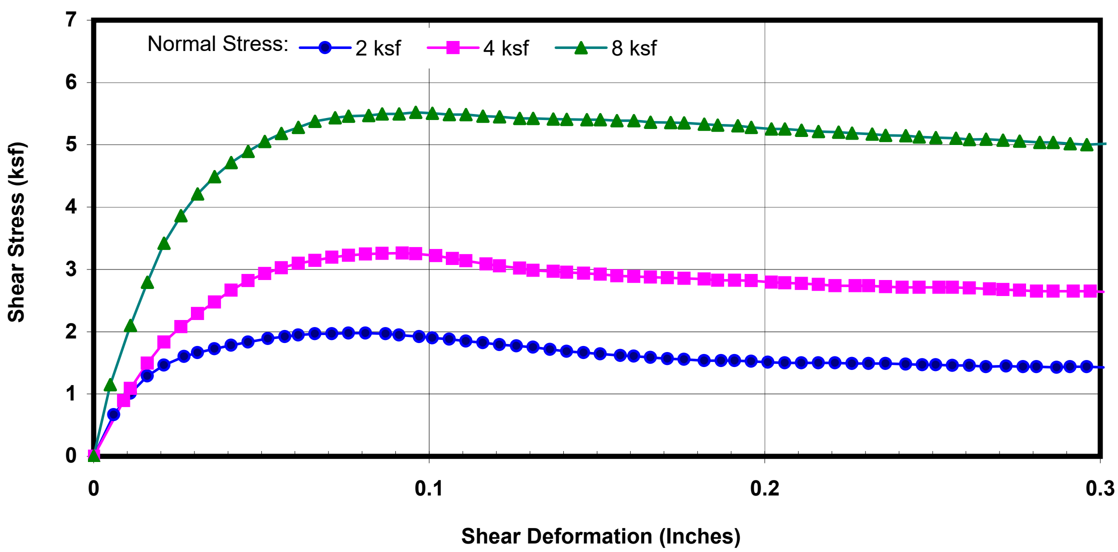
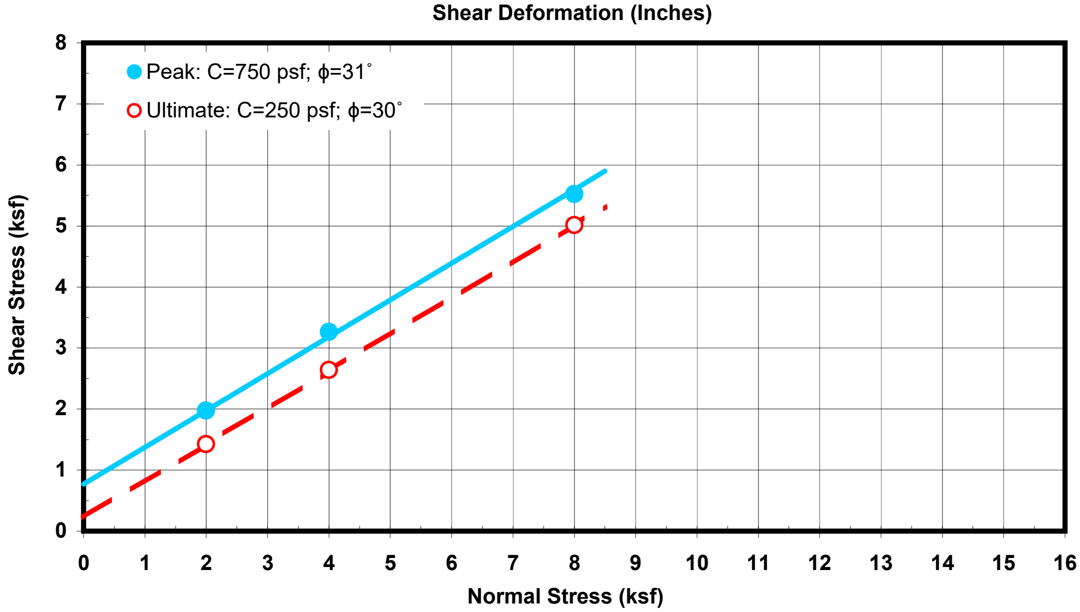
DIRECT SHEAR TEST RESULTS
ASTM D 3080

Client: GeoPentech
Project Name: 5350 Wilshire Development
Project No.: 22010A
Boring No.: GP-2
Sample No.: 9 **Depth (ft):** 45
Sample Type: Mod. Cal.
Soil Description: Clayey Sand
Test Condition: Inundated **Shear Type:** Regular

Tested By: KM **Date:** 03/24/22
Computed By: NR **Date:** 03/30/22
Checked by: AP **Date:** 03/30/22

Wet Unit Weight (pcf)	Dry Unit Weight (pcf)	Initial Moisture Content (%)	Final Moisture Content (%)	Initial Degree Saturation (%)	Final Degree Saturation (%)	Normal Stress (ksf)	Peak Shear Stress (ksf)	Ultimate Shear Stress (ksf)
126.6	109.1	16.1	20.2	80	100	2	1.976	1.426
						4	3.262	2.640
						8	5.520	5.016





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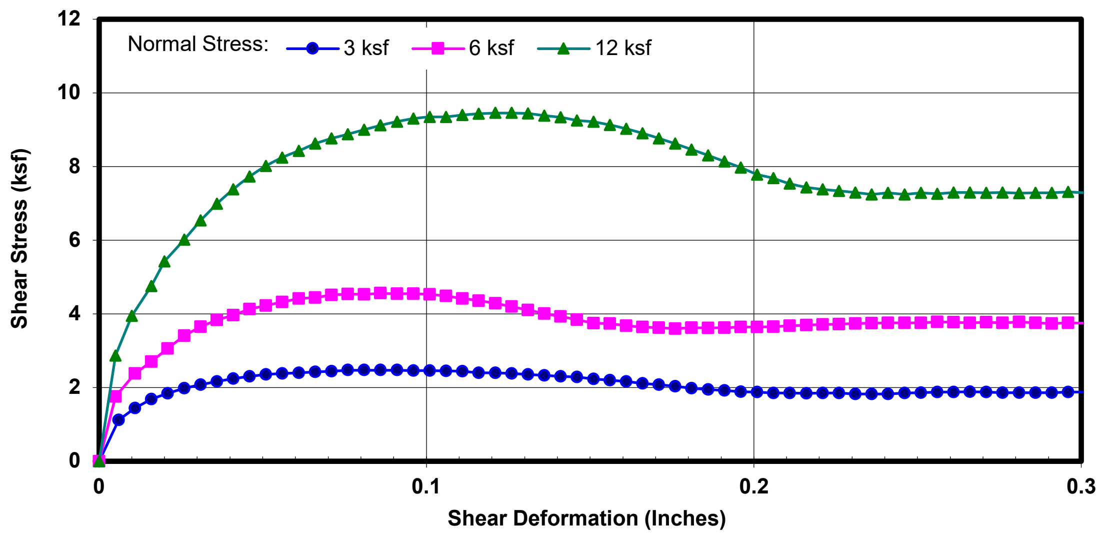
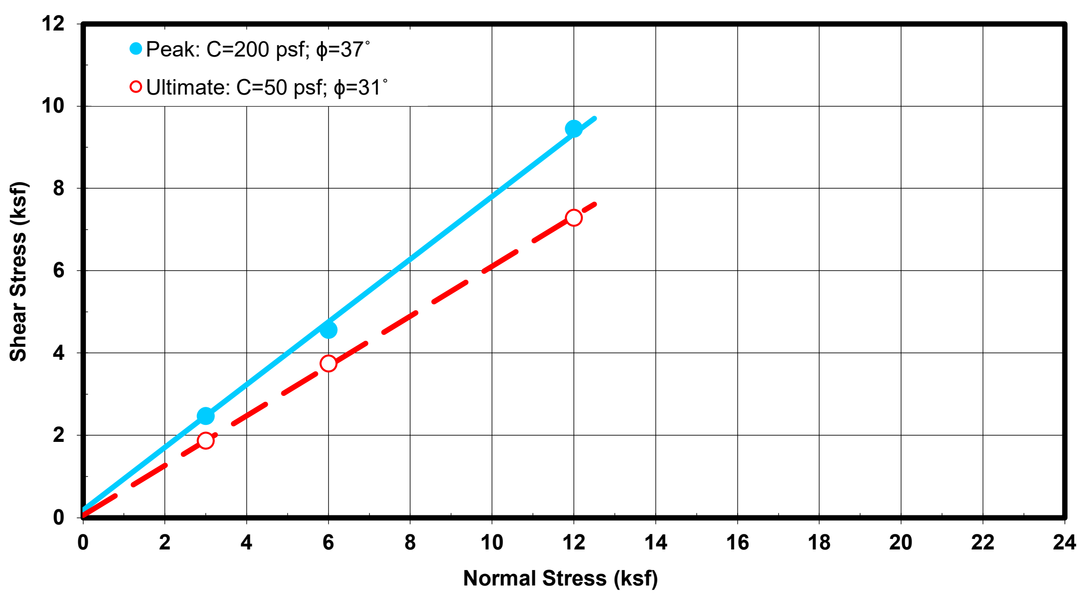
DIRECT SHEAR TEST RESULTS
ASTM D 3080

Client:	GeoPentech	**Tested By:**	KM	**Date:**	03/24/22
Project Name:	5350 Wilshire Development	**Computed By:**	NR	**Date:**	03/30/22
Project No.:	22010A	**Checked by:**	AP	**Date:**	03/30/22
Boring No.:	GP-2				
Sample No.:	13 **Depth (ft):** 65				
Sample Type:	Mod. Cal.				
Soil Description:	Sand w/silt				
Test Condition:	Inundated **Shear Type:** Regular				

Wet Unit Weight (pcf)	Dry Unit Weight (pcf)	Initial Moisture Content (%)	Final Moisture Content (%)	Initial Degree Saturation (%)	Final Degree Saturation (%)	Normal Stress (ksf)	Peak Shear Stress (ksf)	Ultimate Shear Stress (ksf)
128.6	107.2	20.0	21.2	94	100	3	2.472	1.872
						6	4.560	3.744
						12	9.455	7.284





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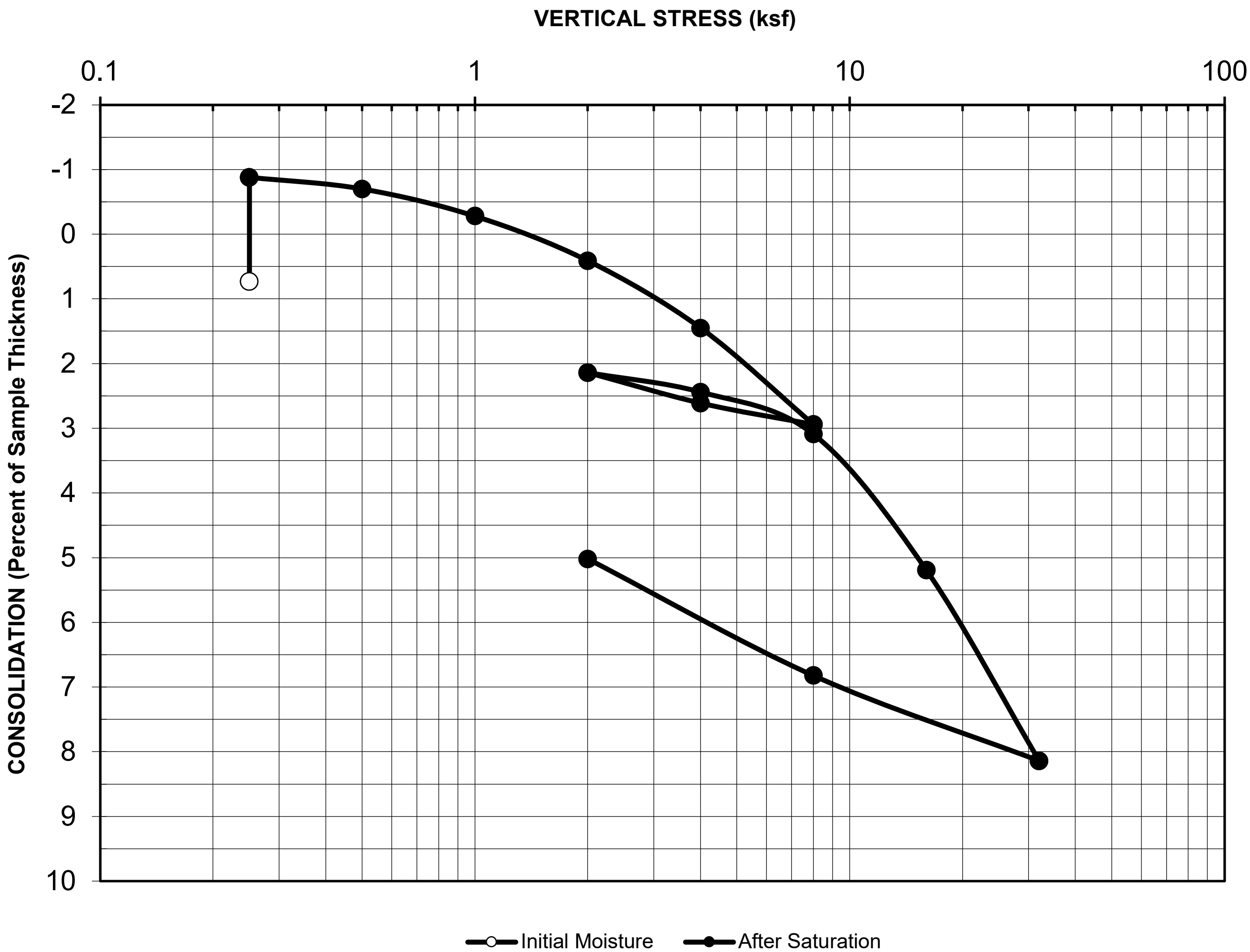

Boring No. :	GP-2	Initial Dry Unit Weight (pcf):	94.2
Sample No.:	5	Initial Moisture Content (%):	27.8
Depth (feet):	25	Final Moisture Content (%):	28.2
Sample Type:	Mod Cal	Assumed Specific Gravity:	2.9
Soil Description:	Sandy Fat Clay	Initial Void Ratio:	0.92
Remarks:	Swell= 1.61% upon inundation		

CONSOLIDATION CURVE
ASTM D 2435

Project Name: 5350 Wilshire Development
Project No.: 22010A
Date: 03/18/22
AP No: 22-0347 **Sheet No:** 1

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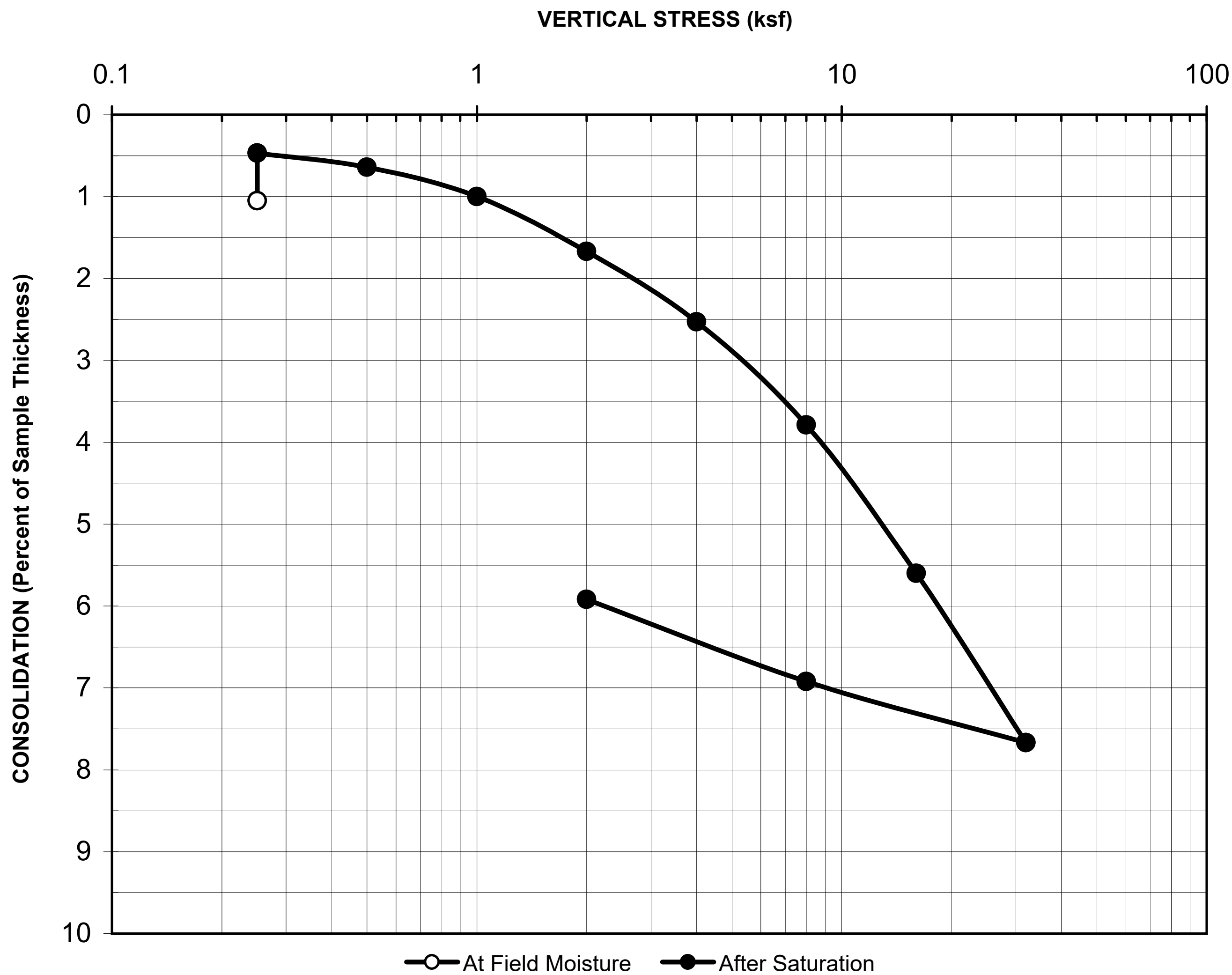

Boring No. :	GP-2	Initial Dry Unit Weight (pcf):	113.3
Sample No.:	9	Initial Moisture Content (%):	16.1
Depth (feet):	45	Final Moisture Content (%):	17.4
Sample Type:	Mod Cal	Assumed Specific Gravity:	2.7
Soil Description:	Clayey Sand	Initial Void Ratio:	0.49
Remarks:	Swell= 0.58% upon inundation		

CONSOLIDATION CURVE
ASTM D 2435

Project Name: 5350 Wilshire Development
Project No.: 22010A
Date: 3/18/2022
AP No: 22-0347 **Sheet No:** 1

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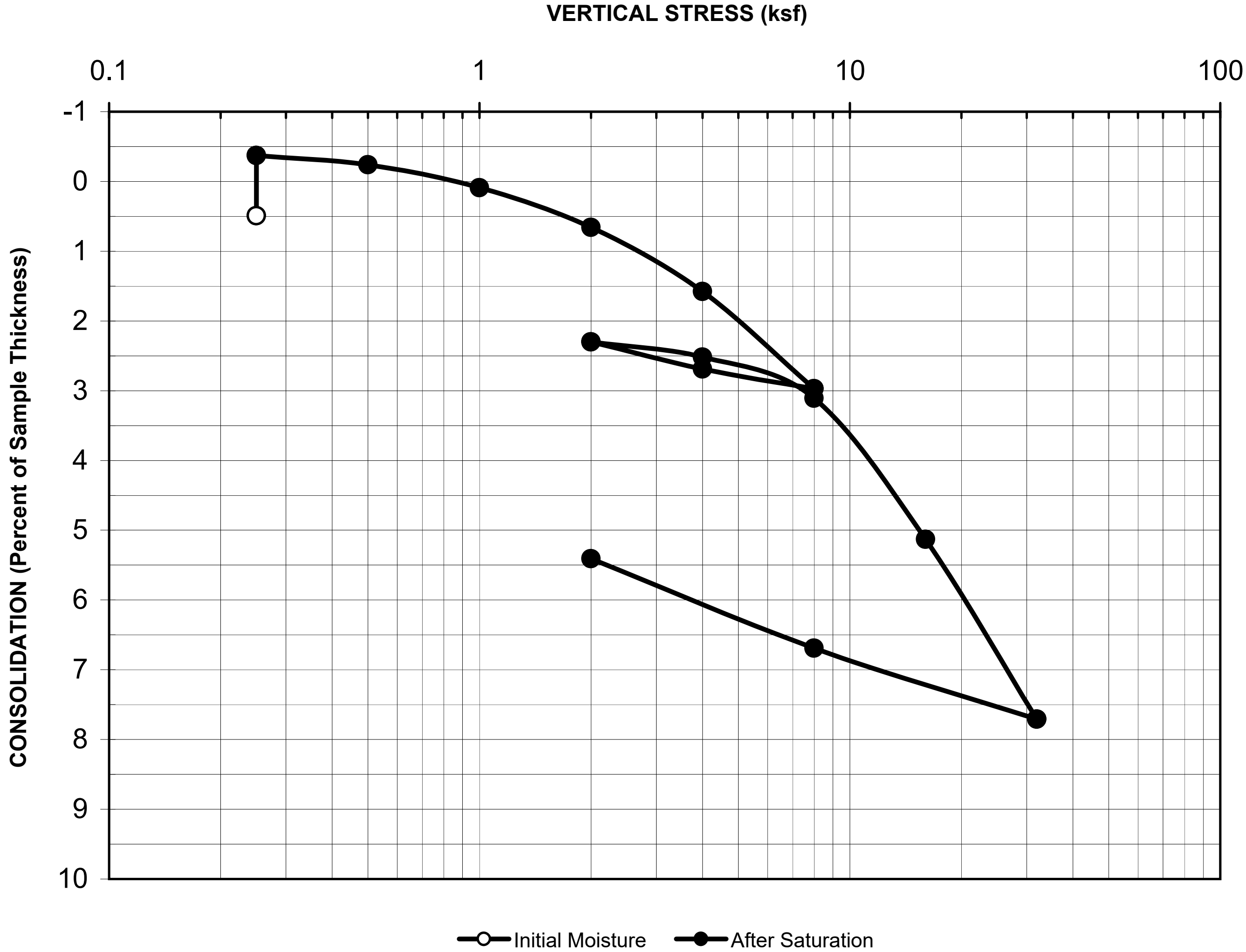

Boring No. :	GP-2	Initial Dry Unit Weight (pcf):	91.4
Sample No.:	19	Initial Moisture Content (%):	30.4
Depth (feet):	95	Final Moisture Content (%):	31.9
Sample Type:	Mod Cal	Assumed Specific Gravity:	2.9
Soil Description:	Claystone	Initial Void Ratio:	0.98
Remarks:	Swell= 0.86% upon inundation		

CONSOLIDATION CURVE
ASTM D 2435

Project Name:	5350 Wilshire Development
Project No.:	22010A
Date:	03/18/22
AP No:	22-0347 **Sheet No:** 1

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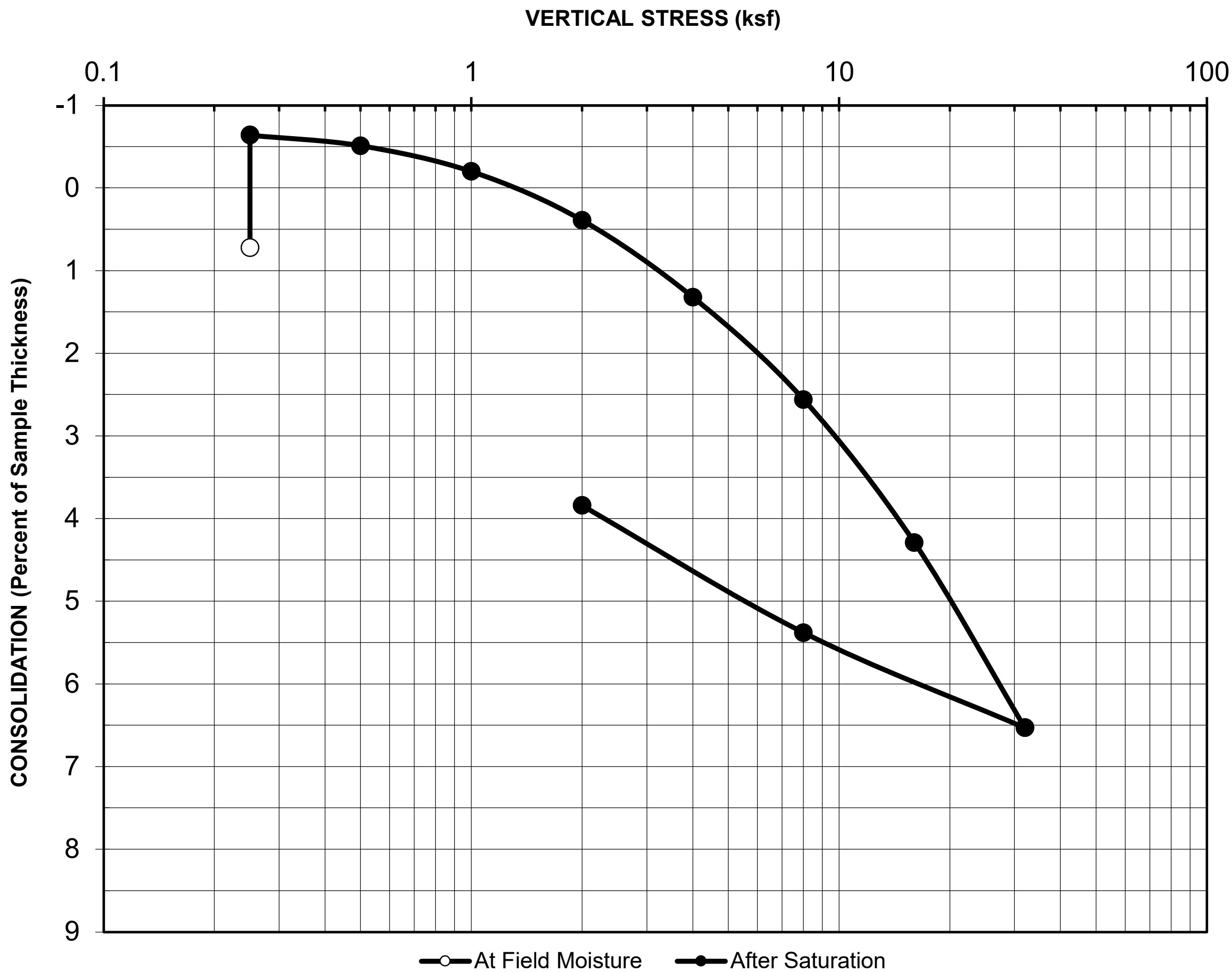

Boring No. :	GP-2	Initial Dry Unit Weight (pcf):	92.8
Sample No.:	22	Initial Moisture Content (%):	30.4
Depth (feet):	115	Final Moisture Content (%):	30.5
Sample Type:	Mod Cal	Assumed Specific Gravity:	2.7
Soil Description:	Claystone	Initial Void Ratio:	0.81
Remarks:	Swell= 1.36% upon inundation		

CONSOLIDATION CURVE
ASTM D 2435

Project Name: 5350 Wilshire Development
Project No.: 22010A
Date: 3/18/2022
AP No: 22-0347 **Sheet No:** 1

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MOISTURE AND DENSITY TEST RESULTS

ASTM D2216 and ASTM D7263 (Method B)

Client: GeoPentech
Project Name: 5350 Wilshire Development
Project No.: 22010A

AP Lab No.: 22-0347
Test Date: 03/22/22

Boring No.	Sample No.	Sample Depth (ft.)	Moisture Content (%)	Dry Density (pcf)
GP-3	3	15	9.4	111.2
GP-3	4	20	22.1	NA
GP-3	5	25	25.2	99.0
GP-3	7	35	18.1	102.1
GP-3	10	50	22.2	NA
GP-3	11	60	17.8	112.3
GP-3	14	90	46.3	NA

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PERCENT PASSING NO. 200 SIEVE

ASTM D1140

Client: GeoPentech
AP Lab No.: 22-0347
Project Name: 5350 Wilshire Development
Test Date: 03/23/22
Project Number: 22010A

Boring No.	Sample No.	Depth (ft)	Percent Fines (%)
GP-3	4	20	73.5
GP-3	7	35	42.3
GP-3	8	40	67.9
GP-3	10	50	57.8
GP-3	11	60	12.2
GP-3	13	80	15.2
GP-3	14	90	70.4

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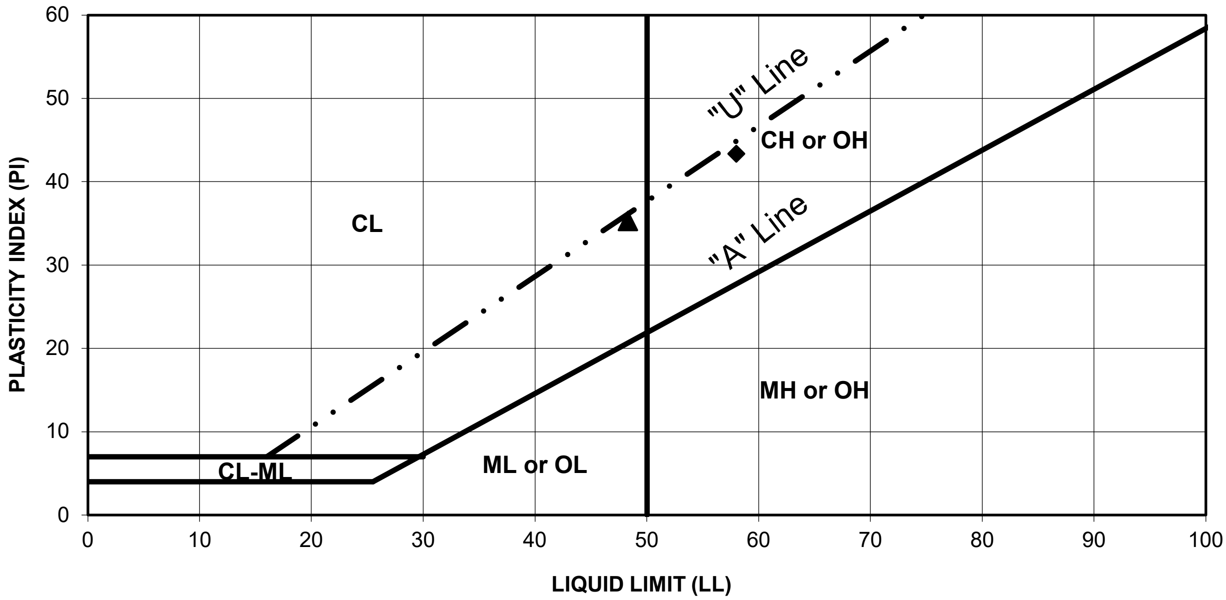
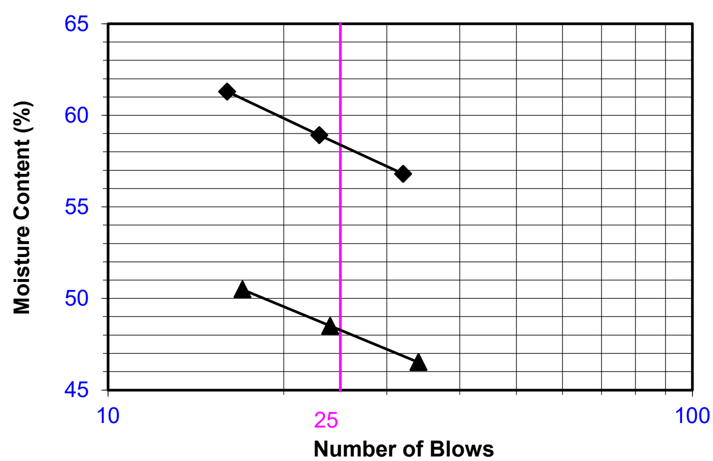
ATTERBERG LIMITS
ASTM D 4318

Client Name: GeoPentech — Tested By: DK — Date: 03/29/22
Project Name: 5350 Wilshire Development — Computed By: NR — Date: 03/30/22
Project No.: 22010A — Checked By: AP — Date: 03/30/22



PROCEDURE USED

- [ ] Wet Preparation
- [X] Dry Preparation
- [X] Procedure A Multipoint Test
- [ ] Procedure B One-point Test



Symbol	Boring Number	Sample Number	Depth (feet)	LL	PL	PI	Plasticity Chart Symbol
♦	GP-3	4	20	58	15	43	CH
▲	GP-3	7	35	48	13	35	CL

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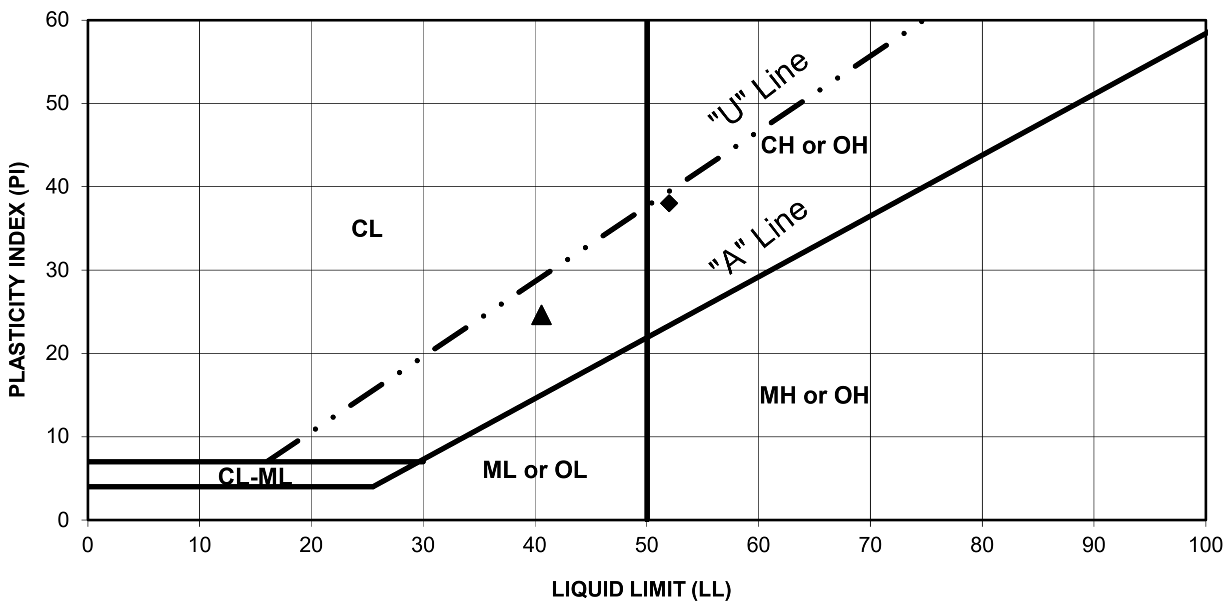
ATTERBERG LIMITS
ASTM D 4318

Client Name: GeoPentech Tested By: DK Date: 03/29/22
Project Name: 5350 Wilshire Development Computed By: NR Date: 03/30/22
Project No.: 22010A Checked By: AP Date: 03/30/22



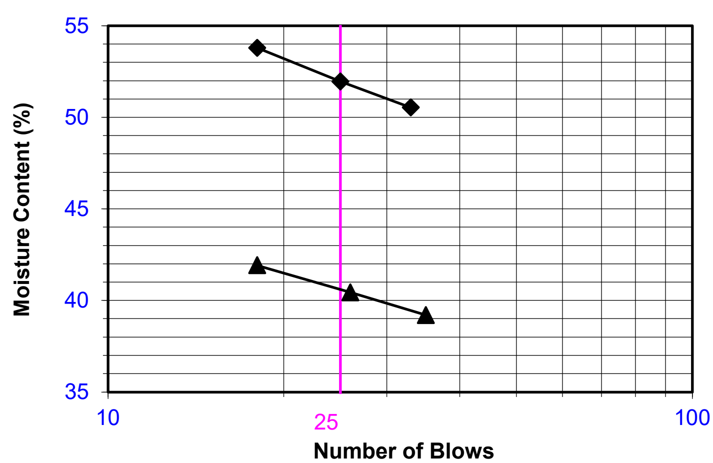
PROCEDURE USED

☐ Wet Preparation

☒ Dry Preparation

☒ Procedure A
Multipoint Test

☐ Procedure B
One-point Test



Symbol	Boring Number	Sample Number	Depth (feet)	LL	PL	PI	Plasticity Chart Symbol
♦	GP-3	8	40	52	14	38	CH
▲	GP-3	10	50	41	16	25	CL

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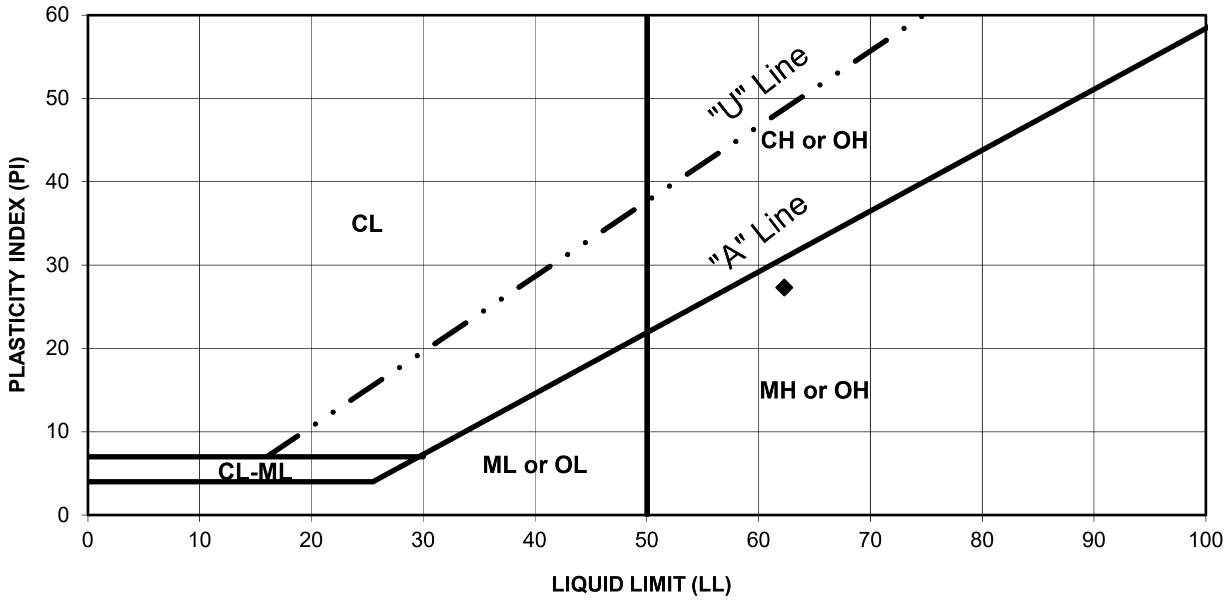
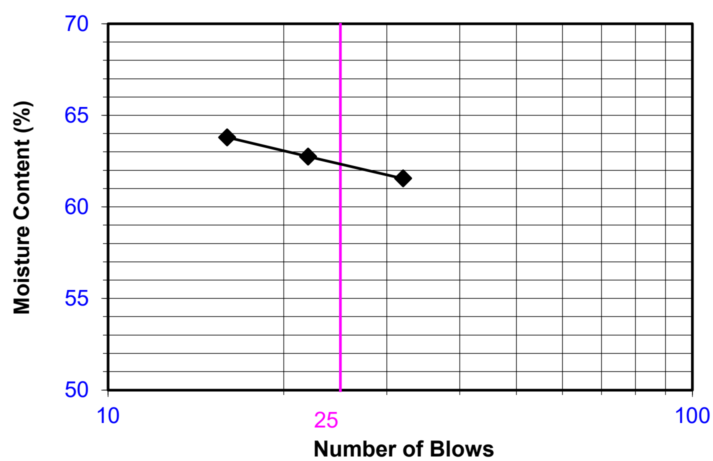
ATTERBERG LIMITS
ASTM D 4318

Client Name:	GeoPentech	Tested By:	DK	Date:	03/29/22
Project Name:	5350 Wilshire Development	Computed By:	NR	Date:	03/30/22
Project No.:	22010A	Checked By:	AP	Date:	03/30/22



PROCEDURE USED

- [ ] Wet Preparation
- [x] Dry Preparation
- [x] Procedure A Multipoint Test
- [ ] Procedure B One-point Test



Symbol	Boring Number	Sample Number	Depth (feet)	LL	PL	PI	Plasticity Chart Symbol
♦	GP-3	14	90	62	35	27	MH

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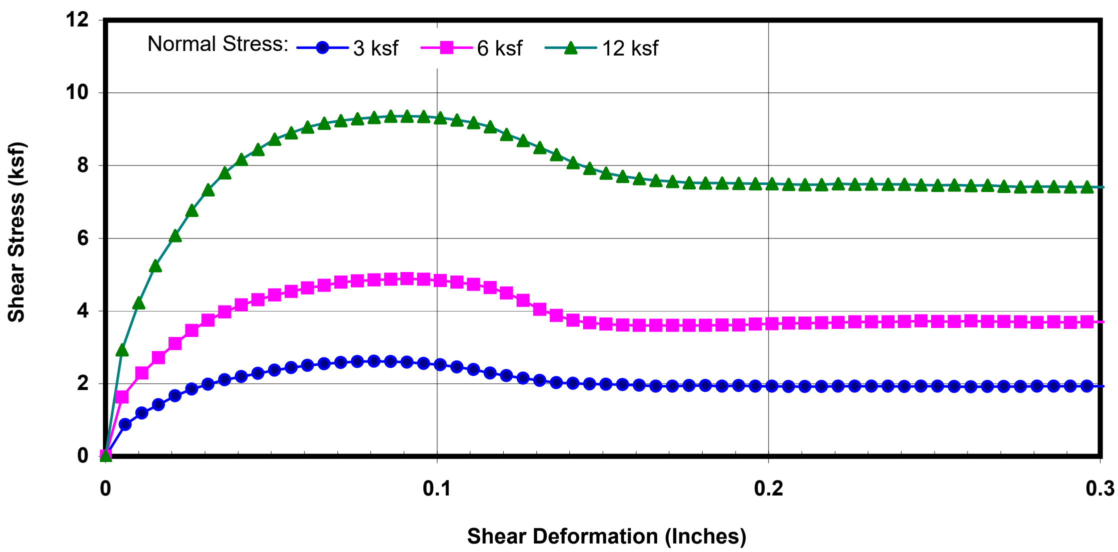
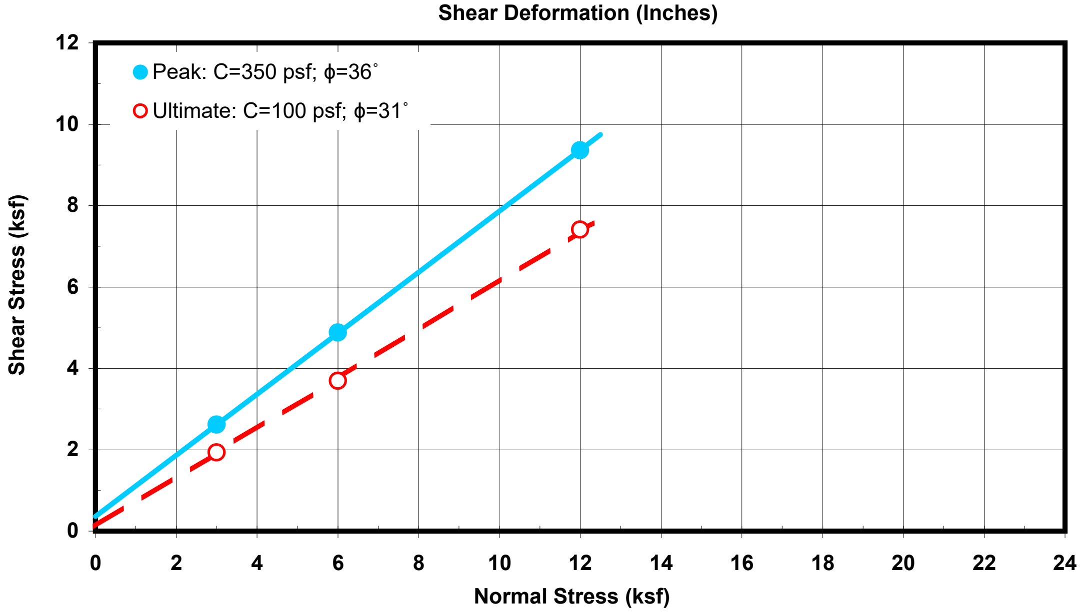
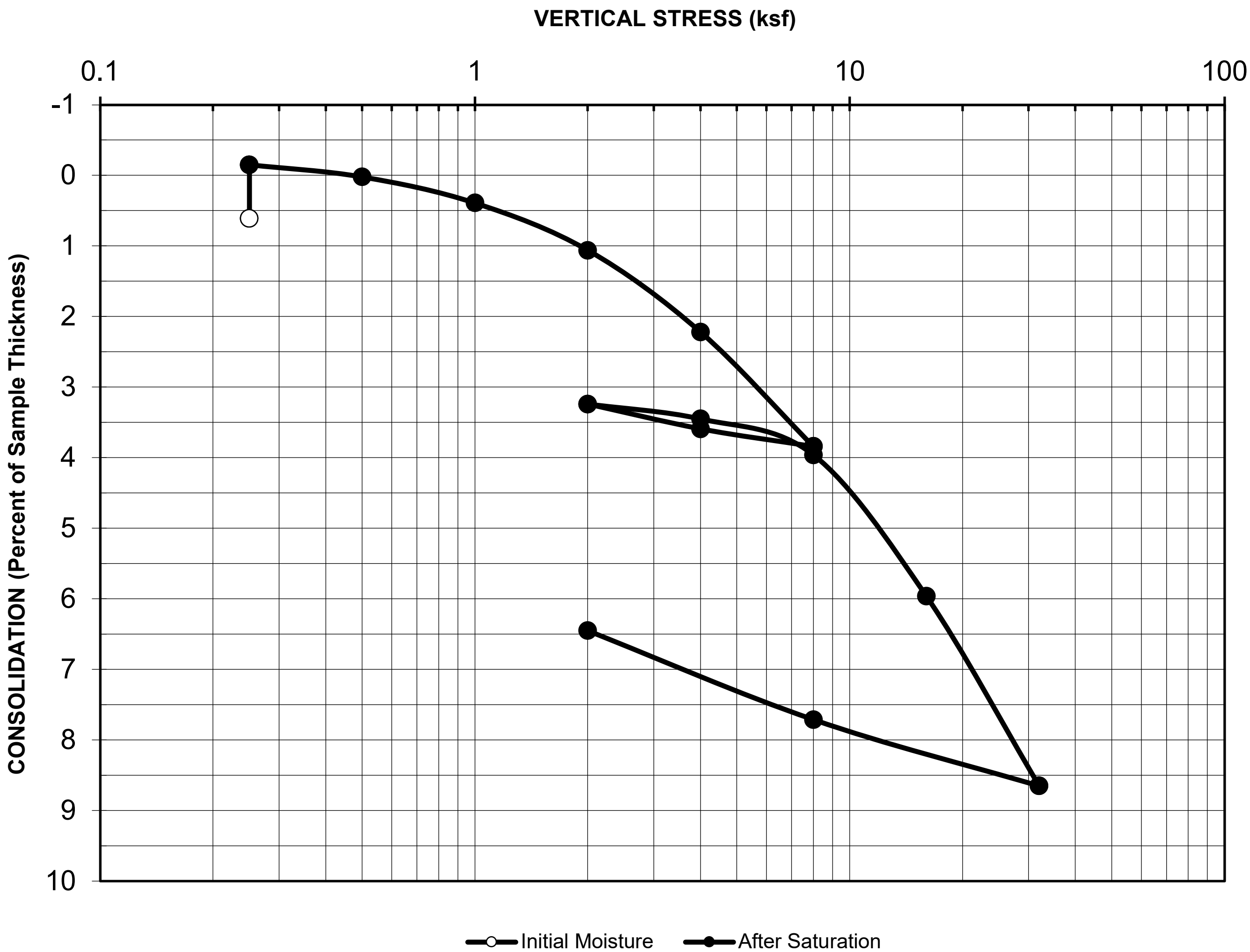
DIRECT SHEAR TEST RESULTS
ASTM D 3080

Client:	GeoPentech	**Tested By:**	KM	**Date:**	03/24/22
Project Name:	5350 Wilshire Development	**Computed By:**	NR	**Date:**	03/30/22
Project No.:	22010A	**Checked by:**	AP	**Date:**	03/30/22
Boring No.:	GP-3				
Sample No.:	13 **Depth (ft):** 80				
Sample Type:	Mod. Cal.				
Soil Description:	Silty Sand w/traces of clay				
Test Condition:	Inundated **Shear Type:** Regular				

Wet Unit Weight (pcf)	Dry Unit Weight (pcf)	Initial Moisture Content (%)	Final Moisture Content (%)	Initial Degree Saturation (%)	Final Degree Saturation (%)	Normal Stress (ksf)	Peak Shear Stress (ksf)	Ultimate Shear Stress (ksf)
125.1	99.9	25.2	25.4	99	100	3	2.616	1.932
						6	4.884	3.696
						12	9.359	7.408







Boring No. :	GP-3	Initial Dry Unit Weight (pcf):	106.9
Sample No.:	7	Initial Moisture Content (%):	18.1
Depth (feet):	35	Final Moisture Content (%):	20.1
Sample Type:	Mod Cal	Assumed Specific Gravity:	2.9
Soil Description:	Clayey Sand w/gravel	Initial Void Ratio:	0.69
Remarks:	Swell= 0.76% upon inundation		

CONSOLIDATION CURVE
ASTM D 2435

Project Name:	5350 Wilshire Development
Project No.:	22010A
Date:	03/18/22
AP No:	22-0347
Sheet No:	1

APPENDIX C

CONE PENETRATION TESTING

C.1 CONE PENETRATION TESTING

The Cone Penetration Testing (CPT) was performed by GeoPentech on March 1st, 2022. The explorations consisted of advancing Three (3) CPTs: CPT-1 to a depth of approximately 70.9 ft, CPT-2 to approximately 69.0 ft, and CPT-3 to approximately 36.3 ft below the ground surface. All three CPTs encountered groundwater at approximately 36 ft below the ground surface. The approximate locations of the CPTs are indicated on Figure 2 in the main report. The work was subcontracted to ConeTec, who provided all equipment, crew, and supplies.

The following pages contain ConeTec's report and data files.

PRESENTATION OF SITE INVESTIGATION RESULTS

5350 Wilshire Development

Prepared for:

GeoPentech

ConeTec Job No: 22-56-23760

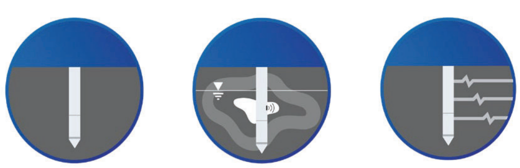
Project Start Date: 2022-Mar-01
Project End Date: 2022-Mar-01
Report Date: 2022-Mar-11

Prepared by:

ConeTec Inc.

820 Aladdin Avenue, San Leandro, CA 95477
Tel: (510) 357-3677

ConeTecCA@conetec.com
www.conetec.com
www.conetecdataservices.com



ABOUT THIS REPORT

The enclosed report presents the results of the site investigation program conducted by ConeTec, Inc. The program consisted of Piezocone Penetration Testing and Pore Pressure Dissipation Testing. Please note that this report, which also includes all accompanying data, are subject to the 3rd Party Disclaimer and Client Disclaimer that follow in the 'Limitations' section of this report.

Project Information	
Client	GeoPentech
Project	5350 Wilshire Development
ConeTec Project Number	22-56-23760
Rig Description	30-ton Truck CPT Rig (C-22)

Coordinates	
Collection Method	Consumer Grade GPS
EPSG Number	32610 (WGS 84 / UTM 10S)

Cone Penetration Test (CPTu)	
Depth Reference	Existing ground surface at the time of the investigation
Sleeve data offset	0.1 Meters

Calculated Geotechnical Parameters Tables	
Additional Information	The Normalized Soil Behaviour Type Chart based on Q_{tn} (SBT Q_{tn}) (Robertson, 2009) was used to classify the soil for this project. A detailed set of calculated CPTu parameters have been generated and are provided in Excel format files in the release folder. The CPTu parameter calculations are based on values of corrected tip resistance (q_t) sleeve friction (f_s) and pore pressure (u_2). Effective stresses are calculated based on unit weights that have been assigned to the individual soil behaviour type zones and the assumed equilibrium pore pressure profile. Soils were classified as either drained or undrained based on the Q_{tn} Normalized Soil Behaviour Type Chart (Robertson, 2009). Calculations for both drained and undrained parameters were included for materials that classified as silt mixtures (zone 4).

Please refer to the list of attached documents following the text of this report. A test summary, location map, and plots are included. Thank you for the opportunity to work on this project.

LIMITATIONS

3rd Party Disclaimer

- The “Report” refers to this report titled 5350 Wilshire Development
- The Report was prepared by ConeTec for GeoPentech

The Report is confidential and may not be distributed to or relied upon by any third parties without the express written consent of ConeTec. Any third parties gaining access to the Report do not acquire any rights as a result of such access. Any use which a third party makes of the Report, or any reliance on or decisions made based on it, are the responsibility of such third parties. ConeTec accepts no responsibility for loss, damage and/or expense, if any, suffered by any third parties as a result of decisions made, or actions taken or not taken, which are in any way based on, or related to, the Report or any portion(s) thereof.

Client Disclaimer

- ConeTec was retained by GeoPentech
- The “Report” refers to this report titled 5350 Wilshire Development
- ConeTec was retained to collect and provide the raw data (“Data”) which is included in the Report.

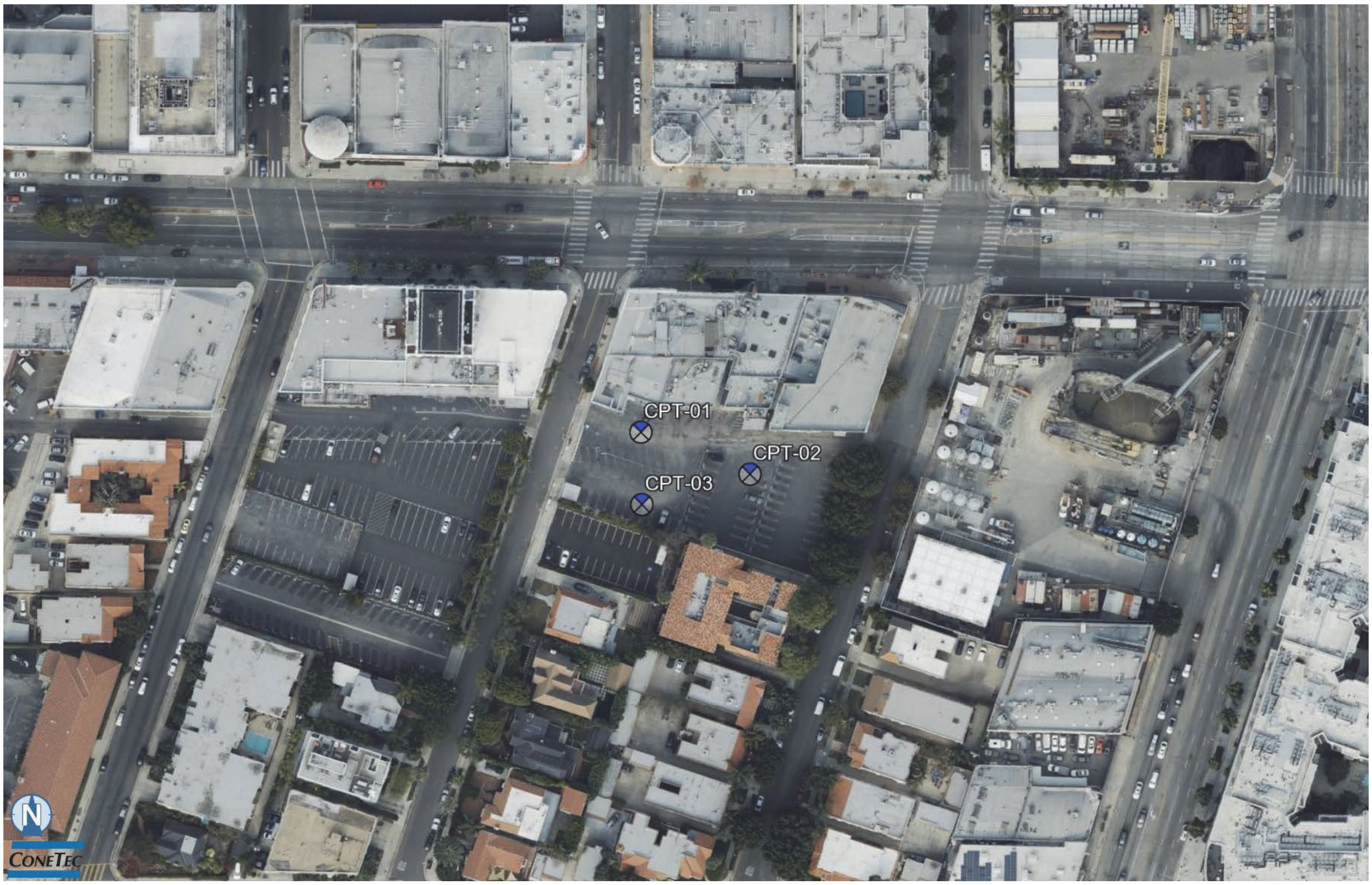
ConeTec has collected and reported the Data in accordance with current industry standards. No other warranty, express or implied, with respect to the Data is made by ConeTec. In order to properly understand the Data included in the Report, reference must be made to the documents accompanying and other sources referenced in the Report in their entirety. Other than the Data, the contents of the Report (including any Interpretations) should not be relied upon in any fashion without independent verification and ConeTec is in no way responsible for any loss, damage or expense resulting from the use of, and/or reliance on, such material by any party.

CONTENTS

The following listed below are included in the report:

- **Site Map**
- **Piezocone Penetration Test (CPTu) Sounding Summary**
- **CPTu Standard Plots, Advanced Plots, and Normalized Plots**
- **SBT Zone Scatter Plots**
- **Pore Pressure Dissipation (PPD) Test Summary**
- **PPD Test Plots**
- **Methodology Statements**
- **Data File Formats**
- **Description of Methods for Calculated CPT Geotechnical Parameters**

SITE MAP



ConeTec Job Number: 22-56-23760

Client: GeoPentech

Project: 5350 Wilshire Development

Report Date: 2022-Mar-11

Sounding Location

All sounding locations are approximate

Cone Penetration Test Summary and Standard Cone Penetration Test Plots

ConeTec

Job No:	22-56-23760
Client:	GeoPentech
Project:	5350 Wilshire Development
Start Date:	01-Mar-2022
End Date:	01-Mar-2022

CONE PENETRATION TEST SUMMARY

Sounding ID	File Name	Date	Cone	Cone Area (cm^2)	Assumed Phreatic Surface[1] (ft)	Final Depth (ft)	Northing[2]	Easting[2]	Elevation[3] (ft)	Refer to Notation Number
CPT-01	22-56-23760_CP01	01-Mar-2022	EC817:T1500F15U35	15	36.0	70.87	3769822	375775	194	4
CPT-02	22-56-23760_CP02	01-Mar-2022	EC817:T1500F15U35	15	36.0	68.98	3769810	375805	191	4
CPT-03	22-56-23760_CP03	01-Mar-2022	EC817:T1500F15U35	15	36.0	36.33	3769802	375775	196	4

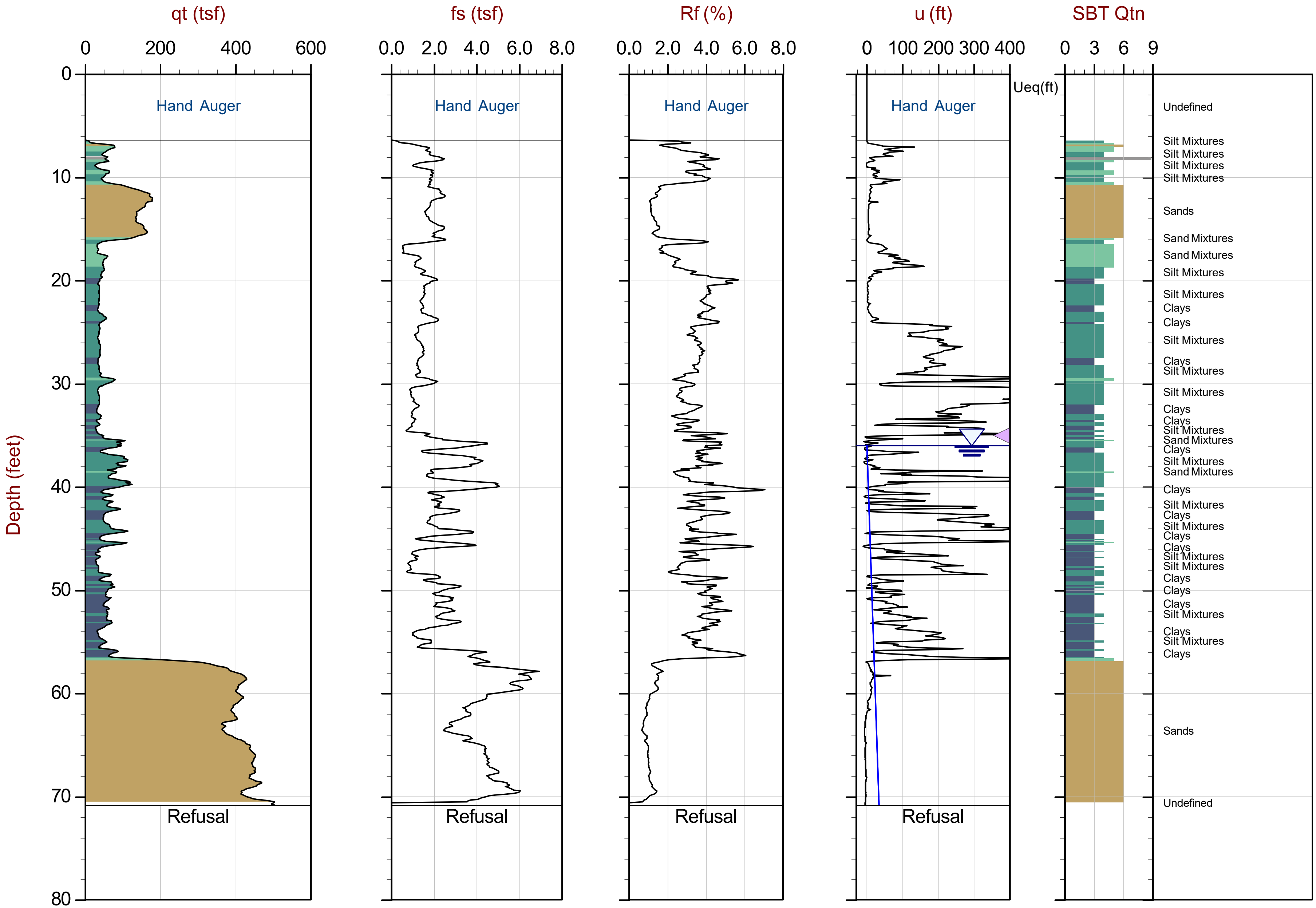
1. The assumed phreatic surface was provided by the client. Hydrostatic conditions were assumed for the calculated parameters.
2. The coordinates were collected using consumer grade GPS equipment. EPSG number: 32610 (WGS84 / UTM Zone 11S).
3. Elevations are referenced to the ground surface and were acquired from the Google Earth Elevation for the recorded coordinates.
4. The assumed phreatic surface was provided by the client based on drill borings performed at the site.

CONETEC

GeoPentech

Job No: 22-56-23760
Date: 2022-03-01 09:58
Site: 5350 Wilshire Development

Sounding: CPT-01
Cone: 817:T1500F15U35



Max Depth: 21.600 m / 70.87 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 22-56-23760_CP01.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 11S N: 3769822m E: 375775m

Equilibrium Pore Pressure (Ueq) | Assumed Ueq | Dissipation, Ueq achieved | Dissipation, Ueq not achieved | Hydrostatic Line

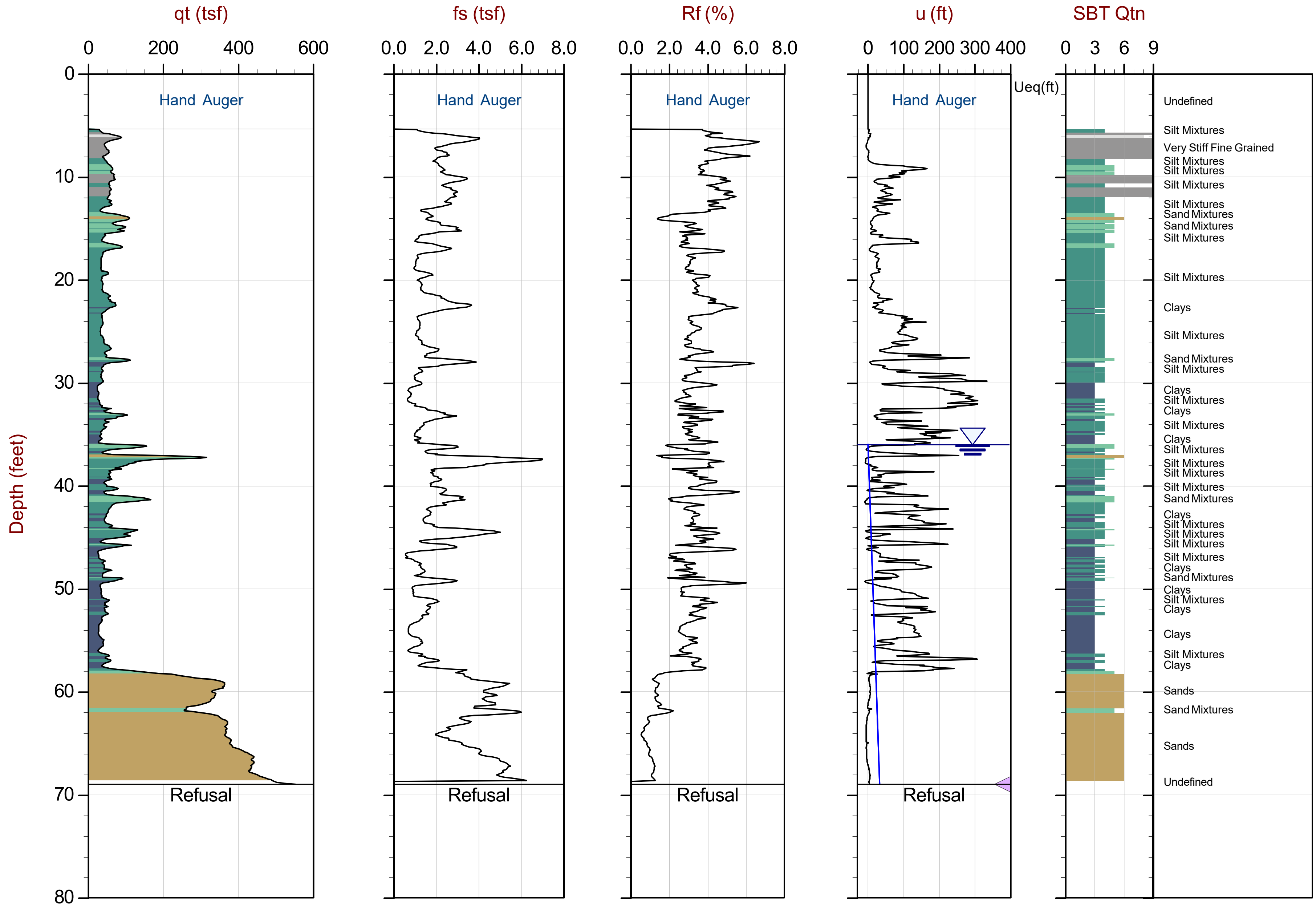
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

GeoPentech

Job No: 22-56-23760
Date: 2022-03-01 08:16
Site: 5350 Wilshire Development

Sounding: CPT-02
Cone: 817:T1500F15U35



Max Depth: 21.025 m / 68.98 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 22-56-23760_CP02.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 11S N: 3769810m E: 375805m

Equilibrium Pore Pressure (Ueq) | Assumed Ueq | Dissipation, Ueq achieved | Dissipation, Ueq not achieved | Hydrostatic Line

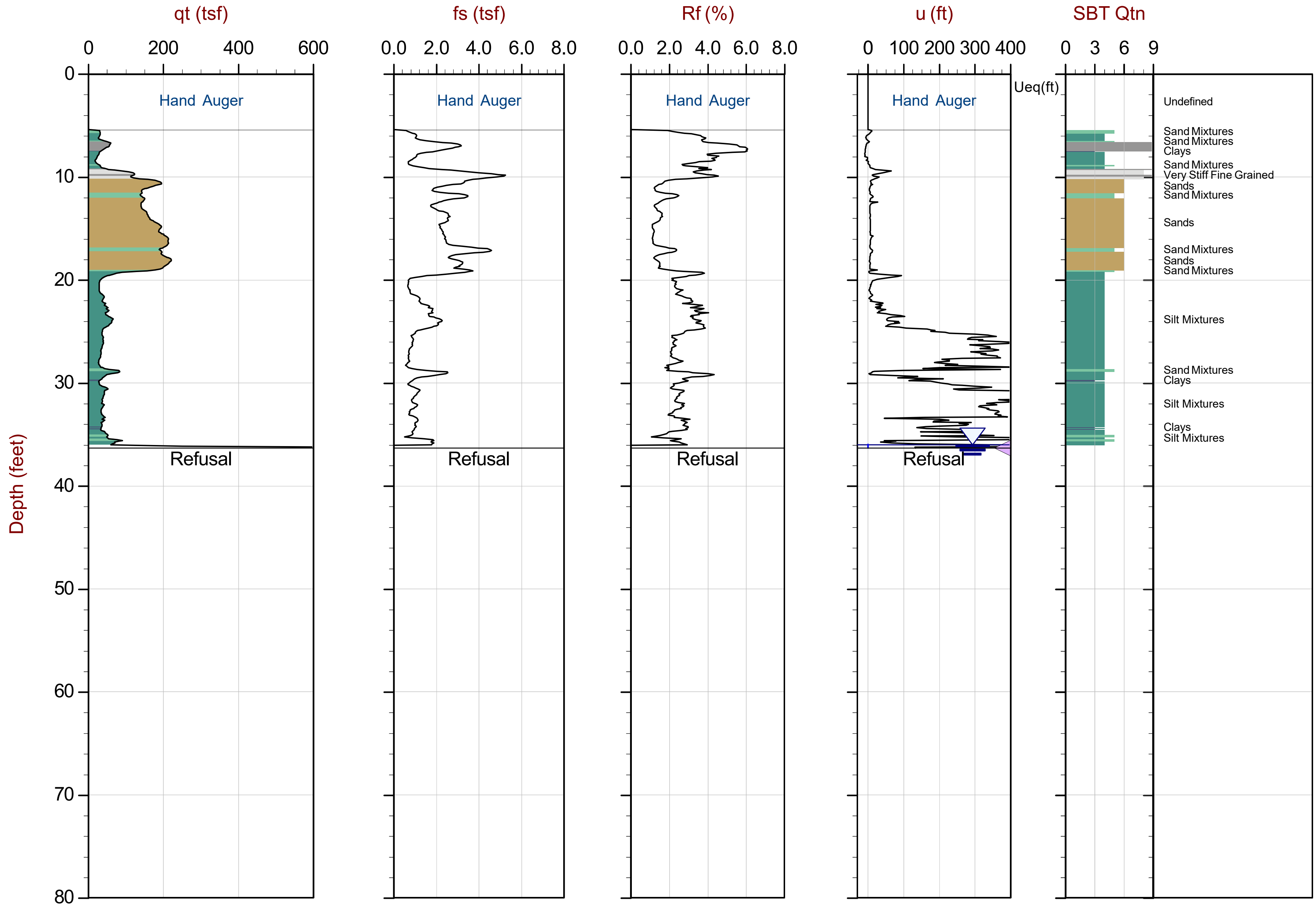
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

GeoPentech

Job No: 22-56-23760
Date: 2022-03-01 11:50
Site: 5350 Wilshire Development

Sounding: CPT-03
Cone: 817:T1500F15U35



Max Depth: 11.075 m / 36.33 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 22-56-23760_CP03.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 11S N: 3769802m E: 375775m

Equilibrium Pore Pressure (Ueq) Assumed Ueq Dissipation, Ueq achieved Dissipation, Ueq not achieved Hydrostatic Line

The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

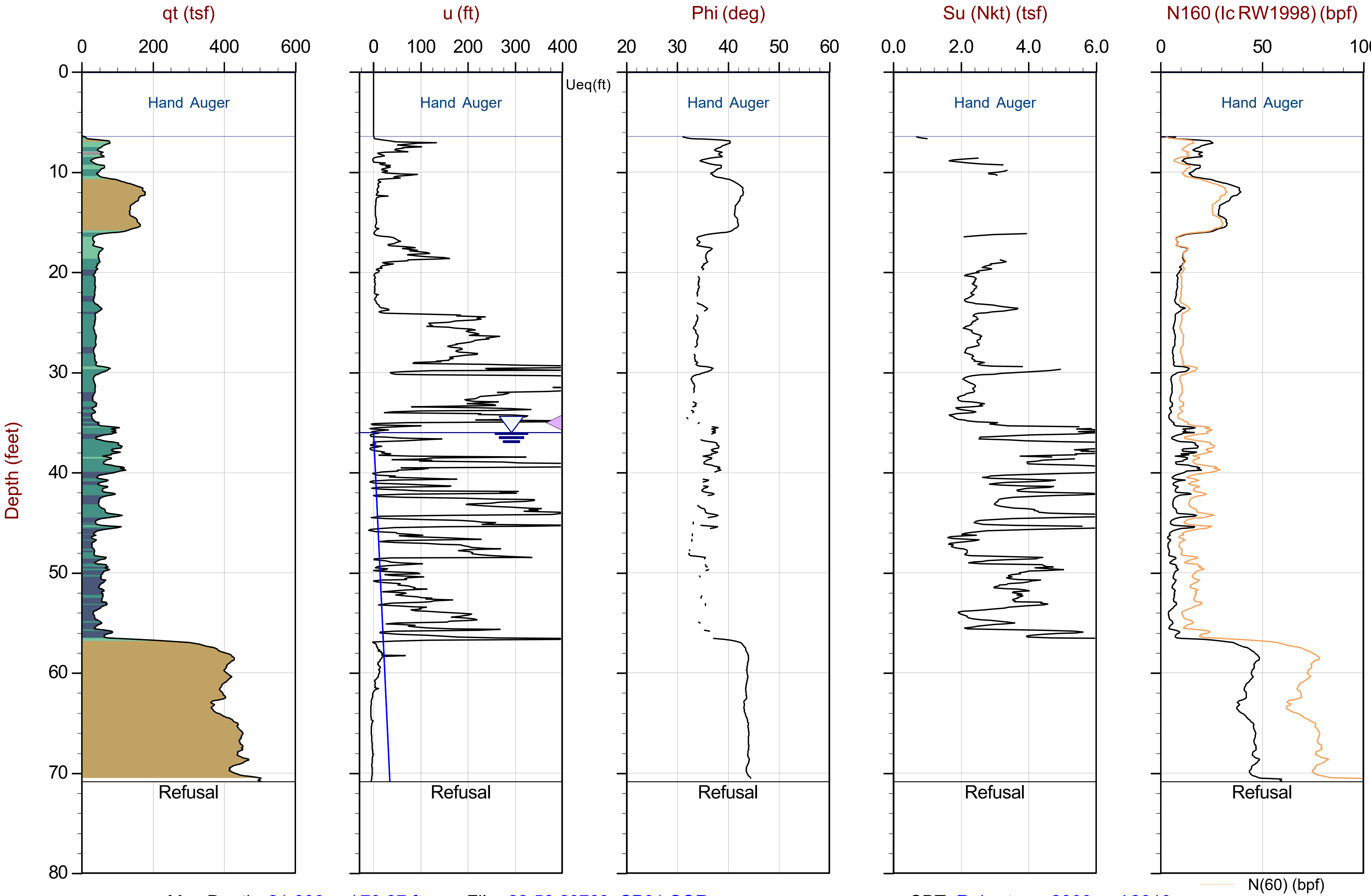
Advanced Cone Penetration Test Plots

ConeTec

GeoPentech

Job No: 22-56-23760
Date: 2022-03-01 09:58
Site: 5350 Wilshire Development

Sounding: CPT-01
Cone: 817:T1500F15U35



Max Depth: 21.600 m / 70.87 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 22-56-23760_CP01.COR
Unit Wt: SBTQtn (PKR2009)
Su Nkt: 15.0

SBT: Robertson, 2009 and 2010
Coords: UTM 11S N: 3769822m E: 375775m

Equilibrium Pore Pressure (Ueq) Assumed Ueq Dissipation, Ueq achieved Dissipation, Ueq not achieved Hydrostatic Line

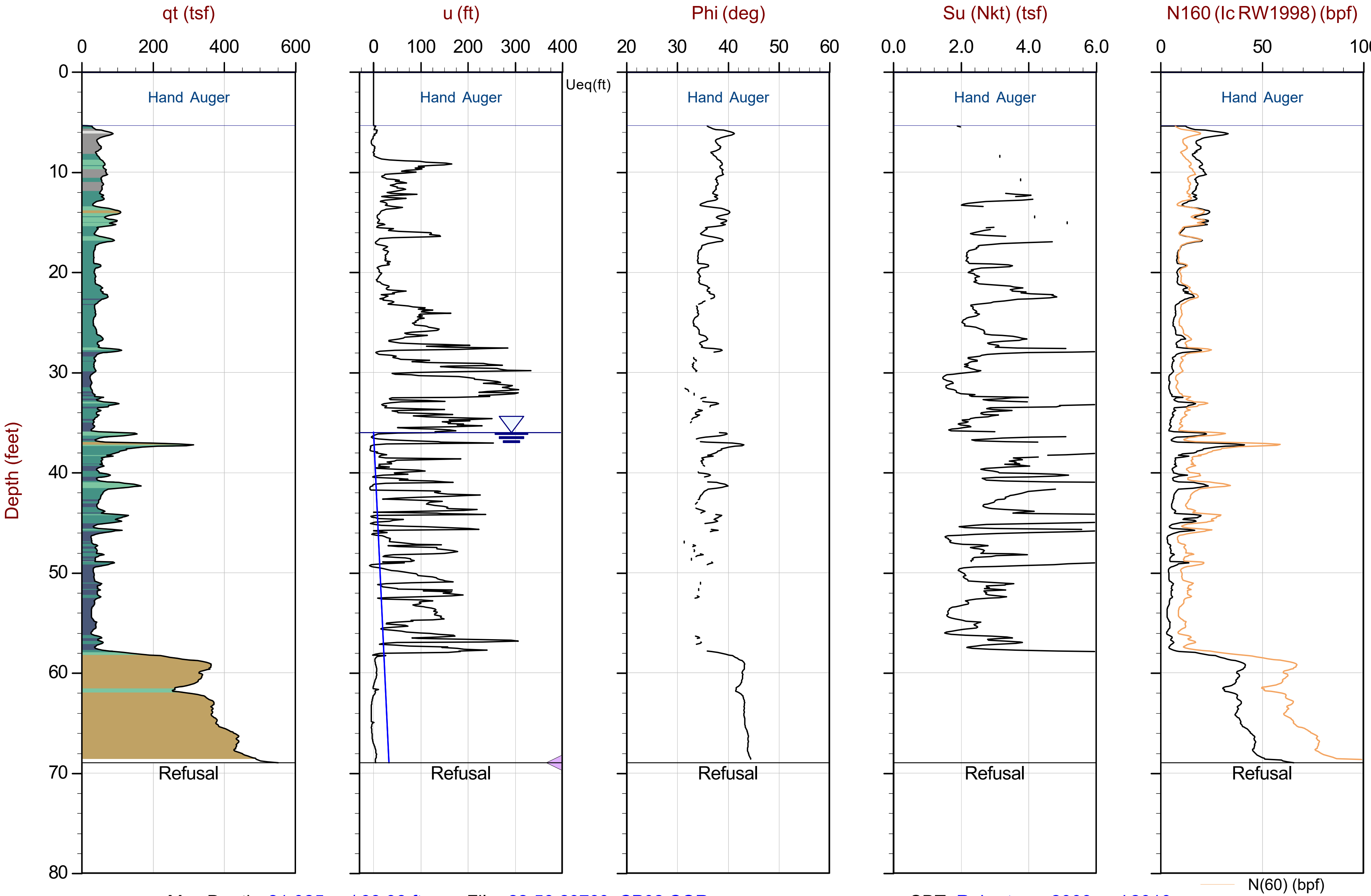
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

GeoPentech

Job No: 22-56-23760
Date: 2022-03-01 08:16
Site: 5350 Wilshire Development

Sounding: CPT-02
Cone: 817:T1500F15U35



Max Depth: 21.025 m / 68.98 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 22-56-23760_CP02.COR
Unit Wt: SBTQtn (PKR2009)
Su Nkt: 15.0

SBT: Robertson, 2009 and 2010
Coords: UTM 11S N: 3769810m E: 375805m

Equilibrium Pore Pressure (Ueq) Assumed Ueq Dissipation, Ueq achieved Dissipation, Ueq not achieved Hydrostatic Line

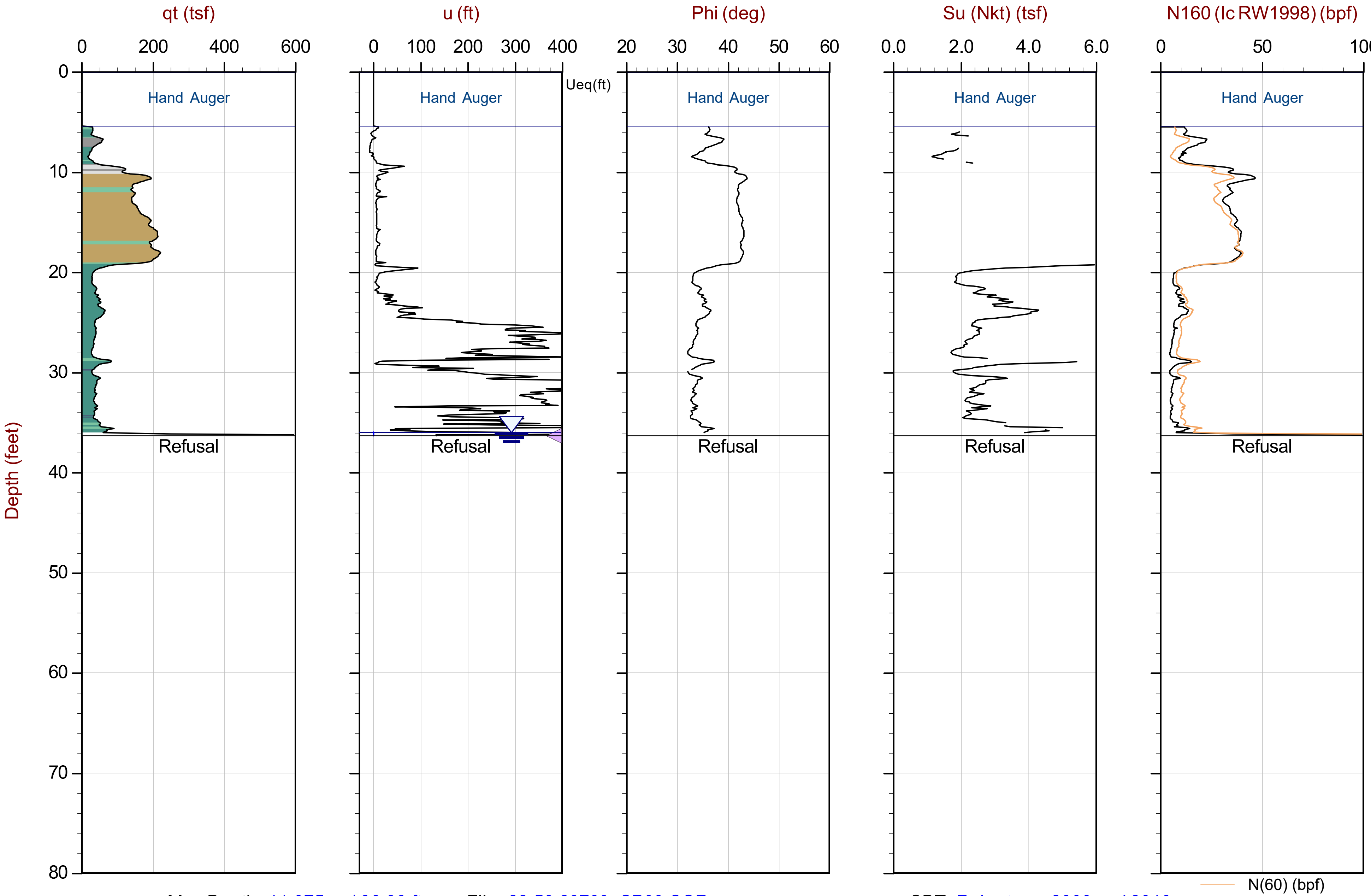
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

GeoPentech

Job No: 22-56-23760
Date: 2022-03-01 11:50
Site: 5350 Wilshire Development

Sounding: CPT-03
Cone: 817:T1500F15U35



Max Depth: 11.075 m / 36.33 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 22-56-23760_CP03.COR
Unit Wt: SBTQtn (PKR2009)
Su Nkt: 15.0

SBT: Robertson, 2009 and 2010
Coords: UTM 11S N: 3769802m E: 375775m

Equilibrium Pore Pressure (Ueq) Assumed Ueq Dissipation, Ueq achieved Dissipation, Ueq not achieved Hydrostatic Line

The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

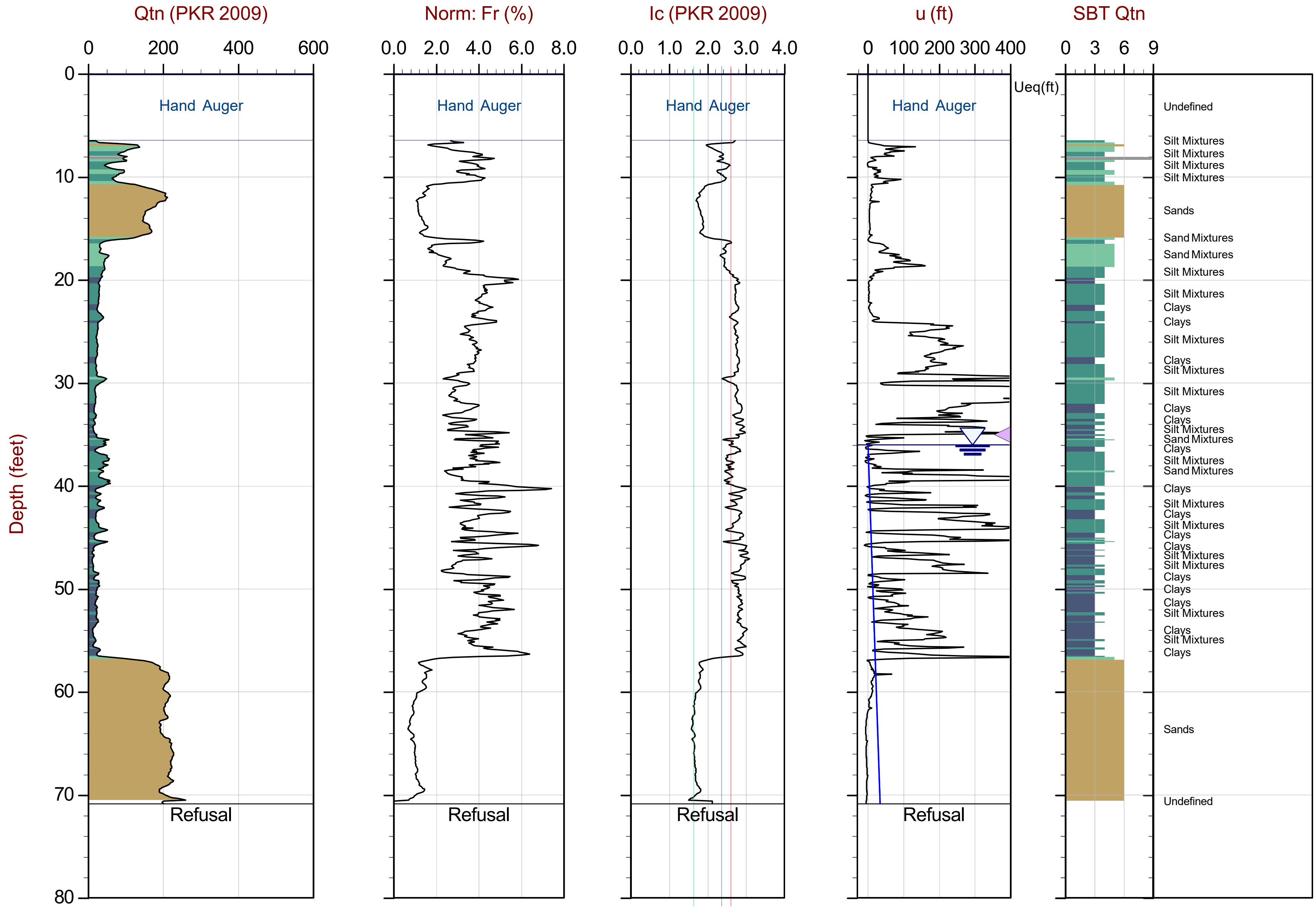
Normalized Cone Penetration Test Plots

CONETEC

GeoPentech

Job No: 22-56-23760
Date: 2022-03-01 09:58
Site: 5350 Wilshire Development

Sounding: CPT-01
Cone: 817:T1500F15U35



Max Depth: 21.600 m / 70.87 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 22-56-23760_CP01.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 11S N: 3769822m E: 375775m

Equilibrium Pore Pressure (Ueq) Assumed Ueq Dissipation, Ueq achieved Dissipation, Ueq not achieved Hydrostatic Line

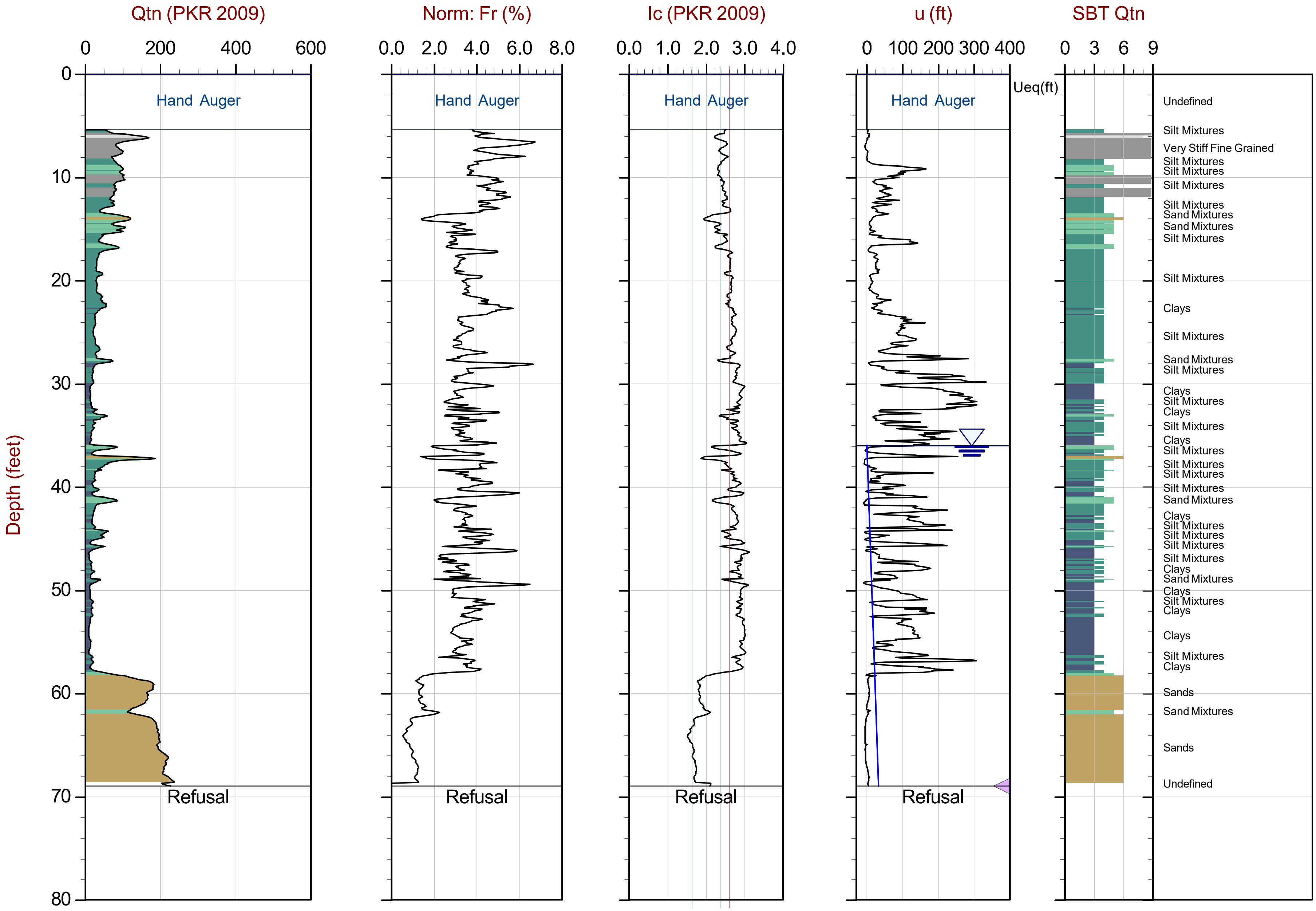
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

GeoPentech

Job No: 22-56-23760
Date: 2022-03-01 08:16
Site: 5350 Wilshire Development

Sounding: CPT-02
Cone: 817:T1500F15U35



Max Depth: 21.025 m / 68.98 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 22-56-23760_CP02.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 11S N: 3769810m E: 375805m

Equilibrium Pore Pressure (Ueq) Assumed Ueq Dissipation, Ueq achieved Dissipation, Ueq not achieved Hydrostatic Line

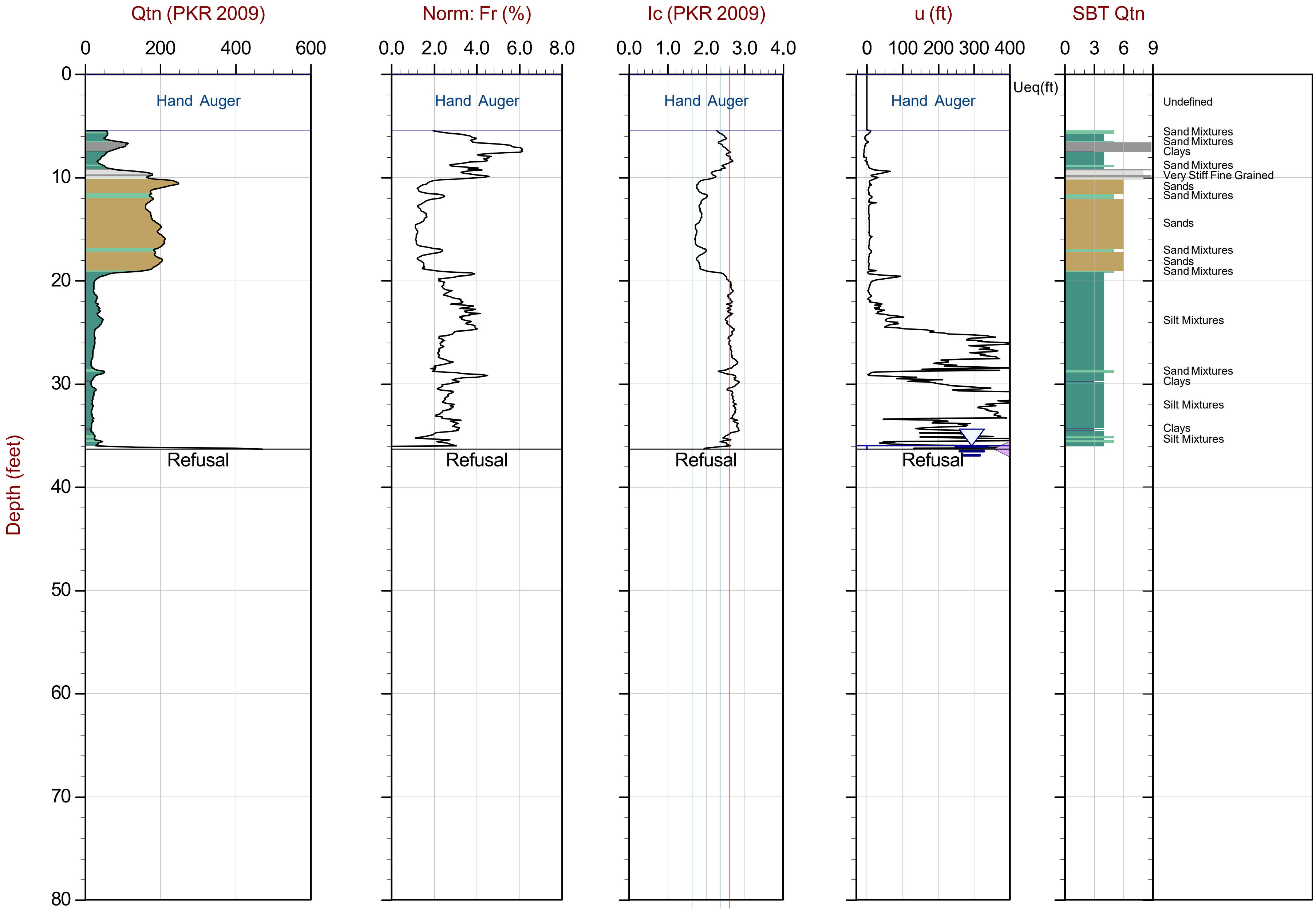
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

GeoPentech

Job No: 22-56-23760
Date: 2022-03-01 11:50
Site: 5350 Wilshire Development

Sounding: CPT-03
Cone: 817:T1500F15U35



Max Depth: 11.075 m / 36.33 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 22-56-23760_CP03.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 11S N: 3769802m E: 375775m

Equilibrium Pore Pressure (Ueq) | Assumed Ueq | Dissipation, Ueq achieved | Dissipation, Ueq not achieved | Hydrostatic Line

The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

Soil Behavior Type (SBT) Scatter Plots

ConeTec

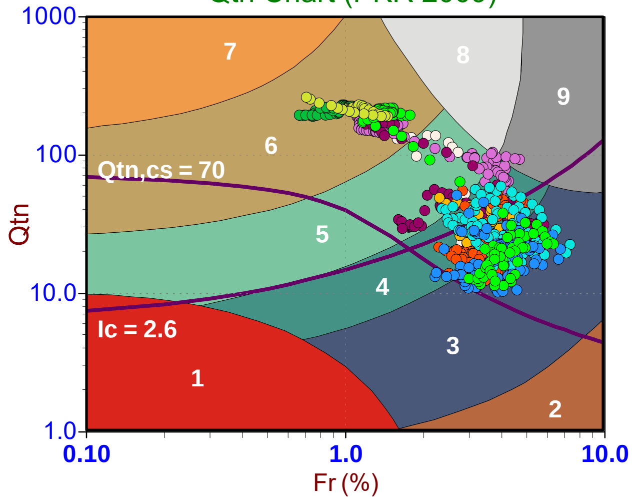
ConeTec

GeoPentech

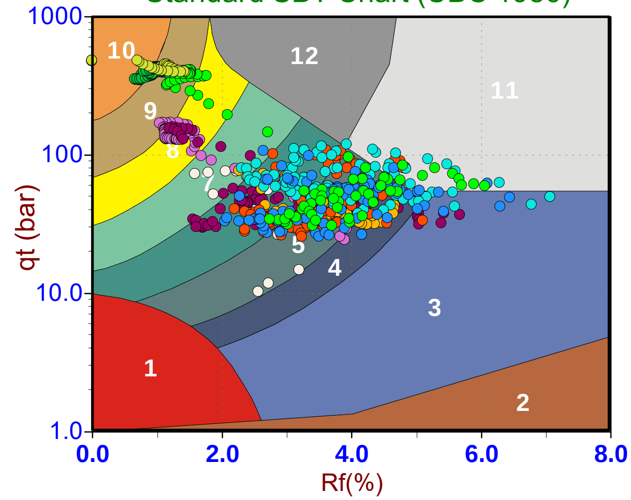
Job No: 22-56-23760
Date: 2022-03-01 09:58
Site: 5350 Wilshire Development

Sounding: CPT-01
Cone: 817:T1500F15U35

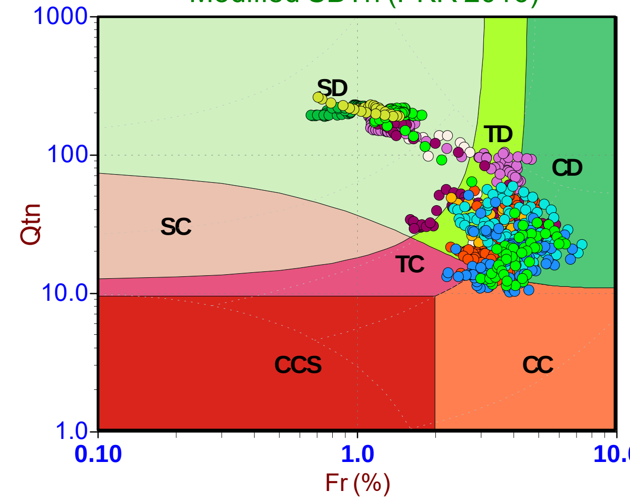
Qtn Chart (PKR 2009)



Standard SBT Chart (UBC 1986)



Modified SBTn (PKR 2016)



Depth Ranges
- >0.0 to 7.5 ft
- >7.5 to 15.0 ft
- >15.0 to 22.5 ft
- >22.5 to 30.0 ft
- >30.0 to 37.5 ft
- >37.5 to 45.0 ft
- >45.0 to 52.5 ft
- >52.5 to 60.0 ft
- >60.0 to 67.5 ft
- >67.5 to 75.0 ft
- >75.0 ft

Legend
- Sensitive, Fine Grained
- Organic Soils
- Clays
- Silt Mixtures
- Sand Mixtures
- Sands
- Gravelly Sand to Sand
- Stiff Sand to Clayey Sand
- Very Stiff Fine Grained

Legend
- Sensitive Fines
- Organic Soil
- Clay
- Silty Clay
- Clayey Silt
- Silt
- Sandy Silt
- Silty Sand/Sand
- Sand
- Gravelly Sand
- Stiff Fine Grained
- Cemented Sand

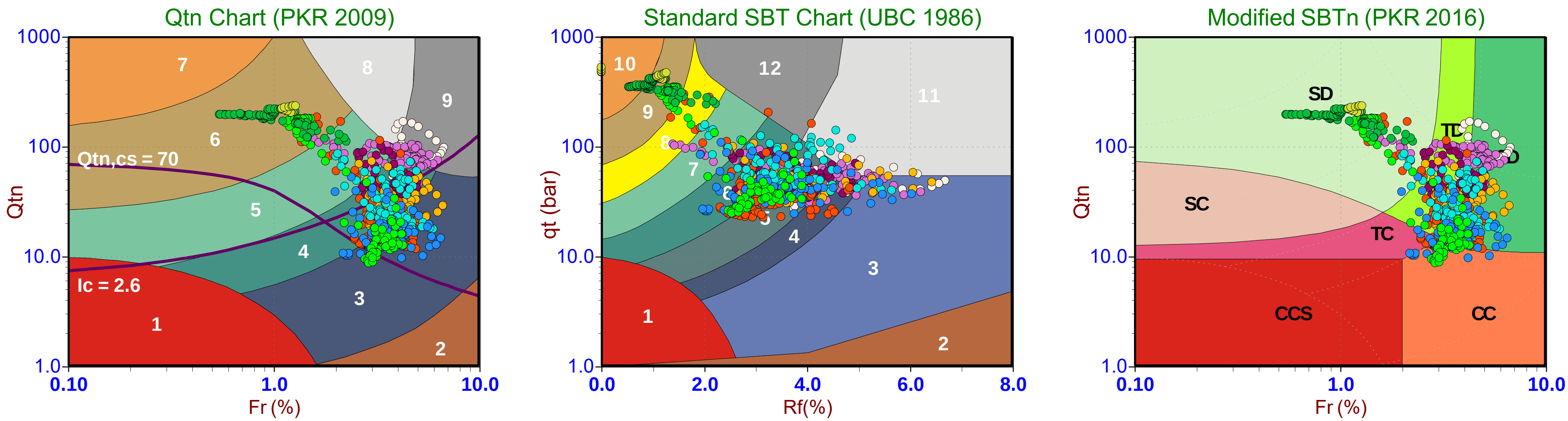
Legend
- CCS (Cont. sensitive clay like)
- CC (Cont. clay like)
- TC (Cont. transitional)
- SC (Cont. sand like)
- CD (Dil. clay like)
- TD (Dil. transitional)
- SD (Dil. sand like)

ConeTec

GeoPentech

Job No: 22-56-23760
Date: 2022-03-01 08:16
Site: 5350 Wilshire Development

Sounding: CPT-02
Cone: 817:T1500F15U35



Depth Ranges
- >0.0 to 7.5 ft
- >7.5 to 15.0 ft
- >15.0 to 22.5 ft
- >22.5 to 30.0 ft
- >30.0 to 37.5 ft
- >37.5 to 45.0 ft
- >45.0 to 52.5 ft
- >52.5 to 60.0 ft
- >60.0 to 67.5 ft
- >67.5 to 75.0 ft
- >75.0 ft

Legend
- Sensitive, Fine Grained
- Organic Soils
- Clays
- Silt Mixtures
- Sand Mixtures
- Sands
- Gravelly Sand to Sand
- Stiff Sand to Clayey Sand
- Very Stiff Fine Grained

Legend
- Sensitive Fines
- Organic Soil
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- Silty Clay
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- Silt
- Sandy Silt
- Silty Sand/Sand
- Sand
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- Stiff Fine Grained
- Cemented Sand

Legend
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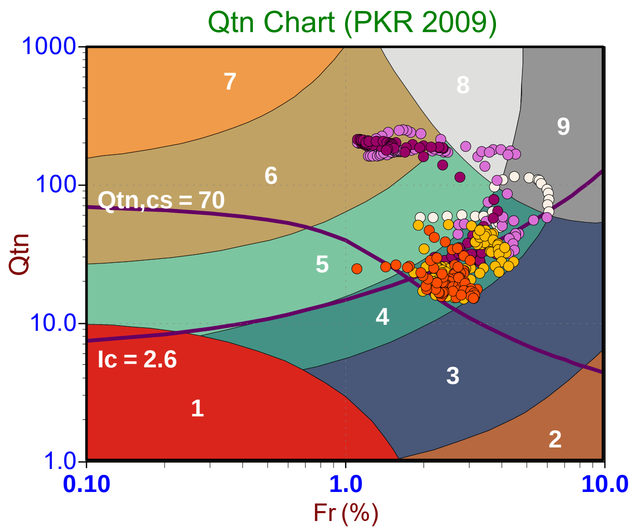
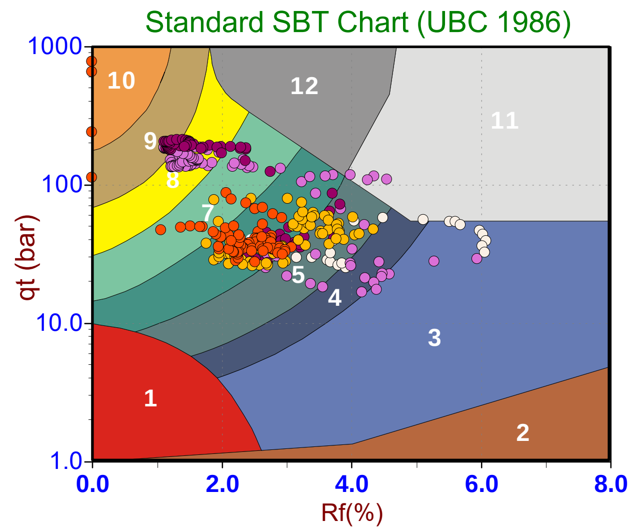
- CC (Cont. clay like)
- TC (Cont. transitional)
- SC (Cont. sand like)
- CD (Dil. clay like)
- TD (Dil. transitional)
- SD (Dil. sand like)

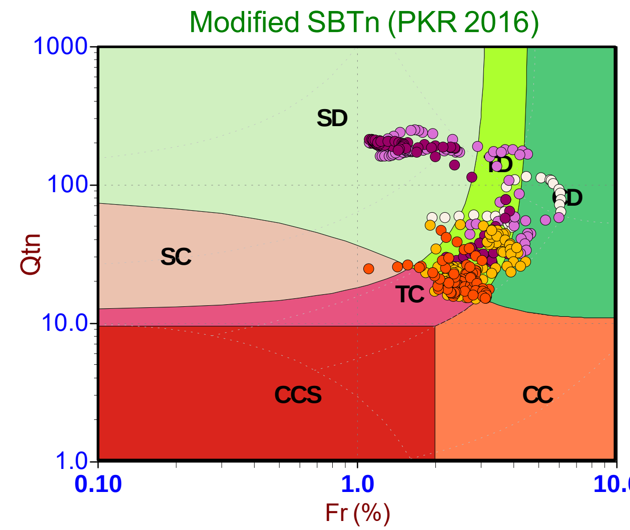
GeoPentech

Job No: 22-56-23760
Date: 2022-03-01 11:50
Site: 5350 Wilshire Development

Sounding: CPT-03
Cone: 817:T1500F15U35







Depth Ranges
- >0.0 to 7.5 ft
- >7.5 to 15.0 ft
- >15.0 to 22.5 ft
- >22.5 to 30.0 ft
- >30.0 to 37.5 ft
- >37.5 to 45.0 ft
- >45.0 to 52.5 ft
- >52.5 to 60.0 ft
- >60.0 to 67.5 ft
- >67.5 to 75.0 ft
- >75.0 ft

Legend
- Sensitive, Fine Grained
- Organic Soils
- Clays
- Silt Mixtures
- Sand Mixtures
- Sands
- Gravelly Sand to Sand
- Stiff Sand to Clayey Sand
- Very Stiff Fine Grained

Legend
- Sensitive Fines
- Organic Soil
- Clay
- Silty Clay
- Clayey Silt
- Silt
- Sandy Silt
- Silty Sand/Sand
- Sand
- Gravelly Sand
- Stiff Fine Grained
- Cemented Sand

Legend
- CCS (Cont. sensitive clay like)
- CC (Cont. clay like)
- TC (Cont. transitional)
- SC (Cont. sand like)
- CD (Dil. clay like)
- TD (Dil. transitional)
- SD (Dil. sand like)

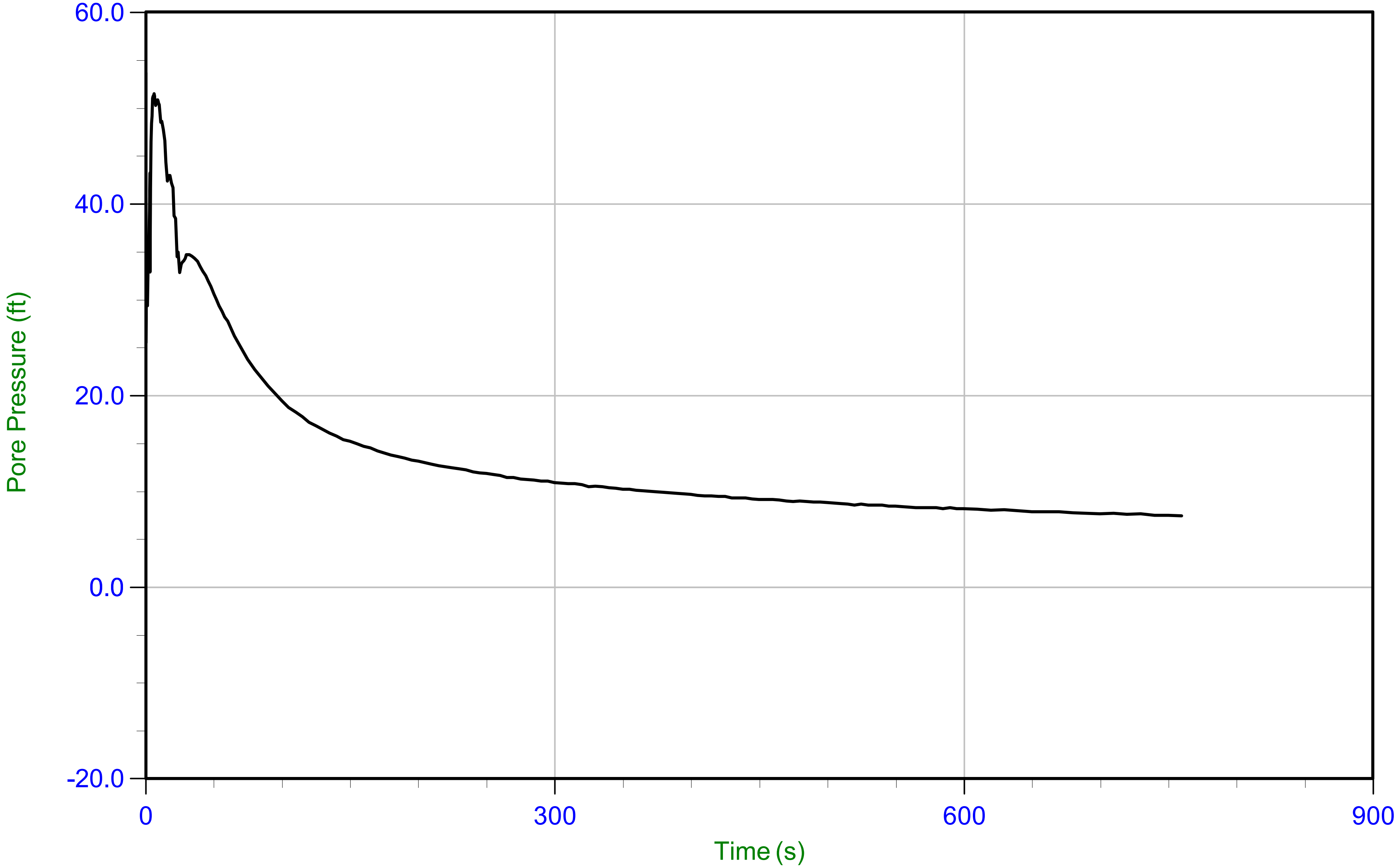
Pore Pressure Dissipation Summary and Pore Pressure Dissipation Plots

Job No:	22-56-23760
Client:	GeoPentech
Project:	5350 Wilshire Development
Start Date:	01-Mar-2022
End Date:	01-Mar-2022

CPTu PORE PRESSURE DISSIPATION SUMMARY

Sounding ID	File Name	Cone Area (cm^2)	Duration (s)	Test Depth (ft)	Estimated Equilibrium Pore Pressure U_{eq} (ft.)	Calculated Phreatic Surface (ft.)
CPT-01	22-56-23760_CP01	15	760	35.02	Not Achieved	
CPT-02	22-56-23760_CP02	15	300	68.98	Not Achieved	
CPT-03	22-56-23760_CP03	15	690	36.33	Not Achieved	

ConeTec	GeoPentech	Job No: 22-56-23760 Date: 03/01/2022 09:58 Site: 5350 Wilshire Development	Sounding: CPT-01 Cone: 817:T1500F15U35 Area=15 cm²

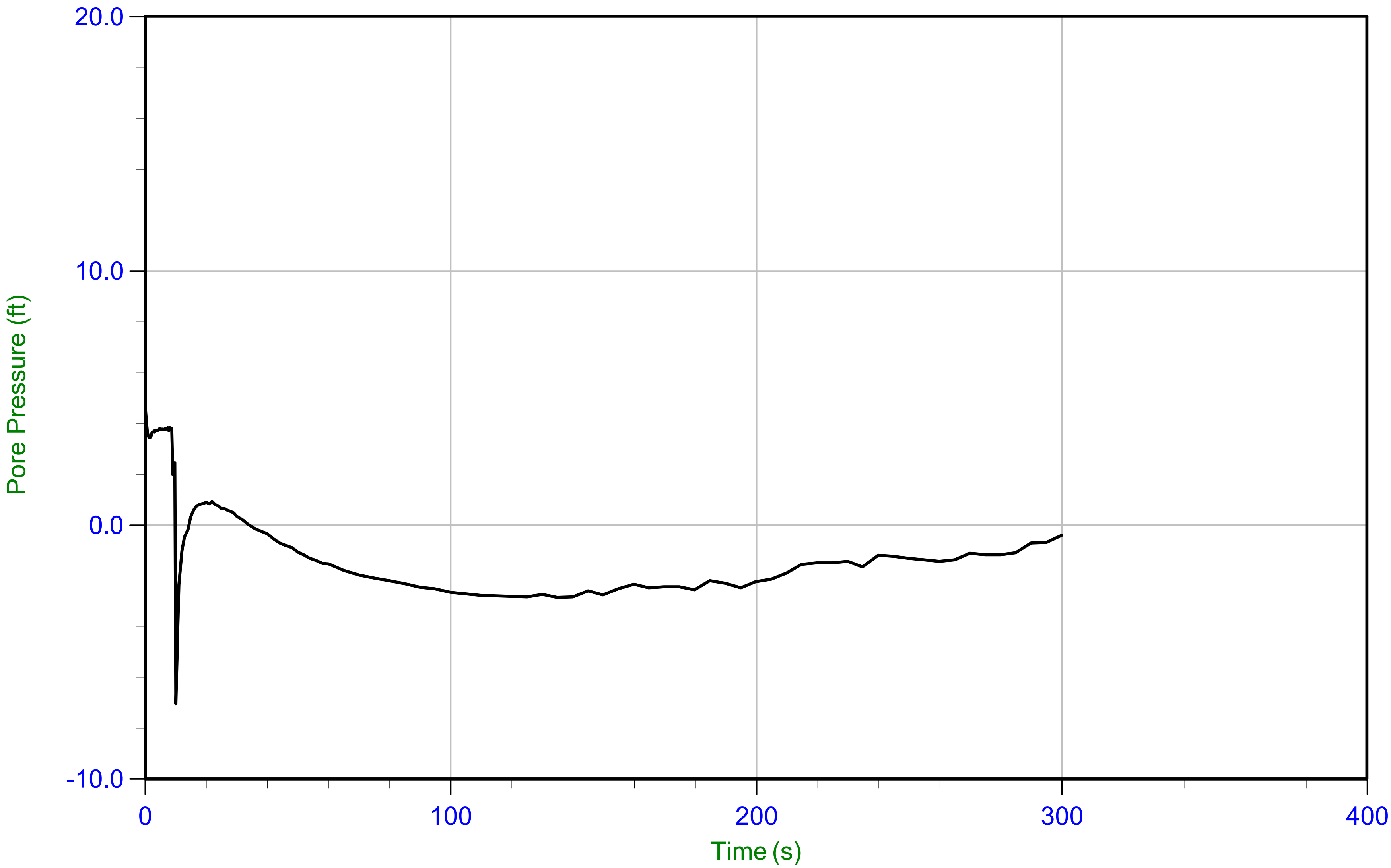


Trace Summary:

Filename: 22-56-23760_CP01.PPD2
Depth: 10.675 m / 35.023 ft
Duration: 760.0 s

u Min: 7.5 ft
u Max: 53.7 ft
u Final: 7.5 ft

ConeTec | GeoPentech | Job No: 22-56-23760 | Sounding: CPT-02
Date: 03/01/2022 08:16 | Cone: 817:T1500F15U35 Area=15 cm²
Site: 5350 Wilshire Development



Trace Summary:

Filename: 22-56-23760_CP02.PPD2
Depth: 21.025 m / 68.979 ft
Duration: 300.0 s

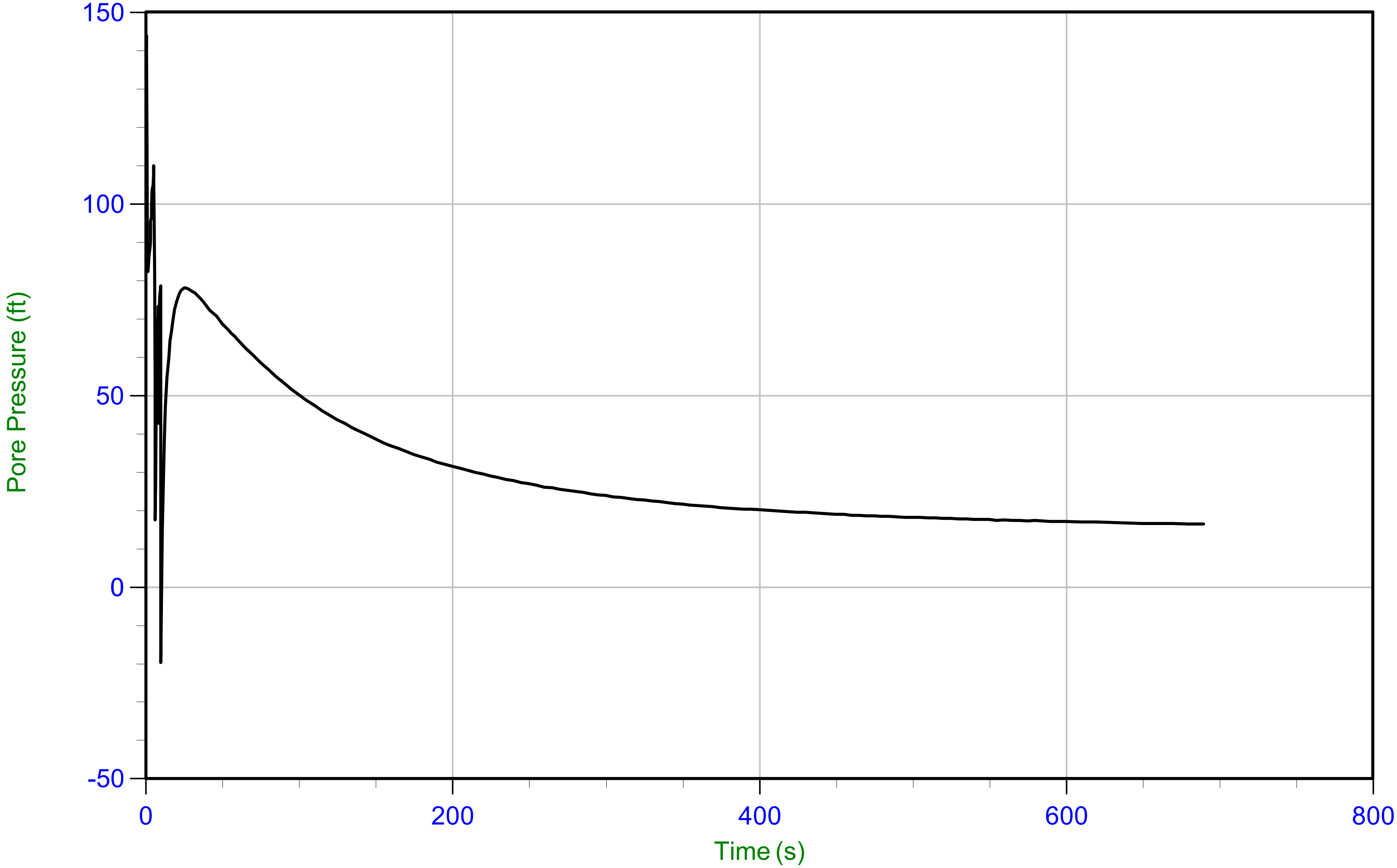
u Min: -7.0 ft
u Max: 4.7 ft
u Final: -0.4 ft

CONETEC

GeoPentech

Job No: 22-56-23760
Date: 03/01/2022 11:50
Site: 5350 Wilshire Development

Sounding: CPT-03
Cone: 817:T1500F15U35 Area=15 cm²



Trace Summary:

Filename: 22-56-23760_CP03.PPD2
Depth: 11.075 m / 36.335 ft
Duration: 690.0 s

u Min: -19.7 ft
u Max: 143.9 ft
u Final: 16.4 ft

Methodology Statements and Data File Formats

Cone penetration tests (CPTu) are conducted using an integrated electronic piezocone penetrometer and data acquisition system manufactured by Adara Systems Ltd., a subsidiary of ConeTec.

ConeTec's piezocone penetrometers are compression type designs in which the tip and friction sleeve load cells are independent and have separate load capacities. The piezocones use strain gauged load cells for tip and sleeve friction and a strain gauged diaphragm type transducer for recording pore pressure. The piezocones also have a platinum resistive temperature device (RTD) for monitoring the temperature of the sensors, an accelerometer type dual axis inclinometer and two geophone sensors for recording seismic signals. All signals are amplified and measured with minimum sixteen-bit resolution down hole within the cone body, and the signals are sent to the surface using a high bandwidth, error corrected digital interface through a shielded cable.

ConeTec penetrometers are manufactured with various tip, friction and pore pressure capacities in both 10 cm^2 and 15 cm^2 tip base area configurations in order to maximize signal resolution for various soil conditions. The specific piezocone used for each test is described in the CPT summary table presented in the first appendix. The 15 cm^2 penetrometers do not require friction reducers as they have a diameter larger than the deployment rods. The 10 cm^2 piezocones use a friction reducer consisting of a rod adapter extension behind the main cone body with an enlarged cross sectional area (typically 44 millimeters diameter over a length of 32 millimeters with tapered leading and trailing edges) located at a distance of 585 millimeters above the cone tip.

The penetrometers are designed with equal end area friction sleeves, a net end area ratio of 0.8 and cone tips with a 60 degree apex angle.

All ConeTec piezocones can record pore pressure at various locations. Unless otherwise noted, the pore pressure filter is located directly behind the cone tip in the "u_2" position (ASTM Type 2). The filter is six millimeters thick, made of porous plastic (polyethylene) having an average pore size of 125 microns (90-160 microns). The function of the filter is to allow rapid movements of extremely small volumes of water needed to activate the pressure transducer while preventing soil ingress or blockage.

The piezocone penetrometers are manufactured with dimensions, tolerances and sensor characteristics that are in general accordance with the current ASTM D5778 standard. ConeTec's calibration criteria also meets or exceeds those of the current ASTM D5778 standard. An illustration of the piezocone penetrometer is presented in Figure CPTu.

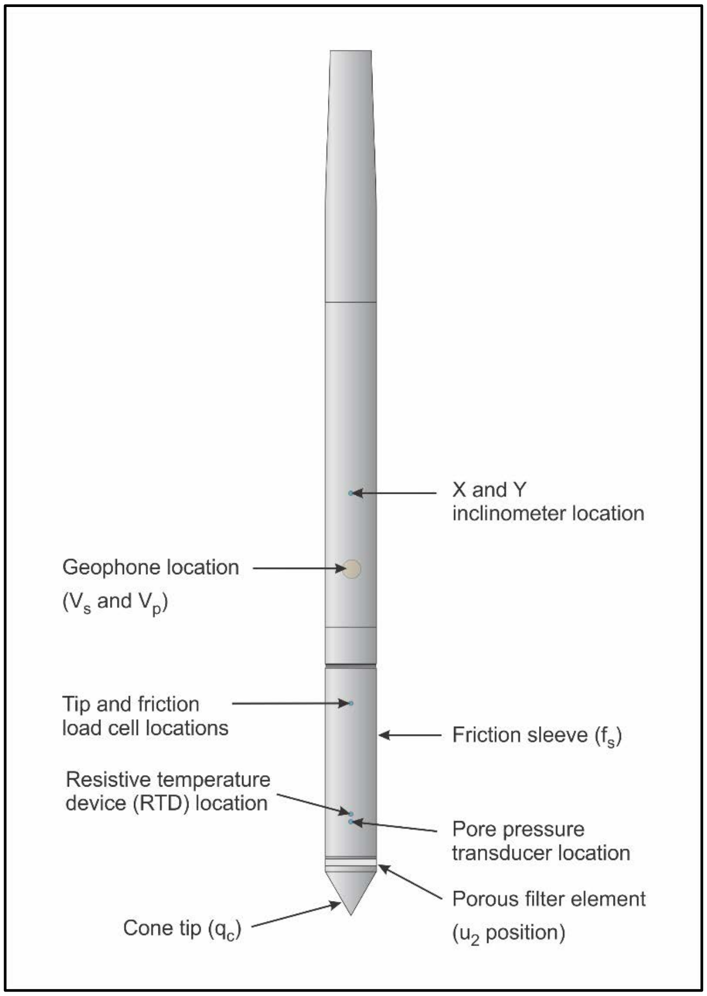


Figure CPTu. Piezocone Penetrometer (15 cm^2)

The ConeTec data acquisition systems consist of a Windows based computer and a signal interface box and power supply. The signal interface combines depth increment signals, seismic trigger signals and the downhole digital data. This combined data is then sent to the Windows based computer for collection and presentation. The data is recorded at fixed depth increments using a depth wheel attached to the push cylinders or by using a spring loaded rubber depth wheel that is held against the cone rods. The typical recording interval is 2.5 centimeters; custom recording intervals are possible.

The system displays the CPTu data in real time and records the following parameters to a storage media during penetration:

- Depth
- Uncorrected tip resistance (q_c)
- Sleeve friction (f_s)
- Dynamic pore pressure (u)
- Additional sensors such as resistivity, passive gamma, ultra violet induced fluorescence, if applicable

All testing is performed in accordance to ConeTec's CPTu operating procedures which are in general accordance with the current ASTM D5778 standard.

Prior to the start of a CPTu sounding a suitable cone is selected, the cone and data acquisition system are powered on, the pore pressure system is saturated with silicone oil and the baseline readings are recorded with the cone hanging freely in a vertical position.

The CPTu is conducted at a steady rate of two centimeters per second, within acceptable tolerances. Typically one meter length rods with an outer diameter of 1.5 inches (38.1 millimeters) are added to advance the cone to the sounding termination depth. After cone retraction final baselines are recorded.

Additional information pertaining to ConeTec's cone penetration testing procedures:

- Each filter is saturated in silicone oil under vacuum pressure prior to use
- Baseline readings are compared to previous readings
- Soundings are terminated at the client's target depth or at a depth where an obstruction is encountered, excessive rod flex occurs, excessive inclination occurs, equipment damage is likely to take place, or a dangerous working environment arises
- Differences between initial and final baselines are calculated to ensure zero load offsets have not occurred and to ensure compliance with ASTM standards

The interpretation of piezocone data for this report is based on the corrected tip resistance (q_t), sleeve friction (f_s) and pore water pressure (u). The interpretation of soil type is based on the correlations developed by Robertson et al. (1986) and Robertson (1990, 2009). It should be noted that it is not always possible to accurately identify a soil behavior type based on these parameters. In these situations, experience, judgment and an assessment of other parameters may be used to infer soil behavior type.

The recorded tip resistance (q_c) is the total force acting on the piezocone tip divided by its base area. The tip resistance is corrected for pore pressure effects and termed corrected tip resistance (q_t) according to the following expression presented in Robertson et al. (1986):

$$q_t = q_c + (1\text{-}a) \bullet u_2$$

where: q_t is the corrected tip resistance
q_c is the recorded tip resistance
u_2 is the recorded dynamic pore pressure behind the tip (u_2 position)
a is the Net Area Ratio for the piezocone (0.8 for ConeTec probes)

The sleeve friction (f_s) is the frictional force on the sleeve divided by its surface area. As all ConeTec piezocones have equal end area friction sleeves, pore pressure corrections to the sleeve data are not required.

The dynamic pore pressure (u) is a measure of the pore pressures generated during cone penetration. To record equilibrium pore pressure, the penetration must be stopped to allow the dynamic pore pressures to stabilize. The rate at which this occurs is predominantly a function of the permeability of the soil and the diameter of the cone.

The friction ratio (R_f) is a calculated parameter. It is defined as the ratio of sleeve friction to the tip resistance expressed as a percentage. Generally, saturated cohesive soils have low tip resistance, high friction ratios and generate large excess pore water pressures. Cohesionless soils have higher tip resistances, lower friction ratios and do not generate significant excess pore water pressure.

A summary of the CPTu soundings along with test details and individual plots are provided in the appendices. A set of files with calculated geotechnical parameters were generated for each sounding based on published correlations and are provided in Excel format in the data release folder. Information regarding the methods used is also included in the data release folder.

For additional information on CPTu interpretations and calculated geotechnical parameters, refer to Robertson et al. (1986), Lunne et al. (1997), Robertson (2009), Mayne (2013, 2014) and Mayne and Peuchen (2012).

The cone penetration test is halted at specific depths to carry out pore pressure dissipation (PPD) tests, shown in Figure PPD-1. For each dissipation test the cone and rods are decoupled from the rig and the data acquisition system measures and records the variation of the pore pressure (u) with time (t).

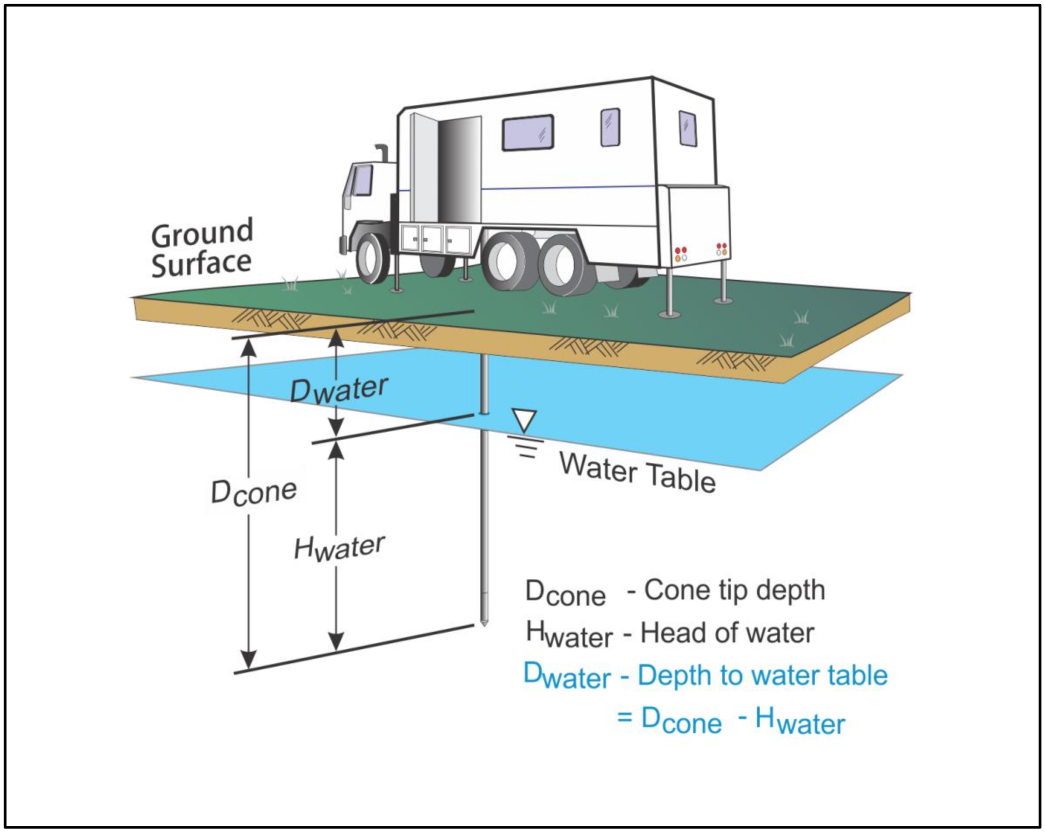


Figure PPD-1. Pore pressure dissipation test setup

Pore pressure dissipation data can be interpreted to provide estimates of ground water conditions, permeability, consolidation characteristics and soil behavior.

The typical shapes of dissipation curves shown in Figure PPD-2 are very useful in assessing soil type, drainage, in situ pore pressure and soil properties. A flat curve that stabilizes quickly is typical of a freely draining sand. Undrained soils such as clays will typically show positive excess pore pressure and have long dissipation times. Dilative soils will often exhibit dynamic pore pressures below equilibrium that then rise over time. Overconsolidated fine-grained soils will often exhibit an initial dilatory response where there is an initial rise in pore pressure before reaching a peak and dissipating.

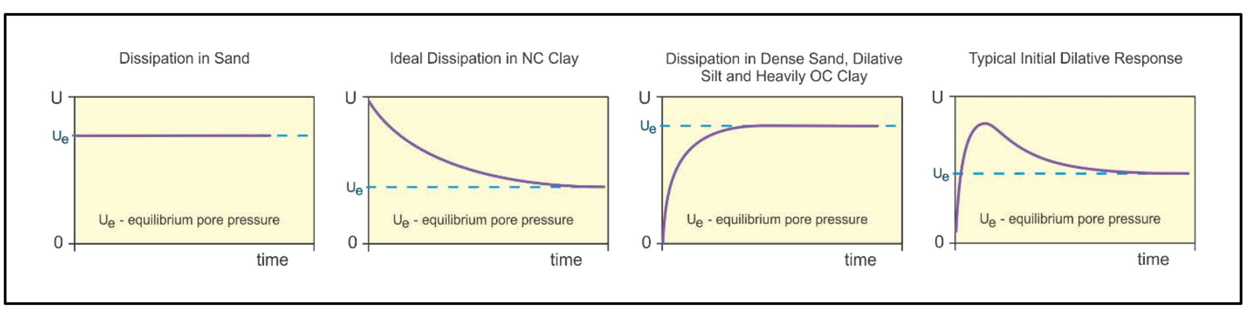


Figure PPD-2. Pore pressure dissipation curve examples

In order to interpret the equilibrium pore pressure (u_{eq}) and the apparent phreatic surface, the pore pressure should be monitored until such time as there is no variation in pore pressure with time as shown for each curve in Figure PPD-2.

In fine grained deposits the point at which 100% of the excess pore pressure has dissipated is known as t_{100}. In some cases this can take an excessive amount of time and it may be impractical to take the dissipation to t_{100}. A theoretical analysis of pore pressure dissipations by Teh and Houlsby (1991) showed that a single curve relating degree of dissipation versus theoretical time factor (T*) may be used to calculate the coefficient of consolidation (c_h) at various degrees of dissipation resulting in the expression for c_h shown below.

$$c_h = \frac{T^* \cdot a^2 \cdot \sqrt{I_r}}{t}$$

Where:
T* is the dimensionless time factor (Table Time Factor)
a is the radius of the cone
I_r is the rigidity index
t is the time at the degree of consolidation

Table Time Factor. T* versus degree of dissipation (Teh and Houlsby (1991))

Degree of Dissipation (%)	20	30	40	50	60	70	80
T* (u_2)	0.038	0.078	0.142	0.245	0.439	0.804	1.60

The coefficient of consolidation is typically analyzed using the time (t_{50}) corresponding to a degree of dissipation of 50% (u_{50}). In order to determine t_{50}, dissipation tests must be taken to a pressure less than u_{50}. The u_{50} value is half way between the initial maximum pore pressure and the equilibrium pore pressure value, known as u_{100}. To estimate u_{50}, both the initial maximum pore pressure and u_{100} must be known or estimated. Other degrees of dissipations may be considered, particularly for extremely long dissipations.

At any specific degree of dissipation the equilibrium pore pressure (u at t_{100}) must be estimated at the depth of interest. The equilibrium value may be determined from one or more sources such as measuring the value directly (u_{100}), estimating it from other dissipations in the same profile, estimating the phreatic surface and assuming hydrostatic conditions, from nearby soundings, from client provided information, from site observations and/or past experience, or from other site instrumentation.

For calculations of c_h (Teh and Houlsby (1991)), t_{50} values are estimated from the corresponding pore pressure dissipation curve and a rigidity index (I_r) is assumed. For curves having an initial dilatory response in which an initial rise in pore pressure occurs before reaching a peak, the relative time from the peak value is used in determining t_{50}. In cases where the time to peak is excessive, t_{50} values are not calculated.

Due to possible inherent uncertainties in estimating I_r, the equilibrium pore pressure and the effect of an initial dilatory response on calculating t_{50}, other methods should be applied to confirm the results for c_h.

Additional published methods for estimating the coefficient of consolidation from a piezocone test are described in Burns and Mayne (1998, 2002), Jones and Van Zyl (1981), Robertson et al. (1992) and Sully et al. (1999).

A summary of the pore pressure dissipation tests and dissipation plots are presented in the relevant appendix.

ASTM D5778-12, 2012, "Standard Test Method for Performing Electronic Friction Cone and Piezocone Penetration Testing of Soils", ASTM International, West Conshohocken, PA. DOI: 10.1520/D5778-12.

Burns, S.E. and Mayne, P.W., 1998, “Monotonic and dilatory pore pressure decay during piezocone tests”, Canadian Geotechnical Journal 26 (4): 1063-1073. DOI: 1063-1073/T98-062.

Burns, S.E. and Mayne, P.W., 2002, “Analytical cavity expansion-critical state model cone dissipation in fine-grained soils”, Soils & Foundations, Vol. 42(2): 131-137.

Jones, G.A. and Van Zyl, D.J.A., 1981, “The piezometer probe: a useful investigation tool”, Proceedings, 10th International Conference on Soil Mechanics and Foundation Engineering, Vol. 3, Stockholm: 489-495.

Lunne, T., Robertson, P.K. and Powell, J. J. M., 1997, “Cone Penetration Testing in Geotechnical Practice”, Blackie Academic and Professional.

Mayne, P.W., 2013, “Evaluating yield stress of soils from laboratory consolidation and in-situ cone penetration tests”, Sound Geotechnical Research to Practice (Holtz Volume) GSP 230, ASCE, Reston/VA: 406-420. DOI: 10.1061/9780784412770.027.

Mayne, P.W. and Peuchen, J., 2012, “Unit weight trends with cone resistance in soft to firm clays”, Geotechnical and Geophysical Site Characterization *4,* Vol. 1 (Proc. ISC-4, Pernambuco), CRC Press, London: 903-910.

Mayne, P.W., 2014, “Interpretation of geotechnical parameters from seismic piezocone tests”, CPT’14 Keynote Address, Las Vegas, NV, May 2014.

Robertson, P.K., Campanella, R.G., Gillespie, D. and Greig, J., 1986, “Use of Piezometer Cone Data”, Proceedings of InSitu 86, ASCE Specialty Conference, Blacksburg, Virginia.

Robertson, P.K., 1990, “Soil Classification Using the Cone Penetration Test”, Canadian Geotechnical Journal, Volume 27: 151-158. DOI: 10.1139/T90-014.

Robertson, P.K., Sully, J.P., Woeller, D.J., Lunne, T., Powell, J.J.M. and Gillespie, D.G., 1992, “Estimating coefficient of consolidation from piezocone tests”, Canadian Geotechnical Journal, 29(4): 539-550. DOI: 10.1139/T92-061.

Robertson, P.K., 2009, “Interpretation of cone penetration tests – a unified approach”, Canadian Geotechnical Journal, Volume 46: 1337-1355. DOI: 10.1139/T09-065.

Sully, J.P., Robertson, P.K., Campanella, R.G. and Woeller, D.J., 1999, “An approach to evaluation of field CPTU dissipation data in overconsolidated fine-grained soils”, Canadian Geotechnical Journal, 36(2): 369-381. DOI: 10.1139/T98-105.

Teh, C.I., and Houlsby, G.T., 1991, “An analytical study of the cone penetration test in clay”, Geotechnique, 41(1): 17-34. DOI: 10.1680/geot.1991.41.1.17.

CPT Data Files (COR Extension)

ConeTec CPT data files are stored in ASCII text files that are readable by almost any text editor. ConeTec file names start with the job number (which includes the two digit year number) an underscore as a separating character, followed by two letters based on the type of test and the sounding ID. The last character position is reserved for an identifier letter (such as b, c, d etc) used to uniquely distinguish multiple soundings at the same location. The CPT sounding file has the extension COR. As an example, for job number 21-02-00001 the first CPT sounding will have file name 21-02-00001_CP01.COR

The sounding (COR) file consists of the following components:

1. Two lines of header information
2. Data records
3. End of data marker
4. Units information

Header Lines

Line 1: Columns 1-6 may be blank or may indicate the version number of the recording software
Columns 7-21 contain the sounding Date and Time (Date is MM:DD:YY)
Columns 23-38 contain the sounding Operator
Columns 51-100 contain extended Job Location information

Line 2: Columns 1-16 contain the Job Location
Columns 17-32 contain the Cone ID
Columns 33-47 contain the sounding number
Columns 51-100 may contain extended sounding ID information

Data Records

The data records contain 4 or more columns of data in floating point format. A comma and spaces separate each data item:

Column 1: Sounding Depth (meters)
Column 2: Tip (q_c), recorded in units selected by the operator
Column 3: Sleeve (f_s), recorded in units selected by the operator
Column 4: Dynamic pore pressure (u), recorded in units selected by the operator
Column 5: Empty or may contain other requested data such as Gamma, Resistivity or UVIF data

End of Data Marker

After the last line of data there is a line containing an ASCII 26 (CTL-Z) character (small rectangular shaped character) followed by a newline (carriage return / line feed). This is used to mark the end of data.

Units Information

The last section of the file contains information about the units that were selected for the sounding. A separator bar makes up the first line. The second line contains the type of units used for depth, q_c, f_s and u. The third line contains the conversion values required for ConeTec's software to convert the recorded data to an internal set of base units (bar for q_c, bar for f_s and meters for u). Additional lines intended for internal ConeTec use may appear following the conversion values.

CPT Data Files (XLS Extension)

Excel format files of ConeTec CPT data are also generated from corresponding COR files. The XLS files have the same base file name as the COR file with a -BSC suffix. The information in the file is presented in table format and contains additional information about the sounding such as coordinate information, and tip net area ratio.

The BSCI suffix is given to XLS files which are enhanced versions of the BSC files and include the same data records in addition to inclination data collected for each sounding.

CPT Dissipation Files (XLS Extension)

Pore pressure dissipation files are provided in Excel format and contain each dissipation trace that exceeds a minimum duration (selected during post-processing) formatted column wise within the spreadsheet. The first column (Column A) contains the time in seconds and the second column (Column B) contains the time in minutes. Subsequent columns contain the dissipation trace data. The columns extend to the longest trace of the data set.

Detailed header information is provided at the top of the worksheet. The test depth in meters and feet, the number of points in the trace and the particular units are all presented at the top of each trace column.

CPT Dissipation files have the same naming convention as the CPT sounding files with a "–PPD" suffix.

Data Records

Each file will contain dissipation traces that exceed a minimum duration (selected during post-processing) in a particular column. The dissipation pore pressure values are typically recorded at varying time intervals throughout the trace; rapidly to start and increasing as the duration of the test lengthens. The test depth in meters and feet, the number of points in the trace and the trace number are identified at the top of each trace column.

Cone Type Designations

Cone ID	Cone Description	Tip Cross Sect. Area (cm^2)	Tip Capacity (bar)	Sleeve Area (cm^2)**	Sleeve Capacity (bar)	Pore Pressure Capacity (bar)
EC###	A15T1500F15U35	15	1500	225	15	35
EC###	A15T375F10U35	15	375	225	10	35
EC###	A10T1000F10U35	10	1000	150	10	35

refers to the Cone ID number
**Outer Cylindrical Area

Description of Methods for Calculated CPT Geotechnical Parameters

CALCULATED CPT GEOTECHNICAL PARAMETERS

A Detailed Description of the Methods Used in ConeTec's CPT Geotechnical Parameter Calculation and Plotting Software

Revision SZW-Rev 14

Revised November 26, 2019

Prepared by Jim Greig, M.A.Sc, P.Eng (BC)

CONETEC

Limitations

The geotechnical parameter output was prepared specifically for the site and project named in the accompanying report subject to objectives, site conditions and criteria provided to ConeTec by the client. The output may not be relied upon by any other party or for any other site without the express written permission of ConeTec Group (ConeTec) or any of its affiliates. For this project, ConeTec has provided site investigation services, prepared factual data reporting and produced geotechnical parameter calculations consistent with current best practices. No other warranty, expressed or implied, is made.

To understand the calculations that have been performed and to be able to reproduce the calculated parameters the user is directed to the basic descriptions for the methods in this document and the detailed descriptions and their associated limitations and appropriateness in the technical references cited for each parameter.

ConeTec's Calculated CPT Geotechnical Parameters as of November 26, 2019

ConeTec's CPT parameter calculation and plotting routine provides a tabular output of geotechnical parameters based on current published CPT correlations and is subject to change to reflect the current state of practice. Due to drainage conditions and the basic assumptions and limitations of the correlations, not all geotechnical parameters provided are considered applicable for all soil types. The results are presented only as a guide for geotechnical use and should be carefully examined for consideration in any geotechnical design. Reference to current literature is strongly recommended. ConeTec does not warranty the correctness or the applicability of any of the geotechnical parameters calculated by the program and does not assume liability for any use of the results in any design or review. For verification purposes we recommend that representative hand calculations be done for any parameter that is critical for design purposes. The end user of the parameter output should also be fully aware of the techniques and the limitations of any method used by the program. The purpose of this document is to inform the user as to which methods were used and to direct the end user to the appropriate technical papers and/or publications for further reference.

The geotechnical parameter output was prepared specifically for the site and project named in the accompanying report subject to objectives, site conditions and criteria provided to ConeTec by the client. The output may not be relied upon by any other party or for any other site without the express written permission of ConeTec Group (ConeTec) or any of its affiliates.

The CPT calculations are based on values of tip resistance, sleeve friction and pore pressures considered at each data point or averaged over a user specified layer thickness (e.g. 0.20 m). Note that q_t is the tip resistance corrected for pore pressure effects and q_c is the recorded tip resistance. The corrected tip resistance (corrected using u_2 pore pressure values) is used for all of the calculations. Since all ConeTec cones have equal end area friction sleeves pore pressure corrections to sleeve friction, f_s, are not required.

The tip correction is: $q_t = q_c + (1-a) \bullet u_2$ (consistent units are implied)

where: q_t is the corrected tip resistance
q_c is the recorded tip resistance
u_2 is the recorded dynamic pore pressure behind the tip (u_2 position)
a is the Net Area Ratio for the cone (typically 0.80 for ConeTec cones)

The total stress calculations are based on soil unit weight values that have been assigned to the Soil Behavior Type (SBT) zones, from a user defined unit weight profile, by using a single uniform value throughout the profile, through unit weight estimation techniques described in various technical papers or from a combination of these methods. The parameter output files indicate the method(s) used.

Effective vertical overburden stresses are calculated based on a hydrostatic distribution of equilibrium pore pressures below the water table or from a user defined equilibrium pore pressure profile (typically obtained from CPT dissipation tests) or a combination of the two. For over water projects the stress effects of the column of water above the mudline have been taken into account as has the appropriate unit weight of water. How this is done depends on where the instruments were zeroed (i.e. on deck or at the mudline). The parameter output files indicate the method(s) used.

A majority of parameter calculations are derived or driven by results based on material types as determined by the various soil behavior type charts depicted in Figures 1 through 5. The parameter output files indicate the method(s) used.

The Soil Behavior Type classification chart shown in Figure 1 is the classic non-normalized SBT Chart developed at the University of British Columbia and reported in Robertson, Campanella, Gillespie and Greig (1986). Figure 2 shows the original normalized (linear method) SBT chart developed by Robertson (1990). The Bq classification charts shown in Figures 3a and 3b incorporate pore pressures into the SBT classification and are based on the methods described in Robertson (1990). Many of these charts have been summarized in Lunne, Robertson and Powell (1997). The

Jefferies and Davies SBT chart shown in Figure 3c is based on the techniques discussed in Jefferies and Davies (1993) which introduced the concept of the Soil Behavior Type Index parameter, I_c. Please note that the I_c parameter developed by Robertson and Fear (1995) and Robertson and Wride (1998) is similar in concept but uses a slightly different calculation method than that used by Jefferies and Davies (1993) as the latter incorporates pore pressure in their technique through the use of the Bq parameter. The normalized Qtn SBT chart shown in Figure 4 is based on the work by Robertson (2009) utilizing a variable stress ratio exponent, n, for normalization based on a slightly modified redefinition and iterative approach for I_c. The boundary curves drawn on the chart are based on the work described in Robertson (2010).

Figure 5 shows a revised behavior based chart by Robertson (2016) depicting contractive-dilative zones. As the zones represent material behavior rather than soil gradation ConeTec has chosen a set of zone colors that are less likely to be confused with material type colors from previous SBT charts. These colors differ from those used by Dr. Robertson.

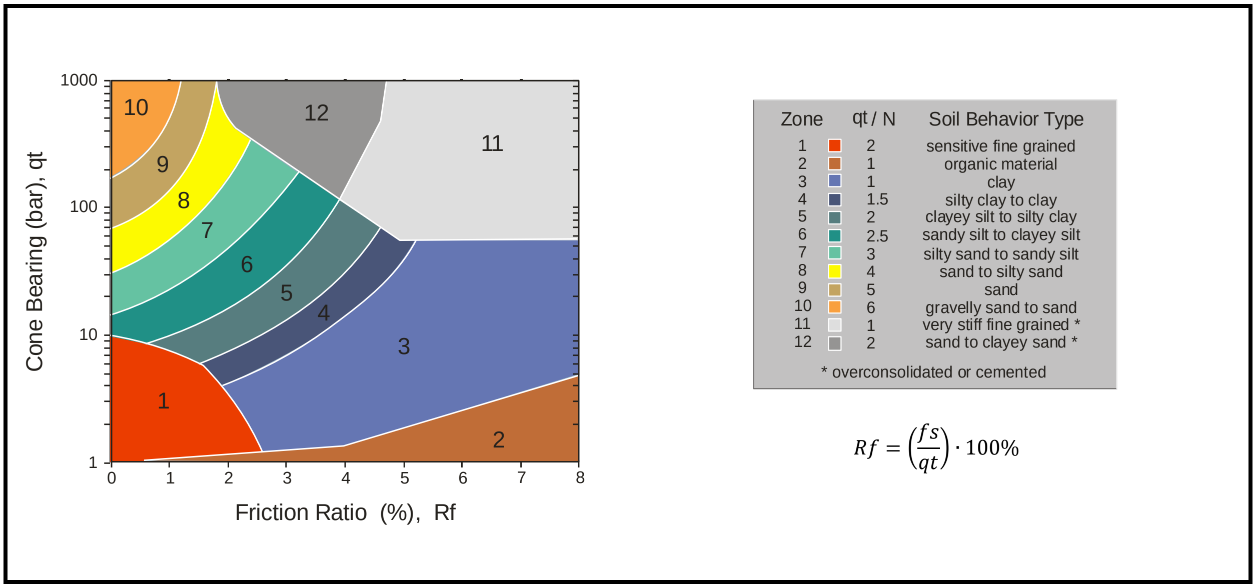


Figure 1. Non-Normalized Soil Behavior Type Classification Chart (SBT)

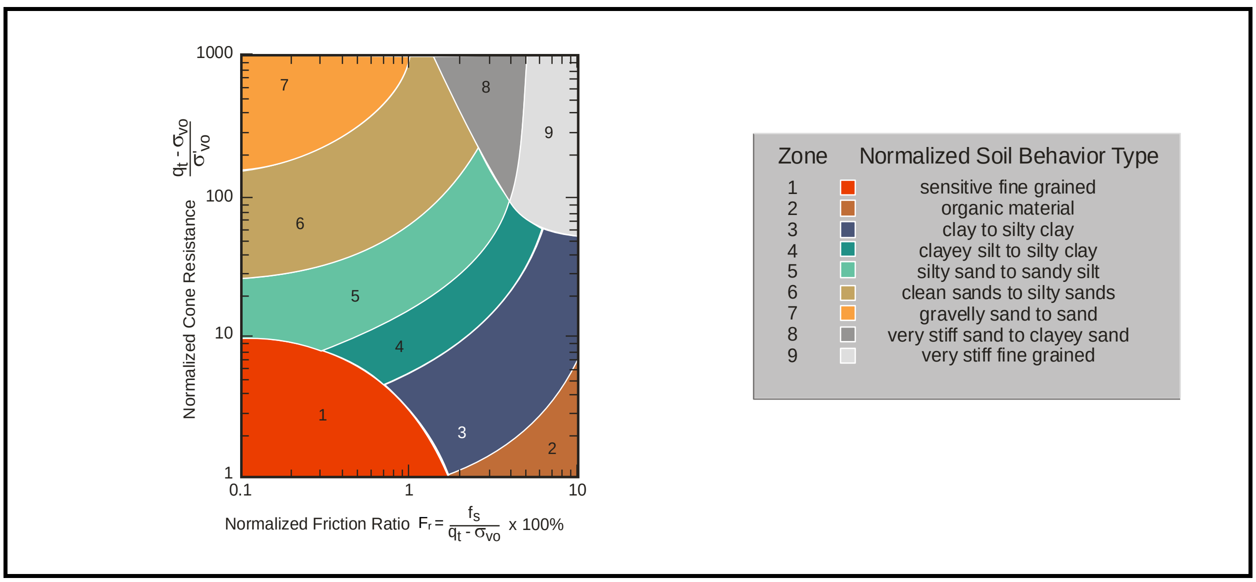


Figure 2. Normalized Soil Behavior Type Classification Chart (SBTn)

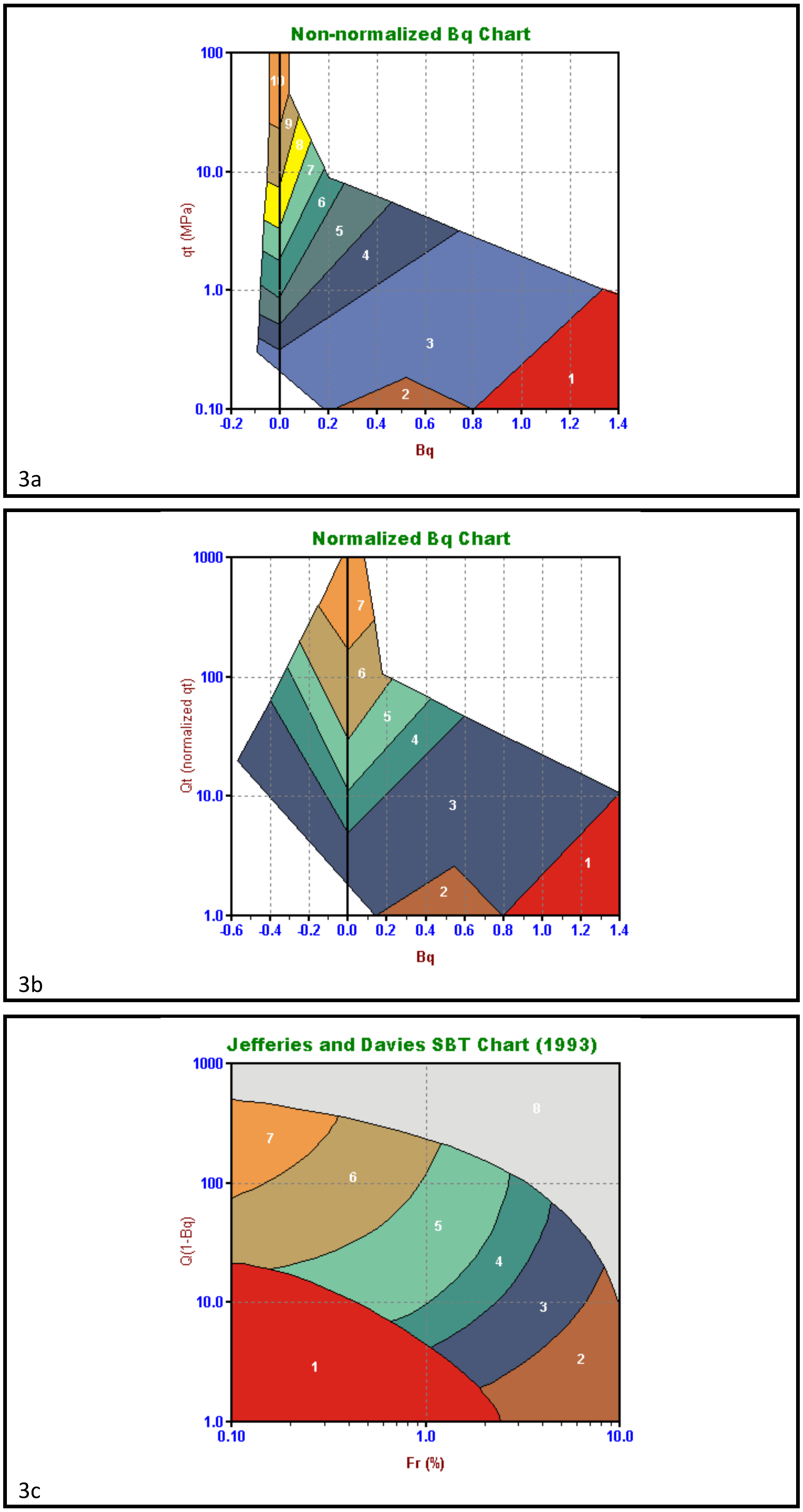


Figure 3. Alternate Soil Behavior Type Charts

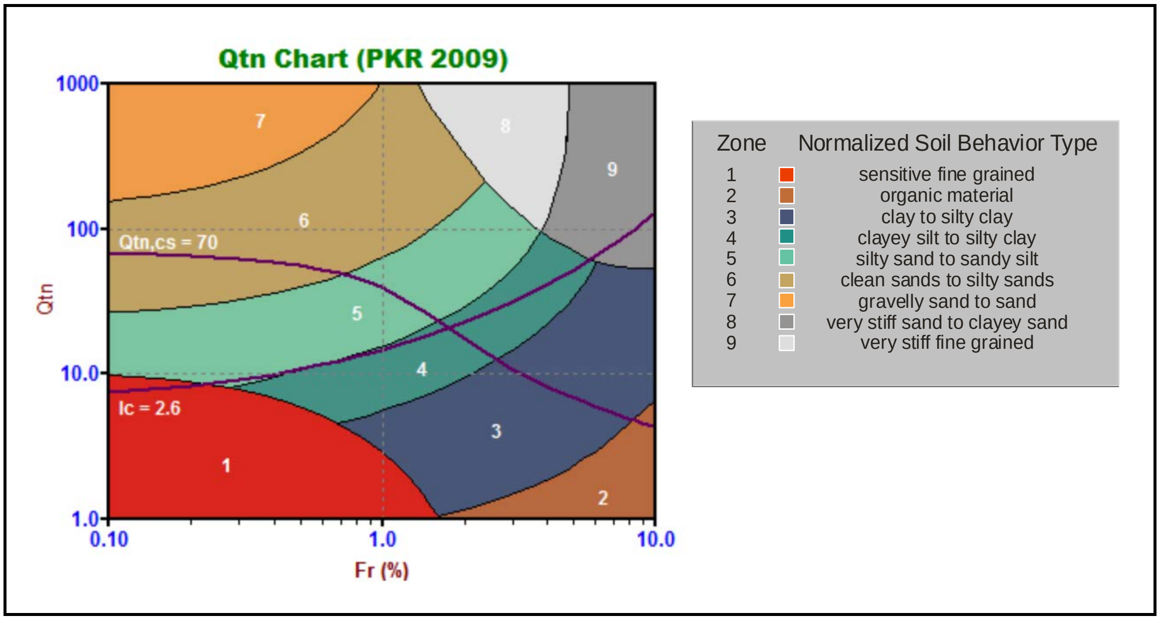


Figure 4. Normalized Soil Behavior Type Chart using Q_{tn} (SBT Qtn)

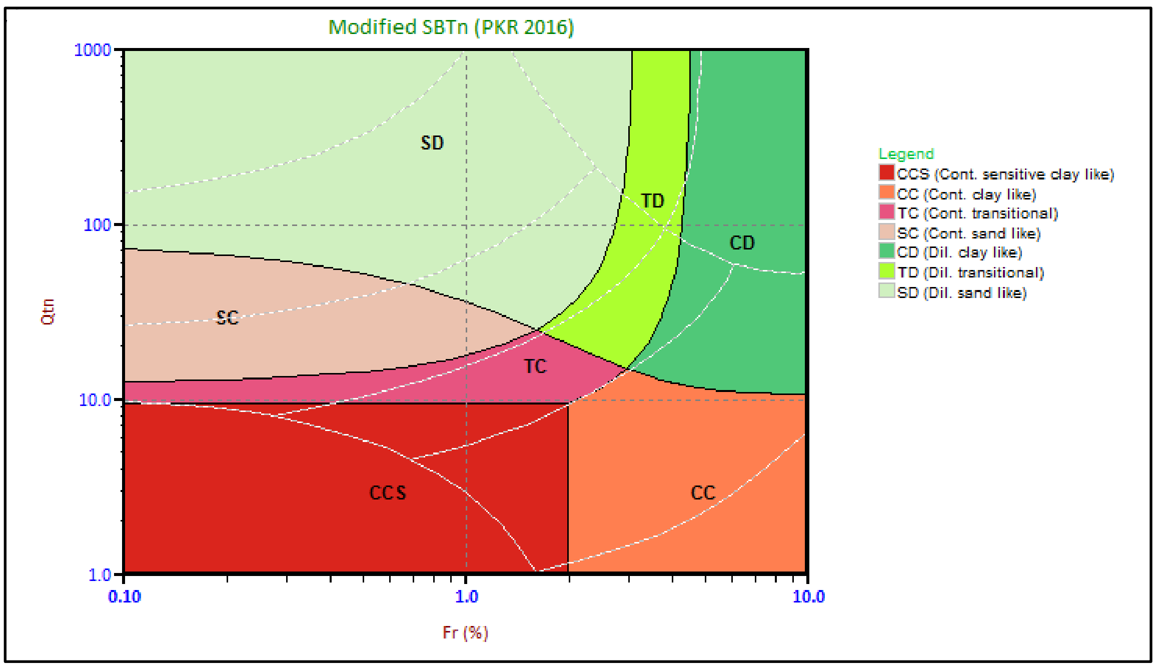


Figure 5. Modified SBTn Behavior Based Chart

Details regarding the geotechnical parameter calculations are provided in Tables 1a and 1b. The appropriate references cited are listed in Table 2. Non-liquefaction specific parameters are detailed in Table 1a and liquefaction specific parameters are detailed in Table 1b.

Where methods are based on charts or techniques that are too complex to describe in this summary the user should refer to the cited material. Specific limitations for each method are described in the cited material.

Where the results of a calculation/correlation are deemed *'invalid'* the value will be represented by the text strings "-9999", "-9999.0", the value 0.0 (Zero) or an empty cell. Invalid results will occur because of (and not limited to) one or a combination of:

1. Invalid or undefined CPT data (e.g. drilled out section or data gap).
2. Where the calculation method is inappropriate, for example, drained parameters in a material behaving as an undrained material (and vice versa).
3. Where input values are beyond the range of the referenced charts or specified limitations of the correlation method.
4. Where pre-requisite or intermediate parameter calculations are invalid.

The parameters selected for output from the program are often specific to a particular project. As such, not all of the calculated parameters listed in Table 1 may be included in the output files delivered with this report.

The output files are typically provided in Microsoft Excel XLS or XLSX format. The ConeTec software has several options for output depending on the number or types of calculated parameters desired or requested by the client. Each output file is named using the original COR file base name followed by a three or four letter indicator of the output set selected (e.g. BSC, TBL, NLI, NL2, IFI, IFI2) and possibly followed by an operator selected suffix identifying the characteristics of the particular calculation run.

Table 1a. CPT Parameter Calculation Methods – Non liquefaction Parameters

Calculated Parameter	Description	Equation	Ref
Depth	Mid Layer Depth *(where calculations are done at each point then Mid Layer Depth = Recorded Depth)*	*[Depth (Layer Top) + Depth (Layer Bottom)]/ 2.0*	CK*
Elevation	Elevation of Mid Layer based on sounding collar elevation supplied by client or through site survey	Elevation = Collar Elevation - Depth	CK*
Avg qc	Averaged recorded tip value (q_c)	$Avgqc = \frac{1}{n}\sum_{i=1}^{n} q_c$ *n=1 when calculations are done at each point*	CK*
Avg qt	Averaged corrected tip (q_t) where: $q_t = q_c + (1-a) \bullet u_2$	$Avgqt = \frac{1}{n}\sum_{i=1}^{n} q_t$ *n=1 when calculations are done at each point*	1
Avg fs	Averaged sleeve friction (f_s)	$Avgfs = \frac{1}{n}\sum_{i=1}^{n} fs$ *n=1 when calculations are done at each point*	CK*
Avg Rf	Averaged friction ratio (R_f) where friction ratio is defined as: $Rf = 100\% \bullet \frac{fs}{q_t}$	$AvgRf = 100\% \cdot \frac{Avgfs}{Avgqt}$ *n=1 when calculations are done at each point*	CK*
Avg u	Averaged dynamic pore pressure *(u)*	$Avgu = \frac{1}{n}\sum_{i=1}^{n} u_i$ *n=1 when calculations are done at each point*	CK*

Calculated Parameter	Description	Equation	Ref
Avg Res	Averaged Resistivity (this data is not always available since it is a specialized test requiring an additional module)	$AvgRes = \frac{1}{n}\sum_{i=1}^{n} Resistivity_i$ *n=1 when calculations are done at each point*	CK*
Avg UVIF	Averaged UVIF ultra-violet induced fluorescence (this data is not always available since it is a specialized test requiring an additional module)	$AvgUVIF = \frac{1}{n}\sum_{i=1}^{n} UVIF_i$ *n=1 when calculations are done at each point*	CK*
Avg Temp	Averaged Temperature (this data is not always available since it requires specialized calibrations)	$AvgTemp = \frac{1}{n}\sum_{i=1}^{n} Temperature_i$ *n=1 when calculations are done at each point*	CK*
Avg Gamma	Averaged Gamma Counts (this data is not always available since it is a specialized test requiring an additional module)	$AvgGamma = \frac{1}{n}\sum_{i=1}^{n} Gamma_i$ *n=1 when calculations are done at each point*	CK*
SBT	Soil Behavior Type as defined by Robertson et al 1986 (often referred to as Robertson and Campanella, 1986)	See Figure 1	1, 5
SBTn	Normalized Soil Behavior Type as defined by Robertson 1990 (linear normalization)	See Figure 2	2, 5
SBT-Bq	Non-normalized Soil Behavior type based on the Bq parameter	See Figure 3	1, 2, 5
SBT-Bqn	Normalized Soil Behavior based on the Bq parameter	See Figure 3	2, 5
SBT-JandD	Soil Behavior Type as defined by Jeffries and Davies	See Figure 3	7
SBT Qtn	Soil Behavior Type as defined by Robertson (2009) using a variable stress ratio exponent for normalization based on I_c	See Figure 4	15
Modified SBTn (contractive /dilative)	Modified SBTn chart as defined by Robertson (2016) indicating zones of contractive/dilative behavior.	See Figure 5	30
Unit Wt.	Unit Weight of soil determined from one of the following user selectable options: 1) uniform value 2) value assigned to each SBT zone 3) value assigned to each SBTn zone 4) value assigned to SBTn zone as determined from Robertson and Wride (1998) based on q_{c1n} 5) values assigned to SBT Qtn zones 6) Mayne fs (sleeve friction) method 7) Robertson 2010 method 8) user supplied unit weight profile The last option may co-exist with any of the other options	See references	3, 5, 15, 21, 24, 29

Calculated Parameter	Description	Equation	Ref
TStress σ_v	Total vertical overburden stress at Mid Layer Depth *A layer is defined as the averaging interval specified by the user where depths are reported at their respective mid-layer depth.* *For data calculated at each point layers are defined using the recorded depth as the mid-point of the layer. Thus, a layer starts half-way between the previous depth and the current depth unless this is the first point in which case the layer start is at zero depth. The layer bottom is half-way from the current depth to the next depth unless it is the last data point.* *Defining layers affects how stresses are calculated since the unit weight attributed to a data point is used throughout the entire layer. This means that to calculate the stresses the total stress at the top and bottom of a layer are required. The stress at mid layer is determined by adding the incremental stress from the layer top to the mid-layer depth. The stress at the layer bottom becomes the stress at the top of the subsequent layer. Stresses are NOT calculated from mid-point to mid-point.* *For over-water work the total stress due to the column of water above the mud line is taken into account where appropriate.*	$TStress = \sum_{i=1}^{n} \gamma_i h_i$ *where* γ_i is layer unit weight h_i is layer thickness	CK*
EStress σ_v'	Effective vertical overburden stress at mid-layer depth	$\sigma_v' = \sigma_v - u_{eq}$	CK*
Equil u u_{eq} or u_0	Equilibrium pore pressure determined from one of the following user selectable options: 1) hydrostatic below water table 2) user supplied profile 3) combination of those above When a user supplied profile is used/provided a linear interpolation is performed between equilibrium pore pressures defined at specific depths. If the profile values start below the water table then a linear transition from zero pressure at the water table to the first defined pointed is used. Equilibrium pore pressures may come from dissipation tests, adjacent piezometers or other sources. Occasionally, an extra equilibrium point ("assumed value") will be provided in the profile that does not come from a recorded value to smooth out any abrupt changes or to deal with material interfaces. These "assumed" values will be indicated on our plots and in tabular summaries.	For hydrostatic option: $u_{eq} = \gamma_w \cdot (D - D_{wt})$ where u_{eq} is equilibrium pore pressure γ_w is unit weight of water D is the current depth D_{wt} is the depth to the water table	CK*
K_0	Coefficient of earth pressure at rest, K_0	$K_0 = (1 - \sin\Phi') \, OCR^{\sin\Phi'}$	17
C_n	Overburden stress correction factor used for $(N_1)_{60}$ and older CPT parameters	$C_n = (P_a/\sigma_v')^{0.5}$ where $0.0 < C_n < 2.0$ *(user adjustable, typically 1.7)* P_a *is atmospheric pressure (100 kPa)*	12
C_q	Overburden stress normalizing factor	$C_q = 1.8 / (0.8 + (\sigma_v'/P_a))$ where $0.0 < C_q < 2.0$ *(user adjustable)* P_a *is atmospheric pressure (100 kPa)*	3, 12

Calculated Parameter	Description	Equation	Ref
N_{60}	SPT N value at 60% energy calculated from q_t/N ratios assigned to each SBT zone. This method has abrupt N value changes at zone boundaries.	*See Figure 1*	5
(N1)60	SPT N_{60} value corrected for overburden pressure	$(N_1)_{60} = C_n \cdot N_{60}$	4
N60Ic	SPT N_{60} values based on the I_c parameter [as defined by Roberston and Wride 1998 (5), or by Robertson 2009 (15)].	$(q_t/P_a)/ N_{60} = 8.5\ (1 - I_c/4.6)$ $(q_t/P_a)/ N_{60} = 10^{(1.1268 - 0.2817Ic)}$ Pa being atmospheric pressure	5 15, 31
(N1)60Ic	SPT N_{60} value corrected for overburden pressure (using N_{60} I_c). User has 3 options.	1) $(N_1)_{60}Ic = C_n \cdot (N_{60}\, I_c)$ 2) $q_{c1n}/ (N_1)_{60}I_c = 8.5\ (1 - I_c/4.6)$ 3) $(Q_{tn})/ (N_1)_{60}I_c = 10^{(1.1268 - 0.2817Ic)}$	4 5 15, 31
Su or Su (Nkt)	Undrained shear strength based on q_t S_u factor N_{kt} is user selectable	$Su = \frac{qt - \sigma_v}{N_{kt}}$	1, 5
Su or Su (Ndu)	Undrained shear strength based on pore pressure S_u factor $N_{\Delta u}$ is user selectable	$Su = \frac{u_2 - u_{eq}}{N_{\Delta u}}$	1, 5
Dr	Relative Density determined from one of the following user selectable options: a) Ticino Sand b) Hokksund Sand c) Schmertmann (1978) d) Jamiolkowski (1985) - All Sands e) Jamiolkowski et al (2003) (various compressibilities, K_o)	See reference (methods a through d) Jamiolkowski et al (2003) reference	5 14
PHI ϕ	Friction Angle determined from one of the following user selectable options (methods a through d are for sands and method e is for silts and clays): a) Campanella and Robertson b) Durgunoglu and Mitchel c) Janbu d) Kulhawy and Mayne e) NTH method (clays and silts)	See appropriate reference	5 5 5 11 23
Delta U/qt	Differential pore pressure ratio (older parameter used before B_q was established)	$= \frac{\Delta u}{qt}$ *where:* $\Delta u = u - u_{eq}$ *and* *u = dynamic pore pressure* u_{eq} *= equilibrium pore pressure*	CK*
Bq	Pore pressure parameter	$Bq = \frac{\Delta u}{qt - \sigma_v}$ where : $\Delta u = u - u_{eq}$ *and* *u = dynamic pore pressure* u_{eq} *= equilibrium pore pressure*	1, 2, 5
Net qt or qtNet	Net tip resistance (used in many subsequent correlations)	$qt - \sigma_v$	CK*
qe	Effective tip resistance (using the dynamic pore pressure u_2 and not equilibrium pore pressure)	$qt - u_2$	CK*

Calculated Parameter	Description	Equation	Ref
qeNorm	Normalized effective tip resistance	$\frac{qt - u_2}{\sigma'_v}$	CK*
Q_t or Norm: Qt	Normalized q_t for Soil Behavior Type classification as defined by Robertson (1990) using a linear stress normalization. Note this is different from Q_{tn}.	$Qt = \frac{qt - \sigma_v}{\sigma'_v}$	2, 5
F_r or Norm: Fr	Normalized Friction Ratio for Soil Behavior Type classification as defined by Robertson (1990)	$Fr = 100\% \cdot \frac{fs}{qt - \sigma_v}$	2, 5
Q(1-Bq)	Q(1-Bq) grouping as suggested by Jefferies and Davies for their classification chart and the establishment of their I_c parameter	$Q \cdot (1 - Bq)$ *where Bq is defined as above and Q is the same as the normalized tip resistance, Q_t, defined above*	6, 7
qc1	Normalized tip resistance, q_{c1}, using a fixed stress ratio exponent, n (this method has stress units)	$q_{c1} = q_t \cdot (Pa/\sigma_v')^{0.5}$ where: Pa = atmospheric pressure	21
qc1 (0.5)	Normalized tip resistance, q_{c1}, using a fixed stress ratio exponent, n (this method is unit-less)	$q_{c1}\,(0.5) = (q_t/P_a) \cdot (Pa/\sigma_v')^{0.5}$ where: Pa = atmospheric pressure	5
qc1 (Cn)	Normalized tip resistance, q_{c1}, based on C_n (this method has stress units)	$q_{c1}(Cn) = C_n * q_t$	5, 12
qc1 (Cq)	Normalized tip resistance, q_{c1}, based on C_q (this method has stress units)	$q_{c1}\,(Cq) = C_q * q_t$ (some papers use q_c)	5, 12
qc1n	normalized tip resistance, q_{c1n}, using a variable stress ratio exponent, n (where n=0.0, 0.70, 1.0) (this method is unit-less)	$q_{c1n} = (q_t / P_a)(P_a/\sigma_v')^n$ where: P_a = atm. Pressure and n varies as described below	3, 5
I_c or Ic (RW1998)	Soil Behavior Type Index as defined by Robertson and Fear (1995) and Robertson and Wride (1998) for estimating grain size characteristics and providing smooth gradational changes across the SBTn chart	$I_c = [(3.47 - \log_{10}Q)^2 + (\log_{10} Fr + 1.22)^2]^{0.5}$ *Where:* $Q = \left(\frac{qt - \sigma_v}{P_a}\right)\left(\frac{P_a}{\sigma'_v}\right)^n$ *Or* $Q = q_{c1n} = \left(\frac{qt}{P_a}\right)\left(\frac{P_a}{\sigma'_v}\right)^n$ *depending on the iteration in determining I_c* *And Fr is in percent* *P_a = atmospheric pressure* *n varies between 0.5, 0.70 and 1.0 and is selected in an iterative manner based on the resulting I_c*	3, 5, 21
Ic (PKR 2009)	Soil Behavior Type Index, I_c (PKR 2009) based on a variable stress ratio exponent n, which itself is based on I_c (PKR 2009). An iterative calculation is required to determine Ic (PKR 2009) and its corresponding n (PKR 2009).	I_c (PKR 2009) = $[(3.47 - \log_{10}Q_{tn})^2 + (1.22 + \log_{10}F_r)^2]^{0.5}$	15

Calculated Parameter	Description	Equation	Ref
n (PKR 2009)	Stress ratio exponent n, based on I_c (PKR 2009). An iterative calculation is required to determine n (PKR 2009) and its corresponding Ic (PKR 2009).	$n\ (PKR\ 2009) = 0.381\ (I_c) + 0.05\ (\sigma_v'/P_a) - 0.15$	15
Qtn (PKR 2009)	Normalized tip resistance using a variable stress ratio exponent based on I_c (PKR 2009) and n (PKR 2009). An iterative calculation is required to determine Qtn (PKR 2009).	$Q_{tn} = [(qt - \sigma_v)/P_a](P_a/\sigma_v')^n$ where P_a = atmospheric pressure (100 kPa) n = stress ratio exponent described above	15
FC	Apparent fines content (%)	$FC=1.75(Ic^{3.25}) - 3.7$ $FC=100$ for $I_c > 3.5$ $FC=0$ for $I_c < 1.26$ $FC = 5\%$ if $1.64 < I_c < 2.6$ AND $F_r<0.5$	3
I_c Zone	This parameter is the Soil Behavior Type zone based on the I_c parameter (valid for zones 2 through 7 on SBTn or SBT Qtn charts)	$I_c < 1.31$ Zone = 7 $1.31 < I_c < 2.05$ Zone = 6 $2.05 < I_c < 2.60$ Zone = 5 $2.60 < I_c < 2.95$ Zone = 4 $2.95 < I_c < 3.60$ Zone = 3 $I_c > 3.60$ Zone = 2	3
State Param or State Parameter or ψ	The state parameter index, ψ, is defined as the difference between the current void ratio, e, and the critical void ratio, e_c. Positive ψ - contractive soil Negative ψ - dilative soil This is based on the work by Been and Jefferies (1985) and Plewes, Davies and Jefferies (1992) - vertical effective stress is used rather than a mean normal stress	See reference	6, 8
Yield Stress σ_p'	Yield stress is calculated using the following methods a) General method b) 1st order approximation using q_tNet (clays) c) 1st order approximation using Δu_2 (clays) d) 1st order approximation using q_e (clays)	All stresses in kPa a) $\sigma_p' = 0.33\cdot(q_t - \sigma_v)^{m'}\ (\sigma_{atm}/100)^{1-m'}$ where $m' = 1 - \frac{0.28}{1+(I_c/2.65)^{25}}$ b) $\sigma_p' = 0.33\cdot(q_t - \sigma_v)$ c) $\sigma_p' = 0.54\cdot(\Delta u_2)$ $\Delta u_2 = u_2 - u_0$ d) $\sigma_p' = 0.60\cdot(q_t - u_2)$	 19 20 20 20
OCR OCR(JS1978) OCR(Mayne2014) OCR (qtNet) OCR (deltaU) OCR (qe) OCR (Vs) OCR (PKR2015)	Over Consolidation Ratio based on a) Schmertmann (1978) method involving a plot plot of S_u/σ_v' /($S_u/\sigma_v')_{NC}$ and OCR b) based on Yield stresses described above c) approximate version based on qtNet d) approximate version based on Δu e) approximate version based on effective tip, q_e f) approximate version based on shear wave velocity, V_s g) based on Qt	 a) requires a user defined value for NC S_u/P_c' ratio b through f) *based on yield stresses* g) OCR = $0.25\cdot(Qt)^{1.25}$	 9 19 20 20 20 18 32

Calculated Parameter	Description	Equation	Ref
Es/qt	Intermediate parameter for calculating Young's Modulus, E, in sands. It is the Y axis of the reference chart.	Based on Figure 5.59 in the reference	5
Es Young's Modulus E	Young's Modulus based on the work done in Italy. There are three types of sands considered in this technique. The user selects the appropriate type for the site from: a) OC Sands b) Aged NC Sands c) Recent NC Sands Each sand type has a family of curves that depend on mean normal stress. The program calculates mean normal stress and linearly interpolates between the two extremes provided in the E_s/q_t chart. Es is evaluated for an axial strain of 0.1%.	Mean normal stress is evaluated from: $\sigma'_m = \frac{1}{3}(\sigma'_v + \sigma'_h + \sigma'_h)^3$ where σ_v'= vertical effective stress σ_h'= horizontal effective stress *and* $\sigma_h = K_o \bullet \sigma_v'$ with K_o assumed to be 0.5	5
Delta U/TStress	Differential pore pressure ratio with respect to total stress	$= \frac{\Delta u}{\sigma_v}$ *where:* $\Delta u = u - u_{eq}$	CK*
Delta U/Estress, P Value, Excess Pore Pressure Ratio	Differential pore pressure ratio with respect to effective stress. Key parameter (P, Normalized Pore Pressure Parameter, Excess Pore Pressure Ratio) in the Winckler et. al. static liquefaction method.	$= \frac{\Delta u}{\sigma'_v}$ *where:* $\Delta u = u - u_{eq}$	25, 25a, CK*
Su/EStress	Undrained shear strength ratio with respect to vertical effective overburden stress using the S_u (N_{kt}) method	$= Su (N_{kt}) / \sigma_v'$	CK*
Gmax	G_{max} determined from SCPT shear wave velocities (not estimated values)	$G_{max} = \rho V_s^2$ where ρ is the mass density of the soil determined from the estimated unit weights at each test depth	27
qtNet/Gmax	Net tip resistance ratio with respect to the small strain modulus G_{max} determined from SCPT shear wave velocities (not estimated values)	$= (qt - \sigma_v) / G_{max}$ where $G_{max} = \rho V_s^2$ and ρ is the mass density of the soil determined from the estimated unit weights at each test depth	15, 28, 30

*CK – common knowledge

Table 1b. CPT Parameter Calculation Methods – Liquefaction Parameters

Calculated Parameter	Description	Equation	Ref
K_{SPT}	Equivalent clean sand factor for $(N_1)60$	$K_{SPT} = 1 + ((0.75/30) \bullet (FC - 5))$	10
K_{CPT} or K_C (RW1998)	Equivalent clean sand correction for q_{c1N}	$K_{cpt} = 1.0$ for $I_c \leq 1.64$ $K_{cpt} = f(I_c)$ for $I_c > 1.64$ (see reference) $K_c = -0.403\ I_c^4 + 5.581\ I_c^3 - 21.63 I_c^2 + 33.75\ I_c - 17.88$	3, 10
Kc (PKR 2010)	Clean sand equivalent factor to be applied to Q_{tn}	$K_c = 1.0$ for $I_c \leq 1.64$ $K_c = -0.403\ I_c^4 + 5.581\ I_c^3 - 21.63 I_c^2 + 33.75\ I_c - 17.88$ for $I_c > 1.64$	16
$(N_1)_{60cs}Ic$	Clean sand equivalent SPT $(N_1)_{60}I_c$. User has 3 options.	1) $(N_1)_{60cs}Ic = \alpha + \beta((N_1)_{60}I_c)$ 2) $(N_1)_{60cs}Ic = K_{SPT} * ((N_1)_{60}I_c)$ 3) $(q_{c1ncs})/ (N_1)_{60cs}I_c = 8.5\ (1 - I_c/4.6)$ $FC \leq 5\%$: $\alpha = 0$, $\beta=1.0$ $FC \geq 35\%$ $\alpha = 5.0$, $\beta=1.2$ $5\% < FC < 35\%$ $\alpha = \exp[1.76 - (190/FC^2)]$ $\beta = [0.99 + (FC^{1.5}/1000)]$	10 10 5
q_{c1ncs}	Clean sand equivalent q_{c1n}	$q_{c1ncs} = q_{c1n} \bullet K_{cpt}$	3
Qtn,cs (PKR 2010)	Clean sand equivalent for Q_{tn} described above - Q_{tn} being the normalized tip resistance based on a variable stress exponent as defined by Robertson (2009)	$Q_{tn,cs} = Q_{tn} \cdot K_c$ (PKR 2016)	16
Su(Liq)/ESv	Liquefied shear strength ratio as defined by Olson and Stark	$\dfrac{Su(Liq)}{\sigma_v'} = 0.03 + 0.0143(q_{c1})$ Note: σ_v' and s_v' are synonymous	13
Su(Liq)/ESv (PKR 2010)	Liquefied shear strength ratio as defined by Robertson (2010)	$\dfrac{Su(Liq)}{\sigma_v'}$ Based on a function involving $Q_{tn,cs}$	16
Su (Liq) (PKR 2010)	Liquefied shear strength derived from the liquefied shear strength ratio and effective overburden stress		16
Cont/Dilat Tip	Contractive / Dilative qc1 Boundary based on $(N_1)_{60}$	$(\sigma_v')_{boundary} = 9.58 \times 10^{-4}\ [(N_1)_{60}]^{4.79}$ *qc1* is calculated from specified qt(MPa)/N ratio	13
CRR	Cyclic Resistance Ratio (for Magnitude 7.5)	$q_{c1ncs} < 50$: $CRR_{7.5} = 0.833\ [q_{c1ncs}/1000] + 0.05$ $50 \leq q_{c1ncs} < 160$: $CRR_{7.5} = 93\ [q_{c1ncs}/1000]^3 + 0.08$	10
Kg	Small strain Stiffness Ratio Factor, Kg	$[Gmax/qt]/[qc1n^{-m}]$ m = empirical exponent, typically 0.75	26

Calculated Parameter	Description	Equation	Ref
SP Distance	State Parameter Distance, Winckler static liquefaction method	Perpendicular distance on Qtn chart from plotted point to state parameter Ψ = -0.05 curve	25
URS NP Fr	Normalized friction ratio point on Ψ = -0.05 curve used in SP Distance calculation		25
URS NP Qtn	Normalized tip resistance (Qtn) point on Ψ = -0.05 curve used in SP Distance calculation		25

Table 2. References

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APPENDIX D

GEOPHYSICAL SURVEYS

D.1 INTRODUCTION

This appendix presents the results of the surface wave geophysical investigation performed in support of soil site class characterization and ground motion development for the design of a mixed-use tower and podium structure with 42 above-ground stories in the tower section and 5 stories in the podium. The structure plans also include 4 subgrade parking levels. The planned structure is located at 708 S Cloverdale Avenue in Los Angeles, California. The geophysical investigation consisted of surface wave surveys using Multichannel Analysis of Surface Waves (MASW) and Refraction Microtremor (ReMi) methods. The geophysical measurements were performed along three survey lines (SW22-1 through SW22-3) at the locations shown in Figure D-1. The purpose of the geophysical surveys was to measure seismic shear-wave (S-wave) velocities at a range of depths to evaluate foundation properties (i.e. V_{S30}) at the site. The geophysical data were collected and processed by an assistant project scientist under the supervision of a California-licensed Professional Geophysicist.

D.2 SURFACE WAVE GEOPHYSICAL METHODS

Both active and passive surface wave surveys were performed at the site. The active surface wave surveys were performed using MASW methods, and the passive surveys were performed using ReMi methods. A detailed description of MASW is provided in Park et al. (1999), and ReMi is described in Louie (2001) and Louie et al. (2021).

In general, the surface wave method records Rayleigh waves generated either with (1) an active source (e.g. sledgehammer) for the MASW method or (2) a passive (ambient) source (e.g. vehicular traffic) for the ReMi method. In a layered medium, Rayleigh surface waves of different frequencies (or wavelengths) propagate at different velocities, referred to as phase velocity. This phase velocity primarily depends on the material stiffness properties (e.g. S-wave velocity) over a depth approximately equal to one wavelength. Consequently, lower frequency, longer wavelength surface wave energy will provide samples to greater survey depths than higher frequency, shorter wavelength energy. Because surface waves of different frequencies (wavelengths) sample different depths, they travel at different velocities (dispersion) in a layered medium. Surface wave geophysical surveys measure the dispersive nature of the geologic medium and produce dispersion curves, which show the variation of Rayleigh wave phase velocity as a function of frequency (or wavelength). Due to the generally lower frequency nature of passive surface wave energy, passive surface wave techniques (i.e. ReMi) have the potential to supplement active surface wave data to achieve deeper investigation depths. For this reason, it is advantageous to perform both types of measurement along the same lines as was done for this project.

After the dispersion curve is generated, the dispersion curve picks are then iteratively fitted to a horizontally layered, laterally continuous, homogeneous-isotropic, S-wave velocity model that would account for the measured surface wave velocity dispersion. The results provide a representative average estimate of the one-dimensional S-wave velocity profile under the array (velocity vs. depth).

D.3 SURFACE WAVE GEOPHYSICAL PROCEDURES

The MASW and ReMi investigations were performed at the site on March 10, 2022. These measurements were collected using a Geometrics Geode seismograph with a linear array of 24 4.5-Hz geophones. As shown on Figure D-1, the three survey lines were performed within the currently existing parking lot. MASW and ReMi measurements were collected with geophones spaced at 10-foot intervals for lines SW22-1 and SW22-2 (total line length of 230 feet) and at 5-foot intervals for line SW22-3 (line length of 115 feet).

For the MASW measurements, the active seismic source consisted of a sledgehammer blow to a ground plate. Shots were performed at 10-foot intervals starting 10 to 20 feet behind the first geophone and finishing 20 to 30 feet in front of the first geophone for lines SW22-1 and SW22-2 (five hit locations). For line SW22-3, shots were performed at 5-foot intervals from 15 feet behind the first geophone to 10 feet in front of the first geophone (six hit locations). At each shot location, the sledgehammer was hit three times and the resultant waveforms for each shot were stacked. A one-second-long record with 0.5 millisecond sample interval was recorded at each shot location. The recorded MASW data were subsequently processed using the program SurfSeis by Kansas Geological Survey. This program uses a modified F-K filter (type of 2-dimensional Fourier transform) to convert the raw seismic data from time and displacement to wave frequency and velocity. The highest amplitude energies along the frequency and phase velocity plot for each shot location were then selected to create a dispersion curve.

Because of the typical lower frequency nature of passive surface wave energy, ReMi measurements were performed to supplement the MASW measurements to deeper investigation depths. A total of ten 32-second-long ReMi records (2 millisecond sample interval) were collected at each survey location along the same geophone arrays that were used for MASW data collection. The source of ambient surface wave energy was primarily vehicular traffic within the neighborhood. The recorded ReMi data were also processed using the Kansas Geological Survey's SurfSeis program. After examining the ReMi records individually to determine which records had sufficient energy to pick a dispersion curve, the curves with the best data were stacked together in the SurfSeis program. Wavefield transformation was then performed on the stacked ReMi records in a similar manner to the MASW processing to create a frequency/phase velocity plot. An overall ReMi dispersion curve was then created from this plot.

For each line, the ReMi dispersion curve picks were combined with the dispersion curve picks generated from MASW to create an overall seismic dispersion curve. The degree of fit of the overlapping ReMi and MASW dispersion picks provided confidence in the results. Additionally, as noted above, the ReMi and MASW data complement each other by generally sampling different frequency ranges of surface wave data. After the data were combined, a best fit polynomial dispersion curve was calculated for modeling. The best fit dispersion curve was then iteratively fitted to a one-dimensional S-wave velocity model using the SurfSeis software. The results provide a one-dimensional vertical profile of S-wave velocity as a function of depth averaged beneath the extent of the survey line.

D.4 SURFACE WAVE GEOPHYSICAL RESULTS

The results of the combined MASW and ReMi surface wave measurements are presented in Figures D-2 through D-4 for lines SW22-1 through SW22-3, respectively. These figures present the MASW, ReMi, and best fit surface wave dispersion curves and the corresponding representative S-wave velocity models. As seen in these figures, the MASW and ReMi dispersion curves are generally in good agreement in the regions that overlap.

Figure D-5 summarizes the surface wave measurement results for the site. This figure shows (1) the S-wave velocity models for lines SW22-1 through SW22-3 plotted as a function of depth below ground surface and (2) the site average S-wave velocity for all the measurements calculated at 1-foot increments.

Based on the results shown on Figure D-5, the V_{S30} was calculated based on the procedures outlined in the National Earthquake Hazards Reduction Program (NEHRP) and UBC. The V_{S30} was calculated from the following equation from these references:

$$v_s = \frac{\sum_{i=1}^{n} d_i}{\sum_{i=1}^{n} \frac{d_i}{v_{si}}}$$

where:

i = distinct different soil and/or rock layer between 1 and n
v_{si} = shear wave velocity in feet per second of layer i
d_i = thickness of any layer within the 100-foot interval
$\sum_{i=1}^{n} d_i$ = 100 feet

Based on this procedure, the site average V_{S30} was calculated from ground surface to 100 feet below ground surface. The V_{S30} below ground surface was calculated as 993 ft/s (303 m/s), which corresponds with NEHRP Site Class D, stiff soil ($600 < V_{S30} \leq 1{,}200$ ft/s). V_{S30} values for depth intervals beginning below ground surface are also shown on Figure D-5.

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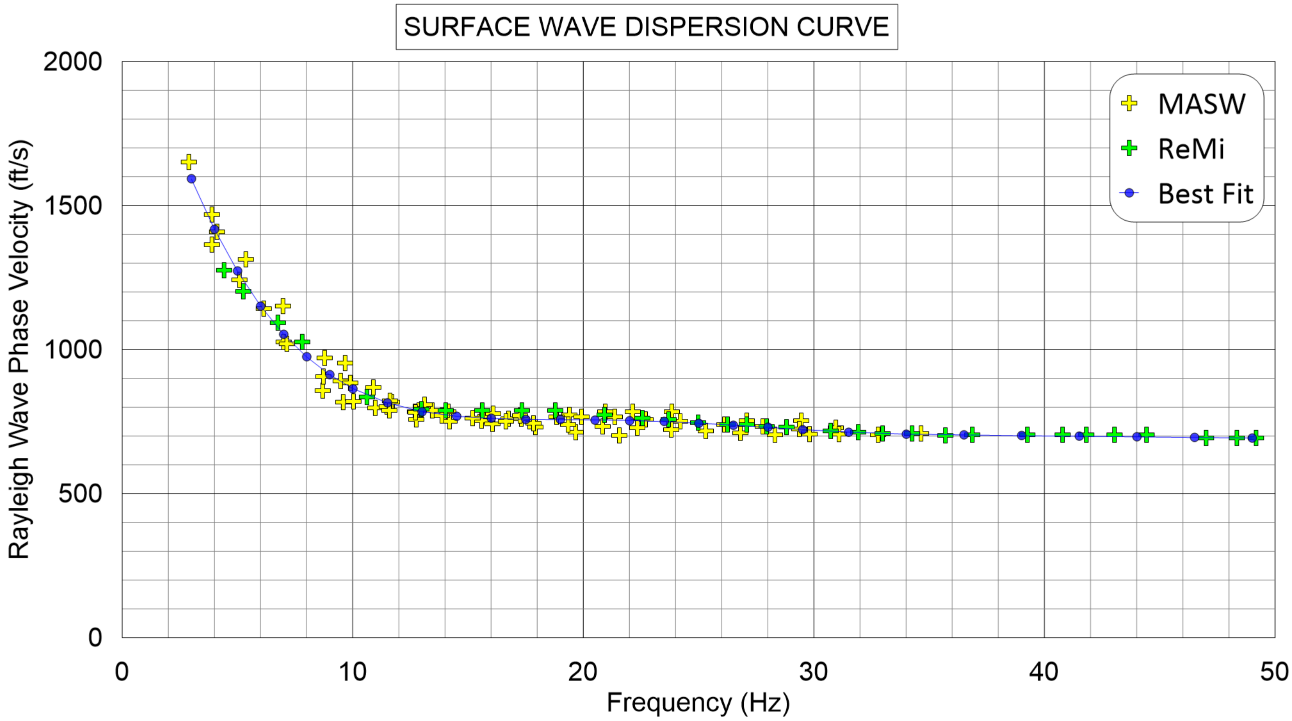
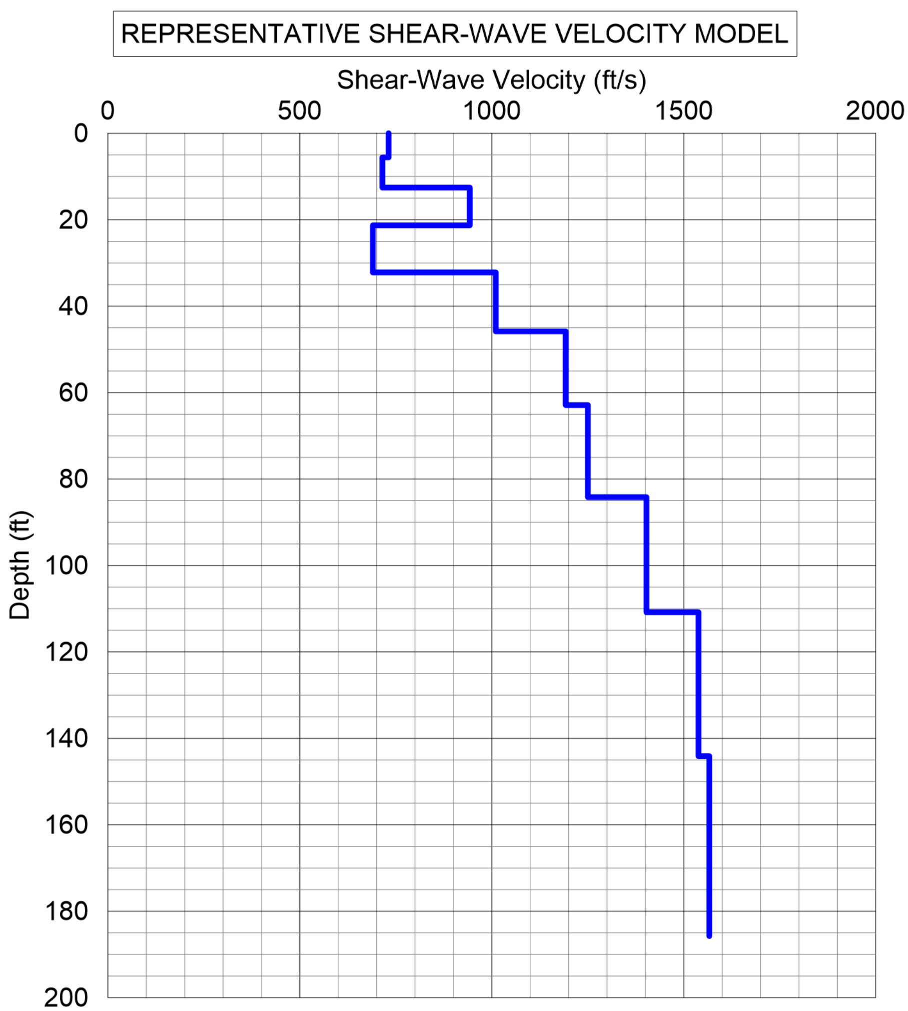
GEOPHYSICAL LINE LOCATIONS

GeoPentech	Project No.: **22010A**	Project: **708 S Cloverdale Ave. Development**	Date: **MAR 2022**	Figure: **D-1**



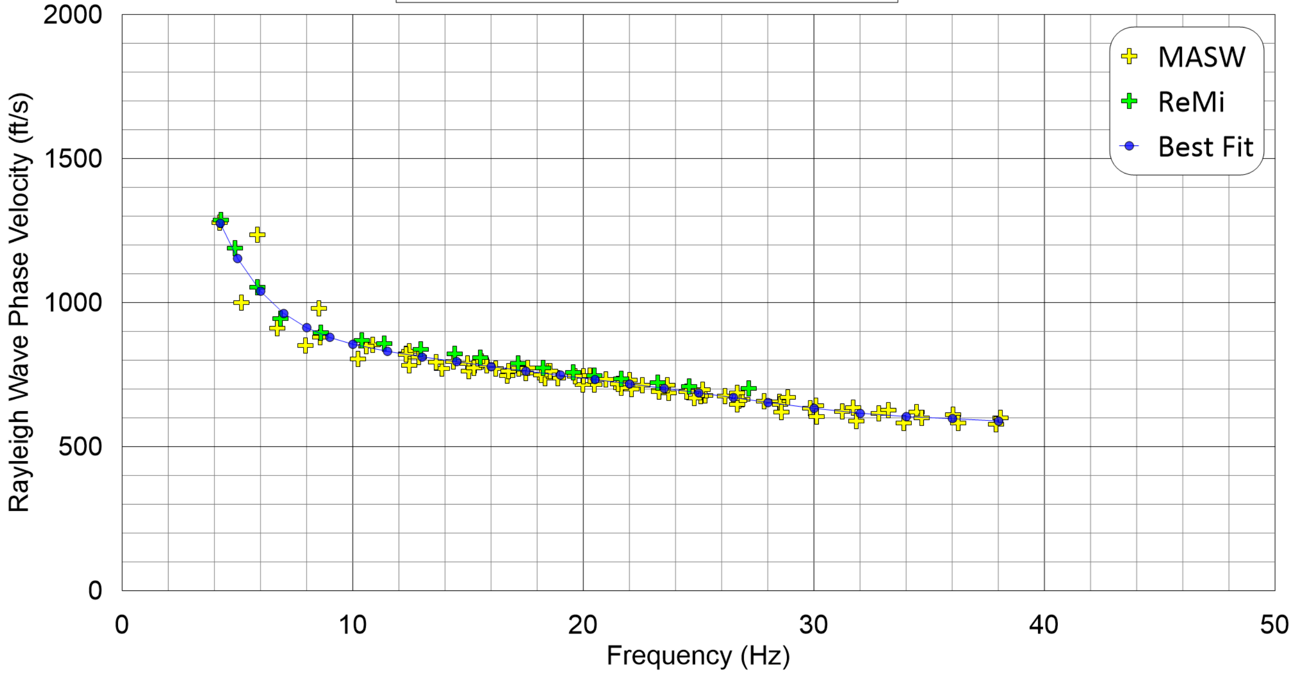


SW22-1: S-WAVE VELOCITY COMBINED SOURCE MODEL

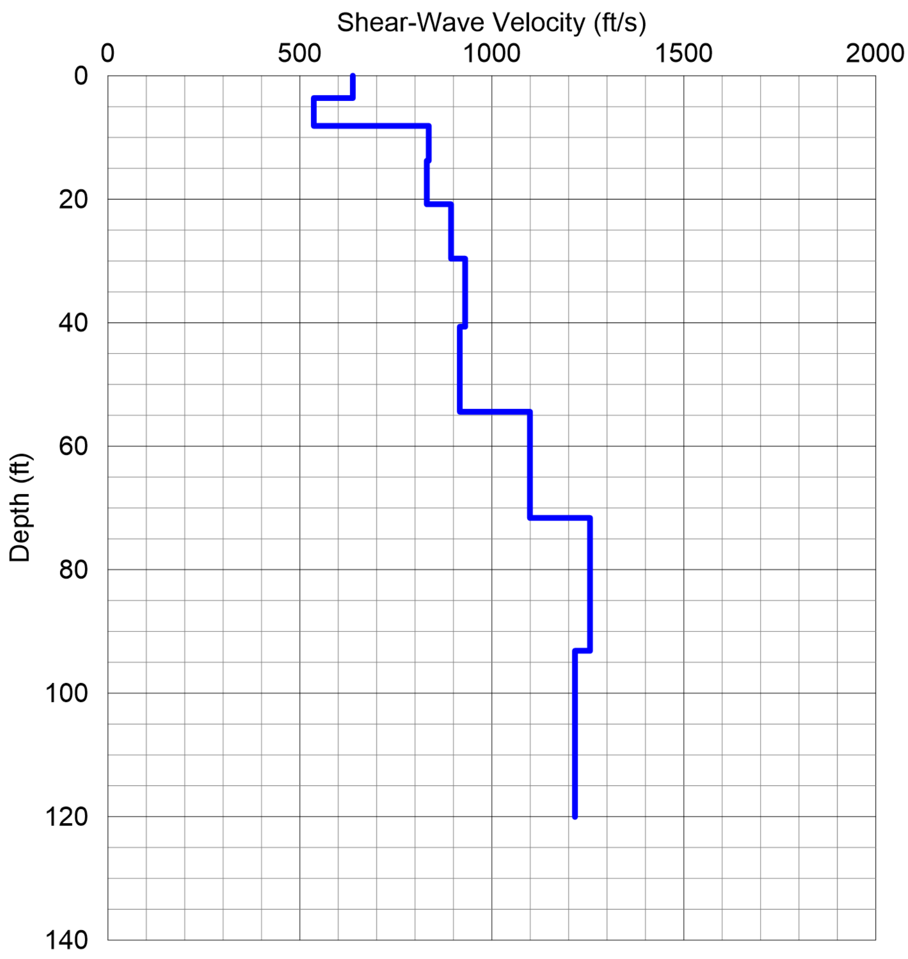
Project No.: 22010A	Project: 708 S Cloverdale Ave. Development	Date: MAR 2022	Figure D-2

GeoPentech

SURFACE WAVE DISPERSION CURVE



REPRESENTATIVE SHEAR-WAVE VELOCITY MODEL

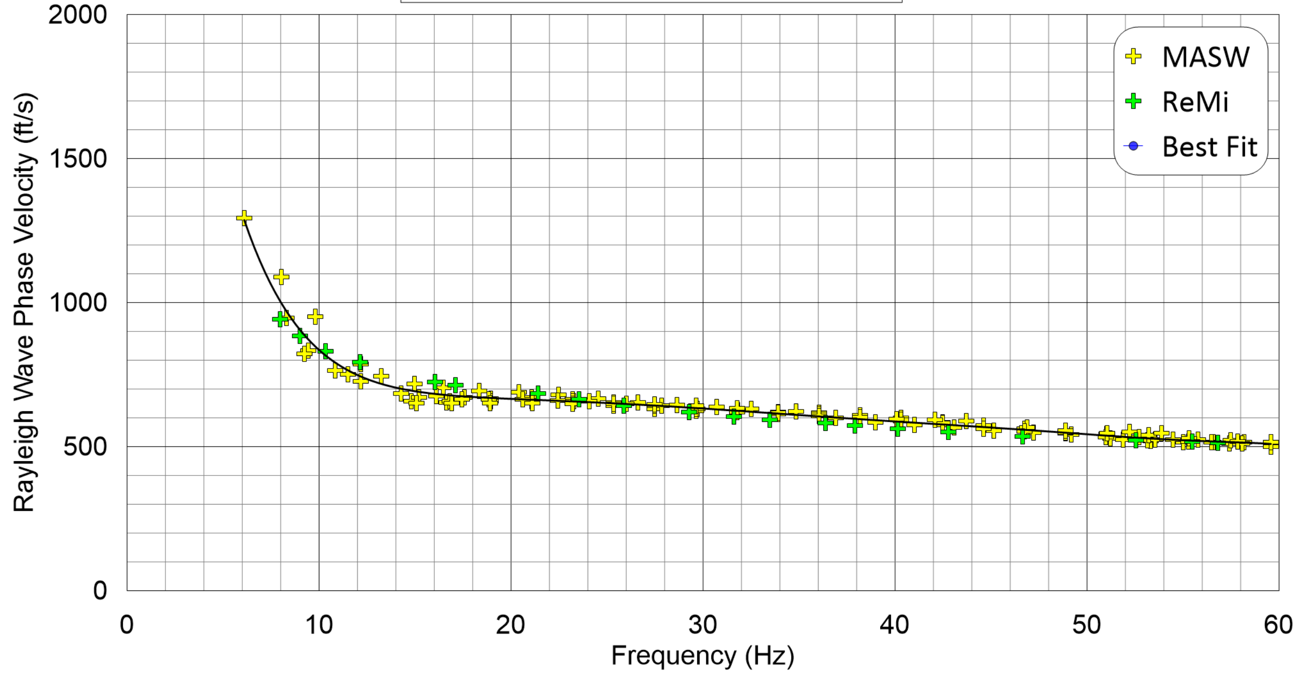


SW22-2: S-WAVE VELOCITY COMBINED SOURCE MODEL

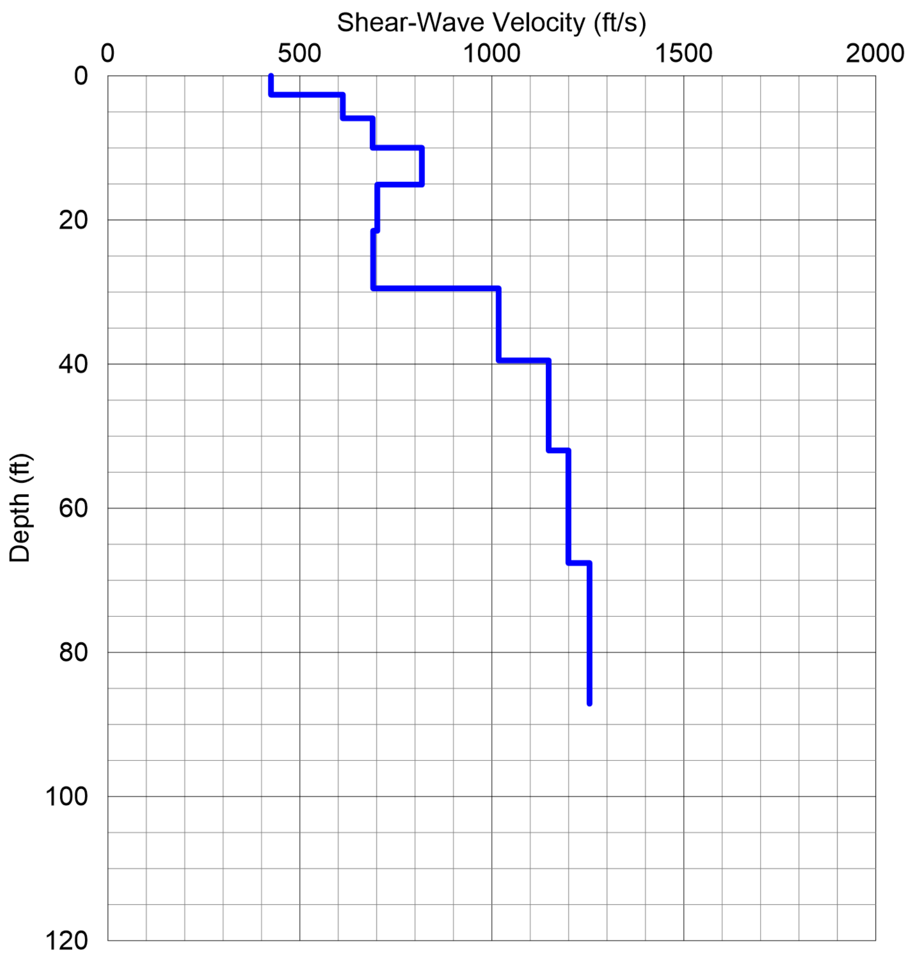
Project No.: 22010A	Project: 708 S Cloverdale Ave. Development	Date: MAR 2022	Figure D-3

GeoPentech

SURFACE WAVE DISPERSION CURVE



REPRESENTATIVE SHEAR-WAVE VELOCITY MODEL



SW22-3: S-WAVE VELOCITY COMBINED SOURCE MODEL

Project No.: 22010A	Project: 708 S Cloverdale Ave. Development	Date: MAR 2022	Figure D-4

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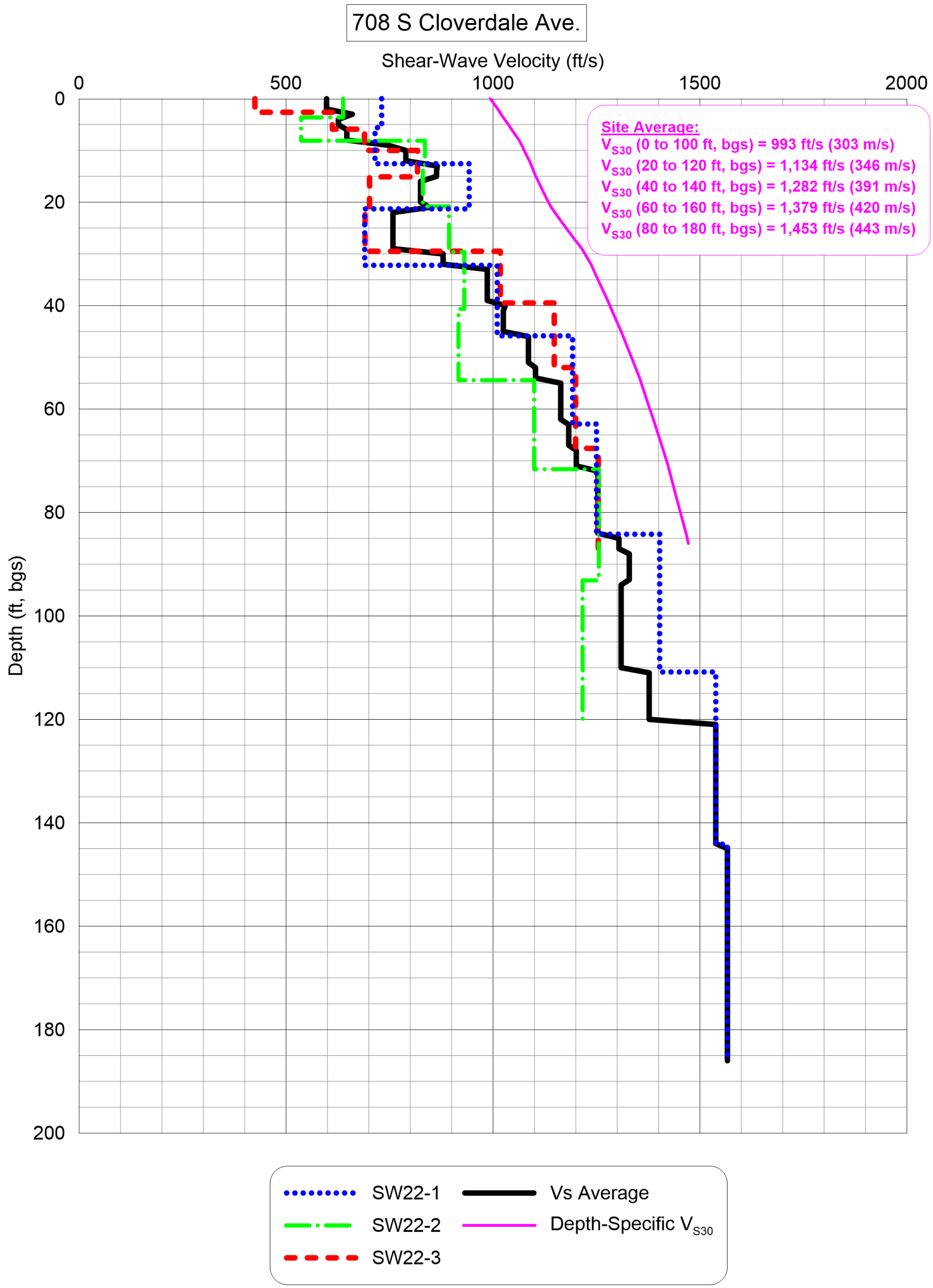

SHEAR WAVE VELOCITY PROFILE SUMMARY

Project No.: 22010A	Project: 708 S Cloverdale Ave. Development	Date: MAR 2022	Figure D-5

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APPENDIX E

GROUND-MOTION ANALYSIS

E.1 INTRODUCTION

This Appendix presents the ground-motion evaluation results for the subject site located on Figure E-1 in Los Angeles, California. Specifically, this Appendix contains the recommended site-specific response spectra. This Appendix will be updated with the earthquake time history analysis results as the structural design progresses. s

The currently proposed development includes the design and construction of a 492-ft tall, 42-story (up to the Roof Terrace) highrise tower that includes four underground parking levels and an amenity podium. The estimated fundamental spectral period of interest of the structure is not finalized at this time, but it is estimated to be about 6.0-seconds, and will be confirmed when the structural design is finalized. For the purposes of the current seismic hazard contribution evaluation, seismic source deaggregation, and acceleration time history record selection, a horizontal period of 5.0-seconds has been selected to represent the key structural period; however, the ground motions are selected such that they present reliable spectral ordinates up to a period of 10 seconds.

We understand that the design for this structure is being carried out in conformance with the 2019 California Building Code (CBC 2019) and ASCE 7-16 requirements using the performance-based design procedure specified by the 2020 Los Angeles Tall Buildings Structural Design Council (LATBSDC). To meet the performance-based design requirements, two levels of seismic evaluation will be completed: [1] a Serviceability Evaluation and [2] a Collapse Prevention Evaluation. The Serviceability Evaluation will be performed using the Service Level Earthquake (SLE) response spectrum, and the Collapse Prevention Evaluation will be performed using the Risk-Targeted Maximum Considered Earthquake (MCE_R) response spectrum. The design of nonstructural components might be based on the Design Response Spectrum (DRS), which is included for completeness.

To fulfill the seismic design requirements, the following site-specific response spectra are developed herein and summarized in this Appendix.

- “Maximum Considered Earthquake” uniform hazard spectrum (also known as the MCE_R response spectrum); This response spectrum is based on risk-targeted, maximum-rotated ordinates at 5% damping and corresponds to a 1% probability of collapse in a 50-year period.

- “Service-Level Earthquake” uniform hazard spectrum (also known as the SLE response spectrum); This response spectrum is based on average horizontal spectral ordinates at 1.62% damping and corresponds to a 50% probability of exceedance in a 30-year period.

- "Design-Level Earthquake" uniform hazard spectrum (also known as a DLE or DBE response spectrum, or DRS). This spectrum is based on maximum-rotated ordinates at 5% damping and corresponds to 2/3 of the MCE_R response spectrum.

The Collapse Prevention Evaluation also requires the development of eleven pairs of earthquake time histories scaled or spectrally matched to the site-specific Maximum Considered Earthquake (MCE_R) response spectrum, in accordance with the requirements of Section 16.2 of ASCE 7-16 and the 2020 LATBSDC guidelines. Because this project will be subject to the performance-based peer-review process, the seed acceleration time histories selected for the Collapse Prevention Evaluation will be reviewed and approved by the Peer Review Panel prior to performing the spectral matching. The final matched acceleration time histories to be used in the nonlinear response analysis will be documented upon receiving approval of the site-specific response spectra and seed time histories by the structural engineering team and the review panel. Note that if the site location or site conditions change appreciably, the ground-motion results presented herein would need to be re-evaluated.

E.2 SEISMIC SITE CHARACTERIZATION

The seismic site characterization for this study consisted of defining the site parameters needed to account for soil non-linearity in ground-motion attenuation models. The shear-wave velocity in the upper 30 meters of the site (V_{S30}) is the primary parameter used to approximate soil non-linearity in the ground-motion models. The remaining site parameters in the ground-motion attenuation models are the basin terms $Z_{1.0}$ and $Z_{2.5}$, which represent the depth to the 1.0 km/s and 2.5 km/s shear-wave velocities, respectively.

As part of this evaluation, shear-wave velocity measurements were collected at the site using Multichannel Analysis of Surface Waves (MASW) and Refraction Microtremor (ReMi) methods along three survey lines. The results and more information on the geophysical methods and analysis procedures is provided in Appendix D of this report. On Figure E-2, the V_{S30} values are calculated for a range of depths below existing ground surface. The data are presented in this format to allow for efficient interpretation of the V_{S30} value at a particular outcropping depth, as well as to provide information on the sensitivity of the V_{S30} to the shallow soils. The V_{S30} values are calculated per ASCE 7-16, Section 20.4.1.

Based on information from SEOR, we understand that the proposed structure consists of four basements attaining a depth of about 66 feet below grade, on a 4 to 14-ft thick mat foundation. Note that the thickness of the mat varies under the footprint of the building. For the purpose of the ground-motion evaluation, we have considered a representative total depth of 70 feet below ground surface in our analysis. Furthermore, it is our understanding that the majority of the seismic loading will be accommodated by the foundation and that lateral loading on the basement walls of the structure is minimal; therefore, the soils at and below the foundation level are expected to control the seismic input. In accordance with the structural properties and Section 3.2.4 of the 2020 LATBSDC guidelines, we recommend the V_{S30} be computed from the 35-ft depth. This corresponds to a V_{S30} value of 1,254 ft/s (382 m/s). This V_{S30} value corresponds to Site Class C (1,200 < $V_{S30} \leq$ 2,500 ft/s) in ASCE 7-16.

The remaining site parameters in the ground-motion attenuation models are the basin terms $Z_{1.0}$ and $Z_{2.5}$. The approximate depths to these interfaces were estimated to be 350 m and 2.25 km, respectively. These estimates were based on the SCEC Community Velocity Model (CVM-S4) by Magistrale et al. (2000 and 2012) and are in general agreement with values previously used for projects in the vicinity.

E.3 ASCE 7-16 CODE-BASED VALUES

Given the site latitude and longitude (34.061789° N, -118.346146° W) and estimated shear-wave velocity, mapped seismic hazard values were queried from the SEAOC/OSHPD Seismic Design Maps Tool application online at https://seismicmaps.org/. As discussed above in Section E.2 of this Appendix, the estimated V_{S30} at the site foundation level is 1,254 ft/s (382 m/s). This V_{S30} value corresponds to site classification for seismic design of Site Class C (1,200 < $V_{S30} \leq$ 2,500 ft/s). The mapped design parameters below are based on this information.

The general procedure ground-motion analysis carried out in accordance with Chapter 16A of the 2019 CBC and Section 11.4.4 of ASCE 7-16 results in mapped acceleration parameters SS and S1 of 2.025 g and 0.721 g, respectively, and site amplification factors Fa and Fv of 1.2 and 1.4, respectively. The general design spectral acceleration parameters S_{DS} and S_{D1} are 1.62 g and 0.673 g, respectively, and Seismic Design Category D for Risk Category II structures. The S_{DS} and S_{D1} values are superseded by the site-specific values presented in this Appendix but have been provided here for completeness.

E.4 SEISMIC HAZARD ANALYSIS

Probabilistic and Deterministic Seismic Hazard Analyses (PSHA and DSHA, respectively) involve the characterization of seismic sources, transmission paths for seismic energy, and the local site conditions. Seismic sources pertinent to ground-motion hazards at the site are characterized based on geologic information. The effects of transmission paths and local site conditions are estimated with ground-motion attenuation relationships, which provide the variation in peak horizontal and/or spectral acceleration with distance and other predictive parameters for a given local site condition. Key information on the computational platforms, seismic sources, and attenuation relationships used in this study is summarized below, followed by the results of the PSHA and DSHA. The resulting response spectra are presented in the following section (Section E.5) of this Appendix.

E.4.1 Seismic Setting

The site is located within a seismically active region of southern California, as evidenced by Quaternary faulting. The locations of Quaternary-active surface-rupturing faults mapped by the US Geological Survey (USGS, 2018) and instrumentally-recorded earthquakes (Hauksson et al., 2018) relative to the project site are shown on Figure E-3a.

The closest Late Quaternary (within the last 15,000 years) surface fault ruptures occurred on the Hollywood Fault (about 4½ km north of the site) and the Newport-Inglewood Fault (about 4½ km southwest). Other nearby faults with Late Quaternary surface rupture include the Santa Monica and the Raymond faults, each located roughly 7 to 13 km from the project site (Figure E-3a).

Several historic earthquakes have occurred within 50 km of the project site, as shown on Figure E-3a. The epicenter for the 1994 Northridge earthquake was approximately 24 km northwest of the project site. Based on interpolating data from nearby recordings in the PEER (2014) database, the event produced peak horizontal ground accelerations (PGA) and peak ground velocities (PGV) of about 0.15 g and 13 cm/s, respectively, at the project site. The 1987 Whittier Narrows earthquake epicenter was approximately 24 km east-southeast of the project site; that event produced PGA and PGV measurements of about 0.15 g and 9 cm/s, respectively, near the project site.

E.4.2 Computations Platforms

The horizontal Deterministic Seismic Hazard Analyses (DSHAs) were performed using the current version of the computer program Hazard (Abrahamson, 2021), herein referred to as HAZ45.

The horizontal Probabilistic Seismic Hazard Analyses (PSHAs) were performed using two computational platforms: HAZ45 (Abrahamson, 2021) and the USGS's PSHA hazard platform used in the National Seismic Hazard Mapping Project, herein referred to as NSHMP-HAZ. Specifically, version v1.1.0 of NSHMP-HAZ was used, which is the latest stable release. As described here below, each platform used an independent source characterization, but the same seismic site conditions (Section E-2) and ground-motion models were integrated in both platforms. The NSHMP-HAZ platform implemented the branch-averaged model based directly on UCERF3, and HAZ45 used an interpretation of UCERF3 with site-specific adjustments (e.g., the latest information available on local faults in the region) and additional epistemic branches to capture uncertainty for key parameters like fault geometry and slip rate. The results were each given 50% weight in calculating the uniform hazard spectra for the horizontal MCE_R and SLE development. Directivity effects were included in the HAZ45 platform, but the NSHMP-HAZ platform does not compute directivity effects.

E.4.3 Seismic Source Characterization

The Seismic Source Characterization (SSC) models used for this project are based on the characterization used by the USGS to develop the 2014 version of National Seismic Hazard Maps (NSHM; Petersen et al., 2014). Both discrete faults and background sources are included.

The NSHMP-HAZ PSHA used the Western US 2014 National Seismic Hazard Map Seismic Source Characterization (SSC) model for this project. This model implements the Uniform California Earthquake Rupture Forecast version 3 (UCERF3; by WGCEP, 2013a,b) branch average models (i.e., both alternatives) for discrete crustal faults and gridded background seismicity. The 2014 versions of the NSHM (Petersen et al., 2014) use the Western US 2014 NSHM SSC model.

The HAZ45 PSHA used our in-house implementation of UCERF3. The source geometries, alternative models, aseismicity factors, and slip rates in the UCERF3 model (WGCEP, 2013a,b) have been implemented in this site-specific SSC model. Additional epistemic uncertainty on slip rate and geometry is included for key nearby sources (i.e., the Hollywood, Santa Monica, Raymond, and Elysian Park faults). The locations of the seismic sources relative to the project site, as implemented in the PSHA, are shown on the fault map on Figure E-3b. The best-estimate parameters (including maximum magnitude, closest distance, slip rate, and style of faulting) for these seismic sources are summarized in Table E-1.

All faults shown on Figure E-3b and listed in Table E-1 were included in the HAZ45 PSHA. In addition to the discrete seismic sources presented in Table E-1, background seismicity that is consistent with the gridded seismicity used in the NSHM calculation was also used in the HAZ45 PSHA. The full set of UCERF3 faults (i.e., those beyond 100 km of the subject site) was implemented in the NSHMP-HAZ PSHA. Specific scenarios evaluated for the DSHA are presented in Table E-2.

E.4.4 Ground-Motion Characterization

Seismic shaking is estimated using empirical ground-motion attenuation relationships and calculated as the pseudo-spectral acceleration (SA) for a given period. Calculated values represent the average horizontal component considering 5% damping.

For this project, four of the five Next Generation Attenuation West 2 (NGA-West2) ground-motion attenuation models were used in the PSHA and DSHA analyses to calculate the horizontal response spectra: Abrahamson et al., (2014) – ASK14; Boore et al., (2014) – BSSA14; Campbell and Bozorgnia, (2014) – CB14; and Chiou and Youngs, (2014) – CY14. The Idriss (2014) model was not used based on the V_{S30} for the site and the applicability criteria for the model. Each of the attenuation relationships was assigned an equal weight of 1/4 to approximately address the "modeling" part of the epistemic uncertainty.

Based on the updated definitions per ASCE 7-16, Section 11.4.1, sites are classified near-fault when significant contribution hazard is noted from sources located within 10 km for $M_W \geq 6$, or within 15 km for $M_W \geq 7$. As discussed below, the project site falls into this category due to the proximity and characteristic earthquake size of the Hollywood, Elysian Park (Upper), Newport-Inglewood, and Santa Monica faults. Directivity effects are therefore considered for these sources in the probabilistic analysis for the horizontal ground motions and were computed using the HAZ45 platform. (It is noted that the NSHM-HAZ platform does not allow for computing directivity effects.) We used directivity models developed by Bayless and Somerville (2013) and Watson-Lamprey (2018). It is noted that directivity effects for the deterministic analysis are not relevant for this specific project site because a combination of the probabilistic MCE_R and the code-based minimum controls at long periods.

E.4.5 PSHA Results

A site-specific Probabilistic Seismic Hazard Analysis (PSHA) was completed to generate hazard curves and equal-hazard response spectra at the site for the Maximum Considered Earthquake (i.e., the MCE_R) and the Service-Level Earthquake (SLE). The basic results of the PSHA are presented in terms of seismic hazard curves, which show the annual probability of exceedance of a given spectral acceleration (SA), including PGA. The annual probability of exceedance is based on the calculated mean number of events per year that result in the spectral acceleration being exceeded at the site. Deaggregation plots are also useful for presenting PSHA results for a specified average return period (ARP) and SA; they show the percentage contribution to the total site seismic hazard based on distance and magnitude. Finally, equal-hazard spectra are used to identify a uniform hazard level (i.e., a specified ARP) over a range of periods.

As discussed above, two computational platforms were used with identical site and ground-motion models; each platform used an independent source characterization. The results were each given 50% weight in calculating the UHS for the MCE_R and SLE development.

E.4.5.1 Source Contribution Hazard Curves

Figures E-4a and E-4b present seismic hazard curves for the spectral periods of 0.2-seconds (which is close to the peak of the response spectrum, as is typical for California hazard) and 5.0-seconds. The total hazard (solid black line) and the contributions of various seismic sources to the total seismic hazard are shown. Table E-3 lists the relative contributions of significant seismic sources at various hazard levels for the 0.2 seconds and the 5.0-second spectral periods for the horizontal ground motions. As indicated on Table E-3 and Figure E-4a, the Hollywood Fault controls the horizontal 0.2-

seconds hazard for average return periods longer than about 500 years. This is expected given the proximity and slip rate of the Hollywood Fault, as listed on Table E-1. Other key contributors to the hazard are the Elysian Park System (mainly the Elysian Park Upper Fault), the Santa Monica Fault, and the Puente Hills System (single-fault model and three-fault model with LA segment as the main contributor). The 5.0-second hazard contributions (Table E-3 and Figure E-4b) are similar to the horizontal 0.2-seconds contributions, with persistent significant hazard contribution from the Hollyood, Santa Monica, Puente Hills and the Elysian Park faults at the 2,475-yr average return period. It is noted that the relative contribution of the San Andreas Fault System (and the other Type A faults) increases as the spectral period increases, and dominates at the short return periods. At the 2,475-yr ARP, the contribution of the Newport Inglewood Onshore Fault exceeds the contributions from the other sources.

E.4.5.2 Deaggregation Plots

Magnitude-distance deaggregations for 0.2-seconds and 5.0-seconds were also evaluated for the following ARPs:

- 43-yr (50% probability of exceedance in 30 years)
- 225-yr (20% probability of exceedance in 50 years)
- 975-yr (5% probability of exceedance in 50 years)
- 2,475-yr (2% probability of exceedance in 50 years)

The deaggregation plots are shown on Figures E-5a and E-5b. The mean magnitude and distance for each deaggregation are also listed on the figures. The vertical axis of the plots show the relative intensity of the magnitude-distance contribution with respect to the epsilon value (number of standard deviations above or below the median). Epsilon values of ±1 correspond to the 16th/84th percentiles; values of ±2 indicate 2nd/98th percentiles; and an epsilon value of zero is the median or 50th percentile.

As shown on Figure E-5a, the 2,475-yr 0.2-second hazard is controlled by M_W 6.0 to 7.5 earthquakes located within 10 km of the site that produce median to 98th percentile ground motions. These magnitude-distance bins correspond to characteristics events on several sources, including the Puente Hills (Alt 1. and LA), Elysian Park (Upper), Compton, and Hollywood (e.g., Table E-1). The 975-yr 0.2-seconds hazard deaggregation is similar to the 2,475-yr deaggregation. The 225-yr 0.2-second hazard deaggregation is also generally similar to the 975-yr and 2,475-yr, albeit with lower intensity ground motions, more contribution from M_W 6.0 to 7.5 events 15 to 25 km away, and more contribution from background seismicity within about 20 km of the site. The 43-yr 0.2-seconds hazard is controlled by background seismicity from M_W 5.0 to 7.0 earthquakes within 40 km of the site. There is also a clear contribution from characteristic events on the San Andreas Fault System located 58 km away. Finally, the peak in the 0.2-seconds 43-yr deaggregation in the M_W 6.0 to 6.5 and 20 to 25 km distance bin is due to low-intensity shaking from characteristic events (with magnitude uncertainty) on the Sierra Madre, Northridge, and Palos Verdes faults.

Figure E-5b shows the deaggregation at the same average return periods for 5.0-seconds. At the 2,475-yr ARP, the largest contributions are still from the local sources; however, as to be expected, within the local sources, the contribution is skewed towards the M-R bins with higher magnitudes and smaller range of distance with respect of the shorter-periods deaggregations. This trend is particularly relevant at the long return periods for the modal distance for the magnitude 7 to 7.5 bin, with attains peak values within 5 km from the site. Some contribution is evident from very high epsilon ground motions produced by characteristic earthquakes on the San Andreas Fault System (M_W 8.2±0.2) about 58 km away from the site, especially at short average return periods. At all average return periods, we notice that the ground-motion hazard presents a clear bimodal distance distribution, where a fair amount of hazard still comes from sources within 15 km of the site, but there is a sharp spike in the 50 to 75 km bin as related to characteristic earthquakes on the San Andreas Fault System. The noticeable but minor spike in the 75 to 100 km bin is due to the contribution from other distant faults with high slip rates (e.g., San Jacinto Fault System located 76 km away from the site). This bimodal hazard distribution is nevertheless characterized by modest contribution associated to magnitude 7 to 8 capable source located about 25 km away from the site. These results are overall consistent with the source contribution to the total hazard discussed above

E.4.5.3 Uniform Hazard Spectra

The results of the PSHA at periods between 0.01 and 10 seconds are aggregated into a uniform hazard spectrum for several return periods and averaged. The 2,475-yr ordinates at 5% damping are also tabulated on Table E-4 in Column 3,4, and 5, and the resulting average UHS is plotted on Figure E-6. The development of the MCE_R spectrum is based on the 2,475-yr uniform hazard spectrum.

The probabilistic MCE_R spectrum, which represents the maximum rotated, risk-targeted ordinates per ASCE 7-16, is shown on Figure E-6. The ordinates are also tabulated on Table E-4 in Column 8. This spectrum was developed using one set of scale factors to adjust the calculated ordinates (which are the average horizontal component of ground motion) to the maximum rotated component of ground motion, and a second set of scale factors was used to adjust the ordinates from hazard representing 2% probability of exceedance in 50 years (the 2,475-yr ARP) to risk, which represents a 1% probability of exceedance in 50 years. The adjustment between average horizontal and maximum rotated component is based on the period-specific ratios in Shahi and Baker (2014). The adjustment between hazard and risk-targeted ordinates is based on the mapped ratios provided by ASCE 7-16 for use by Method 1 (21.2.1.1). At the site latitude and longitude, a scale factor of 0.900 is specified for periods

0.2-second and shorter and a scale factor of 0.899 is used for periods of 1.0-second and longer; scale factors for periods between 0.2- and 1.0-second are linearly interpolated. The incorporation of these scale factors is reflected in the modified probabilistic MCE_R spectrum on Figure E-6, and the process of developing the probabilistic MCE_R spectral ordinates is shown on Table E-4 in Columns 3 through 8.

The Serviceability Evaluation per the 2020 LATBSDC guidelines uses the Service-Level Earthquake (SLE) spectrum, which based on a uniform hazard spectrum reflecting ground motions with a 50% probability of exceedance in 30 years (43-year return period). Accordingly, the results of the horizontal PSHA at periods between 0.01 and 10 seconds are also aggregated into a 43-yr ARP uniform hazard spectrum on Figure E-6. Development of the SLE spectrum, including conversion of the hazard ordinates to the target damping ratio, is discussed below.

E.4.6 DSHA Results

A deterministic seismic hazard analysis (DSHA) was performed for the site following the guidelines provided in ASCE 7-16. Albeit the ASCE 7-16 Supplement 1 introduced an exception to the need of DSHA computation in the event the largest spectral response acceleration of the probabilistic ground motion response spectrum of 21.2.1 is less than 1.2 time the Fa factor (with the latter being determined using Table 11.4.1, with the value of Ss taken as 1.5 for Site Classes A, B, C, and D), such conditions are not encountered in the present project. In fact, the resulting Fa factor for Site Class C is 1.2, thus resulting in a threshold of 1.44 which is less that the peak spectral values attained by the probabilistic MCE_R spectrum. As such, the development of a deterministic ground-motion response spectrum is necessary.

On the basis of the seismic source characterization and the results of the PSHA, several faults were evaluated for the DSHA. Table E-2 lists the key contributors to the DSHA ground motions, as well as the fault parameters used in the analysis. The DSHA scenarios were evaluated using the same ground-motion models and site parameters defined above for the PSHA. Predicted response spectra for each of these DSHA scenarios are shown on Figure E-7. The DSHA ordinates reflect the 84^{th} percentile average horizontal component of ground motion, modified to represent the maximum rotated component of ground motion. The modification for maximum rotated component was performed using the same methodology described for the probabilistic MCE_R development (i.e., the Shahi and Baker, 2014, period-specific ratios). Additional faults, including the nearby Elysian Park (Lower), the San Vicente, the North Salt Lake, the Raymond faults, the Northridge System, and the Sierra Madre faults and were also evaluated and their predicted ground motions were found to contribute less than those sources tabulated above.

Before the ASCE 7-16 Supplement 1 took effect, the deterministic MCE_R response spectrum was defined as the envelope (maximum at each ordinate) of the 84^{th} percentile of DSHA scenarios, but no less than the code-based deterministic minimum developed per ASCE 7-16, Section 21.2.2. In an effort to compute a code-based deterministic minimum response spectrum characterized by realistic spectral shape, the Supplement 1 modifies the approach to develop such minimum: per new provisions, the code-based deterministic minimum is the envelope of the maximum-rotated 84^{th} percentile spectral ordinates, scaled by a single factor such that the maximum response spectral acceleration equals 1.5 times Fa (developed as discussed above). The final deterministic MCE_R response spectrum is still defined as the maximum between the envelope of the maximum-rotated 84^{th} percentile spectral ordinates and the code-based deterministic minimum developed as discussed above.

As observed on Figure E-7, the Compton case controls the deterministic MCE_R spectrum up to about 1.5 seconds, and the Newport-Inglewood Onshore case controls the envelope at longer spectral periods. The code-based deterministic minimum attains smaller spectral amplitudes as compared to the 84^{th} percentile of DSHA scenarios. The deterministic MCE_R spectral ordinates are tabulated in Table E-4 in Column 12, and the process of developing the deterministic MCE_R spectral ordinates is shown on Table E-4 in Columns 9 through 12.

E.5 SITE-SPECIFIC RESPONSE SPECTRA

It is our understanding that the structural evaluation is being carried out in conformance with the 2019 CBC requirements and ASCE 7-16 requirements for performance-based design, using the procedure specified by the 2020 LATBSDC guidelines. Accordingly, two levels of seismic evaluation are required for this project: Serviceability Evaluation and Collapse Prevention Evaluation. The Serviceability Evaluation uses the Service-Level Earthquake (SLE) spectrum, which is represented by a uniform hazard spectrum reflecting ground motions with a 50% probability of exceedance in 30 years (43-yr return period) with a reduced damping ratio (< 5%). The Collapse Prevention Evaluation uses the site-specific MCE_R response spectrum, developed in accordance with the requirements of Section 21.2 of ASCE 7-16. For completeness, the site-specific DRS, developed in accordance with the requirements of Section 21.3 of ASCE 7-16, is also provided for the design of non-structural components. The December 2018 ASCE 7-16 Supplement 1 was followed in developing both the site-specific MCE_R and DRS spectra.

The development of these spectra is discussed below.

E.5.1 Site-Specific MCE_R Response Spectrum

The left panel of Figure E-8 shows the final development of the site-specific horizontal MCE_R response spectrum. The final horizontal MCE_R is developed as the lesser of the deterministic MCE_R and the probabilistic MCE_R response spectra (per ASCE 7-16, Section 21.2.3), but no less than the code-based minimum (per ASCE 7-16, Supplement 1, Section 21.2.3).

As shown in the left panel on Figure E-8, the probabilistic MCE_R spectrum attains less spectral amplitudes as compared to the deterministic MCE_R spectrum. Accordingly, the probabilistic MCE_R spectrum controls at all spectral periods beside for the narrow 0.02- to 0.09-second range and above approximately 6 seconds, where the code-based minimum MCE_R spectrum controls. The final site-specific MCE_R spectrum is shown highlighted in the left panel on Figure E-8, and the spectral ordinates are tabulated in Table E-4, Column 14. The process of developing the site-specific horizontal MCE_R spectral ordinates is shown in Table E-4 in Columns 8 and 12 through 14.

The site-specific horizontal MCE_R developed per ASCE 7-16, Section 21.2 represents the RotD100 spectrum. A compatible RotD50 spectrum was also calculated by “un-rotating” the MCE_R RotD100 using the same period-specific ratios described in Section E.4.5.3. The results are shown in the right panel on Figure E-8, and are used to support the future seed acceleration time history selection.

E.5.2 Site-Specific Design Response Spectrum

The Design Response Spectrum (DRS) was developed as 2/3 of the site-specific MCE_R, but no less than the code-based minimum (which is defined as 80% of the code-based spectrum using ASCE 7-16, Section 11.4.6). The process of developing the DRS is shown on Figure E-9. The final recommended horizontal DRS is shown highlighted on Figure E-9, and the ordinates are tabulated in Table E-5, Column 6. The process of developing the horizontal DRS ordinates is shown in Table E-5 in Columns 3 through 6.

The site-specific seismic design parameters for new structures at the project site were calculated per Section 21.4 of ASCE 7-16 and are listed below. As specified in ASCE 7-16, Section 21.4, the site-specific short-period design acceleration, S_{DS}, is calculated as 90% of the maximum DRS between 0.2-seconds and 5.0-seconds. The 1-second design acceleration, S_{D1}, is calculated as the maximum product of the period and DRS between 1.0- and 2.0-seconds. It is noted that these parameters are based on Fa and Fv values of 1.2 and 1.4, respectively, in accordance with ASCE 7-16, Section 21.3.

- S_{DS} = 1.532 g, based on 90% of the spectral acceleration at a period of 0.3-seconds
- S_{D1} = 0.875 g, based on the spectral acceleration at a period of 1.0-second
- S_{MS} = 2.298 g, based on 1.5 times S_{DS}

- S_{M1} = 1.312 g, based on 1.5 times S_{D1}

E.5.3 Site-Specific SLE Spectrum

The SLE response spectrum, which is based on the 43-year ARP uniform hazard spectrum, is shown on Figure E-10. The SLE response spectrum represents a 50% probability of exceedance in 30 years at a reduced damping ratio (< 5%).

Based on communications from the SEOR, a critical damping value of 1.62% is used in the SLE development in conformance to Section 3.4.4 of the 2020 LATBSDC guidelines factoring in the height of the proposed tower. Specifically, the 43-year ARP uniform hazard spectrum ordinates were converted from 5% spectral damping (as is predicted by the GMPEs in the hazard calculation) to 1.62% damping using the empirically-based Damping Scaling Factor (DSF) relationship in Rezaeian et al. (2012). This model uses magnitude and distance as parameters to estimate period-specific DSFs. The mean magnitude and distance for each spectral ordinate at the 43-yr ARP were used in the DSF calculation.

The final recommended SLE is tabulated in Table E-6 in Column 7. The process of developing the SLE ordinates is also shown in Table E-6.

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TABLE E-1
CHARACTERIZATION[1] OF SIGNIFICANT FAULTS
708 S. CLOVERDALE AVE.

Fault Name	Style of Faulting[2]	Maximum Magnitude (Mw)	Slip Rate (mm/yr)	Closest Rupture Distance From Site (km)
San Vicente	RV	6.1	0.2	1.6
North Salt Lake	RV	5.8	0.1	2.6
Puente Hills (LA)	RV	6.7	0.6	3.7
Newport-Inglewood	SS	7.1	1.2	4.5
Hollywood	OBL	6.5	1.3	4.6
Puente Hills	RV	7.0	0.9	6.7
Santa Monica	OBL	6.7	1.1	6.9
Elysian Park (Upper)	RV	6.5	1.4	7.8
Elysian Park (Lower CFM)	RV	6.8	0.1	10.2
San Pedro Escarpment	RV	7.1	0.2	12
Raymond	OBL	6.5	1.3	13
Compton	RV	7.3	0.8	14
Verdugo	RV	6.8	0.6	15
Malibu Coast	OBL	6.9	0.8	17
Puente Hills (Santa Fe Springs)	RV	6.4	0.8	19
Anacapa-Dume	OBL	7.1	0.7	20
Northridge Hills	RV	6.8	1.0	21
Palos Verdes	SS	7.4	2.3	22
Sierra Madre	RV	7.1	1.5	22
Mission Hills	RV	6.3	0.8	23
Sierra Madre (San Fernando)	RV	6.5	1.6	24
Northridge	RV	6.8	1.3	24
Santa Susana East (connector)	RV	6.2	1.9	24
Santa Monica Bay	RV	6.8	0.1	24
San Gabriel (Extension)	SS	7.1	0.5	28
Redondo Canyon	RV	6.5	0.4	28
Santa Susana	RV	6.9	3.2	28
Elsinore - Whittier[3]	SS	7.0	4.2	28
San Gabriel	OBL	7.2	0.6	29
Puente Hills (Coyote Hills)	RV	6.6	0.8	29
Anaheim	RV	6.2	0.1	33
Clamshell-Sawpit	RV	6.4	0.3	34
Holser	RV	6.6	0.6	37
San Pedro Basin	SS	7.1	1.1	39
Simi-Santa Rosa	OBL	6.8	1.1	42
San Jose	OBL	6.5	0.3	43
Richfield	RV	6.1	0.2	44
Peralta Hills	RV	6.3	0.3	47
Oak Ridge (Onshore)	RV	7.1	2.6	47
Del Valle	RV	6.2	1.0	48
Yorba Linda	RV	6.3	0.1	48
Chino	OBL	6.7	0.9	51
Malibu Coast (Extension)	OBL	6.8	0.5	54
San Joaquin Hills	RV	6.8	1	56
Cucamonga	RV	6.7	1.7	56
San Cayetano	RV	7.0	2.9	57
San Andreas[3]	SS	8.2	29.0	58
Sisar	RV	6.8	0.8	64
Newport-Inglewood (Offshore)	SS	7.1	1.0	66
Fontana (Seismicity)	SS	6.6	0.3	70
San Diego Trough North	SS	7.3	1.6	71
Ventura-Pitas Point	OBL	7.1	1.5	72
Santa Cruz Catalina Ridge	OBL	7.4	1.1	73
Pine Mtn	RV	7.2	0.3	74
Santa Ynez (East)	SS	7.1	1.5	75
San Jacinto[3]	SS	7.9	6.0	76
Oceanside	RV	7.2	0.7	78
Channel Islands Thrust	RV	7.2	1.0	85
Santa Cruz Island	OBL	7.1	0.9	85
Cleghorn	SS	6.6	0.5	85
Oak Ridge (Offshore)	RV	6.8	1.7	86
Mission Ridge-Arroyo Parida-Santa Ana	RV	7.0	1.1	87
Red Mountain	RV	7.4	2.2	93
San Clemente	SS	7.4	1.8	96
Channel Islands Western Deep Ramp	RV	7.2	0.4	98
Garlock[3]	SS	7.4	3.6	100
Big Pine (Central)	RV	6.3	0.4	100
North Frontal (West)	RV	7.1	0.3	103
Coronado Bank	SS	7.4	1.8	103

Notes:

(1) Source characterization based on information published by SCEC/USGS UCERF2 (WGCEP, 2008), 2008 NSHM (Petersen et al., 2008), and UCERF3 (WGCEP, 2013).

(2) SS=Strike-Slip, OBL=Oblique, RV=Reverse or Thrust, NOR=Normal.

(3) Characterization used a distribution of magnitude and slip rates; best estimate for deterministic case shown.

TABLE E-2
DETERMINISTIC SEISMIC HAZARD ANALYSIS FAULT CHARACTERIZATION
708 S. CLOVERDALE AVE.

Fault	Column 1	Column 2	Column 3	Column 4	Column 5	Column 6	Column 7	Column 8	Column 9	Column 10	Column 11	Column 12
	M_W	F_{RV}	F_N	F_{HW}	Z_{TOR}	Z_{BOT}	Dip	W	Z_{HYP}	R_{RUP}	R_{JB}	R_X
Raymond/Hollywood/Santa Monica System	7.0	1	0	0	0	17.3	70	18.4	10.2	4.6	4.6	-4.6
Elysian Park (Upper)	6.5	1	0	0	3.0	15.0	50	15.7	11.0	7.8	7.2	-7.2
Puente Hills (LA)	6.8	1	0	1	2.1	15.0	27	28.4	7.8	3.7	1.8	1.8
Puente Hills (Alt. 1)	7	1	0	0	5.0	13.0	25	18.9	10.2	6.7	4.5	-2.2
Compton	7.3	1	0	1	5.2	15.0	20	28.7	9.4	13.6	0	25.5
Newport-Inglewood Onshore	7.4	0	0	0	0	15.0	90	15.0	10.2	4.5	4.5	4.5
Elsinore	7.8	0	0	0	0	15.4	90	15.4	10.2	28.4	28.4	28.4
San Andreas	8.2	0	0	0	0	13.1	90	13.1	10.2	58	58	58

Key

Column 1	= Moment magnitude.
Column 2	= Reverse-faulting factor: 0 for strike slip, normal, normal-oblique; 1 for reverse, reverse-oblique, thrust.
Column 3	= Normal-faulting factor: 0 for strike slip, reverse, reverse-oblique, thrust and normal-oblique; 1 for normal.
Column 4	= Hanging-wall factor: 1 for site on down-dip side of top of rupture; 0 otherwise.
Column 5	= Depth to top of coseismic rupture (km).
Column 6	= Depth to bottom of the seismogenic crust (km).
Column 7	= Average dip of rupture plane (degrees).
Column 8	= Fault rupture width (km).
Column 9	= Hypocentral depth from the earthquake (km), based on Campbell and Bozorgnia (2014) model.
Column 10	= Closest distance to coseismic rupture (km).
Column 11	= Closest distance to surface projection of coseismic rupture (km).
Column 12	= Horizontal distance from top of rupture measured perpendicular to fault strike (km).

TABLE E-3
PSHA SOURCE CONTRIBUTIONS
708 S. CLOVERDAVE AVE.

0.2-sec

Source	*43-yr*	*225-yr*	*975-yr*	*2,475-yr*
Hollywood	10%	19%	23%	25%
Elysian Park System	8%	12%	13%	13%
Santa Monica	6%	10%	11%	11%
Puente Hills System	6%	9%	10%	11%
Compton	3%	6%	8%	10%
Newport-Inglewood Onshore	3%	6%	7%	8%
Background	14%	9%	7%	6%
San Vicente	1%	2%	3%	3%
Raymond	5%	4%	3%	2%
Santa Susana System	7%	4%	2%	1%
Sierra Madre System	5%	3%	1%	1%
Palos Verdes	2%	2%	1%	1%
San Andreas	6%	2%	1%	0%
Elsinore	1%	1%	0%	0%
San Jacinto	1%	0%	0%	0%
Others	20%	11%	9%	7%

5.0-sec

Source	*43-yr*	*225-yr*	*975-yr*	*2,475-yr*
Newport-Inglewood Onshore	2%	6%	13%	18%
San Andreas	24%	25%	21%	17%
Santa Monica	4%	8%	11%	12%
Hollywood	6%	9%	10%	9%
Puente Hills System	4%	5%	7%	8%
Elysian Park System	5%	7%	8%	7%
Compton	2%	3%	5%	7%
Palos Verdes	2%	4%	5%	5%
Sierra Madre System	4%	3%	3%	2%
Raymond	3%	3%	2%	2%
San Jacinto	9%	4%	2%	1%
Elsinore	4%	2%	1%	1%
Background	3%	2%	1%	1%
Santa Susana System	4%	2%	1%	1%
San Vicente	0%	0%	0%	0%
Others	24%	15%	11%	8%

TABLE E-4
SITE-SPECIFIC HORIZONTAL MCE$_R$ DEVELOPMENT CALCULATION SHEET
708 S. CLOVERDALE AVE.

Column 1	Column 2	Column 3	Column 4	Column 5	Column 6	Column 7	Column 8	Column 9	Column 10	Column 11	Column 12	Column 13	Column 14
		HAZ45 2475-yr UHS (PSHA)	*NSHMP-HAZ 2475-yr UHS (PSHA)*	*Average 2475-yr UHS (PSHA)*	*Risk Collapse Scaling Factors*	*Max. Orientation Scaling Factors*	*Probabilistic MCE$_R$*	*84th %tile DSHA Envelope*	*Max. Direction 84th %tile DSHA Envelope*	*Code-Based Deteterministic Minimum MCE$_R$*	*Deterministic MCE$_R$*	*Code Minimum MCE$_R$*	*Final Site-Specific Horz. MCE$_R$*
Period	*Frequency*	*RotD50*	*RotD50*	*RotD50*		*RotD50*	*RotD100*	*RotD50*	*RotD100*	*RotD100*	*RotD100*	*RotD100*	*RotD100*
(sec)	(Hz)	(g)	(g)	(g)	-	-	(g)	(g)	(g)	(g)	(g)	(g)	(g)
0.01	100	0.929	0.946	0.937	0.900	1.190	1.004	1.018	1.211	0.705	1.211	0.918	1.004
0.02	50	0.937	0.960	0.949	0.900	1.190	1.016	1.027	1.222	0.711	1.222	1.059	1.059
0.03	33.33	0.988	1.010	0.999	0.900	1.190	1.070	1.067	1.270	0.739	1.270	1.199	1.199
0.05	20	1.183	1.195	1.189	0.900	1.190	1.273	1.231	1.465	0.852	1.465	1.480	1.480
0.075	13.33	1.505	1.498	1.501	0.900	1.190	1.608	1.496	1.780	1.036	1.780	1.831	1.831
0.1	10	1.764	1.747	1.755	0.900	1.190	1.880	1.729	2.058	1.198	2.058	1.944	1.944
0.15	6.67	2.067	2.061	2.064	0.900	1.200	2.230	2.044	2.453	1.428	2.453	1.944	2.230
0.2	5	2.227	2.240	2.234	0.900	1.210	2.432	2.298	2.781	1.618	2.781	1.944	2.432
0.25	4	2.300	2.327	2.314	0.900	1.220	2.540	2.428	2.962	1.724	2.962	1.944	2.540
0.3	3.33	2.306	2.346	2.326	0.900	1.220	2.553	2.535	3.093	1.800	3.093	1.944	2.553
0.4	2.5	2.128	2.185	2.157	0.900	1.230	2.387	2.455	3.020	1.758	3.020	1.944	2.387
0.5	2.00	1.942	1.988	1.965	0.900	1.230	2.174	2.242	2.757	1.605	2.757	1.615	2.174
0.75	1.33	1.495	1.528	1.511	0.899	1.240	1.685	1.733	2.149	1.251	2.149	1.077	1.685
1	1	1.161	1.193	1.177	0.899	1.240	1.312	1.309	1.623	0.945	1.623	0.808	1.312
1.5	0.67	0.724	0.761	0.743	0.899	1.240	0.828	0.801	0.994	0.578	0.994	0.538	0.828
2	0.5	0.503	0.535	0.519	0.899	1.240	0.578	0.566	0.702	0.409	0.702	0.404	0.578
3	0.33	0.295	0.325	0.310	0.899	1.250	0.348	0.368	0.459	0.267	0.459	0.269	0.348
4	0.25	0.193	0.220	0.207	0.899	1.260	0.234	0.256	0.323	0.188	0.323	0.202	0.234
5	0.2	0.140	0.164	0.152	0.899	1.260	0.172	0.188	0.237	0.138	0.237	0.162	0.172
7.5	0.13	0.077	0.094	0.086	0.899	1.280	0.099	0.094	0.120	0.070	0.120	0.108	0.108
10	0.1	0.050	0.062	0.056	0.899	1.290	0.065	0.055	0.071	0.041	0.071	0.065	0.065

Note: Significant figures are provided for computational purposes only and do not necessarily reflect accuracies to those significant figures.

Key

Column 1	= Spectral period in seconds.
Column 2	= Spectral frequency (inverse of spectral period) in Hertz.
Column 3	= Mean uniform hazard spectral ordinates for 2,475-yr average return period in units of g for 5% damping, with directivity, based on HAZ45 platform.
Column 4	= Mean uniform hazard spectral ordinates for 2,475-yr average return period in units of g for 5% damping based on NSHMP-HAZ platform.
Column 5	= Averaged mean uniform hazard spectral ordinates for 2,475-yr average return period in units of g for 5% damping; average from Columns 3 and 4.
Column 6	= Site-specific risk coefficient (C_R) from USGS.
Column 7	= Scale factor to obtain maximum-oriented spectral acceleration; from Shahi and Baker (2014).
Column 8	= Probabilistic risk-targeted, maximum considered earthquake ground motion spectral ordinates in units of g for 5% damping.
Column 9	= 84th percentile deterministic hazard spectral ordinates in units of g for 5% damping; ordinates are maximum of all deterministic scenarios, therefore spectrum may not represent a single event.
Column 10	= Deterministic, maximum considered earthquake ground motion spectral ordinates in units of g for 5% damping.
Column 11	= Code-based (ASCE 7-16 Supplement 1, Ch. 21.2.2) deterministic lower limit for risk-targeted, maximum considered earthquake ground motion spectral ordinates in units of g for 5% damping.
Column 12	= Deterministic maximum considered earthquake ground motion spectral ordinates in units of g for 5% damping; maximum value from Columns 10 and 11.
Column 13	= Code minimum (per ASCE 7-16, Supplement 1, Section 21.2.3) for risk-targeted, maximum considered earthquake ground motion spectral ordinates in units of g for 5% damping.
Column 14	= Final risk-targeted, maximum considered earthquake ground motion spectral ordinates in units of g for 5% damping; minimum value from Columns 8 and 12, but no less than Column 13.

TABLE E-5
SITE-SPECIFIC HORIZONTAL DRS DEVELOPMENT CALCULATION SHEET
708 S. CLOVERDALE AVE.

Column 1	Column 2	Column 3	Column 4	Column 5	Column 6
		Code-Based DRS	*80% of Code-Based DRS*	*2/3 of Site-Specific MCE_R*	*Final Site-Specific Horiz. DRS*
Period	*Frequency*	*RotD100*	*RotD100*	*RotD100*	*RotD100*
(sec)	(Hz)	(g)	(g)	(g)	(g)
0.01	100	0.765	0.612	0.669	0.669
0.02	50	0.882	0.706	0.706	0.706
0.03	33.33	0.999	0.799	0.799	0.799
0.05	20	1.233	0.987	0.987	0.987
0.075	13.33	1.526	1.220	1.220	1.220
0.1	10	1.620	1.296	1.296	1.296
0.15	6.67	1.620	1.296	1.486	1.486
0.2	5	1.620	1.296	1.622	1.622
0.25	4	1.620	1.296	1.694	1.694
0.3	3.33	1.620	1.296	1.702	1.702
0.4	2.5	1.620	1.296	1.591	1.591
0.5	2.00	1.346	1.077	1.450	1.450
0.75	1.33	0.897	0.718	1.123	1.123
1	1	0.673	0.538	0.875	0.875
1.5	0.67	0.449	0.359	0.552	0.552
2	0.5	0.336	0.269	0.385	0.385
3	0.33	0.224	0.179	0.232	0.232
4	0.25	0.168	0.135	0.156	0.156
5	0.2	0.135	0.108	0.115	0.115
7.5	0.13	0.090	0.072	0.072	0.072
10	0.1	0.054	0.043	0.043	0.043

Note: Significant figures are provided for computational purposes only and do not necessarily reflect accuracies to those significant figures.

Key

Column 1	= Spectral period in seconds.
Column 2	= Spectral frequency (inverse of spectral period) in Hertz.
Column 3	= Code-based (ASCE 7-16, Ch. 21.3) design spectral ordinates in units of g for 5% damping.
Column 4	= Code minimum (ASCE 7-16, Ch. 21) design ground motion spectral ordinates in units of g for 5% damping; 80% of the value in Column 3.
Column 5	= 2/3 of the final site-specific MCE_R ground motion spectral ordinates in units of g for 5% damping.
Column 6	= Final design ground motion spectral ordinates in units of g for 5% damping; maximum value from Columns 4 and 5.

TABLE E-6
SITE-SPECIFIC HORIZONTAL SLE DEVELOPMENT CALCULATION SHEET
708 S. CLOVERDALE AVE.

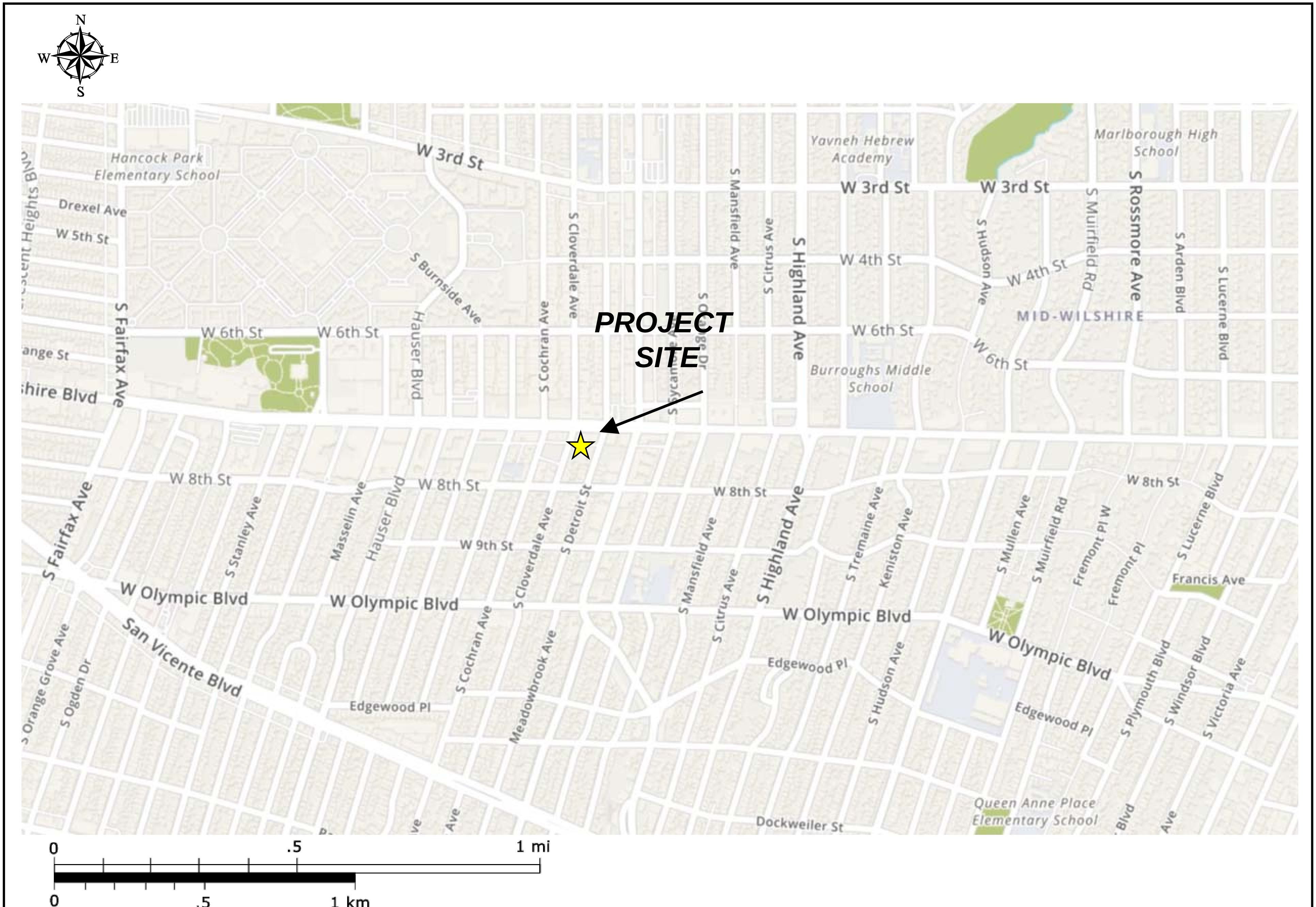
Column 1	Column 2	Column 3	Column 4	Column 5	Column 6	Column 7
		HAZ45 43-yr UHS (PSHA)	*NSHMP-HAZ 43-yr UHS (PSHA)*	*Average 43-yr UHS (PSHA)*	*Damping Scaling Factors*	*SLE @ 1.62% Damping*
Period	*Frequency*	*RotD50*	*RotD50*	*RotD50*		*RotD50*
(sec)	(Hz)	(g)	(g)	(g)	-	(g)
0.01	100	0.177	0.143	0.160	0.999	0.160
0.02	50	0.178	0.144	0.161	1.008	0.162
0.03	33.33	0.188	0.152	0.170	1.040	0.177
0.05	20	0.225	0.182	0.203	1.127	0.229
0.075	13.33	0.286	0.233	0.259	1.233	0.319
0.1	10	0.339	0.279	0.309	1.309	0.404
0.15	6.67	0.403	0.333	0.368	1.369	0.504
0.2	5	0.422	0.346	0.384	1.392	0.534
0.25	4	0.415	0.339	0.377	1.391	0.524
0.3	3.33	0.396	0.323	0.359	1.397	0.502
0.4	2.5	0.343	0.278	0.311	1.399	0.434
0.5	2.00	0.298	0.241	0.270	1.398	0.377
0.75	1.33	0.208	0.167	0.188	1.386	0.260
1	1	0.153	0.121	0.137	1.382	0.189
1.5	0.67	0.095	0.074	0.084	1.375	0.116
2	0.5	0.065	0.051	0.058	1.360	0.079
3	0.33	0.038	0.029	0.034	1.351	0.046
4	0.25	0.026	0.019	0.023	1.330	0.030
5	0.2	0.018	0.014	0.016	1.316	0.021
7.5	0.13	0.010	0.007	0.008	1.278	0.011
10	0.1	0.006	0.004	0.005	1.202	0.006

Note: Significant figures are provided for computational purposes only and do not necessarily reflect accuracies to those significant figures.

Key

Column 1	= Spectral period in seconds.
Column 2	= Spectral frequency (inverse of spectral period) in Hertz.
Column 3	= Mean uniform hazard spectral ordinates for 43-yr average return period in units of g for 5% damping based on HAZ45 platform.
Column 4	= Mean uniform hazard spectral ordinates for 43-yr average return period in units of g for 5% damping based on NSHMP-HAZ platform.
Column 5	= Averaged mean uniform hazard spectral ordinates for 43-yr average return period in units of g for 5% damping; average from Columns 3 and 4.
Column 4	= Damping Scaling Factor used to convert spectral ordinates from 5% damping; developed per Rezaeian et al. (2012).
Column 5	= Service-Level Earthquake ground motion spectral ordinates in units of g for reported damping; developed per Rezaeian et al. (2012).

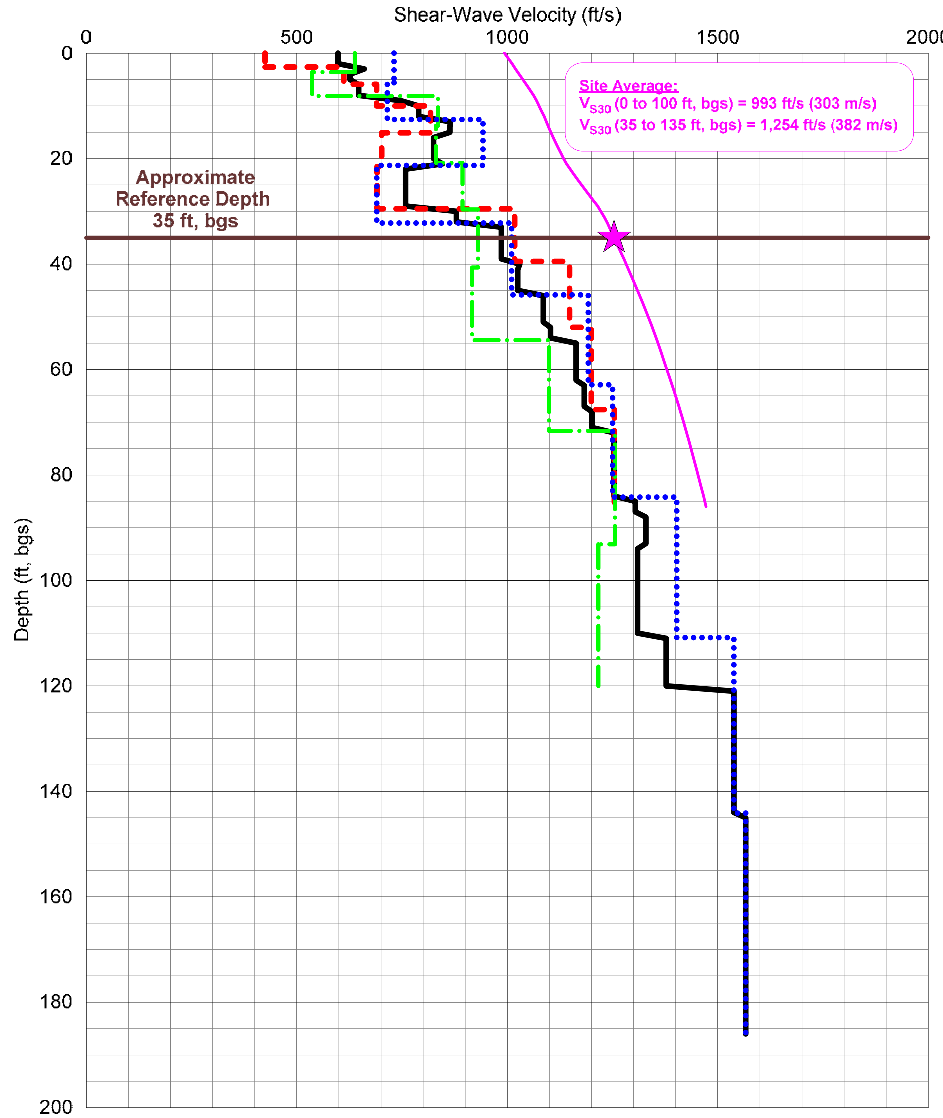
GeoPentech



SITE LOCATION MAP

Date: MAR 2022	Project No.: 22010A	Project: 708 S. CLOVERDALE AVE.	Figure E-1

GeoPentech



LEGEND

Location of Surface Wave Geophysical Measurement

N

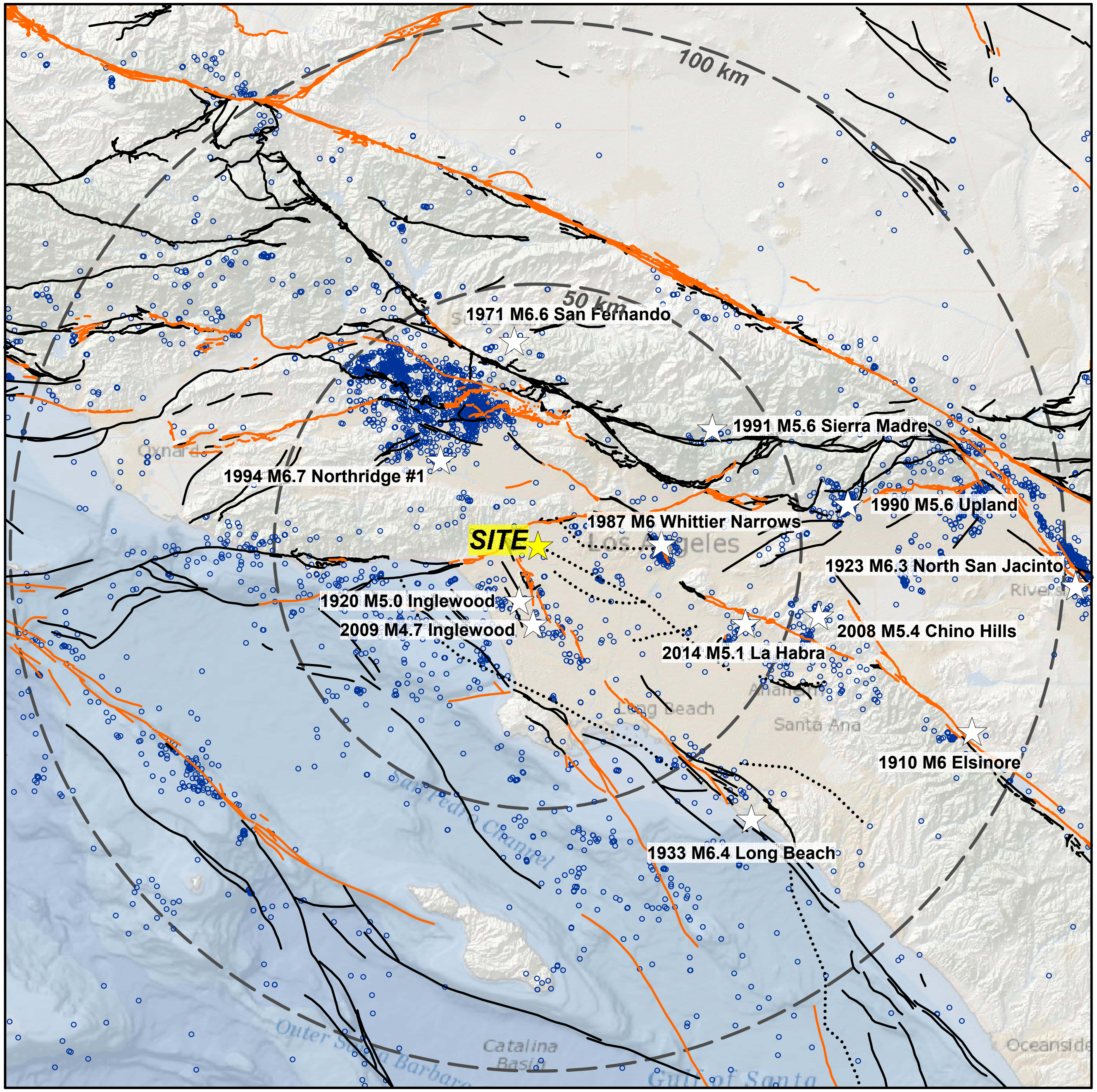
APPROXIMATE SCALE

0 100 FEET

SHEAR-WAVE VELOCITY PROFILE LOCATIONS AND RESULTS

Date: MAR 2022	Project No.: 22010A	Project: 708 S. CLOVERDALE AVE.	Figure E-2

GeoPentech



Legend

USGS Quaternary Fault & Fold Database [(1)]

- Active in < 15,000 years
- Active in < 1,600,000 years
- Blind Thrust

Seismicity [(2)]

- M ≥ 2.5
- Historic Earthquake

Notes:

1. Fault traces are from USGS Quaternary Fault and Fold Database (USGS, 2018).
2. Seismicity (hollow blue dots) is from Hauksson et al. (2018) catalog ("HYS" catalog). Catalog includes all instrumentally-recorded events in southern Calfornia from 01/01/1981 through 12/31/2018. Only M ≥ 2.5 events are shown here. Significant post-1900 earthquakes identified by name (white stars).

Approximate Scale

N

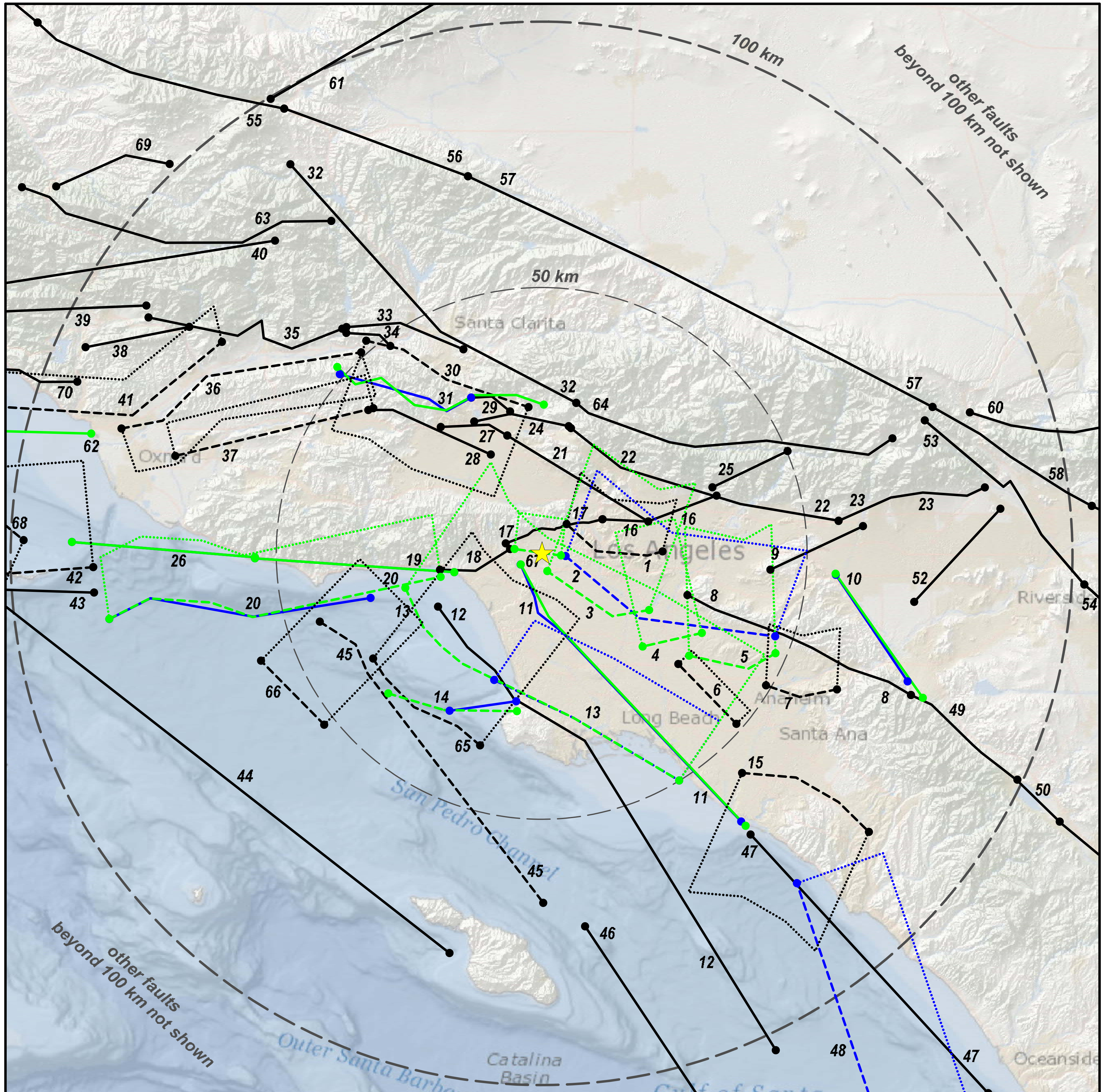
0 km 20 km 40 km 60 km

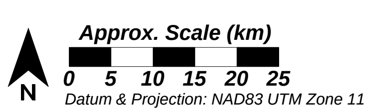
Datum & Projection: NAD83 UTM Zone 11

REGIONAL FAULT & SEISMICITY MAP

Date: MAR 2022	Project No.: 22010A	Project: 708 S. CLOVERDALE AVE.	Figure E-3a

GeoPentech

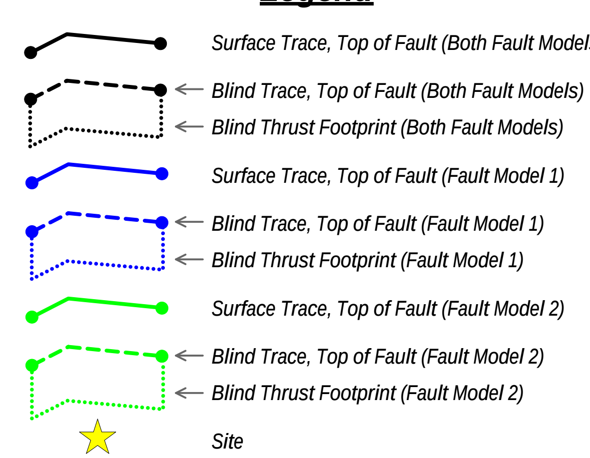




No.	Fault Name
1	Elysian Park (Upper)
2	Puente Hills
3	Puente Hills (LA)
4	Puente Hills (Santa Fe Springs)
5	Puente Hills (Coyote Hills)
6	Anaheim
7	Peralta Hills
8	Elsinore - Whittier
9	San Jose
10	Chino
11	Newport-Inglewood
12	Palos Verdes
13	Compton
14	Redondo Canyon
15	San Joaquin Hills
16	Raymond
17	Hollywood
18	Santa Monica
19	Malibu Coast
20	Anacapa-Dume
21	Verdugo
22	Sierra Madre
23	Cucamonga
24	Sierra Madre (San Fernando)
25	Clamshell-Sawpit
26	Malibu Coast (Extension)
27	Mission Hills
28	Northridge Hills
29	Santa Susana East (connector)
30	Northridge
31	Santa Susana
32	San Gabriel
33	Holser
34	Del Valle
35	San Cayetano

No.	Fault Name
36	Oak Ridge (Onshore)
37	Simi-Santa Rosa
38	Sisar
39	Mission Ridge-Arroyo Parida-Santa Ana
40	Santa Ynez (East)
41	Ventura-Pitas Point
42	Channel Islands Thrust
43	Santa Cruz Island
44	Santa Cruz-Catalina Ridge
45	San Pedro Basin
46	San Diego Trough North
47	Newport-Inglewood Offshore
48	Oceanside Blind Thrust
49	Elsinore - Glen Ivy
50	Elsinore - Temecula/Glen Ivy Stepover
51	Elsinore - Temecula
52	Fontana
53	San Jacinto - San Bernardino Valley
54	San Jacinto - San Jacinto Valley
55	San Andreas - Big Bend
56	San Andreas - North Mojave
57	San Andreas - South Mojave
58	San Andreas - North San Bernardino
59	San Andreas - South San Bernardino
60	Cleghorn
61	Garlock - West
62	Oak Ridge (Offshore)
63	Pine Mtn
64	San Gabriel Extension
65	San Pedro Escarpment
66	Santa Monica Bay
67	San Vicente
68	Channel Islands Western Deep Ramp
69	Big Pine (Central)
70	Red Mountain

Legend



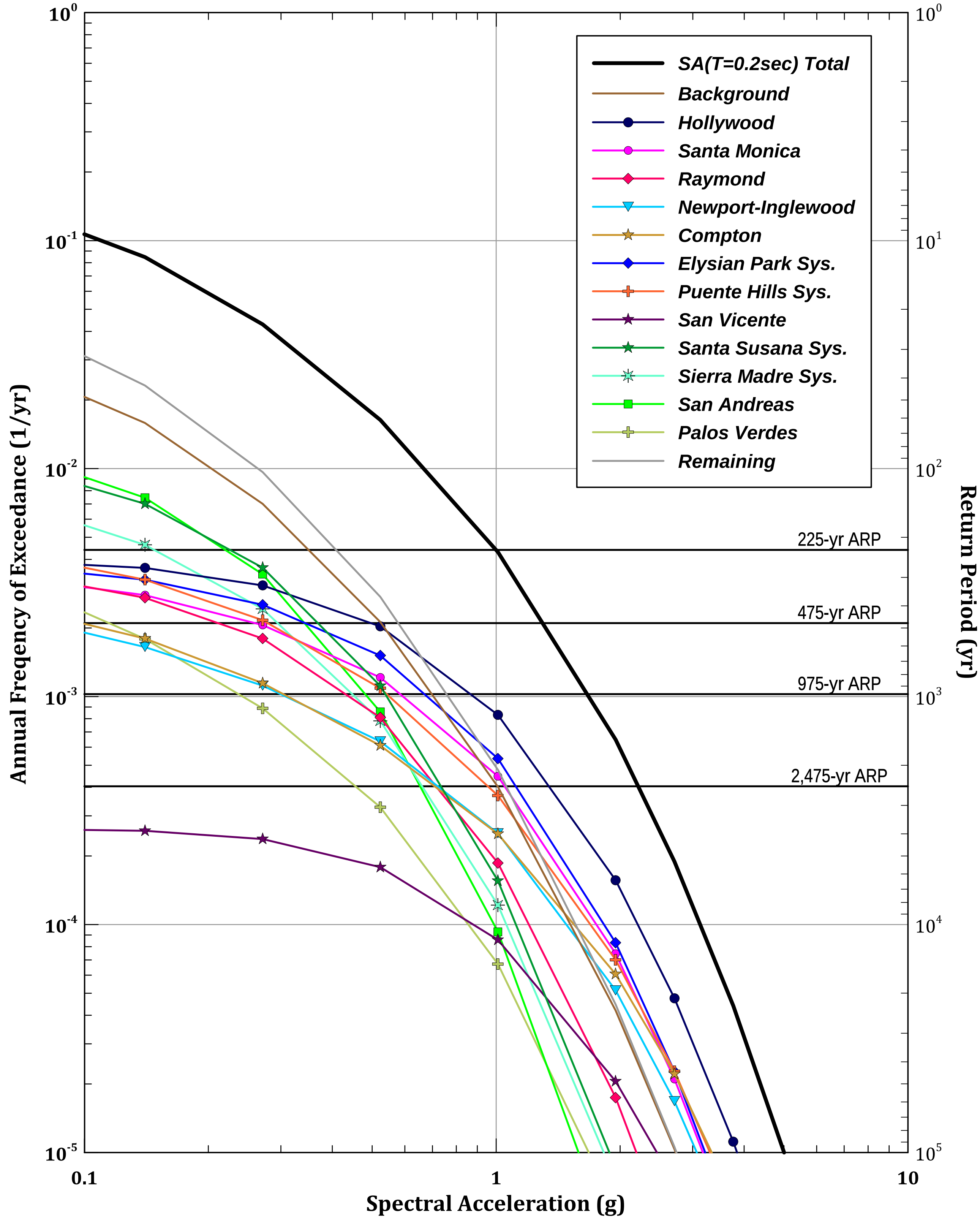
Notes:

1. Fault traces based on UCERF3 (WGCEP, 2013). Fault traces shown here are simplified and as-implemented in the PSHA calculations.
2. All faults within 100 km of site with slip rates greater than 0.05 mm/yr are shown. Slip rates are solution mean rates from UCERF3 (WGCEP, 2013).
3. Fault Models 1 & 2 based on UCERF3 (WGCEP, 2013). Seismic source characterization geometries and slip rates are generally as reported in WGCEP (2013). Magnitude-frequency distributions approximate the SWUS WAACY model (GeoPentech, 2015) with characteristic magnitude calculated from Shaw (2009) regression.

SIMPLIFIED FAULT MAP FOR PSHA

Project: 708 S. CLOVERDALE AVE.		Figure E-3b
Project No.: 22010A	Date: MAR 2022	

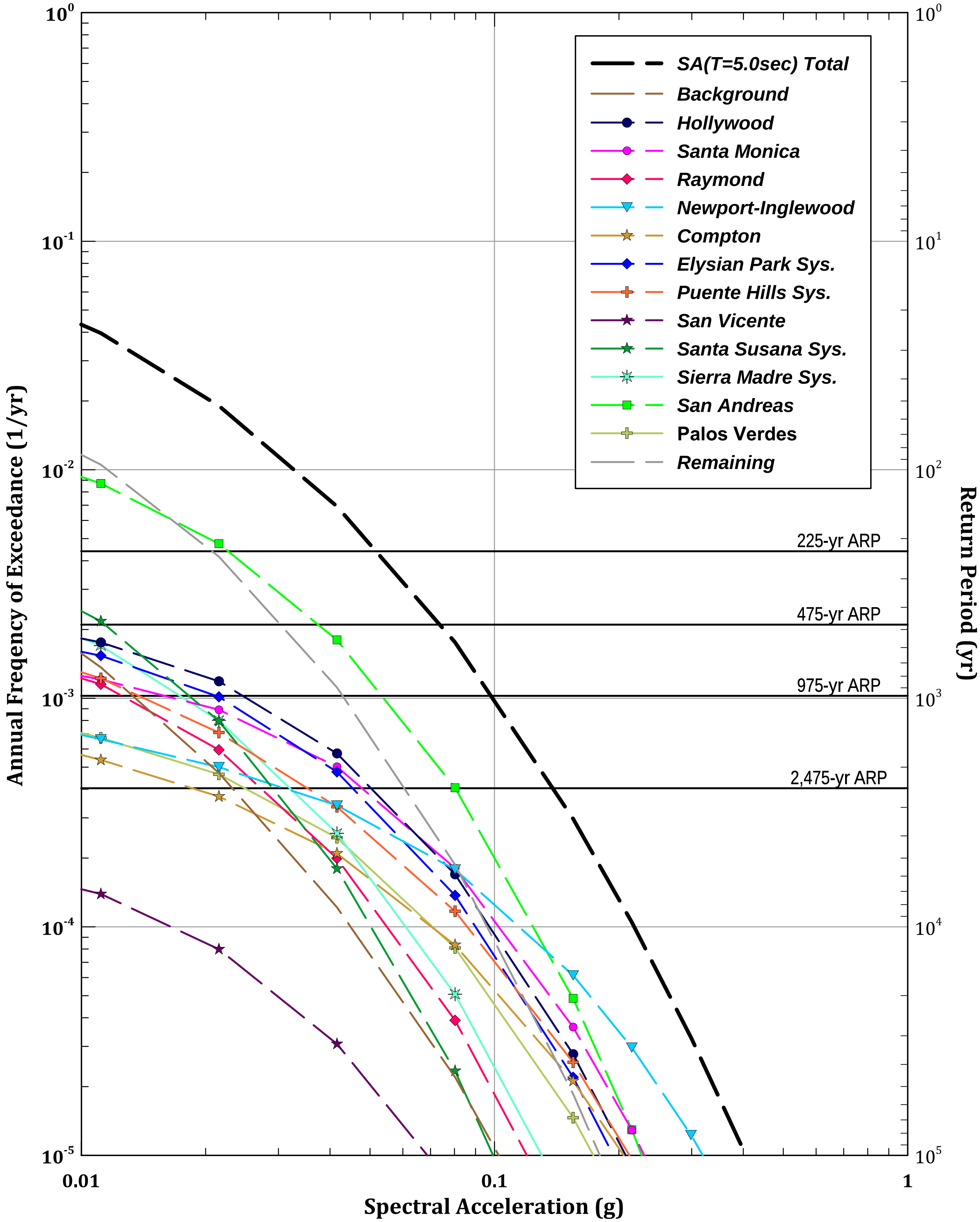
GeoPentech



SOURCE CONTRIBUTIONS TO 0.2-SECOND PERIOD

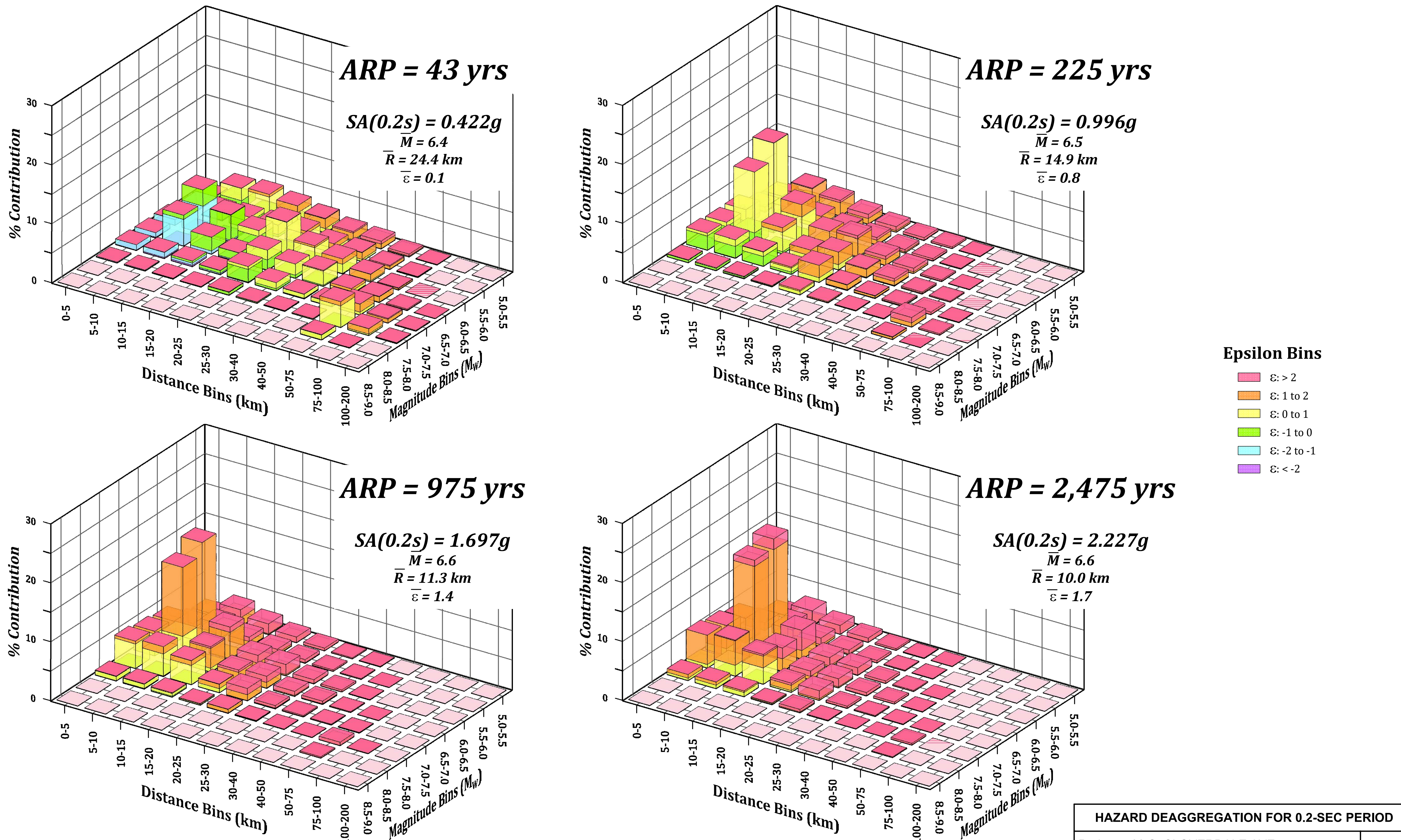
Project No.: 22010A	Project: 708 S. CLOVERDALE AVE.	Date: APR 2022	Figure E-4a

GeoPentech



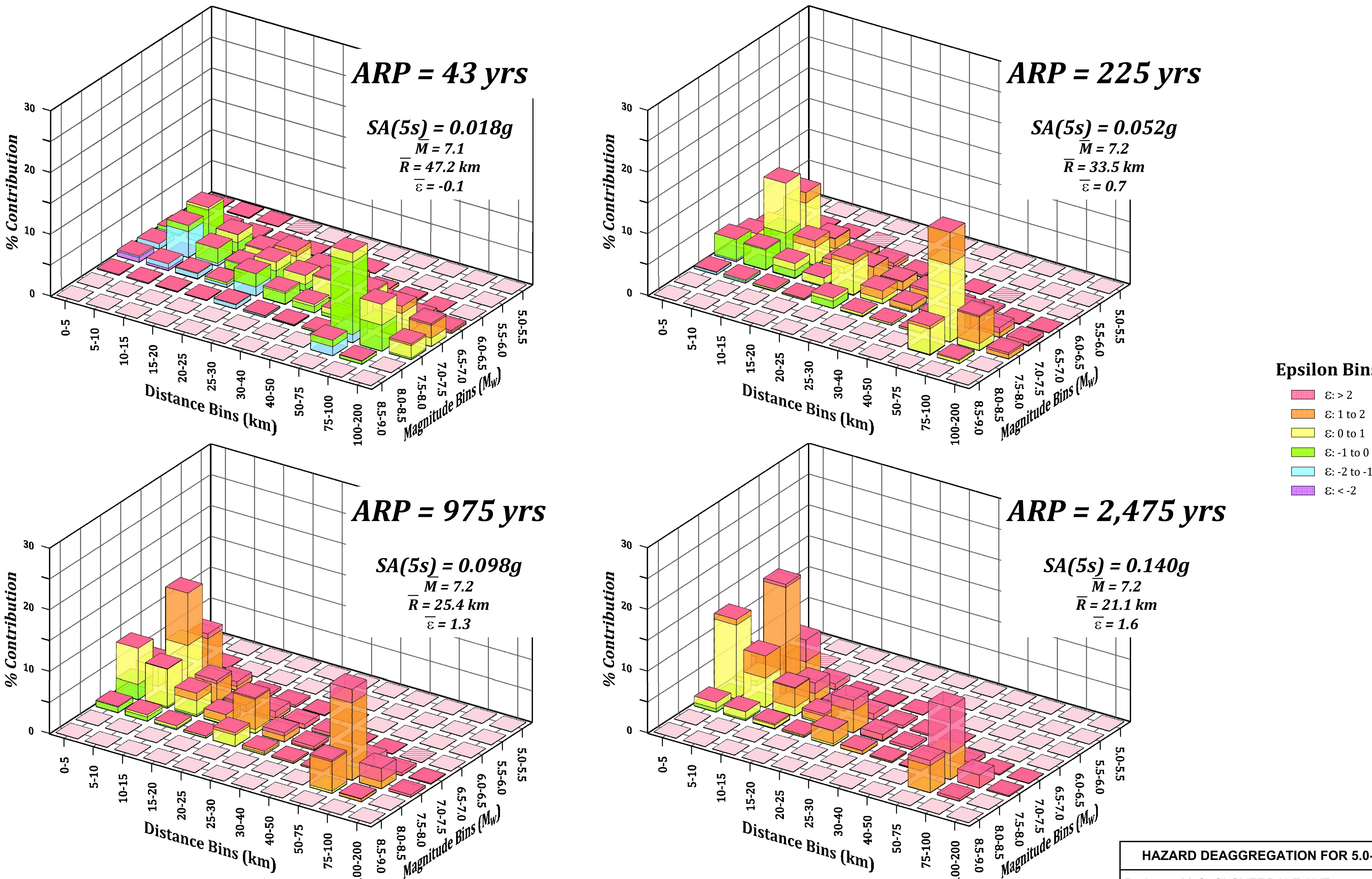
SOURCE CONTRIBUTIONS TO 5.0-SECOND PERIOD			
Project No.: 22010A	Project: 708 S. CLOVERDALE AVE.	Date: APR 2022	Figure E-4b

GeoPentech



HAZARD DEAGGREGATION FOR 0.2-SEC PERIOD		
Project: 708 S. CLOVERDALE AVE.		Figure E-5a
Project No.: 22010A	Date: APR 2022	

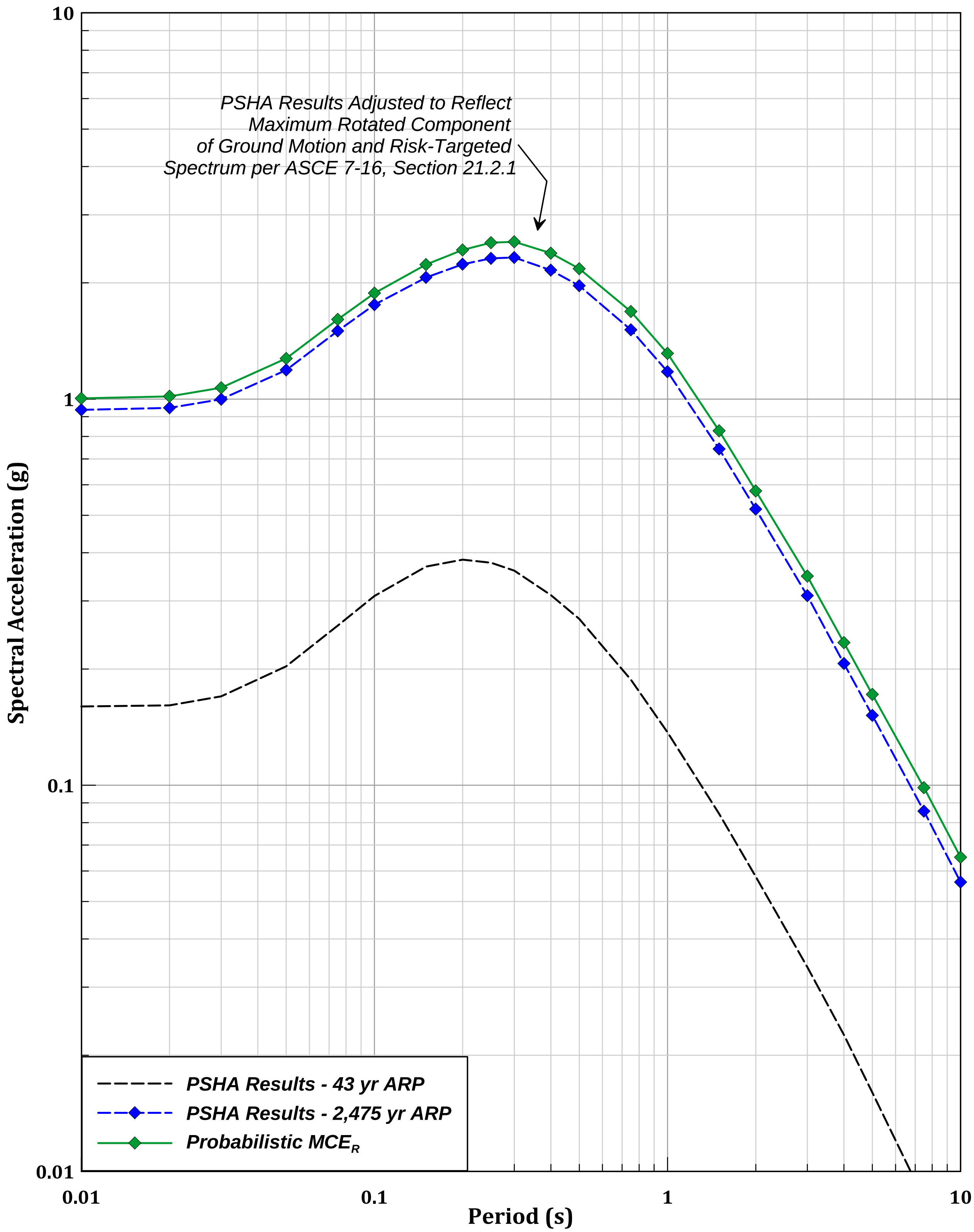
GeoPentech



HAZARD DEAGGREGATION FOR 5.0-SEC PERIOD		
Project: 708 S. CLOVERDALE AVE.		Figure E-5b
Project No.: 22010A	Date: APR 2022	

GeoPentech

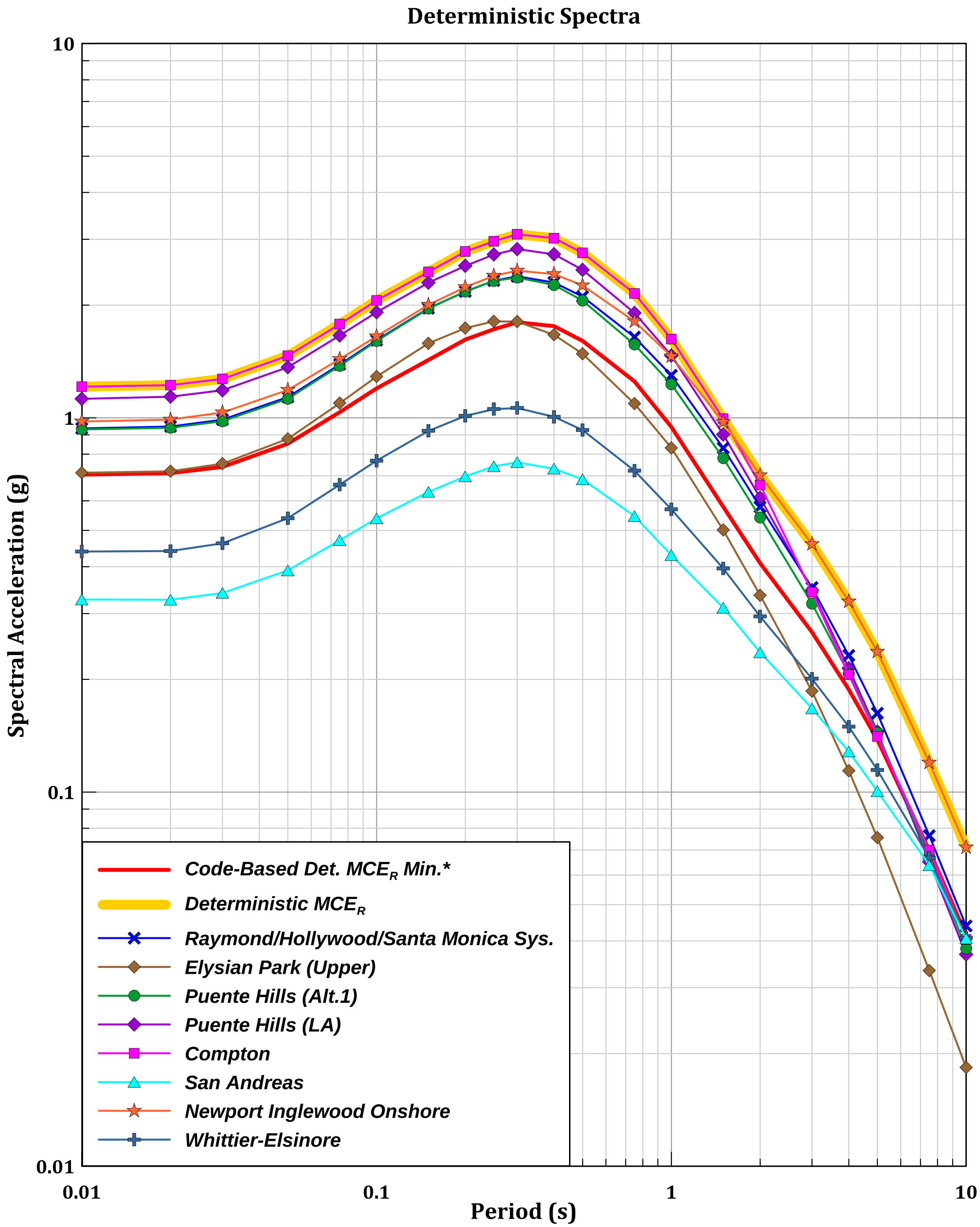
Probabilistic Spectra



Note: All spectra are for Damping (β) = 5.0% unless otherwise indicated.

PROBABILISTIC SPECTRA			
Project No.: 22010A	Project: 708 S. CLOVERDALE AVE.	Date: APR 2022	Figure E-6

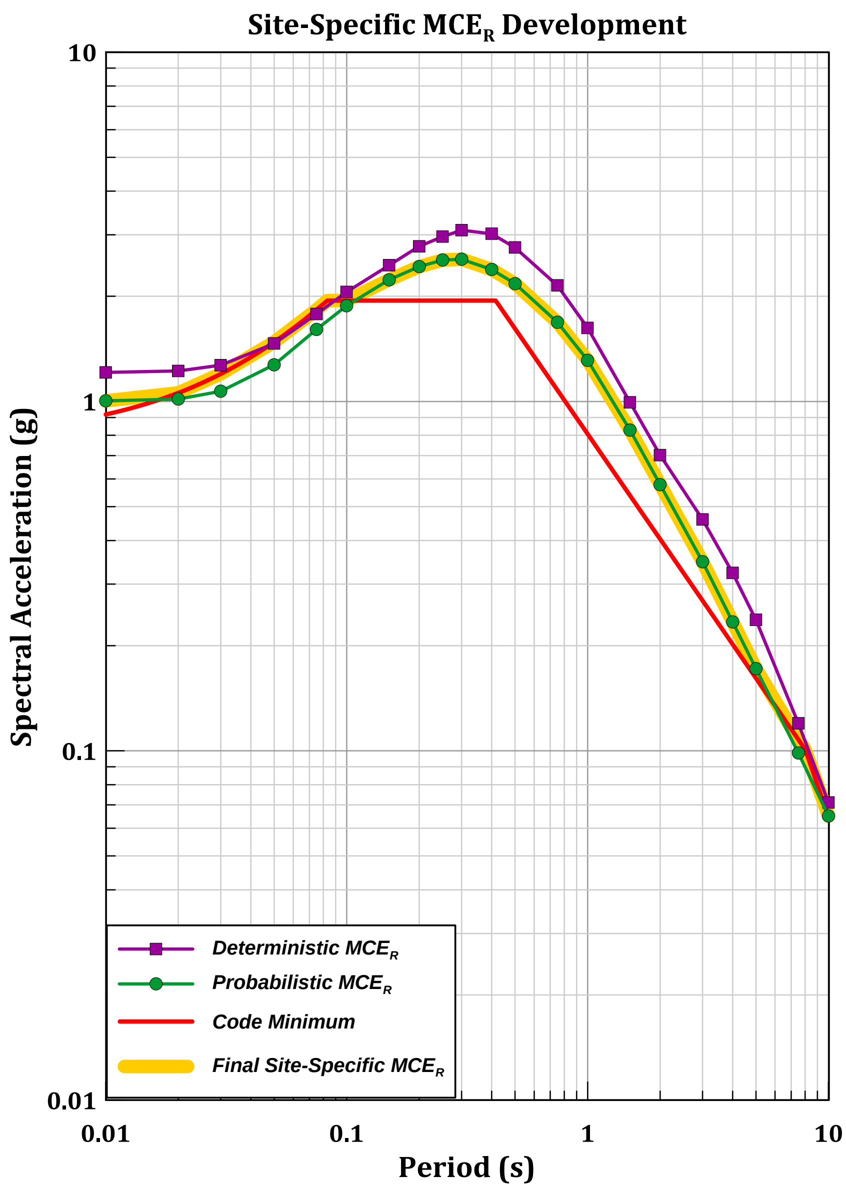
GeoPentech

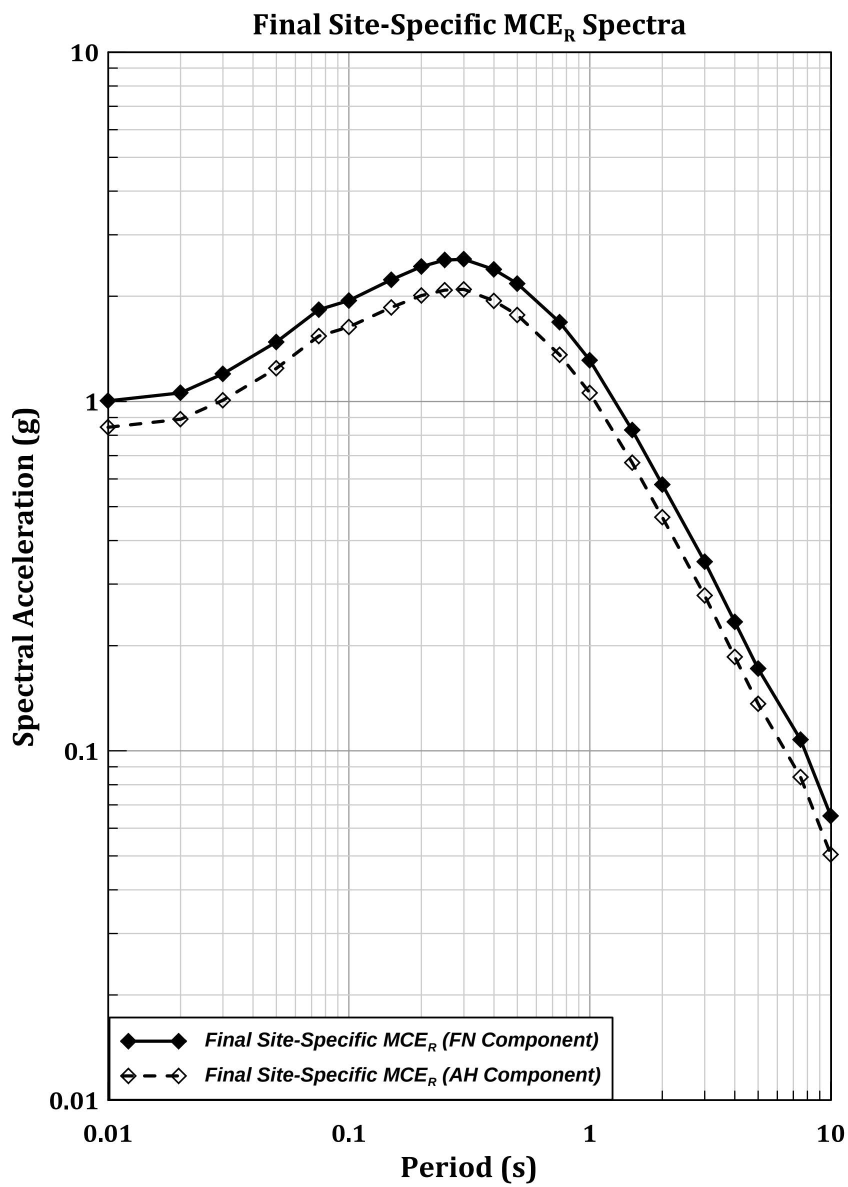


Notes: All spectra are for Damping (β) = 5.0% unless otherwise indicated.
Scenario spectra are 84th %tile maximum rotated ordinates.
Code-Based Det. MCE Min. per ASCE 7-16, Supplement 1, Ch. 21.2.2.

DETERMINISTIC SPECTRA			
Project No.: 22010A	Project: 708 S. CLOVERDALE AVE.	Date: APR 2022	Figure E-7

GeoPentech



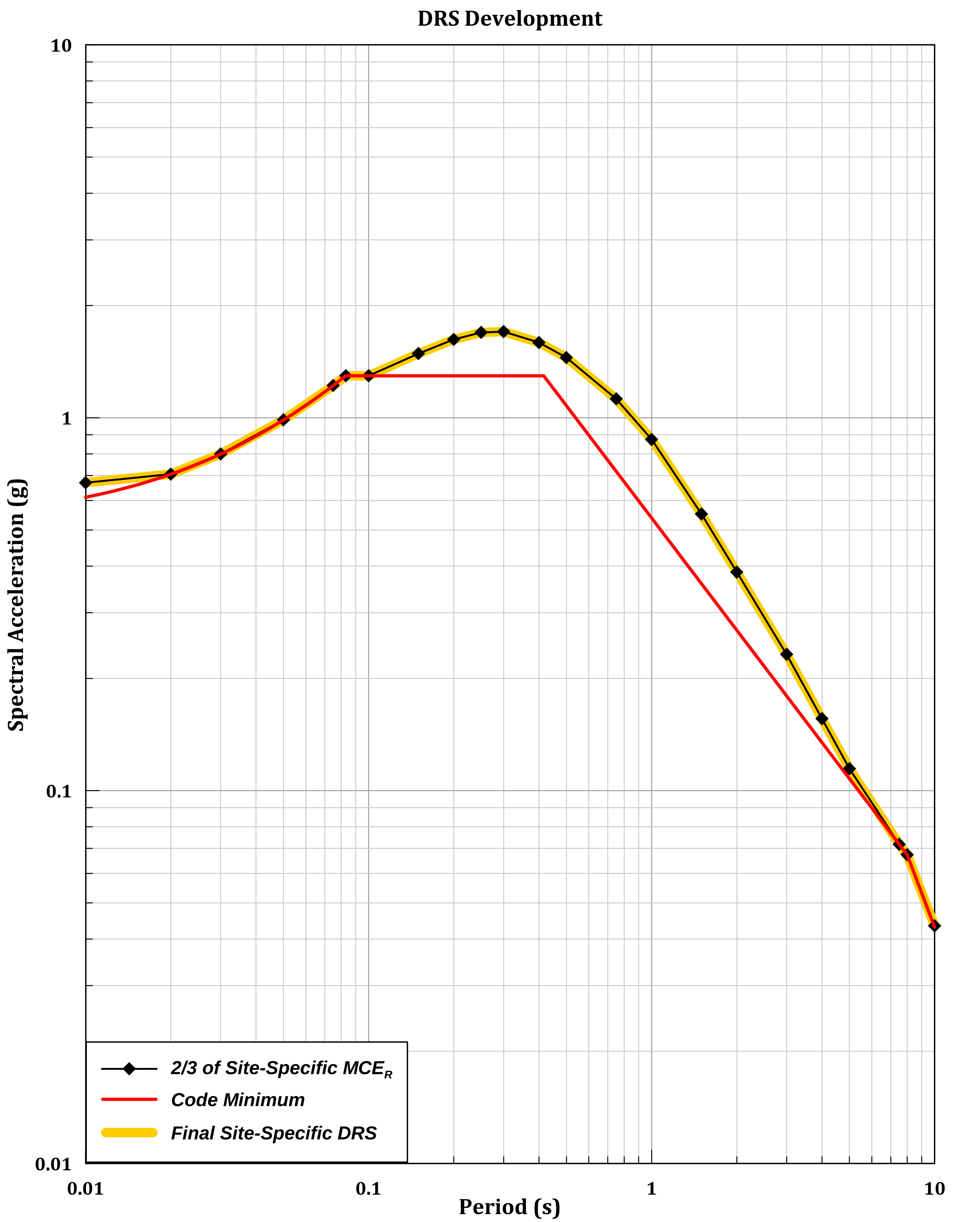


Note: All spectra are for Damping (β) = 5.0% unless otherwise noted.

FINAL SITE-SPECIFIC MCE$_R$ SPECTRA

Date: APR 2022	Project No.: 22010A	Project: 708 S. CLOVERDALE AVE.	Figure E-8

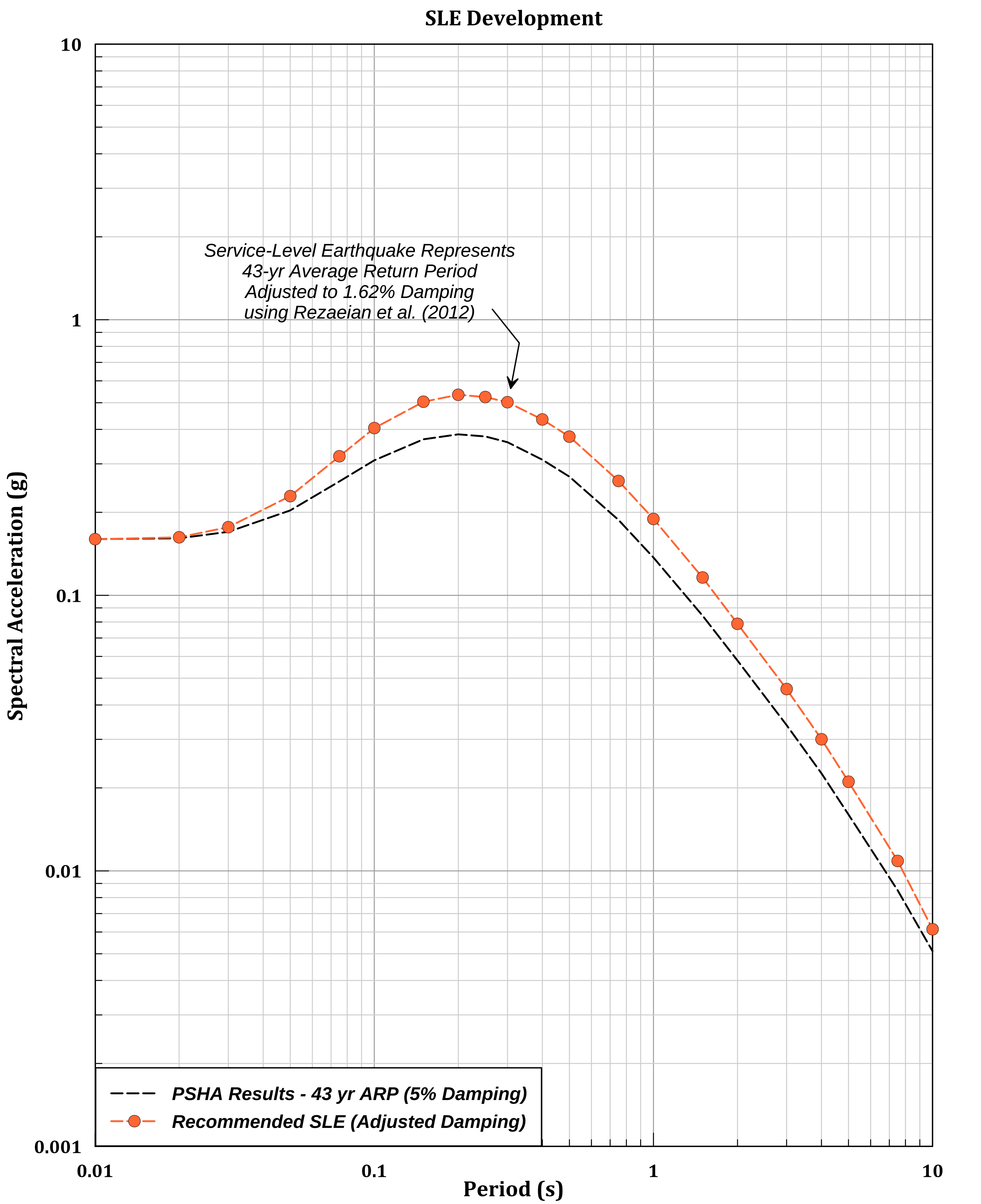
GeoPentech



Note: All spectra are for Damping (β) = 5.0% unless otherwise indicated.

SITE-SPECIFIC DRS DEVELOPMENT			
Project No.: 22010A	Project: 708 S. CLOVERDALE AVE.	Date: APR 2022	Figure E-9

GeoPentech



SITE-SPECIFIC SLE DEVELOPMENT			
Project No.: 22010A	Project: 708 S. CLOVERDALE AVE.	Date: APR 2022	Figure E-10

GeoPentech

APPENDIX F

SETTLEMENT ANALYSIS

F.1 ANALYSIS APPROACH AND RESULTS

Investigations at the site, shown in Figure F-1, encountered clay layers susceptible to consolidation settlement. The objective of the analysis documented in this appendix is to evaluate the potential settlement under the proposed structure (tower and podium) loads to provide an indication of the potential magnitude of settlements.

For this analysis and based on the most recent communications with the Structural Engineer of Record (SEOR), the average pressures on the mat foundation beneath the Tower section is 12 ksf applied at the bottom of an approximately 80 ft excavation, and the loading at the mat foundation beneath the Podium section is 3 ksf applied at the bottom of an approximately 71 ft excavation. Our analyses are performed based on net pressures at the depth of the excavations, i.e., 13.3 ksf under the tower and 2 ksf under the podium, as discussed in the main report section 8.2. The sections below provide further details of the analyses.

Consolidation Settlements

For the consolidation settlement analysis, we utilized the Settle3D software package (version 4.0) by Rocscience, Inc. of Toronto, Ontario. Representative samples from clay layers throughout the site were used to perform consolidation testing the relevant soil settlement parameters were estimated from the test results.

For settlement analyses, an idealized soil profile was developed based on the subsurface sections, shown in Figure F-2, to represent the range of varying conditions beneath the building footprints. We've divided the idealized soil model to consist of four Units; Unit 1 – predominantly consisting of sands and clayey sand mixtures, Unit 2 – clays consisting of overconsolidated lean and fat clays susceptible to consolidation settlements, Unit 3 – sands consisting of poorly-graded sands and silty sand mixtures, and Unit 4 – Fernando Formation claystone consisting of a more-weathered portion (upper) considered susceptible to consolidation, and a less-weathered portion (lower). The material properties for the idealized soil profile, and relevant laboratory test results utilized are tabulated in Figure F-3.

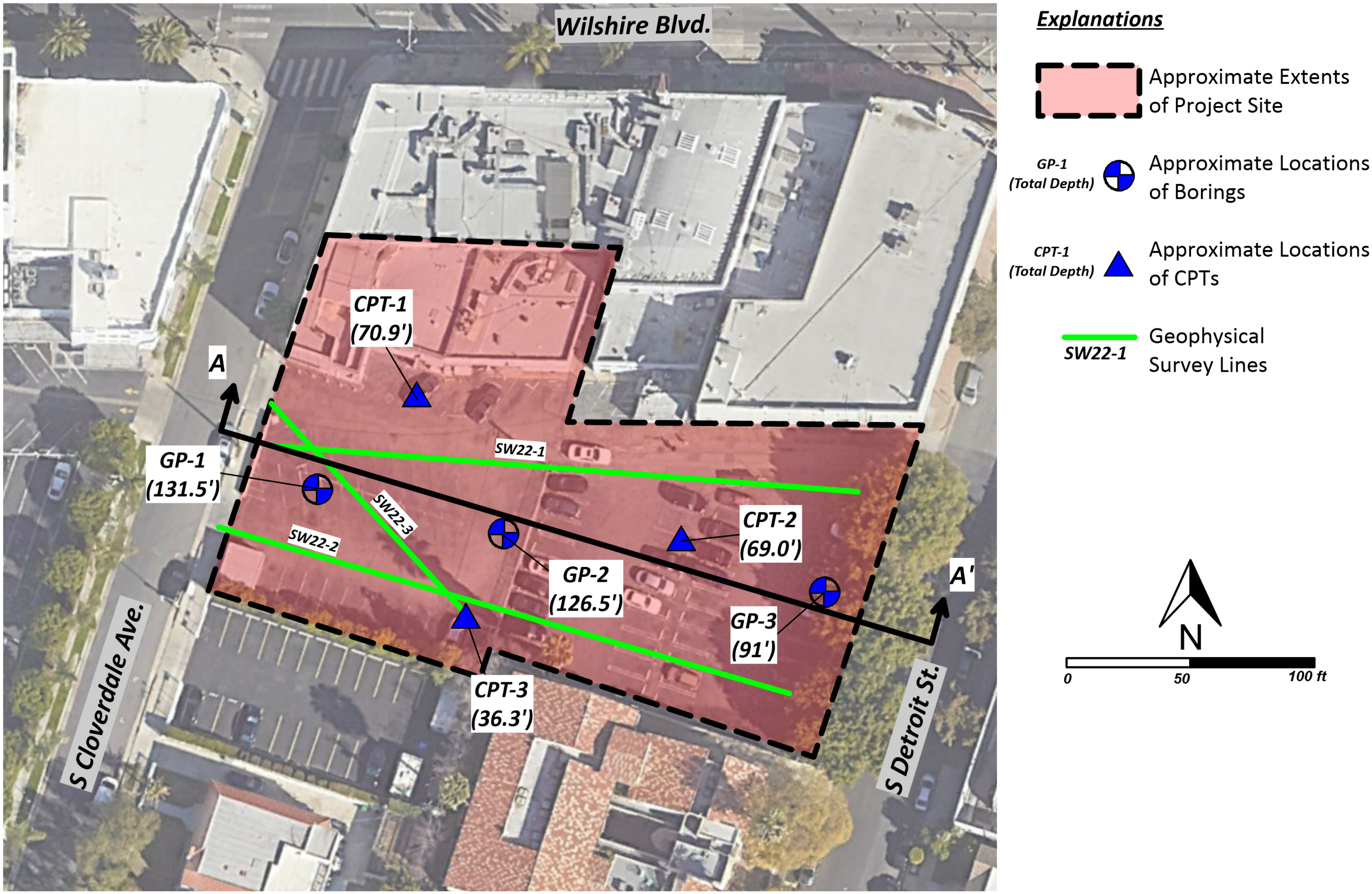
Figure F-4 graphically presents the model inputs and resulting settlements at various locations in the model (query points A, B, and C). Further, it is noted that model properties were developed based on our field investigation and laboratory testing results presented in Appendices A, B, and C and are used in our analysis. Note that a historic high groundwater level of 10 ft below ground surface was assumed for our consolidation analysis. Figure F-5 shows plan and isometric views of the analyzed Settle 3D model. Note that our model calculates consolidation rebound (due to excavation) and settlements (due to net structural loads) in an ultimate manner (i.e., to the end of primary consolidation).

F.2 SUMMARY

Based on the analysis results, we estimate that for loading at the mat foundation under the Tower section (i.e., average net uniform 10.3 ksf), the total consolidation induced settlement (total elastic and primary consolidation settlement) would be on the order of about 3 inches. For loading at the mat foundation under the Podium section (i.e., net uniform 2 ksf), we estimate the total consolidation-induced settlement would be on the order of 1 inch in the central portions of the mat, and approximately 1½ inches in the western portion of the Podium section, closer to the Tower.

References

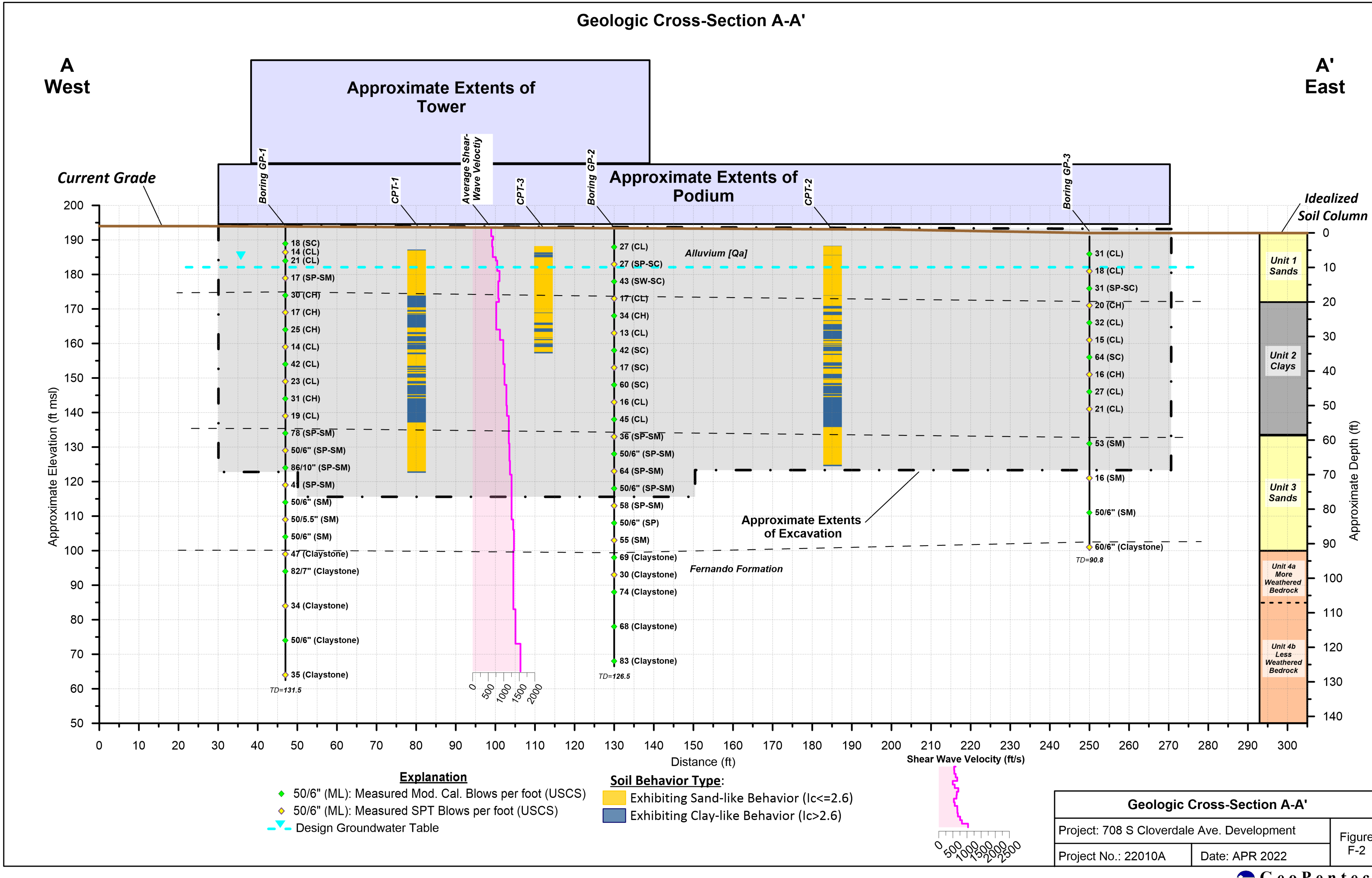
Rocscience, Inc., "Settle3D, Version 4.0." Toronto, Ontario, Canada.



Site Plan

Date: APR 2022	Project No.: 22010A	Project: 708 S Cloverdale Ave. Development	Figure F-1

GeoPentech



Geologic Cross-Section A-A'
A West
A' East
Approximate Extents of Tower
Approximate Extents of Podium
Current Grade
Idealized Soil Column
Boring GP-1
CPT-1
Average Shear-Wave Velocity
CPT-3
Boring GP-2
CPT-2
Boring GP-3
Alluvium [Qa]
Fernando Formation
Approximate Extents of Excavation
18 (SC)
14 (CL)
21 (CL)
17 (SP-SM)
30 (CH)
17 (CH)
25 (CH)
14 (CL)
42 (CL)
23 (CL)
31 (CH)
19 (CL)
78 (SP-SM)
50/6" (SP-SM)
86/10" (SP-SM)
41 (SP-SM)
50/6" (SM)
50/5.5" (SM)
50/6" (SM)
47 (Claystone)
82/7" (Claystone)
34 (Claystone)
50/6" (Claystone)
35 (Claystone)
TD=131.5
27 (CL)
27 (SP-SC)
43 (SW-SC)
17 (CL)
34 (CH)
13 (CL)
42 (SC)
17 (SC)
60 (SC)
16 (CL)
45 (CL)
36 (SP-SM)
50/6" (SP-SM)
64 (SP-SM)
50/6" (SP-SM)
58 (SP-SM)
50/6" (SP)
55 (SM)
69 (Claystone)
30 (Claystone)
74 (Claystone)
68 (Claystone)
83 (Claystone)
TD=126.5
31 (CL)
18 (CL)
31 (SP-SC)
20 (CH)
32 (CL)
15 (CL)
64 (SC)
16 (CH)
27 (CL)
21 (CL)
53 (SM)
16 (SM)
50/6" (SM)
60/6" (Claystone)
TD=90.8
Unit 1 Sands
Unit 2 Clays
Unit 3 Sands
Unit 4a More Weathered Bedrock
Unit 4b Less Weathered Bedrock
Approximate Elevation (ft msl)
Approximate Depth (ft)
Distance (ft)
Shear Wave Velocity (ft/s)
0 500 1000 1500 2000 2500
Explanation
50/6" (ML): Measured Mod. Cal. Blows per foot (USCS)
50/6" (ML): Measured SPT Blows per foot (USCS)
Design Groundwater Table
Soil Behavior Type:
Exhibiting Sand-like Behavior (Ic<=2.6)
Exhibiting Clay-like Behavior (Ic>2.6)
Geologic Cross-Section A-A'
Project: 708 S Cloverdale Ave. Development
Project No.: 22010A
Date: APR 2022
Figure F-2
GeoPentech

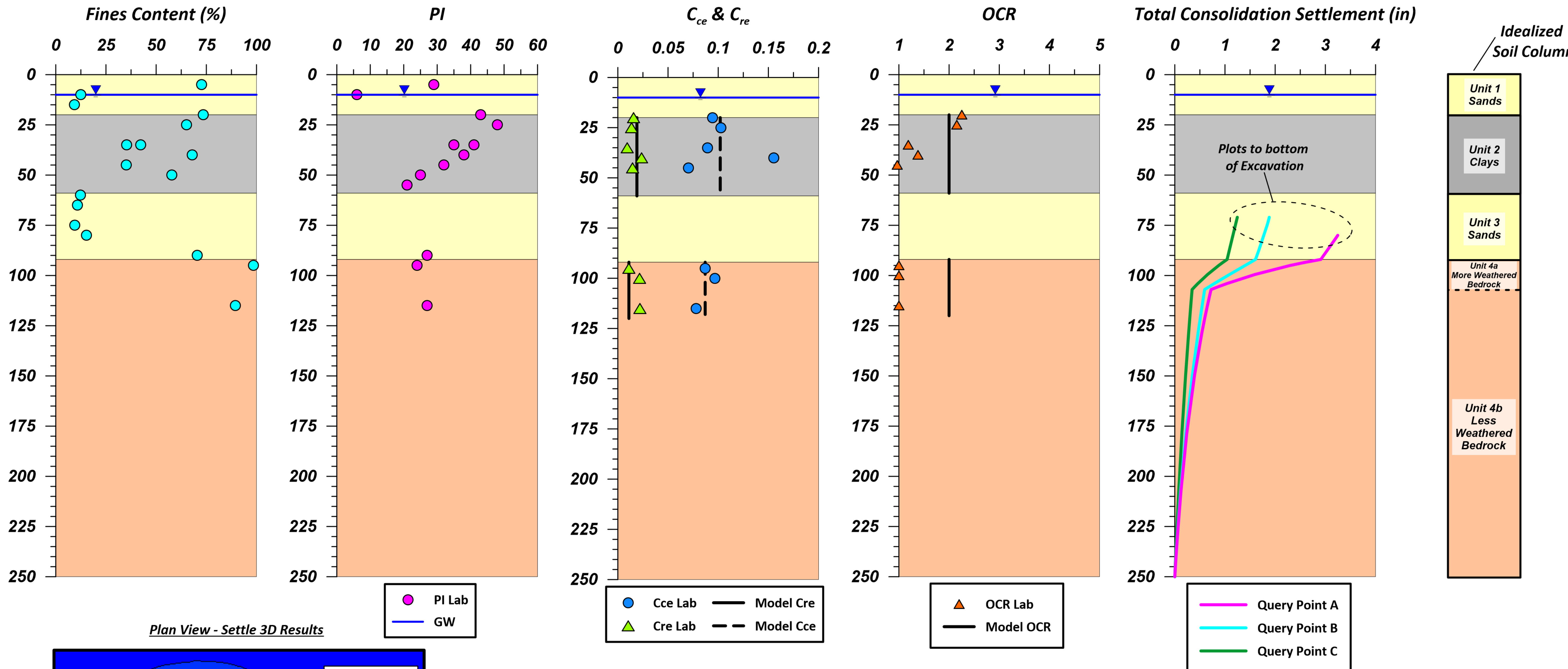
Interpreted Consolidation Laboratory Test Results

Borehole ID	Sample No.	Sample Type	Depth (ft)	USCS	C_c	C_r	σ'_p (psf)	σ'_{v0} (psf)	OCR	e_0	C_{ce}	C_{re}
GP-1	6	Mod Cal	20	CH	0.181	0.030 0.063	5500	2442	2.3	0.92	0.094	0.016 0.033
GP-1	10	Mod Cal	40	CL	0.247	0.038	6700	4848	1.4	0.59	0.155	0.024
GP-1	22	Mod Cal	100	Claystone	0.197	0.044	7500	11374	1.0	1.04	0.097	0.022
GP-2	5	Mod Cal	25	CH	0.197	0.026 0.051	6500	3014	2.2	0.92	0.103	0.014 0.027
GP-2	9	Mod Cal	45	SC	0.105	0.022	5200	5378	1.0	0.49	0.070	0.015
GP-2	19	Mod Cal	95	Claystone	0.172	0.022 0.038	5800	7693	1.0	0.98	0.087	0.011 0.019
GP-2	22	Mod Cal	115	Claystone	0.141	0.040	5500	8987	1.0	0.81	0.078	0.022
GP-3	7	Mod Cal	35	SC	0.151	0.016 0.031	5300	4456.1	1.2	0.69	0.089	0.009 0.018

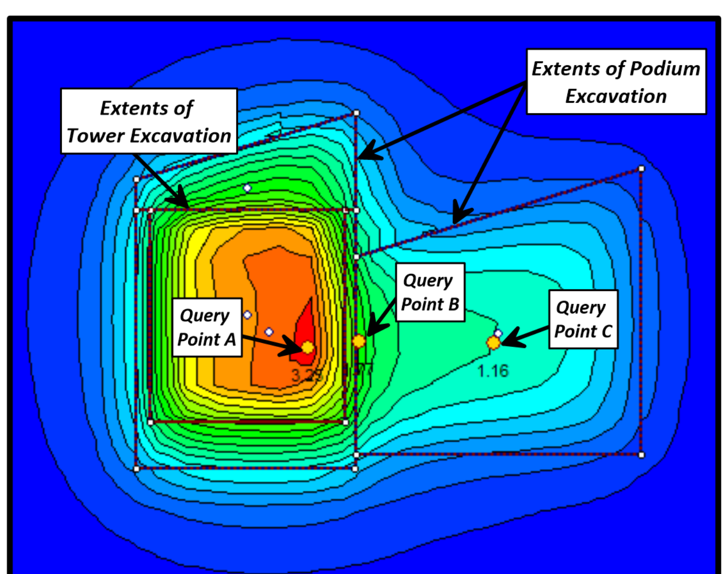
Settle 3D Model Input Parameters

Units	Depth Range (ft)	Layer Thickness (ft)	γ_{wet} (pcf)	E (ksf)	C_{ce}	C_{re}	OCR
Unit 1 - Sands	0-20	20	120	1500	-	-	-
Unit 2 - Clays	20-59	39	120	-	0.102	0.019	2
Unit 3 - Sands	59-92	33	130	3700	-	-	-
Unit 4a - More Weathered Bedrock	92-107	15	120	-	0.087	0.011	2
Unit 4b - Less Weathered Bedrock	107-250	143	125	10000	-	-	-

Representative Soil Column



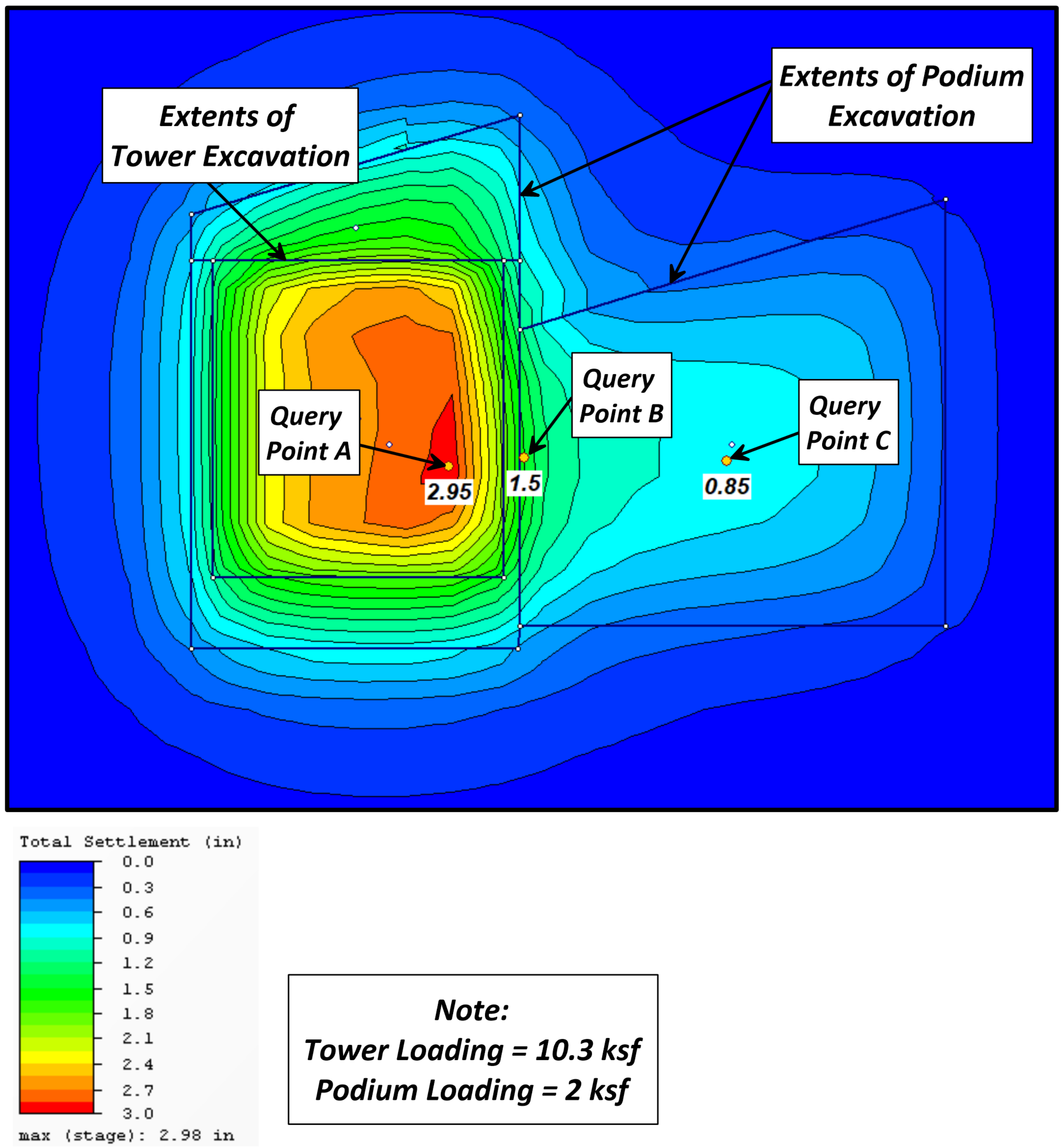
Plan View - Settle 3D Results



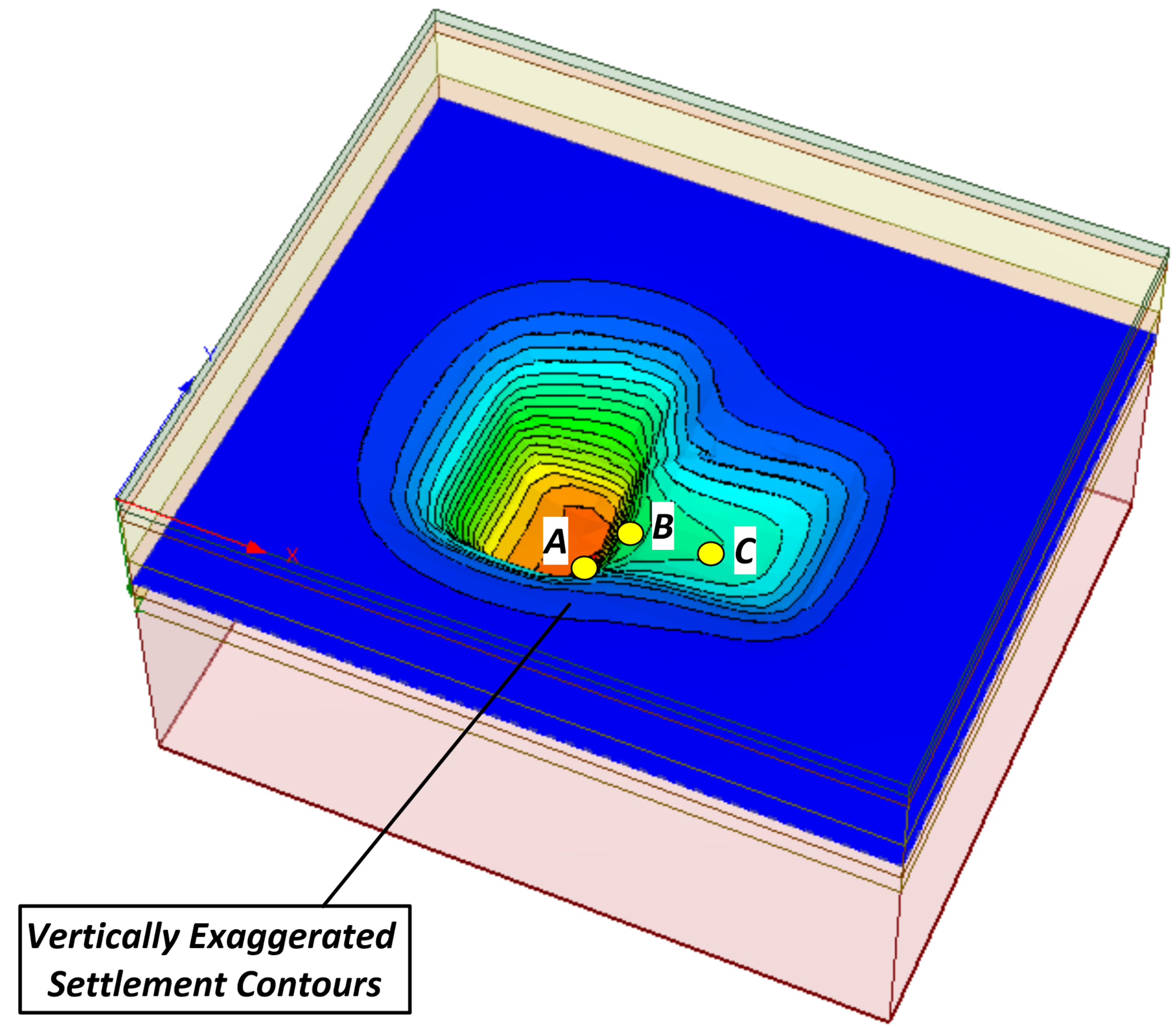
Representative Soil Column		
Project: 708 S Cloverdale Ave. Development		Figure F-4
Project No.: 21121A	Date: APR 2022	

GeoPentech

Plan View - Settle 3D Results



Isometric View - Settle 3D Results



Note:
Tower Loading = 10.3 ksf
Podium Loading = 2 ksf

Settle 3D Model Contour Maps		
Project: 708 S Cloverdale Ave. Development		Figure F-5
Project No.: 22010A	Date: APR 2022	

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