Appendix IS-2

Geotechnical Investigation

Report

GEOTECHNICAL INVESTIGATION REPORT
PROPOSED MIXED-USE DEVELOPMENT

930 N Sycamore Ave / 936 N Sycamore Ave / 940, 942 N Sycamore Ave / 941 N Orange Dr / 937 N Orange Dr / 931 N Orange Dr. Los Angeles, California

Prepared for

Onni Contracting (California), Inc.

315 W. 9th Street, Unit 801
Los Angeles, CA 90015

May 3, 2022

Prepared by

GeoPentech

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

May 3, 2022
Project No.: 21106A

Mrs. Meredith Megarry
Development Manager
1031 S Broadway, Suite 400
Los Angeles, California 90015

Subject: **Geotechnical Investigation Report**
Proposed Mixed-Use Development
930 N Sycamore Ave / 936 N Sycamore Ave / 940, 942 N Sycamore Ave / 941 N Orange Dr / 937 N Orange Dr / 931 N Orange Dr.
Los Angeles, California

Dear Mr. Spector:

This report presents the results of GeoPentech's geotechnical investigation for the proposed mixed-use development to be located at the addresses listed above in Los Angeles, California (also referred to as "940 N Sycamore") . This investigation was performed in general accordance with our agreement dated February 10, 2022, and a Change Order dated March 9, 2022.

This report provides geotechnical recommendations for the design and construction of the project in accordance with the plans provided to us. Results of the field and laboratory tests, as well as findings from our geologic hazard evaluation and ground-motion assessment, 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.

LICENSED PROFESSIONAL ENGINEER
MOHAMMAD ESLAMI
C 92278
EXP. 6/30/23
CIVIL
STATE OF CALIFORNIA

Mandro M. Eslami

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

James Heins

James Heins, EIT
Staff Engineer

REGISTERED PROFESSIONAL ENGINEER
RAMBOD HADIDI
GE 3051
EXP. 6/30/23
GEOTECHNICAL
STATE OF CALIFORNIA

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

REGISTERED GEOLOGIST
STEVE DUKE
No. 2269
CERTIFIED ENGINEERING GEOLOGIST
Exp. 7/31/23
STATE OF CALIFORNIA

Steve Duke, PG, PGp, CEG, CHg
Associate

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 mid-rise tower to be located on one parcel in Los Angeles, California (34.088184° N, -118.342244° W) (Project). The project is referred to as "940 N Sycamore" on project plans. The location of the Project site is shown on Figure 1, and the extent of the project site is shown on Figure 2. The Project site spans the following addresses and parcel numbers:

Street Address, Los Angeles, CA	APN
930 N Sycamore Ave. 936 N Sycamore Ave. 940 N Sycamore Ave. 942 N Sycamore Ave. 931 N orange Dr. 937 N Orange Dr. 941 N orange Dr.	5532-010-050

This report was prepared in accordance with the agreement between GeoPentech and Onni Group dated February 10, 2022, and a Change Order dated March 9, 2022.

2.0 PROJECT DESCRIPTION

Our understanding of the Project is based on the information provided with Request for Proposal (RFP) package, which includes a set of architectural drawings provided by SCB Architects dated March 3, 2022, as well as communications, and online meetings with the project team.

We understand that the proposed development includes the construction of a thirteen (13) story above grade tower and four (4) below grade levels in a parcel of land south of Romaine Street and between North Sycamore Avenue and North Orange Drive in the city of Los Angeles. The below-grade levels and the first six (6) stories of above-ground will be used as parking. The tower will be about 196 feet above grade and will include four underground basement levels down to about 62 feet below grade (i.e., top of slab at the lowest level). The mat foundation will be 5' thick with localized sections up to 11' thick down to 67 and 73 feet below grade. The approximate extents of the proposed structure are shown on Figure 2, and Figures 3a and 3b present 3D architectural view and architectural cross-sections of the proposed development, respectively.

We understand that the design for this structure will be carried out in conformance with the 2019 California Building Code (CBC 2019) and ASCE 7-16 requirements. We also understand that the Project, including geotechnical aspects of the design, will be submitted for review and approval to Los

Angeles Department of Building and Safety (LADBS). Furthermore, because the Project is taller than 160 ft above grade, its seismic design could be subject to LADBS's Peer Review Process.

This report presents the results of GeoPentech's geotechnical investigation (including field exploration) as well as recommendations for the design and construction of the proposed development.

3.0 SCOPE OF WORK

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

- ***Review of Existing Information:*** Performed a review of existing geotechnical, geologic, and seismic information for the site as well as the currently proposed development plans.
- ***Field Investigation and Laboratory Testing:*** Completed field work to investigate the nature and stratigraphy of the subsurface materials and to obtain soil samples for laboratory testing. The field investigation included drilling two (2) borings to depths of 132 and 161.5 feet below the existing ground surface within the footprint of the proposed tower. The approximate locations of GeoPentech's borings are shown on Figure 2. Select soil samples were taken to a soils lab for testing, including moisture and density, grain size analysis, Atterberg limits, consolidation, and direct shear tests.
- ***Geologic and Seismic Hazards Evaluation:*** Evaluated site subsurface conditions, and geologic setting, and assessed seismic and geologic 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 ASCE 7-16.
- ***Engineering Analysis:*** Performed engineering evaluations of the geotechnical data to develop recommendations for the design and construction of the foundations, walls below grade, shoring, excavation, earthwork criteria, and paving.
- ***Reporting:*** Prepared this report to present the results of the geotechnical investigation.

4.0 EXISTING SITE CONDITIONS

The site currently consists of paved parking lot with four small one-story structures and no vegetation. As shown on Figure 2, the site is bounded by N Sycamore Avenue to the west, an existing single-story commercial building to the north (7000 Romaine), North Orange Drive to the east, and an existing 7-

story building to the south. The site is relatively flat with a surface area of approximately 43,000 square feet.

We understand that the existing buildings at the site will be demolished prior to the construction of the proposed development. The existing ground surface elevation is approximately 278 feet (NAVD 88) and varies by about 1 to 2 feet across the site.

5.0 FIELD EXPLORATION AND LABORATORY TESTING

5.1 Boring Exploration

Two (2) borings (GP-1 and GP-2) were completed by GeoPentech at the site. GP-1 and GP-2 were advanced at the locations shown on Figure 2 to total depths of 132 and 161.5 feet below ground surface, respectively. Both borings were drilled using 8-inch diameter hollow-stem auger drilling equipment, and drilling mud was added to the auger when drilling below the groundwater to mitigate potential uplift at the bottom of the borehole. Standard Penetration Test (SPT) samples and modified California (MC) 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 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 distribution
- 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 complete results of laboratory tests along with the test results are presented in Appendix B. The laboratory testing is also summarized on the boring logs in Appendix A.

5.3 Geophysical Surveys

Geophysical surveys were performed to measure shear-wave (S-wave) velocity within the soil strata underlying 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 the 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 C.

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 1 mile 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 1¾ km north), Newport-Inglewood fault (located about 6.5 km to the southwest); the Santa Monica fault (located about 7 km to the southwest); and the Verdugo fault (located about 12 km northeast). The San Andreas Fault is located approximately 56 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 undocumented artificial fill soils underlain by Quaternary-age alluvial fan deposits (alluvium). Figure 6 show geologic Cross Section A-A', 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 characteristic of the geologic units based on the laboratory test results performed during this investigation is presented in Table 1.

Artificial Fill

The borings advanced at the site encountered artificial fill from the ground surface to a depth of about 10 feet. The fill generally consisted of moist, dark brown, very stiff, clay (CH and CL). The fill is likely the result of past demolition and construction activities at the site. Note that deeper fill, including debris, which was not encountered in the borings, may also exist on site, and the density/strength of the fill may also vary across the site.

Quaternary Alluvium

Quaternary age alluvium was encountered below the artificial fill in Borings GP-1 and GP-2 to depths of about 100 and 110 feet below the ground surface, respectively. The alluvium encountered generally consisted of stiff to hard clay (CH and CL) and medium dense to very dense sand (SC, SM, and SP-SC). The upper portion of the alluvium predominantly consisted of fine-grained soils to a depth of between about 60 and 80 feet below the ground surface. Below that depth, the alluvium predominantly consisted of coarse-grained soils. 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 (up to about 162 feet). The bedrock encountered consisted of hard interbedded siltstone and claystone. We classify the upper portions of the bedrock as weathered to the depths drilled and consider the formational material below the depths drilled as less weathered material due to very high blowcounts and refusal.

6.3 Groundwater

Groundwater was observed during drilling initially at a depth of 20 feet below the ground surface in Boring GP-1 and rose to approximately 16 feet after about 30 minutes. Groundwater was also initially encountered in Boring GP-2 at a depth of 25 feet below the ground surface and rose to about 18 feet after about 30 minutes.

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 approximately 10 to 20 feet below the ground surface, which is consistent with the encountered groundwater depth during field exploration. 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. Furthermore, there may be a potential for perched water to occur locally in sandy zones above the 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 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 1 mile south of the Earthquake Fault Zone for the Hollywood Fault Zone. Additionally, the site is not located within 1,000 feet of a mapped Holocene-active fault based on a review of mapping by (USGS, 2021), as shown on Figure 4a. Therefore, the site is not considered susceptible to surface fault rupture hazards.

7.2 Seismic Shaking

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 D of the report. The following site-specific response spectra are developed for the design of the project:

- 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). Note that the MCE_R response spectrum captures maximum rotated (i.e., RotD100) conditions that are deemed appropriate to support the design process.
- A “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.

For completeness, a “Service-Level Earthquake” uniform hazard spectrum with average horizontal spectral ordinates at 1.95% damping (corresponding to a 50% probability of exceedance in a 30-year period; i.e., a 43-year return period) 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 $M_W \geq 7$.

The code-based, site-specific “Design Level” or DRS uniform hazard spectrum with risk-targeted, maximum-rotated ordinates at 5% damping has also been provided for reference and comparison to site-specific values. However, the code-based values are superseded by the site-specific values.

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. This is consistent with the results of our field investigation, which did not encounter soils susceptible to liquefaction. As such, liquefaction is not considered to be a hazard at this site.

7.4 Seismically-Induced Settlement

Seismically-induced settlement may 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 has the potential to cause serious structural damage.

As part of the site development, the upper approximately 67 to 73 feet of the site will be excavated and the soils removed for the new basement levels which will extend to below the groundwater, thereby removing all the unsaturated soils that are potentially susceptible to 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. 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, 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 12 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 278 feet above mean sea level and is relatively far from the shoreline. As such, the site is not considered susceptible to tsunami hazards.

7.9 Landslide

A potential for landsliding 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 zone (CGS, 2018). Therefore, the potential for landsliding 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 Buffer Zone, as defined by the City of Los Angeles and subject to the City’s methane code. We recommend that a methane study should be performed by a methane specialist to provide specific methane mitigation recommendations for the design and construction of the project.

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 67 to 73 ft, about 57 to 63 ft below the groundwater level, 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.

Foundation System: Due to relatively high building loads (average bearing pressure of 6,000 psf), 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.

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 2019 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 = 2.089g and S_1 = 0.749g 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 D and are defined below. These parameters were developed in accordance with ASCE 7-16, Section 21.3.

S_{DS} = 1.606 g, based on 90% of the spectral acceleration at a period of 0.3-seconds

S_{D1} = 0.890 g, based on the spectral acceleration at a period of 1.0-second

S_{MS} = 2.409 g, based on 1.5 times S_{DS}

S_{M1} = 1.355 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 D.

8.2 Foundation Recommendations

Preliminary loading conditions provided to us by the Structural Engineer indicate an average bearing pressure of about 6,000 pounds per square foot (psf) under the footprint of the building. Considering four (4) below-grade parking levels and assuming a slab thickness of 5 ft, excavation of the upper 67-

73 feet of soils is anticipated. Due to relatively high building loads, we recommend a mat foundation for the building.

Mat Foundation

The proposed building is recommended to be supported on a mat foundation resting on native alluvial deposits. A mat foundation founded on native alluvium at a depth of approximately 67-73 ft from existing grade may be designed using an allowable bearing capacity of 6,000 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 E), we estimate the settlement of the proposed building on a mat foundation in the manner recommended will be less than 3 inches for an average mat bearing pressure of 6,000 psf. Differential settlement is estimated to be about half of the total settlement across the mat in either direction.

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 foundation.

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 Walls Below Grade

Lateral Earth Pressure

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 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 subterranean walls, hydrostatic pressure can be ignored in the wall design. 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 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.

<u>Drainage</u>

Given the shallow groundwater, we anticipate that the building walls below grade will be designed to resist hydrostatic pressures. 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.

8.5 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.

8.6 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 67-73 feet of the site will be excavated for the proposed basement level and foundation, 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 Lateral Pressures

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 would not be feasible.

For the design of the shoring system, we recommend the following lateral earth pressures for drained and submerged conditions, respectively. For cantilever piles we recommend using the triangular lateral pressure with a maximum pressure equal to 40H (psf, drained) and 24H (psf, submerged). For the design of braced or tied-back shoring, we recommend using a trapezoidal pressure distribution with a maximum pressure equal to 24H (psf, drained) and 15H (psf, submerged), where H is the retained height in feet. For submerged conditions, hydrostatic pressures due to groundwater should also be included. 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.

Excavation Support System

The selection and design of an appropriate excavation support system should be coordinated with a qualified shoring engineer. Table 2 provides a summary of engineering properties for the geologic units present within the project site that can be used for the design of the excavation support system. Additional recommendations on excavation support systems are provided in the following:

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.

8.7 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.

8.8 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.

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

American Society of Civil Engineers, 2016, Minimum Design Loads for Buildings and Other Structures, ASCE Standard 7-16.

ASTM International, West Conshohocken, PA, www.astm.org.

California Building Standards Code (CBC), (2019). California Code of Regulations. California Building Standards Commission Based on the 2018 International Building Code, Sacramento, CA.

California Department of Transportation, Materials Engineering and Testing Services, 2021, “Corrosion Guidelines, Version 3.2”, dated May 2021.

California Department of Water Resources, 2014, “Summary of Recent, Historical, and Estimated Potential for Future Land Subsidence in California, Technical Memorandum.”

California Geologic Survey (CGS), 1986, Special Studies Zones for the Hollywood 7.5-Minute Quadrangle.

California Geologic Survey (CGS), 1998, Seismic Hazard Report for the Hollywood 7.5-Minute Quadrangle, Los Angeles County, California: Open File Report 98-17.

California Geologic Survey (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, Plates 8 and 9, scale 1:100,000.

California Geological Survey (CGS), 2014, Earthquake Zones of Required Investigation, Hollywood Quadrangle, released 6 November 2014, scale 1:24,000.

Department of City Planning Los Angeles, California, 1996, "Safety Element of the Los Angeles City General Plan," adopted by the City Council on November 26, 1996.

Federal Emergency Management Agency (FEMA), 2008, “Flood Insurance Study for Los Angeles County,” September 26, 2008.

ISRM (International Society for Rock Mechanics), 1978. “Rock Characterization Testing and Monitoring (ISRM Suggested Methods).” Brown, E.T., ed., Pergamon Press, London, 211 pp.

Los Angeles, County of, 1990, "Seismic Safety Element."

Rocscience, Inc., 2014-2016, “Settle3D, Version 4.0.” Toronto, Ontario, Canada.

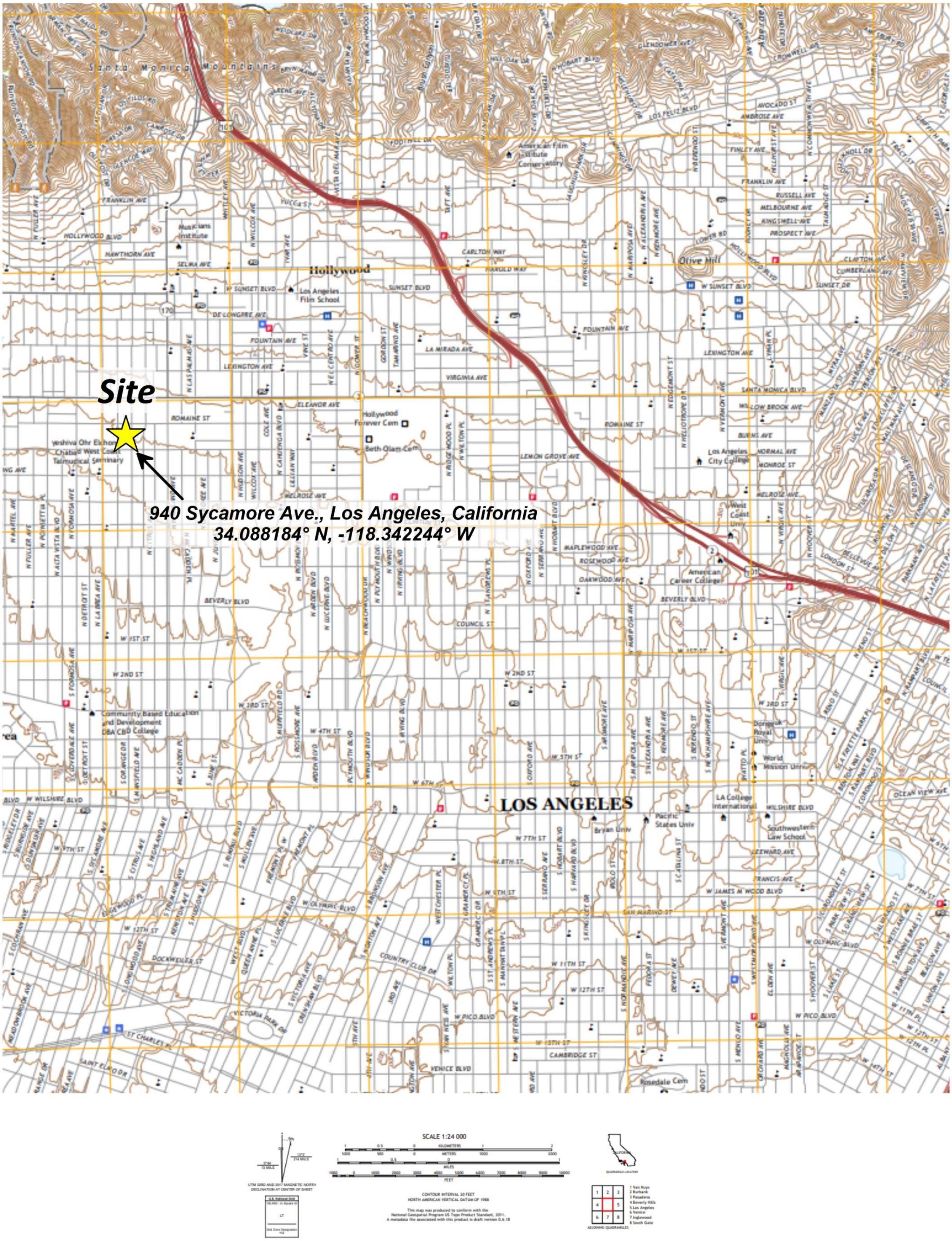
United States Geological Survey, 2018, Quaternary fault and fold database for the United States, accessed December 2021, available at [http//earthquakes.usgs.gov/regional/qfaults/].

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	Low to High Plasticity Clays (CL and CH)	0 to 10	Range Median (# Tests)	18 18 (1)	132 132 (1)	-	-	-	-	-	-	-	-	-	-
Quaternary Alluvium	Low to High Plasticity Clays (CL and CH)	10 to 60	Range Median (# Tests)	15 to 30 20 (10)	124 to 133 130 (10)	-	-	10 to 63 50 (8)	10 to 40 29 (9)	33 to 59 49 (9)	34 34 (1)	300 300 (1)	1.3 to 2.1 1.8 (3)	0.081 to 0.118 0.103 (3)	0.009 to 0.030 0.015 (5)
	Sands (SC, SM, and SP-SC)	60 to 100	Range Median (# Tests)	19 to 31 22 (5)	127 to 130 128 (2)	0 to 1 1 (2)	28 to 87 58 (2)	5 to 71 26 (7)	17 to 18 18 (2)	31 to 33 32 (2)	31 to 35 32 (3)	350 to 950 450 (3)	-	-	-
Fernando Formation	More weathered	100 to 150	Range Median (# Tests)	21 to 27 23 (6)	125 to 129 127 (4)	-	-	31 to 85 70 (3)	8 to 25 13 (6)	34 to 50 39 (6)	-	-	-	-	0.010 to 0.021 0.014 (4)
	Less weathered	>150	-	-	-	-	-	-	-	-	-	-	-	-	-

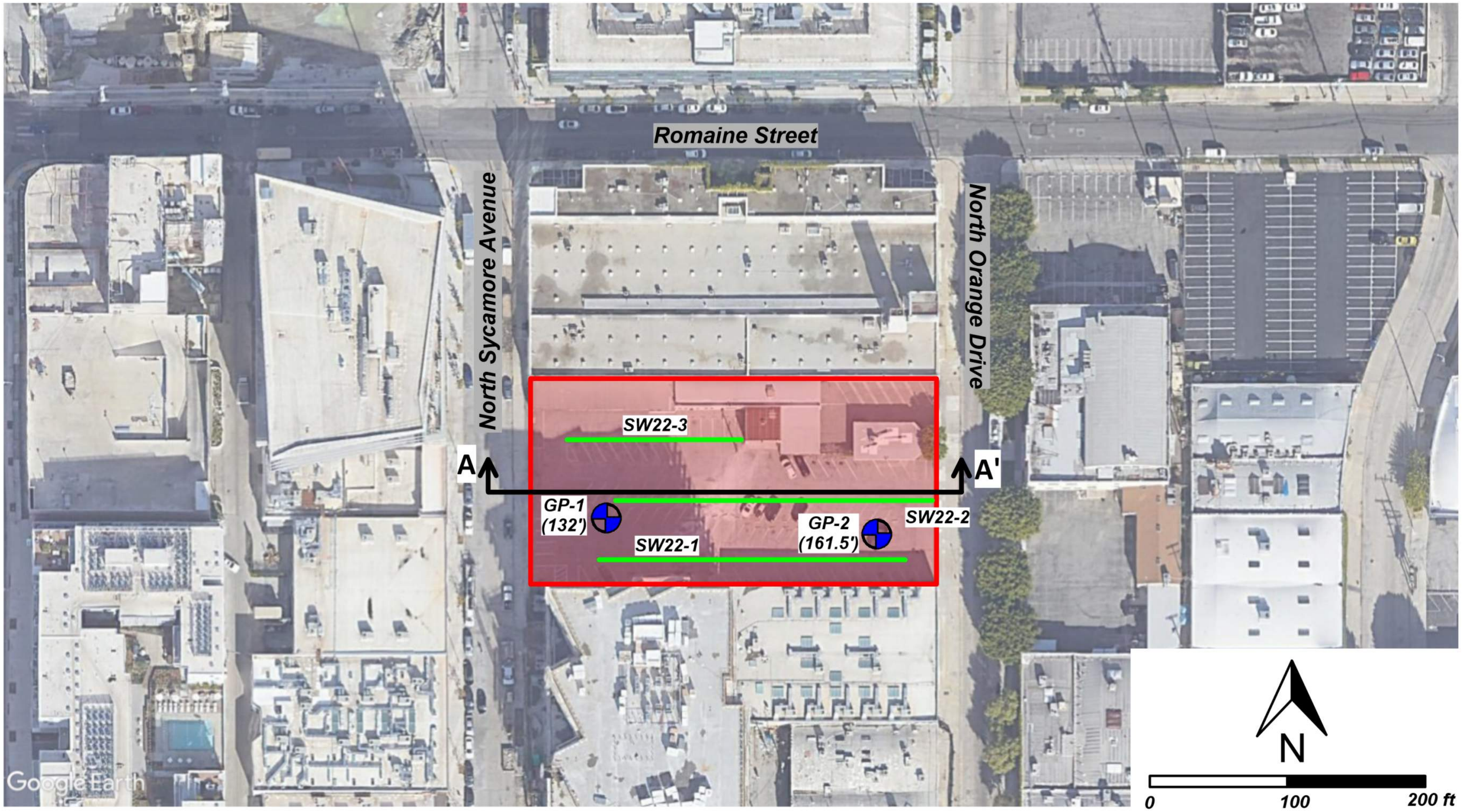
Table 2 – Summary of Engineering Properties for Design

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	Low to High Plasticity Clays (CL and CH)	0 to 10	132	0.110	0.011	30	-	0.25
Quaternary Alluvium	Low to High Plasticity Clays (CL and CH)	10 to 60	130	0.110	0.011	34	300	0.35
	Sands (SC, SM, and SP-SC)	60 to 100	130	-	-	32	-	-
Fernando Formation	More weathered	100 to 150	127	-	0.010	-	-	Very Weak to Weak (ISRM, 1978)
	Less weathered	>150	130	-	-	-	-	-



SITE LOCATION MAP			
Project No.: 21106A	Project: 940 Sycamore Development	Date: APR 2022	Figure 1

GeoPentech



Explanations

Approximate Footprint of Proposed Development

GP-1 (Total Depth) *Boring Locations*

SW22-1 *Geophysical Survey Lines*

Cross Section

SITE PLAN

Date: APR 2022	Project No.: 21106A	Project: 940 N Sycamore Ave. Development	Figure 2

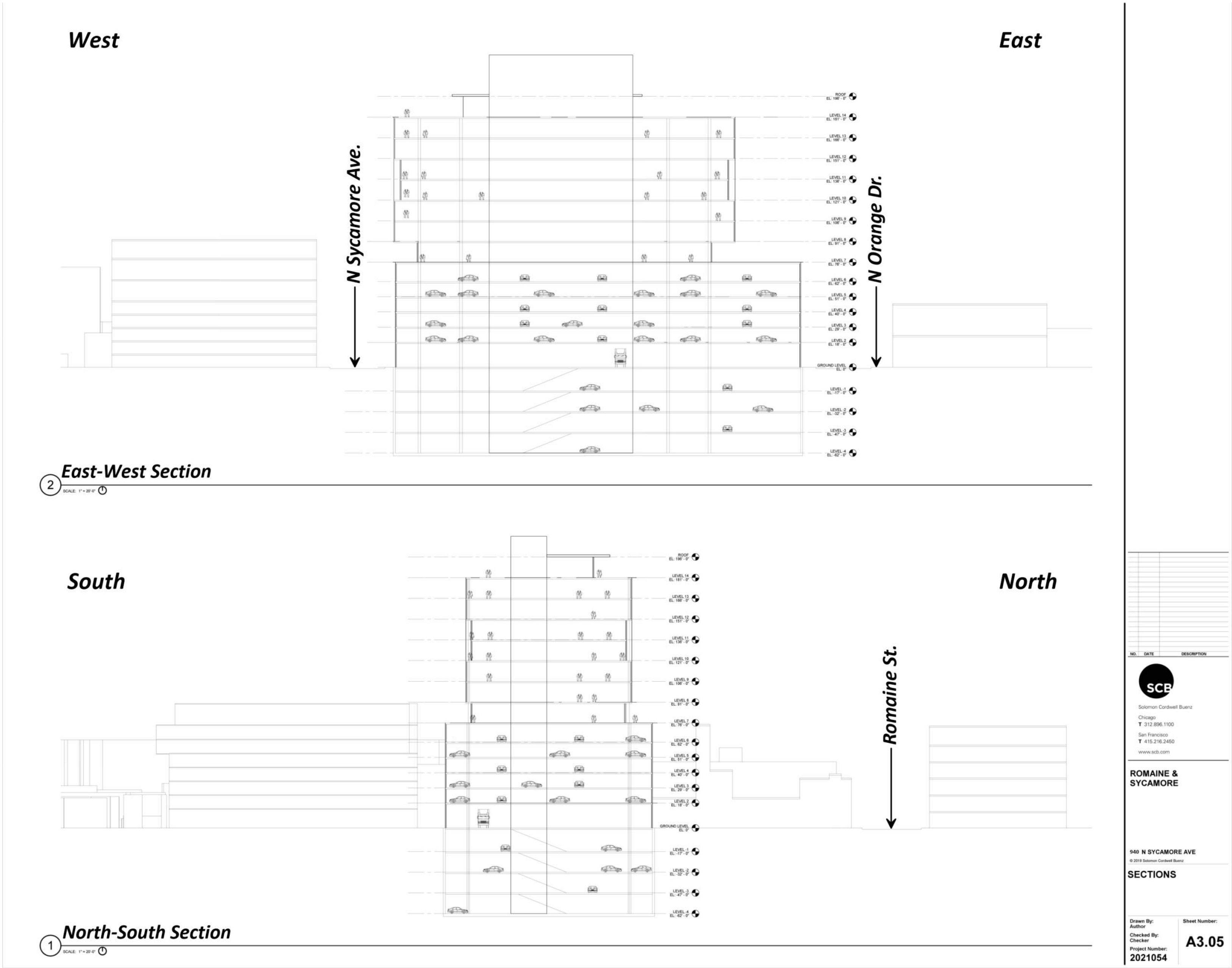
GeoPentech

Source: SCB Architectural Package (dated 03/03/2022)

3D ARCHITECTURAL VIEW

Date: APR 2022	Project No.: 21106A	Project: 940 N Sycamore Ave. Development	Figure 3a

GeoPentech

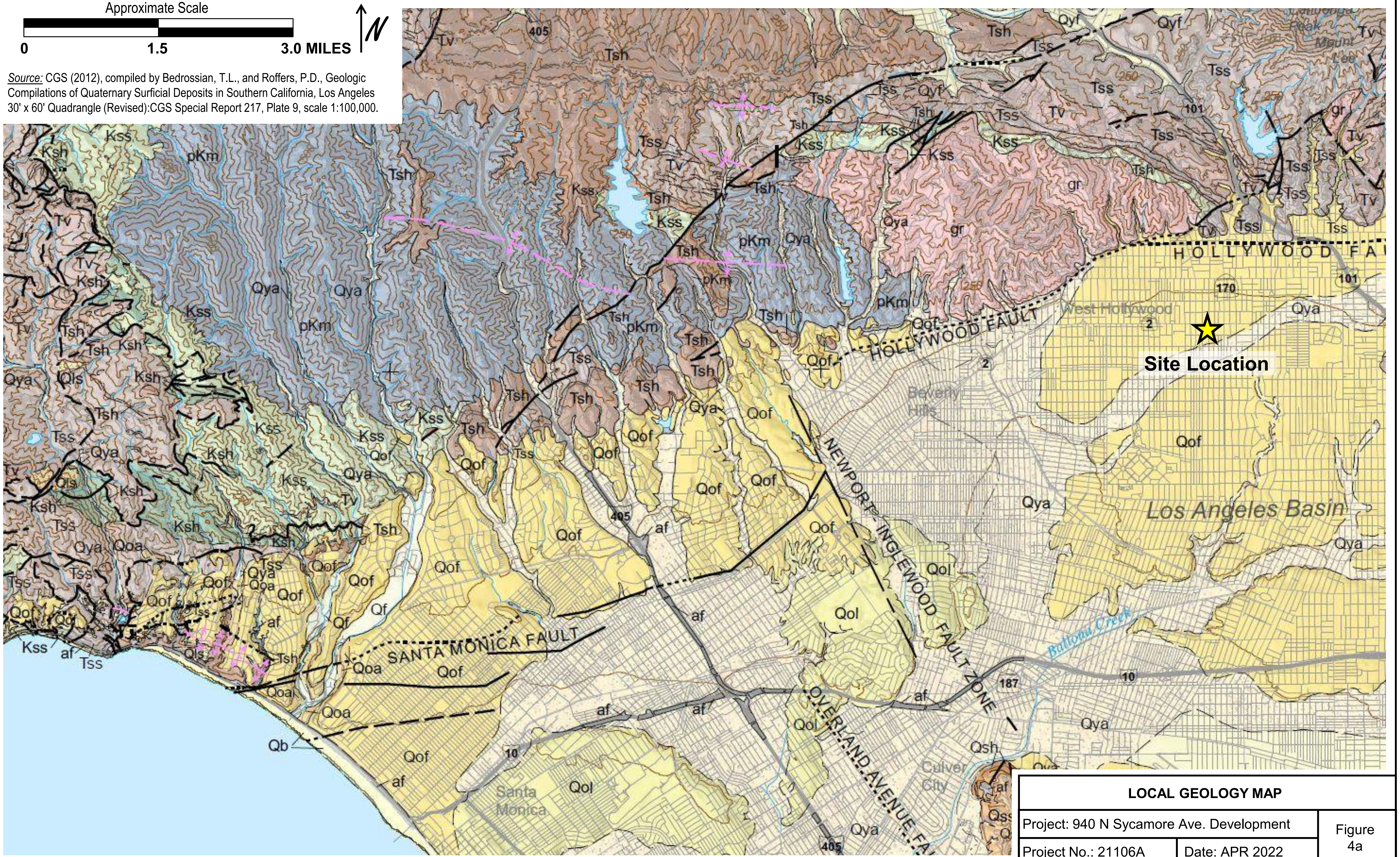


Source: SCB Architectural Package (dated 04/06/2022)

2D ARCHITECTURAL SECTION

Date: APR 2022	Project No.: 21106A	Project: 940 N Sycamore Ave. Development	Figure 3b

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: 940 N Sycamore Ave. Development		Figure 4a
Project No.: 21106A	Date: APR 2022	

GeoPentech

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

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.

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

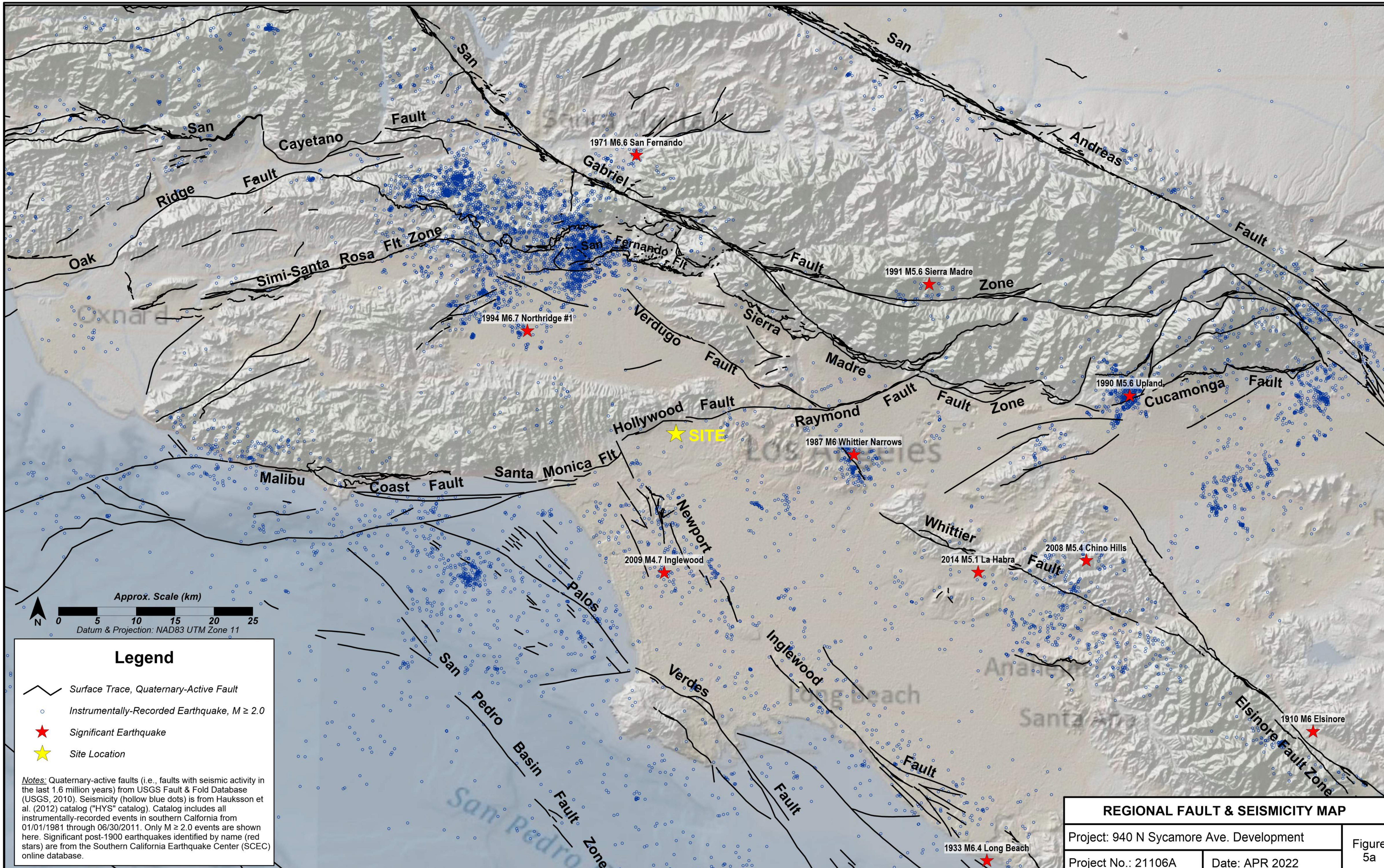
Spring

Road

County boundary

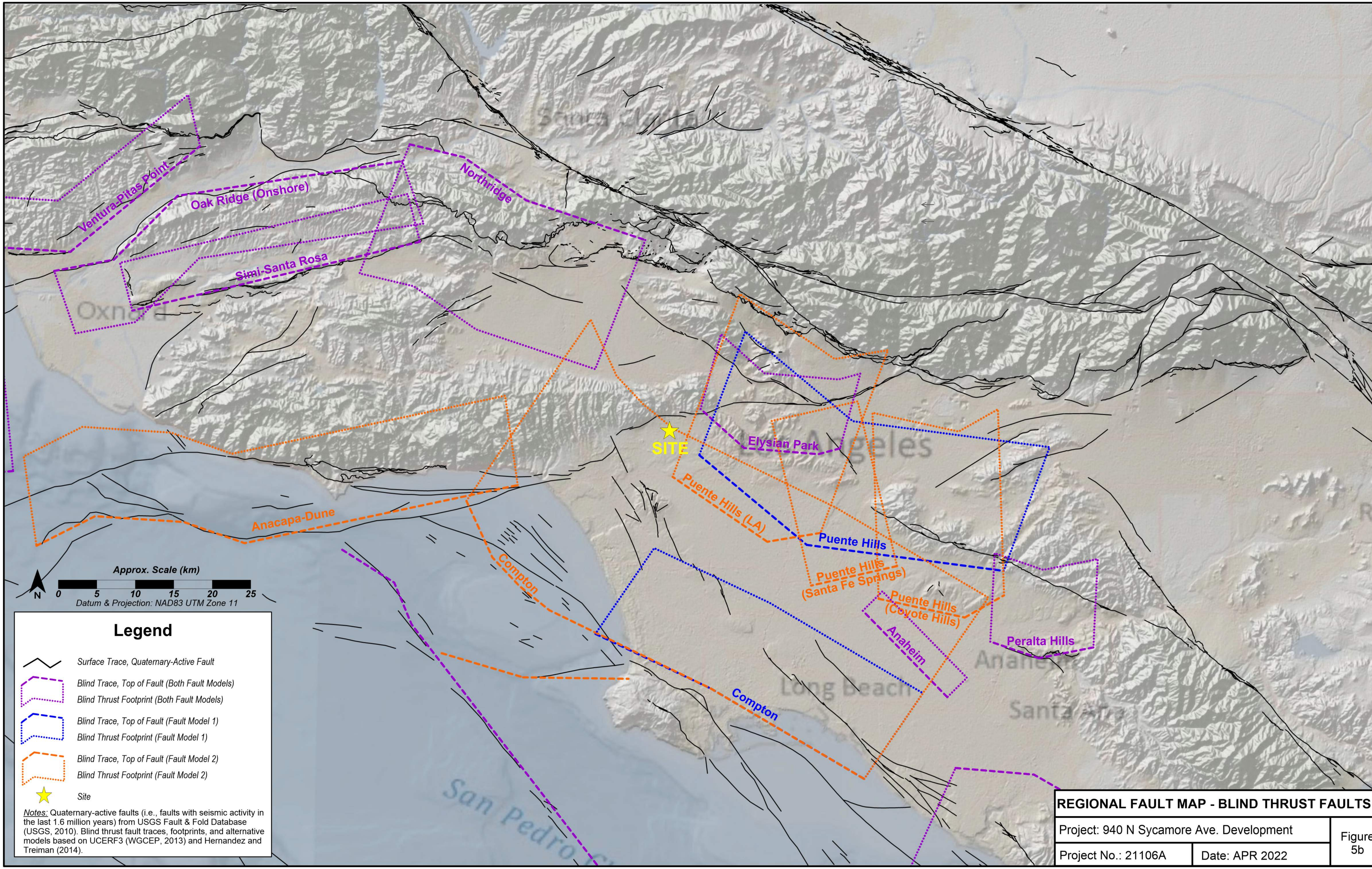
LOCAL GEOLOGY MAP LEGEND		
Project: 940 N Sycamore Ave. Development		Figure 4b
Project No.: 21106A	Date: APR 2022	

GeoPentech



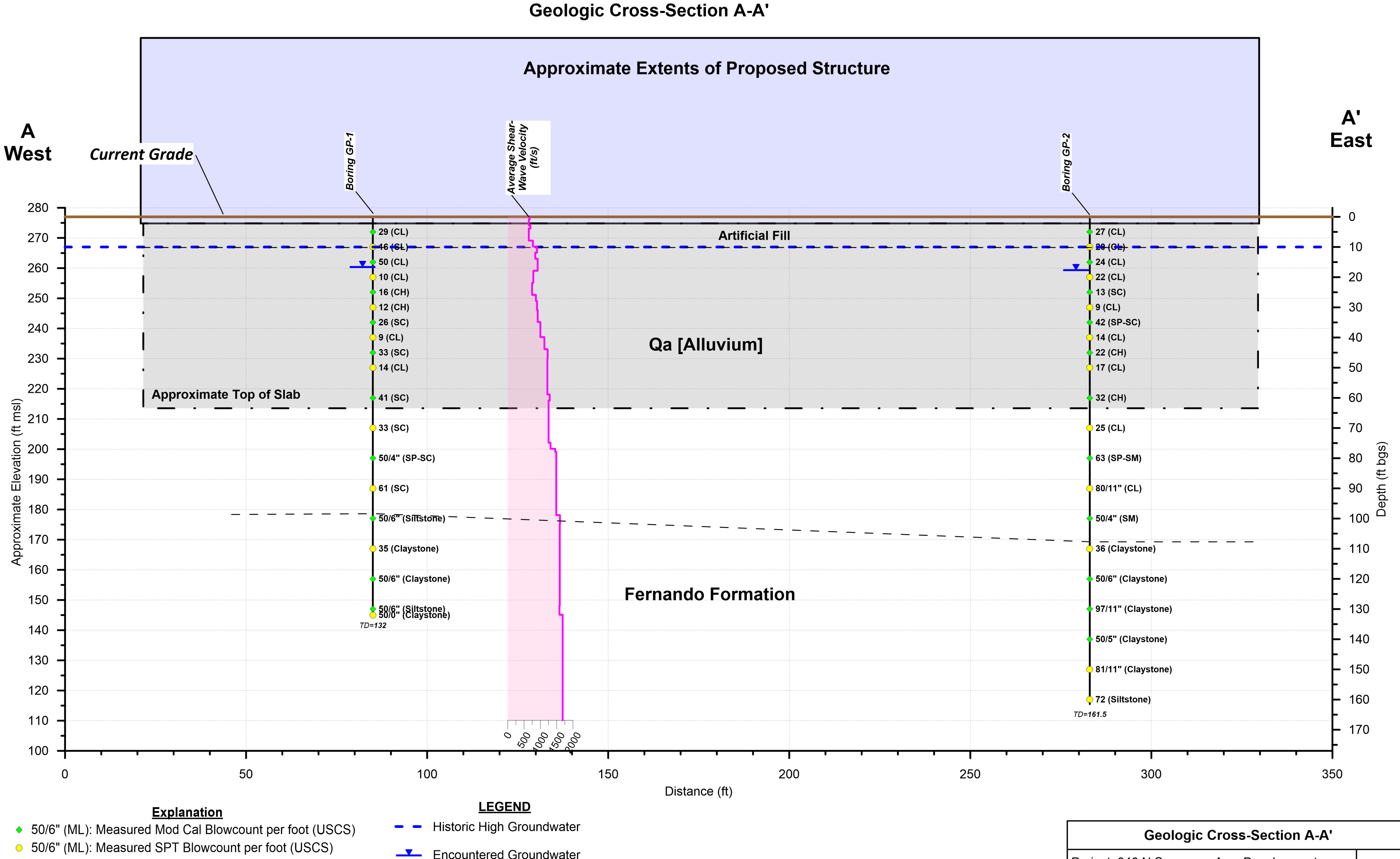
REGIONAL FAULT & SEISMICITY MAP		
Project: 940 N Sycamore Ave. Development		Figure 5a
Project No.: 21106A	Date: APR 2022	

GeoPentech



Ventura-Pitas Point
Oak Ridge (Onshore)
Northridge
Simi-Santa Rosa
Oxnard
Santa Clarita
SITE
Elysian Park
Los Angeles
Puente Hills (LA)
Puente Hills
Puente Hills (Santa Fe Springs)
Puente Hills (Coyote Hills)
Anaheim
Peralta Hills
Anacapa-Dune
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: 940 N Sycamore Ave. Development
Project No.: 21106A
Date: APR 2022
Figure 5b

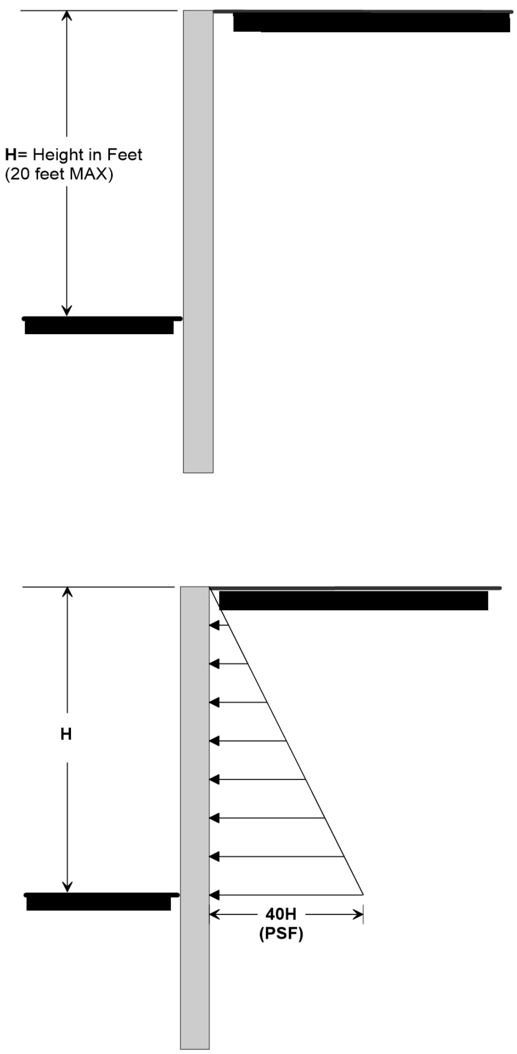
GeoPentech



Geologic Cross-Section A-A'		
Project: 940 N Sycamore Ave. Development		Figure 6
Project No.: 21106A	Date: APR 2022	

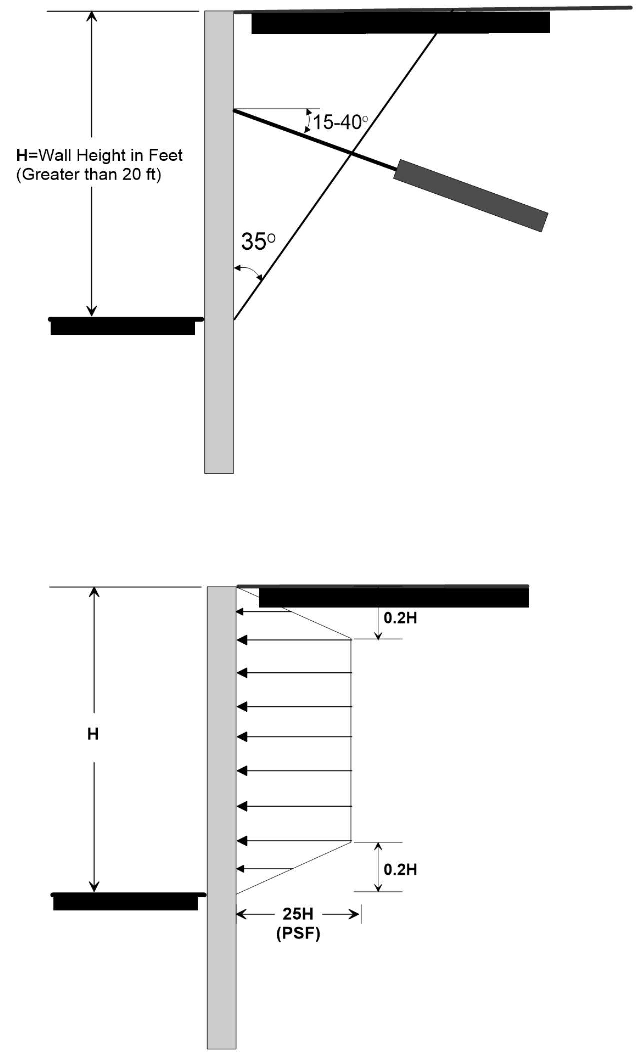
GeoPentech

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



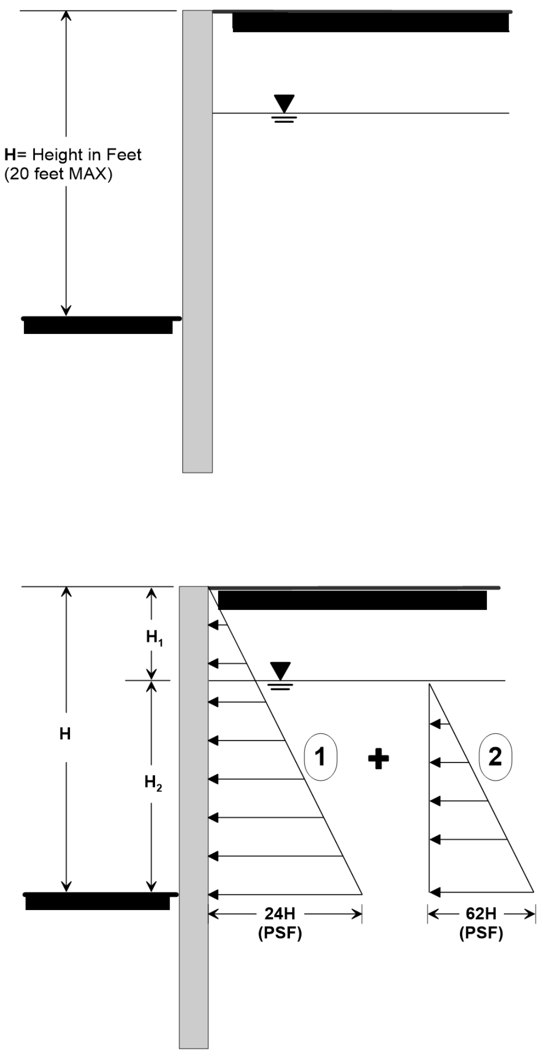
1. Maximum Lateral Earth Pressure is 25H with a trapezoidal distribution.
2. The shoring should be designed to resist applicable surcharge loads from adjacent structures, stockpiled material, or traffic loads.

TEMPORARY SHORING DETAILS - DRAINED

Project No.: 21106A	Project: 940 N Sycamore Ave. Development	Date: APR 2022	Figure 7

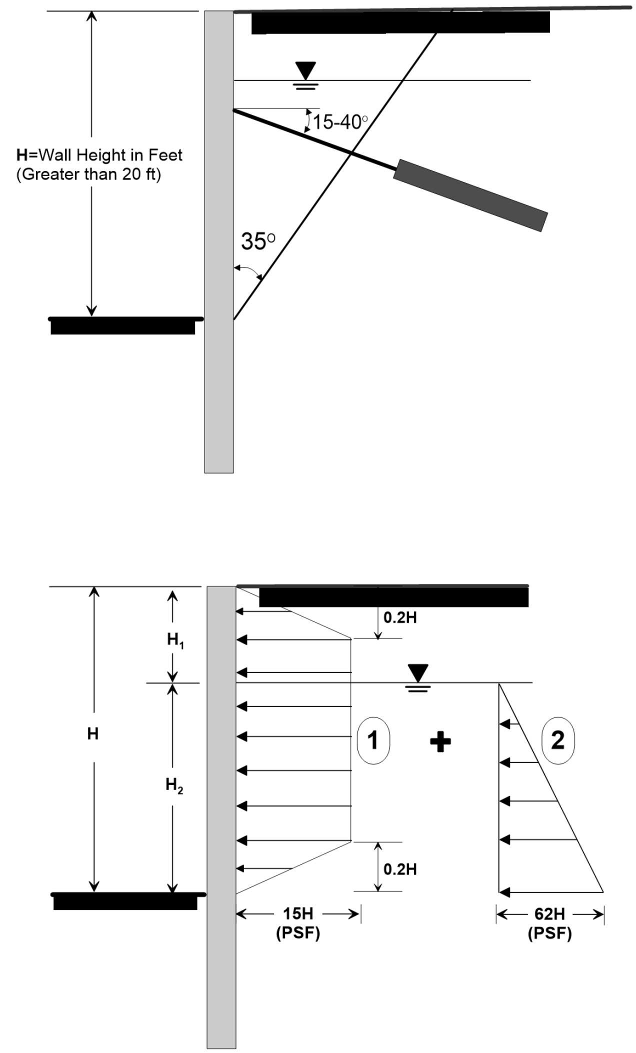
GeoPentech

Figure 8a
Cantilever Shoring - Submerged



1. Recommended values are based on Groundwater depth of 10 ft bgs (H_1=10') and a total wall height of 65 ft (H=65').
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 - Submerged



1. Recommended values are based on a Groundwater depth of 10 ft bgs (H_1=10') and a total wall height of 65 ft (H=65').
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.: 21106A	Project: 940 N Sycamore 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 two days on March 10-11, 2022 (Borings GP-1 and GP-2). The explorations consisted of advancing two borings: GP-1 to a depth of approximately 132 ft, and GP-2 to approximately 161.5 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. 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: 940 N Sycamore Ave. Development
Project Location: 940 N Sycamore Ave.
Project Number: 21106A

Key to Log of Boring

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

- **LEAN CLAY (CL)**
- **POORLY GRADED SAND W/ SILT (SP-SM)**
- **FAT CLAY (CH)**
- **POORLY GRADED SAND W/ CLAY (SP-SC)**
- **Clayey SAND (SC)**
- **Silty SAND (SM)**
- **CLAYSTONE**
- **SILTSTONE**

TYPICAL SAMPLER GRAPHIC SYMBOLS

- Bulk Sample
- Modified Calfiornia
- Standard Penetration Test

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: 940 N Sycamore Ave. Development
Project Location: 940 N Sycamore Ave.
Project Number: 21106A

Log of GP-1

Date(s) Drilled	**3/10/2022**	Logged By	**R. Wakefield**	Checked By	**R. Hadidi**
Drilling Method	**Hollow Stem Auger**	Drill Bit Size/Type	**8" HSA**	Total Depth of Borehole	**132.0 feet**
Drill Rig Type	**CME 75**	Drilling Contractor	**Martini Drilling**	Approximate Surface Elevation	**277 ft**
Groundwater Level(s)	**16' bgs**	Sampling Method	**Bulk, SPT, MC**	Hammer Data	**Automatic Trip Hammer 140-lbs/30" drop**
Borehole Location	**34.08814°, -118.34250°**	Borehole Completion	**Borehole backfilled with neat cement slurry using tremie pipe from bottom of hole to surface.**		

Elevation, feet	Depth, feet	Type	Number	Blows Per 6"	Recovery	Graphic Log	MATERIAL DESCRIPTION	Dry Unit Weight, pcf	Water Content, %	REMARKS
	0		Bulk-1				**[Asphalt]** 3" asphalt 3" base			
275							**[Artificial Fill (Af)]** **Fat CLAY (CH)**, very moist, very dark brown 10YR 2/2, medium plasiticty			
	5		1	7 10 19			**Lean CLAY (CL)**, very stiff, moist, very dark brown 10YR 2/2, medium plasticity, no HCL reaction	111.5	18.1	
270										
	10		2	4 7 9			**[Alluvium (Qal)]** **Lean CLAY (CL)**, very stiff, moist, dark yellowish brown 10YR 6/4, medium plasticity, no HCL reaction			
265										
	15		3	9 20 30			becomes hard ∇	109.2	20.2	FC=59.4% LL=43 PI=25
260										
	20		4	3 4 6			becomes stiff, saturated			Groundwater Encountered, rose to 15.8' bgs after 30 minutes. Begin adding drilling mud.
255										
	25		5	5 6 10			**Fat CLAY (CH)**, very stiff, saturated, yellowish brown, high plasticity, no HCL reaction	102.2	24.7	LL=55 PI=40
250										
	30									

GeoPentech
Geotechnical & Geoscience Consultants

Elevation, feet	Depth, feet	SAMPLES Type	Number	Blows Per 6"	Recovery	Graphic Log	MATERIAL DESCRIPTION	Dry Unit Weight, pcf	Water Content, %	REMARKS
	30		6	3 5 7			becomes stiff			LL=51 PI=36
245										
	35		7	7 12 14			**Clayey SAND (SC)**, medium dense, saturated, dark yellowish brown 10YR 4/6, fine sand, slightly plastic, no HCL reaction	112.5	18.6	FC=38.7% LL=33 PI=19 CONSOL
240										
	40		8	3 4 5			**Lean CLAY (CL)**, stiff, saturated, dark yellowish brown 10YR 4/6, fine grained sand, medium plasticity, no HCL reaction			
235										
	45		9	4 13 20			**Clayey SAND (SC)**, medium dense, saturated, dark yellowish brown 10YR 4/6, fine sand, no HCL reaction, trace rounded gravels	111.1	19.9	FC=21.4% LL=33 PI=10
230										
	50		10	4 6 8			**Lean CLAY (CL)**, stiff, saturated, dark yellowish brown 10YR 4/6, fine grained sand, medium to high plasticity, no HCL reaction			
225										
	55									
220										
	60		11	11 18 23			**Clayey SAND (SC)**, medium dense, saturated, yellowish brown 10YR 5/6, fine sand, fine to coarse sand, no HCL reaction			FC=28% LL=33 PI=17 DS
215										
	65									

Elevation, feet	Depth, feet	Type	Number	Blows Per 6"	Recovery	Graphic Log	MATERIAL DESCRIPTION	Dry Unit Weight, pcf	Water Content, %	REMARKS
	65									
210										
	70		12	10 16 17			becomes dense, interbedded with Lean CLAY zones		20.6	FC=25.8% LL=31 PI=18
205										
	75									
200										
	80		13	18 50/4"	10"		**Poorly Graded SAND with Clay (SP-SC)**, very dense, saturated, yellowish brown 10YR 5/4, fine to medium Sand, no HCL reaction	108.6	19.3	FC=5.2% DS
195										
	85									
190										
	90		14	13 27 34			**Clayey SAND (SC)**, very dense, saturated, yellowish brown 10YR 5/4, fine to medium Sand, slightly plastic, no HCL reaction		30.9	G=0% S=87% F=13%
185										
	95									
180										
	100									

Elevation, feet	Depth, feet	SAMPLES Type	Number	Blows Per 6"	Recovery	Graphic Log	MATERIAL DESCRIPTION	Dry Unit Weight, pcf	Water Content, %	REMARKS
	100		15	25 50/6"	12"		**[Fernando Formation]** **SILTSTONE**, hard, fresh, dark olive gray 5YR 3/2, saturated, slight plasticity, slight HCL reaction, trace fossils, breaks along rough laminations horizontal to MC rings	104.2	22.5	LL=38 PI=10 CONSOL
175										
	105									
170										
	110		6	6 12 23			**CLAYSTONE**, hard, fresh, dark olive gray 5YR 3/2, saturated, medium plasticity, slight HCL reaction, trace fossils, breaks along rough laminations horizontal to MC rings			LL=50 PI=25
165										
	115									
160										
	120		17	22 50/6"	12"		same as above			
155										
	125									
150										
	130		18	30 50/6"	12"		**SILTSTONE**, hard, fresh, dark olive gray 5YR 3/2, saturated, slight plasticity, slight HCL reaction, trace fossils, breaks along rough laminations horizontal to MC rings	98.5	26.6	LL=37 PI=9
145			19	50/0"	NR		**Total depth: 132' bgs** Borehole backfilled with neat cement slurry using tremie pipe from bottom of hole to surface.			
	135									

Project: 940 N Sycamore Ave. Development
Project Location: 940 N Sycamore Ave.
Project Number: 21106A

Log of GP-1

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
	135						Groundwater encountered at 20' bgs, rose to 15.8' bgs after 30 minutes			
140										
	140									
135										
	145									
130										
	150									
125										
	155									
120										
	160									
115										
	165									
110										
	170									

Project: 940 N Sycamore Ave. Development Project Location: 940 N Sycamore Ave. Project Number: 21106A	# Log of GP-2 Sheet 1 of 5

Date(s) Drilled	**3/11/2022**	Logged By	**R. Wakefield**	Checked By	**R. Hadidi**
Drilling Method	**Hollow Stem Auger**	Drill Bit Size/Type	**8" HSA**	Total Depth of Borehole	**161.5 feet**
Drill Rig Type	**CME 75**	Drilling Contractor	**Martini Drilling**	Approximate Surface Elevation	**278 ft**
Groundwater Level(s)	**18' bgs**	Sampling Method	**Bulk, SPT, MC**	Hammer Data	**Automatic Trip Hammer 140-lbs/30" drop**
Borehole Location	**34.08810°, -118.34189°**	Borehole Completion	**Borehole backfilled with neat cement slurry using tremie pipe 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]** 3" asphalt 6" base			Bulk-1: CORR
							[Artificial Fill (Af)] **Lean CLAY (CL)**, very moist, very dark brown 10YR 2/2, medium plasiticty			
275										
	5		1	4 10 17			**Lean CLAY (CL)**, very stiff, moist, very dark brown 10YR 2/2, medium plasticity, no HCL reaction			
270										
	10		2	4 7 13			**[Alluvium (Qal)]** **Lean CLAY (CL)**, very stiff, moist, dark yellowish brown 10YR 6/4, medium plasticity, slight HCl reaction		14.6	
265										
	15		3	4 10 13			no HCl reaction	104.5	22.4	LL=49 PI=29
260							∇			
	20		4	6 10 12			slight HCl reaction			FC=63.1%
255										
	25		5	3 6 7			**Clayey SAND (SC)**, medium dense, saturated, dark yellowish brown 10YR 4/6, fine sand, slightly plastic, no HCL reaction	95.4	29.5	Groundwater Encountered, rose to 17.8' bgs after 30 minutes. Begin adding drilling mud. FC=41.1% LL=38 PI=14
250										
	30									

Report: GP SOIL BA LOG; File: 21106A 948 SYCAMORE.GPJ; 5/9/2023

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
	30		6	2 4 5			**Sandy CLAY (CL)**, stiff, moist, dark yellowish brown 10YR 6/4, medium plasticity, slight HCl reaction			
245										
	35		7	5 22 20			**Poorly Graded SAND with Clay (SP-SC)**, medium dense, saturated, dark yellowish brown 10YR 4/6, fine sand, no HCL reaction, trace fine gravels		19.6	FC=10.4% DS
240										
	40		8	3 6 8			**Sandy CLAY (CL)**, stiff, saturated, dark yellowish brown 10YR 4/6, medium plasticity, no HCL reaction			
235										
	45		9	6 12 20			**Fat CLAY (CH)**, very stiff, saturated, dark yellowish brown 10YR 4/6, high plasticity, no HCL reaction	100.8	26.5	LL=59 PI=38 CONSOL
230										
	50		10	4 8 9			**Sandy CLAY (CL)**, very stiff, saturated, dark yellowish brown 10YR 4/6, medium plasticity, no HCL reaction			FC=58.7%
225										
	55									
220										
	60		11	10 17 25			**Fat CLAY (CH)**, very stiff, saturated, dark yellowish brown 10YR 4/6, high plasticity, no HCL reaction	110.5	19.4	FC=60.6% LL=52 PI=35 CONSOL
215										
	65									

Project: 940 N Sycamore Ave. Development
Project Location: 940 N Sycamore Ave.
Project Number: 21106A

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
	65									
210										
	70		12	5 9 16			**Sandy CLAY (CL)**, very stiff, saturated, dark yellowish brown 10YR 4/6, medium plasticity, no HCL reaction			
205										
	75									
200										
	80		13	13 30 33			**Poorly Graded SAND with Silt (SP-SM)**, dense, saturated, yellowish brown 10YR 5/4, fine Sand, no HCL reaction	102.5	23.7	FC=11.2% DS
195										
	85									
190										
	90		14	27 30 50/5"			**Sandy CLAY (CL)**, hard, saturated, yellowish brown 10YR 5/4, medium plasticity, no HCL reaction			G=1% S=28% F=71%
185										
	95									
180										
	100									

Report: GP SOIL BA LOG; File: 21106A 948 SYCAMORE.GPJ; 5/9/2023

GeoPentech
Geotechnical & Geoscience Consultants

Elevation, feet	Depth, feet	Type	Number	Blows Per 6"	Recovery	Graphic Log	MATERIAL DESCRIPTION	Dry Unit Weight, pcf	Water Content, %	REMARKS
	100		15	12 50/4"			**Silty SAND (SM)**, very dense, saturated, pale brown 10YR 6/3, fine Sand, no HCL reaction		22.1	FC=31.1%
175										
	105									
170										
	110		16	32 16 20			[Fernando Formation] **CLAYSTONE**, hard, dark olive gray 5YR 3/2, saturated, medium plasticity, no HCL reaction			
165										
	115									
160										
	120		17	27 50/6"			has a slight HCL reaction	102.2	24.2	FC=70.4% LL=47 PI=25 CONSOL
155										
	125									
150										
	130		18	21 47 50/5"			becomes more silty			
145										
	135									

Project: 940 N Sycamore Ave. Development
Project Location: 940 N Sycamore Ave.
Project Number: 21106A

Log of GP-2

Elevation, feet	Depth, feet	Type	Number	Blows Per 6"	Recovery	Graphic Log	MATERIAL DESCRIPTION	Dry Unit Weight, pcf	Water Content, %	REMARKS
	135									
140										
	140		19	29 50/5"			same as above	106.1	21.4	FC=84.6% LL=39 PI=15 DS CONSOL
135										
	145									
130										
	150		20	16 31 50/5"			same as above			
125										
	155									
120										
	160		21	17 26 44			**SILTSTONE**, hard, fresh, dark olive gray 5YR 3/2, saturated, slight plasticity, slight HCL reaction		26.9	LL=34 PI=8
115							**Total depth: 161.5' bgs** Borehole backfilled with neat cement slurry using tremie pipe from bottom of hole to surface. Groundwater encountered at 25' bgs, rose to 17.8' bgs after 30 minutes			
	165									
110										
	170									

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	1	5	CL	18.1	111.5												
GP-1	2	10	CL														
GP-1	3	15	CL	20.2	109.2	43	25			59.4							
GP-1	4	20	CL														
GP-1	5	25	CH	24.7	102.2	55	40										
GP-1	6	30	CH			51	36										
GP-1	7	35	SC	18.6	112.5	33	19			38.7							CONSOL
GP-1	8	40	CL														
GP-1	9	45	SC	19.9	111.1	33	10			21.4							
GP-1	10	50	CL														
GP-1	11	60	SC			33	17			28					32	950	
GP-1	12	70	SC	20.6		31	18			25.8							
GP-1	13	80	SP-SC	19.3	108.6					5.2					35	450	
GP-1	14	90	SC	30.9				0	87	13							
GP-1	15	100	Siltstone	22.5	104.2	38	10										CONSOL
GP-1	16	110	Claystone			50	25										
GP-1	17	120	Claystone														
GP-1	18	130	Siltstone	26.6	98.5	37	9										
GP-1	19	132	Claystone														
GP-2	B-1	0-5	CL								1009	225	48	8.2			
GP-2	1	5	CL														
GP-2	2	10	CL	14.6													
GP-2	3	15	CL	22.4	104.5	49	29										
GP-2	4	20	CL							63.1							
GP-2	5	25	SC	29.5	95.4	38	14			41.1							
GP-2	6	30	CL														
GP-2	7	35	SP-SC	19.6						10.4					34	300	
GP-2	8	40	CL														
GP-2	9	45	CH	26.5	100.8	59	38										CONSOL
GP-2	10	50	CL							58.7							
GP-2	11	60	CH	19.4	110.5	52	35			60.6							CONSOL
GP-2	12	70	CL														

*See Lab results in Appendix B

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	13	80	SP-SM	23.7	102.5					11.2					31	350	
GP-2	14	90	CL					1	28	71							
GP-2	15	100	SM	22.1						31.1							
GP-2	16	110	Claystone														
GP-2	17	120	Claystone	24.2	102.2	47	25			70.4							CONSOL
GP-2	18	130	Claystone														
GP-2	19	140	Claystone	21.4	106.1	39	15			84.6					*	*	CONSOL
GP-2	20	150	Claystone														
GP-2	21	160	Siltstone	26.9		34	8										

*See Lab results in Appendix B

AP Engineering and Testing, Inc.
DBE|MBE|SBE
2607 Pomona Boulevard | Pomona, CA 91768
t. 909.869.6316 | f. 909.869.6318 | www.aplaboratory.com

MOISTURE AND DENSITY TEST RESULTS

ASTM D2216 and ASTM D7263 (Method B)

Client: GeoPentech
Project Name: 940 Sycamore Development
Project No.: 21106A
AP Lab No.: 22-0346
Test Date: 03/21/22

Boring No.	Sample No.	Sample Depth (ft.)	Moisture Content (%)	Dry Density (pcf)
GP-1	1	5	18.1	111.5
GP-1	3	15	20.2	109.2
GP-1	5	25	24.7	102.2
GP-1	7	35	18.6	112.5
GP-1	9	45	19.9	111.1
GP-1	12	70	20.6	NA
GP-1	13	80	19.3	108.6
GP-1	14	90	30.9	NA
GP-1	15	100	22.5	104.2
GP-1	18	130	26.6	98.5

AP Engineering and Testing, Inc.
DBE|MBE|SBE
2607 Pomona Boulevard | Pomona, CA 91768
t. 909.869.6316 | f. 909.869.6318 | www.aplaboratory.com

MOISTURE AND DENSITY TEST RESULTS

ASTM D2216 and ASTM D7263 (Method B)

Client: GeoPentech
Project Name: 940 Sycamore Development
Project No.: 21106A
AP Lab No.: 22-0346
Test Date: 03/21/22

Boring No.	Sample No.	Sample Depth (ft.)	Moisture Content (%)	Dry Density (pcf)
GP-2	2	10	14.6	NA
GP-2	3	15	22.4	104.5
GP-2	5	25	29.5	95.4
GP-2	7	35	19.6	NA
GP-2	9	45	26.5	100.8
GP-2	11	60	19.4	110.5
GP-2	13	80	23.7	102.5
GP-2	15	100	22.1	DISTURBED
GP-2	17	120	24.2	102.2
GP-2	19	140	21.4	106.1
GP-2	21	160	26.9	NA

AP Engineering and Testing, Inc.
DBE|MBE|SBE
2607 Pomona Boulevard | Pomona, CA 91768
t. 909.869.6316 | f. 909.869.6318 | www.aplaboratory.com

PERCENT PASSING NO. 200 SIEVE

ASTM D1140

Client: GeoPentech
Project Name: 940 Sycamore Development
Project Number: 21106A
AP Lab No.: 22-0346
Test Date: 03/22/22

Boring No.	Sample No.	Depth (ft)	Percent Fines (%)
GP-1	3	15	59.4
GP-1	7	35	38.7
GP-1	9	45	21.4
GP-1	11	60	28.0
GP-1	12	70	25.8
GP-1	13	80	5.2

PERCENT PASSING NO. 200 SIEVE

ASTM D1140

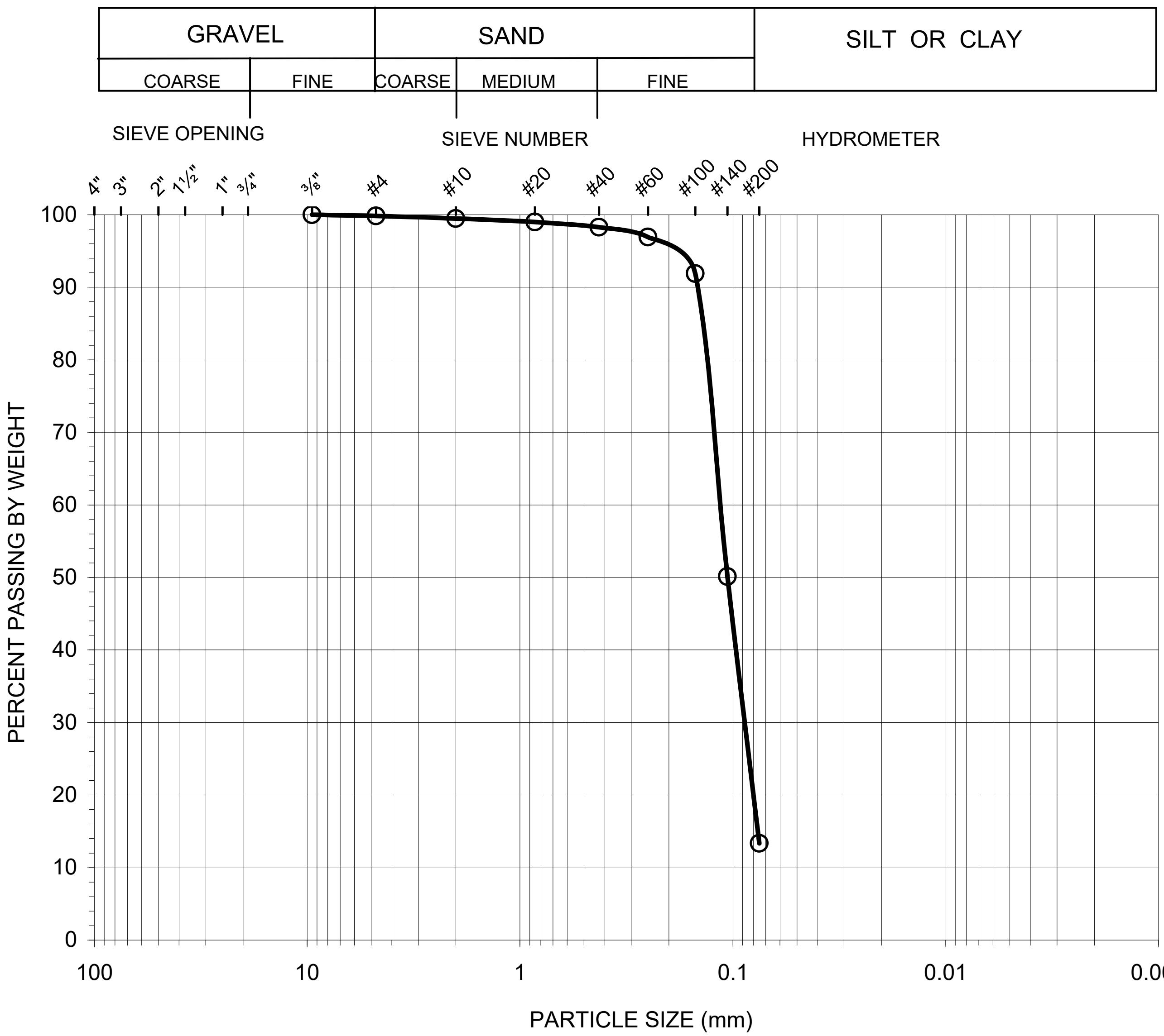
Client: GeoPentech
AP Lab No.: 22-0346
Project Name: 940 Sycamore Development
Test Date: 03/22/22
Project Number: 21106A

Boring No.	Sample No.	Depth (ft)	Percent Fines (%)
GP-2	4	20	63.1
GP-2	5	25	41.1
GP-2	7	35	10.4
GP-2	10	50	58.7
GP-2	11	60	60.6
GP-2	13	80	11.2
GP-2	15	100	31.1
GP-2	17	120	70.4
GP-2	19	140	84.6

AP Engineering and Testing, Inc.
DBE|MBE|SBE
2607 Pomona Boulevard | Pomona, CA 91768
t. 909.869.6316 | f. 909.869.6318 | www.aplaboratory.com

GRAIN SIZE DISTRIBUTION CURVE
ASTM D 6913

Client Name:	GeoPentech	Tested by:	JT	Date:	03/23/22
Project Name:	940 Sycamore Development	Computed by:	NR	Date:	03/24/22
Project No.:	21106A	Checked by:	AP	Date:	03/29/22

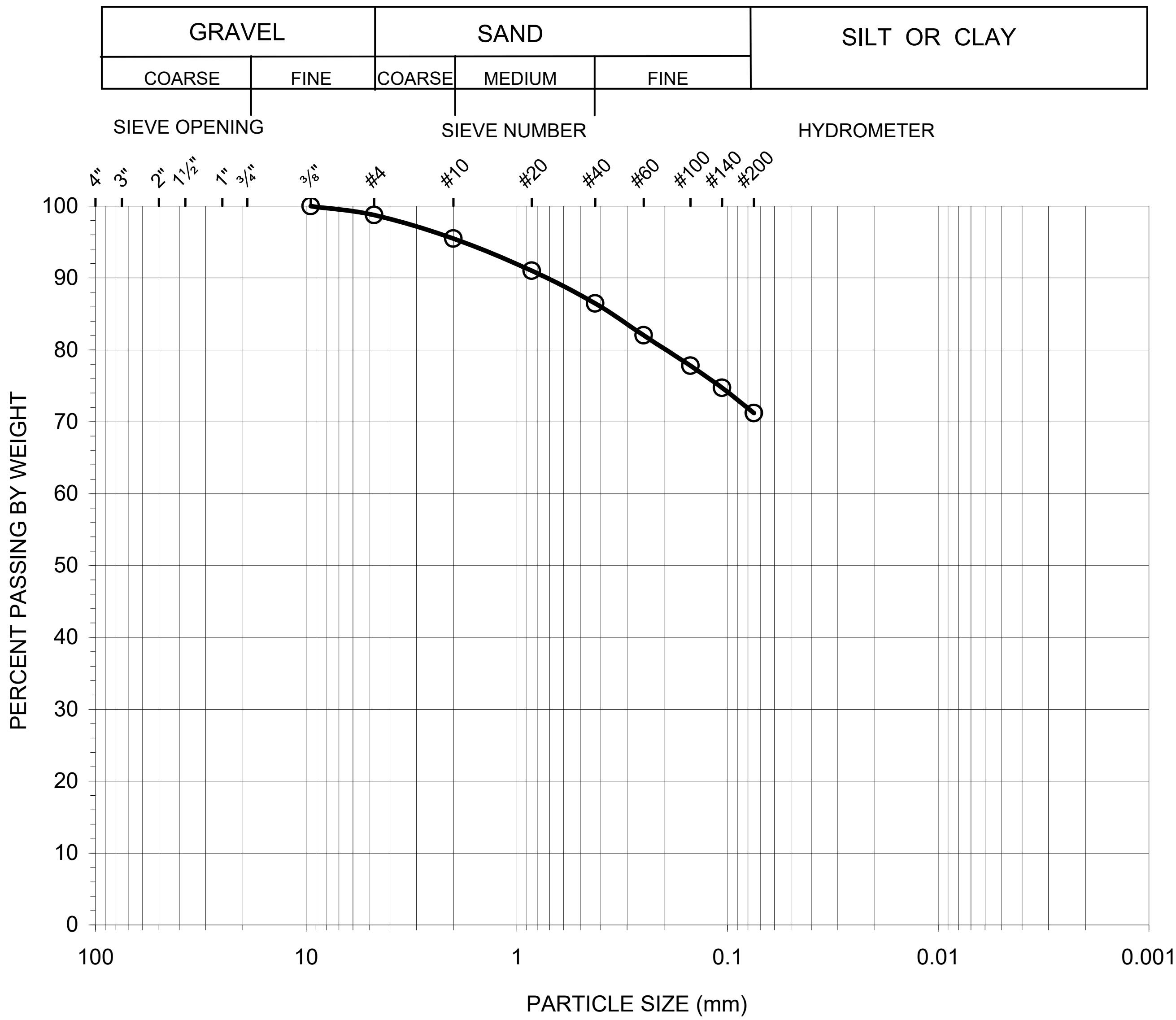


Symbol	Boring No.	Sample No.	Sample Depth (feet)	Percent			Atterberg Limits LL:PL:PI	Soil Type U.S.C.S
				Gravel	Sand	Silt & Clay		
○	GP-1	14	90	0	87	13	N/A	SM

AP Engineering and Testing, Inc.
DBE|MBE|SBE
2607 Pomona Boulevard | Pomona, CA 91768
t. 909.869.6316 | f. 909.869.6318 | www.aplaboratory.com

GRAIN SIZE DISTRIBUTION CURVE
ASTM D 6913

Client Name:	GeoPentech	Tested by:	JT	Date:	03/23/22
Project Name:	940 Sycamore Development	Computed by:	NR	Date:	03/24/22
Project No.:	21106A	Checked by:	AP	Date:	03/29/22



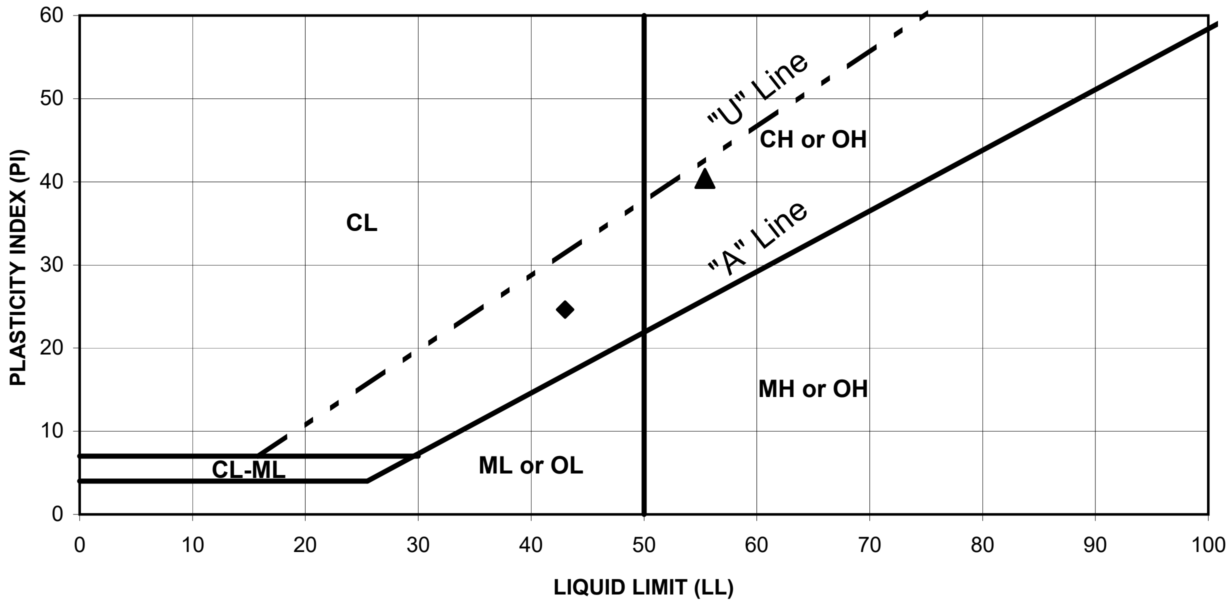
Symbol	Boring No.	Sample No.	Sample Depth (feet)	Percent			Atterberg Limits LL:PL:PI	Soil Type U.S.C.S
				Gravel	Sand	Silt & Clay		
○	GP-2	14	90	1	28	71	N/A	CL*

**Note: Based on visual classification of sample*

AP Engineering and Testing, Inc.
DBE|MBE|SBE
2607 Pomona Boulevard | Pomona, CA 91768
t. 909.869.6316 | f. 909.869.6318 | www.aplaboratory.com

ATTERBERG LIMITS
ASTM D 4318

Client Name:	GeoPentech	Tested By:	DK	Date:	03/20/22
Project Name:	940 Sycamore Development	Computed By:	NR	Date:	03/23/22
Project No.:	21106A	Checked By:	AP	Date:	03/29/22



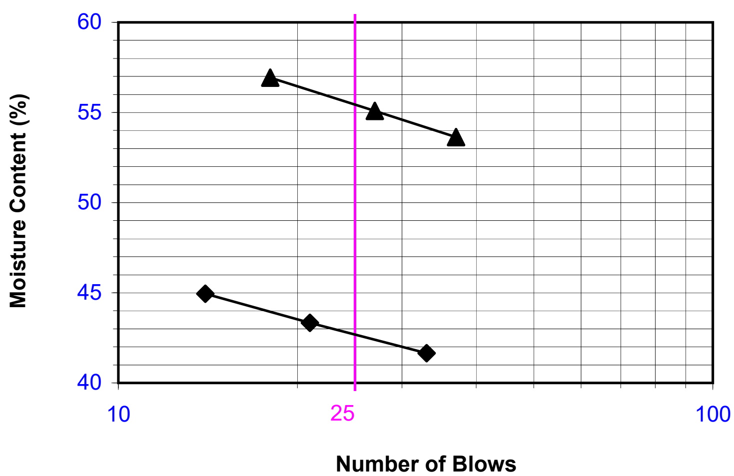
PROCEDURE USED

☐ Wet Preparation

☒ Dry Preparation

☒ Procedure A
Multipoint Test

☐ Procedure B
One-point Test

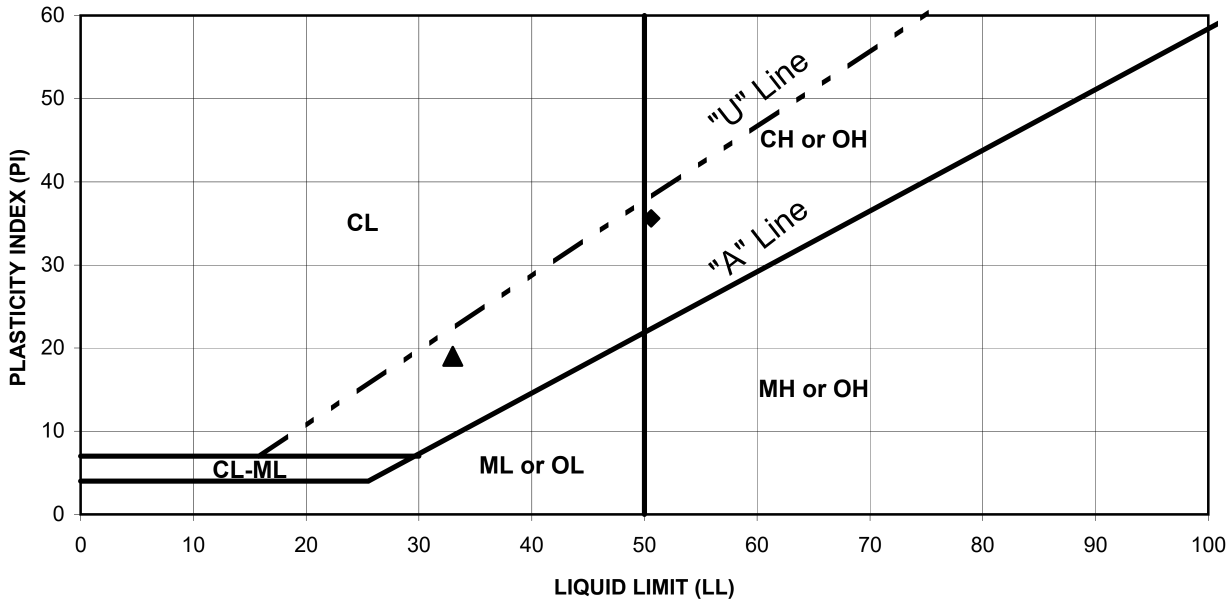


Symbol	Boring Number	Sample Number	Depth (feet)	LL	PL	PI	Plasticity Chart Symbol
♦	GP-1	3	15	43	18	25	CL
▲	GP-1	5	25	55	15	40	CH

AP Engineering and Testing, Inc.
DBE|MBE|SBE
2607 Pomona Boulevard | Pomona, CA 91768
t. 909.869.6316 | f. 909.869.6318 | www.aplaboratory.com

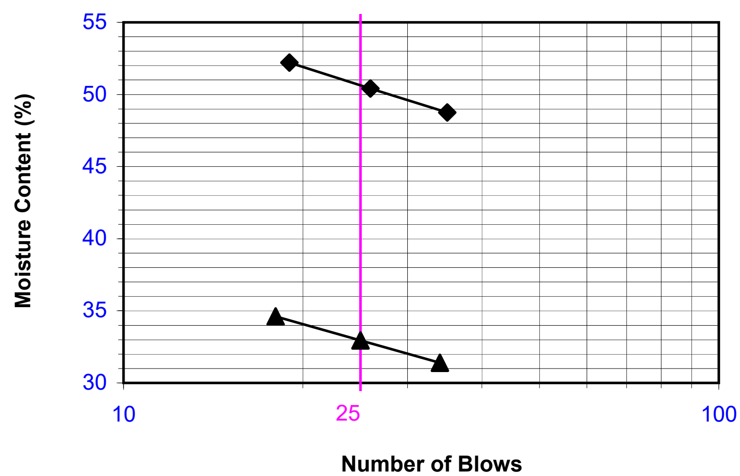
ATTERBERG LIMITS
ASTM D 4318

Client Name: GeoPentech — Tested By: DK — Date: 03/20/22
Project Name: 940 Sycamore Development — Computed By: NR — Date: 03/23/22
Project No.: 21106A — Checked By: AP — Date: 03/29/22



PROCEDURE USED

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

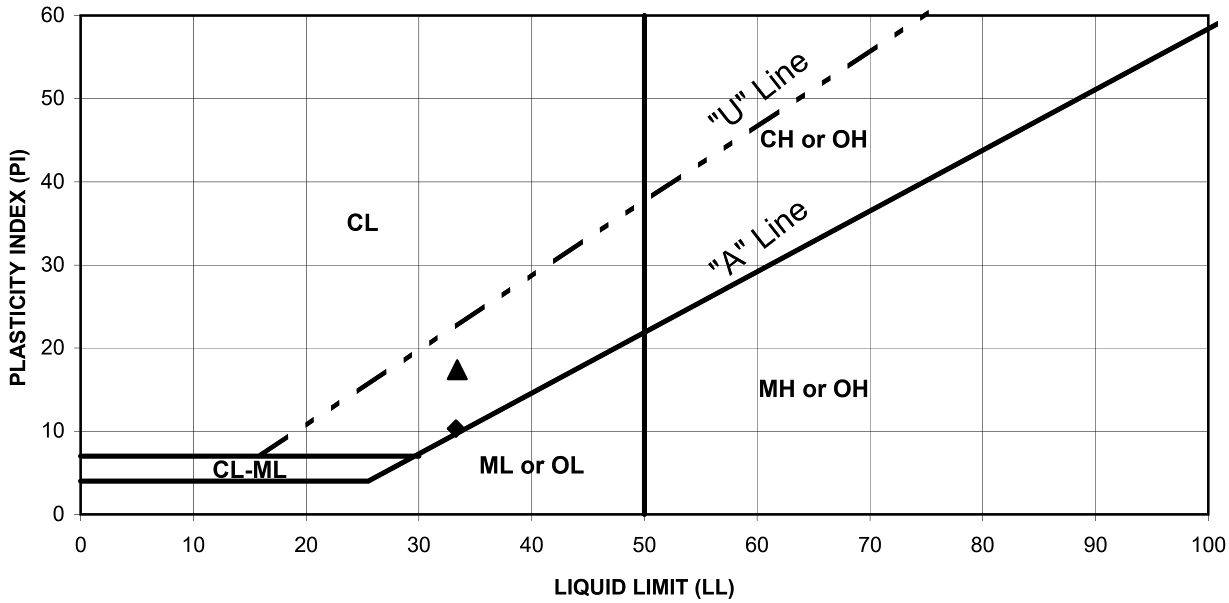


Symbol	Boring Number	Sample Number	Depth (feet)	LL	PL	PI	Plasticity Chart Symbol
♦	GP-1	6	30	51	15	36	CH
▲	GP-1	7	35	33	14	19	CL

AP Engineering and Testing, Inc.
DBE|MBE|SBE
2607 Pomona Boulevard | Pomona, CA 91768
t. 909.869.6316 | f. 909.869.6318 | www.aplaboratory.com

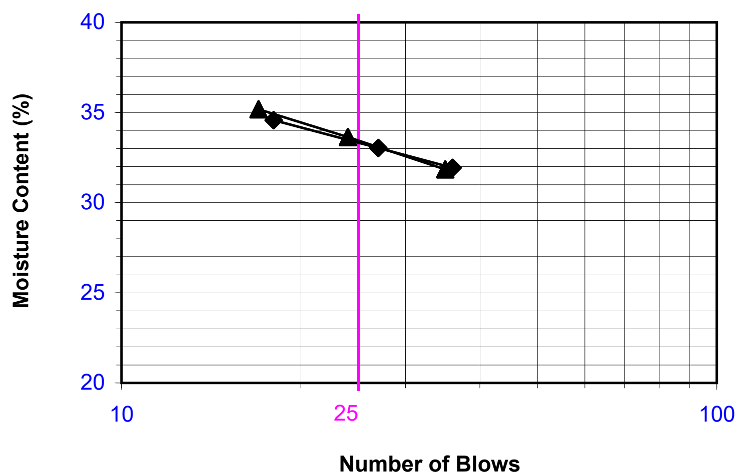
ATTERBERG LIMITS
ASTM D 4318

Client Name:	GeoPentech	Tested By:	DK	Date:	03/20/22
Project Name:	940 Sycamore Development	Computed By:	NR	Date:	03/23/22
Project No.:	21106A	Checked By:	AP	Date:	03/29/22



PROCEDURE USED

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

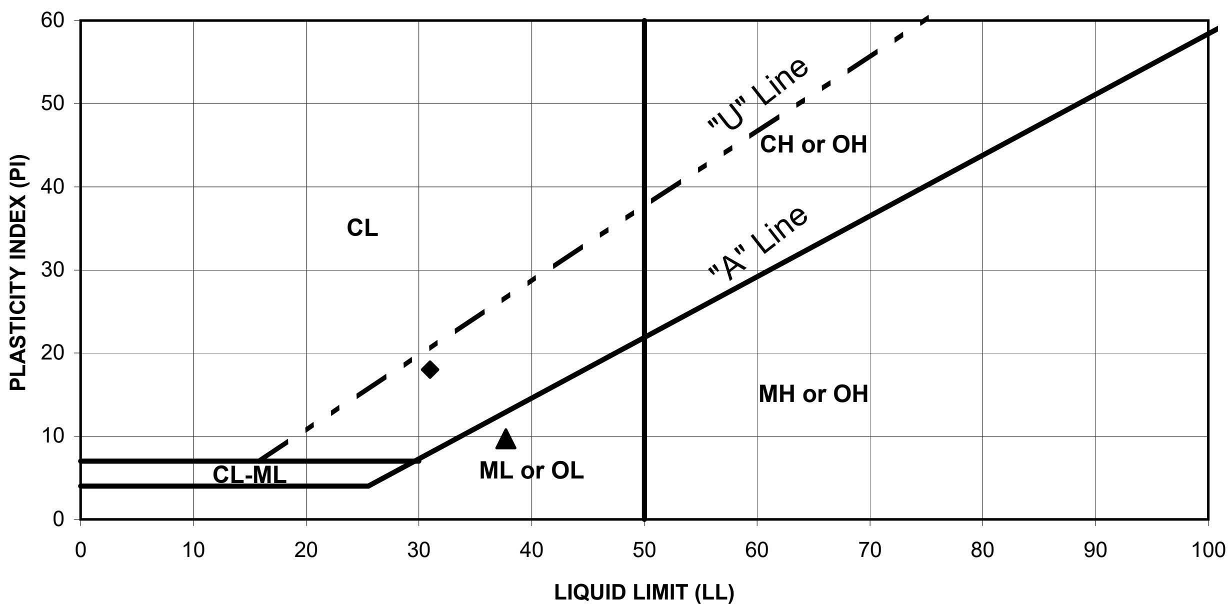


Symbol	Boring Number	Sample Number	Depth (feet)	LL	PL	PI	Plasticity Chart Symbol
♦	GP-1	9	45	33	23	10	CL
▲	GP-1	11	60	33	16	17	CL

AP Engineering and Testing, Inc.
DBE|MBE|SBE
2607 Pomona Boulevard | Pomona, CA 91768
t. 909.869.6316 | f. 909.869.6318 | www.aplaboratory.com

ATTERBERG LIMITS
ASTM D 4318

Client Name:	GeoPentech	Tested By:	DK	Date:	03/20/22
Project Name:	940 Sycamore Development	Computed By:	NR	Date:	03/23/22
Project No.:	21106A	Checked By:	AP	Date:	03/29/22



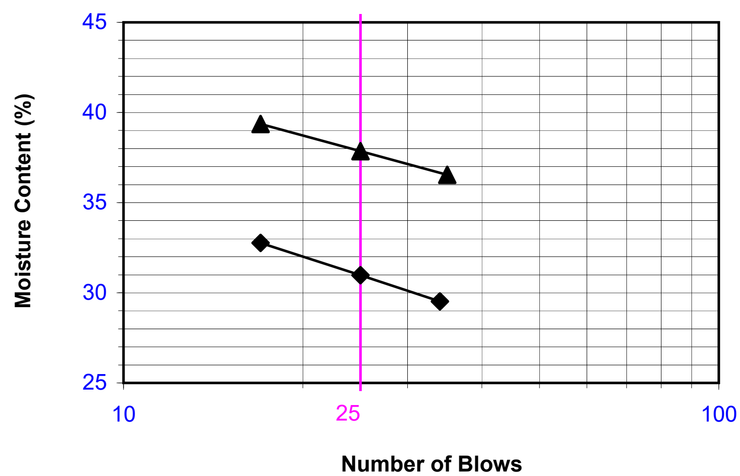
PROCEDURE USED

☐ Wet Preparation

☒ Dry Preparation

☒ Procedure A
Multipoint Test

☐ Procedure B
One-point Test

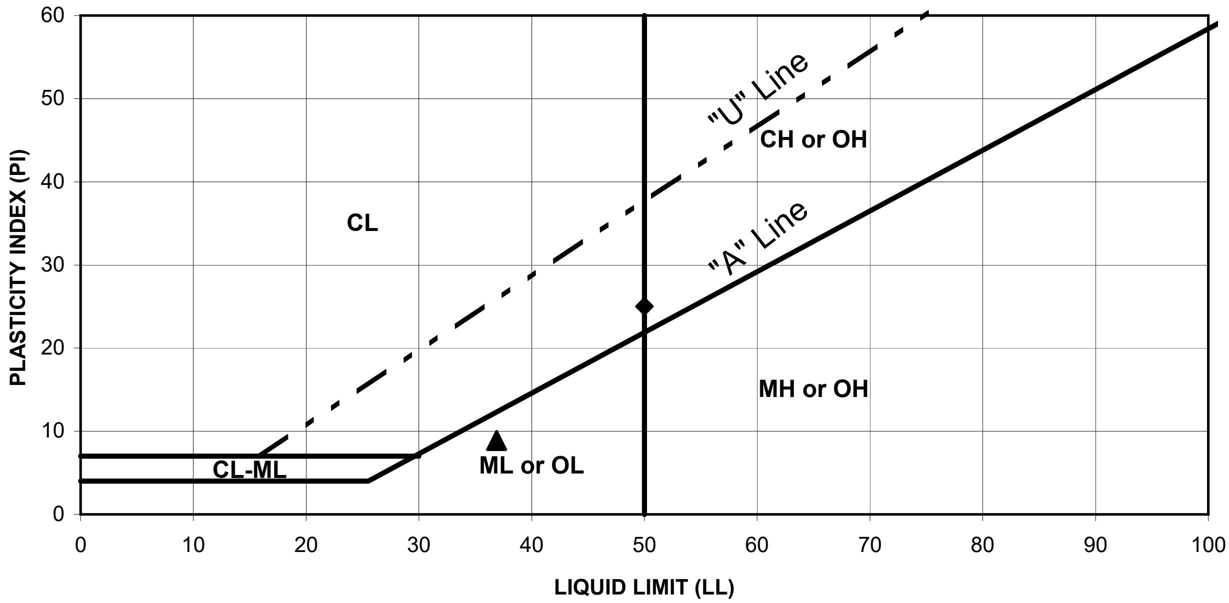


Symbol	Boring Number	Sample Number	Depth (feet)	LL	PL	PI	Plasticity Chart Symbol
♦	GP-1	12	70	31	13	18	CL
▲	GP-1	15	100	38	28	10	ML

AP Engineering and Testing, Inc.
DBE|MBE|SBE
2607 Pomona Boulevard | Pomona, CA 91768
t. 909.869.6316 | f. 909.869.6318 | www.aplaboratory.com

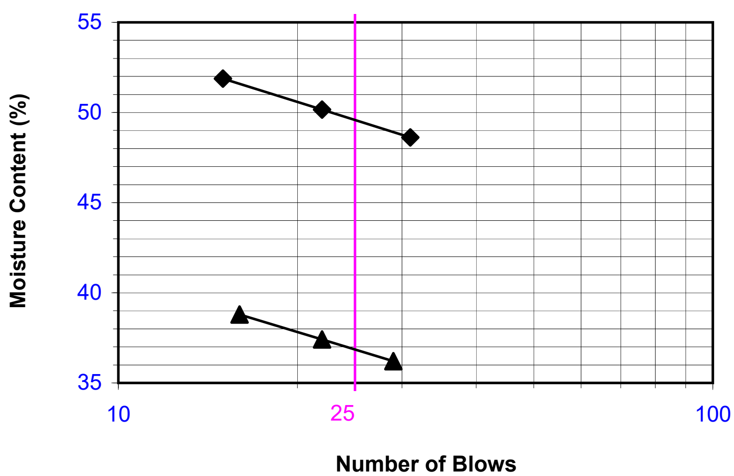
ATTERBERG LIMITS
ASTM D 4318

Client Name: GeoPentech — Tested By: DK — Date: 03/20/22
Project Name: 940 Sycamore Development — Computed By: NR — Date: 03/23/22
Project No.: 21106A — Checked By: AP — Date: 03/29/22



PROCEDURE USED

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

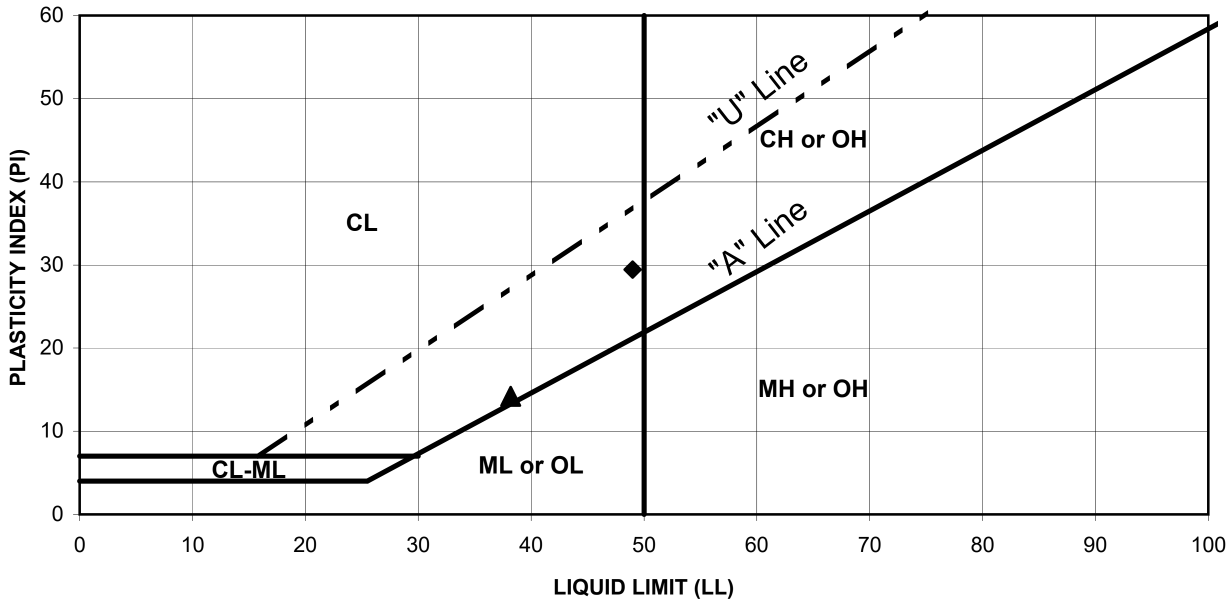


Symbol	Boring Number	Sample Number	Depth (feet)	LL	PL	PI	Plasticity Chart Symbol
♦	GP-1	16	110	50	25	25	CH
▲	GP-1	18	130	37	28	9	ML

AP Engineering and Testing, Inc.
DBE|MBE|SBE
2607 Pomona Boulevard | Pomona, CA 91768
t. 909.869.6316 | f. 909.869.6318 | www.aplaboratory.com

ATTERBERG LIMITS
ASTM D 4318

Client Name:	GeoPentech	Tested By:	LS	Date:	03/22/22
Project Name:	940 Sycamore Development	Computed By:	NR	Date:	03/24/22
Project No.:	21106A	Checked By:	AP	Date:	03/29/22



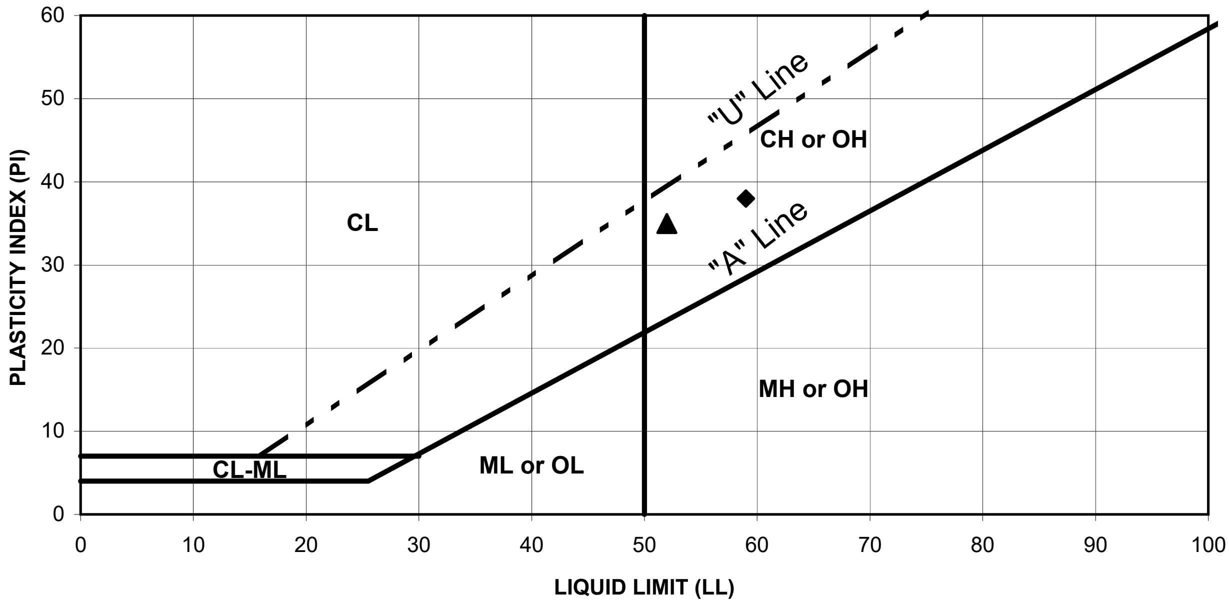
PROCEDURE USED

- [ ] 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	3	15	49	20	29	CL
▲	GP-2	5	25	38	24	14	CL

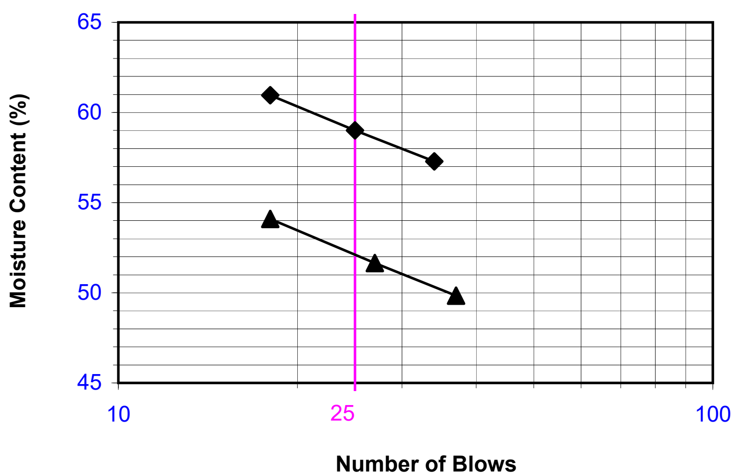
ATTERBERG LIMITS
ASTM D 4318

Client Name: GeoPentech Tested By: LS Date: 03/22/22
Project Name: 940 Sycamore Development Computed By: NR Date: 03/24/22
Project No.: 21106A Checked By: AP Date: 03/29/22



PROCEDURE USED

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

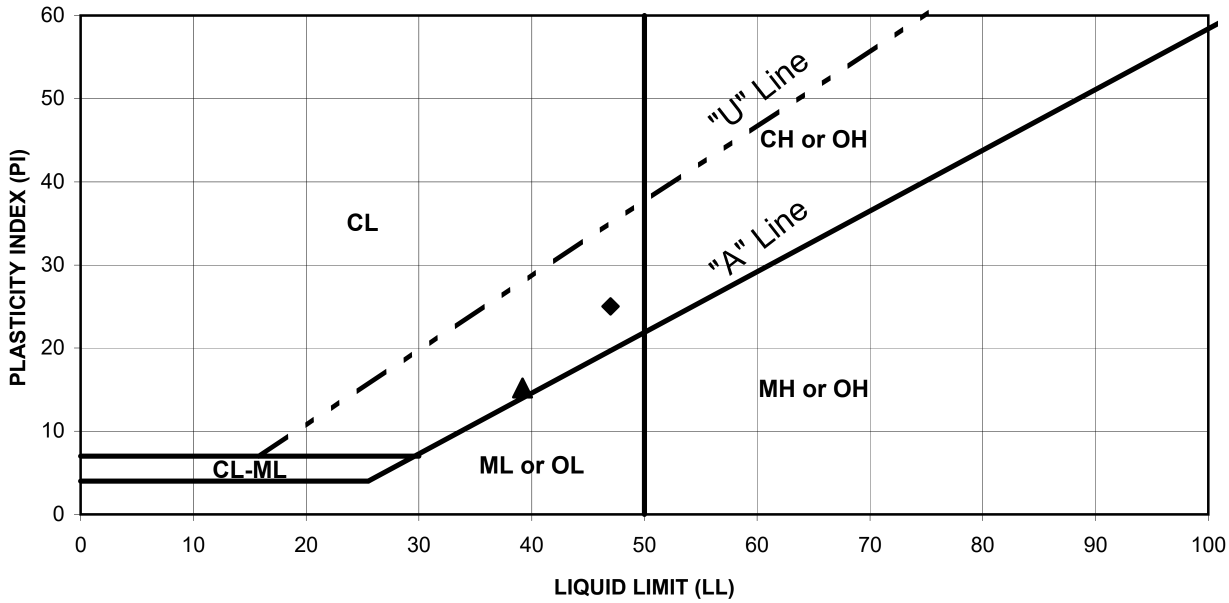


Symbol	Boring Number	Sample Number	Depth (feet)	LL	PL	PI	Plasticity Chart Symbol
♦	GP-2	9	45	59	21	38	CH
▲	GP-2	11	60	52	17	35	CH

AP Engineering and Testing, Inc.
DBE|MBE|SBE
2607 Pomona Boulevard | Pomona, CA 91768
t. 909.869.6316 | f. 909.869.6318 | www.aplaboratory.com

ATTERBERG LIMITS
ASTM D 4318

Client Name: GeoPentech | Tested By: LS | Date: 03/22/22
Project Name: 940 Sycamore Development | Computed By: NR | Date: 03/24/22
Project No.: 21106A | Checked By: AP | Date: 03/29/22



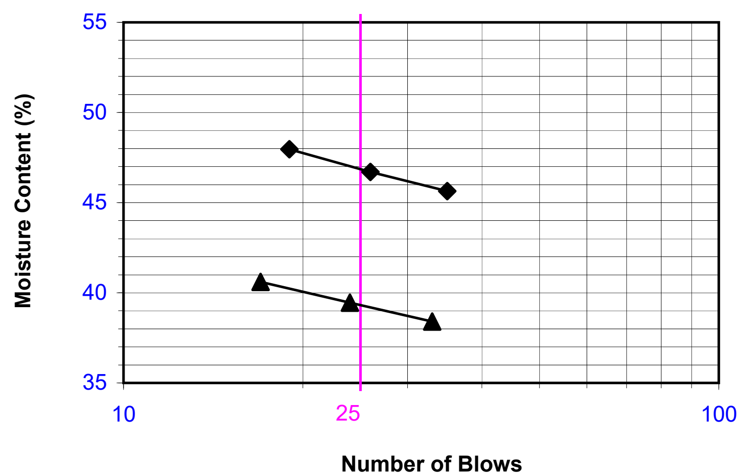
PROCEDURE USED

☐ Wet Preparation

☒ Dry Preparation

☒ Procedure A
Multipoint Test

☐ Procedure B
One-point Test

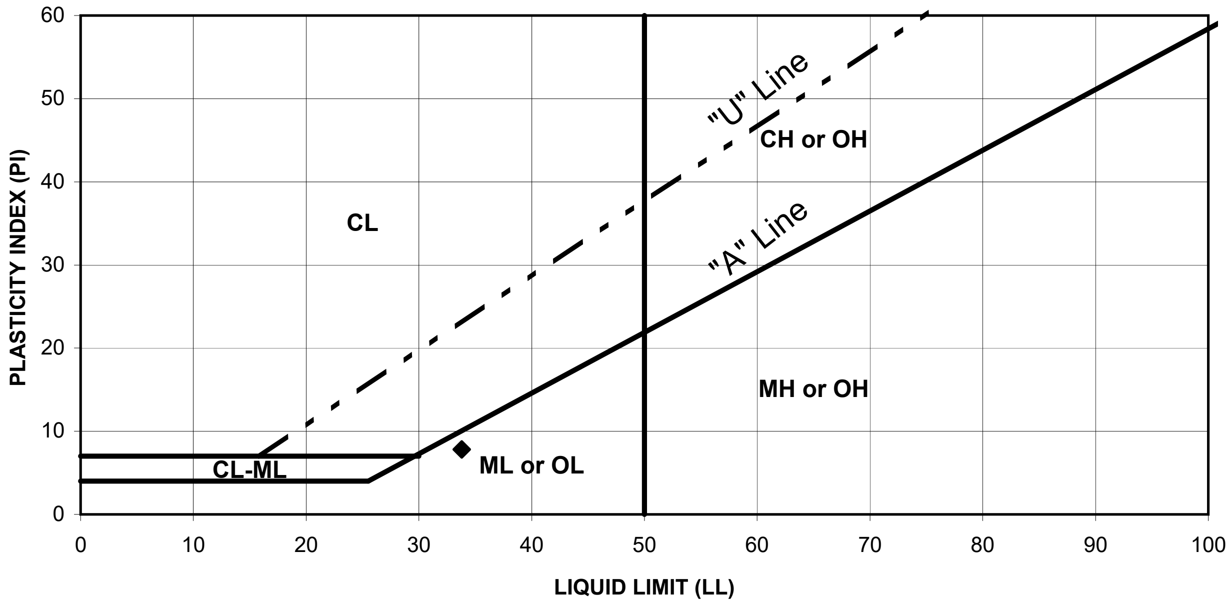


Symbol	Boring Number	Sample Number	Depth (feet)	LL	PL	PI	Plasticity Chart Symbol
♦	GP-2	17	120	47	22	25	CL
▲	GP-2	19	140	39	24	15	CL

AP Engineering and Testing, Inc.
DBE|MBE|SBE
2607 Pomona Boulevard | Pomona, CA 91768
t. 909.869.6316 | f. 909.869.6318 | www.aplaboratory.com

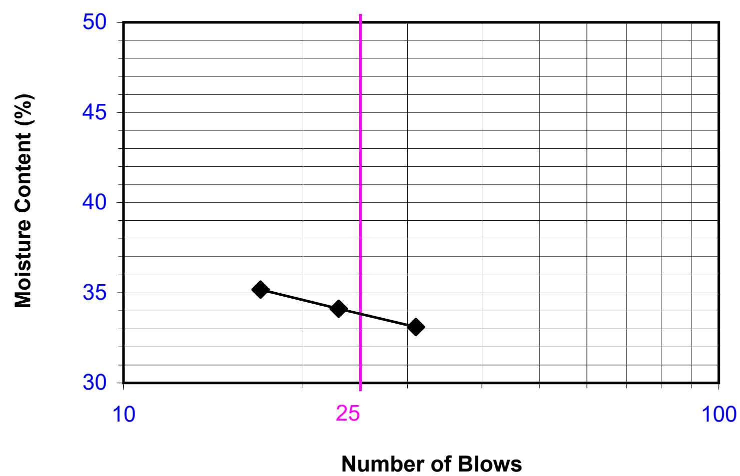
ATTERBERG LIMITS
ASTM D 4318

Client Name:	GeoPentech	Tested By:	LS	Date:	03/22/22
Project Name:	940 Sycamore Development	Computed By:	NR	Date:	03/24/22
Project No.:	21106A	Checked By:	AP	Date:	03/29/22



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	21	160	34	26	8	ML

AP Engineering and Testing, Inc.
DBE|MBE|SBE
2607 Pomona Boulevard | Pomona, CA 91768
t. 909.869.6316 | f. 909.869.6318 | www.aplaboratory.com

CORROSION TEST RESULTS

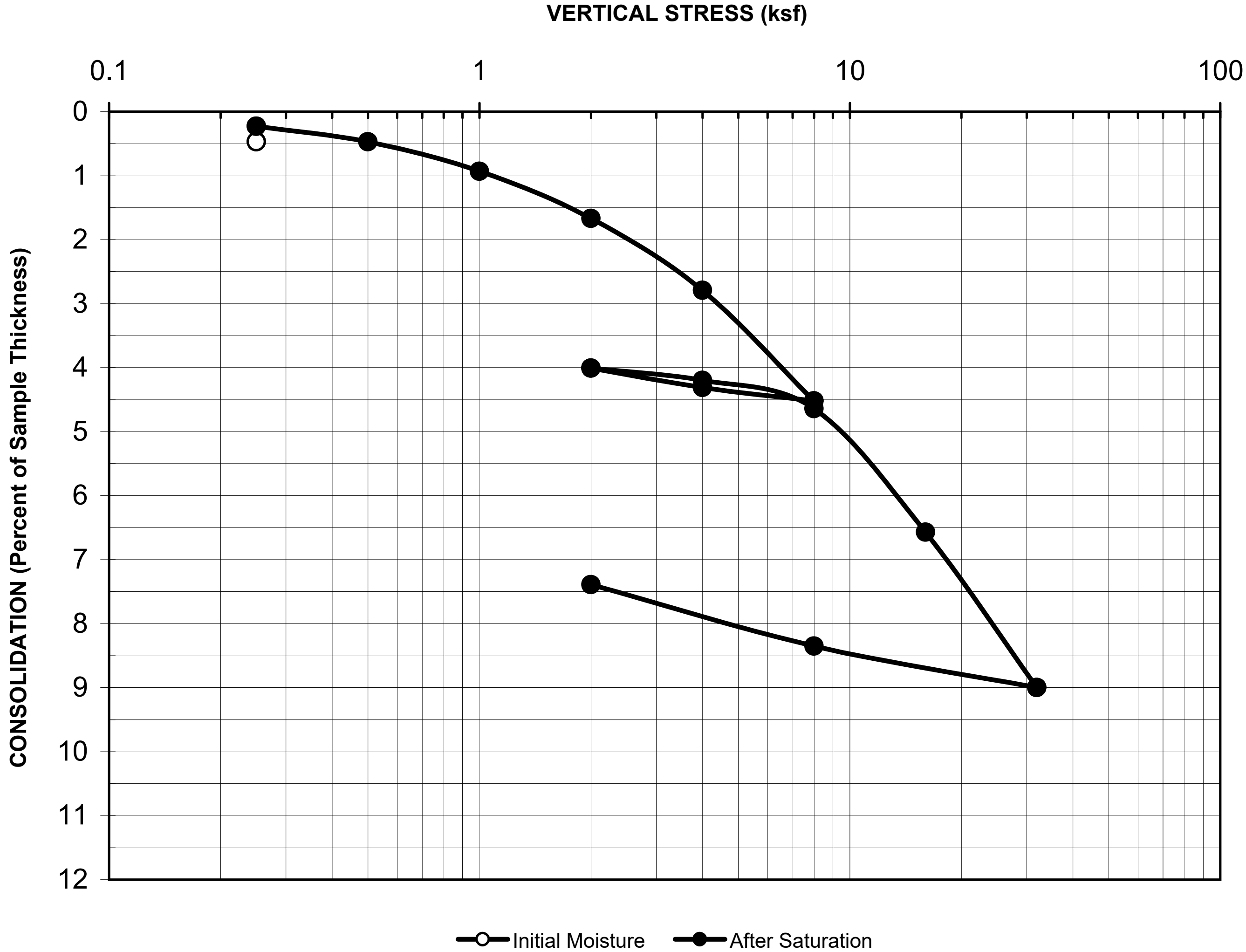
Client Name: GeoPentech
Project Name: 940 Sycamore Development
Project No.: 21106A

AP Job No.: 22-0346
Date: 03/25/22

Boring No.	Sample No.	Depth (feet)	Soil Description	Minimum Resistivity (ohm-cm)	pH	Sulfate Content (ppm)	Chloride Content (ppm)
GP-2	B-1	0-5	Clay	1,009	8.2	225	48

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

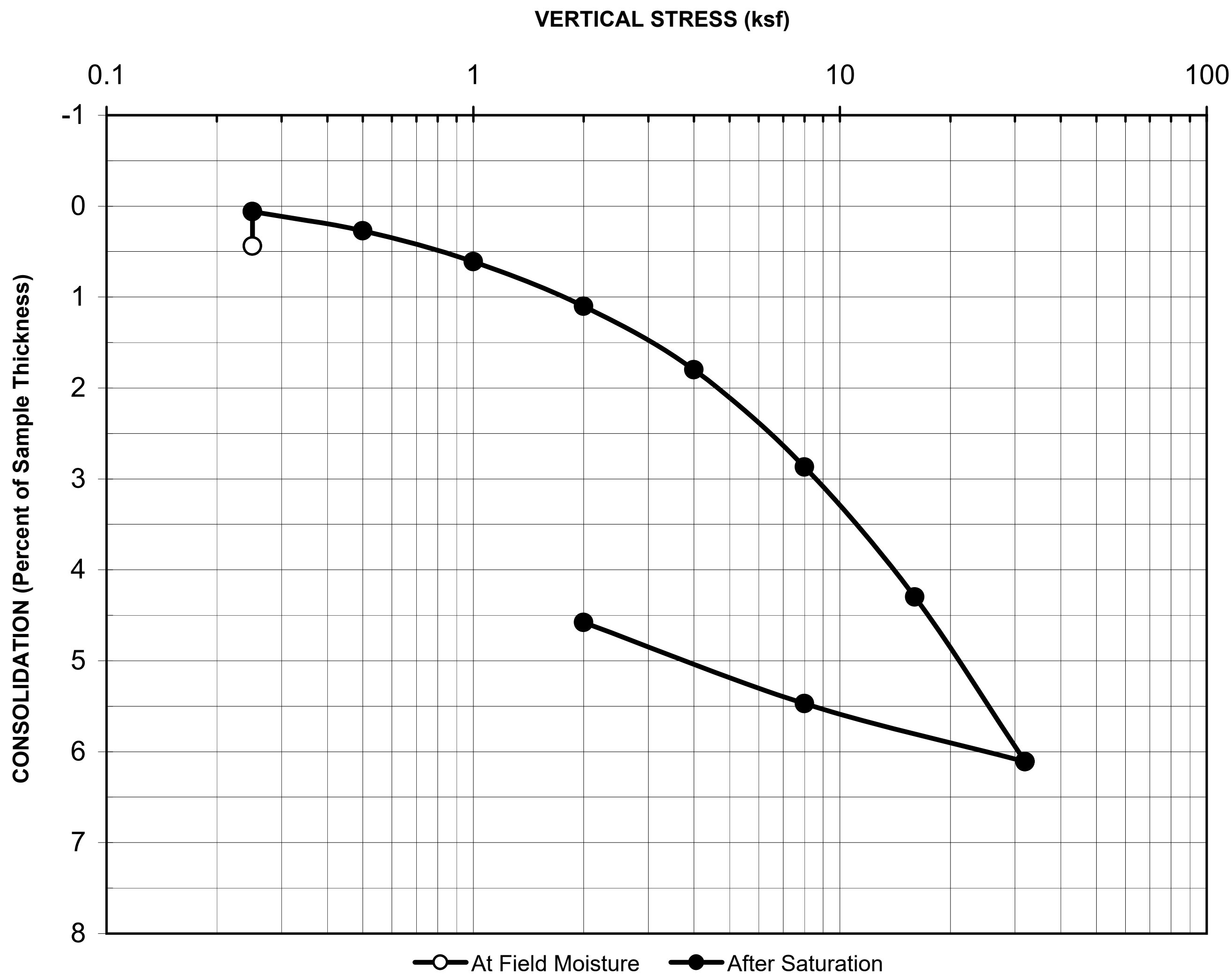
AP Engineering and Testing, Inc.
DBE|MBE|SBE
2607 Pomona Boulevard | Pomona, CA 91768
t. 909.869.6316 | f. 909.869.6318 | www.aplaboratory.com



Boring No. :	GP-1	Initial Dry Unit Weight (pcf):	111.1
Sample No.:	7	Initial Moisture Content (%):	18.6
Depth (feet):	35	Final Moisture Content (%):	18.7
Sample Type:	Mod Cal	Assumed Specific Gravity:	2.9
Soil Description:	Clayey Sand	Initial Void Ratio:	0.63
Remarks:	Swell= 0.24% upon inundation		

CONSOLIDATION CURVE
ASTM D 2435

Project Name:	940 Sycamore Development
Project No.:	21106A
Date:	03/18/22
AP No:	22-0346 **Sheet No:** 1

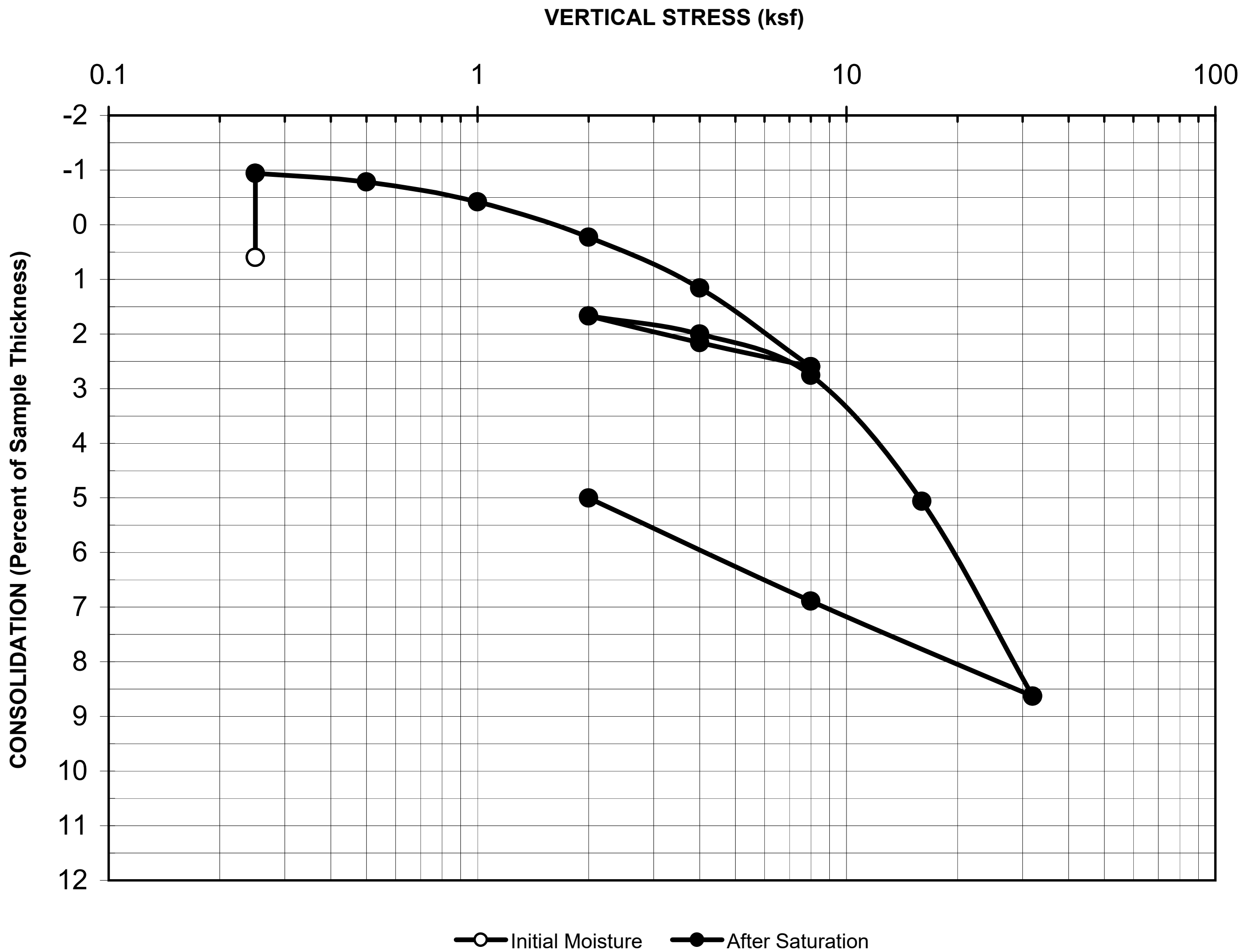


Boring No. :	GP-1	Initial Dry Unit Weight (pcf):	104.3
Sample No.:	15	Initial Moisture Content (%):	22.5
Depth (feet):	100	Final Moisture Content (%):	24.0
Sample Type:	Mod Cal	Assumed Specific Gravity:	2.7
Soil Description:	Siltstone	Initial Void Ratio:	0.61
Remarks:	Swell= 0.38% upon inundation		

CONSOLIDATION CURVE
ASTM D 2435

Project Name: 940 Sycamore Development
Project No.: 21106A
Date: 3/18/2022
AP No: 22-0346 **Sheet No:** 1

AP Engineering and Testing, Inc.
DBE|MBE|SBE
2607 Pomona Boulevard | Pomona, CA 91768
t. 909.869.6316 | f. 909.869.6318 | www.aplaboratory.com

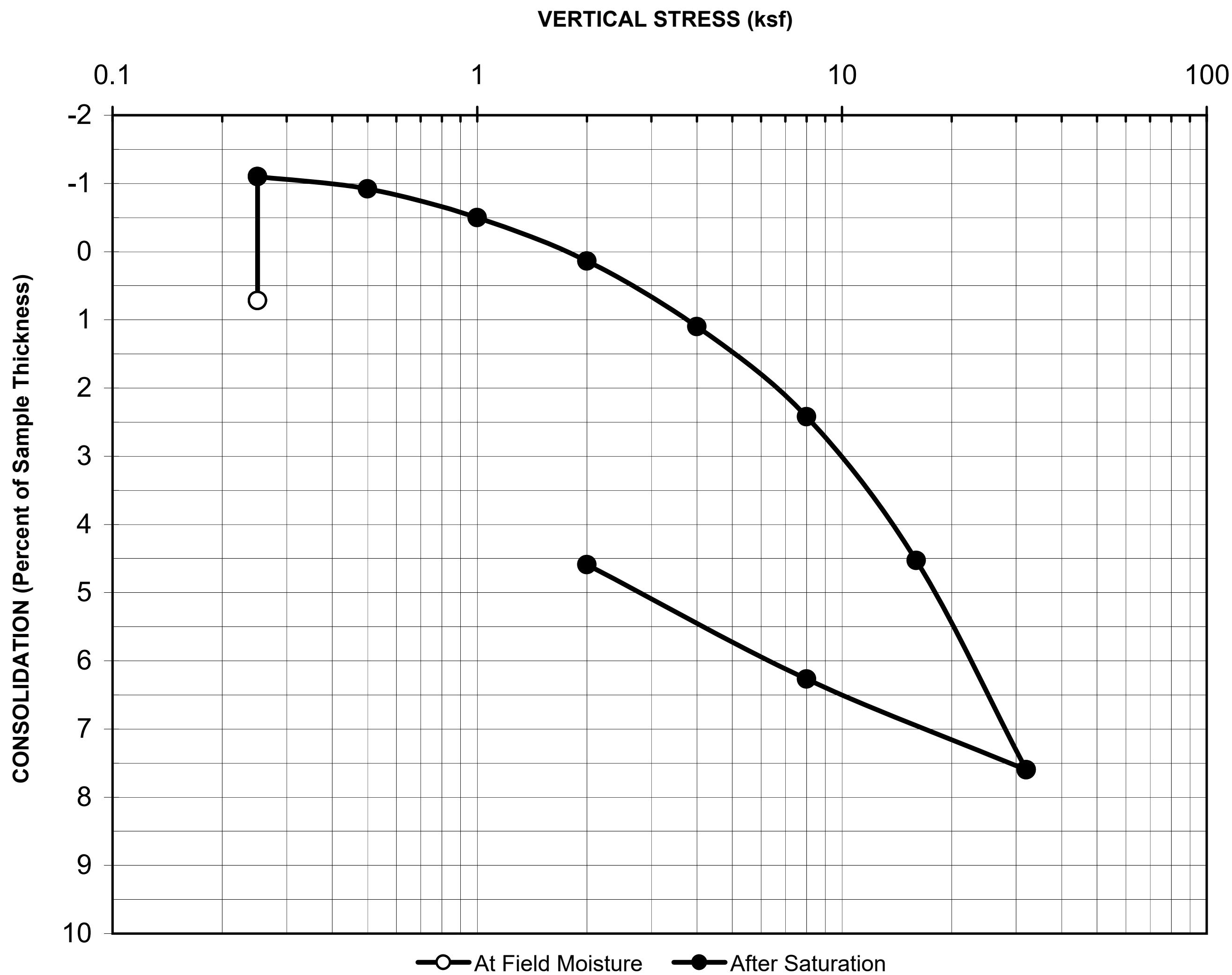


Boring No. :	GP-2	Initial Dry Unit Weight (pcf):	98.4
Sample No.:	9	Initial Moisture Content (%):	26.5
Depth (feet):	45	Final Moisture Content (%):	26.4
Sample Type:	Mod Cal	Assumed Specific Gravity:	2.9
Soil Description:	Fat Clay	Initial Void Ratio:	0.84
Remarks:	Swell= 1.54% upon inundation		

CONSOLIDATION CURVE
ASTM D 2435

Project Name:	940 Sycamore Development
Project No.:	21106A
Date:	03/18/22
AP No:	22-0346 **Sheet No:** 1

AP Engineering and Testing, Inc.
DBE|MBE|SBE
2607 Pomona Boulevard | Pomona, CA 91768
t. 909.869.6316 | f. 909.869.6318 | www.aplaboratory.com

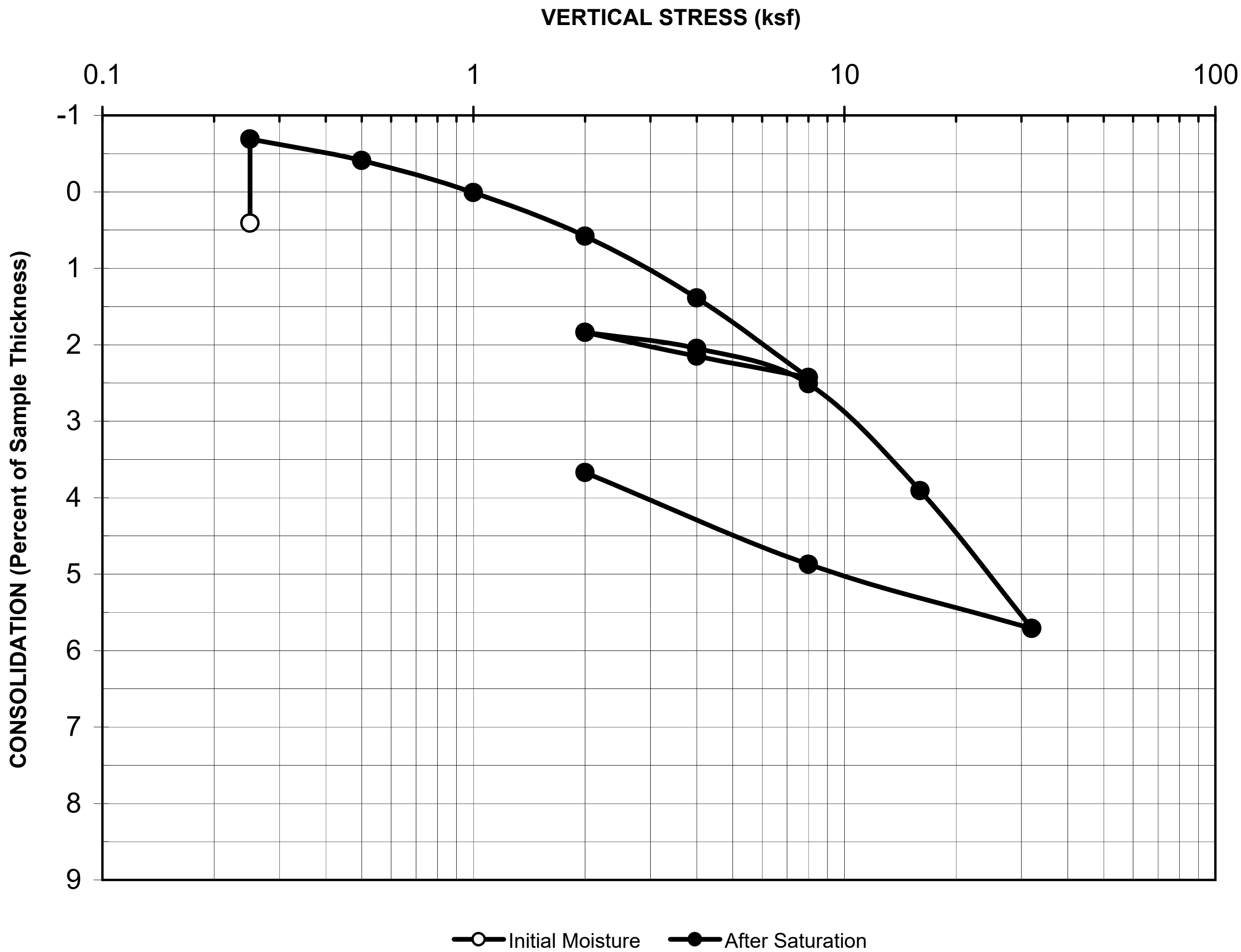


Boring No. :	GP-2	Initial Dry Unit Weight (pcf):	111.1
Sample No.:	11	Initial Moisture Content (%):	19.4
Depth (feet):	60	Final Moisture Content (%):	19.7
Sample Type:	Mod Cal	Assumed Specific Gravity:	2.7
Soil Description:	Sandy Fat Clay	Initial Void Ratio:	0.52
Remarks:	Swell= 1.82% upon inundation		

CONSOLIDATION CURVE
ASTM D 2435

Project Name: 940 Sycamore Development
Project No.: 21106A
Date: 3/18/2022
AP No: 22-0346 **Sheet No:** 1

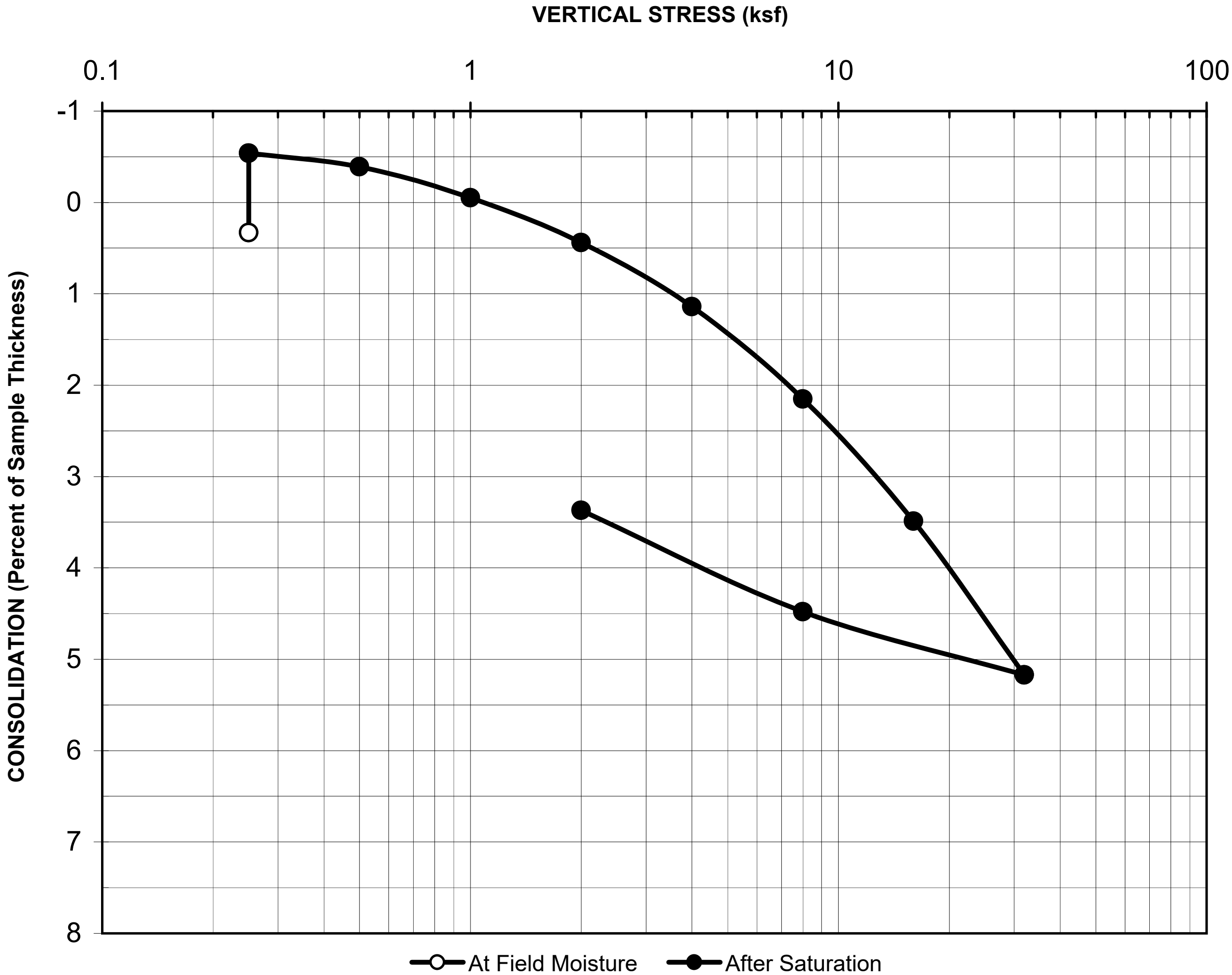
AP Engineering and Testing, Inc.
DBE|MBE|SBE
2607 Pomona Boulevard | Pomona, CA 91768
t. 909.869.6316 | f. 909.869.6318 | www.aplaboratory.com



Boring No. :	GP-2	Initial Dry Unit Weight (pcf):	103.5
Sample No.:	17	Initial Moisture Content (%):	24.2
Depth (feet):	120	Final Moisture Content (%):	24.1
Sample Type:	Mod Cal	Assumed Specific Gravity:	2.9
Soil Description:	Sandy Claystone	Initial Void Ratio:	0.75
Remarks:	Swell= 1.10% upon inundation		

CONSOLIDATION CURVE
ASTM D 2435

Project Name: 940 Sycamore Development
Project No.: 21106A
Date: 03/18/22
AP No: 22-0346 **Sheet No:** 1



Boring No. :	GP-2	Initial Dry Unit Weight (pcf):	105.8
Sample No.:	19	Initial Moisture Content (%):	21.4
Depth (feet):	140	Final Moisture Content (%):	22.7
Sample Type:	Mod Cal	Assumed Specific Gravity:	2.7
Soil Description:	Sandy Claystone	Initial Void Ratio:	0.59
Remarks:	Swell= 0.87% upon inundation		

CONSOLIDATION CURVE
ASTM D 2435

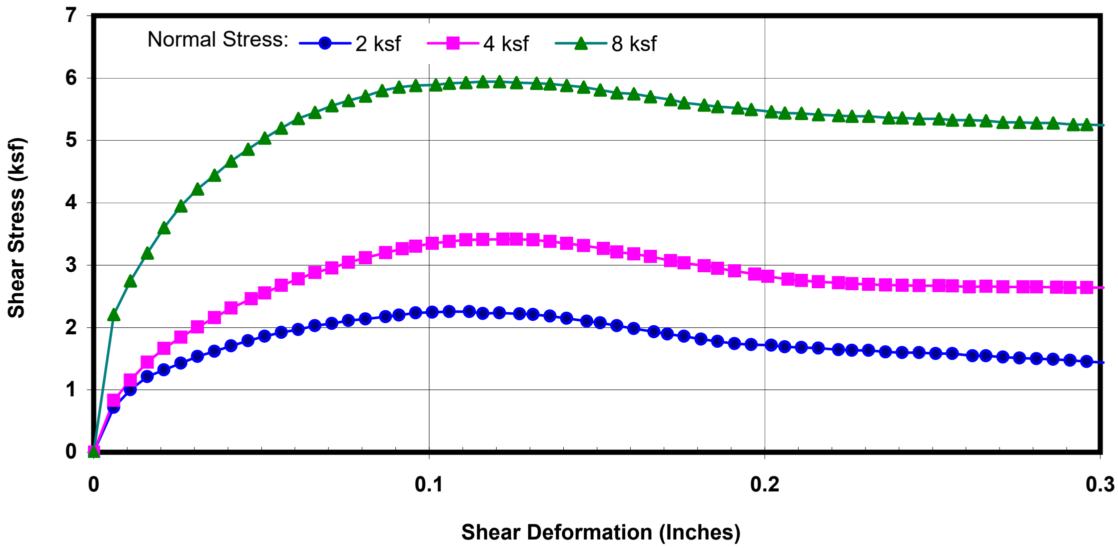
Project Name:	940 Sycamore Development
Project No.:	21106A
Date:	3/18/2022
AP No:	22-0346 **Sheet No:** 1

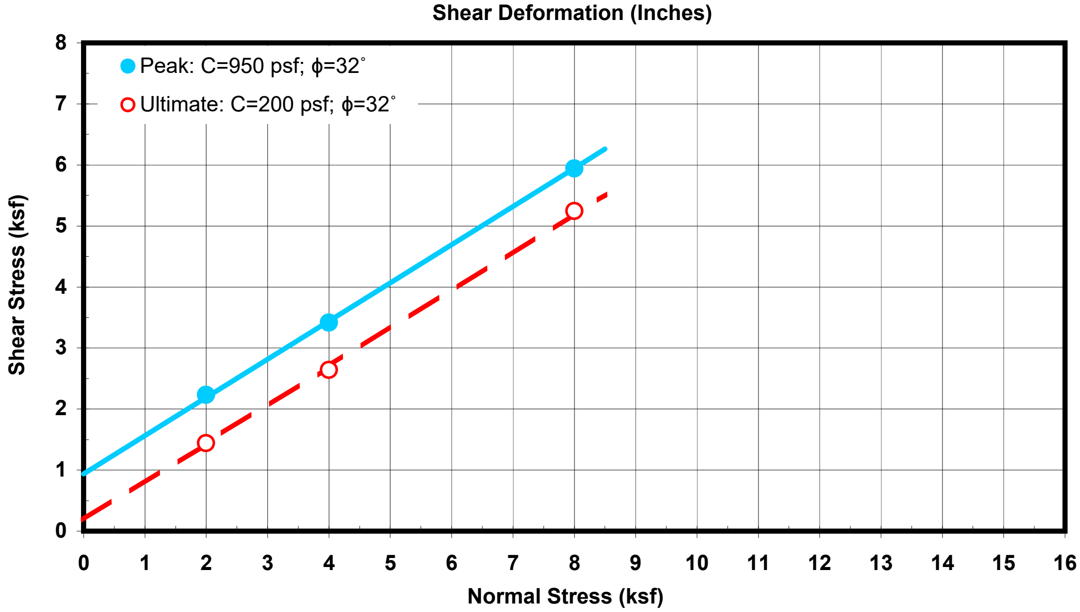
AP Engineering and Testing, Inc.
DBE|MBE|SBE
2607 Pomona Boulevard | Pomona, CA 91768
t. 909.869.6316 | f. 909.869.6318 | www.aplaboratory.com

DIRECT SHEAR TEST RESULTS
ASTM D 3080

Client:	GeoPentech	**Tested By:**	ST	**Date:**	03/22/22
Project Name:	940 Sycamore Development	**Computed By:**	NR	**Date:**	03/24/22
Project No.:	21106A	**Checked by:**	AP	**Date:**	03/29/22
Boring No.:	GP-1				
Sample No.:	11 **Depth (ft):** 60				
Sample Type:	Mod. Cal.				
Soil Description:	Clayey 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)
133.5	113.5	17.6	18.0	98	100	2	2.232	1.440
						4	3.414	2.640
						8	5.940	5.244





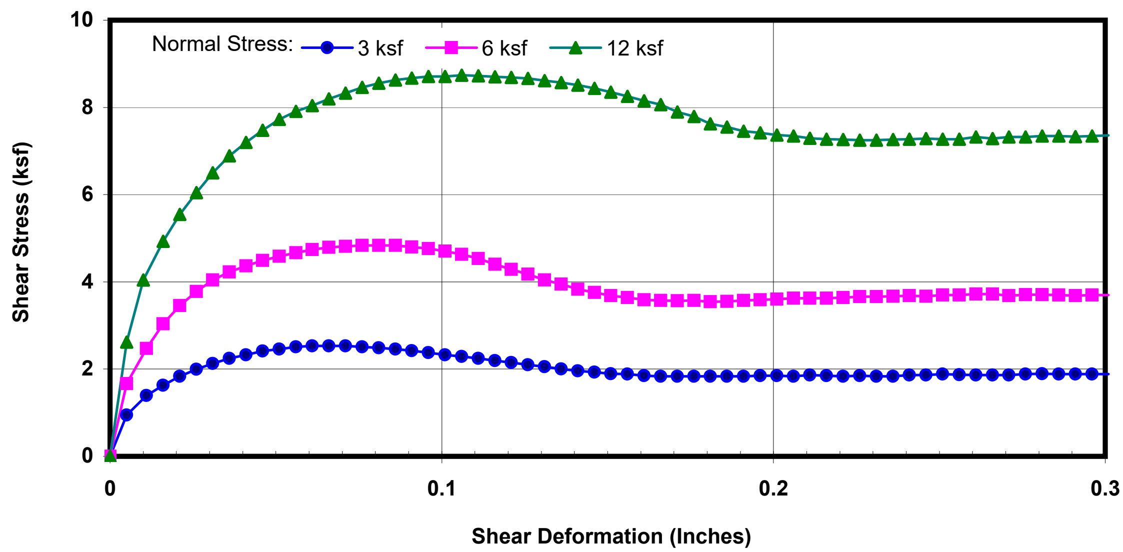
AP Engineering and Testing, Inc.
DBE|MBE|SBE
2607 Pomona Boulevard | Pomona, CA 91768
t. 909.869.6316 | f. 909.869.6318 | www.aplaboratory.com

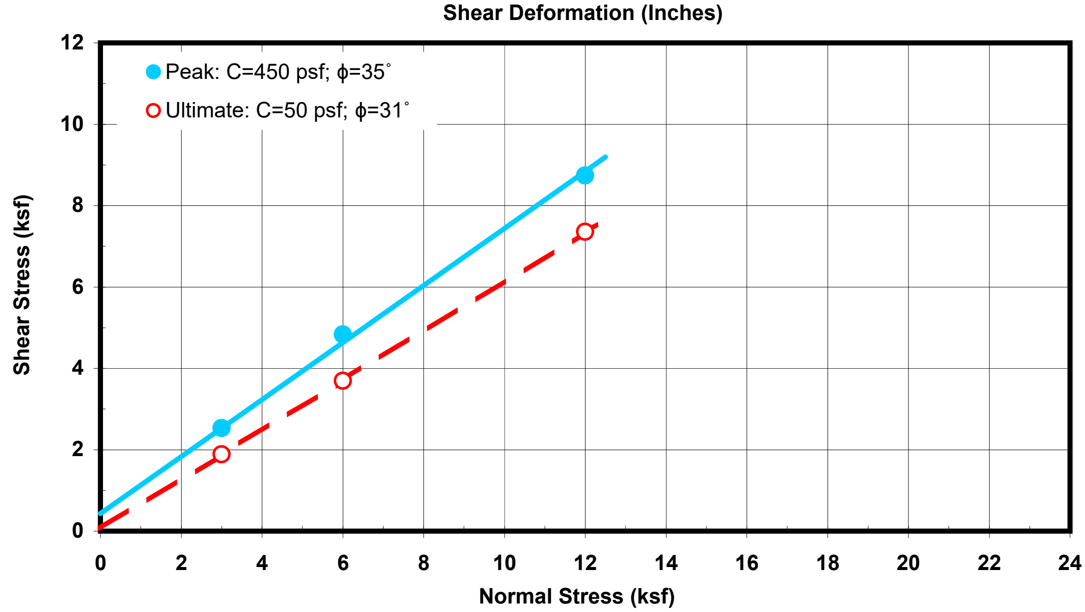
DIRECT SHEAR TEST RESULTS
ASTM D 3080

Client: GeoPentech
Project Name: 940 Sycamore Development
Project No.: 21106A
Boring No.: GP-1
Sample No.: 13 **Depth (ft):** 80
Sample Type: Mod. Cal.
Soil Description: Sand w/silt
Test Condition: Inundated **Shear Type:** Regular

Tested By: ST **Date:** 03/22/22
Computed By: NR **Date:** 03/24/22
Checked by: AP **Date:** 03/29/22

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)
128.6	107.8	19.3	20.9	93	100	3	2.532	1.884
						6	4.836	3.696
						12	8.735	7.356



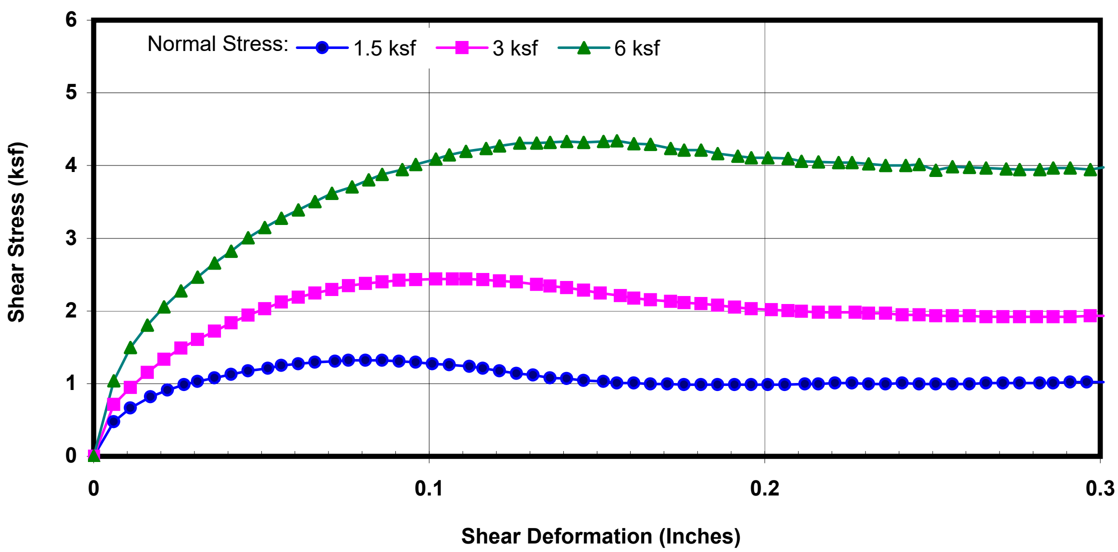


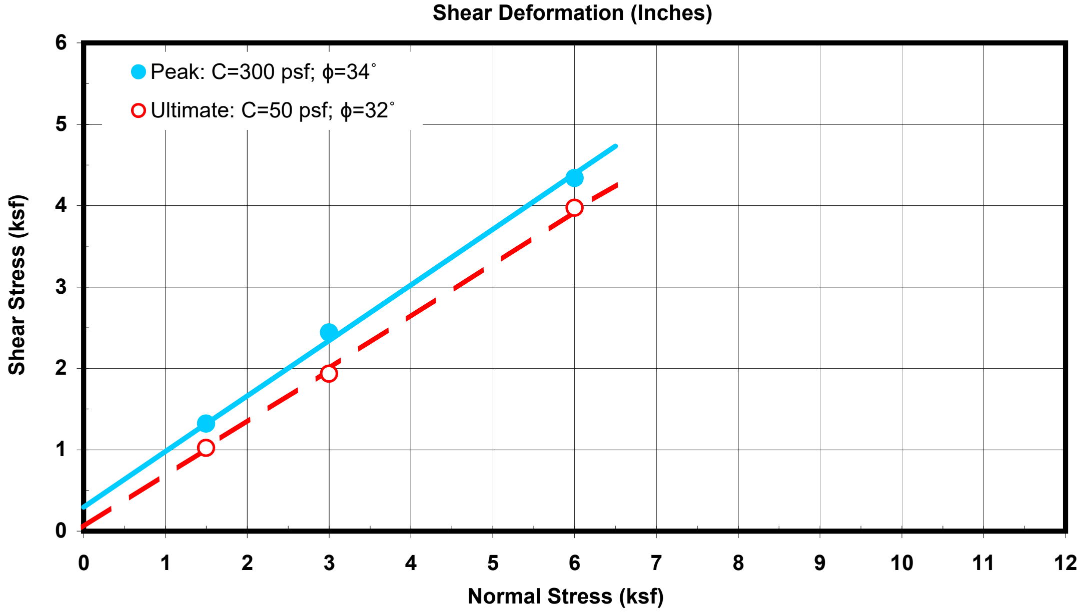
AP Engineering and Testing, Inc.
DBE|MBE|SBE
2607 Pomona Boulevard | Pomona, CA 91768
t. 909.869.6316 | f. 909.869.6318 | www.aplaboratory.com

DIRECT SHEAR TEST RESULTS
ASTM D 3080

Client:	GeoPentech	**Tested By:** JT	**Date:** 03/22/22
Project Name:	940 Sycamore Development	**Computed By:** NR	**Date:** 03/24/22
Project No.:	21106A	**Checked by:** AP	**Date:** 03/29/22
Boring No.:	GP-2		
Sample No.:	7 **Depth (ft):** 35		
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)
131.2	109.7	19.6	19.9	99	100	1.5	1.320	1.020
						3	2.440	1.932
						6	4.339	3.974





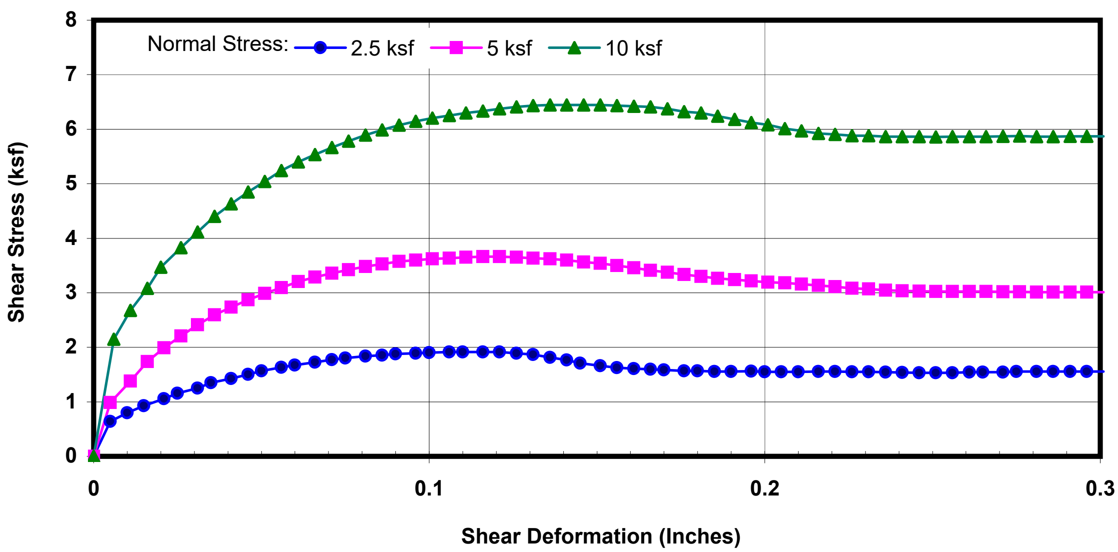
AP Engineering and Testing, Inc.
DBE|MBE|SBE
2607 Pomona Boulevard | Pomona, CA 91768
t. 909.869.6316 | f. 909.869.6318 | www.aplaboratory.com

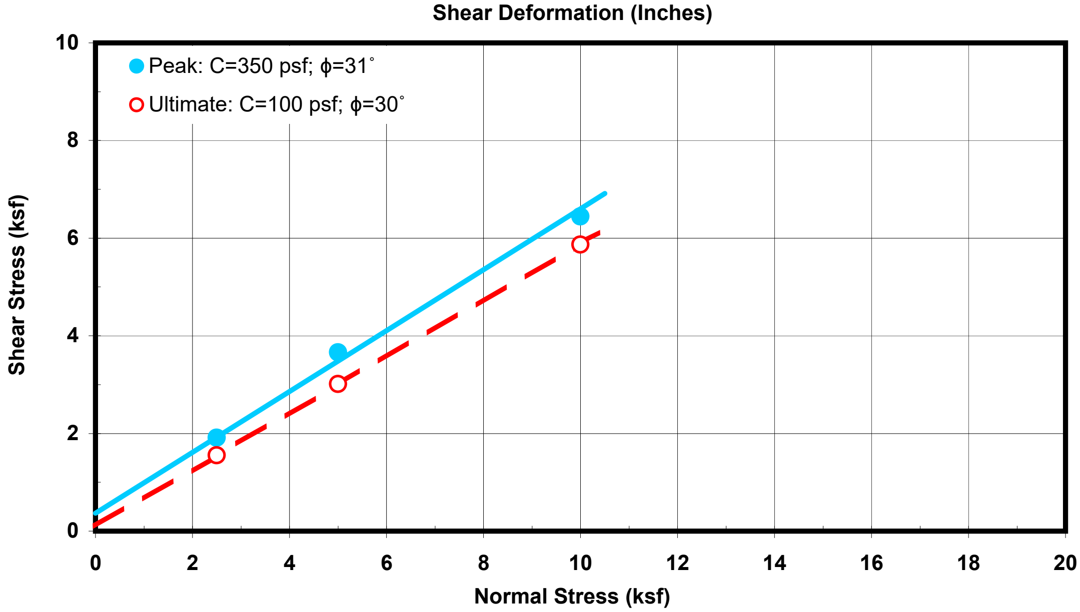
DIRECT SHEAR TEST RESULTS
ASTM D 3080

Client: GeoPentech
Project Name: 940 Sycamore Development
Project No.: 21106A
Boring No.: GP-2
Sample No.: 13 **Depth (ft):** 80
Sample Type: Mod. Cal.
Soil Description: Sand w/silt
Test Condition: Inundated **Shear Type:** Regular

Tested By: ST **Date:** 03/23/22
Computed By: NR **Date:** 03/24/22
Checked by: AP **Date:** 03/29/22

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)
125.4	101.3	23.7	24.5	97	100	2.5	1.912	1.555
						5	3.660	3.012
						10	6.444	5.868





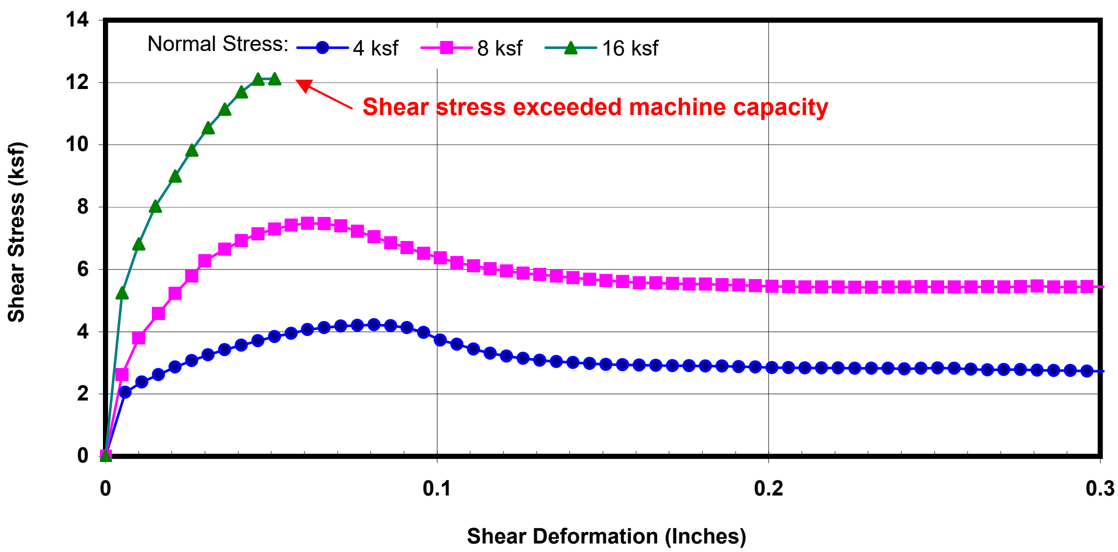
AP Engineering and Testing, Inc.
DBE|MBE|SBE
2607 Pomona Boulevard | Pomona, CA 91768
t. 909.869.6316 | f. 909.869.6318 | www.aplaboratory.com

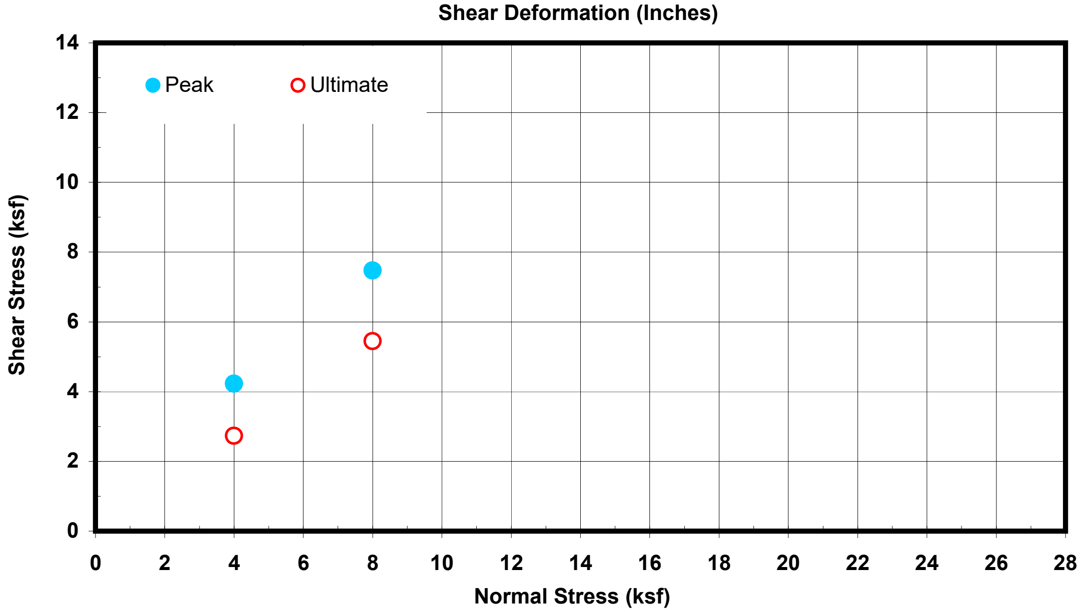
DIRECT SHEAR TEST RESULTS
ASTM D 3080

Client: GeoPentech
Project Name: 940 Sycamore Development
Project No.: 21106A
Boring No.: GP-2
Sample No.: 19 **Depth (ft):** 140
Sample Type: Mod. Cal.
Soil Description: Claystone
Test Condition: Inundated **Shear Type:** Regular

Tested By: ST **Date:** 03/23/22
Computed By: NR **Date:** 03/24/22
Checked by: AP **Date:** 03/29/22

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)
128.1	105.5	21.4	22.1	97	100	4	4.224	2.736
						8	7.476	5.448
						16	0.000	0.000





APPENDIX C

GEOPHYSICAL SURVEYS

C.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 commercial tower with 13 above-ground stories and 4 subgrade parking levels located south of Romaine Street between North Sycamore Avenue and North Orange Drive 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 C-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. VS30) at the site. The geophysical data were collected and processed by an assistant project scientist under the supervision of a California-licensed Professional Geophysicist.

C.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).

C.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 C-1, the three survey lines were performed along a west-to-east orientation 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 20 feet behind the first geophone and finishing 30 feet in front of the first geophone for lines SW22-1 and SW22-2. For line SW22-3, shots were performed at 5-foot intervals from 30 feet behind the first geophone to 5 feet behind the first geophone. 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.

C.4 Surface Wave Geophysical Results

The results of the combined MASW and ReMi surface wave measurements are presented in Figures C-2 through C-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 C-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 C-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 1,046 ft/s (319 m/s), which corresponds with NEHRP Site Class D, stiff soil (600 < V_{S30} ≤ 1,200 ft/s). V_{S30} values for depth intervals beginning below ground surface are also shown on Figure C-5.

C.5 References

Louie, J.N. (2001). Faster, Better: Shear-wave Velocity to 100 Meters Depth from Refraction Microtremor Arrays: Bulletin of the Seismological Society of America, v. 91, no. 2, p. 347-364.

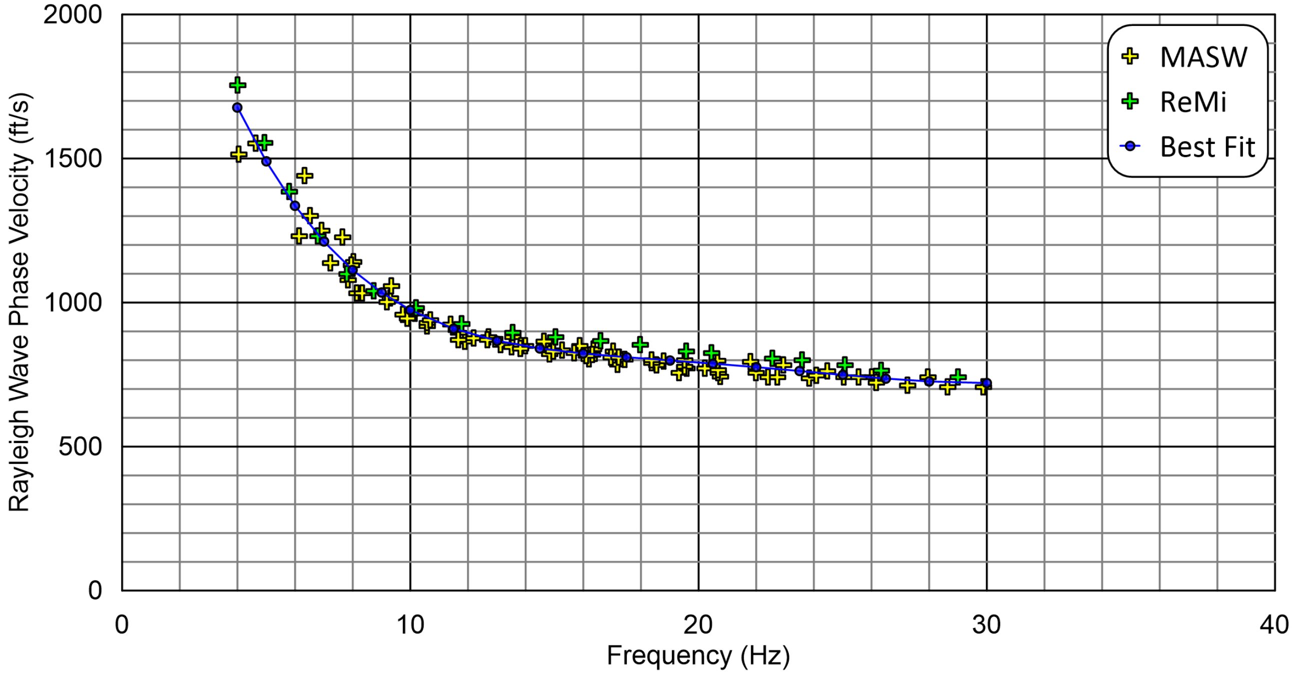
Louie, J.N., Pancha, A., and Kissane, B. (2021). Guidelines and pitfalls of refraction microtremor surveys: Journal of Seismology, published online June 7, 2021, https://doi.org/10.1007/s10950-021-10020-5.

Park, C.B., Miller, R.D., and Xia, J. (1999). Multichannel analysis of surface waves: Geophysics, v. 64, no. 3, pp. 800-808.

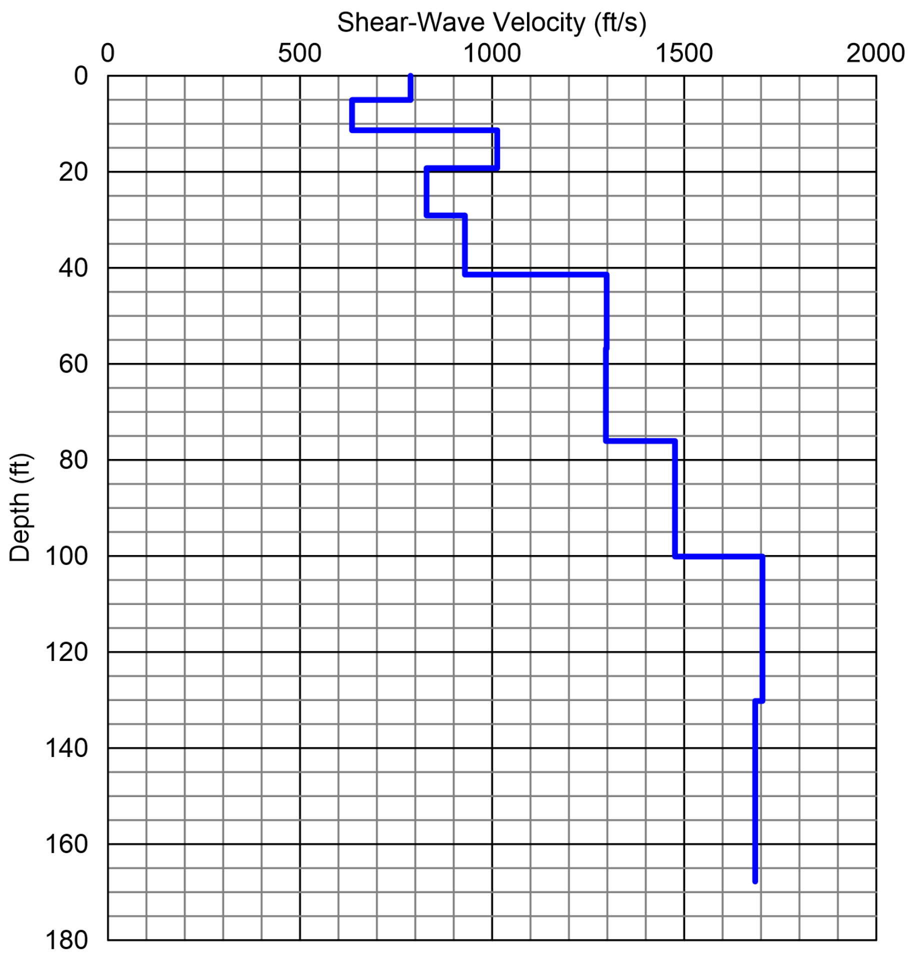
SITE MAP

GeoPentech	Project No.: **21106A**	Project: **940 N. Sycamore Ave.**	Date: **MAR 2022**	Figure: **C-1**

SURFACE WAVE DISPERSION CURVE



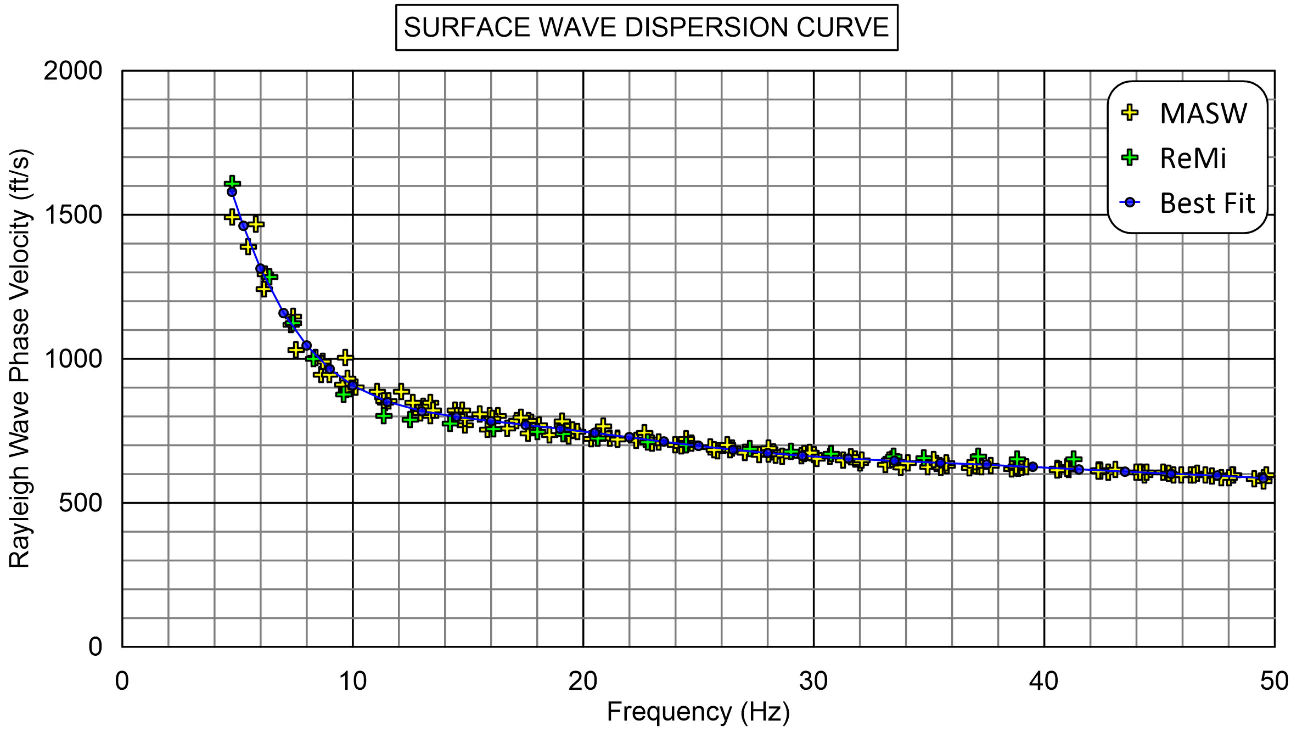
REPRESENTATIVE SHEAR-WAVE VELOCITY MODEL

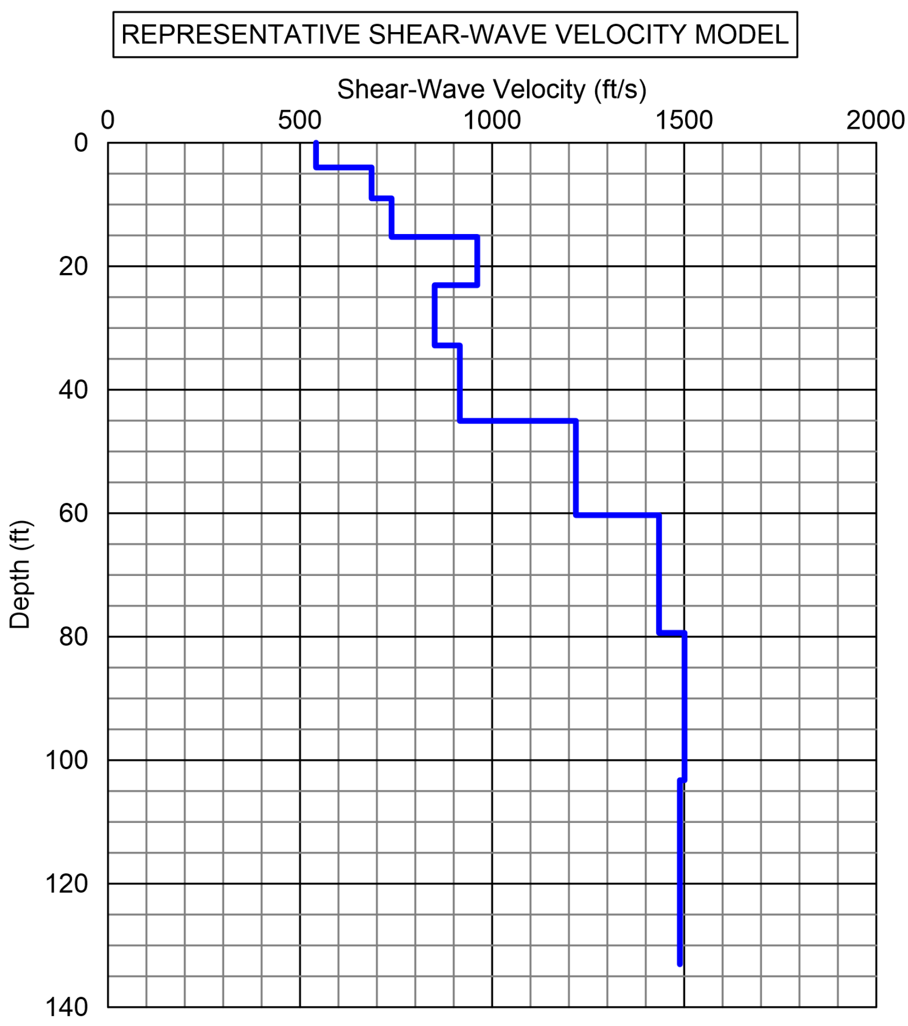


SW22-1: S-WAVE VELOCITY COMBINED SOURCE MODEL

Project No.: 21106A	Project: 940 N. Sycamore Ave.	Date: MAR 2022	Figure C-2

GeoPentech

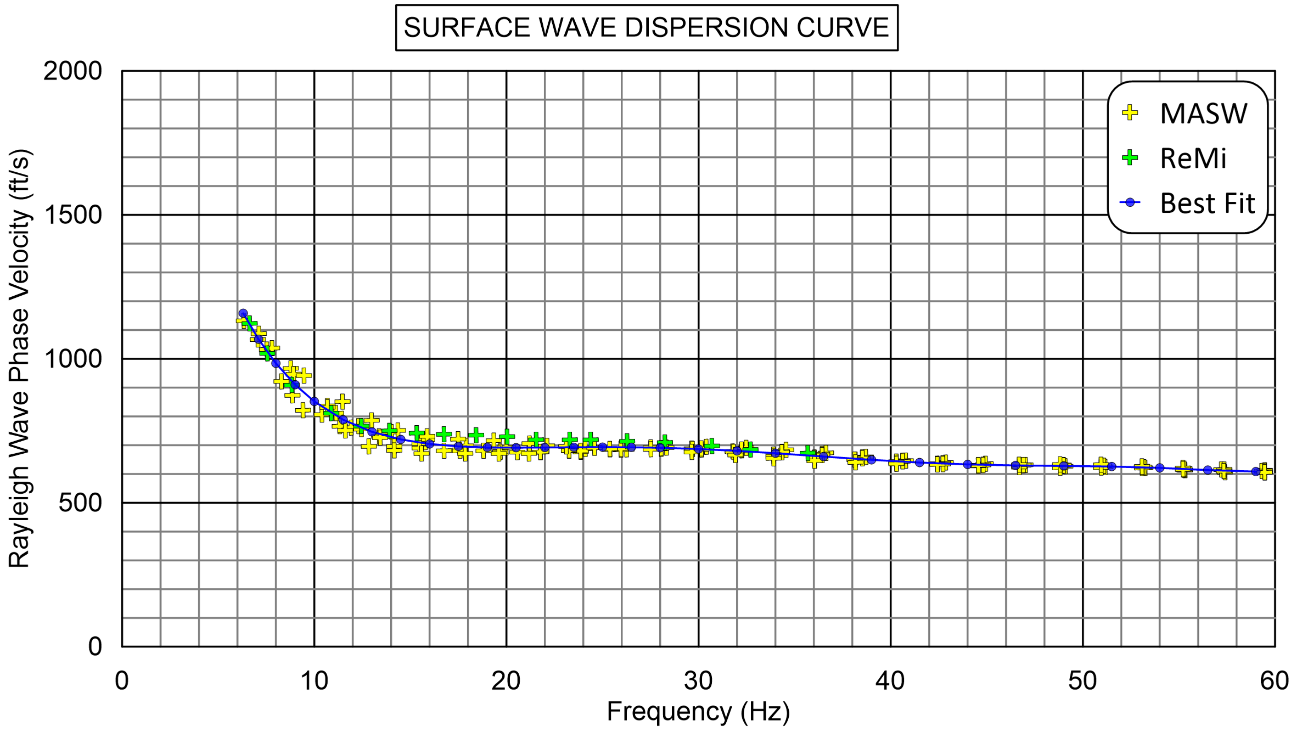


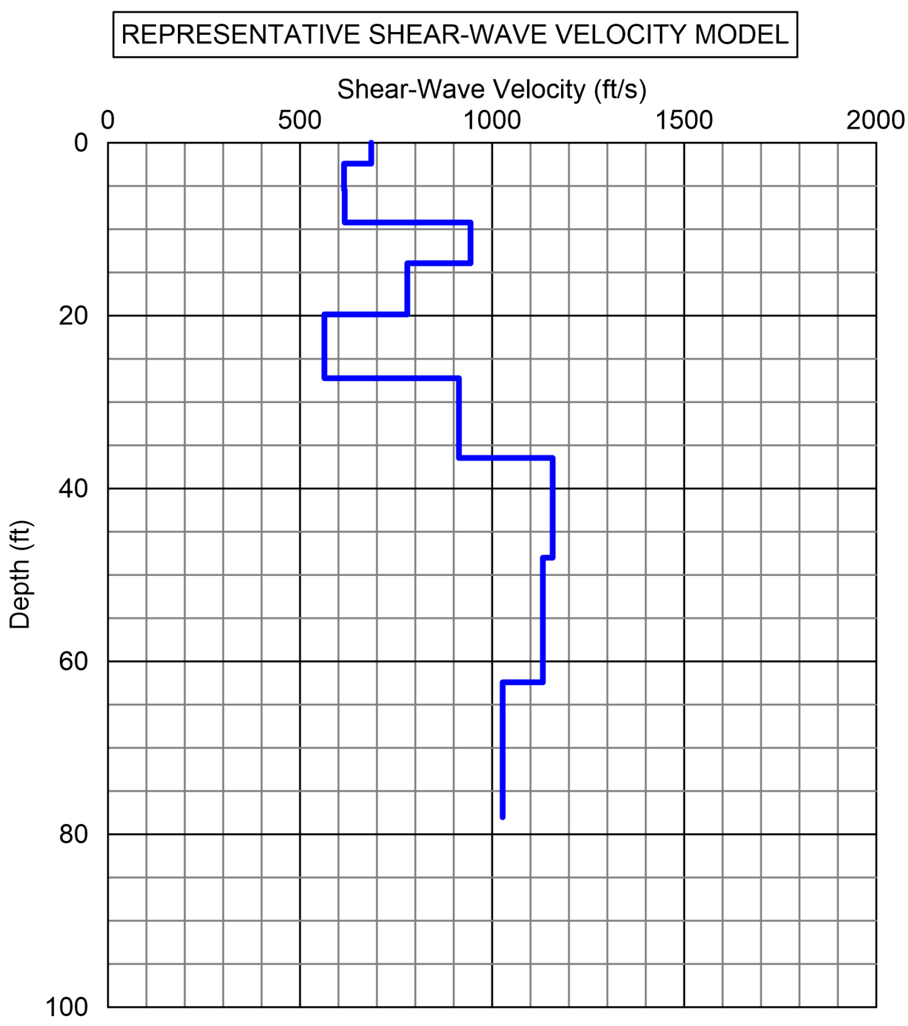


SW22-2: S-WAVE VELOCITY COMBINED SOURCE MODEL

Project No.: 21106A	Project: 940 N. Sycamore Ave.	Date: MAR 2022	Figure C-3

GeoPentech

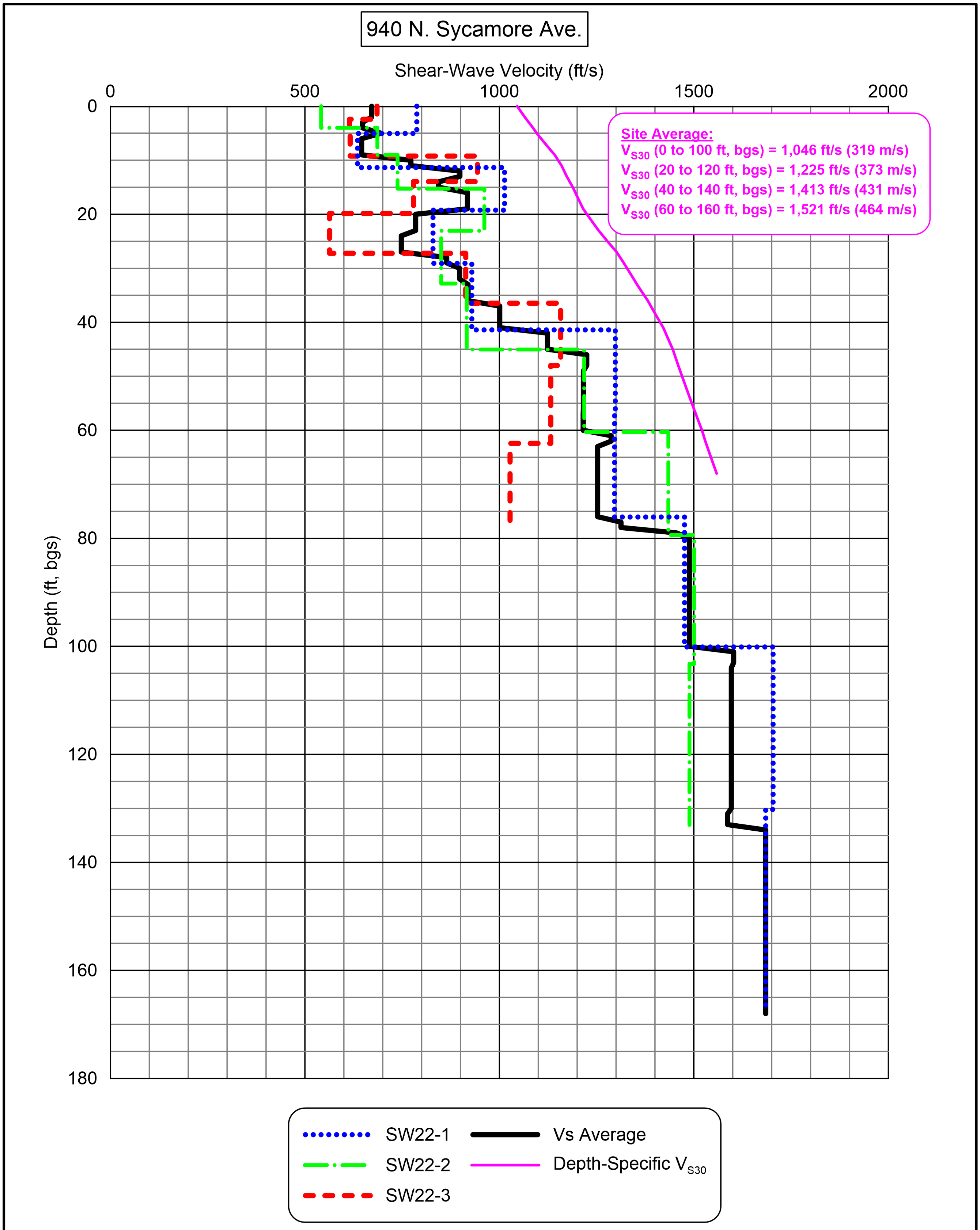




SW22-3: S-WAVE VELOCITY COMBINED SOURCE MODEL

Project No.: 21106A	Project: 940 N. Sycamore Ave.	Date: MAR 2022	Figure C-4

GeoPentech



SHEAR WAVE VELOCITY PROFILE SUMMARY

Project No.: 21106A	Project: 940 N. Sycamore Ave.	Date: MAR 2022	Figure C-5

GeoPentech

APPENDIX D

GROUND-MOTION ANALYSIS

D.1 INTRODUCTION

This Appendix presents the ground-motion evaluation results for the subject site located on Figure D-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.

The currently proposed development includes the design and construction of a 196-ft tall, 13-story midrise tower that includes four parking levels attaining an approximate depth of 62 feet below grade with the mat foundation extending to approximately 67-73 feet below grade. The estimated fundamental spectral period of interest of the structure is not finalized at this time, but it is estimated to be about 2.0-seconds, and will be confirmed when the structural design is finalized.

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. To meet the design requirements, the following levels of seismic evaluation will be completed: [1] Collapse Prevention Evaluation will be performed using the Risk-Targeted Maximum Considered Earthquake (MCE_R) response spectrum, and [2] Design of nonstructural components will be based on the Design Response Spectrum (DRS). Should the project require a Serviceability Evaluation, a Service Level Earthquake (SLE) response spectrum 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.

- “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.

- “Service-Level Earthquake” uniform hazard spectrum (also known as the SLE response spectrum); This response spectrum is based on average horizontal spectral ordinates at 1.95% damping and corresponds to a 50% probability of exceedance in a 30-year period.

At this time, consistent with the recommendations by the project’s Structural Engineer of Record, Mr. Harrison Glotman of Glotman and Simpson, we understand that the structural evaluation is anticipated to be based on elastic response spectrum analysis and will not need spectrum-compatible time histories during this phase. Note that if the site location or site conditions change appreciably, the ground-motion results presented herein would need to be re-evaluated.

D.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 C of this report. On Figure D-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 62 feet below grade, on a 5-ft thick mat slab foundation. 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, Commentary of Chapter 20 in ASCE 7-16, and Section 3.2.4 of the 2020 Los Angeles Tall Buildings Structural Design Council (LATBSDC) guidelines, we recommend the V_{S30} be computed from the 30-ft depth. This corresponds to a V_{S30} value of 1,328 ft/s (405 m/s). This V_{S30} value corresponds to Site Class C (1,200 < V_{S30} < 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 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.

D.3 ASCE 7-16 CODE-BASED VALUES

Given the site latitude and longitude (located near -118.342244°W, 34.088184°N) 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 D.2 of this Appendix, the estimated V_{S30} at the site foundation level is 1,328 ft/s (405 m/s). This V_{S30} value corresponds to site classification for seismic design of Site Class C (1,200 < V_{S30} < 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 S_S and S_1 of 2.089 g and 0.749 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 1.671 g and 0.699 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.

D.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 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 D.5) of this Appendix.

D.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 D-3a.

The closest Late Quaternary (within the last 15,000 years) surface fault ruptures occurred on Hollywood Fault roughly 2 km north of the site, based on the fault locations mapped in the Hollywood Quadrangle Earthquake Zones of Required Investigation Map for the Alquist-Priolo Fault Zone. Other nearby faults with Late Quaternary surface rupture include the Santa Monica, Raymond, and Newport-Inglewood faults, each located roughly 7 to 12 km from the project site (Figure D-3a).

Several historic earthquakes have occurred within 50 km of the project site, as shown on Figure D-3a. The epicenter for the 1994 Northridge earthquake was approximately 23½ km northwest of the project site. Based on the recording stations in the NGA/PEER database, the event produced peak horizontal ground accelerations (PGA) and peak ground velocities (PGV) of about 0.20 g to 0.33 g and 20 cm/s to 23 cm/s, respectively, near the project site. The 1987 Whittier Narrows earthquake epicenter was approximately 23 km east-southeast of the project site; that event produced PGA and PGV measurements of about 0.15 g and 8 cm/s, respectively, near the project site.

D.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 D-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.

D.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 D-3b. The best-estimate parameters (including maximum magnitude, closest distance, slip rate, and style of faulting) for these seismic sources are summarized in Table D-1.

All faults shown on Figure D-3b and listed in Table D-1 were included in the HAZ45 PSHA. In addition to the discrete seismic sources presented in Table D-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 D-2.

D.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 of the 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 the deterministic MCE_R spectral ordinates exceed the probabilistic MCE_R ones, and the code-based minimum controls at long periods.

D.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.

D.4.5.1 Source Contribution Hazard Curves

Figures D-4a and D-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 2.0-seconds (assumed to represent the fundamental mode of the building). The total hazard (solid black line) and the contributions of various seismic sources to the total seismic hazard are shown. Table D-3 lists the relative contributions of significant seismic sources at various hazard levels for the 0.2 seconds and the 2.0-second spectral periods for the horizontal ground motions. As indicated on Table D-3 and Figure D-4a, the Hollywood Fault controls the horizontal 0.2-seconds hazard for average return periods longer than about 100 years. This is expected given the proximity and slip rate of the Hollywood Fault, as listed on Table D-1. Other key contributors to the hazard are the Elysian Park System (mainly the Elysian Park Upper Fault) and the Puente Hills System (single-fault model and three-fault model with LA segment as the main contributor). The 2.0-second hazard contributions (Table D-3 and Figure D-4b) are similar to the horizontal 0.2-seconds contributions, with significant hazard contribution from the Hollyood 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.

D.4.5.2 Deaggregation Plots

Magnitude-distance deaggregations for 0.2-seconds and 2.0-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 D-5a and D-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 D-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 D-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 20 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 30 km of the site. There is also a clear contribution from characteristic events on the San Andreas located 55.5 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 D-5b shows the deaggregation at the same average return periods for 2.0-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 notwithstanding the range of distance does not appear changed with respect of the shorter-periods deaggregations. 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 55.5 km away from the site, especially at short average return periods. At the 225-yr and 43-yr 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 30 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 and other distant faults with high slip rates.

D.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 D-4 in Column 3,4, and 5, and the resulting average UHS is plotted on Figure D-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 D-6. The ordinates are also tabulated on Table D-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 0.897 is specified for all periods. The incorporation of these scale factors is reflected in the modified probabilistic MCE_R spectrum on Figure D-6, and the process of developing the probabilistic MCE_R spectral ordinates is shown on Table D-4 in Columns 3 through 8.

The Serviceability Evaluation per the 2020 LATBSDC guidelines uses the Service-Level Earthquake (SLE) spectrum, which is 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 D-6. Development of the SLE spectrum, including conversion of the hazard ordinates to the target damping ratio, is discussed below.

D.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 D-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 D-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 D-7, the Puente Hills (LA) Fault and the Compton Fault sources cases present very similar spectral accelerations between PGA up to about 0.75 seconds, with the Puente Hills (LA) Fault exceeding the other case. A periods between 1 and 4 seconds, the combined Raymond-Hollywood-Santa Monica case present the largest demand, whereas at longer periods the Newport-Inglewood case trades off in controlling the deterministic MCE_R spectrum. The deterministic MCE_R spectral ordinates are tabulated in Table D-4 in Column 12, and the process of developing the deterministic MCE_R spectral ordinates is shown on Table D-4 in Columns 9 through 12.

D.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. 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. 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. For completeness, our analyses also encompass Serviceability Evaluation by developing 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 development of these spectra is discussed below.

D.5.1 Site-Specific MCE_R Response Spectrum

The left panel of Figure D-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 D-8, the probabilistic MCE_R spectrum controls at all spectral periods beside; however, the final horizontal site-specific MCE_R is adjusted such that none of the spectral ordinates fall below the code-based minimum. The code-based minimum controls in the narrow 0.02 - to 0.1-second period range and at periods above 7.5 seconds. The final site-specific MCE_R spectrum is shown highlighted in the left panel on Figure D-8, and the spectral ordinates are tabulated in Table D-4, Column 14. The process of developing the site-specific horizontal MCE_R spectral ordinates is shown in Table D-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 D.4.5.3. The results are shown in the right panel on Figure D-8, and can be used to support the future seed acceleration time history selection, should the need arise in the future. For the design purpose, the RotD100 spectral ordinates correspond to our recoomended spectra.

D.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 D-9. The final recommended horizontal DRS is shown highlighted on Figure D-9, and the ordinates are tabulated in Table D-5, Column 6. The process of developing the horizontal DRS ordinates is shown in Table D-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 1.0 and 2.5, respectively, in accordance with ASCE 7-16, Section 21.3.

- S_{DS} = 1.606 g, based on 90% of the spectral acceleration at a period of 0.3-seconds
- S_{D1} = 0.890 g, based on the spectral acceleration at a period of 1.0-second
- S_{MS} = 2.409 g, based on 1.5 times S_{DS}
- S_{M1} = 1.355 g, based on 1.5 times S_{D1}

D.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 D-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 2.57% 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 2.57% 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 D-6 in Column 7. The process of developing the SLE ordinates is also shown in Table D-6.

D.6 REFERENCES

Abrahamson, N.A. and Gregor, N. (2021). Personal communication. Haz45 FORTRAN source files.

Abrahamson, N.A., Silva, W.J., and Kamai, R. (2014). Summary of the ASK14 Ground Motion Relation for Active Crustal Regions: Earthquake Spectra, vol. 30 No. 3, pp. 1025-1055.

American Society of Civil Engineers - ASCE (2018). Minimum Design Loads and Associated Criteria for Buildings and Other Structures Supplement 1. ASCE Standard ASCE/SEI 7-16. American Society of Civil Engineers.

American Society of Civil Engineers - ASCE (2017). Minimum Design Loads for Buildings and Other Structures. ASCE Standard ASCE/SEI 7-16. American Society of Civil Engineers.

Ancheta, T.D., Darragh, R.B., Stewart, J.P., Seyhan, E., Silva, W.J., Chiou, B.S.-J., Wooddell, K.E., Graves, R.W., Kottke, A.R., Boore, D.M., Kishida, T., and Donahue, J.L. (2014). NGA-West2 Database: Earthquake Spectra, Vol. 30(3), 989-1005.

Bayless, J., and Somerville, P. (2013). Bayless-Somerville Directivity Model, Chapter 3 of PEER Report No. 2013/09, P. Spudich (Editor), Pacific Earthquake Engineering Research Center, Berkeley, CA.

Boore, D.M., Stewart, J.P., Seyhan, E., and Atkinson, G.M. (2014). NGA-West2 Equations for Predicting PGA, PGV, and 5% Damped PSA for Shallow Crustal Earthquakes: Earthquake Spectra, vol. 30 No. 3, pp. 1057-1085.

California Building Standards Code (CBC), (2019). California Code of Regulations. California Building Standards Commission Based on the 2018 International Building Code, Sacramento, CA.

Campbell, K.W., and Bozorgnia, Y. (2014). NGA-West2 Ground Motion Model for the Average Horizontal Components of PGA, PGV, and 5% Damped Linear Acceleration Response Spectra: Earthquake Spectra, vol. 30 No. 3, pp. 1087-1115.

Chiou, B.S.-J., and Youngs, R.R. (2014). Update of the Chiou and Youngs NGA Model for the Average Horizontal Component of Peak Ground Motion and Response Spectra: Earthquake Spectra, vol. 30 No. 3, pp. 1117-1153.

Hauksson, E., Yang, W., and Shearer, P.M. (2012, updated online 2018). Waveform Relocated Earthquake Catalog for Southern California (1981 to June 2011): Bulletin of the Seismological Society of America, vol. 102, no. 5, p. 2239-2244.

Idriss, I.M. (2014). An NGA-West2 Empirical Model for Estimating the Horizontal Spectral Values Generated by Shallow Crustal Earthquakes: Earthquake Spectra, vol. 30 No. 3, pp. 1155-1177.

Los Angeles Tall Buildings Structural Design Council (LATBSDC) (2020). An Alternative Procedure for Seismic Analysis and Design of Tall Buildings Located in the Los Angeles Region, 94 pp.

Magistrale, H., Day S., Clayton R.W., Graves R.W. (2000). The SCEC Southern California Reference Three Dimensional Seismic Velocity Model Version 2, Bulletin of the Seismological Society of America, Vol. 90, p. S65–S76. Updated in 2012.

Pacific Earthquake Engineering Research (PEER) (2014). PEER NGA West 2 Ground Motion Database, available at [http://peer.berkeley.edu/ngawest2/databases/] extracted March 2014.

Petersen, M.D., Moschetti, Morgan P., Powers, P.M., Mueller, C.S., Hallar, K.M., Frankel, A.D., Zeng, Y., Rezaeian, S., Harmsen, S.C., Boyd, O.S., Field, N., Chen, R., Rukstales, K.S., Luco, N., Wheeler, R.L., Williams, R.A., and Olsen, A.H. (2014). Documentation for the 2014 update of the United States National Seismic Hazard Maps: U.S. Geological Survey Open File Report 2014-1091, 255 pp.

Petersen, M.D., Frankel, A.D., Harmsen, S.C., Mueller, C.S., Hallar, K.M., Wheeler, R.L., Wesson, R.L., Zeng, Y., Boyd, O.S., Perkins, D.M., Luco, N., Field, E.H., Wills, C.J., and Rukstales, K.S. (2008). Documentation for the 2008 update of the United States National Seismic Hazard Maps: U.S. Geological Survey Open File Report 2008-1128, 128 pp.

Powers, P.M., Rezaeian, S., Shumway, A.M., Petersen, M.D., Luco, N., Boyd, O.S., ... & Thompson, E.M. (2021). The 2018 update of the US National Seismic Hazard Model: Ground motion models in the western US. Earthquake Spectra, doi:10.1177/87552930211011200.

Shahi, S.K. and Baker, J.W. (2014). NGA-West2 Models for Ground Motion Directionality: Earthquake Spectra, vol. 30 No. 3, pp. 1285-1300.

Shaw, B.E. (2009). Constant stress drop from small to great earthquakes in magnitude-area scaling, Bull. Seismol. Soc. Am., Vol. 99, p. 871, 2009.

Southern California Earthquake Center (SCEC) (2016). Significant Earthquakes and Faults: Historical Earthquakes & Significant Faults in Southern CA, available at [http://www.data.scec.org/significant/index.html] accessed August 2016.

United States Geological Survey (USGS) (2018). Quaternary fault and fold database for the United States, accessed January 2019, available at [http//earthquakes.usgs.gov/regional/qfaults/].

Watson-Lamprey, J. (2018). Capturing Directivity Effects in the Mean and Aleatory Variability of the NGA-West 2 Ground Motion Prediction Equations, Pacific Earthquake Engineering Research Center PEER Report 2018/04.

Working Group on California Earthquake Probabilities (WGCEP) (2013a). Uniform California earthquake rupture forecast, Version 3 (UCERF3) – The time-independent model: U.S. Geological Survey Open-File Report 2013-1165, 97 pp., California Geological Survey Special Report 228, and Southern California Earthquake Center Publication 1792, available at [http://pubs.usgs.gov/of/2013/1165/].

WGCEP (2013b). "WGCEP Fault Sections and Models Application" available at [http://www.wgcep.org/tools-fault_db] (java app); FM3.1 and FM3.2 accessed July 2014.

TABLE D-1
CHARACTERIZATION[1] OF SIGNIFICANT FAULTS
940 N. SYCAMORE AVE.

Fault Name	Style of Faulting[2]	Maximum Magnitude (Mw)	Slip Rate (mm/yr)	Closest Rupture Distance From Site (km)
North Salt Lake	RV	5.8	0.1	0.5
Hollywood	OBL	6.5	1.3	1.8
San Vicente	RV	6.1	0.2	3.2
Puente Hills (LA)	RV	6.7	0.6	5.2
Elysian Park (Upper)	RV	6.5	1.4	5.8
Newport-Inglewood	SS	7.1	1.2	6.6
Puente Hills	RV	7.0	0.9	7.3
Santa Monica	OBL	6.7	1.1	7.3
Elysian Park (Lower CFM)	RV	6.8	0.1	10.8
Raymond	OBL	6.5	1.3	12
Verdugo	RV	6.8	0.6	12
San Pedro Escarpment	RV	7.1	0.2	13
Compton	RV	7.3	0.8	15
Malibu Coast	OBL	6.9	0.8	18
Northridge Hills	RV	6.8	1.0	19
Sierra Madre	RV	7.1	1.5	19
Puente Hills (Santa Fe Springs)	RV	6.4	0.8	20
Mission Hills	RV	6.3	0.8	21
Anacapa-Dume	OBL	7.1	0.7	21
Sierra Madre (San Fernando)	RV	6.5	1.6	21
Santa Susana East (connector)	RV	6.2	1.9	22
Northridge	RV	6.8	1.3	22
Palos Verdes	SS	7.4	2.3	24
San Gabriel (Extension)	SS	7.1	0.5	25
Santa Susana	RV	6.9	3.2	25
Santa Monica Bay	RV	6.8	0.1	26
San Gabriel	OBL	7.2	0.6	26
Elsinore - Whittier[3]	SS	7.0	4.2	29
Puente Hills (Coyote Hills)	RV	6.6	0.8	30
Redondo Canyon	RV	6.5	0.4	31
Clamshell-Sawpit	RV	6.4	0.3	33
Holser	RV	6.6	0.6	35
Anaheim	RV	6.2	0.1	35
Simi-Santa Rosa	OBL	6.8	1.1	40
San Pedro Basin	SS	7.1	1.1	41

Fault Name	Style of Faulting[2]	Maximum Magnitude (Mw)	Slip Rate (mm/yr)	Closest Rupture Distance From Site (km)
San Jose	OBL	6.5	0.3	43
Richfield	RV	6.1	0.2	44
Del Valle	RV	6.2	1.0	46
Oak Ridge (Onshore)	RV	7.1	2.6	46
Peralta Hills	RV	6.3	0.3	48
Yorba Linda	RV	6.3	0.1	49
Chino	OBL	6.7	0.9	51
Malibu Coast (Extension)	OBL	6.8	0.5	55
San Cayetano	RV	7.0	2.9	55
Cucamonga	RV	6.7	2	55
San Andreas[3]	SS	8.2	29.0	56
San Joaquin Hills	RV	6.8	0.5	58
Sisar	RV	6.8	0.8	64
Newport-Inglewood (Offshore)	SS	7.1	1.0	68
Fontana (Seismicity)	SS	6.6	0.3	70
Ventura-Pitas Point	OBL	7.1	1.5	71
Pine Mtn	RV	7.2	0.3	72
Santa Ynez (East)	SS	7.1	1.5	73
San Diego Trough North	SS	7.3	1.6	73
San Jacinto[3]	SS	7.9	6.0	75
Santa Cruz Catalina Ridge	OBL	7.4	1.1	75
Oceanside	RV	7.2	0.7	81
Cleghorn	SS	6.6	0.5	84
Channel Islands Thrust	RV	7.2	1.0	85
Santa Cruz Island	OBL	7.1	0.9	85
Mission Ridge-Arroyo Parida-Santa Ana	RV	7.0	1.1	86
Oak Ridge (Offshore)	RV	6.8	1.7	86
Red Mountain	RV	7.4	2.2	93
Garlock[3]	SS	7.4	3.6	97
Channel Islands Western Deep Ramp	RV	7.2	0.4	98
Big Pine (Central)	RV	6.3	0.4	99
San Clemente	SS	7.4	1.8	99
North Frontal	RV	7.1	0.3	102
Coronado Bank	SS	7.4	1.8	106

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 D-2
DETERMINISTIC SEISMIC HAZARD ANALYSIS FAULT CHARACTERIZATION
940 N. SYCAMORE 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	1.8	1.8	1.8
Elysian Park (Upper)	6.5	1	0	0	3.0	15.0	50	15.7	11.0	5.8	5.0	-5.0
Puente Hills (LA)	6.8	1	0	1	2.1	15.0	27	28.4	7.8	5.2	0	4.8
Puente Hills (Alt. 1)	7	1	0	1	5.0	13.0	25	18.9	10.2	7.3	0	0.0
Compton	7.3	1	0	1	5.2	15.0	20	28.7	9.4	14.5	0	28.0
Newport-Inglewood Onshore	7.4	0	0	0	0	15.0	90	15.0	10.2	6.6	6.6	6.6
Elsinore	7.8	0	0	0	0	15.4	90	15.4	10.2	29.0	29.0	29.0
San Andreas	8.2	0	0	0	0	13.1	90	13.1	10.2	55.5	55.5	55.5

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 D-3
PSHA SOURCE CONTRIBUTIONS
940 N. SYCAMORE AVE.

0.2-sec

Source	*43-yr*	*225-yr*	*975-yr*	*2,475-yr*
Hollywood	11%	24%	32%	37%
Elysian System	8%	13%	15%	14%
Puente Hills System	6%	8%	9%	10%
Santa Monica	6%	9%	9%	8%
Compton	3%	5%	6%	7%
Background	13%	8%	5%	5%
Newport Inglewood Onshore	3%	4%	4%	4%
San Vicente	1%	2%	3%	3%
Raymond	5%	5%	3%	2%
Santa Susana System	8%	4%	2%	1%
Sierra Madre System	6%	3%	2%	1%
San Andreas	6%	2%	1%	0%
Elsinore	1%	0%	0%	0%
San Jacinto	1%	0%	0%	0%
Others	22%	13%	9%	7%

2.0-sec

Source	*43-yr*	*225-yr*	*975-yr*	*2,475-yr*
Hollywood	8%	17%	25%	28%
Elysian System	6%	12%	15%	16%
Santa Monica	5%	9%	11%	12%
Puente Hills System	4%	7%	9%	10%
Newport Inglewood Onshore	2%	5%	7%	8%
Compton	2%	4%	6%	8%
San Andreas	17%	11%	6%	3%
Raymond	4%	4%	3%	2%
Sierra Madre System	5%	4%	2%	2%
Background	5%	3%	2%	1%
San Vicente	1%	1%	1%	1%
Santa Susana System	7%	3%	1%	1%
Elsinore	3%	1%	0%	0%
San Jacinto	5%	2%	0%	0%
Others	26%	18%	12%	7%

TABLE D-4
SITE-SPECIFIC HORIZONTAL MCE_R DEVELOPMENT CALCULATION SHEET
940 N. SYCAMORE 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 Detetermistic 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	0.998	0.980	0.989	0.897	1.190	1.055	1.023	1.218	0.717	1.218	0.946	**1.055**
0.02	50	1.009	0.996	1.003	0.897	1.190	1.070	1.039	1.236	0.727	1.236	1.090	**1.090**
0.03	33.33	1.068	1.052	1.060	0.897	1.190	1.131	1.087	1.294	0.761	1.294	1.234	**1.234**
0.05	20	1.288	1.254	1.271	0.897	1.190	1.357	1.271	1.512	0.890	1.512	1.522	**1.522**
0.075	13.33	1.639	1.574	1.607	0.897	1.190	1.715	1.553	1.848	1.088	1.848	1.881	**1.881**
0.1	10	1.916	1.831	1.874	0.897	1.190	2.000	1.795	2.136	1.257	2.136	2.005	**2.005**
0.15	6.67	2.237	2.154	2.195	0.897	1.200	2.363	2.117	2.540	1.495	2.540	2.005	**2.363**
0.2	5	2.400	2.332	2.366	0.897	1.210	2.568	2.320	2.808	1.652	2.808	2.005	**2.568**
0.25	4	2.471	2.413	2.442	0.897	1.220	2.672	2.434	2.970	1.748	2.970	2.005	**2.672**
0.3	3.33	2.469	2.424	2.446	0.897	1.220	2.677	2.507	3.059	1.800	3.059	2.005	**2.677**
0.4	2.5	2.233	2.215	2.224	0.897	1.230	2.454	2.355	2.896	1.704	2.896	2.005	**2.454**
0.5	2.00	2.031	2.009	2.020	0.897	1.230	2.228	2.135	2.626	1.545	2.626	1.678	**2.228**
0.75	1.33	1.558	1.540	1.549	0.897	1.240	1.723	1.636	2.029	1.194	2.029	1.119	**1.723**
1	1	1.202	1.198	1.200	0.897	1.240	1.335	1.260	1.562	0.919	1.562	0.839	**1.335**
1.5	0.67	0.740	0.762	0.751	0.897	1.240	0.835	0.798	0.990	0.582	0.990	0.559	**0.835**
2	0.5	0.509	0.535	0.522	0.897	1.240	0.580	0.549	0.681	0.401	0.681	0.419	**0.580**
3	0.33	0.299	0.326	0.312	0.897	1.250	0.350	0.332	0.415	0.244	0.415	0.280	**0.350**
4	0.25	0.194	0.222	0.208	0.897	1.260	0.235	0.215	0.271	0.160	0.271	0.210	**0.235**
5	0.2	0.140	0.165	0.152	0.897	1.260	0.172	0.159	0.200	0.118	0.200	0.168	**0.172**
7.5	0.13	0.077	0.095	0.086	0.897	1.280	0.098	0.081	0.103	0.061	0.103	0.112	**0.112**
10	0.1	0.050	0.062	0.056	0.897	1.290	0.065	0.048	0.061	0.036	0.061	0.067	**0.067**

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 D-5
SITE-SPECIFIC HORIZONTAL DRS DEVELOPMENT CALCULATION SHEET
940 N. SYCAMORE 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	0.788	0.631	0.704	**0.704**
0.02	50	0.908	0.727	0.727	**0.727**
0.03	33.33	1.028	0.823	0.823	**0.823**
0.05	20	1.268	1.014	1.014	**1.014**
0.075	13.33	1.567	1.254	1.254	**1.254**
0.1	10	1.671	1.337	1.337	**1.337**
0.15	6.67	1.671	1.337	1.575	**1.575**
0.2	5	1.671	1.337	1.712	**1.712**
0.25	4	1.671	1.337	1.782	**1.782**
0.3	3.33	1.671	1.337	1.785	**1.785**
0.4	2.5	1.671	1.337	1.636	**1.636**
0.5	2.00	1.398	1.119	1.486	**1.486**
0.75	1.33	0.932	0.746	1.149	**1.149**
1	1	0.699	0.559	0.890	**0.890**
1.5	0.67	0.466	0.373	0.557	**0.557**
2	0.5	0.350	0.280	0.387	**0.387**
3	0.33	0.233	0.186	0.233	**0.233**
4	0.25	0.175	0.140	0.157	**0.157**
5	0.2	0.140	0.112	0.115	**0.115**
7.5	0.13	0.093	0.075	0.075	**0.075**
10	0.1	0.056	0.045	0.045	**0.045**

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.

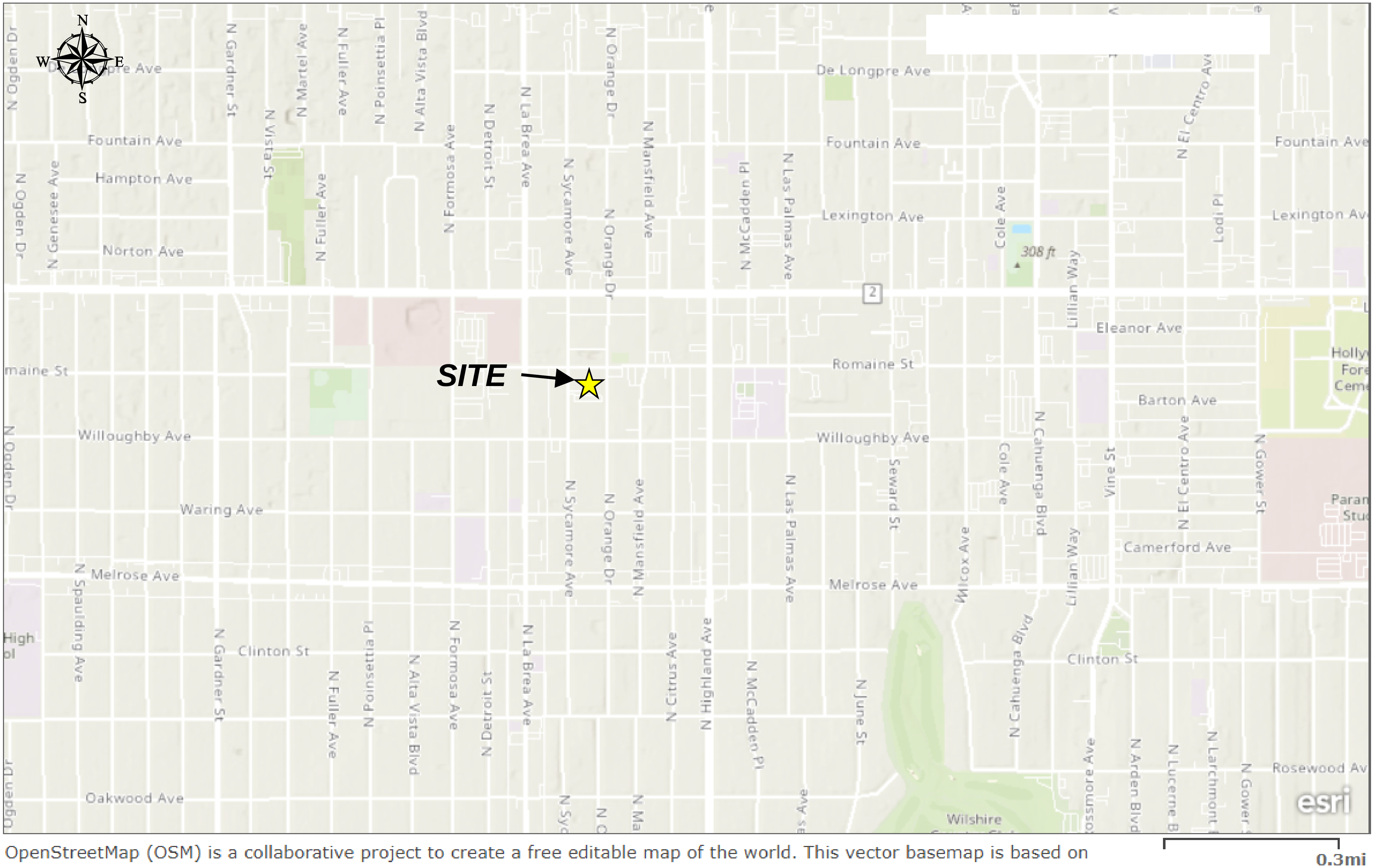
TABLE D-6
SITE-SPECIFIC HORIZONTAL SLE DEVELOPMENT CALCULATION SHEET
940 N. SYCAMORE AVE.

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 @ 2.57% Damping*
Period	*Frequency*	*RotD50*	*RotD50*	*RotD50*		*RotD50*
(sec)	(Hz)	(g)	(g)	(g)	-	(g)
0.01	100	0.181	0.142	0.162	1.000	0.162
0.02	50	0.183	0.144	0.163	1.005	0.164
0.03	33.33	0.194	0.152	0.173	1.022	0.177
0.05	20	0.233	0.183	0.208	1.071	0.223
0.075	13.33	0.296	0.234	0.265	1.130	0.300
0.1	10	0.351	0.280	0.315	1.175	0.370
0.15	6.67	0.414	0.332	0.373	1.210	0.452
0.2	5	0.430	0.343	0.386	1.226	0.474
0.25	4	0.421	0.333	0.377	1.226	0.463
0.3	3.33	0.400	0.316	0.358	1.231	0.441
0.4	2.5	0.344	0.270	0.307	1.233	0.378
0.5	2.00	0.297	0.233	0.265	1.233	0.327
0.75	1.33	0.207	0.162	0.184	1.228	0.226
1	1	0.152	0.117	0.134	1.226	0.165
1.5	0.67	0.094	0.072	0.083	1.223	0.101
2	0.5	0.065	0.049	0.057	1.214	0.069
3	0.33	0.038	0.028	0.033	1.211	0.040
4	0.25	0.026	0.019	0.022	1.200	0.027
5	0.2	0.018	0.013	0.016	1.195	0.019
7.5	0.13	0.010	0.007	0.008	1.176	0.010
10	0.1	0.006	0.004	0.005	1.131	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).



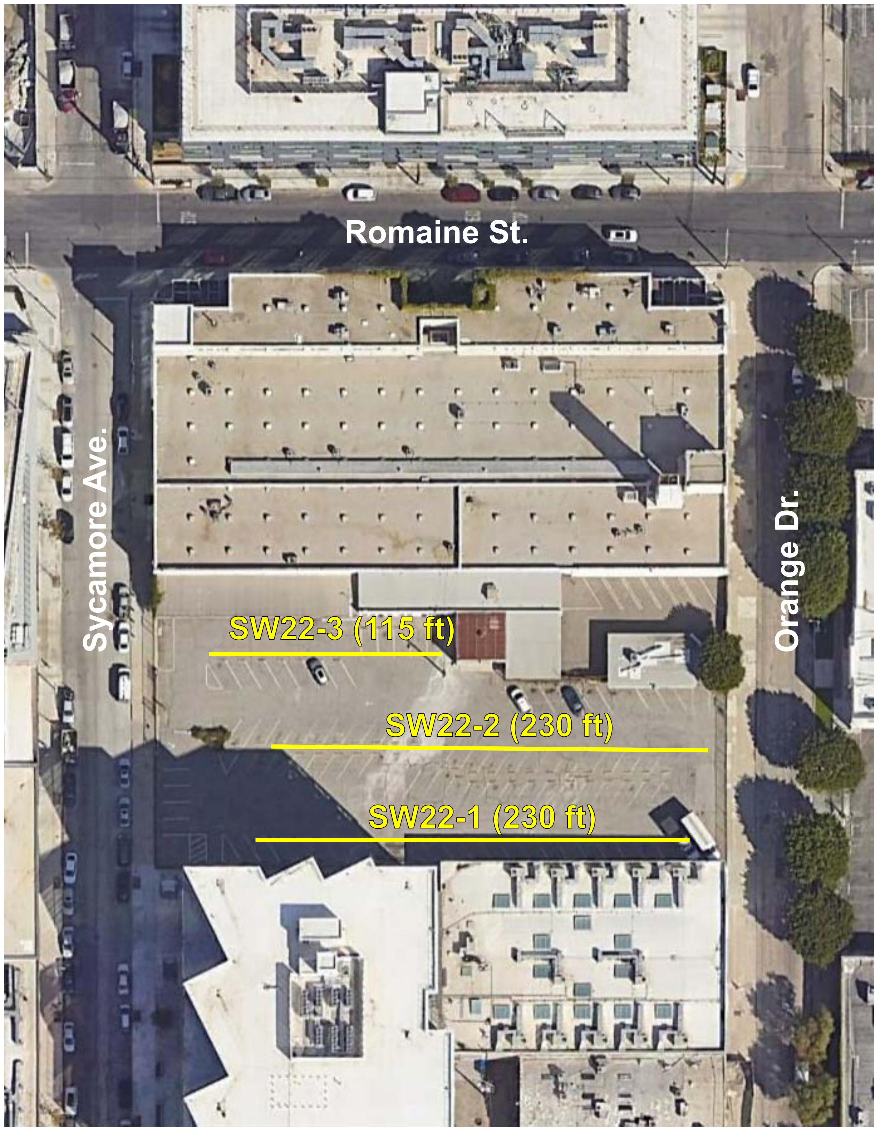
OpenStreetMap (OSM) is a collaborative project to create a free editable map of the world. This vector basemap is based on the Daylight map distribution of OSM data and is hosted by Esri. The vector web map is presented in OSM cartography.

Esri, NASA, NGA, USGS, FEMA | Esri Community Maps Contributors, County of Los Angeles, California State Parks, Esri, HERE, Garmin, SafeGraph, GeoTechnologies, Inc, METI/NASA, USGS, Bureau of Land Management, EPA, NPS, US Census Bureau, USDA

SITE LOCATION MAP

Date: MAR 2022	Project No.: 21106A	Project: 940 N. SYCAMORE AVE.	Figure D-1

GeoPentech



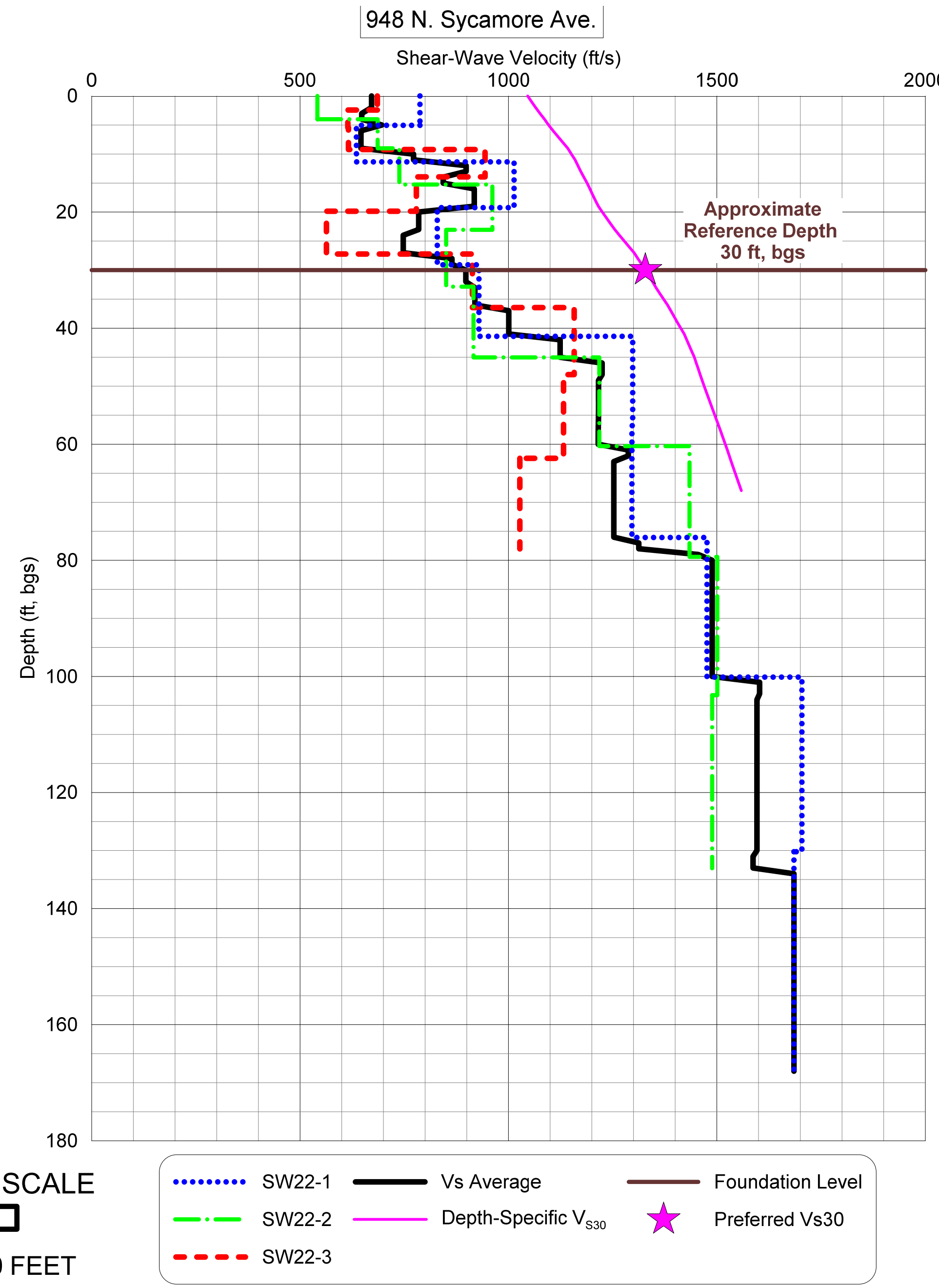
LEGEND

Location of Surface Wave Geophysical Measurement

N

APPROXIMATE SCALE

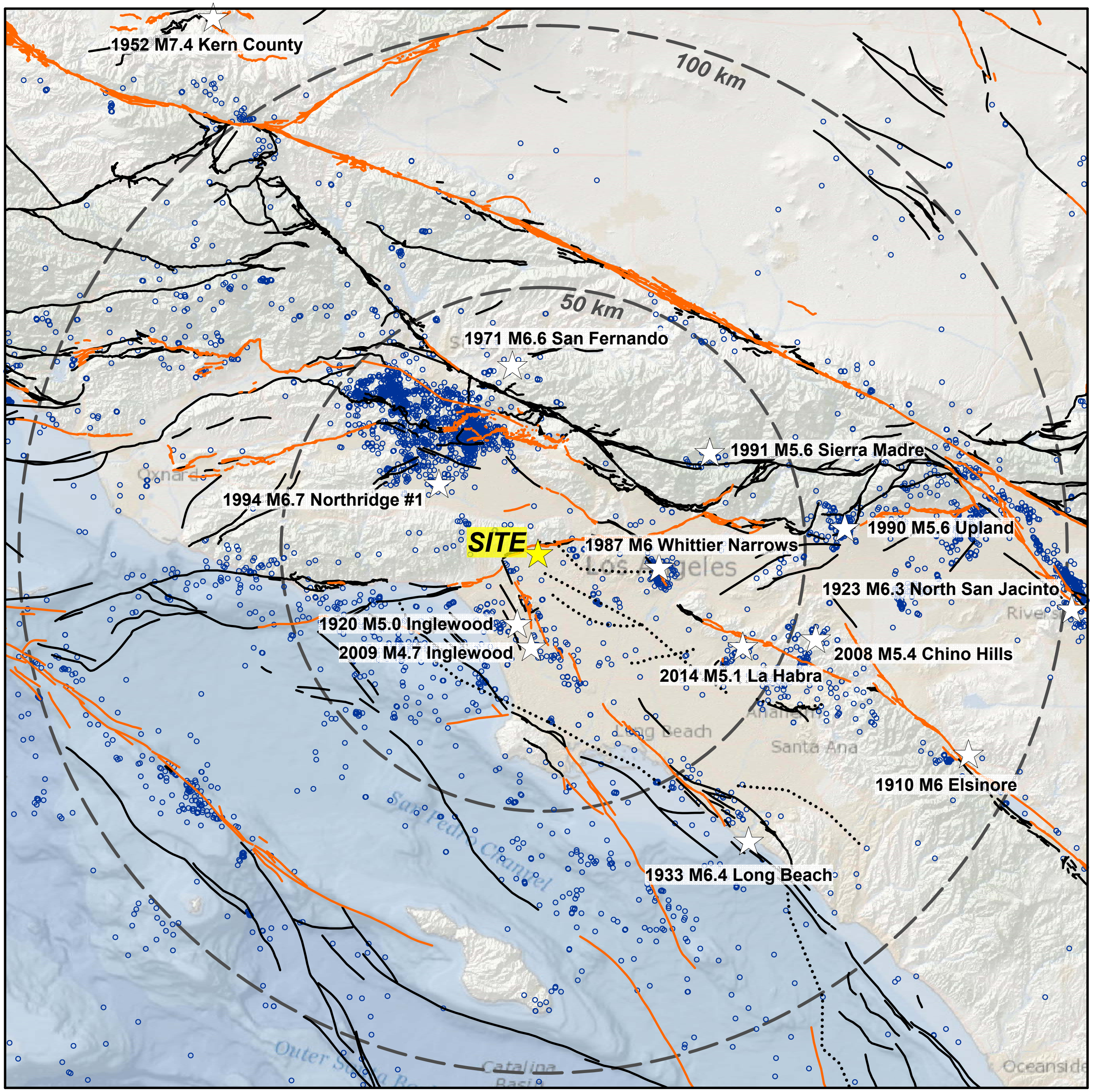
0 100 FEET



SHEAR-WAVE VELOCITY PROFILE LOCATIONS AND RESULTS

Date: APR 2022	Project No.: 21106A	Project: 940 N. SYCAMORE AVE.	Figure D-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

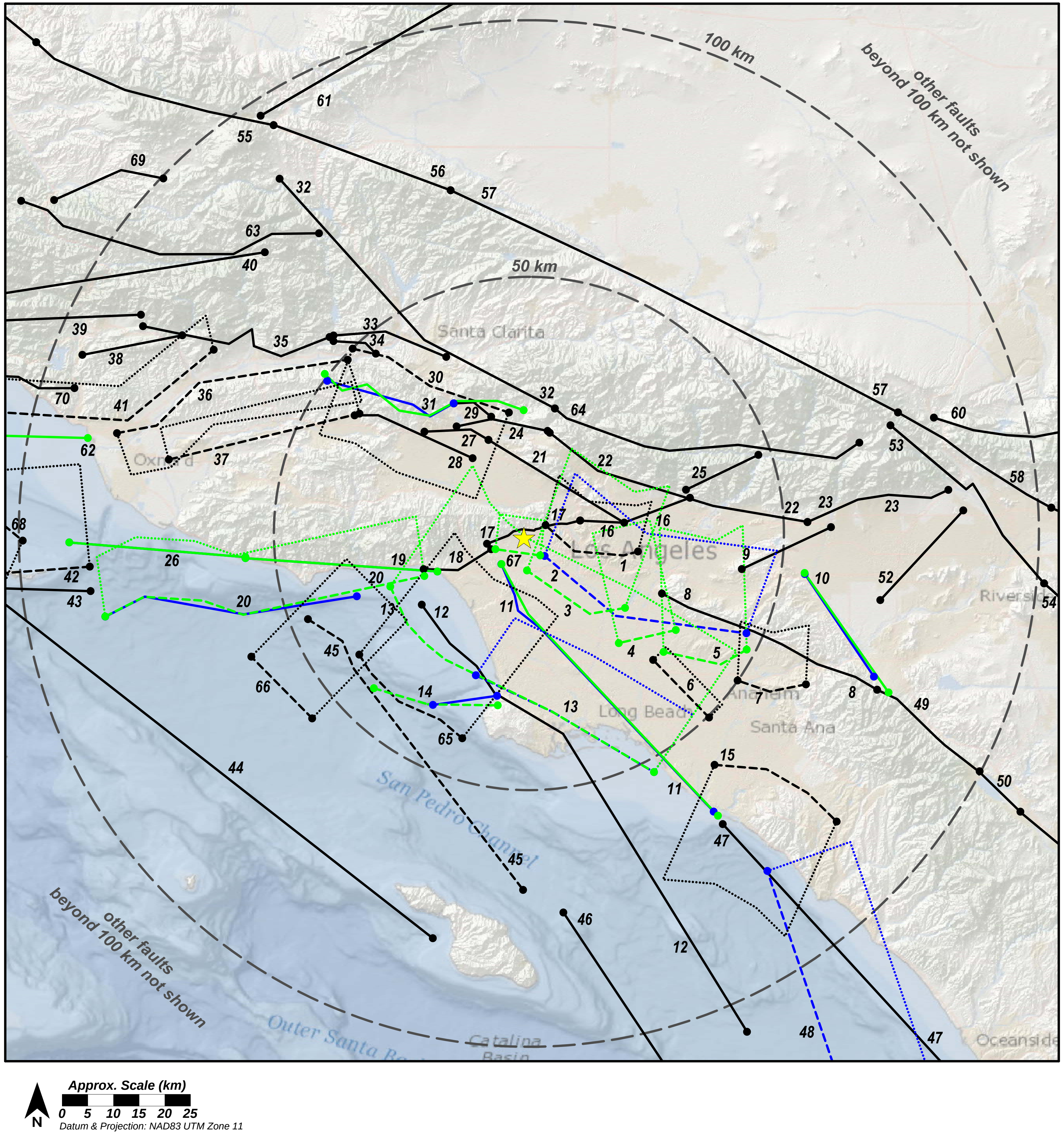
0 km 20 km 40 km 60 km

Datum & Projection: NAD83 UTM Zone 11

REGIONAL FAULT & SEISMICITY MAP

Date: MAR 2022	Project No.: 21106A	Project: 940 N. SYCAMORE AVE.	Figure D-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

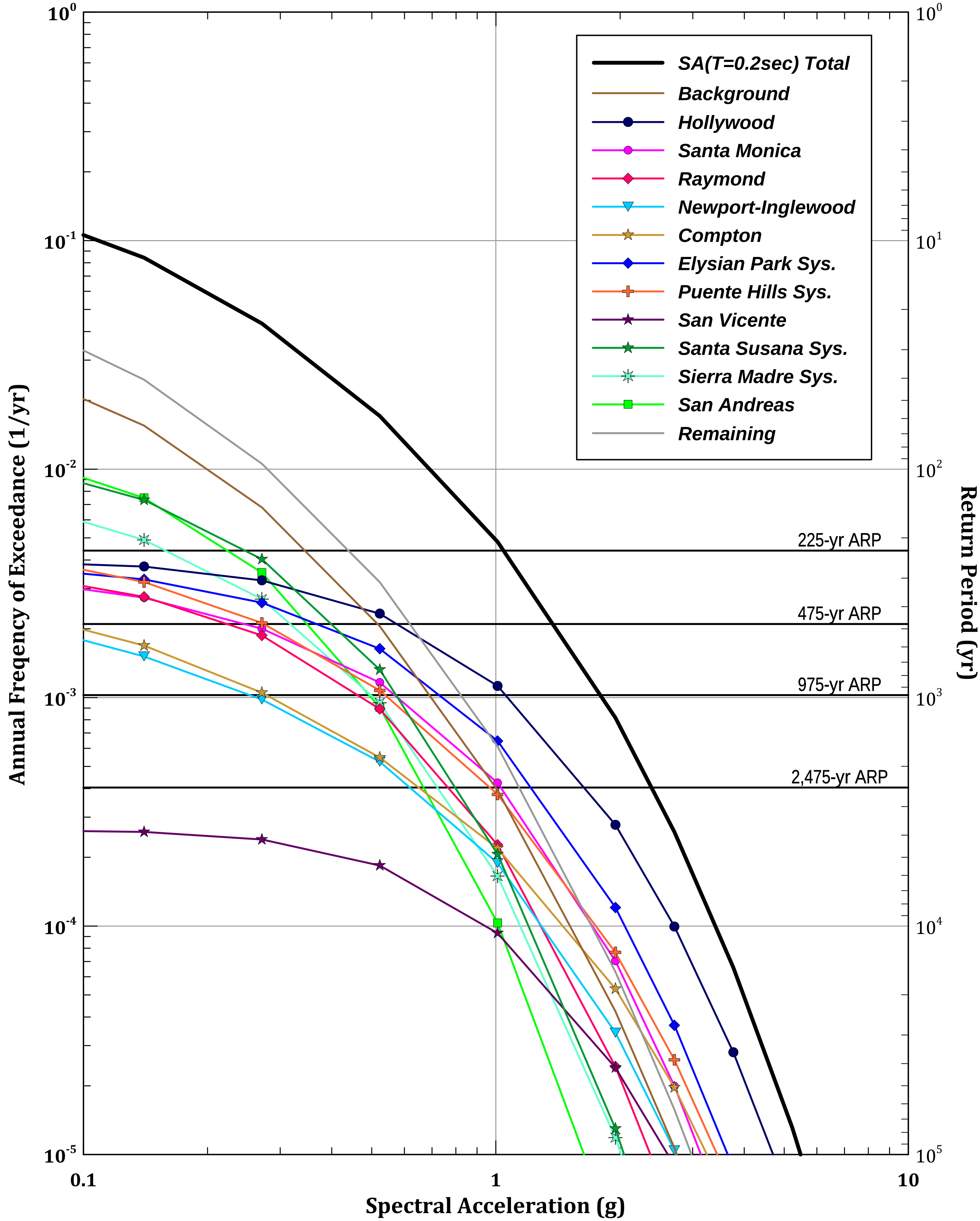
Surface Trace, Top of Fault (Both Fault Models)
Blind Trace, Top of Fault (Both Fault Models)
Blind Thrust Footprint (Both Fault Models)
Surface Trace, Top of Fault (Fault Model 1)
Blind Trace, Top of Fault (Fault Model 1)
Blind Thrust Footprint (Fault Model 1)
Surface Trace, Top of Fault (Fault Model 2)
Blind Trace, Top of Fault (Fault Model 2)
Blind Thrust Footprint (Fault Model 2)
Site

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		Figure D-3b
Project: 940 N. SYCAMORE AVE.		
Project No.: 21106A	Date: MAR 2022	

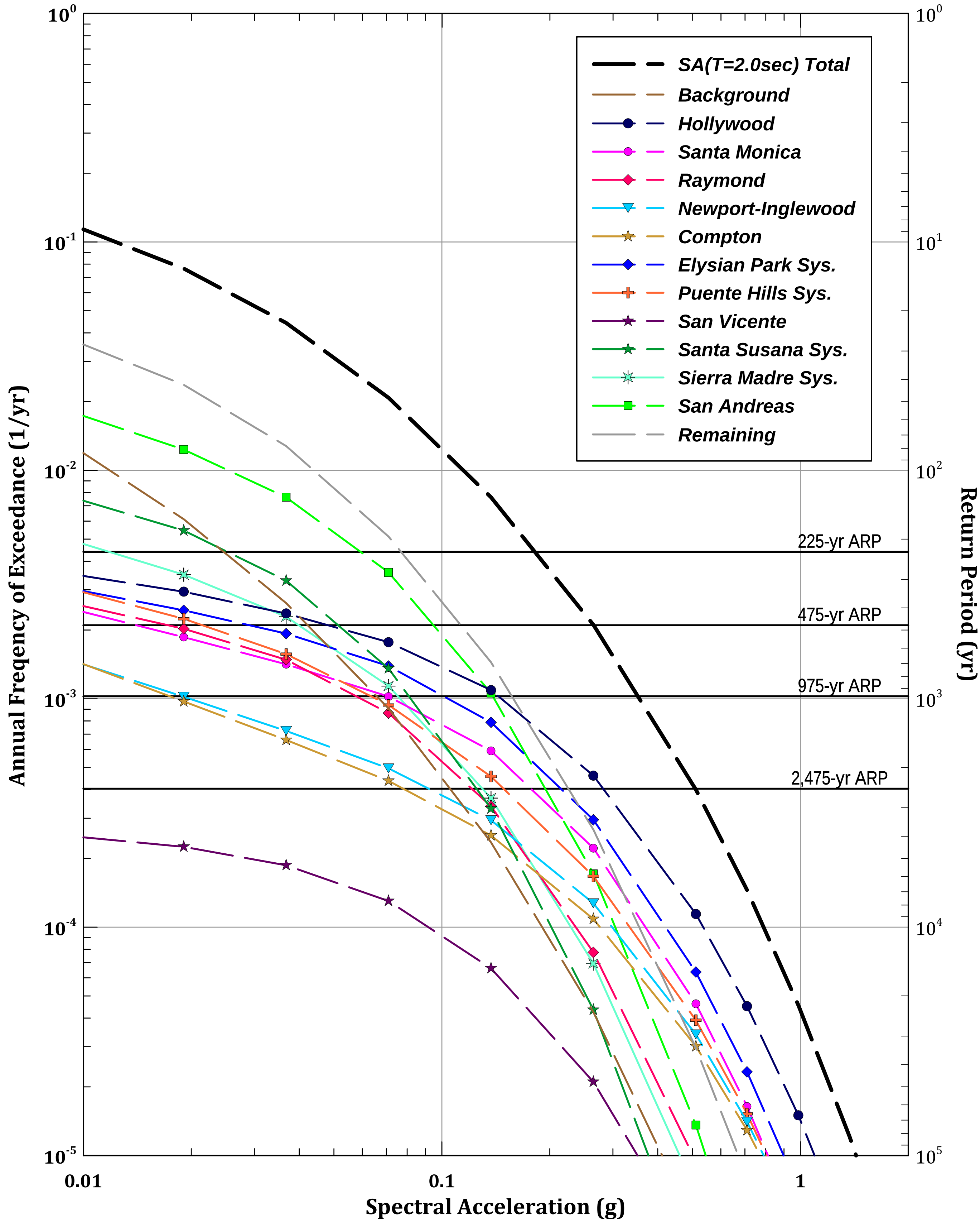
GeoPentech



SOURCE CONTRIBUTIONS TO 0.2-SECOND PERIOD

Project No.: 21106A	Project: 940 N. SYCAMORE AVE.	Date: APR 2022	Figure D-4a

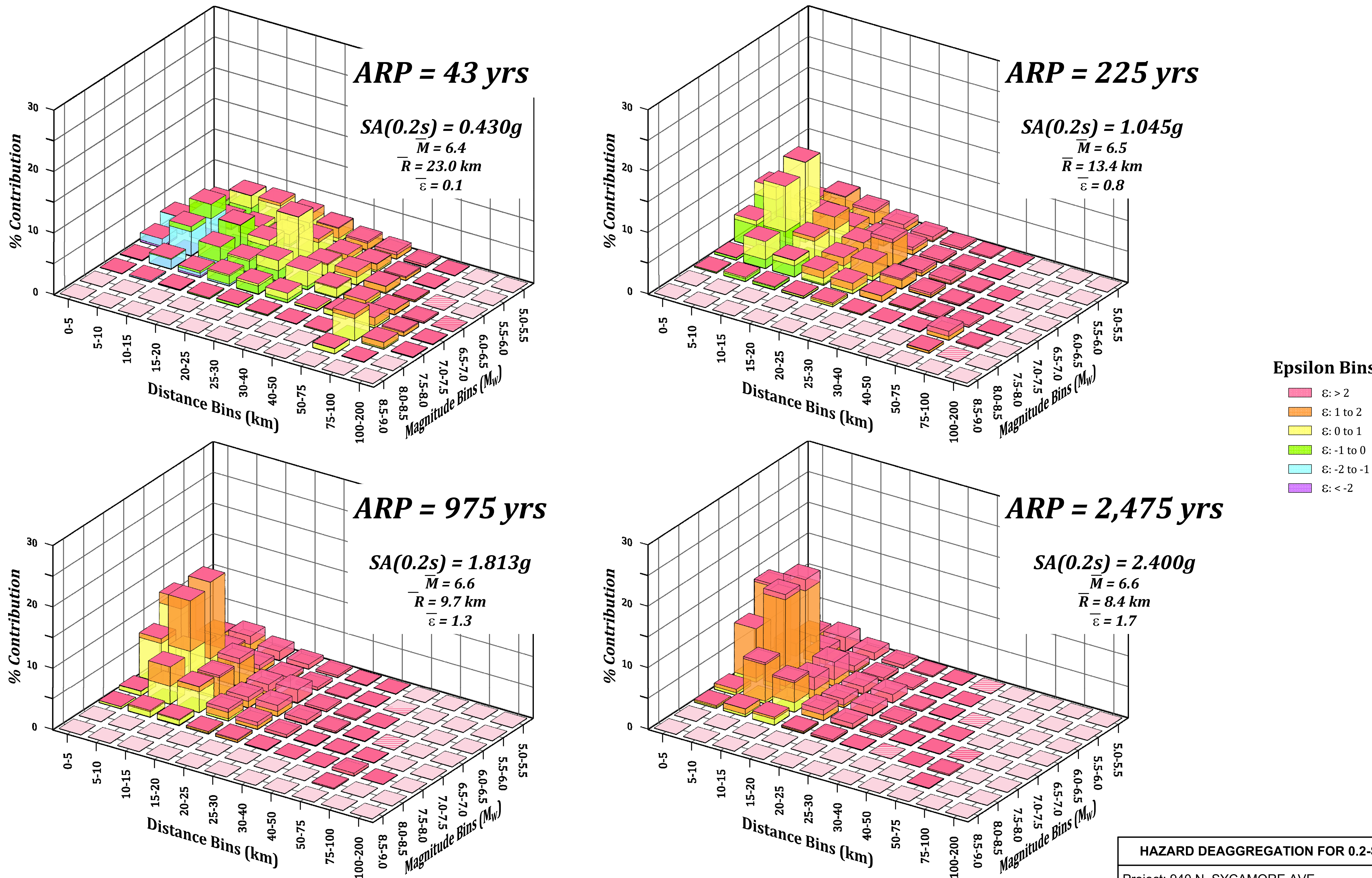
GeoPentech



SOURCE CONTRIBUTIONS TO 2.0-SECOND PERIOD

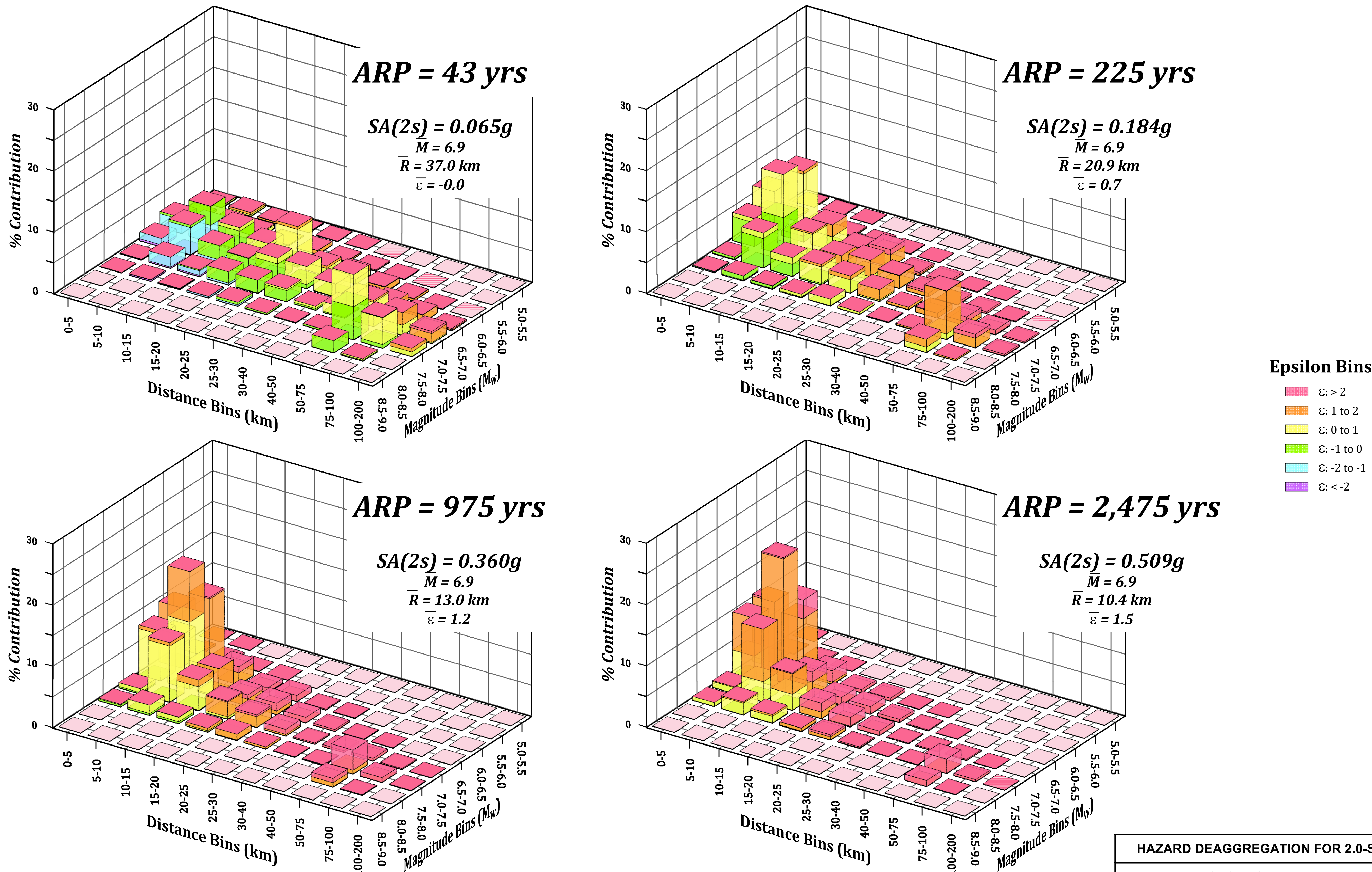
Project No.: 21106A	Project: 940 N. SYCAMORE AVE.	Date: APR 2022	Figure D-4b

GeoPentech



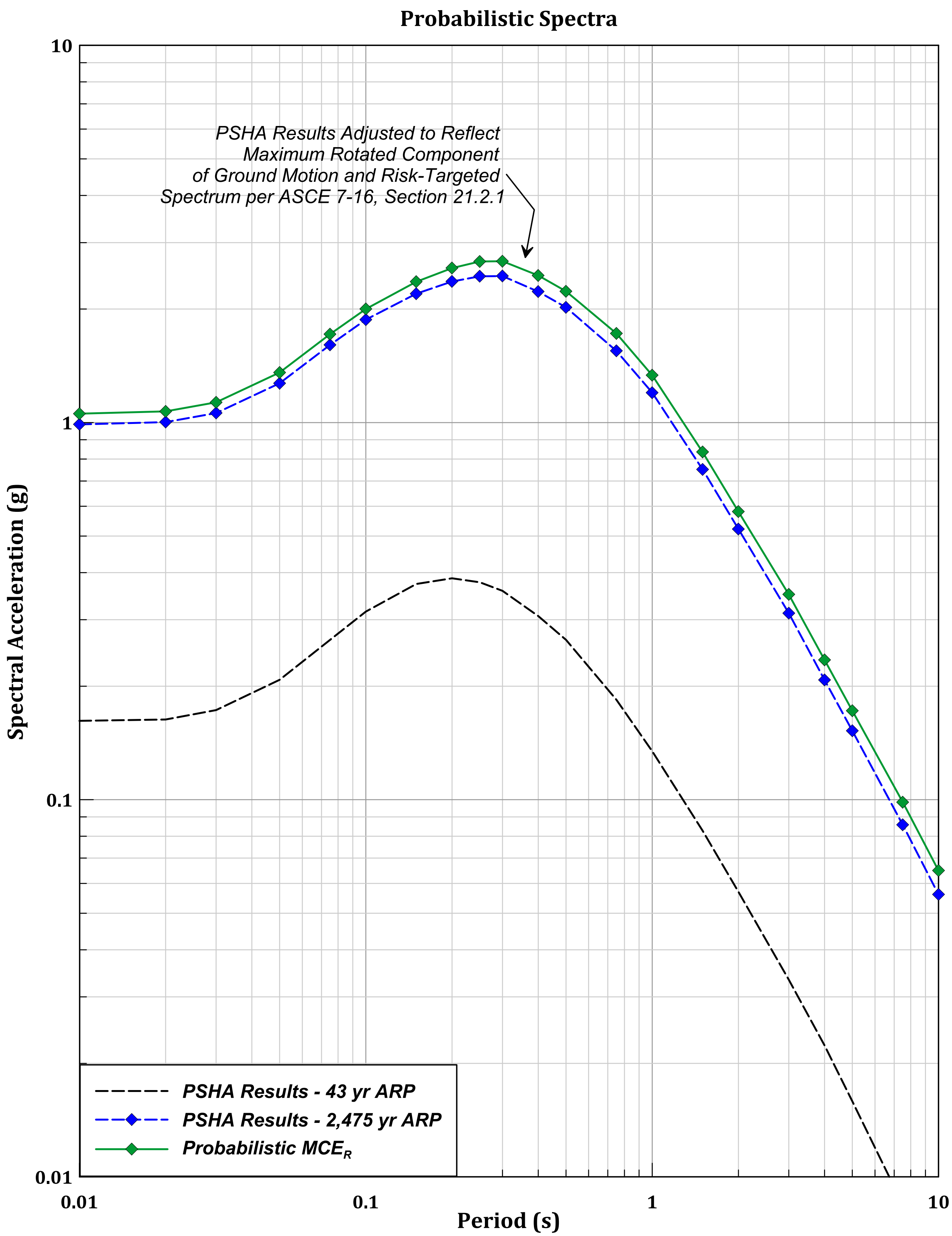
HAZARD DEAGGREGATION FOR 0.2-SEC PERIOD		
Project: 940 N. SYCAMORE AVE.		Figure D-5a
Project No.: 21106A	Date: APR 2022	

GeoPentech



HAZARD DEAGGREGATION FOR 2.0-SEC PERIOD		
Project: 940 N. SYCAMORE AVE.		Figure D-5b
Project No.: 21106A	Date: APR 2022	

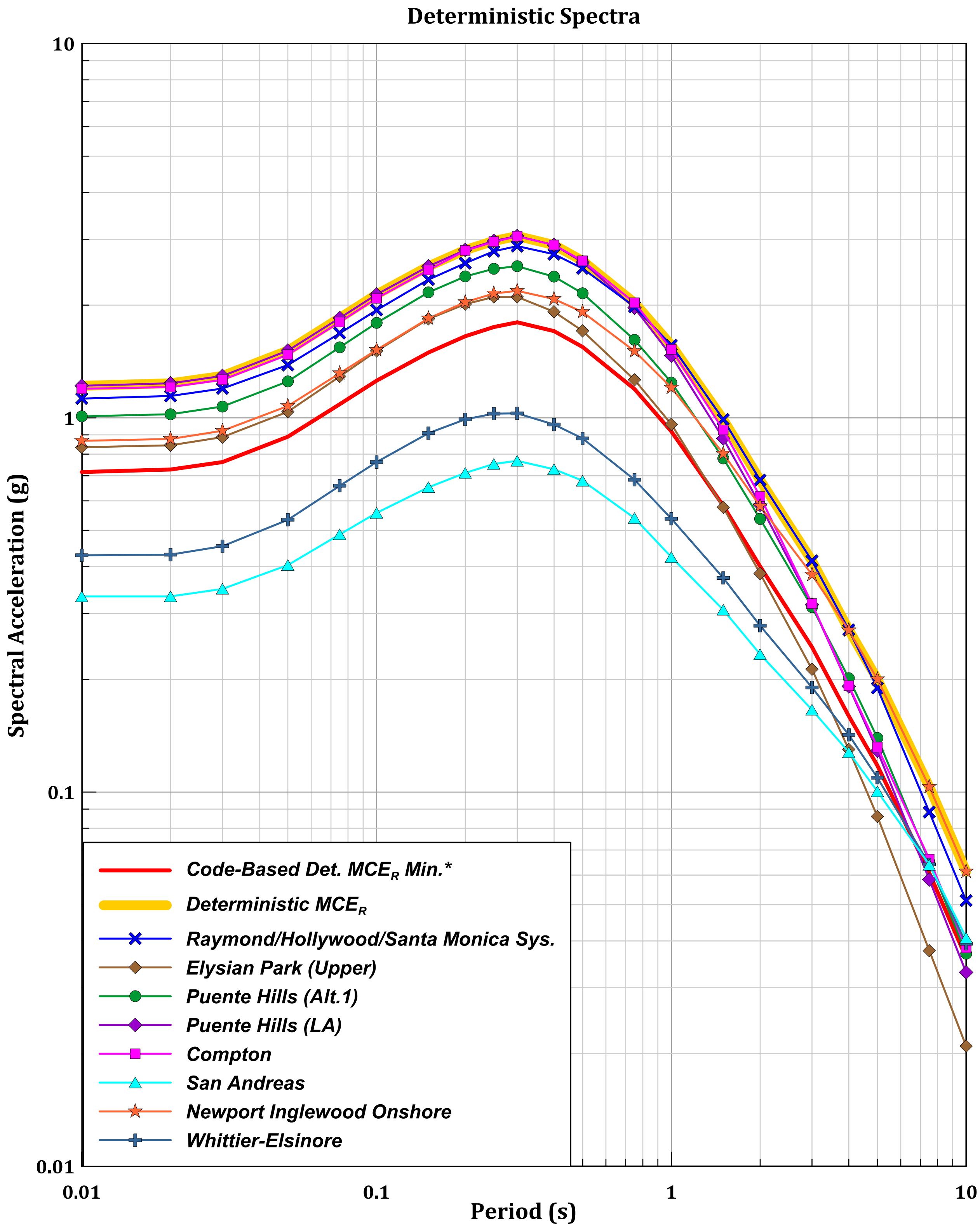
GeoPentech



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

PROBABILISTIC SPECTRA			
Project No.: 21106A	Project: 940 N. SYCAMORE AVE.	Date: APR 2022	Figure D-6

GeoPentech

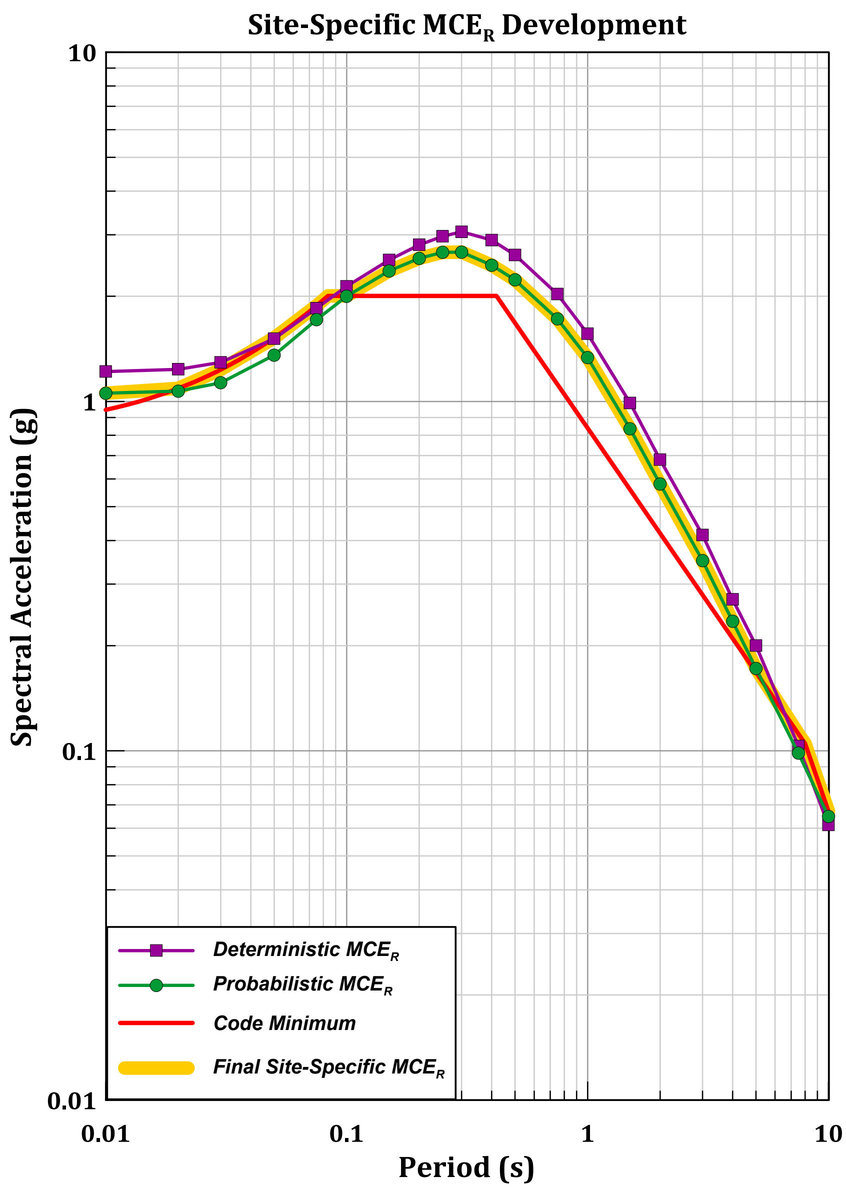


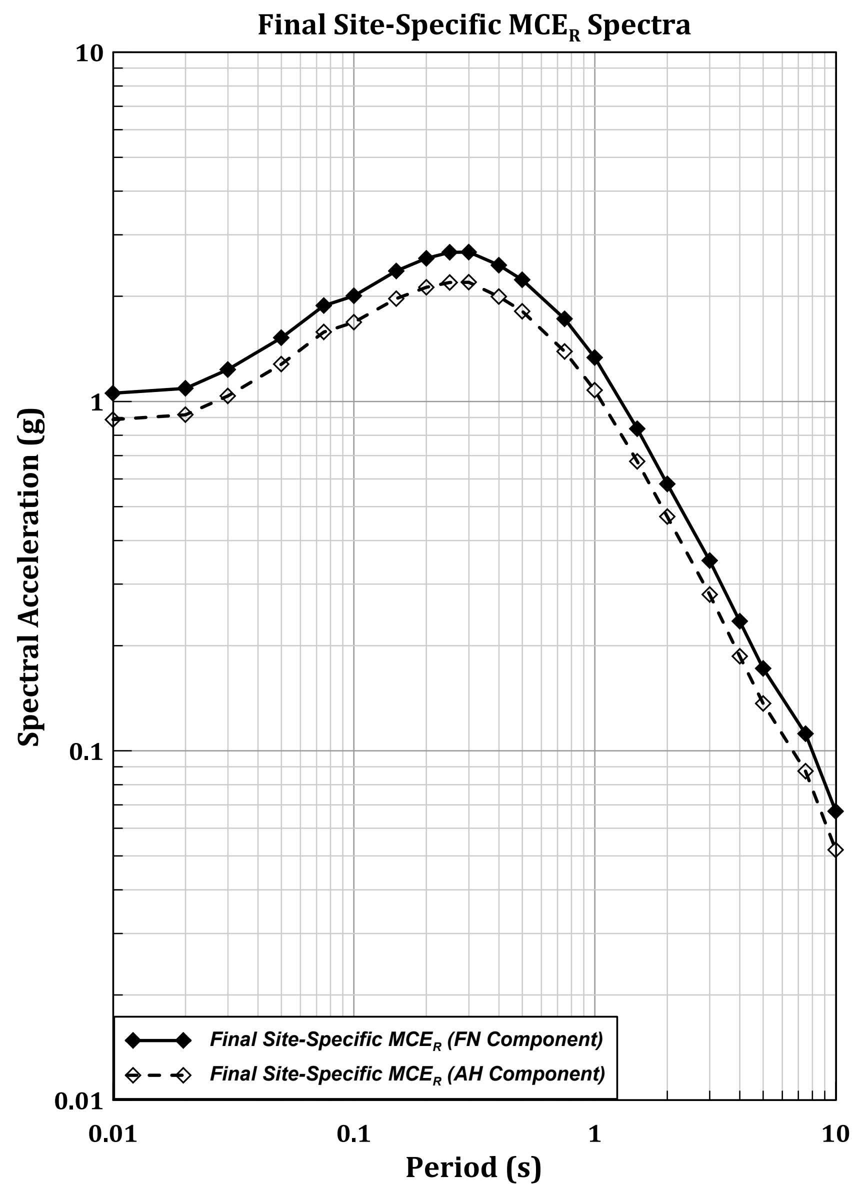
Notes: All spectra are for Damping (β) = 5.0% unless otherwise indicated.
Scenario spectra are 84th %tile maximum rotated ordinates.
Code-Based Det. MCE Min. per ASCE 7-16, Supplement 1, Ch. 21.2.2.

DETERMINISTIC SPECTRA

Project No.: 21106A	Project: 940 N. SYCAMORE AVE.	Date: APR 2022	Figure D-7

GeoPentech



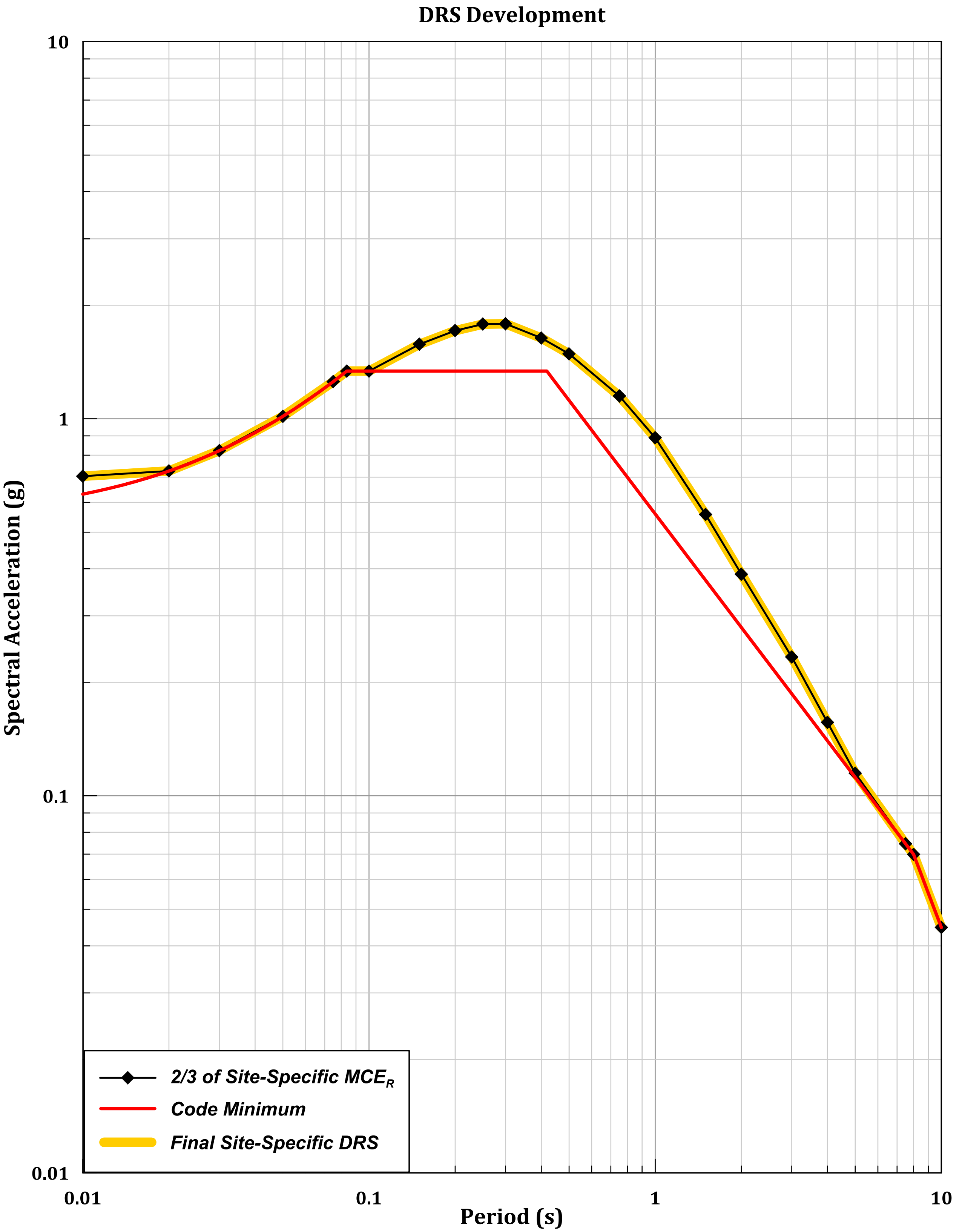


Note: All spectra are for Damping (β) = 5.0% unless otherwise noted.

FINAL SITE-SPECIFIC MCE$_R$ SPECTRA

Date: APR 2022	Project No.: 21106A	Project: 940 N. SYCAMORE AVE.	Figure D-8

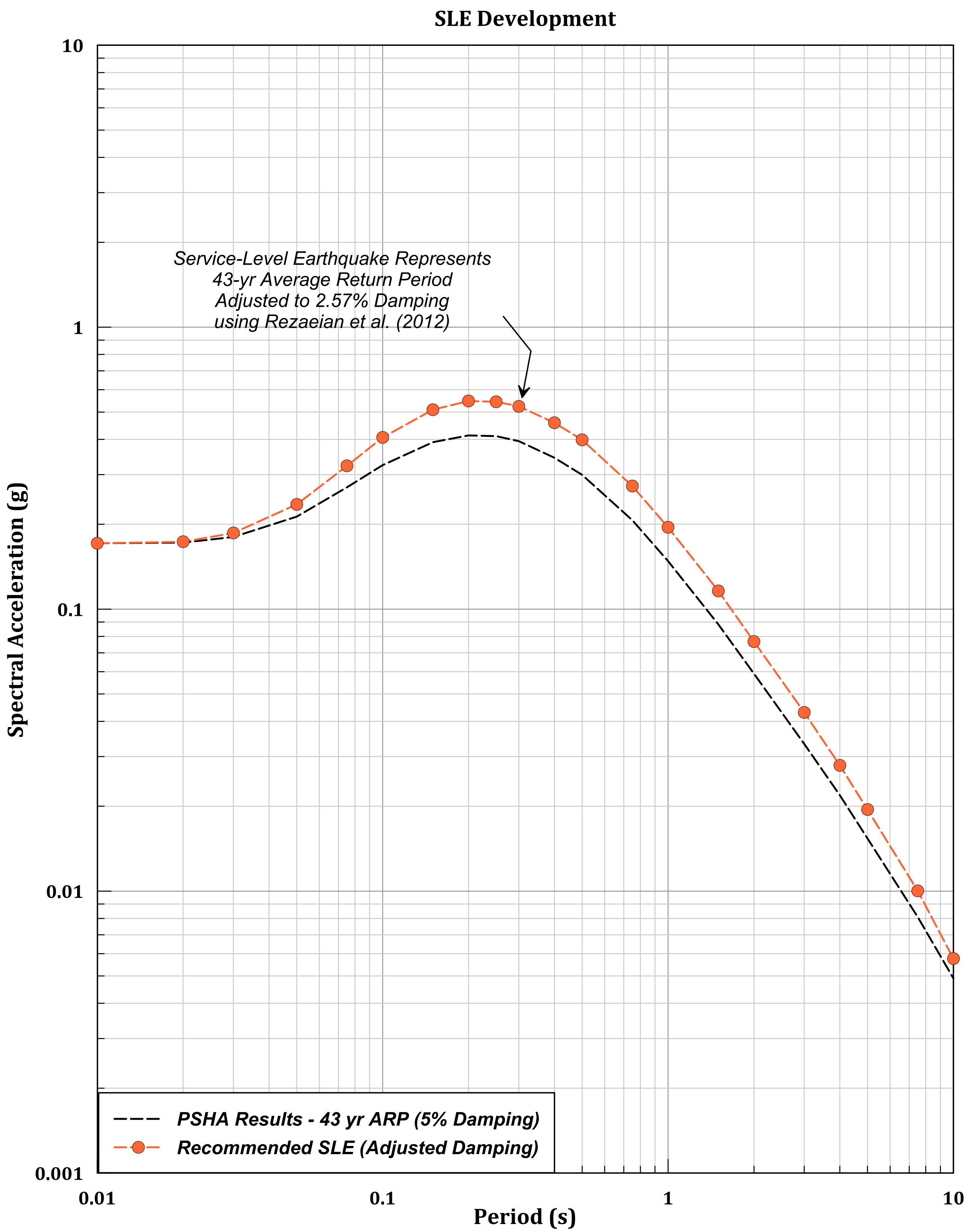
GeoPentech



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

SITE-SPECIFIC DRS DEVELOPMENT			
Project No.: 21106A	Project: 940 N. SYCAMORE AVE.	Date: APR 2022	Figure D-9

GeoPentech



SITE-SPECIFIC SLE DEVELOPMENT

Project No.: 21106A	Project: 940 N. SYCAMORE AVE.	Date: APR 2022	Figure D-10

GeoPentech

APPENDIX E

SETTLEMENT ANALYSIS

E.1 ANALYSIS APPROACH AND RESULTS

Investigations at the site, shown in Figure E-1, encountered formational material susceptible to consolidation settlement. The objective of the analysis documented in this appendix is to evaluate the potential settlement under the proposed structure loading. 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 proposed structure is considered to be 6 ksf applied at the bottom of a 67 ft excavation.

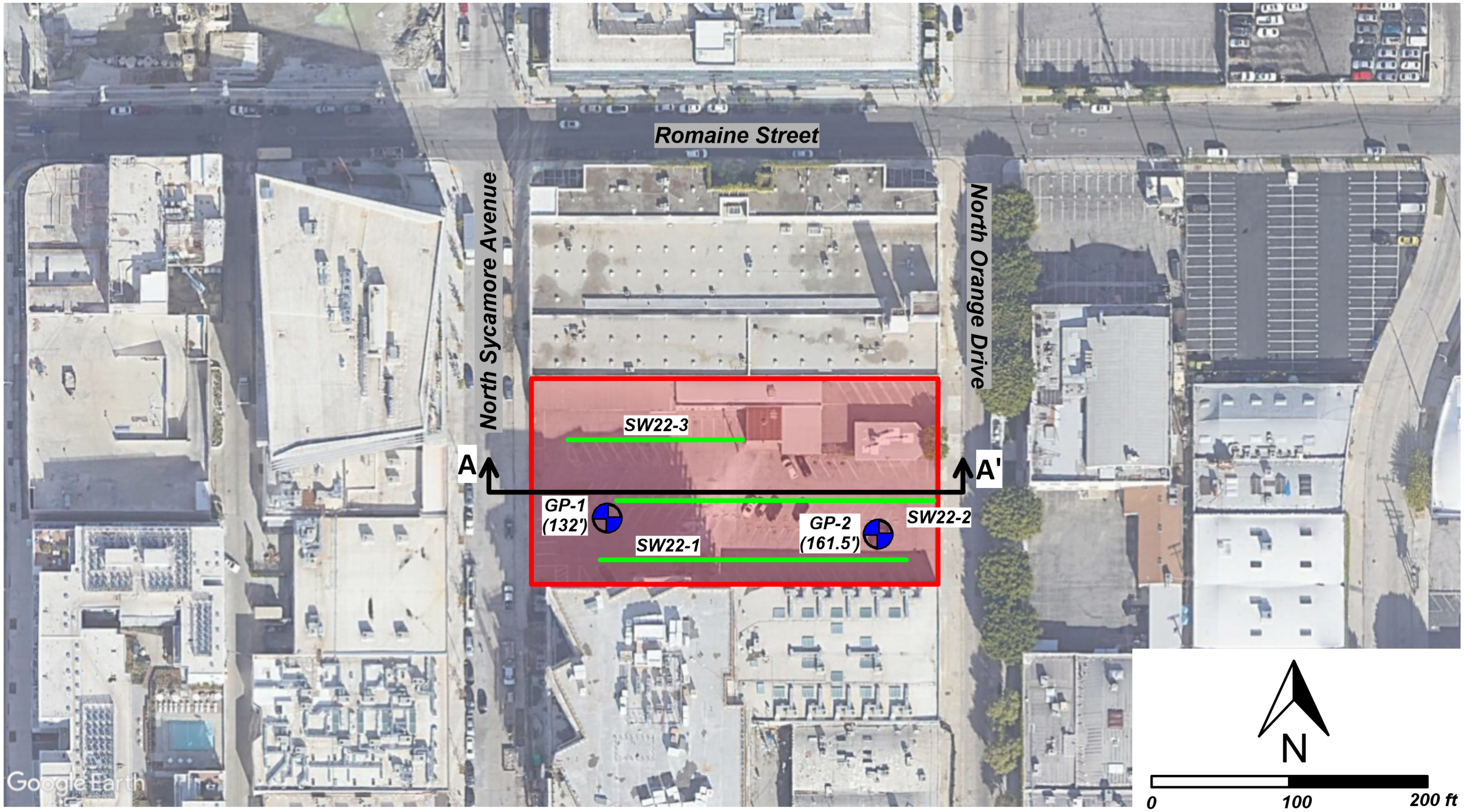
Consolidation Settlements

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 based on the Casagrande method for use in the analysis.

For settlement analyses, an idealized soil profile was developed based on the subsurface units, shown in Figure E-2 to represent the range of varying conditions beneath the building footprint. The material properties for the idealized soil profile are tabulated in Figure E-3.

Figure E-4 shows plan and isometric views of the analyzed Settle 3D model, and graphically presents the resulting settlements at various locations in the model (query points A and B). 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. Furthermore, our model calculates consolidation rebound (due to excavation) and settlements (due to structural loads) in an ultimate manner.

Based on the analysis results, we estimate that for the proposed structure loading (i.e. average uniform 6,000 psf), the total consolidation induced settlement (total elastic and primary consolidation settlement) would be on the order of about 3 inches.



Explanations

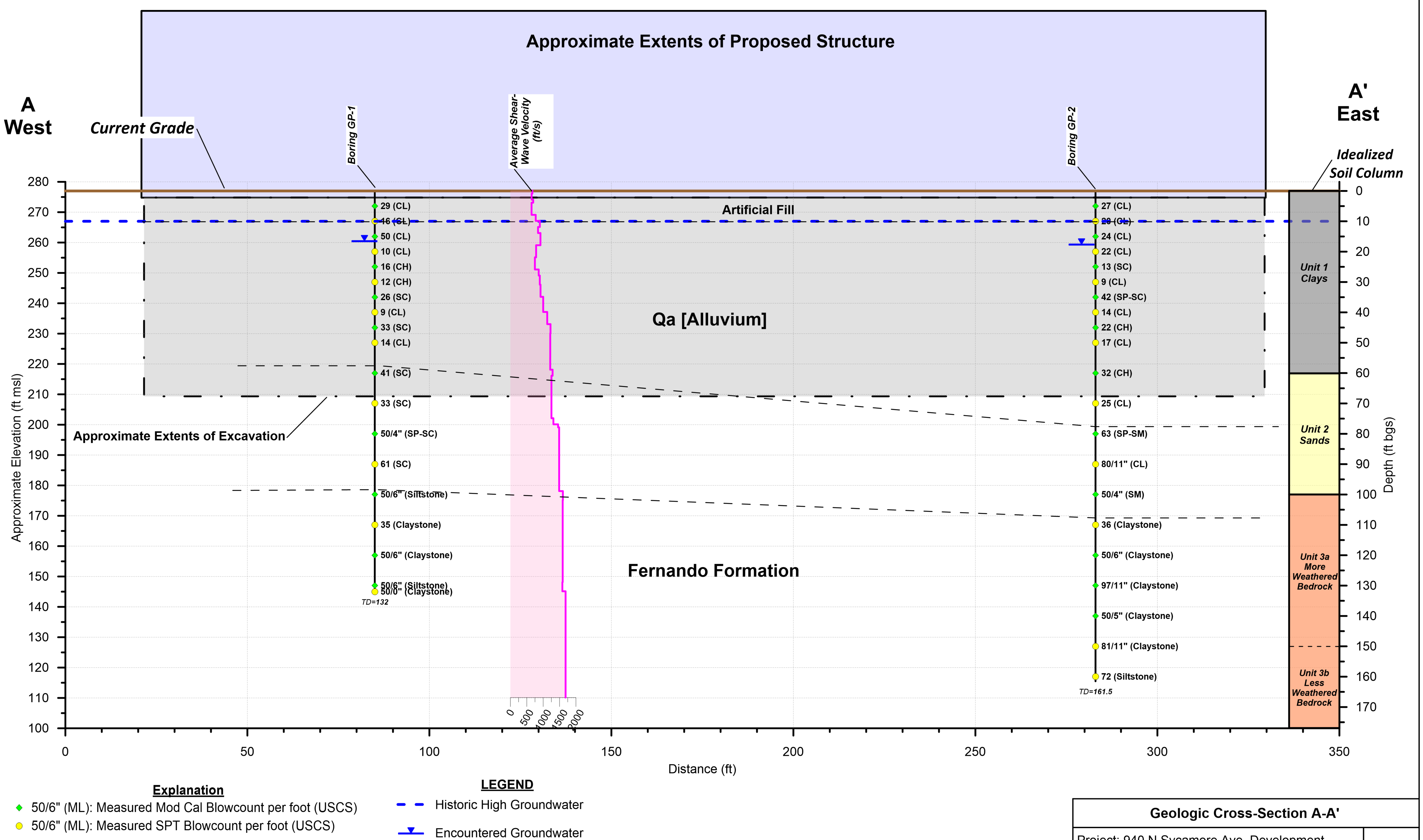
	Approximate Footprint of Proposed Development
GP-1 (Total Depth)	Boring Locations
SW22-1	Geophysical Survey Lines
	Cross Section

SITE PLAN

Date: APR 2022	Project No.: 21106A	Project: 940 N Sycamore Ave. Development	Figure E-1

GeoPentech

Geologic Cross-Section A-A'



Geologic Cross-Section A-A'		
Project: 940 N Sycamore Ave. Development		Figure E-2
Project No.: 21106A	Date: APR 2022	

GeoPentech

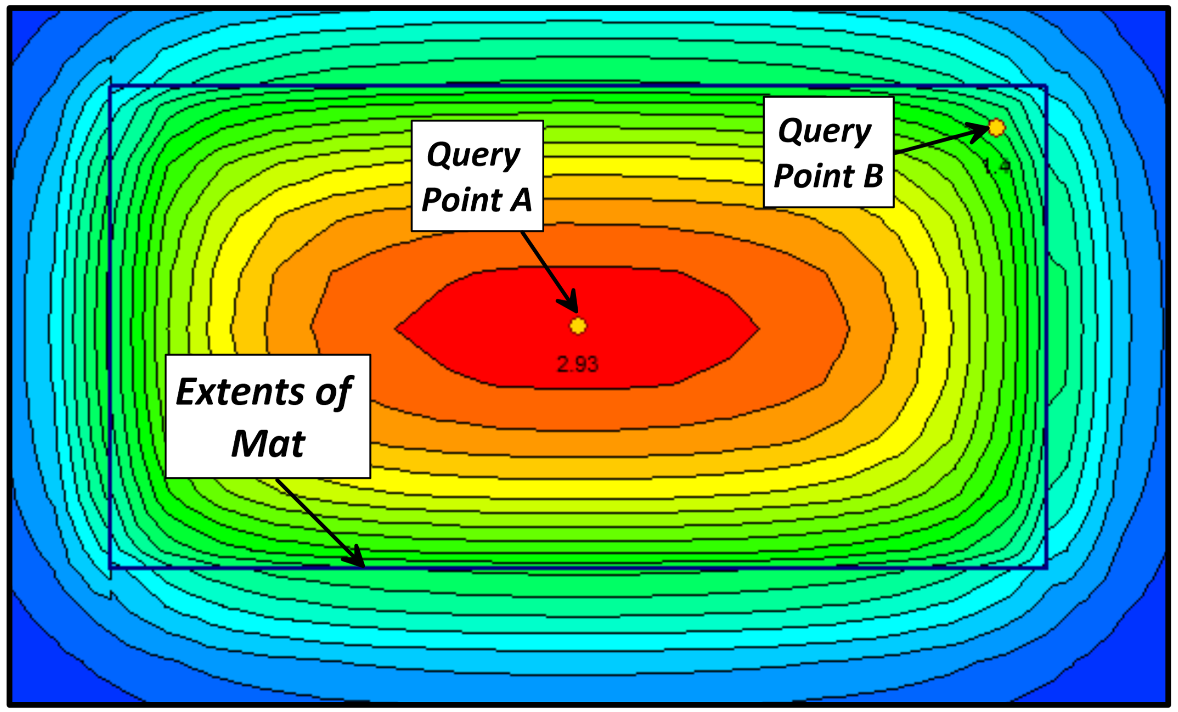
Interpreted Consolidation Laboratory Test Results

Borehole ID	Sample No.	Sample Type	Depth (ft)	USCS	C_{ce}	C_{re}	σ'_p (psf)	σ'_{v0} (psf)	OCR	e_0
GP-1	7	Mod Cal	35	SC	0.081	0.009 0.013	4500	3422	1.3	0.63
GP-1	15	Mod Cal	100	Siltstone	0.058	0.013	6200	7461	1.0	0.61
GP-2	9	Mod Cal	45	CH	0.118	0.015 0.030	8100	3866	2.1	0.84
GP-2	11	Mod Cal	60	CH	0.103	0.025	9100	5108	1.8	0.52
GP-2	17	Mod Cal	120	Claystone	0.061	0.010 0.021	10500	8680	1.2	0.75
GP-2	19	Mod Cal	140	Claystone	0.056	0.015	9000	10233	1.0	0.59

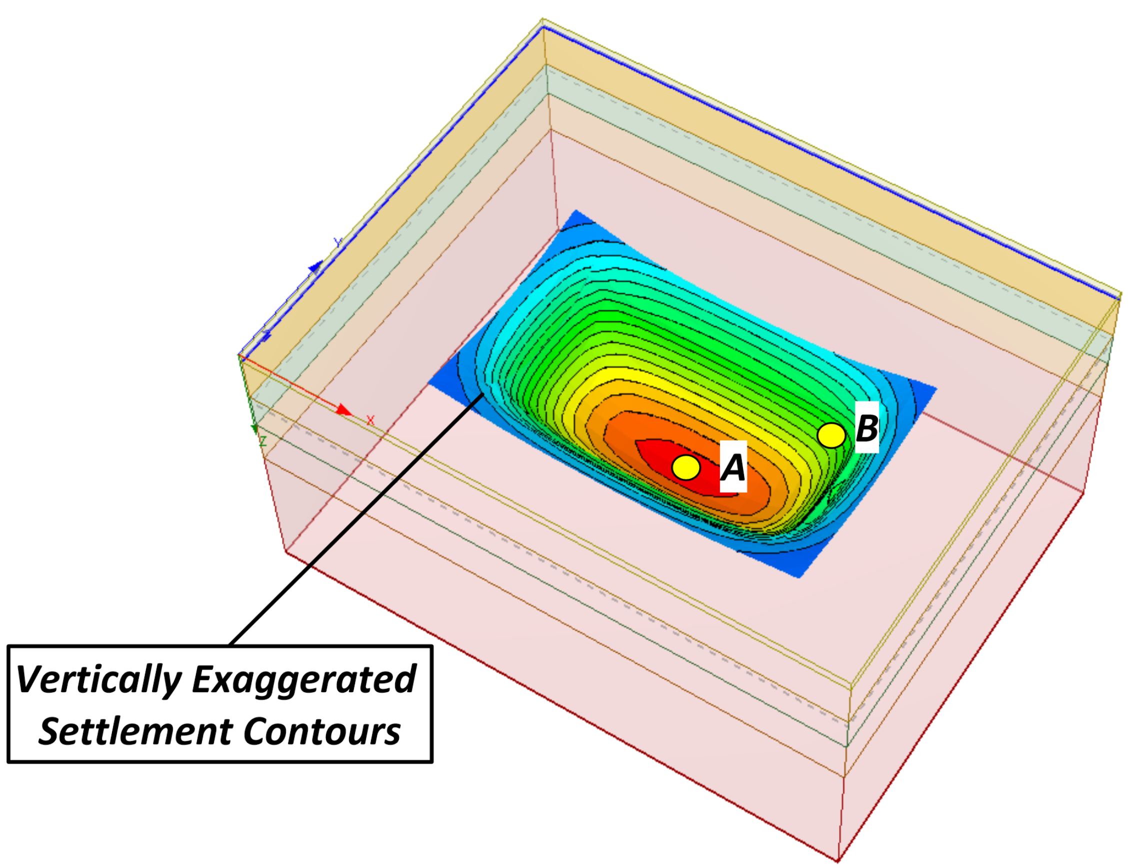
Settle 3D Model Input Parameters

Units	Depth Range (ft)	Layer Thickness (ft)	γ_{wet} (pcf)	E (ksf)	C_{ce}	C_{re}	OCR
Unit 1 - Clays	0-60	60	130	-	0.110	0.011	2
Unit 2 - Sands	60-100	40	130	3700	-	-	-
Unit 3a - More Weathered Bedrock	100-150	50	127	-	0.058	0.009	2
Unit 3b - Less Weathered Bedrock	150-300	150	130	10000	-	-	-

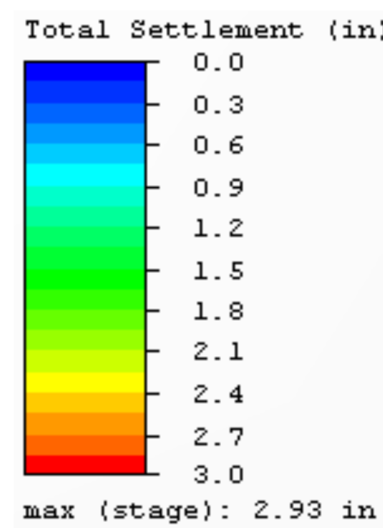
Plan View - Settle 3D Results

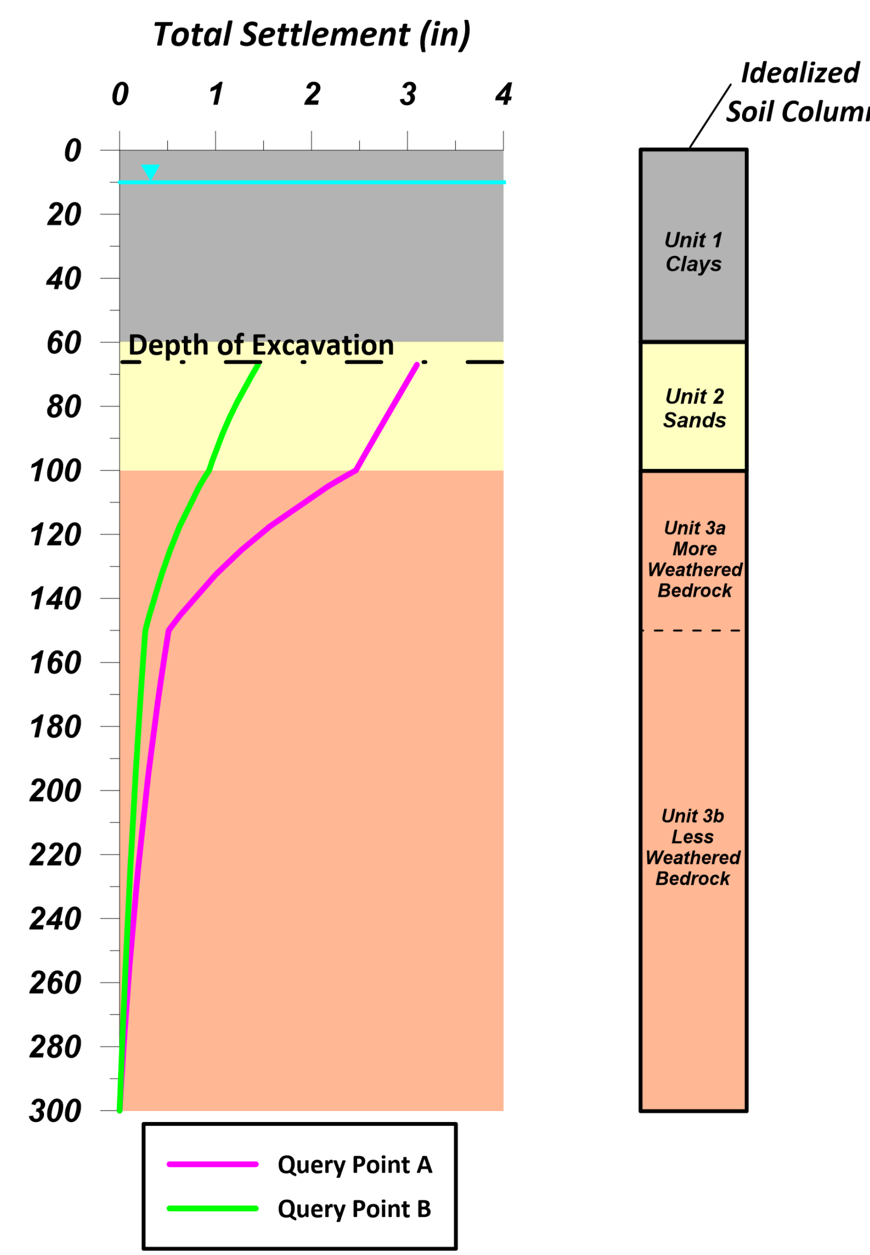


Isometric View - Settle 3D Results



Contour Legend





Note:
Mat Loading = 6 ksf

Settle 3D Model Results		
Project: 940 N Sycamore Ave. Development		Figure E-4
Project No.: 21106A	Date: APR 2022	

GeoPentech