Geotechnical Report

GEOTECHNICAL INVESTIGATION
PROPOSED DEVELOPMENT AT
1201 SOUTH GRAND AVENUE
LOS ANGELES, CALIFORNIA

Prepared for
City-Century LLC
865 S. Figueroa Street, Suite 2330
Los Angeles, CA 90017

May 26, 2020

Prepared by
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GeoPentech

May 26, 2020
Project No. 15083A

Mr. Simon Kaplan
City-Century LLC
865 S. Figueroa Street, Suite 2330
Los Angeles, CA 90017

SUBJECT: Geotechnical Investigation Report
1201 S. Grand Avenue (12th and Grand)
Los Angeles, California

Dear Mr. Kaplan:

This report presents the results of GeoPentech's geotechnical investigation for the proposed development that will be located at 1201 South Grand Ave. in Los Angeles, California. This investigation was performed in accordance with our proposal dated January 17, 2018, and authorizations dated February 23, 2018, and May 13, 2020.

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

Very truly yours,
GeoPentech, Inc.

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1 INTRODUCTION

This geotechnical investigation report is provided for the proposed tower and associated podium structure to be located at 1201 South Grand Avenue, in downtown Los Angeles, near the intersection of West 12th Street and South Grand Avenue, as shown on Figures 1 and 2b.

2 PROJECT UNDERSTANDING

Our understanding of the project is based on an entitlement design plan set by MVE+Partners (see Figure 2a), and our previous experience at similar projects in downtown Los Angeles. Based on the drawings provided to us and our discussions with City-Century, we understand that the proposed development consists of a 40-story mixed-use residential tower underlain by 2 to 3 subterranean basement levels. We further understand that the residential tower will be surrounded by a podium and parking structure which will have 8 levels above ground and 2 to 3 subterranean levels.

We understand that the design for the structure will be carried out in conformance with the 2019 California Building Code (CBC 2019) and ASCE 7-16 requirements inclusive of Supplement 1 (ASCE 2018). We also understand that the seismic design for this project will be performance-based and therefore will be subject to the City of Los Angeles Panel Peer Review process. The guidelines for the development of ground motions on this project will be based on "An Alternative Procedure for Seismic Analysis and Design of Tall Building Located in the Los Angeles Region," 2017 edition, by the Los Angeles Tall Buildings Structural Design Council (LATBSDC), inclusive of the 2018 Supplements (LATBSDS, 2018). Note that this version of the guidelines by LATBSDC references ASCE 7-16 for development of design seismic ground motions.

At this time, reports of previous geotechnical investigations at the project site were not available for our review; however, we have compiled available data from neighboring projects to provide background on the site conditions and supplement our current field investigation. It is our understanding that no structural loading conditions are available at this time; as such, for the purposes of this report, we assumed preliminary loading conditions based on our experience from previous similar development as described in Section 8.1, Foundations.

3 SCOPE OF WORK

The scope of work included the following tasks:

- Reviewed the schematic design plans.
- Reviewed geotechnical reports and construction records by GeoPentech and others at adjacent properties.
- Completed a site investigation, which included:
  - Drilling 2 hollow-stem auger borings (B-1 and B-2) extending to depths 151.5 feet and 101 feet to obtain samples using SPT and California Samplers at approximately 5 to 10-foot intervals to depths of 60 feet and at 10-foot intervals thereafter.

  - Performing two surface-wave geophysical surveys to measure seismic wave velocities of the subsurface materials as a function of depth to supplement the two borings in characterization of the subsurface materials and assess the subsurface conditions.
  - Performing one geophysical downhole seismic survey to characterize the shear and compression wave velocity profile at the site. The downhole seismic measurements were performed in borehole B-1 to a depth of 151.5 feet below the existing ground surface.
  - Performing percolation testing in borehole B-2 to evaluate the infiltration characteristics of the subsurface materials for stormwater management system design.

- Performed laboratory testing of soil samples, including analyses of static physical soil properties.
- Evaluated the subsurface conditions, geologic setting, seismic conditions, and geologic-seismic hazards affecting the area and their potential impact on the subject project.
- Performed engineering analyses and developed recommendations for design and construction of the proposed tower and associated podium structure.
- Prepared this geotechnical report.

4 EXISTING SITE CONDITIONS

The proposed project site is located at 1201 South Grand Avenue, which is currently occupied by a surface parking lot towards the southwest side and an existing office building on the northeast side. The existing office building is three stories high, is founded on a shallow foundation with no basement, and will be replaced by the currently proposed development. The site is bound by a 20-ft public alley to the northwest, South Grand Avenue to the southeast, West 12th Street to the northeast, and an existing 1-story building at 1225 S. Grand to the southwest. The site is relatively flat at an elevation of about 240 ft msl.

5 FIELD EXPLORATION AND LABORATORY TESTING

5.1 Available Subsurface Data

GeoPentech has completed geotechnical field investigations and observation of new construction in downtown Los Angeles near the subject site. Pertinent GeoPentech projects include:

1) The proposed City-Century development at 1233 S. Grand Avenue and consisted of drilling two (2) borings ranging in depth from 82 to 135 feet and pushing five (5) CPTs to depths ranging from 10.5 to 25.7 feet below existing grade (see Figure 2b), and

2) The property at 1200 S. Grand Avenue (located across the street and southeast of the project site as shown on Figure 2b), which was investigated with ten (10) borings, ranging in depth from 51.5 to 101.3 feet and included in our Geotechnical Investigation Report dated May 13, 2013.

GeoPentech also reviewed two publicly available geotechnical investigation reports for nearby projects completed by other consultants from LADBS records. These following reports were reviewed:

1) Investigation performed by Applied Earth Sciences (AES, 2013) for the site at 1249 S. Grand Avenue, southwest of the subject property (see Figure 2b). As documented by AES (2013), three borings to depths of 41-feet encountered fill materials overlying stiff/dense alluvial sandy silts, silty clays, and gravelly to clean sands. No groundwater was encountered. The soils report was approved by LADBS under Log #81416.

2) Investigation performed by GeoDesign (GeoDesign, 2005) for the project at 1155 S. Grand Avenue, located directly across 12th Avenue northeast of the current project site. The project consisted of a 28-story tower with associated podium and parking structure with several below grade levels. The investigation included seven (7) new borings to depths of up to 71 feet below existing grade and encountered about 5 feet of fill overlying medium stiff to very stiff silt and clay and very dense poorly graded sands with gravel. No groundwater was encountered. The soils report for 1155 South Grand Avenue was submitted to LADBS on April 19, 2005 under Log #47999.

5.1.1 Current Field Investigation

GeoPentech's current field investigation was performed over two weekends from February 24, 2018 to March 3, 2018. The investigation consisted of drilling two borings (B-1 and B-2), completing one downhole seismic survey in boring B-1 at the location of the proposed tower, conducting 2 percolation tests in boring B-2, and collecting surface-wave geophysical measurements along two survey lines located along the north and east edges of the property. The approximate locations of the borings and geophysical lines are shown on Figure 2b.

Two borings (B-1 and B-2) were drilled to depths of approximately 151.5 and 101 feet, below existing grade, respectively. Soils encountered during drilling were visually classified in general accordance with the Unified Soil Classification System (USCS). Soil samples were obtained at approximately 5- to 10-foot intervals, with 5-foot spacing generally in the shallower portions of the borings and 10-foot spacing in the deeper sections. Samples were obtained using either a Standard Penetration Test (SPT) sampler or a Modified California (MC) sampler. The logs of the borings are provided in Appendix A. Selected soil samples were submitted to the geotechnical laboratory of Leighton Group, Inc. in Irvine, California to evaluate their pertinent physical and engineering properties. The laboratory testing assigned includes determinations of moisture content and unit weight, sieve analysis, Atterberg limits, consolidation, direct shear, and corrosion tests. The results of the tests are included in Appendix B along with Leighton Group's documentation letter dated April 6, 2018. GeoPentech has reviewed the results of the laboratory tests and concurs with the testing procedures and results.

Downhole seismic measurements were conducted at the subject site in boring B-1 following the installation of temporary casing during drilling. The downhole seismic survey method directly measures the in-situ vertically-propagating compression (P) and horizontally-polarized shear (SH) wave velocities as a function of depth within the geologic material adjacent to a borehole. Measurement procedures followed ASTM D7400-08, "Standard Test Methods for Downhole Seismic Testing." The measurements were made by GeoPentech personnel, and the collected data was reduced for use in our ground motion evaluation. GeoPentech also performed surface wave geophysical measurements along two survey lines (SW18-1 and SW18-2) along the north and east sides of the property. The locations of the geophysical measurements are shown on Figure 2b. The purpose of the surface wave surveys was to supplement the boring and downhole seismic information. The results of the seismic downhole measurements within borehole B-1 and surface measurements along lines SW18-1 and SW18-2 are presented in Appendix A and discussed in Section 6.3 below.

Lastly, we also performed infiltration testing in boring B-2 to evaluate drainage characteristics of the alluvium in-situ and provide guidance for design of a potential infiltration system for the project. The results of this testing and design recommendations are included in Section 8.6 below.

6 GEOLOGIC CONDITIONS

6.1 Regional Geology and Seismicity

Regionally, the site is located near the boundary of the Peninsular Ranges physiographic province and 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. Locally, the site is within the Los Angeles Basin, about 5 miles south of the Santa Monica Mountains range front and about two miles south of the Elysian Hills at the northern edge of the Los Angeles Basin. The Elysian Hills, along with the Repetto Hills to the east, constitute a group of low hills between the Santa Monica Mountains and the Puente Hills. The Elysian Hills, which are separated from the Repetto Hills by the existing Los Angeles River, are composed primarily of Pliocene to Miocene age sedimentary rocks, locally overlain by Pleistocene age alluvial materials and Holocene age alluvium in the intervening drainages and lowlands (Lamar, 1970). The Holocene sediments south of the Elysian Hills, where the site is located, were deposited along and adjacent to the ancestral Los Angeles River.

As shown on Figure 3a, the site is roughly two miles west of the Los Angeles River, within young alluvial valley deposits ("Qya" as described on Figure 3b) and near the mapped approximate boundary of young alluvial fan deposits ("Qyf"). Both units are Late Quaternary (Late Pleistocene to Holocene) in age. The alluvial valley deposits are described as slightly consolidated clays, silts, sands, and gravels deposited along stream valleys and alluvial flats of larger rivers. The alluvial fan deposits are described as slightly consolidated boulders, cobbles, gravels, sands, and silts (Figures 3a and 3b; CGS, 2012).

The project site is located within a seismically active region of southern California, as indicated on Figure 4a. Recent examples of the seismic activity in the region include the 1987 Whittier Narrows earthquake and the 1994 Northridge earthquake. Figure 4a shows the site location relative to mapped active faults in the region, as identified by the US Geological Survey (USGS, 2010). Significant faults near the site mapped with late Quaternary surface displacement include the Hollywood Fault (located about 8 km north of the site); the Newport-Inglewood Fault (located about 10 km to the west); the Raymond Fault (located about 11 km to the northeast); and the Santa Monica Fault (located about 14 km to the northwest). The San Andreas Fault is located approximately 57 km to the northeast.

Potentially active blind thrust faults are also believed to exist in the region, as shown on Figure 4b. These include the Puente Hills Blind Thrust (Shaw and Shearer, 1999) and the Upper Elysian Park Thrust (Oskin et al., 2000). 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 4b, the site is located on the hanging walls of the Puente Hills, Puente Hills (LA), and Compton Alt. 2 blind thrust faults. The distance between the site and the fault planes are approximately 5 km, 4 km, and 14 km, respectively. Additionally, the site is located on the footwall of the Elysian Park (Upper), with a closest distance of about 5 km.

Finally, as discussed in Section 7.1 below, the site is not located within a currently-established Alquist-Priolo (AP) Special Studies Zone. Although the site is located within the Hollywood Quadrangle, it is not affected by the new Earthquake Zone of Required Investigation for the Hollywood Fault (CGS, 2014), as the AP Zone for the Hollywood Fault is almost 8 km north of the site.

6.2 Site Geology and Subsurface Conditions

Figure 3a shows a geologic map of the site area by CGS (2012), which shows the surface of the site is underlain by Late Pleistocene to Holocene alluvial fan and valley deposits (referred to herein as Quaternary alluvium). This is consistent with the materials observed by GeoPentech during our field exploration and other observations at nearby downtown Los Angeles projects, including our work at the 1200 S. Grand Avenue site across the street (see Section 5.1). Specifically, the Quaternary alluvium at the site is mantled by artificial fill. Artificial fill was encountered in borings B-1 and B-2 to a depth of about 5 feet. Based on our experience in the downtown area, it is possible that deeper fill may exist at the site. Construction records documenting the details of the fill materials, placement, and depth were not available. Additionally, bedrock was not encountered in our borings to the depths explored during our field investigation.

Figures 5 and 6 show simplified geologic cross-sections A-A' and B-B' across the subject property. The locations of the geologic cross sections A-A' and B-B' are shown on Figure 2b. Descriptions of each of the geologic units are discussed below.

- Artificial Fill: Fill soils, generally about 5 feet thick, were encountered in the current borings at the site. The fill encountered consisted of stiff to hard clayey silts with sand and gravel with brick fragments and concrete. Note that deeper fill, including debris, and remnants of previous foundations may also be present at various depths across the site, based on our experience with similar projects in the area. The density/strength of the fill may also vary across the site.
- Quaternary Alluvium: Underlying the fill, the alluvium generally consists of dense to very dense sands, silty to clayey sands, and sandy silts with gravel, with a layer of very stiff to hard silty to sandy clay. The clay layer was encountered in both borings (B-1 and B-2) at depths ranging between about 45 and 70 feet below existing grade and was approximately 20 to 25 feet thick. SPT blowcounts in the alluvium ranged from 13 to 100+ blows/ft, with an average of over 70 blows/ft. Note that lower blowcounts were generally measured in more clayey material. Also note that many blowcounts were likely affected by gravel and may not reflect the material matrix density/stiffness but rather the influence of oversize particles. However, the blowcounts overall indicate dense to very dense/stiff to hard material. The expansion potential of the clay layers encountered from depths of about 45 to 70 feet is estimated to vary from low to medium across the site based on the results of the Atterberg Limits.

The site is located within the active Downtown Los Angeles Oil Field and within a methane zone defined by the City of Los Angles as discussed further in Sections 7.12 and 7.13, respectively.

6.3 Seismic Downhole Measurements and Results

Downhole seismic measurements were collected within borehole B-1 to a depth of 150 feet below ground surface, as described in Appendix A. A summary of the P-wave and SH-wave layer velocities and depths for the various geologic units logged within these boreholes is presented in the table below.

SUMMARY OF SH-WAVE AND P-WAVE VELOCITY LAYERS IN BORING NUMBER B-1

PREDOMINANT LITHOLOGY	Depth Range (ft)	SH-WAVE Velocity (ft/sec)	P-WAVE Velocity (ft/sec)
Clayey Silt with sand (ML) [Fill]	0 to 5	670	1,500
Dense to very dense silty Sand to Sand with silt (SM to SW-SM) and Stiff to hard silty Clay (CL) to clayey Silt (ML) with sand [Alluvium]	5 to 65	1,360	2,660
Very dense silty Sand, Sand, and gravelly Sand (SM, SP and SW) and hard sandy Silt (ML) [Alluvium]	65 to 150	1,940	3,390

The V_{S30} for Boring B-1 was calculated between a depth of 35 to 135 feet, resulting in a design value of 1,700 ft/s as described in Appendix A.

6.4 Groundwater

Free groundwater was not observed during drilling in the current investigation to the maximum 150-foot depth explored. According to the seismic hazard zone report for the Hollywood Quadrangle, historically highest groundwater in the vicinity of the site may be around 100 to 110 feet below the ground surface. It is noted that no free groundwater was observed during drilling, and downhole seismic P-wave velocity measurements within B-1 suggest the geologic material adjacent to the borehole is unsaturated throughout the depths explored.

It should be recognized that groundwater levels can fluctuate over time, depending on seasonal rainfall and other influences. Furthermore, although no groundwater was observed during drilling within the site limits, there may be a potential for perched water seepage to occur locally in sandy zones of the alluvium deposits.

7 EVALUATION OF GEOLOGIC AND SEISMIC HAZARDS

An evaluation of the potential impacts on the site from potential geologic and seismic hazards is presented in the following sections.

7.1 Fault Surface Rupture

The subject site is not within a currently-established Alquist-Priolo (AP) Special Studies Zone. Therefore, the potential for fault surface rupture at the site is considered low.

As previously noted, the site is located on the hanging wall of the Puente Hills Blind Thrusts seismic source planes. Although blind thrusts do not represent discrete surface rupture hazards to the site, they are potential sources of seismic shaking and possibly distributed coseismic ground deformation.

7.2 Seismic Shaking and Site Class

We understand that the design for the tower will be carried out in conformance with the 2019 CBC and ASCE 7-16 requirements using the performance-based design procedure specified by LATBDC, 2018. 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 C of the report.

Site-specific shear-wave velocity measurements were conducted during the field investigation and were used to determine the site class for seismic design according to Chapter 20 of ASCE 7-16. As the proposed development includes 2 to 3 subterranean basement levels, the $V_{S,30}$ value was calculated using shear-wave velocity measurements between 35- and 135- feet below the ground surface. The shear wave velocity data recently collected by GeoPentech at the project site indicates a $V_{S,30}$ value of 1,700 ft/s (518 m/s). This $V_{S,30}$ value corresponds to site classification for seismic design of Site Class C (1,200 $< V_{S30} <$ 2,500 ft/s).

To fulfill the seismic design requirements, the following site-specific response spectra are developed:

- A "Maximum Considered Event" uniform hazard spectrum with risk-targeted, maximum rotated ordinates at 5% damping; also known as a site-specific MCE_R response spectrum (corresponding to a 1% probability of collapse in a 50-year period; i.e., a modified 2,475-year return period spectrum). Note that because the site is classified as near-source due to the presence of hazard-significant sources within 15 km of the site, the MCE_R-level spectrum is provided for two alternative orientations: a Fault Normal (FN) MCE_R spectrum and a Fault Parallel (FP) MCE_R spectrum.
- A "Service-Level Earthquake" uniform hazard spectrum with average horizontal spectral ordinates at 1.7% damping (corresponding to a 50% probability of exceedance in a 30-year period; i.e., a 43-year return period)

For completeness, the code-compliant, site-specific "Design Level" or DRS uniform hazard spectrum with risk-targeted, maximum-rotated ordinates at 5% damping has also been provided. In addition, the selection of seed time-histories for the nonlinear response analysis was carried out to identify existing recordings from earthquakes that have characteristics similar to the events that control the hazard in the period range of interest at the MCE_R hazard level. The selected seed time histories are included in Appendix C.

7.3 Code-Based Seismic Design Parameters

Given the site latitude and longitude (located near 34.04006° N, 118.26375° W) and site shear-wave velocity (discussed above), mapped seismic hazard values were queried from the USGS online seismic design map application at https://earthquake.usgs.gov/ws/designmaps/asce7-16.html . Using the ASCE 7-16 standard, the mapped design parameters for a Site Class C, Risk Category I, II, or III structure at this location yield a Seismic Design Category D. Based on this information, the general procedure ground-motion analysis carried out in accordance with Section 11.4 of ASCE 7-16 results in design spectral acceleration parameters S_{DS} and S_{D1} of 1.550 g and 0.642 g, respectively. Note that these values are superseded in this report by the site-specific values presented in the ground-motion evaluation conducted for the site (see Appendix C) and are provided here for completeness.

7.4 Liquefaction Potential

Liquefaction potential is greatest where the groundwater level is shallow, and submerged, relatively loose sands occur within a depth of about 15 meters (50 feet) or less below the ground surface. Liquefaction potential generally decreases as the density, grain size, clay content, and gravel content increase. In addition, higher loading due to higher ground acceleration and shaking duration also increase liquefaction potential.

According to the Seismic Hazard Zone information on the Hollywood Quadrangle Zones of Required Investigation Map (CGS, 2014), the site is not within an area identified as having a potential for liquefaction. This classification is consistent with our site-specific observations, which indicate that the soils beneath the site are predominantly dense to very dense sands and very stiff to hard sandy to silty clays. In addition, free groundwater was not encountered within the borings at the site (up to 150 feet bgs). Therefore, the potential for liquefaction and the associated ground deformation beneath the site is considered to be negligible.

7.5 Seismically-Induced Settlement

Seismically-induced settlement is often caused when unsaturated loose to medium-dense granular soils with relatively low fines-content are densified during ground shaking. The granular materials encountered in our exploratory borings are not in the loose to medium-dense category and are generally dense to very dense. The remainder of the soil encountered consists of very stiff to hard silty to sandy clay. Therefore, the potential for seismically-induced settlement at the site is considered negligible.

7.6 Subsidence

Ground surface subsidence generally results from the extraction of fluids or gas from the subsurface that can result in a gradual lowering of the overlying ground surface. Subsidence can also occur when subsurface peat deposits oxidize and undergo volume loss. Although the site is located over the Los Angeles Downtown Oil Field, subsidence of the area above this oil field has not been reported. Additionally, the subsurface soils are not known to contain significant quantities of peat that would create a potential for subsidence. Therefore, the potential for subsidence is considered low.

7.7 Seismically-induced Flooding

The potential hazard for seismically-induced flooding is generally associated with a body of water located adjacent to the site, or from seismically-induced failure of a reservoir located on drainage upstream of the site. According to the FEMA flood insurance map for the area (FEMA, 2008), the site is not located within any defined dam or debris basin inundation area, nor is it within a defined flood boundary. As such, seismically-induced flooding is not considered a hazard at the site.

7.8 Landslide

A potential for landslide is often indicated in areas of moderate to steep terrain that are underlain by unfavorably oriented geologic layering or discontinuities. The site is located on relatively flat terrain, the underlying sedimentary units are relatively flat lying, and no landslides are mapped in the vicinity of the site (CGS, 2014). In addition, the site is not in a designated earthquake-induced landslide hazard zone (CGS, 2014). Therefore, a potential for landslide is considered negligible.

7.9 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 around 240 feet (msl) and is relatively far from the shoreline. As such, the potential for damage from a tsunami at the site is considered remote.

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 considered to be 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 Oil Wells

The site is within the active Downtown Los Angeles Oil Field. Based on a review of DOGGR maps, the site does not contain any known active, inactive, or abandoned oil, gas, or geothermal wells. However, both currently and previously active injector and producer wells are located in the vicinity of the site. In particular, three plugged producer wells are located at 1211 S. Olive Street, about 250-350 feet to the southeast, and a cluster of injector and producer wells, which are directionally drilled throughout the area of the oil field, are located at the “Broadway Drillsite” at the 1300 block of South Broadway, approximately 1100 to 1200 feet southeast of the site. Wells at 1211 S. Olive Street were uncovered during recent construction and were plugged to current standards; however, construction details of the wells are unknown, and previous production records are unavailable from DOGGR.

7.13 Methane

The site is located within a methane zone defined by the City of Los Angeles (2004). At this time we understand that a methane study has been performed at the site by Methane Specialists and is documented in a report dated May 10, 2018. Based on the report, the project may require a passive methane mitigation system. It is our understanding that the methane system design will be completed once the project configuration is finalized.

7.14 Other Geologic Hazards

Other geologic conditions including expansive soils, radon gas, presence of naturally occurring asbestos in geologic formations, hydro -collapse, and clays and cyclic softening are not considered to be hazards at the site.

8 GEOTECHNICAL RECOMMENDATIONS

8.1 Foundations

8.1.1 General

In general, approximately the upper 5 feet of the subsurface consists of undocumented fill that is not suitable for support of the building loads. The fill is likely to be variable, and deeper fill, debris or unsuitable native soils could also be encountered in isolated locations across the site. It is anticipated that all the fill and debris will be removed as part of the excavation for the proposed three levels of subterranean parking.

Based on our review of the schematic design plans provided to us by the project's Architects (MVE+Partners) and dated 05/04/2020 (see Figure 2a), we understand that the proposed tower structure will be supported on a continuous mat foundation system with the associated podium/parking structures supported on either an extension of the tower mat or on individual spread and strip footings at the same elevation. As discussed in Section 2.0, no structural loading conditions are available at this time; as such, for the purposes of this draft report, we assumed preliminary loading conditions based on our experience from previous similar development as shown in the following sections.

From a geotechnical standpoint, the proposed combined foundation system is feasible. The following section provides specific recommendations for the design and construction of the foundations.

8.1.2 Tower Mat Foundation

The following table summarizes the tower information and the assumed loading conditions for the tower. As discussed previously in Section 2.0 (Project Understanding), we anticipate the tower will have 2 to three subterranean levels and will be supported on a mat foundation system. The following section addresses the assumed configuration with three subterranean levels.

Proposed Tower	
Number of Stories (Tower)	40 stories above grade Underlain by 3 subterranean levels
Footprint Area	~ 90-foot by 112-foot (central portion of site)
Assumed Elevation of Bottom of Foundations	~ 200 to 205 feet (msl)
Assumed Thickness of Mat Foundation	5 to 10 feet
Assumed Nominal Loading – Tower Area (DL+LL)	8,000 psf

- The bottom of the mat foundations for the proposed building is anticipated to extend to an elevation of about 200 feet msl (that is about 40 feet below existing ground surface at this location). The mat foundation at these elevations can be established on the alluvium deposits comprised of the dense to very dense sand and silty to clayey sands.

- Based on the drawings provided to us, and assuming that the mat foundation will extend about 10 feet beyond the footprint of the tower, the foundation size for the proposed mat is estimated to be about 100 feet by 125 feet and we have assumed that it will be about 5-10 feet thick with the thicker portions near the central core. Note that we have no details of the proposed foundations at this time.

- Based on an allowable average bearing capacity of 8,000 psf for the tower portion of the structure, the settlement at the center of the proposed mat is estimated to be about 2 inches and about ¾ inches at the corners of the tower portion. The average allowable bearing should be confirmed once details of the structural design are available.

- A vertical unit modulus of subgrade reaction, K, of 300 pounds per cubic inch (pci) may be used in the design of the mat foundations. This value is a unit value for use with a one-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 (in feet).

Note that the modulus of subgrade reaction should be confirmed once details of the foundation design are available and we can coordinate with the structural engineer.

- Lateral loads may be resisted by friction and by the passive resistance of the soils. The allowable passive resistance in predominantly dense native soils encountered at depths greater than 25 feet below existing grade may be assumed to be equal to the pressure developed by a fluid with a density of 350 pounds per cubic foot along the embedded side of the mat or footing. The allowable passive resistance should be limited to a maximum of 9,000 psf. A coefficient of friction of 0.4 may be used between footings and the underlying native soils. This value should be reduced to 0.25 if used in combination with the passive earth pressure. A one-third increase in the passive value may be used for wind or seismic loads.

8.1.3 Podium/Parking Structure Foundation

The Podium/Parking structures are 8 stories and underlain by 2 to 3 levels of subterranean parking. These structures can be supported on a single continuous mat foundation extending under the entire structure including the tower portion of the building. This continuous mat can be designed with the parameters given for the tower mat above. Alternatively, the podium portion of the building can be supported independently from the tower on individual spread and strip footings. The following recommendations can be used for spread and strip footings. The following section addresses the assumed configuration with three subterranean levels.

Proposed Podium/Parking	
Number of Stories (Podium/Parking)	8 Stories above grade Underlain by 3 subterranean levels
Footprint Area	~21,700 SF (area between tower and site perimeter)
Assumed Elevation of Bottom of Foundations	~ 200 to 205 feet MSL
Assumed Nominal Loading – Podium Area (DL+LL)	2,000 psf

- The foundations of the 8-story Podium/Parking above ground structure will be located at the bottom of the subterranean parking levels and should be established on the alluvium deposits comprised of the dense to very dense sand and silty to clayey sands.
- If spread footings are used, foundations with a minimum width of 10 feet and established at a depth of at least 4 feet below the lowest adjacent finished floor level, may be designed for a maximum allowable bearing capacity of 3,500 psf. Total settlements of spread footings is anticipated to be about ¾ inch and differential settlements between two similarly loaded footings is estimated to be about ½ inch.
- Continuous footings, with a minimum width of 3 feet and established at a depth of at least 3 feet below the lowest adjacent finished floor level, may be designed for an allowable bearing capacity of 3,000 psf. Settlement of continuous footings is estimated to be about ¾ inch and differential settlement over a horizontal distance of 30 feet to be about ½ inch.
- The recommended allowable bearing capacities are net values, and as such the weight of concrete in the footings can be taken as 50 pounds per cubic foot (pcf) and the weight of soil backfill may be neglected.
- The above allowable bearing values are for dead plus live loads and may be increased by one-third to accommodate transient loads that include wind or seismic loads.
- Lateral loads may be resisted by friction and by the passive resistance of the soils. The allowable passive resistance in predominantly dense native soils encountered at depths greater than 25 feet below existing grade may be assumed to be equal to the pressure developed by a fluid with a density of 350 pounds per cubic foot along the embedded side of the mat or footing. The allowable passive resistance should be limited to a maximum of 9,000 psf. A coefficient of friction of 0.4 may be used between footings and the underlying native soils. This value should be reduced to 0.25 if used in combination with the passive earth pressure. A one-third increase in the passive value may be used for wind or seismic loads.
- Using the estimated values of the settlement for the Podium/Parking portion of the structure and the estimated settlements for the Tower portion, the differential settlement between the two buildings (Tower and Podium/Parking) may vary from about ½ to 1 inch.
- The recommended values for the podium footings should be confirmed once details of the structural design are made available.

8.2 Walls-Below-Grade

8.2.1 Lateral Earth Pressure

Subterranean parking and basement walls should be designed to resist lateral earth pressures plus any surcharges from adjacent loads. 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 40 pounds per cubic foot (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 55 pcf. The recommended earth pressures are calculated assuming that a drainage system will be installed, so that hydrostatic pressure will not develop behind the subterranean walls.

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.

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 an incremental seismic lateral pressure of 23H (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 can be combined with the active earth pressure mentioned previously. If designing for static loading only, the at-rest lateral earth pressure should be used.

As noted previously, lateral earth pressure surcharges due to footings or foundations located within a 1:1 line projected upwards from the bottom of the wall should be included. Based on the proximity of the existing 1-story building located on the southwest side of the site at 1225 South Grand Avenue, it may impose a surcharge on the new basement walls. Further discussion and preliminary values for design are included in section 8.4.3.

In addition, the buildings directly across the alleyway north of the project, could also impose a surcharge depending on the details of their foundations and distance from the new building. Details for these buildings are not available at this time, but can be evaluated if their footings are located within a 1:1 line. Other surcharge conditions that are not directly addressed in this report can be handled on a case-by-case basis.

8.2.2 Drainage

Walls below-grade and temporary shoring should be designed to resist hydrostatic pressures (equivalent fluid pressure of 62.4 pounds per cubic foot), or be provided with positive drainage behind the wall. Miradrain 6000 (or the equivalent) or pea gravel wrapped in filter fabric, placed behind wall sections (between soldier pile shoring, if applicable), would provide satisfactory drainage. The drain 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 drainage pipe should discharge to pipes through the base of the wall, with a minimum of one pipe between each pair of soldier piles, and should be connected to an appropriate positive gravity drainage system. The pipe should be sloped at least 2 inches in 100 feet and surrounded by filter gravel.

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 can be used. The crushed rock or gravel should have less than 5% passing a No. 200 sieve. Subterranean walls should also be appropriately waterproofed.

Please note that in addition to these recommendations, typical practice in Los Angeles is to install a "rock pocket" in each shoring bay to collect groundwater that may accumulate behind the shoring and direct it to the basement wall drainage system. LADBS also typically requires installation and inspection of these "rock pockets" unless the basement is designed to resist hydrostatic pressures.

8.3 Floor Slab Support

If the subgrade is prepared as recommended in the following section on earthwork, floor slabs may be supported on grade where applicable. The existing undocumented fill and upper loose/soft natural soils are not considered suitable for support of floor slabs. Existing undocumented fill and upper loose/soft natural soils should be removed and replaced as engineered fill. Based on our current understanding, all undocumented fill at the site is expected to be removed as part of the mass excavation.

Note that as indicated in Section 7.13, a methane study (Methane Specialists, 2018) has been conducted for the project. In addition to the recommendations below, the floor slab support system would need to be coordinated with methane mitigation measures required for the project.

If vinyl or other moisture-sensitive floor covering is planned, we recommend that any floor slabs on grade in those areas be underlain by a capillary break consisting of a vapor-retarding membrane over a 4-inch-thick layer of gravel. A 2-inch-thick layer of sand should be placed between the gravel and the membrane to decrease the possibility of damage to the membrane. We suggest the following gradation for the gravel:

Sieve Size	Percent Passing
¾"	90 - 100
No. 4	0 - 10
No. 100	0 - 3

A low-slump concrete should be used to minimize possible curling of the slab. A 2-inch thick layer of coarse sand can be placed over the vapor retarding membrane to reduce slab curling. If this sand bedding is used, care should be taken during the placement of the concrete to prevent displacement of the sand. This sand layer may be omitted if the potential for slab curling can be addressed to the satisfaction of the structural engineer by other means. The concrete slab should be allowed to cure properly before placing vinyl or other moisture-sensitive floor covering.

We recommend that all earthwork and slab-on-grade construction be observed by a qualified geotechnical engineer firm to document the conditions of the final subgrade soils immediately prior to the slab-on-grade construction.

8.4 Sulfate Attack and Corrosion Potential of Soils

The corrosion potential was evaluated based on the laboratory testing performed by Leighton Consulting Inc. Laboratory testing performed by Leighton for the current study included pH, minimum electrical resistivity, and chloride and soluble sulfate content tests and results are provided in Appendix B.

Testing for sulfates on these three samples indicated mostly negligible content sulfate concentration, based on guidance from the American Concrete Institute (ACI-318-05), with test results indicating less than 150 ppm. Based on the negligible sulfate attack potential, no special type of cement is required for concrete structures and pipe; however, the structural engineer should check the mix design for adequacy from the structural standpoint.

Corrosion guidelines from Caltrans (2018) indicate that for structural elements, the Department considers a site to be corrosive if one or more of the following conditions exist for the representative soil and/or water samples taken at the site:

- Sulfate concentration is 1500 ppm or greater,
- Chloride concentration is 500 ppm or greater, or
- pH is 5.5 or less.

Based on laboratory testing on three samples (B-1@45’, B-2@15’, and B-2@50’), the on-site soils do not meet the Department’s definition for a corrosive environment for structures. However, electrical resistivity testing of the samples indicated a range from relatively high (about 8800 ohm-cm) to relatively low (about 1500 ohm-cm). Due to the potential variation across the site, if there are corrosion concerns, we recommend that a corrosion consultant be contacted to provide appropriate measures against corrosion.

8.5 Excavations and Temporary Shoring

8.5.1 <u>Excavations</u>

Excavations as deep as about 45 feet below existing grade are anticipated for the proposed development. Based on our experience on nearby sites and our discussions with the project team, we anticipate that shoring consisting of soldier piles and tie-back anchors with lagging will be considered with the potential for internal raker bracing for some portions of the site. Based on the site topography and the understanding that the excavation will extend to the limits of the property, we do not anticipate any sloping ground behind the shoring. Temporary excavations up to a height of 4 feet can be cut vertically. Excavations deeper than 4 feet should be shored or sloped back for safety. If space is available, excavations can be made with temporary slopes of 1:1 (horizontal to vertical) to a maximum depth of 30 feet.

Unshored excavations should not extend below a plane drawn at 1½:1 (horizontal to vertical) extending downward from adjacent existing footings. Note that on the southwest side of the site an existing 1-story building abuts the property line and shoring along this side of the site will need to accommodate the surcharge imposed by the building and limit deflections to avoid damaging it. Further discussion of this condition is included in Section 8.4.3.

The anticipated excavation depths on the order of 45 feet that are planned as part of the proposed development will be partially within the upper artificial fill and partially within the underlying alluvium deposits. Temporary excavation slopes with inclination of 1:1 (horizontal to vertical) can be used as-needed provided that the temporary slope excavations are monitored during construction and additional support measures are available in the event of excessive sloughing. Excessive caving was not noted in the borings performed; however, the chances of caving will increase within larger scale excavations and should be anticipated in particularly granular materials and/or where seepage may occur. It is important that all surface water be directed away from excavation slopes so as to reduce the chance of erosion and seepage.

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

8.5.2 Groundwater Control

Free groundwater has not been observed during drilling at the site, historically highest groundwater reported by CDMG in the vicinity is greater than 100 feet bgs, and p-wave velocity measurements at the site indicate unsaturated conditions to the full depth of exploration. Based on this information and the anticipated excavation depths of about 45 feet, no groundwater is anticipated to be encountered during construction. However, it is possible that seeps may occur at localized areas. We anticipate that groundwater if encountered, can be handled by collecting the water in sumps and pumping. The contractor should be prepared to handle groundwater or surficial runoff where encountered. Note that depending on when the construction occurs, groundwater levels could fluctuate and vary from when measurements are made.

It should be noted that controlling and maintaining of groundwater and surficial water during construction is the responsibility of the contractor. As part of the groundwater control, the quality of collected water should also be checked in accordance with applicable permits and regulations for water discharge.

8.5.3 Shoring

Braced or tied-back soldier pile shoring is recommended for the proposed excavations adjacent to existing streets and/or structures. For the design of braced or tied-back shoring, we recommend using a trapezoidal pressure distribution as shown on Figure 7. For level grade behind the shoring, the maximum pressure is equal to 22H in units of pounds per square foot, where H is the retained height in feet.

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 pounds per square foot, acting as a result of an assumed 300 pounds per square foot 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.

8.5.4 Existing Adjacent Building – Southwest Side of Site (1225 South Grand Avenue)

Currently, an existing 1-story building is located directly adjacent to the property line on the southwest side of the project site at 1225 South Grand Avenue. Based on potholing performed for the project at 1233 S. Grand, the existing building has a shallow footing system founded near the existing grade and has no basement levels extending below grade. In addition, the footings and building structure are comprised largely of masonry construction (bricks) which may be vulnerable to movement. The footings for this building will likely surcharge the proposed residential tower and podium structure as well as any temporary shoring utilized. Shoring along this portion of the proposed excavation should be designed to accommodate the loads imposed by the existing building to the north and limit deflections to less than ½-inch throughout construction.

Based on our understanding of information gathered during the potholing performed at 1233 S. Grand and subsequent evaluation by City-Century's structural consultant at the time, the foundations for the building at 1225 S. Grand are founded near the ground surface and have loads varying from about 12 kips to 41.5 kips per column with wall loads of about 2.4 kips/ft to 3.5 kips/ft.

Furthermore, several buildings are located directly across the alleyway on the north side of the new project site. The structural details of these buildings are unknown at this time and it is not clear whether they are close enough to the new project to potentially impose a surcharge.

For design purposes horizontal loads equivalent to ½ the vertical pressure imposed by the foundations of the adjacent buildings can be conservatively applied uniformly over the height of the adjacent shoring and basement walls. Foundations located within an area within a 1:1 line projected up from the bottom of the new basement should be included in the surcharge. This estimate can be refined with additional analysis once the details of the nearby buildings becomes available.

The details of the foundations for the building directly adjacent to 1201 S. Grand and the buildings north of the alleyway should be confirmed prior to design and construction of the new project.

8.5.5 Design of Soldier Piles

Soldier pile shoring consisting of steel beams placed in drilled holes, backfilled with concrete and braced or restrained by tieback anchors can be used to support the excavations. Lagging will be required between the soldier piles. Soldier piles should be installed at a maximum spacing of three diameters (center to center). For the design of soldier piles spaced at least 3 diameters apart on-center, the allowable lateral bearing value (passive pressure) below the bottom of the proposed excavation may be assumed to be 500 pounds per square foot per foot of depth up to a maximum of 5,000 pounds per square foot for piles embedded in native soils.

To develop the full lateral values, firm contact between the soldier piles and the in-situ soils must be achieved. Structural concrete may be used for that portion of the soldier pile below the bottom of the excavation. Lean-mix or non-structural concrete may be used below the excavated level, but should have sufficient strength to adequately transfer the imposed loads to the surrounding soils.

The soldier piles below the excavated level may be used to resist downward loads. The frictional resistance between the soldier piles and the soils below the excavated level may be taken as 500 pounds per square foot. This value is based on the assumption that full bearing will be developed between the steel soldier beam and the concrete and also between the concrete and the retained earth.

If alternative methods such as vibrating piles instead of placing soldier piles in drilled holes are considered by the shoring contractor, we recommend that soldier pile driving methods be limited to locations at least 40 feet from adjacent existing buildings (see "Monitoring" section below). For such cases the beams should be vibrated into place for the portion below the excavated level only. Note that if pre-drilling for the soldier beams extends beyond the excavated level, the allowable passive pressure should be reduced to 350 pounds per square foot per foot of depth up to a maximum of 3,500 pounds per square foot.

8.5.6 Lagging

Due to the granular nature of the subsurface materials, continuous lagging should be used. The soldier piles should be designed for the full anticipated lateral pressure; however, the pressure on the lagging will be less due to arching effects in the soils. Therefore, the lagging can be designed for the recommended earth pressure but limited to a maximum value of 400 pounds per square foot. Careful installation of the lagging will be necessary to achieve bearing against the retained earth. Where solid bearing is not achieved, cement slurry should be used to fill gaps between the lagging and retained earth.

8.5.7 Tied-Back Anchor Design

Lateral loads can be resisted by tieback friction anchors. For design of the anchors, 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 20 feet beyond the potential active wedge and to a greater length as necessary to develop the desired capacities.

For the design of pressure-grouted anchors we recommend the use of 2,500 psf grout-to-ground bond strength along the bonded zone. If anchors are not pressure-grouted and the grout in the bonded zone is placed by gravity, a value of 750 psf should be used for the grout-to-ground bond. The as-constructed capacities of the anchors should be determined by testing as outlined below under the Tie-back Anchor Testing section. As recommended below, all anchors should be tested.

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 for group effects is necessary.

8.5.8 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 to the surface. 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 facilitate placement of the sand 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. Excavation next to the shoring should not proceed below the level required for tieback installation until the anchors have been installed.

8.5.9 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. As an initial guideline, 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 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 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 subsequently 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.

8.5.10 Deflections

Predicting actual deflections of a shored embankment is difficult given the complex nature of the construction environment. It should, however, be realized that some deflection is likely to 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 elements of structures that are adjacent to the shored excavation. Note that the existing building on the north side of the site is directly adjacent to the property line. The shoring in this area should be designed to limit deflections to less than ½-inch.

8.5.11 Monitoring

Monitoring the performance of the shoring system following installation is recommended. The monitoring should at a minimum consist of periodic surveying of the lateral and vertical locations of the tops of all the soldier piles. An initial survey should be taken prior to the first level of excavation so that an accurate baseline may be established. It may also be beneficial to install inclinometer(s) along with the shoring system where the shoring is adjacent to existing structures. Depending on the details of the adjacent structure, an inclinometer could provide better information to assess whether the shoring system is performing adequately to limit deflections and potential deformations under the adjacent building.

Also note that installing soldier beams using a vibratory hammer next to existing buildings could cause undesirable shaking in structures off the site and installation by drilling for the full depth as discussed previously under Design of Soldier Piles. We also recommend that vibration monitoring be performed for all piles installed at a distance of 40 feet or less from existing buildings or structures. We recommend that the initial survey and monitoring program also include the adjacent existing structures, particularly the one-story building at 1225 S. Grand. Photographs and videos of the existing permanent structures are recommended as part of the documentation process.

8.5.12 Rakers

As an alternative to tieback anchors, raker braces may be used to brace the soldier-pile shoring walls. Raker bracing, where used, should be supported by continuous temporary concrete footings (deadmen) spanning across multiple raker locations. For design of such temporary footings, with the rakers inclined at 35 to 60 degrees from the vertical, a bearing value of 4,000 pounds per square foot (psf) may be used. The concrete dead man footings should be placed at the bottom of the excavations and founded at a depth of at least 15 feet below the existing ground surface in native alluvium and extending at least 4 feet below the lowest adjacent grade. Lateral resistance may be determined using a passive pressure equivalent fluid weight of 300 pcf. The allowable passive resistance should be limited to 6,000 psf. A coefficient of friction of 0.4 may be used between the footings and the underlying native soils. This value should be reduced to 0.25 if used in combination with the passive earth pressure. The upper 3 feet of soil should be neglected for lateral resistance calculations.

Excavation next to the shoring should not proceed below the level required for raker installation until the bracing has been installed. A temporary slope of 1:1 (Horizontal to Vertical) for the soil embankment can be used to provide internal stability while installing the bracing elements. Note that temporary benches next to the shoring should be a minimum of 4 ft wide horizontally.

8.6 Earthwork

8.6.1 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.

8.6.2 Subgrade Preparation and Moisture Conditioning

Areas excavated to receive fill should be cleared and stripped of all debris, deleterious matter, organic material and vegetation, and remnants resulting from demolition of existing foundations or utilities. 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.

8.6.3 Footing 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 undocumented fill is encountered at the bottom of excavation, additional removals may be required. The bottom of excavations should be proof-rolled so as to allow placement of any required fill at 95% relative compaction in accordance with ASTM D-1557, or the placement of concrete or concrete slurry mix as backfill where required. Compacted fill should be placed immediately upon approval of the prepared subgrade by the geotechnical engineer of record.

Where footing 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 footings.

8.6.4 Material for Fill

All granular (sands, silty sands with gravel) can be used as engineered fill. The clay soils encountered range from low to moderately expansive and are not suitable for use as compacted fill beneath concrete walks and paving. If the clay soils encountered are mixed with granular soils, they can be reused, however, care must be taken to ensure the soils are thoroughly mixed and uniform before placement. Oversize material (larger than 6 inches in diameter) should not be used in the fill. Any required import material should be approved by the geotechnical engineer of record prior to being placed at the site.

8.6.5 Compaction

The preparation of the subgrade, footing excavations 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 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 and slabs should be compacted to a minimum of 95 percent maximum dry density as determined in accordance with ASTM D1557. 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 lifts generally no greater than 8 to 12 inches thick, loose measurement. The moisture content of the fill material should be within 2 percentage points of optimum as determined by ASTM D1557.

8.7 Stormwater Infiltration

Based on observations during drilling and laboratory testing at the project site, the interval between 0 and 70 feet below existing grade is not suitable for stormwater infiltration. Field tests conducted in Boring B-2 at the project site indicate the soils encountered from approximately 75 feet to 80 feet below existing grade are sandy and appear favorable for stormwater infiltration. Details of the infiltration testing and results are included in Appendix A. The table below summarizes the infiltration test results.

SOILS AND INFILTRATION TEST RESULTS

Boring	Depth	Soil Type	Estimated Infiltration Rate inches/hour
B-2	75-80 ft	Silty SAND (SM)	19 in/hr

Based on these results, we recommend a design infiltration rate of about 19 in/hr for the sandy interval that may extend between depths of 70 to 90 feet below existing grade at and near Boring B-2. (Note that the total depth of the planned dry wells will need to be at least 10 feet above the highest historical groundwater table, which is estimated by the State to be about 100 feet below ground surface.) If the design capacity of the proposed wells at this location is insufficient, additional wells could be drilled near Boring B-2 using this design infiltration rate and depth range. If the well is proposed at a location other than near Boring B-2, the location should be brought to our attention for review, and further location-specific testing may be required; however, we generally recommend the infiltration well(s) be installed near the location of boring B-2, away from the more heavily loaded tower structure. In accordance with the City SUSMP guidelines, infiltration wells should be set back from property lines a minimum of ten (10) feet. Note that further guidance for stormwater infiltration systems and required setbacks available in Document No. P/BC 2017-118 from LADBS should also be followed.

Regular maintenance of the installed drywells should be performed on a periodic basis. Monitoring after large storms should also be performed. If changes in the infiltration rate are noted over time, this should be brought to our attention so we can re-evaluate the potential limitations of the system. Pre-treatment of the stormwater should be done to remove particulate matter and prevent clogging of the well.

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 slab-on-grade and mat 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 installation of the shoring system and testing of the tie-back anchors where applicable.
- 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 GENERAL CONDITIONS

The conclusions and recommendations presented in this report are based upon GeoPentech's understanding of the project and the assumption that the subsurface conditions encountered during construction do not deviate appreciably from those disclosed by the field exploration. In addition, we have made assumptions regarding the structural loading necessary to provide foundation design recommendations. If our assumptions differ from the actual design conditions, the differences should be brought to our attention so that we can modify our recommendations.

The information presented in this report is intended to be used for design and construction based on the assumptions made in this report and the information provided to us. This information is subject to change once the locations, configurations, layout, or features of the proposed buildings are changed. It is the responsibility of the Owner to bring any changes in the proposed structures and any deviations of the subsurface conditions to the attention of GeoPentech.

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 and in this locale.

10 REFERENCES

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

American Society of Civil Engineers (ASCE) (2018). Supplement 1 to the ASCE Standard ASCE/SEI 7-16, Minimum Design Loads for Buildings and Other Structures, effective December 12, 2018, American Society of Civil Engineers.

Applied Earth Sciences (AES), 2013, Report of Geotechnical Investigation, Proposed Apartment Building Project Lots 14 and 15 of Tract No. 17683, 1247-1251 South Grand Avenue and 401-417 West Pico Boulevard, Los Angeles, California, prepared for Mr. David Poorbaba, 9 August 2013.

California Building Standards Code (CBC), 2019, California Code of Regulations. California Building Standards Commission Based on the 2015 International Building Code, Sacramento, CA.

California Department of Water Resources (DWR), 1961, Planned Utilization of the Ground Water Basins of the Coastal Plain of Los Angeles County. Bulletin No. 104.

California Division of Mines and Geology (CDMG), 1998, Seismic Hazard Zone Report for the Burbank 7.5-Minute Quadrangle, Los Angeles County, California, Seismic Hazard Zone Report 016, 63 pp.

California Geological Survey (CGS), 2014, Zones of Required Investigation Map, Earthquake Fault Zones and Seismic Hazard Zones, Hollywood 7.5 Minute Quadrangle, Dated November 6, 2014 for Earthquake Fault Zones, and March 25, 1999 for Seismic Hazard Zones.

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.

City of Los Angeles, 2004, Methane and Methane Buffer Zones. Map prepared by GIS Mapping, Bureau of Engineering, Department of Public Works, dated March 3, 2004.

Dibblee, T.W., 2010, Geologic Map of the Hollywood and Burbank (South ½) Quadrangles, Los Angeles County, California, Dibblee Geological Foundation Map #DF-30, Second Printing, Scale = 1:24,000.

Dibblee, T.W., 1991, Geologic Map of the Hollywood and Burbank (South ½) Quadrangles, Los Angeles County, California, Dibblee Geological Foundation Map #DF-30, First Printing, Scale = 1:24,000.

FEMA, 2008, Federal Emergency Management Agency, Flood Insurance Rate Maps, Panel 06037C1620F.

GeoPentech, Inc., 2013, Geotechnical Investigation, Proposed Development at 12th and Grand, 1200 South Grand Avenue, Los Angeles, California, prepared for G12 Partners, LLC, 13 March 2013, 22 pp. plus figures and appendices.

Hernandez J., and Treiman, J. (2014), California Geological Survey, Fault Evaluation Report FER 253, The Hollywood Fault in the Hollywood 7.5’ Quadrangle, Los Angeles County, California. Dated February 14, 2014.

Lamar, D.L., 1970, Geology of the Elysian Park-Repetto Hills area, Los Angeles County, California: California Division of Mines and Geology Special Report 101, 45 p.

LATBSDC, 2018, An Alternative Procedure for Seismic Analysis and Design of Tall Buildings Located in the Los Angeles Region. A Consensus Document by the Los Angeles Tall Buildings Structural Design Council, 2017 Edition with 2018 Supplements, dated March 20, 2018.

Methane Specialists, 2018. Site Methane Investigation Report for High-rise condominium complex with three subterranean parking levels, 1201 S. Grand Avenue, Los Angeles, CA 90015, dated May 10, 2018.

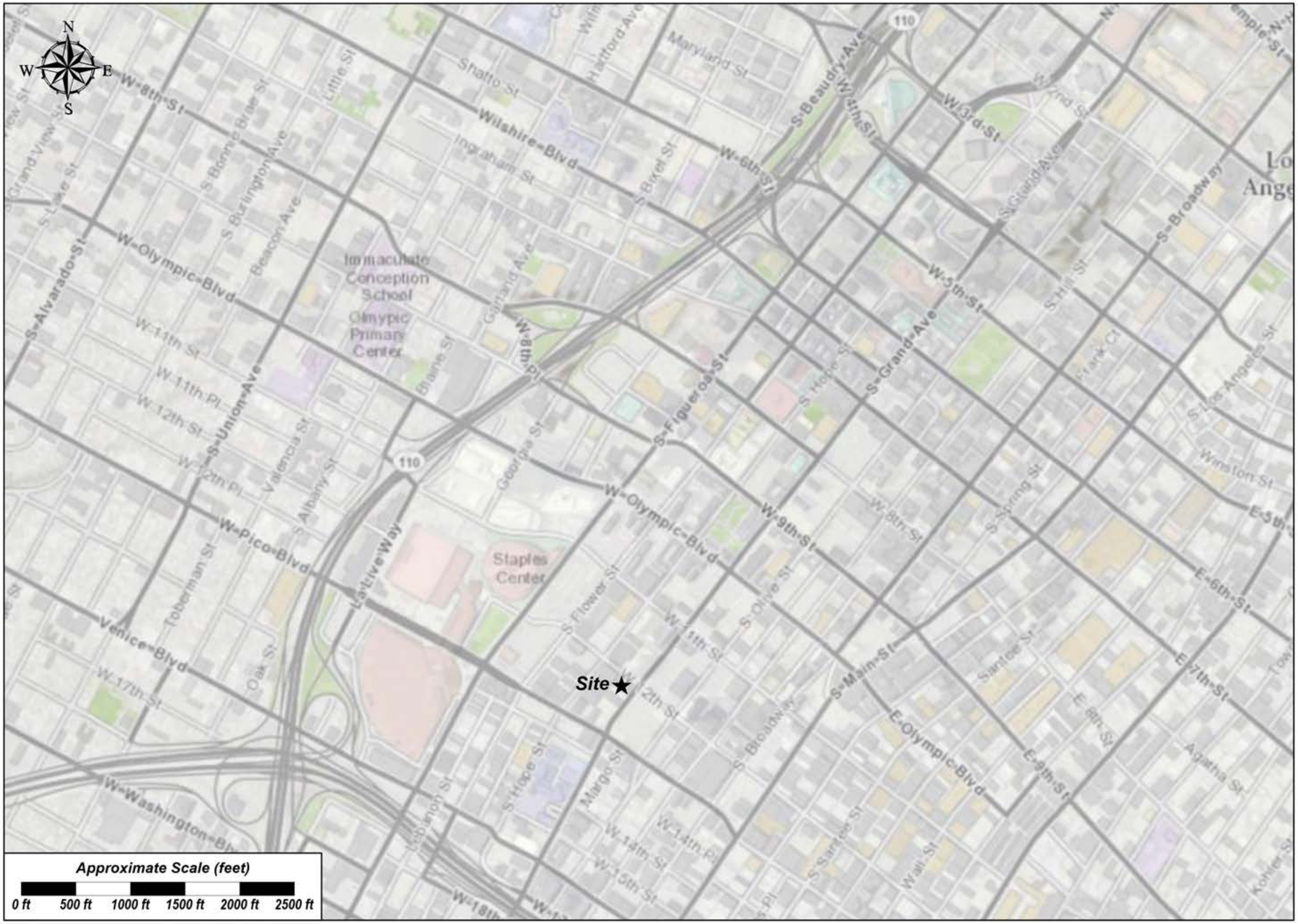
Oskin, M., Sieh, K., Rockwell, T., and others, 2000, Active parasitic folds on the Elysian Park anticline, Los Angeles, California: Implications for seismic hazard in central Los Angeles: Geological Society of America Bulletin, v. 112, p. 693-707.

Shaw, J.H., and Shearer, P.M., 1999. An Elusive Blind-Thrust Fault Beneath Metropolitan Los Angeles: Science, vol. 283, p. 1516-1518.

Southern California Earthquake Center (SCEC), 2014, Significant Earthquakes and Faults: Historical Earthquakes & Significant Faults in Southern CA, available at [http://www.data.scec.org/significant/index.html] accessed August 2014.

United States Geological Survey (USGS), 2010, Quaternary fault and fold database for the United States, updated November 2010, available at [http//earthquakes.usgs.gov/regional/qfaults/].

Working Group on California Earthquake Probabilities (WGCEP) (2013). 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/].

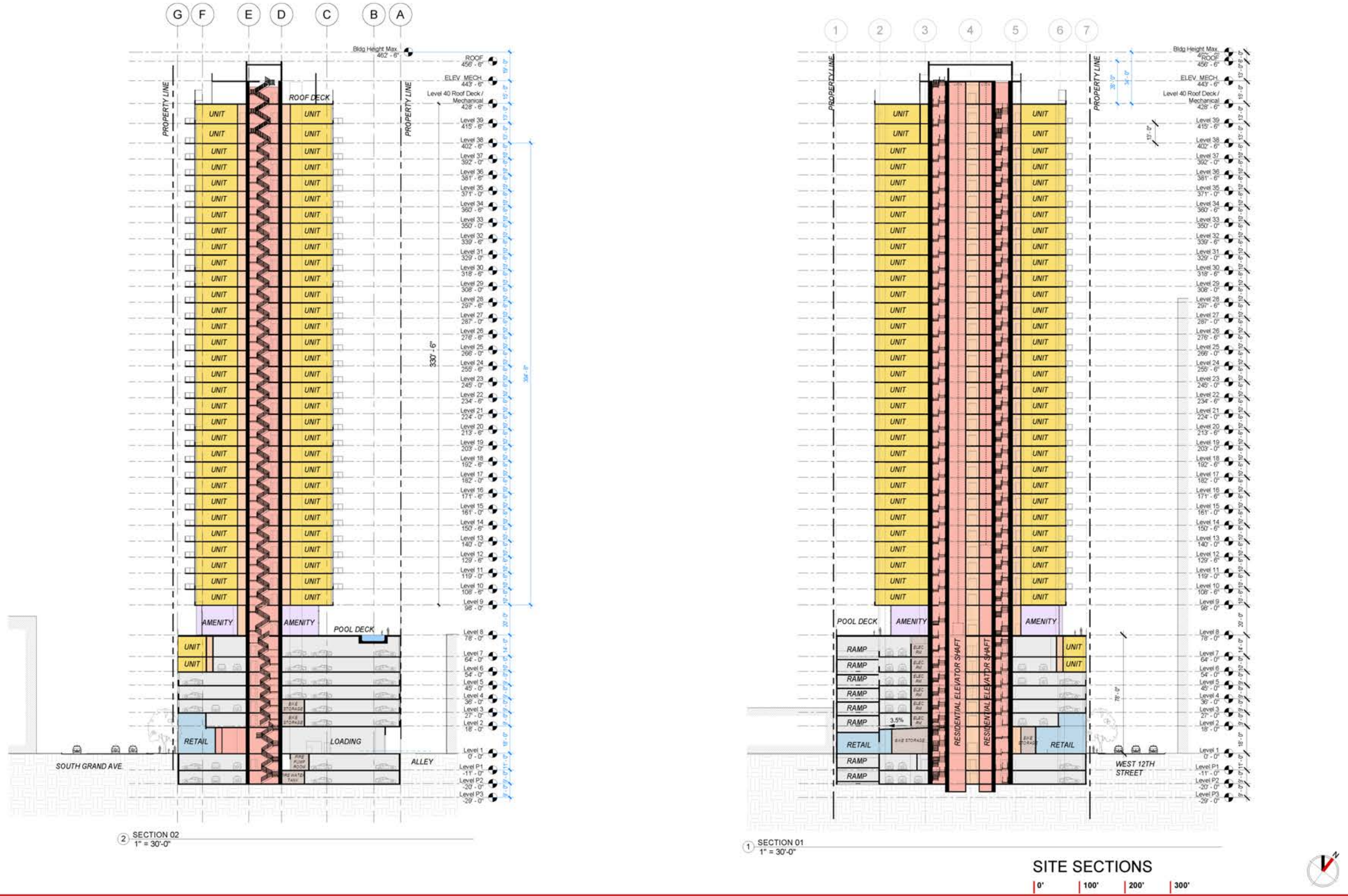


Site Location.srf

SITE LOCATION MAP

Date: MAY 2020	Project No.: 15083A	Project: 1201 S. GRAND AVENUE	Figure 1

GeoPentech



City Century

1201 SOUTH GRAND

LOS ANGELES, CA 90015

MVE - PARTNERS

PROJECT #1910129

5/04/2020

A3.1

Source:
1201 South Grand Entitlement plans by MVE+Partners dated 5/4/2020

PROPOSED DEVELOPMENT		
Project: 1201 S. GRAND AVENUE GEOTECH.		Figure 2a
Project No.: 15083A	Date: MAY 2020	

GeoPentech

Field Explor 8-11-15.srf

FIELD EXPLORATION LOCATIONS

Date: APR 2018	Project No.: 15083A	Project: 1201 S. GRAND AVENUE GEOTECHNICAL INVESTIGATION	Figure 2b

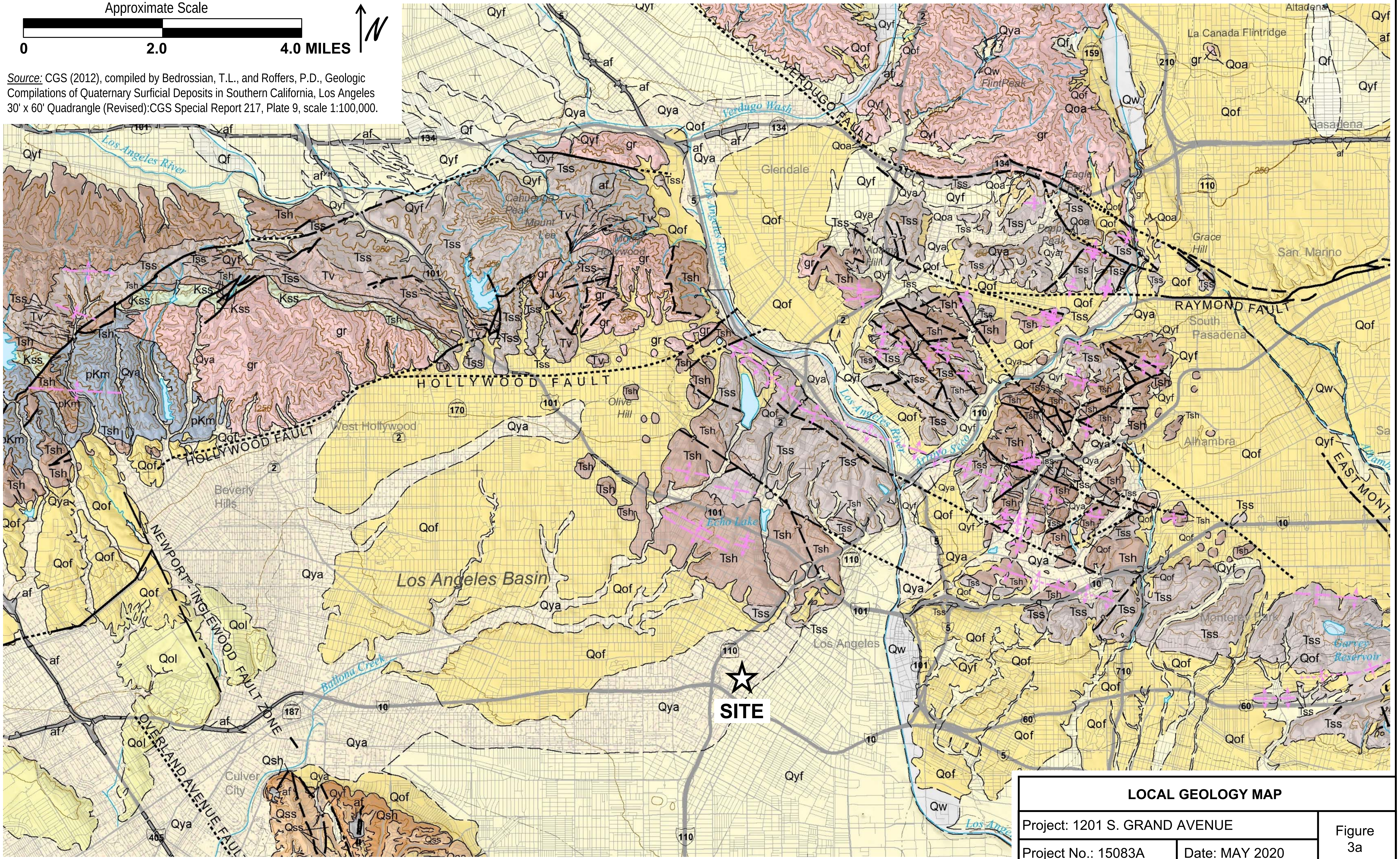
GeoPentech

Approximate Scale

0 2.0 4.0 MILES

N

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: 1201 S. GRAND AVENUE		Figure 3a
Project No.: 15083A	Date: MAY 2020	

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

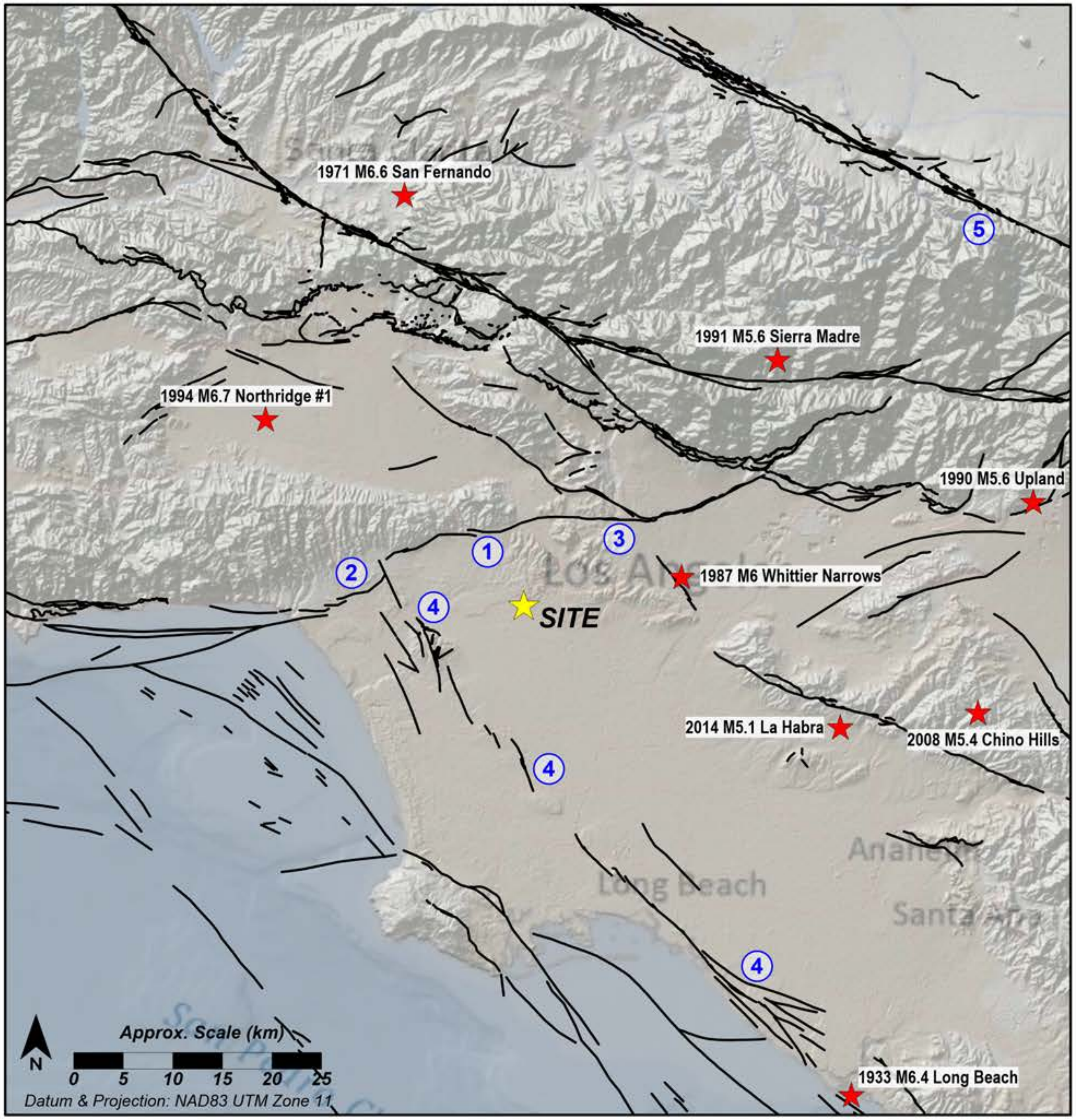
Spring

Road

County boundary

LOCAL GEOLOGY MAP LEGEND		
Project: 1201 S. GRAND AVENUE		Figure 3b
Project No.: 15083A	Date: MAY 2020	

GeoPentech



Legend

Surface Trace, Quaternary-active Fault

Significant Earthquake

1 HOLLYWOOD FAULT
2 SANTA MONICA FAULT
3 RAYMOND FAULT
4 NEWPORT-INGLEWOOD FAULT
5 SAN ANDREAS FAULT

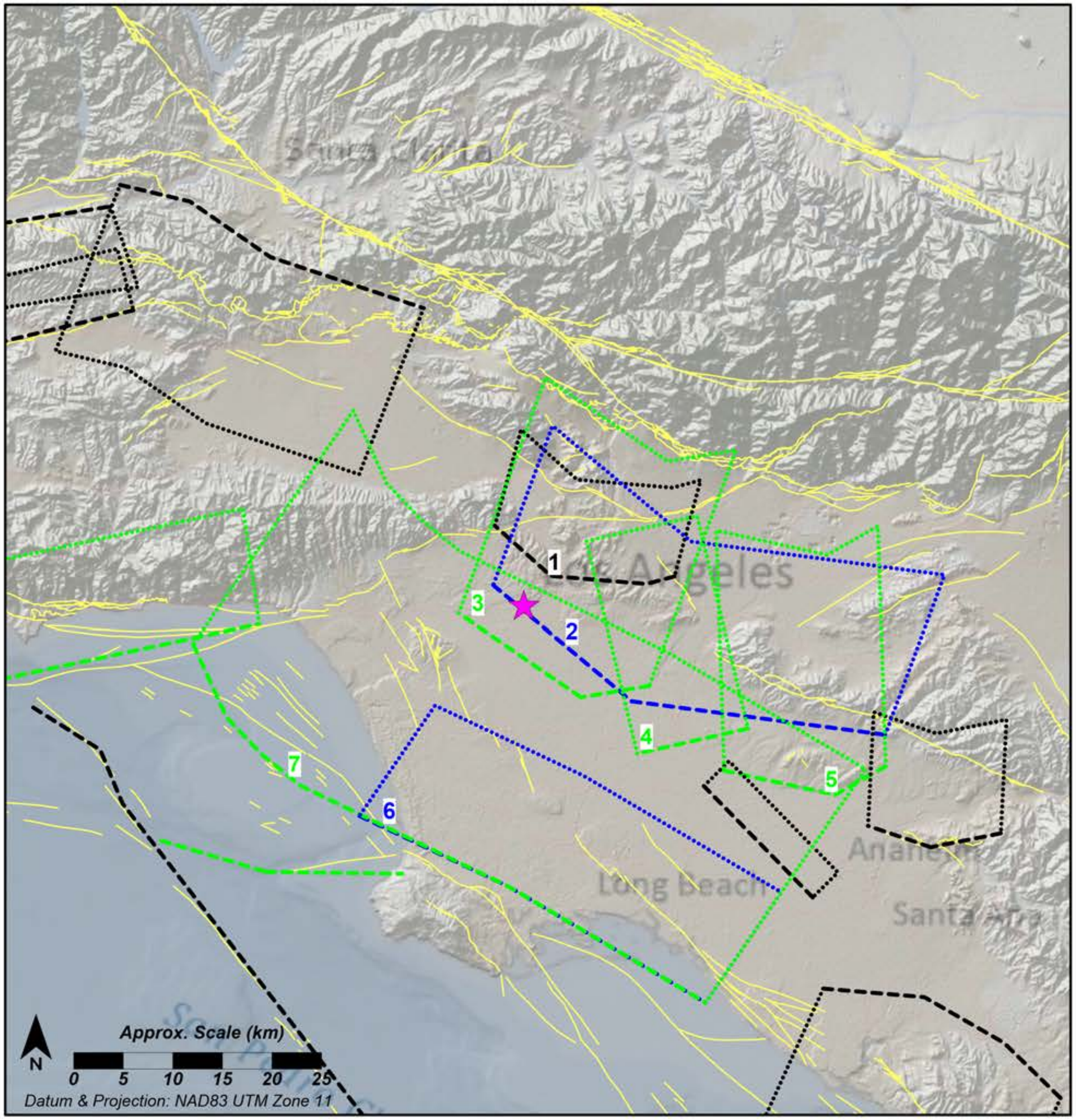
Note: Quaternary-active faults (i.e., faults with seismic activity in the last 1.6 million years) from USGS Fault & Fold Database (USGS, 2010). Significant post-1900 earthquakes, identified by name (white stars), from the Southern California Earthquake Center (SCEC) online database.

HISTORIC SEISMICITY MAP WITH QUATERNARY-ACTIVE FAULTS

Date: MAY 2020	Project No.: 15083A	Project: 1201 S. GRAND AVENUE	Figure 4a

GeoPentech

50km Seismicity_updated.srf



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

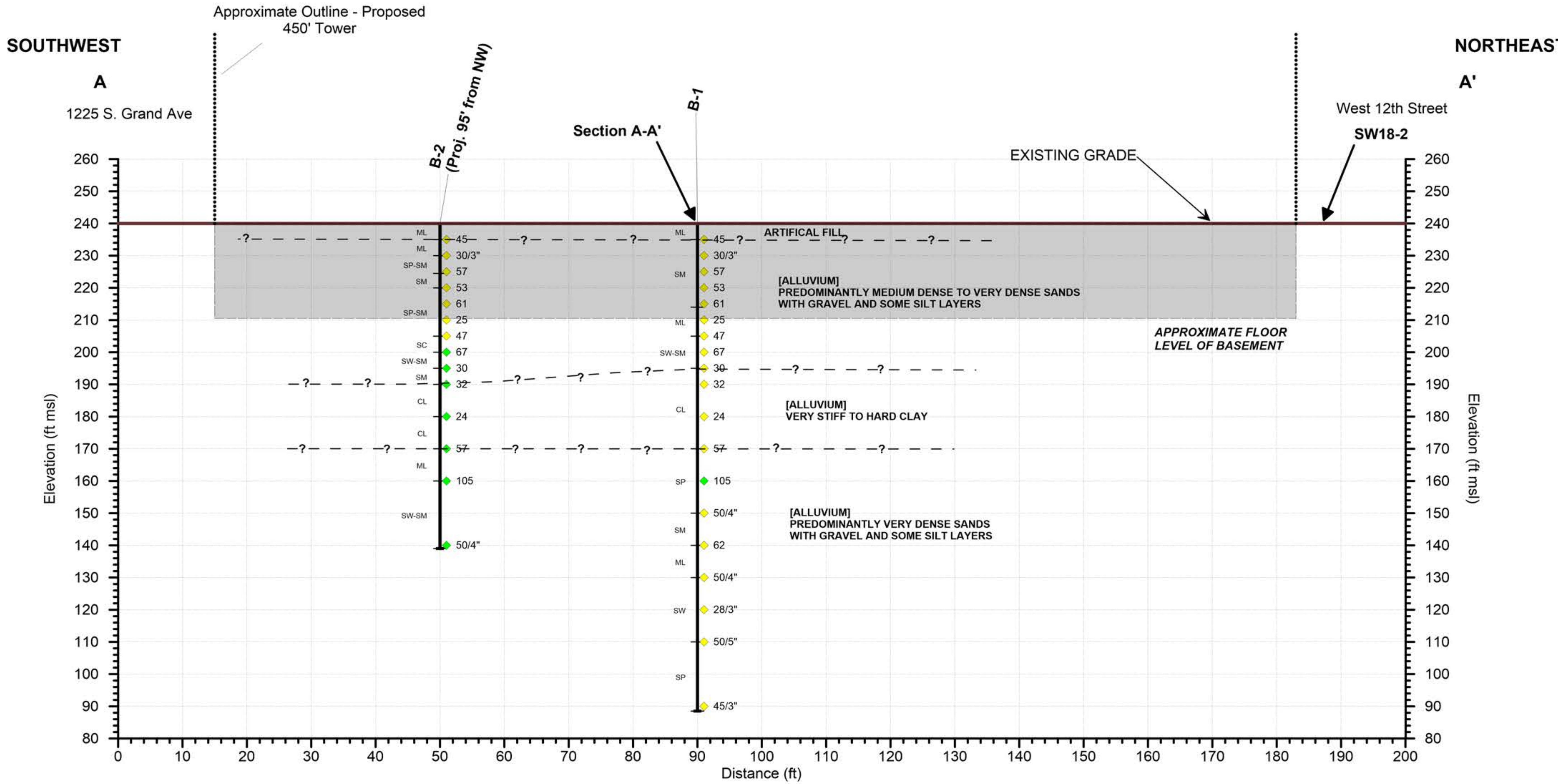
1 ELYSIAN PARK (UPPER) FAULT
2 PUENTE HILLS FAULT
3 PUENTE HILLS (LA) FAULT
4 PUENTE HILLS (SANTA FE SPRINGS) FAULT
5 PUENTE HILLS (COYOTE HILLS) FAULT
6 COMPTON FAULT ALT. 1
7 COMPTON FAULT ALT. 2

Note: 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).

50km Faults with Blind.srf

QUATERNARY-ACTIVE FAULTS & BLIND THRUSTS			
Date: MAY 2020	Project No.: 15083A	Project: 1201 S. GRAND AVENUE	Figure 4b

GeoPentech



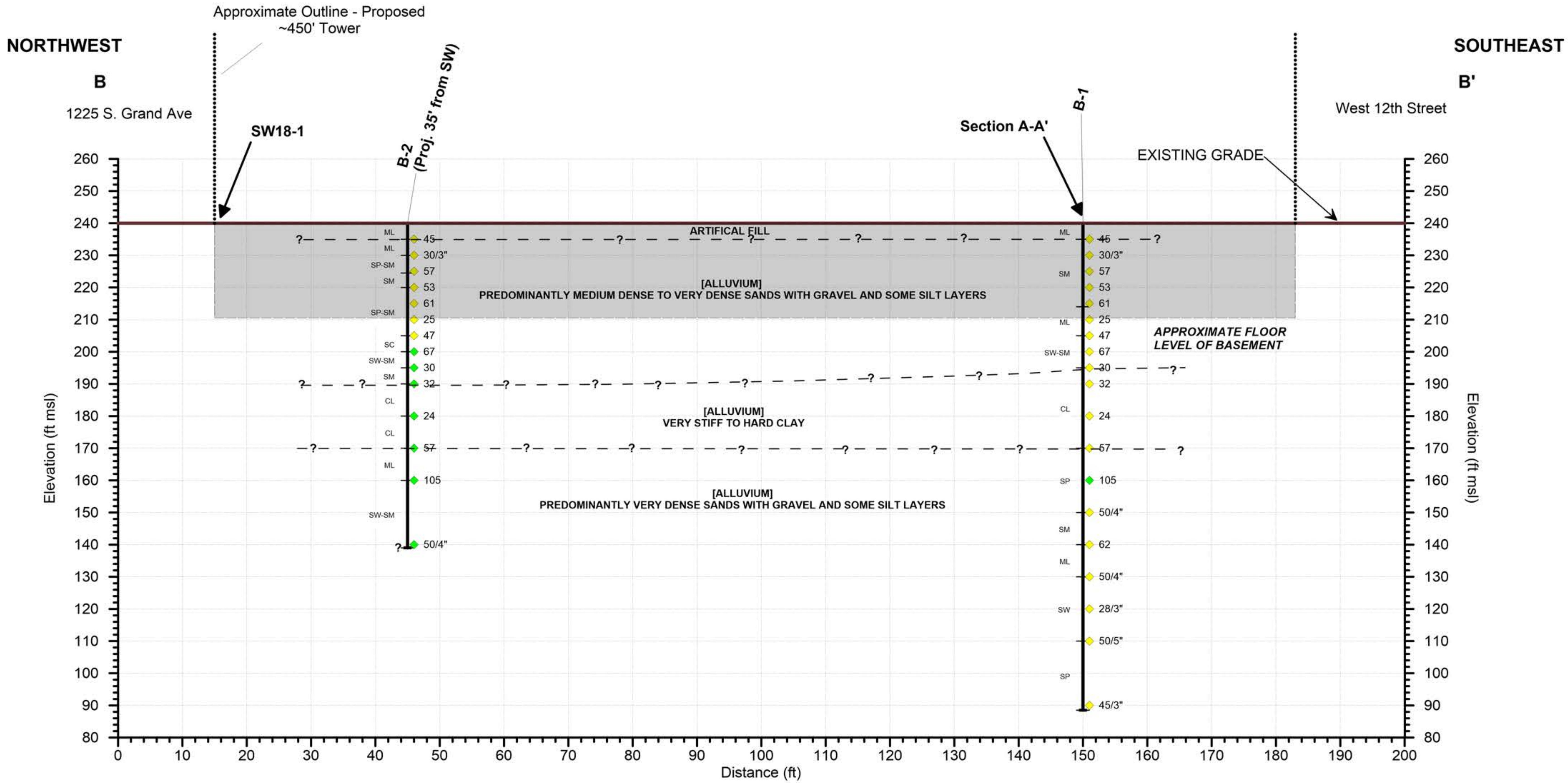
LEGEND

- ?- - -?- Geologic Contact (queried where uncertain)
- 26 Measured SPT Blowcount per foot
- 26 Measured Mod. CA Blowcount per foot

Notes:
Location of geologic cross-section shown on Figure 2.
2:1 Vertical to Horizontal Exaggeration.

GEOLOGIC CROSS SECTION A-A'		
Project: 1201 S. GRAND AVENUE GEOTECH.		Figure 5
Project No.: 15083A	Date: MAY 2020	

GeoPentech



LEGEND

–?– – –?– Geologic Contact (queried where uncertain)

◇ 26 Measured SPT Blowcount per foot

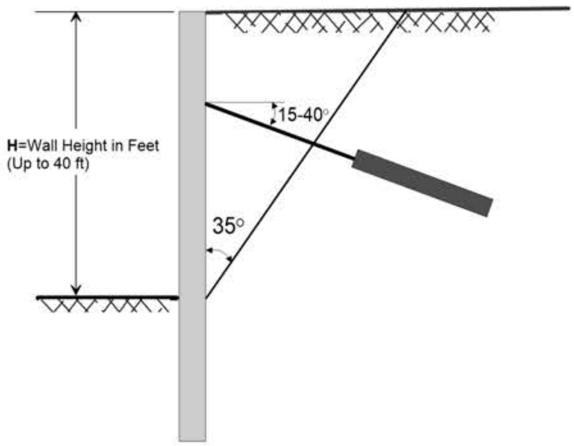
◆ 26 Measured Mod. CA Blowcount per foot

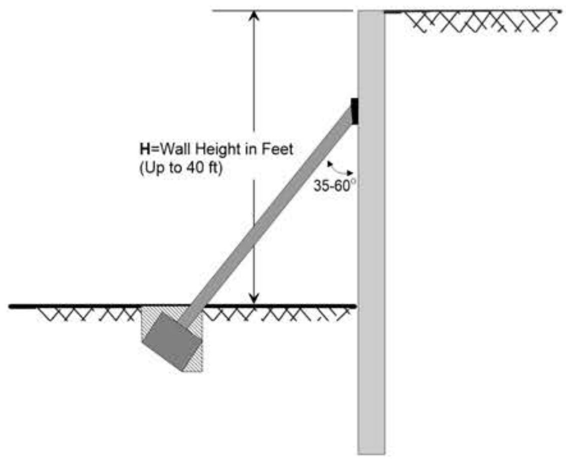
Notes:
Location of geologic cross-section shown on Figure 2.
2:1 Vertical to Horizontal Exaggeration.

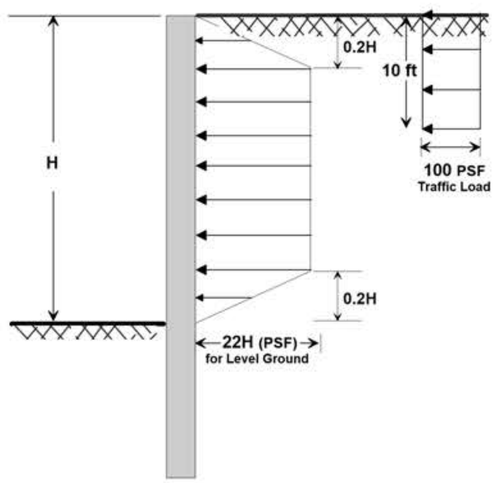
GEOLOGIC CROSS SECTION B-B'		
Project: 1201 S. GRAND AVENUE GEOTECH.		Figure 6
Project No.: 15083A	Date: MAY 2020	

GeoPentech

Braced/Tiedback Shoring







- Maximum Lateral Earth Pressure is 22H for level ground with a trapezoidal distribution.
- Tie-Back Anchor
  - 750 psf for gravity grouted anchors
  - 2,500 psf for pressure grouted anchors.
- Rakers - 4,000 psf for allowable bearing resistance. Extend min of 4 ft below adjacent grade.

Notes:

1. The lateral pressures above assume no build up of hydrostatic pressure behind the shoring.
2. Soldier Pile Design - Spaced at 2 diameters on center. Maximum tributary area of 3 pile diameters. Passive pressure below bottom of excavation is 500 psf per foot of depth up to a maximum value of 5,000 psf for piles installed without pre-drilling below the bottom of the excavation. If pre-drilling extends below the bottom of the excavation level, the passive pressure should be reduced to 350 psf per foot up to a maximum of 3,500 psf. Frictional resistance between soldier pile and soil below excavation is 500 psf.
3. Lagging - Design for full anticipated lateral pressure indicated above, but limit to a maximum value of 400 psf.
4. Traffic - A lateral pressure of 100 psf acting uniformly on the upper 10 feet can be used for traffic loads.
5. The shoring should be designed to resist applicable surcharge loads such as those from adjacent structures and stockpiled material.

APPENDIX A

FIELD EXPLORATION

Appendix A - Field Exploration

Introduction

The field investigations were performed between February 24, 2018 and March 3, 2018. The explorations consisted of advancing two (2) hollow-stem auger borings (B-1 and B-2), performing downhole seismic measurements in boring B-1, surface-wave geophysical measurements, and infiltration testing in boring B-2. The approximate locations of the borings and surface-wave geophysical measurements are indicated on Figure 2 in the main text.

Hollow-Stem Auger Drilling

Borings B-1 and B-2 were drilled on February 24, 2018 and March 3, 2018 using a truck-mounted CME 85 rig equipped with hollow-stem auger drilling equipment. An engineer monitored the drilling operations and prepared a field record of soils observed and drilling conditions. The drilling was subcontracted to BC2 Environmental, who provided all drilling equipment, crew, and supplies.

During drilling, soil samples were obtained at approximate depths with intervals ranging between 5 feet and 10 feet using either a Standard Penetration Test (SPT) sampler or a modified California (CA) sampler. SPT and CA samples were collected by driving a sampler approximately 18 inches into the soil at the bottom of the boring using a 140-pound auto hammer falling approximately 30 inches.

The SPT sampler used a cutting shoe and barrel with nominal inside diameters of 1.375 and 1.5 inches, respectively, and a nominal outside diameter of 2 inches. Liners were not used, but sand catchers were used in the cutting shoe for some SPT samples. The SPT samples were placed in sealed plastic bags and labeled as appropriate. The CA sampler cutting shoe and barrel have nominal inside diameters of 2.38 and 2.5 inches, respectively, and a nominal outside diameter of 3 inches. Nominal 6-inch long, 2.4-inch diameter brass tubes were used to line the barrel, and sand catchers were used in some CA samples. Plastic end caps were placed on the CA tubes to help preserve the moisture content of the samples. Select soil samples were tested at the geotechnical laboratory of Leighton in Irvine, California to evaluate their physical and pertinent engineering properties. Descriptions of the laboratory testing and the test results are presented in Appendix B.

After recovering the sample, the engineer noted the depth interval, recorded a description of the recovered soil 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 general accordance with the Unified Soil Classification System (USCS). Field observations were later updated with laboratory test results as appropriate.

Upon completion of drilling, logging, and sampling Boring B-1, a temporary 2-inch diameter PVC solid casing (i.e. no slotted screen portions) was installed to facilitate downhole seismic testing. The annular space was backfilled with cement-bentonite grout from the total depth of the hole up to approximately 5 feet below ground surface (i.e., cement-bentonite backfill between 5 feet and 150 feet bgs), about 3 feet of hydrated bentonite chips (i.e., hydrated bentonite chips between 5 feet and 2 feet bgs), and completed with concrete to the surface (i.e., concrete between 0 feet and 2 feet bgs). A steel plate was used to protect the top of the casing temporarily until abandonment of the casing the following weekend. Subsequently, downhole measurements were collected within the open casing and following collection of the measurements, the casing was abandoned by filling with cement-bentonite grout. The casing was filled by tremie completely from the bottom to the ground surface (i.e. from between 0 feet and 150 feet bgs).

During the drilling, logging, and sampling in Boring B-2, a temporary 2-inch diameter PVC casing with screen was used to complete infiltration testing as described below in the Infiltration Testing Section. After the percolation testing was completed, the temporary casing was removed, and the open borehole was abandoned with cement-bentonite grout to the ground surface with a concrete plug at the top to restore the ground surface.

The activities and findings of the borehole drilling and logging effort are provided along with a key on computer-generated boring logs.

Downhole Seismic Measurements

Downhole seismic tests were collected within Boring B-1 on February 28, 2018. The downhole seismic test method makes direct measurements of in-situ vertically propagating compression (P) and horizontally polarized shear (SH) wave velocities as a function of depth within the geologic material adjacent to a borehole. Measurement procedures followed ASTM D7400-08, "Standard Test Methods for Downhole Seismic Testing."

Downhole Seismic Methods and Procedures

A seismic source was used to generate a seismic wave (P or SH) at the ground surface. The seismic source was offset horizontally from the borehole a distance of 5 feet. The P-wave seismic source consisted of a ground plate that was struck vertically with a sledgehammer. The SH-wave seismic source consisted of an 8-foot long by 6 by 4-inch wood beam capped on both ends with a steel plate and loaded in place by the front end of a vehicle that was parked on top of the beam. The ends of this beam were positioned equidistant from the borehole. Initially, one end of the beam was struck horizontal with a sledgehammer to produce an SH-wave (forward hit). Next, the opposite end of the beam was struck horizontally with a sledgehammer to produce an opposite polarity SH-wave (reverse hit). The combination of the two opposite polarity SH-waves were used to determine SH travel times.

A downhole receiver positioned at a selected depth within the cased borehole was used to record the arrival of the seismic wave (P or SH). A three component triaxial borehole geophone (one vertical-channel and two orthogonal horizontal channels), which could be firmly pneumatically fixed against the PVC casing sidewall, was used to collect the downhole seismic measurements. Multiple downhole seismic measurements were performed at successive receiver depths within the borehole. The receiver depth was referenced to ground surface, and measurements were made at receiver intervals of 5 feet from the ground surface to the bottom of the hole (150 feet).

A Geometrics S12 signal enhancing seismograph was used to record the response of the downhole receiver. The seismic source (sledgehammer) contained a trigger that was connected to and initiated the seismograph recording, thus measuring the travel time between seismic source and downhole receiver. Downhole seismic test records were digitally recorded and stored with a 0.062 ms sample interval.

The recorded digital downhole seismic records were analyzed using the OYO Corporation program PickWin Version 4.1.1.7. The digital waveforms were analyzed to identify arrival times. The first prominent departure of the vertical receiver trace was identified as the P-wave first arrival. The SH-wave forward and reverse hits recorded on the two horizontal receiver channels were superimposed. The SH-wave first arrival was identified at the location of the first prominent relatively low-frequency departure of the forward hit and an 180° polarity change is noted to have occurred on the reverse hit. For analysis, no filter was applied to the P waveforms, and a 17 Hz low-cut filter and 134 Hz high-cut filter was applied to the SH waveforms.

After correcting the P and SH-wave travel time for the source offset, the P and SH-wave travel-times were plotted versus depth. P and SH layer and interval velocities were calculated as the slope of lines drawn through the plotted data.

Downhole Seismic Results

The results of the seismic downhole measurements collected within Boring B-1 are presented on Figure A-1. Figure A-1 shows (1) a table of the measured P and SH-wave travel-times and depths; (2) a table of the interpreted P and SH-wave layer velocities and depth ranges; (3) a table of the calculated P and SH-wave interval velocities; and (4) a plot of the P and SH-wave travel-times as a function of depth showing the interpreted layer velocities. Table A-1 summarizes the interpreted P and SH layer velocities and depths shown on Figure A-1 for the various geologic units logged in Boring B-1.

TABLE A-1
SUMMARY OF SH-WAVE AND P-WAVE VELOCITY LAYERS WITHIN BORING NUMBER B-1

PREDOMINANT LITHOLOGY	Depth Range (ft)	SH-WAVE Velocity (ft/sec)	P-WAVE Velocity (ft/sec)
Clayey Silt with sand (ML) [Fill]	0 to 5	670	1,500
Dense to very dense silty Sand to Sand with silt (SM to SW-SM) and Stiff to hard silty Clay (CL) to clayey Silt (ML) with sand [Alluvium]	5 to 65	1,360	2,660
Very dense silty Sand, Sand, and gravelly Sand (SM, SP and SW) and hard sandy Silt (ML) [Alluvium]	65 to 150	1,940	3,390

The V_{S30} was calculated based on the procedures outlined in the 2010 California Building Code, "2010 California Existing Building Code, Title 24, Part 10, Section 1613A.5.5 – Site Classification for Seismic Design." The V_{S30} was calculated from Equation 16A-40 of this reference which states:

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

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 V_{S30} for Boring B-1 was calculated between a depth of 0 to 100 feet and 35 to 135 feet. The results are summarized on Table A-2.

TABLE A-2
CALCULATED V_{S30} WITHIN BORING NUMBER B-1

DEPTH RANGE (ft, below ground surface)	V_{S30} (ft/sec)
0 to 100	1,410
35 to 135	1,700

Surface-Wave Geophysical Measurements

The surface-wave surveys were performed using Multi-channel Analysis of Surface Waves (MASW) and Refraction Microtremor (ReMi) methods. The geophysical surveys were performed along two survey lines (SW18-1 and SW18-2) at the approximate locations shown on Figure 2 of the main text. The purpose of the geophysical surveys was to measure seismic shear-wave (S-wave) velocities and depths to evaluate foundation properties.

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 provided in (Louie, 2001).

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.

Surface-Wave Geophysical Procedures

The MASW and ReMi investigations were performed at the site on February 28, 2018. These measurements were collected using a Geometrics S12 seismograph with a linear array of twelve, 4.5-Hz geophones. Geophones were linearly spaced at 10-foot or 15-foot intervals for the measurements.

For the MASW measurements, the active seismic source consisted of a sledgehammer blow to a ground plate. The MASW measurements were collected along a 12-channel array (either 110 or 165 feet long), and shots were performed at equal station intervals (either 10 or 15-feet) starting at the end geophone to 4 to 5 station intervals (50 to 60 feet) beyond the end geophone. At each shot location, the sledgehammer was hit approximately four to seven times and the resultant waveform was stacked. A 1,024-millisecond long record (0.5 millisecond sample interval) was recorded at each shot location. The recorded MASW data was subsequently processed using the program SurfSeis by Kansas Geological Survey. This program performs a wavefield transformation to convert the seismic data from time-distance space to frequency-phase velocity space. The highest amplitude energy in the frequency-phase velocity space was selected for the 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. The ReMi measurements were collected along a 12-channel array (about 165 feet long). For the ReMi measurements, a total of ten 32,768 millisecond long records (2 millisecond sample interval) were recorded at each survey location. The source of ambient surface wave energy was primarily from vehicular traffic travelling along the adjacent roadways. The recorded ReMi data was subsequently processed using the program SeisOpt ReMi by Optim Software. The program performs a slowness-frequency waveform transformation to the recorded surface wave records to separate Rayleigh waves from other seismic arrivals. The ReMi dispersion curves are picked as the lower bound envelope of the surface wave energy, which represents the slowest surface wave energy (highest slowness). In theory, the slowest identifiable surface wave energy represents the energy that is propagating parallel to the survey line, and higher velocity energy propagating oblique to the line would be seen as higher velocity.

For each line, the ReMi dispersion curve was combined with the dispersion curve generated from MASW for modeling. The degree of fit of the overlapping ReMi and MASW measurements 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. The results provide a one-dimensional vertical profile of S-wave velocity as a function of depth averaged beneath the area of the line.

Surface-Wave Geophysical Results

The results of the combined MASW and ReMi surface wave measurements are presented in Figures A-2 and A-3 for SW18-1 and SW18-2, 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.

Infiltration Testing

One (1) infiltration test was conducted in Borehole B-2 at depths of 75 to 80 feet below ground surface at the end of drilling. The soil boring log at the end of this appendix provides a description of the soils sampled from the test interval. Samples collected within the tested zone were evaluated for grain size distribution at the geotechnical laboratory of Leighton Group in Santa Ana, California. Descriptions of the laboratory testing and the test results are presented in Appendix B.

Figure A-4 shows the infiltration test setup schematically. To facilitate infiltration testing in Boring B-2, temporary 2-inch diameter PVC casing with a 5-foot interval of slotted casing (20-slot) was installed between depths of 75 feet and 80 feet, and solid casing was installed between the ground surface and 75 feet. The annular space around the slotted interval was filled with pea gravel to maintain the stability of the borehole during testing, and the augers were pulled back to the top of the sand-packed interval. An approximately 2-foot thick layer of bentonite chips was used to seal the top of the test interval. Water was pumped into the PVC casing, allowing the water level to rise within the sand pack multiple times and saturate the sediments adjacent the test section. When sediments adjacent the boring were close to saturation, changes in water level in the test section were recorded using an electronic water level transducer/data logger that was placed near the base of the PVC casing. Once the rate of water level decline was relatively stable, the final rate of water level decline was measured with the transducer/data logger system. These data are used to evaluate the infiltration capacity of the tested zone. Figure A-5 shows the results of three test measurements, all of which show consistent rates of water level decline.

Infiltration Rate Evaluation

The infiltration rate of the sediments immediately adjacent to the test interval was estimated from the rate of decline of the water that was added during the infiltration testing. These results are used to assess the potential for using drywells to discharge stormwater at the site.

Under partially saturated conditions, such as for the planned stormwater drywell system, the estimated infiltration rate from the test results provides a reasonable value for use in designing the infiltration system. Typically, the volumetric approach is used to provide a corrected infiltration rate (e.g., see the Standard Urban Stormwater Mitigation Plan [SUSMP], 2000); however, due to the high infiltration capacity of the materials in the test interval, water added to the casing discharged at a rate equal to the flow into the casing, such that changes in water level occurred only within the sand-packed test interval; therefore a volumetric correction related to the decline in water level is not needed.

Infiltration Analysis Results

Based on observations during drilling Boring B-2, soils between about 50 feet and 70 feet below existing grade are predominantly clayey. These fine-grained soils (clay layers) between 50 feet and 70 feet in Boring B-2 have poor infiltration capacity. The upper 50 feet of the subsurface materials overlying the clay layers are also not considered suitable for infiltration because the underlying clay layer would act as a barrier to the water flow and would create perched water conditions. Such perched water conditions would cause the wetting of the underlying clay layer, and when saturated and subjected to heavy loading conditions from the tower structure, the clay layer may compress resulting in differential settlements beneath the tower structure. Based on this, the interval between 0 and 50 feet below existing grade is not considered suitable for stormwater infiltration.

Infiltration testing was completed in Boring B-2 between 75 feet and 80 feet below existing grade. Soils observed during drilling Boring B-2 indicate coarse-grained materials (sands) throughout the infiltration test depth. Laboratory test results for the samples collected at depths of 70 feet and 80 feet in Boring B-2 show sandy silt (with a fines content of 69%) and well-graded sand with silt and gravel (with a fines content of about 11%), respectively. The results of the infiltration testing (Figure A-5) indicate an infiltration rate of 19 inches/hour (in/hr). For comparison, the SUSMP (2000) indicates that infiltration in soils with infiltration rates less than 0.3 inches/hour is not feasible.

Groundwater was not observed in Boring B-1 at the site, which extended to a depth of 151.5 feet below existing grade. The historic groundwater table at the site has been reported at a depth of about 100 to 110 feet (CDMG, 1998).

Infiltration Analysis Conclusions

As discussed above, the interval between 0 and 70 feet below existing grade is not considered suitable for stormwater infiltration. Field tests conducted at Boring B-2 at the project site indicate the soils encountered from approximately 75 feet to 80 feet below existing grade are sandy and appear favorable for stormwater infiltration using an appropriately designed and maintained drywell drainage system. Table A-3 summarizes the infiltration test results.

TABLE A-3
SOILS AND INFILTRATION TEST RESULTS

Boring	Depth	Soil Type	Estimated Infiltration Rate inches/hour
B-2	75-80 ft	Sandy Silt to Well-Graded Sand with Silt and Gravel	19 in/hr

Based on these results, we recommend a design infiltration rate of about 19 in/hr for the sandy interval that may extend between depths of 70 to 90 feet below existing grade at and near Boring B-

2. (Note that the total depth of the planned dry wells will need to be at least 10 feet above the historic high groundwater table, which is estimated by the State to be 100 feet below ground surface.) If the design capacity of the proposed wells at these this location is insufficient, additional wells could be drilled within the site using this design infiltration rate and depth range. However, the planned locations should be brought to our attention for review prior to final design. Furthermore, if the well is proposed at a location other than near Boring B-2, the location should be brought to our attention for review.

Maintenance of the installed drywell(s) should be performed on a regular basis. Monitoring of the drywell effectiveness at discharging stormwater during large storms should also be performed. If changes in the infiltration rate are noted over time, this should be brought to our attention so we can re-evaluate the potential limitations of the system. Pre-treatment of the stormwater should be implemented to remove particulate matter and prevent clogging of the well.

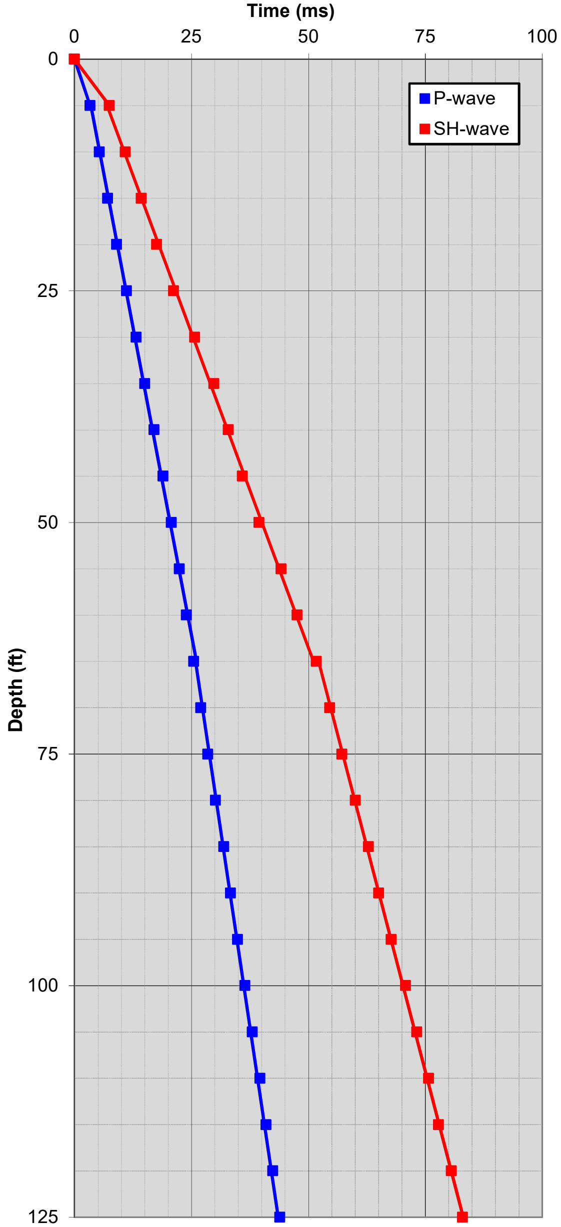
It should be noted that infiltrating stormwater can saturate the soils below the foundations of the proposed building as well as the adjacent area. The City SUSMP guidelines require infiltration wells to be set back from property lines a minimum of ten (10) feet.

SEISMIC WAVE TRAVEL TIMES

Depth (ft)	P-time (ms)	P-layer	SH-wave (ms)	SH-layer
0	0	1	0	1
5	4.72	12	10.50	12
10	5.91	2	12.12	2
15	7.49	2	15.08	2
20	9.26	2	18.11	2
25	11.37	2	21.59	2
30	13.36	2	26.08	2
35	15.21	2	30.09	2
40	17.13	2	33.12	2
45	19.01	2	36.08	2
50	20.80	2	39.63	2
55	22.49	2	44.37	2
60	24.02	2	47.75	2
65	25.55	23	51.82	23
70	27.03	3	54.68	3
75	28.56	3	57.29	3
80	30.23	3	60.15	3
85	31.99	3	62.95	3
90	33.40	3	65.16	3
95	34.89	3	67.81	3
100	36.48	3	70.84	3
105	38.08	3	73.24	3
110	39.67	3	75.78	3
115	41.02	3	77.87	3
120	42.39	3	80.61	3
125	43.94	3	83.03	3
130	45.17	3	85.22	3
135	46.58	3	88.28	3
140	47.78	3	91.21	3
145	48.97	3	93.40	3
150	50.29	3	95.74	3

Source Offset (ft)
5

TRAVEL TIME PLOT



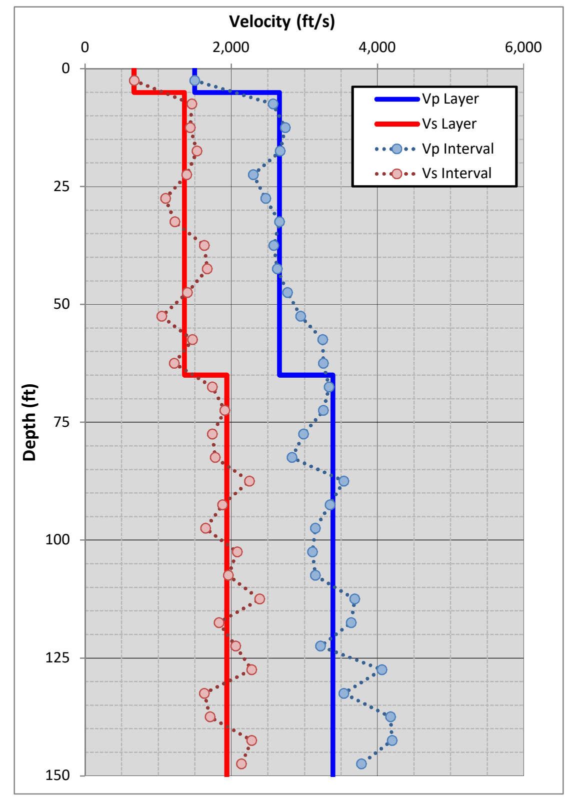
INTERVAL VELOCITES

Depth Range	P-Velocity (ft/s)	SH-Velocity (ft/s)
0 to 5	1,500	670
5 to 10	2,570	1,460
10 to 15	2,740	1,440
15 to 20	2,670	1,530
20 to 25	2,300	1,390
25 to 30	2,470	1,100
30 to 35	2,660	1,230
35 to 40	2,580	1,630
40 to 45	2,630	1,670
45 to 50	2,770	1,400
50 to 55	2,950	1,050
55 to 60	3,250	1,470
60 to 65	3,260	1,220
65 to 70	3,340	1,740
70 to 75	3,260	1,910
75 to 80	2,990	1,740
80 to 85	2,830	1,780
85 to 90	3,540	2,250
90 to 95	3,350	1,880
95 to 100	3,150	1,650
100 to 105	3,110	2,080
105 to 110	3,150	1,960
110 to 115	3,690	2,390
115 to 120	3,640	1,830
120 to 125	3,220	2,060
125 to 130	4,060	2,280
130 to 135	3,540	1,630
135 to 140	4,180	1,710
140 to 145	4,200	2,280
145 to 150	3,780	2,140

LAYER VELOCITES

Layer	P-Depth (ft)	P-Velocity (ft/s)	SH-Depth (ft)	SH-Velocity (ft/s)
1	0 to 5	1,500	0 to 5	670
2	5 to 65	2,660	5 to 65	1,360
3	65 to 150	3,390	65 to 150	1,940
4				
5				
6				
7				
8				
9				
10				

VELOCITY MODEL



V_{S30} CALCULATION

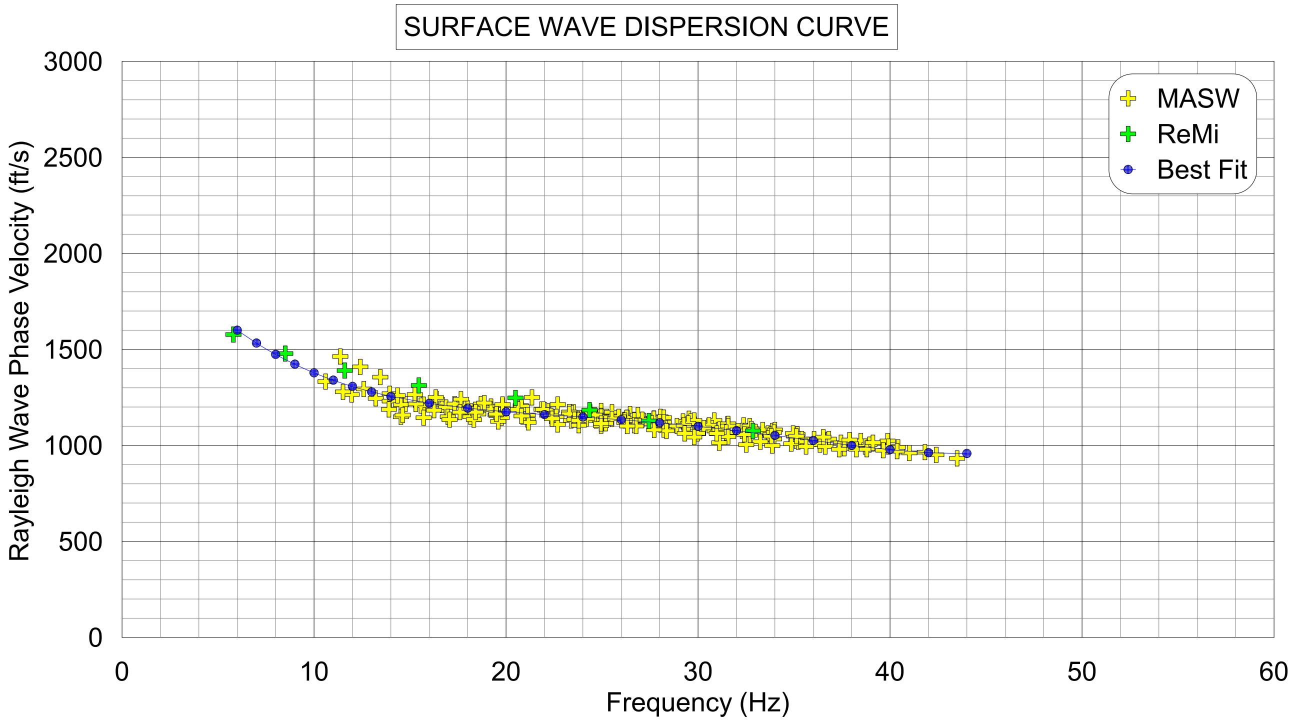
Vs30 (ft/s)	Depth (ft)
1,410	0 to 100
1,700	35 to 135

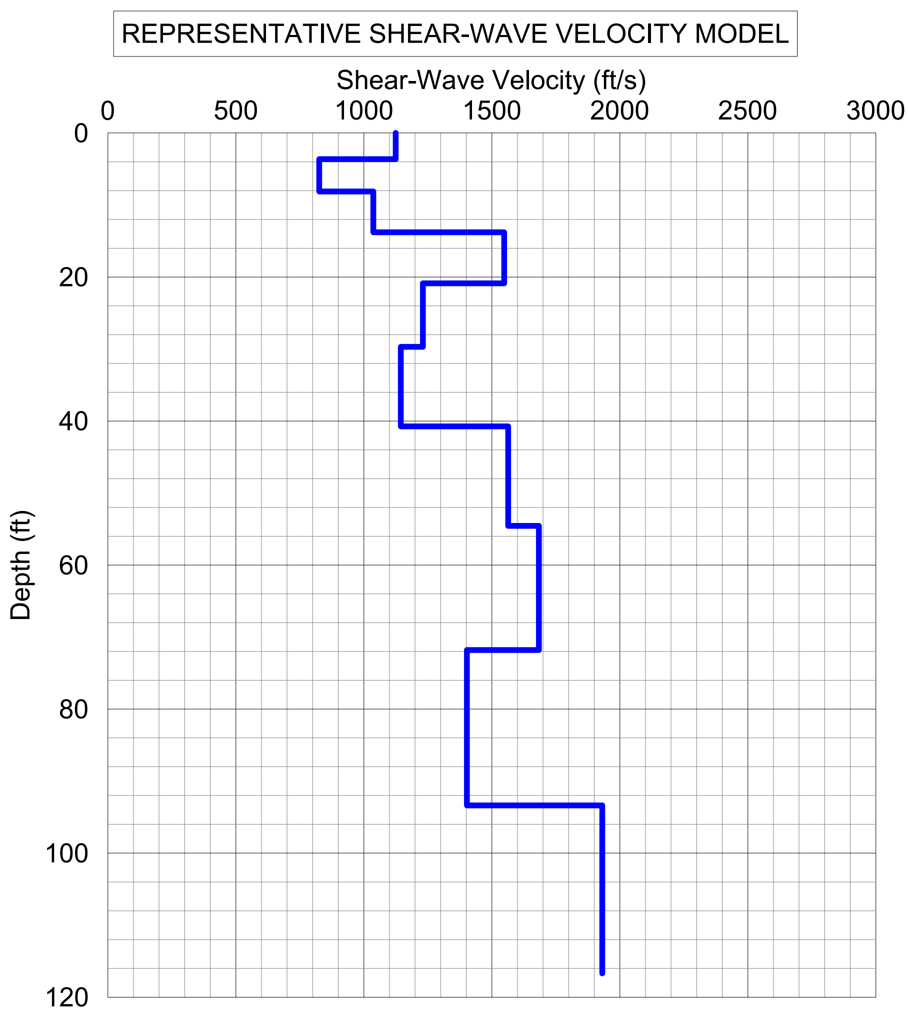
GeoPentech

SEISMIC DOWNHOLE TEST RESULTS
BORING B-1

PROJECT: 1201 SOUTH GRAND AVENUE

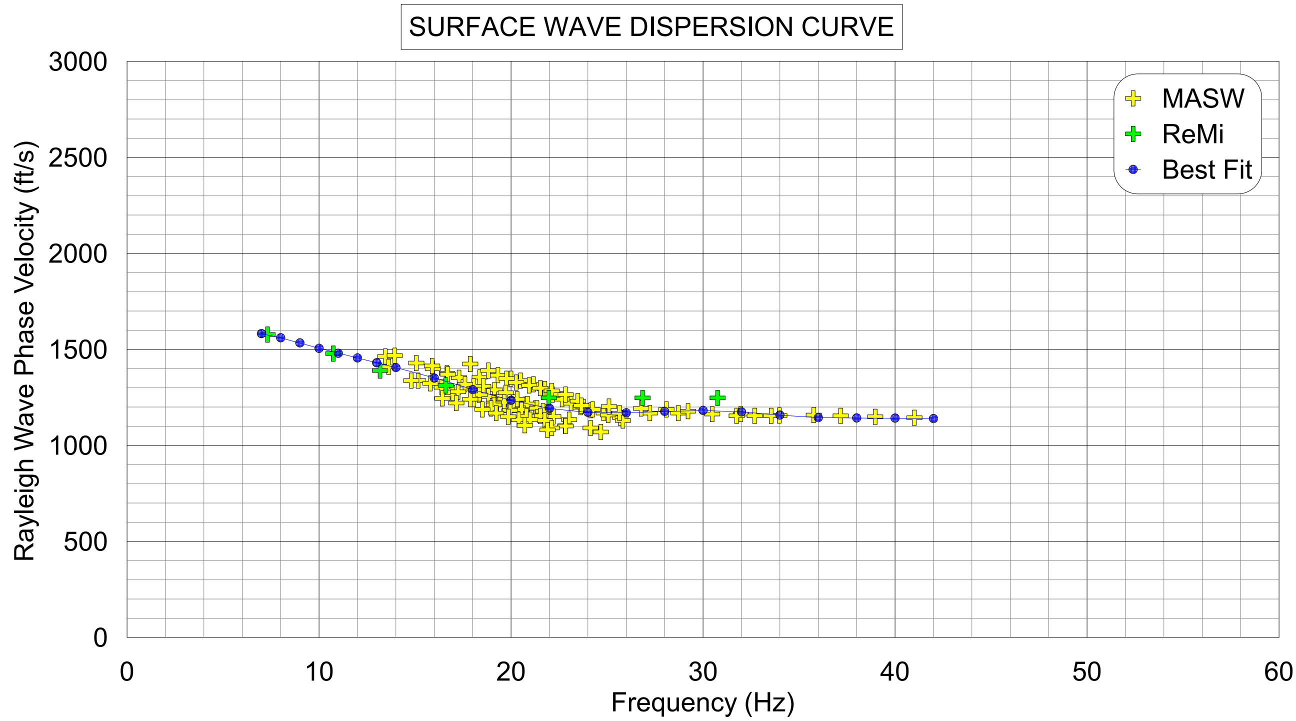
PROJECT #: 15083A | DATE: APR 2018 | FIGURE: A-1

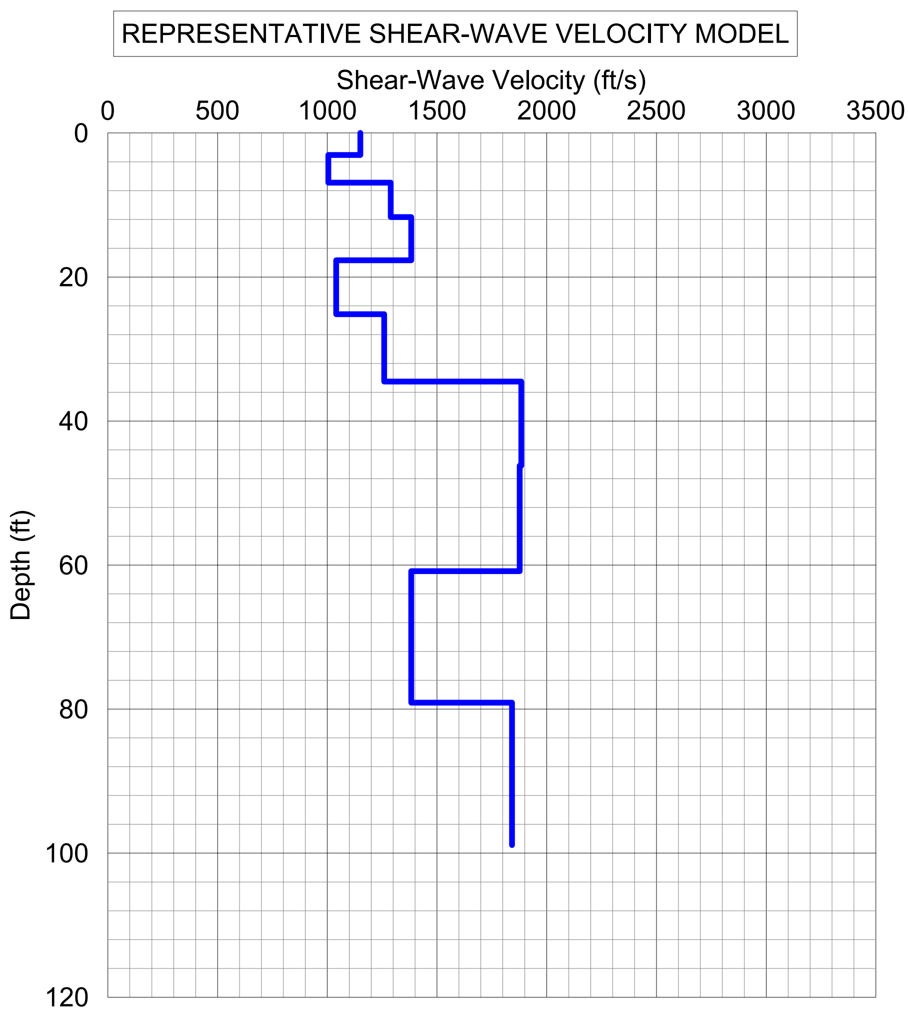




SW18-1: S-WAVE VELOCITY COMBINED SOURCE MODEL			
Project No.: 15083A	Project: 1201 SOUTH GRAND AVENUE	Date: APR 2018	Figure A-2

GeoPentech





SW18-2: S-WAVE VELOCITY COMBINED SOURCE MODEL

Project No.: 15083A	Project: 1201 SOUTH GRAND AVENUE	Date: APR 2018	Figure A-3

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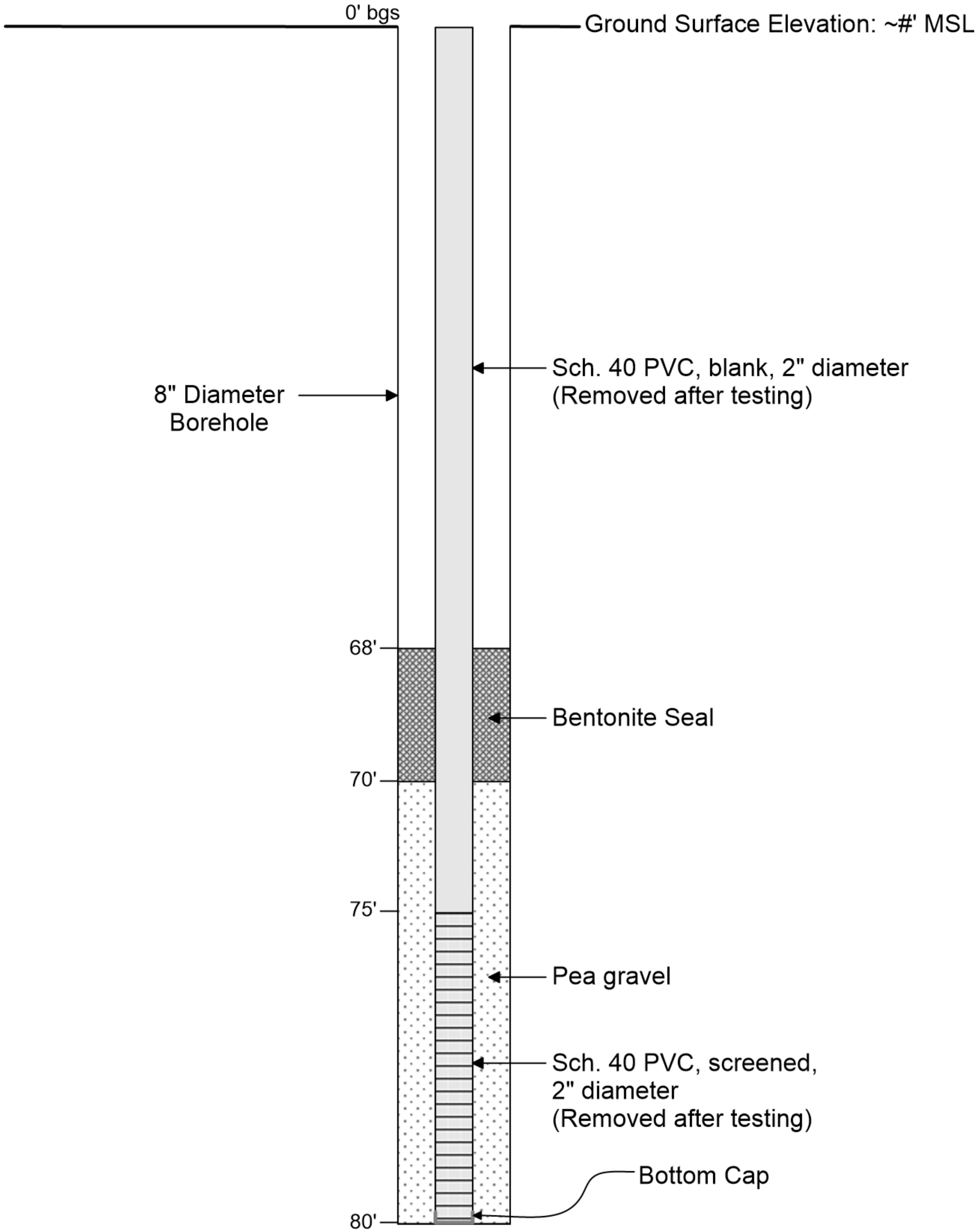
Location: Northeast side of existing parking lot (see Fig 2)	**Date Installed:** 3/3/18	**Method:** See Text
Installed By: BC2 Environmental	**Observed By:** Sean Munter	**Total Depth (ft bgs):** 80
Screened Interval: 75-80'	**Remarks:**	

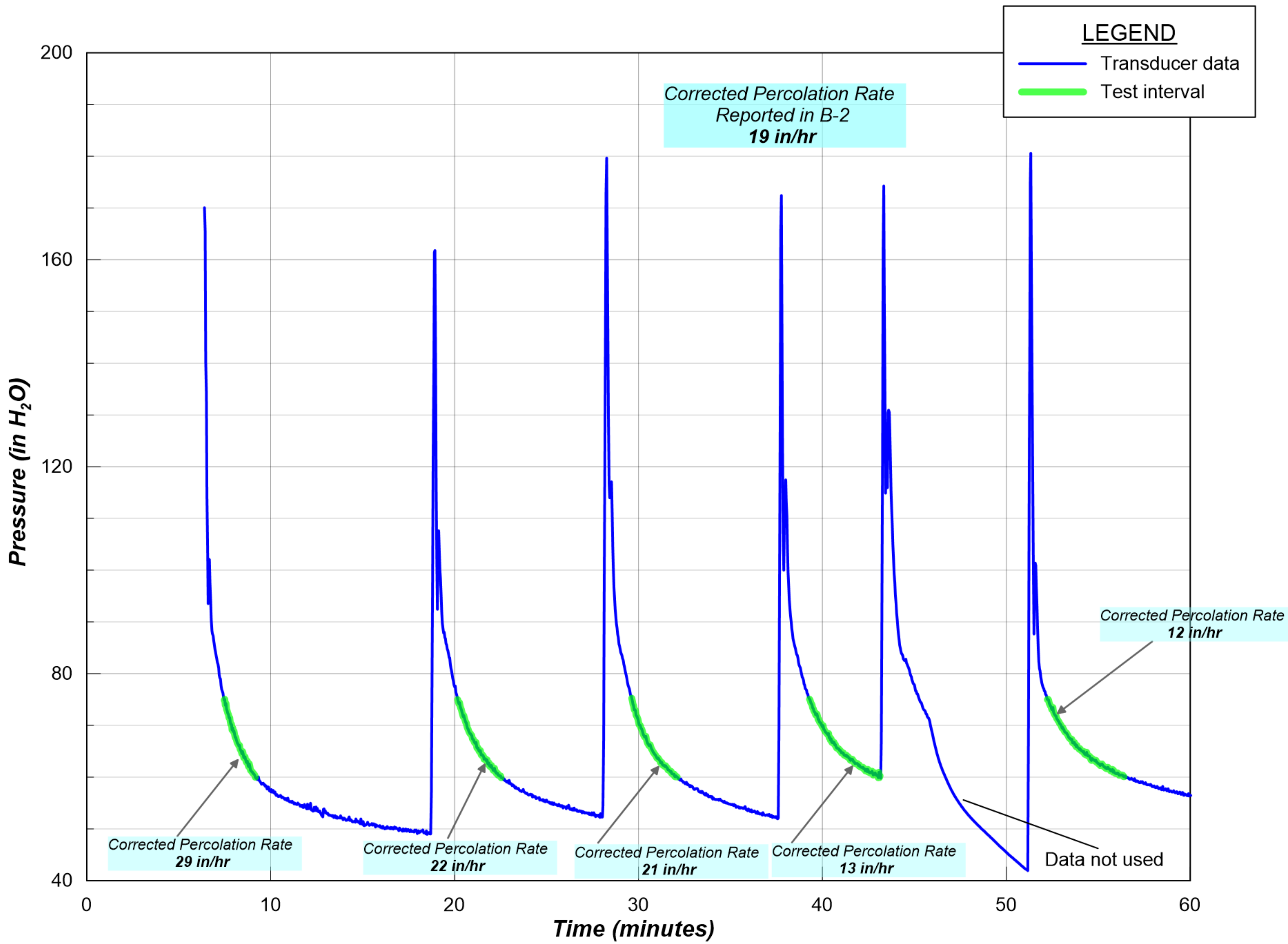


Note: Drawing not to scale

DIAGRAM FOR PERCOLATION TEST IN BORING B-2

Project No.: 15083A	Project: 1201 S. GRAND AVENUE GEOTECHNICAL INVESTIGATION	Date: Apr 2018	Figure A-4

GeoPentech



PERCOLATION TEST DATA FOR BORING B-2

Date: APR 2018	Project No.: 15083A	Project: 1201 S. GRAND AVE	Figure A-5

GeoPentech

Project: 1201 South Grand Avenue - Geotechnical Investigation Project Location: Los Angeles, CA Project Number: 15083A	Key to Log of Boring Sheet 1 of 1

Elevation, feet	Depth, feet	SAMPLES Type	Number	Blows / 6"	Recovery, %	Graphic Log	MATERIAL DESCRIPTION	Water Content, %	Dry Unit Weight, pcf	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 over inches driven

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

Asphalt

Concrete

Well-graded SAND (SW)

Poorly-graded SAND (SP)

Well-graded SAND with Silt (SW-SM)

Poorly-graded SAND with Silt (SP-SM)

Silty SAND (SM)

Clayey SAND (SC)

Lean CLAY (CL)

SILT (ML)

TYPICAL SAMPLER GRAPHIC SYMBOLS

California Modified Sampler

Standard Penetration Test

OTHER LABORATORY TEST ABBREVIATIONS

COMP	Compaction by modified effort (ASTM D1557)
CONS	One-dimensional consolidation test (ASTM D2435)
CORR	Chemical tests to determine soil corrosivity
DS	Consolidated drained direct shear test (ASTM D3080)
EI	Expansion Index (ASTM D4829), EI at 50% saturation
ER	Minimum soil electrical resistivity (DOT CA 532/643)
FC	Fines Content (ASTM D1140), % <#200 sieve
SA	Sieve Analysis (ASTM D422), % <#200 sieve
HYD	Hydrometer Analysis on fine-grained soils
LL	Liquid Limit from Atterberg Limits test (ASTM D4318)
PI	Plasticity Index; NP indicates non-plastic determination

OTHER GRAPHIC SYMBOLS

—— Contact between strata

– – – Inferred contact between strata or gradational change

Change within material properties within a stratum

Depth of note

Soil classifications are based on the Unified Soil Classification System. Descriptions and stratum lines are interpretive; field descriptions may 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.

Project: 1201 South Grand Avenue - Geotechnical Investigation Project Location: Los Angeles, CA Project Number: 15083A	**Log of B-1** Sheet 1 of 5

Date(s) Drilled	**02/24/2018**	Logged By	**D. Wahl**	Checked By	**S. Tatusian**
Drilling Method	**Hollow Stem Auger**	Drill Bit Size/Type	**8" Bullet Bit**	Total Depth of Borehole	**151.5 feet**
Drill Rig Type	**CME - 85**	Drilling Contractor	**BC2 Environmental**	Approximate Surface Elevation	**240'**
Groundwater Level(s)	**Not Observed During Drilling**	Sampling Method	**SPT, Cal Mod**	Hammer Data	**Automatic hammer 140lbs/30" drop**
Borehole Location	**~34' South of Building and 29' West of Grand Gate**	Borehole Completion	**Backfilled with cement-bentonite grout (See end notes for details)**		

Elevation, feet	Depth, feet	SAMPLES Type	Number	Blows / 6"	Recovery	Graphic Log	MATERIAL DESCRIPTION	Water Content, %	Dry Unit Weight, pcf	REMARKS
240	0						**ASPHALT**, patch, 1 - 3" thick, cobbles and gravel to 12" diameter, granite cobble, mechanically broken, angular debris, brick fragments [FILL] **Clayey SILT with Sand (ML)**, moist, low to medium plasticity, stiff to hard			Hand auger to 5' bgs
235	5		1	7 20 25			[ALLUVIUM] **Silty SAND (SM)**, dense to very dense, moist, reddish yellow (5YR 7/6), fine to coarse gravel to 1" diameter, subrounded to angular, granitic, trace clay; no reaction to HCl			Color change at about 6', light orange brown
230	10		2	8 30/3"				1.8		Double hits
225	15		3	12 35 22			becomes strong brown (7.5YR 4/6)			Double hits
220	20		4	15 30 23			becomes brown (7.5YR 5/4), cobbles, mechanically broken gravel	4.0		SA: 11% < #200
215	25		5	25 35 26			**Clayey SILT with Sand (ML)**, stiff, moist, strong brown (7.5YR 5/8), low to nonplastic			Easier driving, last 4", 1 hit per inch
210	30									

Elevation, feet	Depth, feet	Type	Number	Blows / 6"	Recovery	Graphic Log	MATERIAL DESCRIPTION	Water Content, %	Dry Unit Weight, pcf	REMARKS
210	30		6	6 11 14			becomes stiff to very stiff, strong brown (7.5YR 5/8), low to medium plasticity, less gravel; mottled with greyish brown (10YR 5/2) ~35%	15.6		
205	35		7	12 23 24			**SAND with Silt (SW-SM)**, dense to very dense, moist, reddish yellow (5YR 6/6), coarse SAND, less gravel, trace subangular gravel to 3/8"			
200	40		8	32 29 38			becomes with trace iron oxide staining	5.0		SA: 9% < #200
195	45		9	7 15 15			**Silty CLAY with Sand (CL)**, very stiff to hard, moist, very dark grayish brown (2.5Y 3/2), low plasticity, interbedded with **Silty SAND (SM)**, dense, moist, fine to coarse SAND; mottled with olive brown (2.5Y 4/3)	15.1		SA: 61% < #200 LL = 38 PI = 26 ER = 1400 ohm-cm CORR
190	50		10	8 16 16						
185	55									
180	60		11	7 10 14			becomes very stiff, mottled with orange and gray and very dark brown (7.5YR 2.5/3), medium plasticity, trace fine SAND; some iron oxide staining	24.0		LL = 50 PI = 33
175	65									

Elevation, feet	Depth, feet	SAMPLES Type	Number	Blows / 6"	Recovery	Graphic Log	MATERIAL DESCRIPTION	Water Content, %	Dry Unit Weight, pcf	REMARKS
175	65									
170	70		12	14 27 30			**Poorly-graded SAND (SP)**, very dense, moist, yellowish brown (10YR 5/4), fine to medium SAND, trace subrounded gravel to 1/4"	3.8		
165	75									Driller notes difficult drilling @ ~75' bgs; possible cobble
160	80		13	48 52 53			becomes with some silt; no reaction to HCl	2.7	104.0	DS Rig chattering @ ~82' bgs
155	85									
150	90		14	50/4"			**Silty SAND (SM)**, very dense, moist, light yellowish brown (2.5Y 6/4), fine to coarse SAND; abundant granitic clasts, decomposed; some iron oxide staining; thin (~1") interbedded SILT (ML), hard	3.6		subrounded gravel in cuttings; ~1/2" to 3/4" diameter @ 89' bgs
145	95									
140	100									

Elevation, feet	Depth, feet	SAMPLES Type	Number	Blows / 6"	Recovery	Graphic Log	MATERIAL DESCRIPTION	Water Content, %	Dry Unit Weight, pcf	REMARKS
140	100		15	15 27 35			**Sandy SILT (ML)**, hard, moist, light olive gray (5Y 6/2), nonplastic, very fine sand, finely bedded to laminated *(the rought int.?)*; some iron oxide staining; mottled with orangish brown	23.2		SA: 74% < #200
										Cobble fragments in shoe (granitic)
135	105									
										added water @ ~108' bgs to facilitate drilling gravel fragments in shoe
130	110		16	34 50/4"			**Well-graded SAND (SW)**, very dense, moist, reddish brown (5YR 4/4), fine to coarse SAND, subrounded, some subrounded gravel from 1/4" to 1" diameter, mechanically broken granitic clasts; some iron oxide staining	5.5		
125	115									Add water @ 115' bgs
120	120		17	36 28/3"			**Gravelly SAND (SW)**, very dense, moist, yellowish red (5YR 4/6), fine to coarse grained SAND, subrounded, gravel subangular to angular to 3/4" diameter	6.4		
115	125									
110	130		18	18 50/5"			**Poorly-graded SAND (SP)**, very dense, moist, strong brown (7.5YR 5/6), fine SAND			
105	135									

Elevation, feet	Depth, feet	Type	Number	Blows / 6"	Recovery	Graphic Log	MATERIAL DESCRIPTION	Water Content, %	Dry Unit Weight, pcf	REMARKS
105	135							7.5		SA: 6% < #200
										Add water @ 137' bgs
100	140									
95	145									Add water @ 145' bgs
90	150		19	37 45/3"			becomes **Poorly Graded SAND with Silt (SP-SM)**	6.2		
							Total Depth = 151.5' bgs			
							On 2/25/2018 Installed 2" Black PVC with end cap to full depth Lean cement-bentonite grout used to fill annulus On 3/03/2018 PVC subsequently filled to surface with lean cement grout and upper 5' removed Surface restored with concrete patch			
85	155									
80	160									
75	165									
70	170									

Project: 1201 South Grand Avenue - Geotechnical Investigation
Project Location: Los Angeles, CA
Project Number: 15083A

Log of B-2

Date(s) Drilled	**03/03/2018**	Logged By	**S. Munter**	Checked By	**S. Tatusian**
Drilling Method	**Hollow Stem Auger**	Drill Bit Size/Type	**8"- Diameter**	Total Depth of Borehole	**101.0 feet**
Drill Rig Type	**CME - 85**	Drilling Contractor	**BC2 Environmental**	Approximate Surface Elevation	
Groundwater Level(s)	**Not Observed During Drilling**	Sampling Method	**SPT, Cal Mod**	Hammer Data	**Automatic hammer 140lbs/30" drop**
Borehole Location		Borehole Completion	**Backfilled with cement-bentonite grout; capped with concrete over bentonite chips (See end notes for details)**		

Elevation, feet	Depth, feet	SAMPLES Type	Number	Blows / 6"	Recovery	Graphic Log	MATERIAL DESCRIPTION	Water Content, %	Dry Unit Weight, pcf	REMARKS
	0						**ASPHALT**			
							CONCRETE			
							[FILL] **Sandy SILT with Gravel (ML)**, medium dense, slightly moist grading to moist with depth, very dark grayish brown (10YR 3/2), slightly plastic SILT, fine to coarse sand, fine to coarse gravel to 3" fragments of brick and concrete; no reaction to HCl			
	5		1	5 6 7	13.5"		**[ALLUVIUM]** **SILT with Sand (ML)**, slightly moist, dark brown (10YR 3/3), slightly plastic SILT, fine to coarse sand, trace fine gravel; no reaction to HCl			
	10		2	5 7 9	8"		**Well-graded SAND with Silt and Grael (SW-SM)**, medium dense, slightly moist, brown (10YR 4/3), fine to coarse SAND, fine to coarse gravel to at least 1"; no reaction to HCl	1.8		Sample 2: Contains mechanically broken gravel
	15		3A 3B	8 50/5"	8"		becomes dark gray (5Y 4/1)			
							Silty SAND (SM), very dense, moist, light olive brown (2.5Y 5/3), fine to coarse SAND; no reaction to HCl			Sample 3A/3B: Contains mechanically broken gravel SA: 5.3% < #200 ER = 8775 ohm-cm CORR
	20		4	8 25 39	12"		**Well-graded SAND with Silt and Gravel (SW-SM)**, very dense, slightly moist, light olive brown (2.5Y 5/3), fine to coarse SAND, fine to coarse subrounded gravel of granitic rock to 3"; no reaction to HCl	4.7		Sample 4: Contains mechanically broken gravel
	25		5A 5B	25 50/6"			becomes moist			Sample 5A: Likely slough Sample 5B: Contains mechanically broken gravel
	30									

GeoPentech
Geotechnical & Geoscience Consultants

Elevation, feet	Depth, feet	Type	Number	Blows / 6"	Recovery	MATERIAL DESCRIPTION	Water Content, %	Dry Unit Weight, pcf	REMARKS
	30		6	13 33 50/4"	8.5"	becomes olive brown (2.5Y 4/3)	4.8		Sample 6: Contains mechanically broken gravel Gravel in cuttings are subrounded and up to 3" in size
	35		7	19 29 22	18"	**Clayey SAND with Gravel (SC)**, very dense, moist, yellowish brown (10YR 5/4), fine to coarse SAND, medium plastic clay, fine to coarse gravel of granitic rock; no reaction to HCl			Sample 7: Contains mechanically broken gravel
	40		8A 8B	29 50/3"	67% 100%	**SAND with Silt and Gravel (SW-SM)**, very dense, moist, yellowish brown (10YR 5/4), fine to coarse SAND, slightly plastic silt, fine to coarse gravel of granitic rock; no reaction to HCL	4.5	110.0	SA: 9% < #200 DS
	45		9A 9B	27 50/6"	100% 100%	**Silty SAND (SM)**, very dense, moist, yellowish brown (10YR 5/4), fine to medium SAND; no reaction to HCl	9.4	101.0	SA: 36.7% < #200 LL = NP PI = NP DS
	50		10A 10B	6 16 36	100% 100%	**Sandy CLAY (CL)**, hard, moist, olive brown (2.5Y 4/4), slightly plastic SILT; no reaction to HCl	14.1 18.9	115.0 111.0	Water added to facilitate drilling @ 50' bgs LL = 33 PI = 16 DS ER = 1600 ohm-cm CORR @50.5 bgs LL = 31 PI = 15 DS, CONS
	55								
	60		11A 11B	9 18 29	100% 100%	**Silty CLAY (CL)**, hard, moist, brown (10YR 4/3), medium plastic CLAY, trace fine sand; no reaction to HCl	18.8	111.0	SA: 72% < #200 LL = 28 PI = 10 SA: 53% < #200 DS, CONS
	65								

Elevation, feet	Depth, feet	SAMPLES Type	Number	Blows / 6"	Recovery	Graphic Log	MATERIAL DESCRIPTION	Water Content, %	Dry Unit Weight, pcf	REMARKS
	65									
	70		12A 12B	11 50/4"	67% 100%		**Sandy SILT (ML)**, very dense, moist, brown (10YR 4/3), trace coarse SAND, trace subrounded gravel to 1"; no reaction to HCl	23.9	107.0	SA: 69% < #200
	75									
	80		13	50/6"	83%		**Well-graded SAND with Silt and Grael (SW-SM)**, very dense, moist, olive brown (2.5Y 4/3), fine to coarse SAND, fine to coarse subrounded gravel of granitic rock to 1.5"; no reaction to HCl			Sample 13: Contains mechanically broken gravel SA: 11.3% < #200
	85									
	90									
	95									
	100									

Elevation, feet	Depth, feet	SAMPLES Type	Number	Blows / 6"	Recovery	Graphic Log	MATERIAL DESCRIPTION	Water Content, %	Dry Unit Weight, pcf	REMARKS
	100		14A 14B	23 50/6"	100% 100%		**SILT (ML)**, hard, moist, dark grayish brown (2.5Y 4/2), nonplastic SILT; FeO staining, no reaction to HCl	30.9	94.0	SA: 84.8% < #200 Cuttings stored in drums (9 total; 1 only half full). Left onsite for pickup and disposal later
							Total Depth = 101' bgs			
	105									
	110									
	115									
	120									
	125									
	130									
	135									

APPENDIX B

LABORATORY TESTING

Appendix B – Laboratory Testing

General

The laboratory testing program performed by GeoPentech for the proposed project site included moisture content, dry density, Atterberg limits, particle size distribution, direct shear, consolidation, and corrosion. The geotechnical testing was conducted at the laboratory facilities of Leighton in Irvine, California. The tests were performed in general accordance with applicable procedures of the American Society for Testing and Materials (ASTM), the State of California Department of Transportation, Standard Test Methods (DOT CA), and United States Environmental Protection Agency Test Methods (USEPA). The results of laboratory tests are summarized in Table B-1, on the boring logs in Appendix A, and on figures presented in Appendix B. The results of the laboratory testing were provided in a letter by Leighton dated April 9, 2018 and are included in this Appendix. GeoPentech has reviewed the results of the laboratory testing and finds them acceptable. Brief descriptions of the testing and the test results are presented in the following sections.

Moisture Content and Dry Density

For selected modified California samples (CA), 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. The moisture content and dry density of the samples tested are presented on the boring logs in Appendix A at the corresponding sample depth and are summarized in Table B-1.

Atterberg Limits

Atterberg limits test is a classification test that is performed on cohesive soils (i.e. silty and clayey soils) to measure their plastic limit (PL) and liquid limit (LL) from which the plasticity index (PI) can be calculated. The measured values can be plotted on a plasticity chart, which is used as an aid in classifying the soil material and its behavior. These tests were performed in accordance with ASTM D4318. The results of the Atterberg Limits tests are shown on the boring logs (Appendix A) and summarized in Table B-1.

Particle-Size Analysis

For selected CA, SPT, and bulk samples, the distribution of particle sizes larger than 0.075-mm (retained on the No. 200 sieve) was determined by sieving in general accordance with ASTM D6913. The grain size distribution curves are plotted below, and the percentages of gravel, sand, and fines (material passing the Standard No. 200 sieve) are presented in Table B-1. The percentage of fines measured in the grain size distribution is also presented on the boring logs (Appendix A) for convenience and are noted by “SA”.

Direct Shear

Direct shear tests were performed on selected samples in accordance with ASTM D3080. Shear stress and sample deformation were monitored throughout the tests. The results of the direct shear tests are presented below.

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, and are presented below.

Corrosivity Tests

Soil samples were tested for electrical resistivity, pH, sulfate content, and chloride content. These tests were performed on relatively undisturbed samples in general accordance with DOT CA test methods 417, 422, and 643. The test results were used to determine the corrosivity potential of the soil on underground improvements for the proposed structure. The results of the corrosivity tests are summarized in Table B-1.

TABLE B-1
SUMMARY OF LABORATORY TESTING
1201 S. GRAND

Location				Classification		Initial Condition			Atterberg		Gradation			Direct Shear (peak)				UC	Compaction		EI	R-value	Chem				Consolidation
Boring Number	Sample/ Specimen Number	Sample Type	Depth (ft)	Geologic Unit [1]	USCS Symbol / rock type	Water content (%)	Total unit weight (pcf)	Dry unit weight (pcf)	Liquid Limit	Plasticity Index	Gravel (%)	Sand (%)	Fines (%)	Preparation	Normal stress sequence (ksf)	Friction angle (deg)	Cohesion (ksf)	Unconfined compression, strength (psi)	Optimum water content (%) (Modified Proctor Test)	Maximum dry density (pcf) (Modified Proctor Test)	Expansion Index @ 50% S	R-value	Min. resistivity (ohm-cm)	Sulfate content (ppm)	Chloride content (ppm)	Soil pH	
B1	2	SPT	10-11.5		SP-SM	1.8																					
B1	4	SPT	20-21.5		SW-SM	4.0					40	49	11														
B1	6	SPT	30-31.5		CL-ML	15.6																					
B1	8	SPT	40-41.5		SW-SM	5.0					16	75	9														
B1	9a	SPT	45-46		CL	15.1																					
B1	9a & 9b	SPT	45-46.5		CL				38	26			61										1400	109	72	6.46	
B1	11	SPT	60-61.5		CH	24.0			50	33																	
B1	12	SPT	70-71.5		SP-SM	3.8																					
B1	13b	Cal Mod	81-81.5		SM	2.7	106	104							4,8,12	36	0.0										
B1	14	SPT	90-91.5		SP-SM	3.6																					
B1	15	SPT	100-101.5		ML	23.2					0.0	26.0	74.0														
B1	16	SPT	110-111.5		SP-SM	5.5																					
B1	17	Bulk	120-121.5		SP-SM	6.4																					
B1	18	SPT	135-136.5		SP-SM	7.5							6														
B1	19	SPT	150-151.5		SP-SM	6.2																					
B2	2	SPT	10-11.5		SP-SM	1.8																					
B2	4	SPT	20-21.5		SP-SM	4.7																					
B2	3a, 3b & 4	SPT	15-21.5										5.3										8775	87	50	6.71	
B2	6	SPT	30-31.5		SP-SM	4.8							8.4														
B2	8b	Cal Mod	40.3-40.8		SW-SM	4.5	115	110			24	67	9		3,5,10	36	0										
B2	9b	Cal Mod	45.5-46		SM	9.4	111	101	NP	NP			36.7		1,2,5	35	0.097										
B2	10a	Cal Mod	50-50.5		CL	14.1	131	115	33	16					3,5,10	25	1.162						1600	139	21	6.34	X

Note: [1] af = Artificial Fill, Qvof = Very Old Alluvium

TABLE B-1
SUMMARY OF LABORATORY TESTING
1201 S. GRAND

Location				Classification		Initial Condition			Atterberg		Gradation			Direct Shear (peak)				UC	Compaction		EI	R-value	Chem				Consolidation
Boring Number	Sample/ Specimen Number	Sample Type	Depth (ft)	Geologic Unit [1]	USCS Symbol / rock type	Water content (%)	Total unit weight (pcf)	Dry unit weight (pcf)	Liquid Limit	Plasticity Index	Gravel (%)	Sand (%)	Fines (%)	Preparation	Normal stress sequence (ksf)	Friction angle (deg)	Cohesion (ksf)	Unconfined compression, strength (psi)	Optimum water content (%) (Modified Proctor Test)	Maximum dry density (pcf) (Modified Proctor Test)	Expansion Index @ 50% S	R-value	Min. resistivity (ohm-cm)	Sulfate content (ppm)	Chloride content (ppm)	Soil pH	
B2	10b	Cal Mod	50.5-51		CL	18.9	131	111	31	15					3,5,10	31	0.49										
B2	11a	Cal Mod	60-60.5		CL	18.8	132	111			0	28	72														X
B2	11b	Cal Mod	60.5-61		CL-ML				28	10	0	47	53		4,7,12	34	0.276										
B2	12a	Cal Mod	70-70.3		ML	23.9	132	107			0	31	69														
B2	13	Cal Mod	80-80.4		SW-SM								11.3														
B2	14b	Cal Mod	100.5-101		CL-ML	30.9	123	94					84.8														

Note: [1] af = Artificial Fill, Qvof = Very Old Alluvium

Leighton Consulting, Inc.
A LEIGHTON GROUP COMPANY

Friday, April 06, 2018

Leighton Project No. 11440.010

GeoPentech, Inc.
5251 California Avenue, Suite 210
Irvine, CA 92617

Attention: Mr. Douglas Wahl, PE

Subject: Geotechnical Laboratory Testing Results
1201 S. Grand Avenue Geotechnical Investigation
GeoPentech Project No. 15083A

In accordance with your request and Laboratory Assignment Schedule received March 9, 2018, Leighton Consulting, Inc. (Leighton) performed geotechnical laboratory testing of soil samples from the above-referenced project. Leighton's scope of work was limited to geotechnical laboratory testing of soil samples brought to our Irvine laboratory by others. We did not perform any work outside of our laboratory for this project and we are unaware of the chain-of-custody for these samples brought to our laboratory. Test reports were delivered to you electronically in the form of PDF files on March 28, 29 and 30, 2018. The tests were performed in our approved LADBS (City of Los Angeles Department of Building and Safety) testing agency laboratory license no. TA10069, and conducted in essential accordance with the standard test methods listed below.

GEOTECHNICAL LABORATORY TESTING

- ASTM D2216 Laboratory Determination of Water (Moisture) Content of Soil and Rock by Mass
- ASTM D2937 Density of Soil in Place by the Drive-Cylinder Method
- ASTM D6913 Particle-Size Distribution (Gradation) of Soils Using Sieve Analysis
- ASTM D422 Particle-Size Analysis of Soils
- ASTM D1140 Amount of Material in Soils Finer Than the No. 200 (75-µm) Sieve
- ASTM D4318 Liquid Limit, Plastic Limit, and Plasticity Index of Soils
- ASTM D3080 Direct Shear Test of Soils Under Consolidated Drained Conditions
- ASTM D2435 One-Dimensional Consolidation Properties of Soils Using Incremental Loading

- CTM 417-B, 422, 643 Sulfate Content, Chloride Content, pH and Resistivity of Soils

LIMITATIONS

The soil specimens were tested for GeoPentech, Inc., based on their needs, directions, and requirements at the time. The results of geotechnical laboratory testing are not authorized for use by, and are not to be relied upon by any party except GeoPentech, Inc., with whom Leighton contracted for the work. Use of or reliance on the geotechnical laboratory test reports by any other party is at that party's risk. Unauthorized use of or reliance on the test reports constitutes an agreement to defend and indemnify Leighton from and against any liability which may arise as a result of such use or reliance, regardless of any fault, negligence, or strict liability of Leighton.

We appreciate being of continued service to GeoPentech, Inc. If you have any questions, please contact us at your convenience at **(866) *LEIGHTON***, directly at the phone extensions and e-mail addresses below.

Respectfully submitted,

LEIGHTON CONSULTING, INC.

James Ward
Laboratory Project Coordinator
Extension 4249, jward@leightongroup.com

Roderick Marcia, PE 70150
Materials Testing Engineering Manager
Extension 4294, rmarcia@leightongroup.com

JW/RM:rm

Distribution: (1) Addressee – PDF only

Attachment: References (1-page)

R E F E R E N C E S

ASTM International, Annual Book of ASTM Standards, Section 4: Construction, Volumes 04.08 and 04.09: Soil and Rock (I and II), 2018.

CTM 417-B (Part II): State of California, Department of Transportation, California Test 417-B (Part II), October 1, 1973.

CTM 422 (Part II): State of California, Department of Transportation, California Test 422 (Part II), 1978.

CTM 643: State of California, Department of Transportation, California Test 643, June 2007, http://www.dot.ca.gov/hq/esc/ctms/pdf/CT_643jun07.pdf

Boring No.	B-1	B-1	B-2	B-2	B-2	B-2	B-2	
Sample No.	9a & 9b	18	3a, 3b & 4	6	9b	13	14b	
Depth (ft.)	45-46.5	135-136.5	15-21.5	30-31.5	45.5-46	80-80.4	100.5-101	
Sample Type	S	S	S	S	C	C	C	
Soil Identification	Olive brown sandy lean clay s(CL)	Light olive brown poorly-graded sand with silt (SP-SM)	Olive brown poorly-graded sand with silt and gravel (SP-SM)g	Olive brown poorly-graded sand with silt and gravel (SP-SM)g	Yellowish brown silty sand (SM)	Olive brown poorly-graded sand with silt and gravel (SP-SM)g	Yellowish brown silty clay with sand (CL-ML)s	
Moisture Correction								
Wet Weight of Soil + Container (g)	0.0	0.0	0.0	0.0	0.0	0.0	0.0	
Dry Weight of Soil + Container (g)	0.0	0.0	0.0	0.0	0.0	0.0	0.0	
Weight of Container (g)	1.0	1.0	1.0	1.0	1.0	1.0	1.0	
Moisture Content (%)	0.0	0.0	0.0	0.0	0.0	0.0	0.0	
Sample Dry Weight Determination								
Weight of Sample + Container (g)	340.0	463.3	790.8	412.9	603.4	682.1	463.4	
Weight of Container (g)	108.0	76.7	76.0	77.2	244.2	74.9	99.7	
Weight of Dry Sample (g)	232.0	386.6	714.8	335.7	359.2	607.2	363.7	
Container No.:								
After Wash								
Method (A or B)	A	A	A	A	A	A	A	
Dry Weight of Sample + Cont. (g)	199.1	440.1	752.9	384.7	471.7	613.4	155.0	
Weight of Container (g)	108.0	76.7	76.0	77.2	244.2	74.9	99.7	
Dry Weight of Sample (g)	91.1	363.4	676.9	307.5	227.5	538.5	55.3	
% Passing No. 200 Sieve	**60.7**	**6.0**	**5.3**	**8.4**	**36.7**	**11.3**	**84.8**	
% Retained No. 200 Sieve	39.3	94.0	94.7	91.6	63.3	88.7	15.2	

Leighton

PERCENT PASSING
No. 200 SIEVE
ASTM D 1140

Project Name: 1201 S. Grand Avenue Geotechnical Investigation
Project No.: 15083A
Tested By: OHF/RM Date: 03/14/18

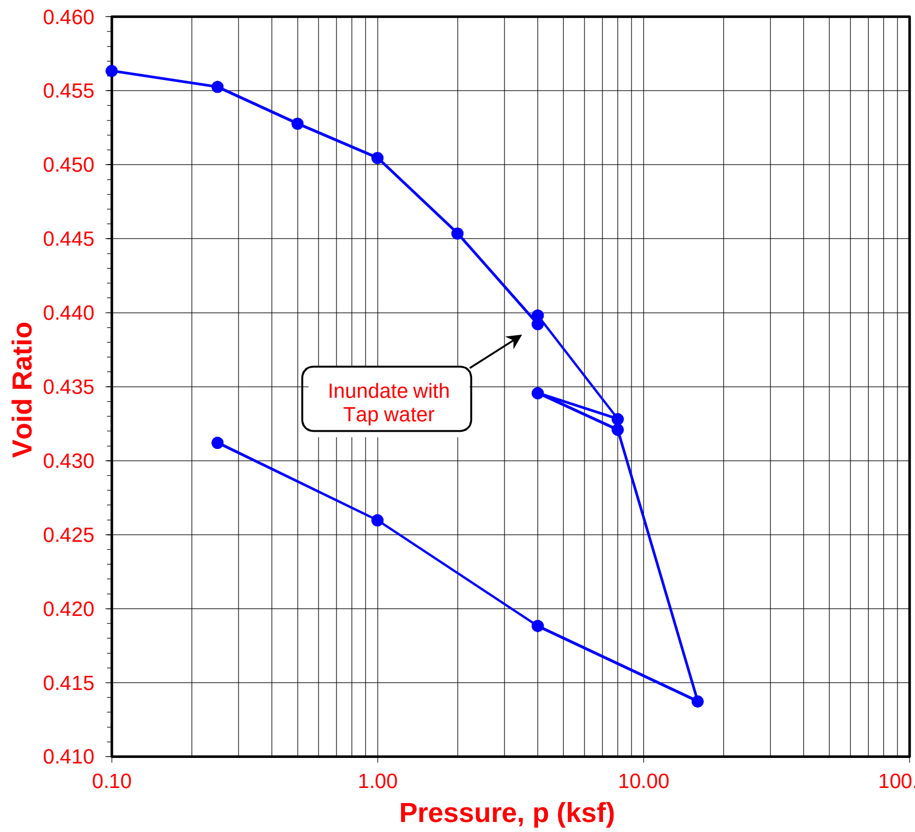
Leighton

ONE-DIMENSIONAL CONSOLIDATION PROPERTIES of SOILS

ASTM D 2435

Project Name:	1201 S. Grand Avenue Geotechnical Investigation	Tested By:	G. Bathala	Date:	03/14/18
Project No.:	15083A	Checked By:	J. Ward	Date:	03/29/18
Boring No.:	B-2	Depth (ft.):	50-50.5		
Sample No.:	10a	Sample Type:	C		
Soil Identification:	Light olive brown sandy lean clay s(CL)				

Sample Diameter (in.)	2.415
Sample Thickness (in.)	1.000
Wt. of Sample + Ring (g)	203.61
Weight of Ring (g)	44.88
Height after consol. (in.)	0.9824
Before Test	
Wt.Wet Sample+Cont. (g)	177.87
Wt.of Dry Sample+Cont. (g)	162.79
Weight of Container (g)	55.80
Initial Moisture Content (%)	14.1
Initial Dry Density (pcf)	115.7
Initial Saturation (%)	83
Initial Vertical Reading (in.)	0.2972
After Test	
Wt.of Wet Sample+Cont. (g)	243.10
Wt. of Dry Sample+Cont. (g)	219.17
Weight of Container (g)	39.54
Final Moisture Content (%)	17.76
Final Dry Density (pcf)	114.1
Final Saturation (%)	100
Final Vertical Reading (in.)	0.2764
Specific Gravity (assumed)	2.70
Water Density (pcf)	62.43

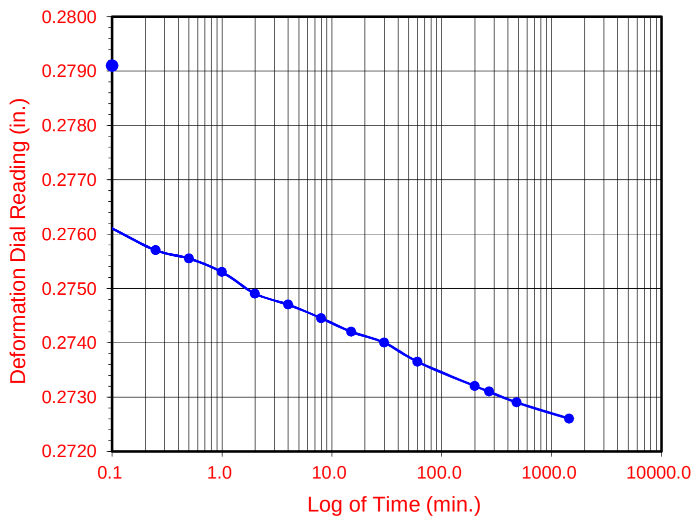


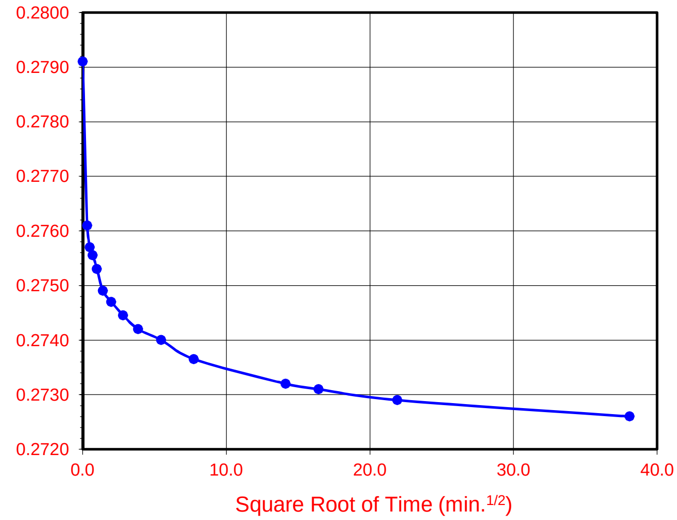
Pressure (p) (ksf)	Final Reading (in.)	Apparent Thickness (in.)	Load Compliance (%)	Deformation % of Sample Thickness	Void Ratio	Corrected Deforma-tion (%)
0.10	0.2969	0.9997	0.00	0.04	0.456	0.04
0.25	0.2955	0.9983	0.06	0.17	0.455	0.11
0.50	0.2928	0.9956	0.16	0.44	0.453	0.28
1.00	0.2897	0.9925	0.31	0.75	0.450	0.44
2.00	0.2846	0.9874	0.47	1.26	0.445	0.79
4.00	0.2787	0.9815	0.64	1.85	0.439	1.21
4.00	0.2791	0.9819	0.64	1.81	0.440	1.17
8.00	0.2726	0.9754	0.81	2.46	0.433	1.65
4.00	0.2745	0.9773	0.74	2.27	0.435	1.53
8.00	0.2720	0.9748	0.82	2.52	0.432	1.70
16.00	0.2576	0.9604	1.00	3.96	0.414	2.96
4.00	0.2632	0.9660	0.79	3.40	0.419	2.61
1.00	0.2705	0.9733	0.55	2.67	0.426	2.12
0.25	0.2764	0.9792	0.32	2.08	0.431	1.76

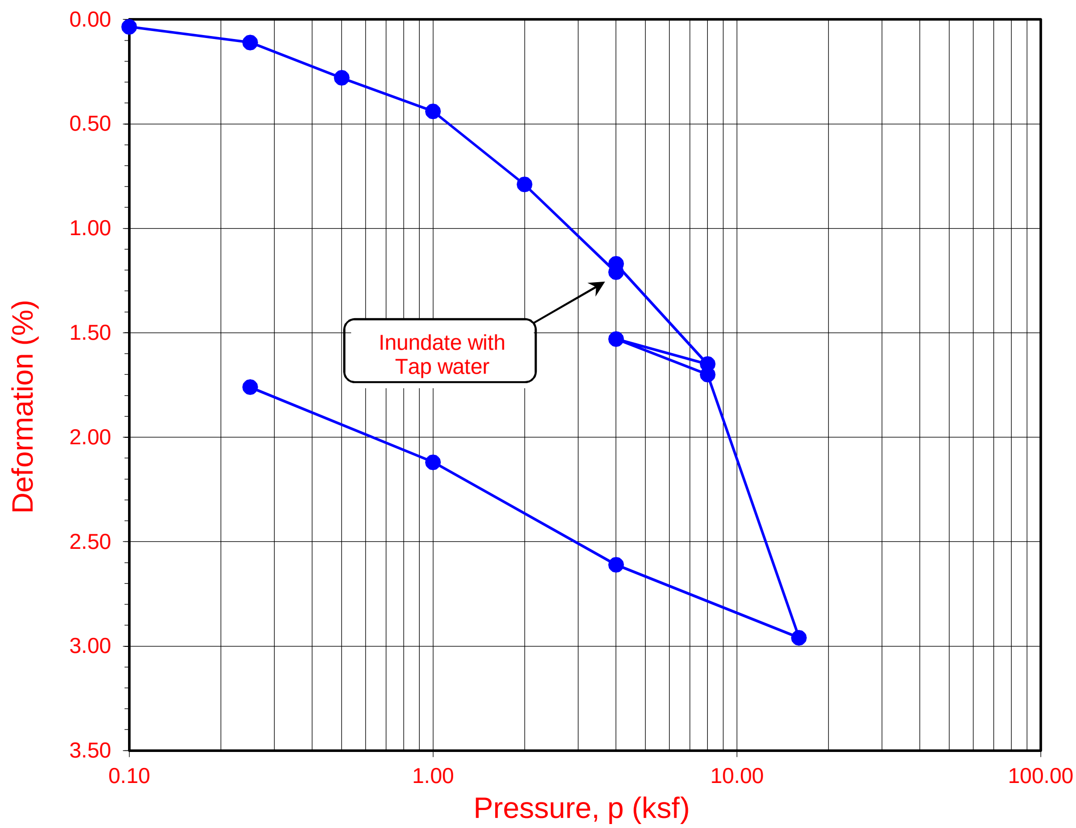
Time Readings @ 8.0 ksf

Date	Time	Elapsed Time (min)	Square Root of Time	Dial Rdgs. (in.)
3/19/18	8:25:00	0.0	0.0	0.2791
3/19/18	8:25:06	0.1	0.3	0.2761
3/19/18	8:25:15	0.2	0.5	0.2757
3/19/18	8:25:30	0.5	0.7	0.2756
3/19/18	8:26:00	1.0	1.0	0.2753
3/19/18	8:27:00	2.0	1.4	0.2749
3/19/18	8:29:00	4.0	2.0	0.2747
3/19/18	8:33:00	8.0	2.8	0.2745
3/19/18	8:40:00	15.0	3.9	0.2742
3/19/18	8:55:00	30.0	5.5	0.2740
3/19/18	9:25:00	60.0	7.7	0.2737
3/19/18	11:45:00	200.0	14.1	0.2732
3/19/18	12:55:00	270.0	16.4	0.2731
3/19/18	16:25:00	480.0	21.9	0.2729
3/20/18	8:35:00	1450.0	38.1	0.2726

Time Readings @ 8.0 ksf







Boring No.	Sample No.	Depth (ft.)	Moisture Content (%)		Dry Density (pcf)		Void Ratio		Degree of Saturation (%)	
			Initial	Final	Initial	Final	Initial	Final	Initial	Final
B-2	**10a**	**50-50.5**	**14.1**	**17.8**	**115.7**	**114.1**	**0.457**	**0.431**	**83**	**100**

Soil Identification: Light olive brown sandy lean clay s(CL)

Leighton

ONE-DIMENSIONAL CONSOLIDATION PROPERTIES of SOILS
ASTM D 2435

Project No.: 15083A

1201 S. Grand Avenue Geotechnical Investigation

03-18

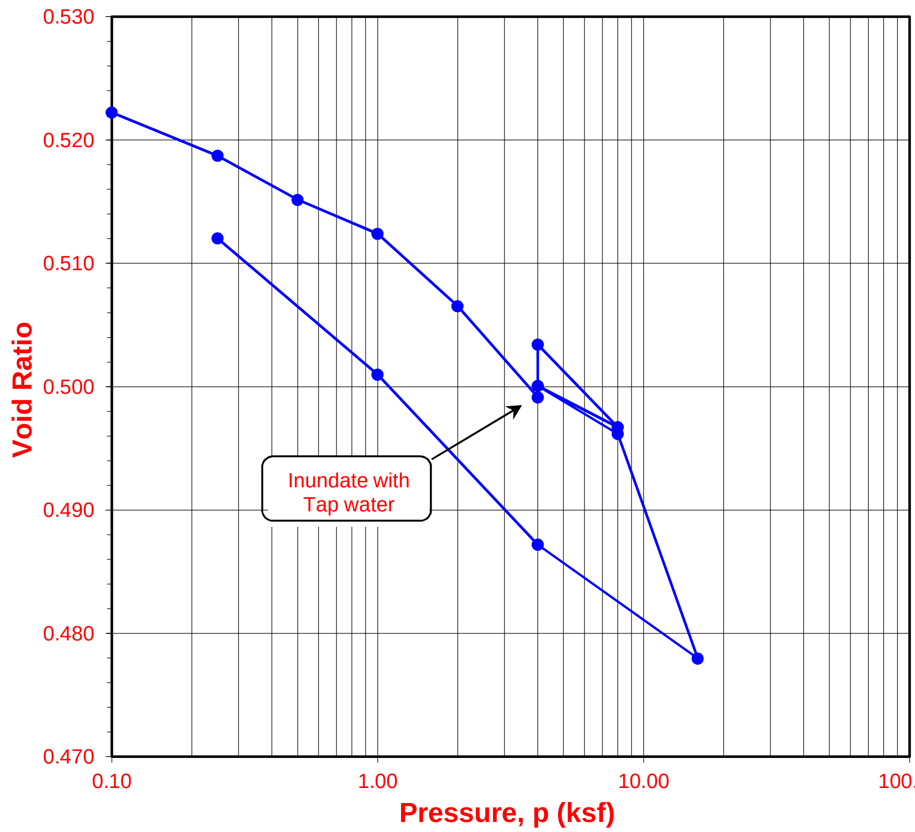
Leighton

ONE-DIMENSIONAL CONSOLIDATION PROPERTIES of SOILS

ASTM D 2435

Project Name:	1201 S. Grand Avenue Geotechnical Investigation	Tested By: G. Bathala	Date:	03/14/18
Project No.:	15083A	Checked By: J. Ward	Date:	03/29/18
Boring No.:	B-2	Depth (ft.): 60-60.5		
Sample No.:	11a	Sample Type: C		
Soil Identification:	Dark yellowish brown lean clay with sand (CL)s			

Sample Diameter (in.)	2.415
Sample Thickness (in.)	1.000
Wt. of Sample + Ring (g)	203.74
Weight of Ring (g)	45.77
Height after consol. (in.)	0.9924
Before Test	
Wt.Wet Sample+Cont. (g)	701.77
Wt.of Dry Sample+Cont. (g)	625.41
Weight of Container (g)	218.30
Initial Moisture Content (%)	18.8
Initial Dry Density (pcf)	110.6
Initial Saturation (%)	97
Initial Vertical Reading (in.)	0.3294
After Test	
Wt.of Wet Sample+Cont. (g)	243.67
Wt. of Dry Sample+Cont. (g)	218.75
Weight of Container (g)	39.92
Final Moisture Content (%)	18.73
Final Dry Density (pcf)	111.5
Final Saturation (%)	99
Final Vertical Reading (in.)	0.3181
Specific Gravity (assumed)	2.70
Water Density (pcf)	62.43

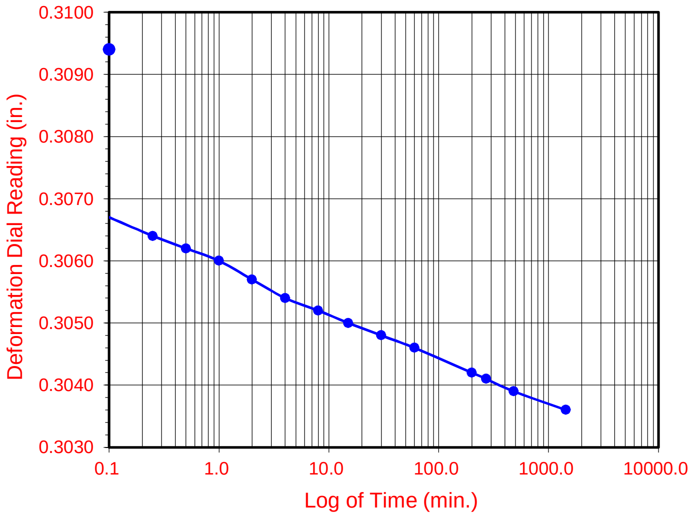


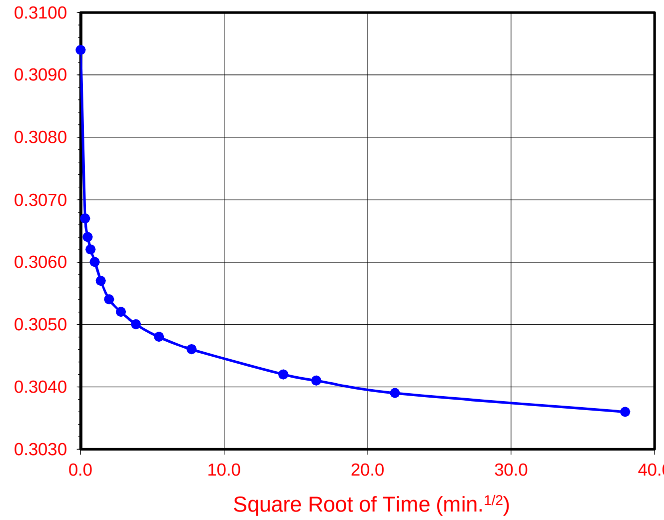
Pressure (p) (ksf)	Final Reading (in.)	Apparent Thickness (in.)	Load Compliance (%)	Deformation % of Sample Thickness	Void Ratio	Corrected Deformation (%)
0.10	0.3285	0.9991	0.00	0.10	0.522	0.10
0.25	0.3258	0.9964	0.04	0.37	0.519	0.33
0.50	0.3224	0.9930	0.14	0.70	0.515	0.56
1.00	0.3192	0.9898	0.28	1.02	0.512	0.74
2.00	0.3135	0.9841	0.47	1.60	0.507	1.13
4.00	0.3066	0.9772	0.67	2.28	0.499	1.61
4.00	0.3094	0.9800	0.67	2.00	0.503	1.33
8.00	0.3036	0.9742	0.81	2.58	0.497	1.77
4.00	0.3066	0.9772	0.73	2.28	0.500	1.55
8.00	0.3032	0.9738	0.82	2.63	0.496	1.81
16.00	0.2895	0.9601	0.99	3.99	0.478	3.00
4.00	0.2976	0.9682	0.79	3.19	0.487	2.40
1.00	0.3090	0.9796	0.55	2.04	0.501	1.49
0.25	0.3181	0.9887	0.37	1.14	0.512	0.76

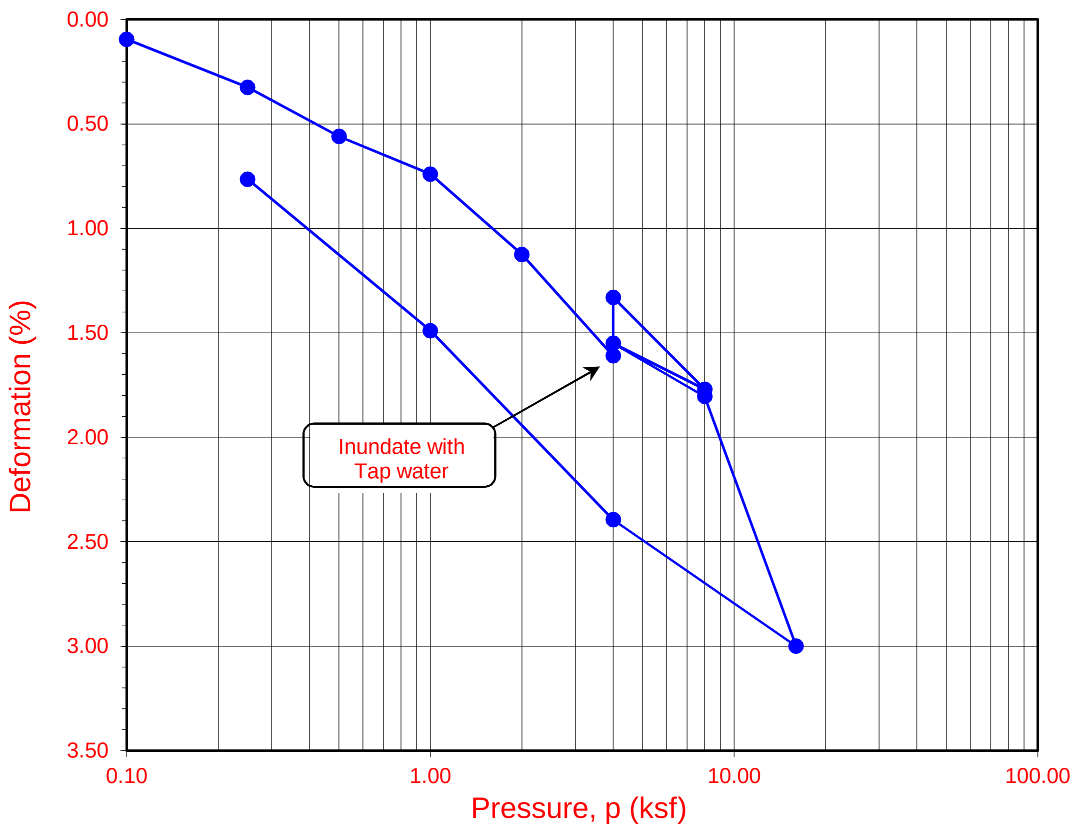
Time Readings @ 8.0 ksf

Date	Time	Elapsed Time (min)	Square Root of Time	Dial Rdgs. (in.)
3/19/18	8:30:00	0.0	0.0	0.3094
3/19/18	8:30:06	0.1	0.3	0.3067
3/19/18	8:30:15	0.2	0.5	0.3064
3/19/18	8:30:30	0.5	0.7	0.3062
3/19/18	8:31:00	1.0	1.0	0.3060
3/19/18	8:32:00	2.0	1.4	0.3057
3/19/18	8:34:00	4.0	2.0	0.3054
3/19/18	8:38:00	8.0	2.8	0.3052
3/19/18	8:45:00	15.0	3.9	0.3050
3/19/18	9:00:00	30.0	5.5	0.3048
3/19/18	9:30:00	60.0	7.7	0.3046
3/19/18	11:50:00	200.0	14.1	0.3042
3/19/18	13:00:00	270.0	16.4	0.3041
3/19/18	16:30:00	480.0	21.9	0.3039
3/20/18	8:30:00	1440.0	37.9	0.3036

Time Readings @ 8.0 ksf







Boring No.	Sample No.	Depth (ft.)	Moisture Content (%)		Dry Density (pcf)		Void Ratio		Degree of Saturation (%)	
			Initial	Final	Initial	Final	Initial	Final	Initial	Final
B-2	**11a**	**60-60.5**	**18.8**	**18.7**	**110.6**	**111.5**	**0.524**	**0.512**	**97**	**99**

Soil Identification: Dark yellowish brown lean clay with sand (CL)s

Leighton

ONE-DIMENSIONAL CONSOLIDATION PROPERTIES of SOILS
ASTM D 2435

Project No.: 15083A

1201 S. Grand Avenue Geotechnical Investigation

03-18

TESTS for SULFATE CONTENT
CHLORIDE CONTENT and pH of SOILS

Leighton

Project Name:	1201 S. Grand Avenue Geotechnical Investigation	Tested By :	G. Berdy	Date:	03/15/18
Project No. :	15083A	Data Input By:	J. Ward	Date:	03/30/18

Boring No.	B-1	B-2	B-2	
Sample No.	9a & 9b	3a, 3b & 4	10a	
Sample Depth (ft)	45-46.5	15-21.5	50-50.5	
Soil Identification:	Olive brown s(CL)	Olive brown (SP-SM)g	Light olive brown s(CL)	
Wet Weight of Soil + Container (g)	192.13	184.43	187.13	
Dry Weight of Soil + Container (g)	189.28	184.08	180.48	
Weight of Container (g)	58.27	57.71	52.57	
Moisture Content (%)	2.18	0.28	5.20	
Weight of Soaked Soil (g)	100.75	100.16	100.35	

SULFATE CONTENT, DOT California Test 417, Part II

Beaker No.	116	310	2	
Crucible No.	19	12	9	
Furnace Temperature (°C)	860	860	860	
Time In / Time Out	8:30/9:15	8:30/9:15	8:30/9:15	
Duration of Combustion (min)	45	45	45	
Wt. of Crucible + Residue (g)	23.7557	22.6907	21.2028	
Wt. of Crucible (g)	23.7531	22.6886	21.1996	
Wt. of Residue (g) (A)	0.0026	0.0021	0.0032	
PPM of Sulfate (A) x 41150	106.99	86.41	131.68	
PPM of Sulfate, Dry Weight Basis	**109**	**87**	**139**	

CHLORIDE CONTENT, DOT California Test 422

ml of Extract For Titration (B)	30	30	30	
ml of AgNO3 Soln. Used in Titration (C)	0.9	0.7	0.4	
PPM of Chloride (C -0.2) * 100 * 30 / B	70	50	20	
PPM of Chloride, Dry Wt. Basis	**72**	**50**	**21**	

pH TEST, DOT California Test 643

pH Value	**6.46**	**6.71**	**6.34**	
Temperature °C	19.7	20.4	19.4	

SOIL RESISTIVITY TEST

DOT CA TEST 643

Project Name:	1201 S. Grand Avenue Geotechnical Investigation	Tested By :	G. Berdy	Date:	03/20/18
Project No. :	15083A	Data Input By:	J. Ward	Date:	03/30/18
Boring No.:	B-1	Depth (ft.) :	45-46.5		
Sample No. :	9a & 9b				

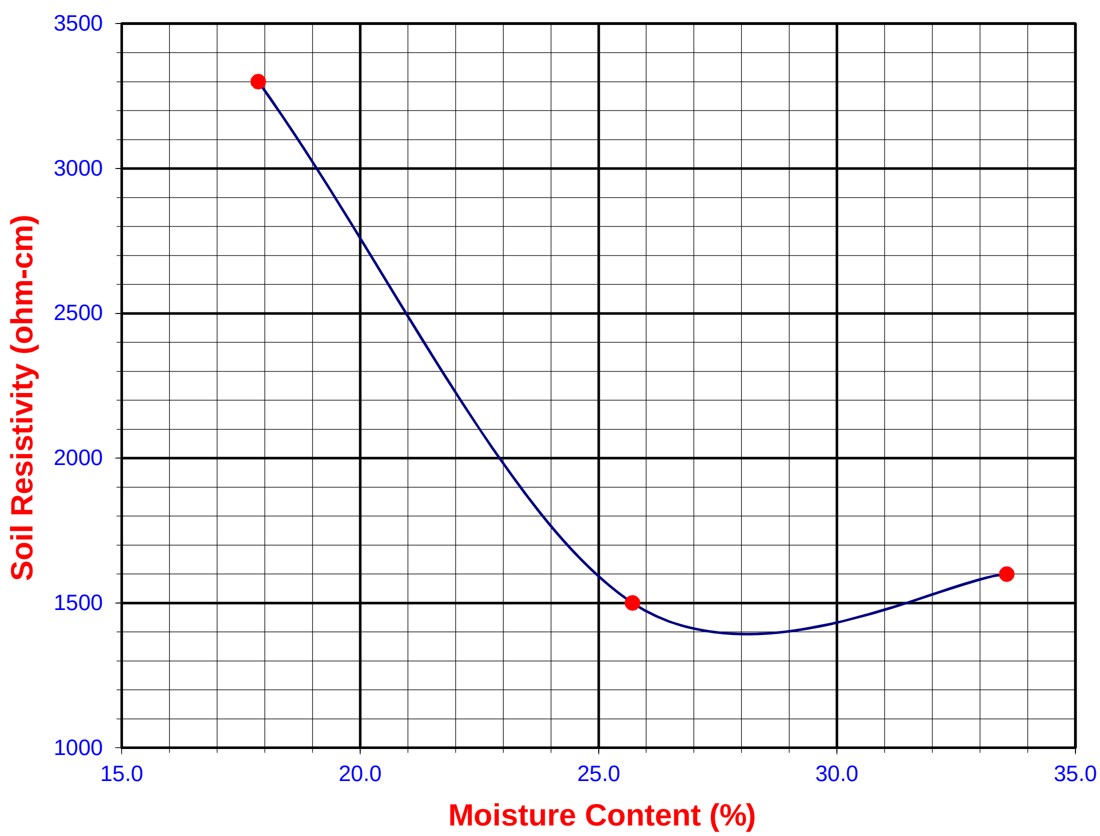
Soil Identification:* Olive brown s(CL)

*California Test 643 requires soil specimens to consist only of portions of samples passing through the No. 8 US Standard Sieve before resistivity testing. Therefore, this test method may not be representative for coarser materials.

Specimen No.	Water Added (ml) (Wa)	Adjusted Moisture Content (MC)	Resistance Reading (ohm)	Soil Resistivity (ohm-cm)
1	20	17.87	3300	3300
2	30	25.71	1500	1500
3	40	33.56	1600	1600
4				
5				

Moisture Content (%) (MCi)	2.18
Wet Wt. of Soil + Cont. (g)	192.13
Dry Wt. of Soil + Cont. (g)	189.28
Wt. of Container (g)	58.27
Container No.	
Initial Soil Wt. (g) (Wt)	130.23
Box Constant	1.000
MC =(((1+Mci/100)x(Wa/Wt+1))-1)x100	

Min. Resistivity (ohm-cm)	Moisture Content (%)	Sulfate Content (ppm)	Chloride Content (ppm)	Soil pH	
				pH	Temp. (°C)
DOT CA Test 643		DOT CA Test 417 Part II	DOT CA Test 422	DOT CA Test 643	
1400	**28.1**	**109**	**72**	**6.46**	**19.7**



SOIL RESISTIVITY TEST

DOT CA TEST 643

Project Name:	1201 S. Grand Avenue Geotechnical Investigation	Tested By :	G. Berdy	Date:	03/20/18
Project No. :	15083A	Data Input By:	J. Ward	Date:	03/30/18
Boring No.:	B-2	Depth (ft.) :	15-21.5		
Sample No. :	3a, 3b & 4				

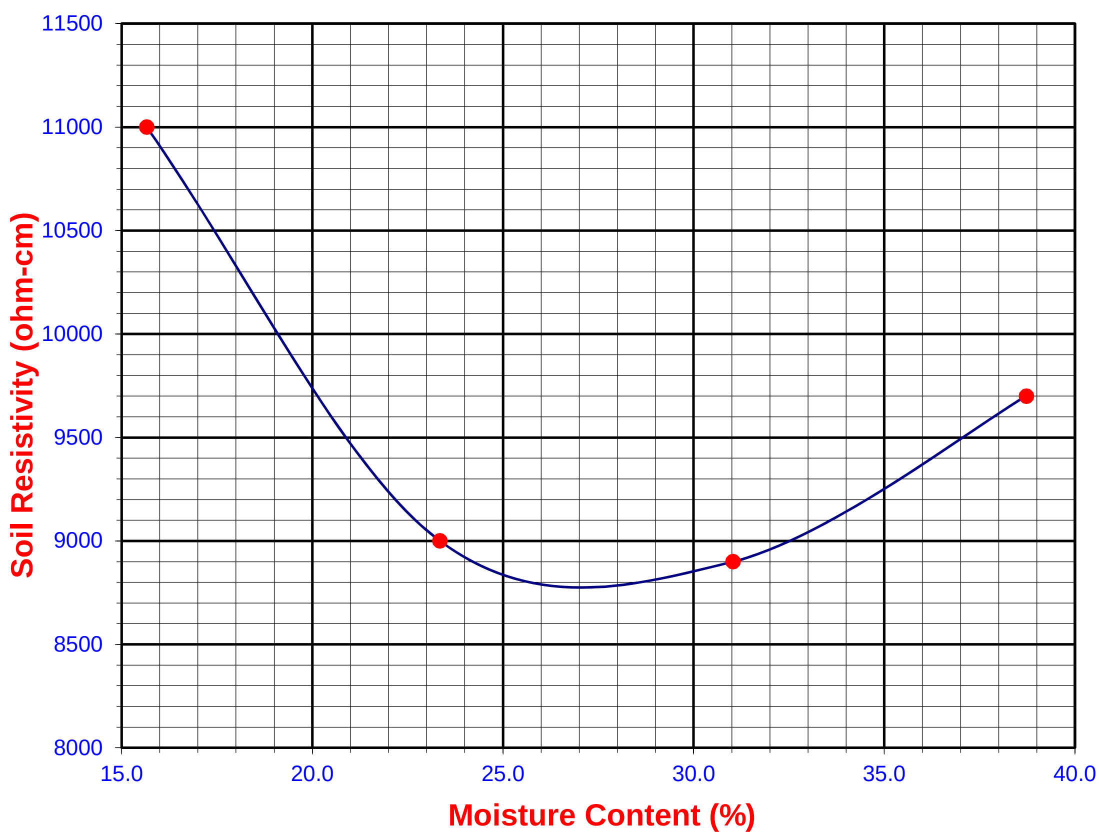
Soil Identification:* Olive brown (SP-SM)g

*California Test 643 requires soil specimens to consist only of portions of samples passing through the No. 8 US Standard Sieve before resistivity testing. Therefore, this test method may not be representative for coarser materials.

Specimen No.	Water Added (ml) (Wa)	Adjusted Moisture Content (MC)	Resistance Reading (ohm)	Soil Resistivity (ohm-cm)
1	20	15.66	11000	11000
2	30	23.35	9000	9000
3	40	31.04	8900	8900
4	50	38.73	9700	9700
5				

Moisture Content (%) (MCi)	0.28
Wet Wt. of Soil + Cont. (g)	184.43
Dry Wt. of Soil + Cont. (g)	184.08
Wt. of Container (g)	57.71
Container No.	
Initial Soil Wt. (g) (Wt)	130.40
Box Constant	1.000
MC =(((1+Mci/100)x(Wa/Wt+1))-1)x100	

Min. Resistivity (ohm-cm)	Moisture Content (%)	Sulfate Content (ppm)	Chloride Content (ppm)	Soil pH	
				pH	Temp. (°C)
DOT CA Test 643		DOT CA Test 417 Part II	DOT CA Test 422	DOT CA Test 643	
8775	**27.1**	**87**	**50**	**6.71**	**20.4**



SOIL RESISTIVITY TEST

DOT CA TEST 643

Leighton

Project Name:	1201 S. Grand Avenue Geotechnical Investigation	Tested By :	G. Berdy	Date:	03/20/18
Project No. :	15083A	Data Input By:	J. Ward	Date:	03/30/18
Boring No.:	B-2	Depth (ft.) :	50-50.5		
Sample No. :	10a				

Soil Identification:* Light olive brown s(CL)

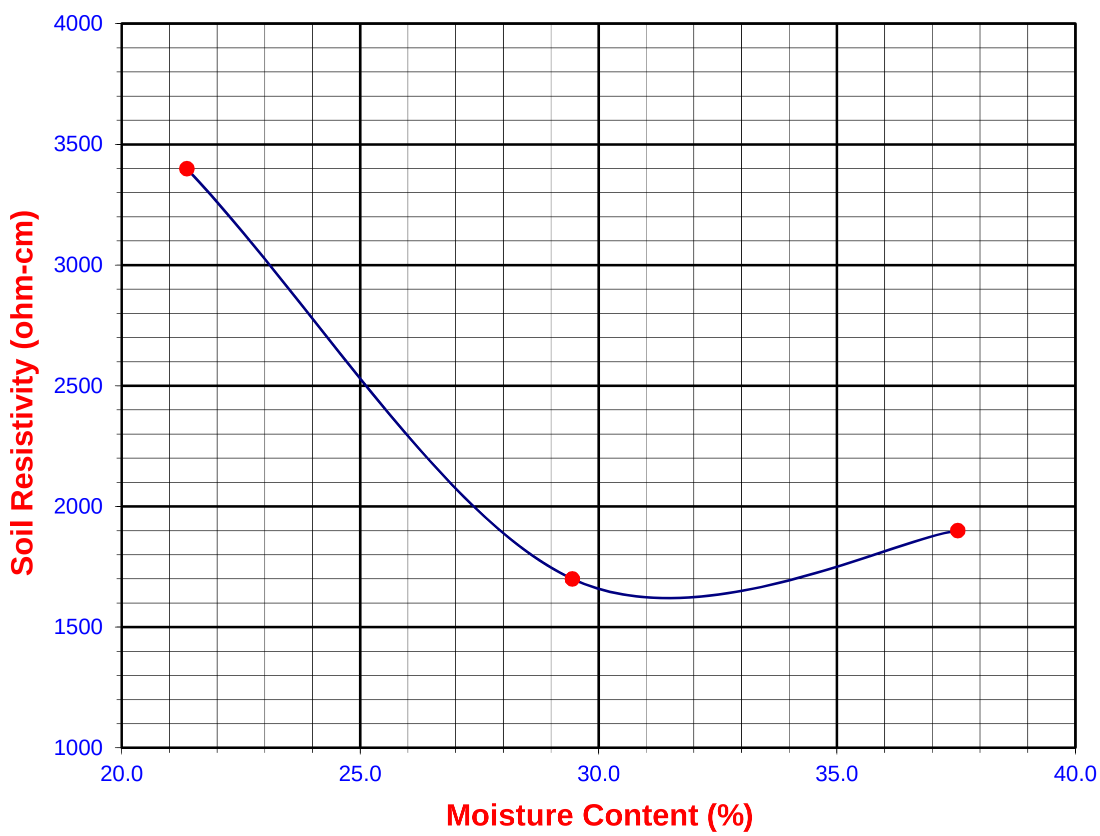
*California Test 643 requires soil specimens to consist only of portions of samples passing through the No. 8 US Standard Sieve before resistivity testing. Therefore, this test method may not be representative for coarser materials.

Specimen No.	Water Added (ml) (Wa)	Adjusted Moisture Content (MC)	Resistance Reading (ohm)	Soil Resistivity (ohm-cm)
1	20	21.37	3400	3400
2	30	29.45	1700	1700
3	40	37.53	1900	1900
4				
5				

Moisture Content (%) (MCi)	5.20
Wet Wt. of Soil + Cont. (g)	187.13
Dry Wt. of Soil + Cont. (g)	180.48
Wt. of Container (g)	52.57
Container No.	
Initial Soil Wt. (g) (Wt)	130.14
Box Constant	1.000

MC =(((1+Mci/100)x(Wa/Wt+1))-1)x100

Min. Resistivity (ohm-cm)	Moisture Content (%)	Sulfate Content (ppm)	Chloride Content (ppm)	Soil pH	
				pH	Temp. (°C)
DOT CA Test 643		DOT CA Test 417 Part II	DOT CA Test 422	DOT CA Test 643	
1600	**31.5**	**139**	**21**	**6.34**	**19.4**



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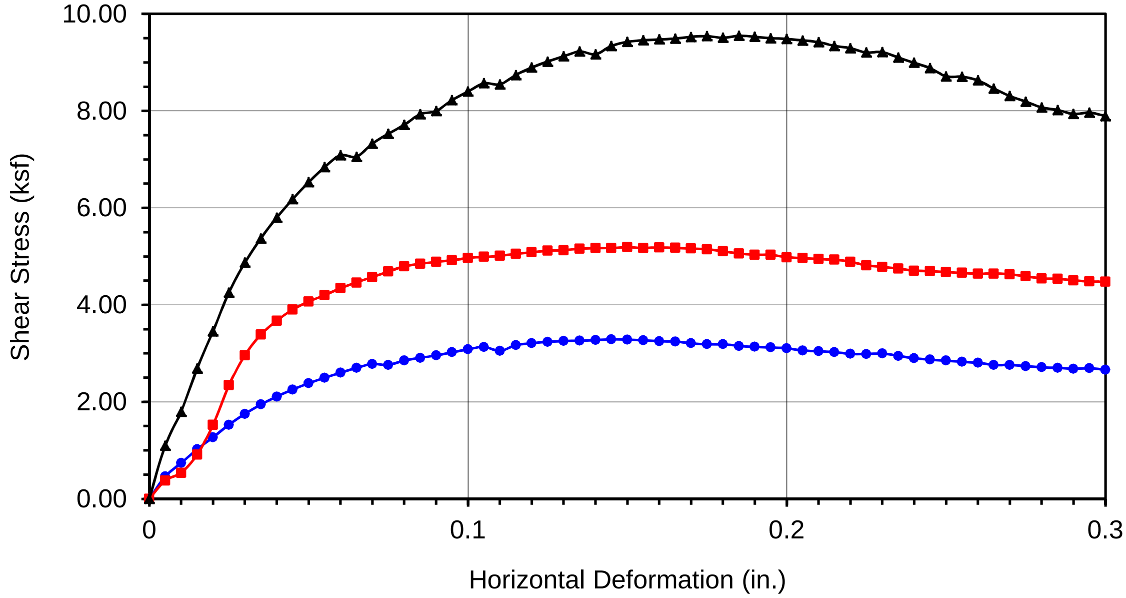
DIRECT SHEAR TEST

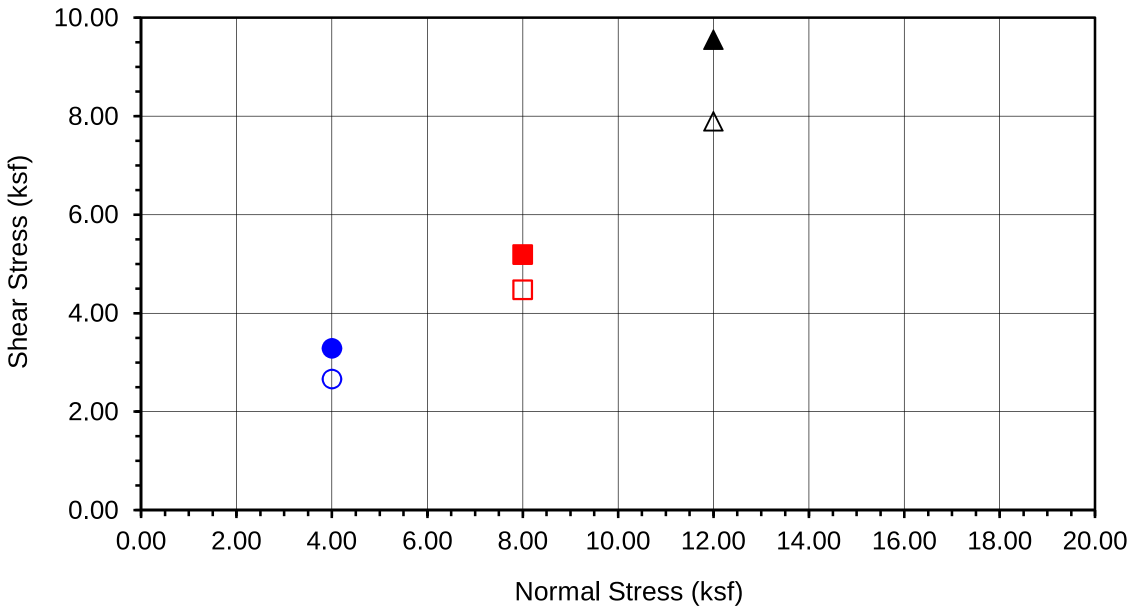
Consolidated Drained - ASTM D 3080

Project Name: 1201 S. Grand Avenue Geotechnical Investigation
Project No.: 15083A
Boring No.: B-1
Sample No.: 13b
Soil Identification: Yellowish brown silty sand (SM)

Tested By: R. Manning — Date: 03/20/18
Checked By: J. Ward — Date: 03/29/18
Sample Type: C
Depth (ft.): 81-81.5

Sample Diameter(in):	2.415	2.415	2.415
Sample Thickness(in.):	1.000	1.000	1.000
Weight of Sample + ring(gm):	165.10	169.90	169.61
Weight of Ring(gm):	41.63	42.84	41.73
Before Shearing			
Weight of Wet Sample+Cont.(gm):	325.91	325.91	325.91
Weight of Dry Sample+Cont.(gm):	318.95	318.95	318.95
Weight of Container(gm):	63.37	63.37	63.37
Vertical Rdg.(in): Initial	0.3130	0.3819	0.0000
Vertical Rdg.(in): Final	0.3438	0.4127	-0.0460
After Shearing			
Weight of Wet Sample+Cont.(gm):	197.03	181.81	201.91
Weight of Dry Sample+Cont.(gm):	173.27	157.64	179.25
Weight of Container(gm):	57.25	37.68	57.75
Specific Gravity (Assumed):	2.70	2.70	2.70
Water Density(pcf):	62.43	62.43	62.43





Boring No.	**B-1**
Sample No.	**13b**
Depth (ft)	**81-81.5**

Sample Type:

C

Soil Identification:
Yellowish brown silty sand (SM)

Normal Stress (kip/ft²)	4.000	8.000	12.000
Peak Shear Stress (kip/ft²)	● 3.288	■ 5.190	▲ 9.551
Shear Stress @ End of Test (ksf)	○ 2.663	□ 4.477	△ 7.894
Deformation Rate (in./min.)	0.0033	0.0033	0.0033
Initial Sample Height (in.)	1.000	1.000	1.000
Diameter (in.)	2.415	2.415	2.415
Initial Moisture Content (%)	2.72	2.72	2.72
Dry Density (pcf)	100.0	102.9	103.5
Saturation (%)	10.7	11.5	11.7
Soil Height Before Shearing (in.)	0.9692	0.9692	0.9540
Final Moisture Content (%)	20.5	20.1	18.7

Leighton

DIRECT SHEAR TEST RESULTS

Consolidated Drained - ASTM D 3080

Project No.: 15083A

1201 S. Grand Avenue Geotechnical Investigation

03-18

Leighton

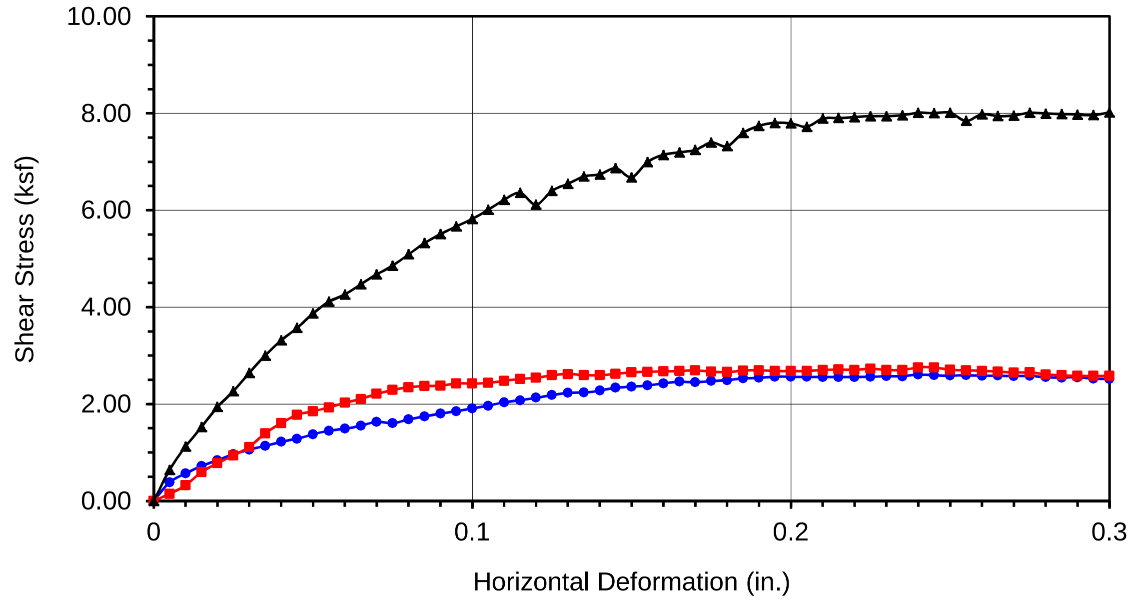
DIRECT SHEAR TEST

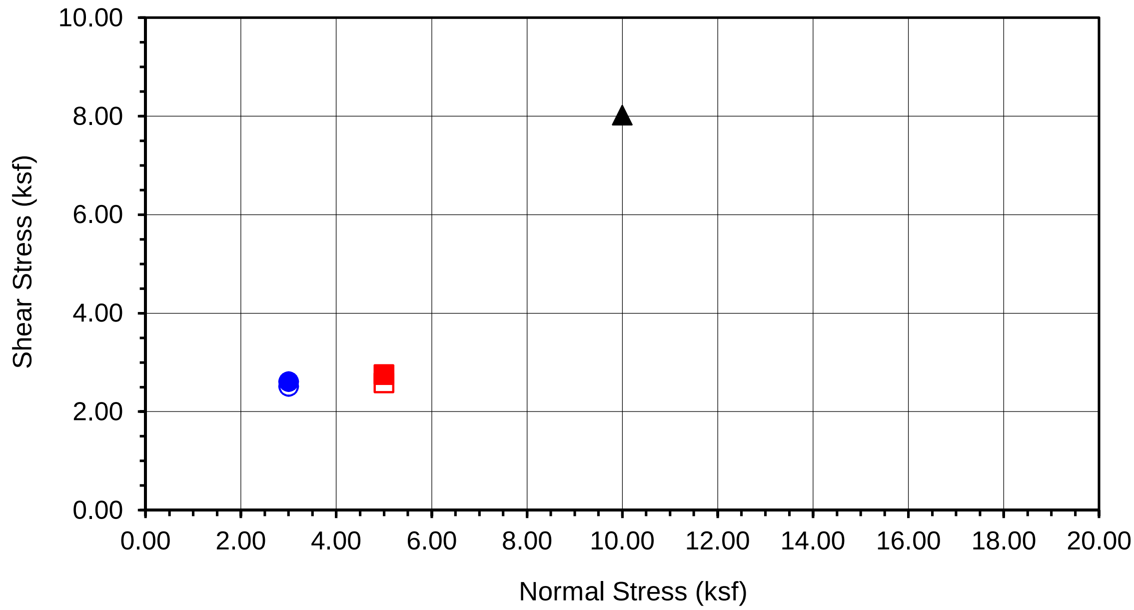
Consolidated Drained - ASTM D 3080

Project Name:	1201 S. Grand Avenue Geotechnical Investigation	Tested By:	R. Manning	Date:	03/13/18
Project No.:	15083A	Checked By:	J. Ward	Date:	03/29/18
Boring No.:	B-2	Sample Type:	C		
Sample No.:	8b	Depth (ft.):	40.3-40.8		

Soil Identification: Olive brown well-graded sand with silt and gravel (SW-SM)g

Sample Diameter(in):	2.415	2.415	2.415
Sample Thickness(in.):	1.000	1.000	1.000
Weight of Sample + ring(gm):	173.18	176.48	176.30
Weight of Ring(gm):	45.59	45.88	42.46
Before Shearing			
Weight of Wet Sample+Cont.(gm):	227.74	227.74	227.74
Weight of Dry Sample+Cont.(gm):	220.46	220.46	220.46
Weight of Container(gm):	57.20	57.20	57.20
Vertical Rdg.(in): Initial	0.0000	0.3224	0.3163
Vertical Rdg.(in): Final	-0.0782	0.4011	0.4290
After Shearing			
Weight of Wet Sample+Cont.(gm):	209.34	189.93	177.18
Weight of Dry Sample+Cont.(gm):	191.57	171.12	162.23
Weight of Container(gm):	74.35	51.04	37.21
Specific Gravity (Assumed):	2.70	2.70	2.70
Water Density(pcf):	62.43	62.43	62.43





Boring No.	**B-2**
Sample No.	**8b**
Depth (ft)	**40.3-40.8**

Sample Type:

C

Soil Identification:
Olive brown well-graded sand with silt and gravel (SW-SM)g

Normal Stress (kip/ft²)	3.000	5.000	10.000
Peak Shear Stress (kip/ft²)	● 2.609	■ 2.757	▲ 8.020
Shear Stress @ End of Test (ksf)	○ 2.515	□ 2.584	△ 8.020
Deformation Rate (in./min.)	0.0025	0.0025	0.0025
Initial Sample Height (in.)	1.000	1.000	1.000
Diameter (in.)	2.415	2.415	2.415
Initial Moisture Content (%)	4.46	4.46	4.46
Dry Density (pcf)	101.6	104.0	106.6
Saturation (%)	18.3	19.4	20.7
Soil Height Before Shearing (in.)	0.9218	0.9213	0.8873
Final Moisture Content (%)	15.2	15.7	12.0

Leighton

DIRECT SHEAR TEST RESULTS
Consolidated Drained - ASTM D 3080

Project No.: 15083A
1201 S. Grand Avenue Geotechnical Investigation
03-18

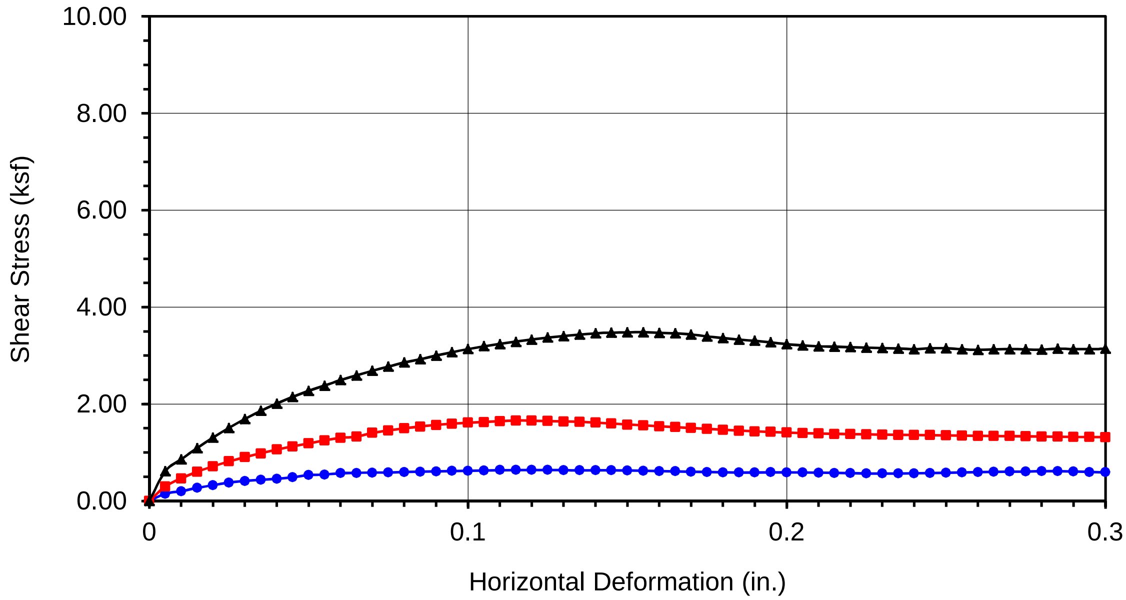
DIRECT SHEAR TEST

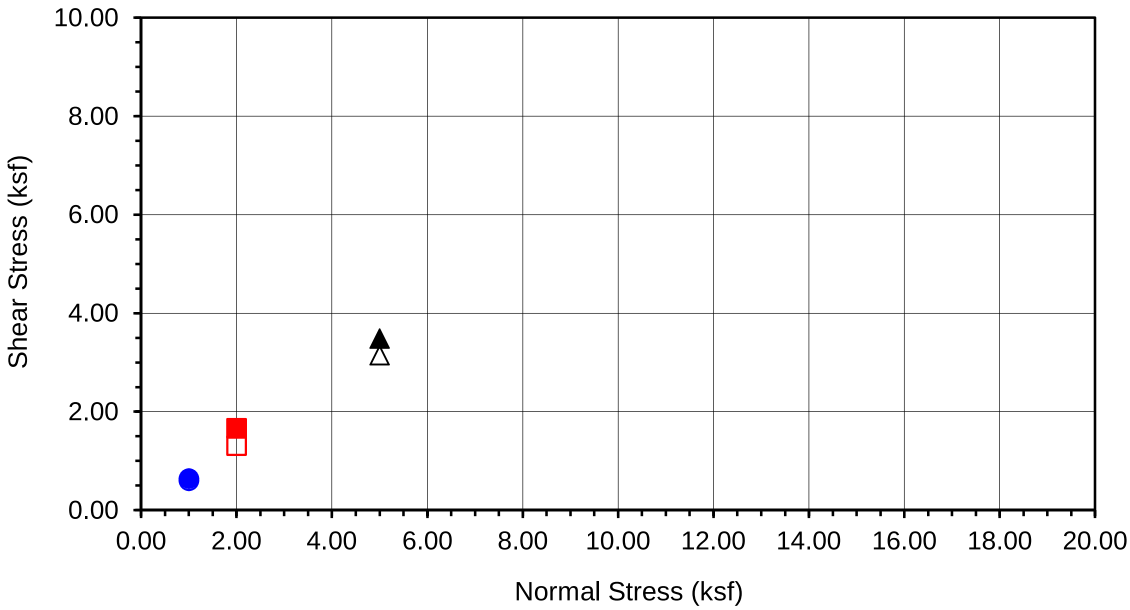
Consolidated Drained - ASTM D 3080

Leighton

Project Name:	1201 S. Grand Avenue Geotechnical Investigation	Tested By:	R. Manning	Date:	03/15/18
Project No.:	15083A	Checked By:	J. Ward	Date:	03/29/18
Boring No.:	B-2	Sample Type:	C		
Sample No.:	9b	Depth (ft.):	45.5-46		
Soil Identification:	Yellowish brown silty sand (SM)				

Sample Diameter(in):	2.415	2.415	2.415
Sample Thickness(in.):	1.000	1.000	1.000
Weight of Sample + ring(gm):	176.96	187.24	181.87
Weight of Ring(gm):	42.52	46.00	40.60
Before Shearing			
Weight of Wet Sample+Cont.(gm):	189.78	189.78	189.78
Weight of Dry Sample+Cont.(gm):	178.41	178.41	178.41
Weight of Container(gm):	57.57	57.57	57.57
Vertical Rdg.(in): Initial	0.0000	0.3316	0.3002
Vertical Rdg.(in): Final	-0.0178	0.3863	0.3660
After Shearing			
Weight of Wet Sample+Cont.(gm):	208.06	203.20	202.34
Weight of Dry Sample+Cont.(gm):	183.06	177.11	177.14
Weight of Container(gm):	63.28	57.25	57.60
Specific Gravity (Assumed):	2.70	2.70	2.70
Water Density(pcf):	62.43	62.43	62.43





Boring No.	**B-2**
Sample No.	**9b**
Depth (ft)	**45.5-46**

Sample Type:

C

Soil Identification:
Yellowish brown silty sand (SM)

Normal Stress (kip/ft²)	1.000	2.000	5.000
Peak Shear Stress (kip/ft²)	● 0.641	■ 1.660	▲ 3.483
Shear Stress @ End of Test (ksf)	○ 0.594	□ 1.314	△ 3.141
Deformation Rate (in./min.)	0.0017	0.0017	0.0017
Initial Sample Height (in.)	1.000	1.000	1.000
Diameter (in.)	2.415	2.415	2.415
Initial Moisture Content (%)	9.41	9.41	9.41
Dry Density (pcf)	102.2	107.4	107.4
Saturation (%)	39.1	44.6	44.6
Soil Height Before Shearing (in.)	0.9822	0.9453	0.9342
Final Moisture Content (%)	20.9	21.8	21.1

Leighton

DIRECT SHEAR TEST RESULTS
Consolidated Drained - ASTM D 3080

Project No.: 15083A
1201 S. Grand Avenue Geotechnical Investigation
03-18

DS B-2, 9b @ 45.5-46

DIRECT SHEAR TEST

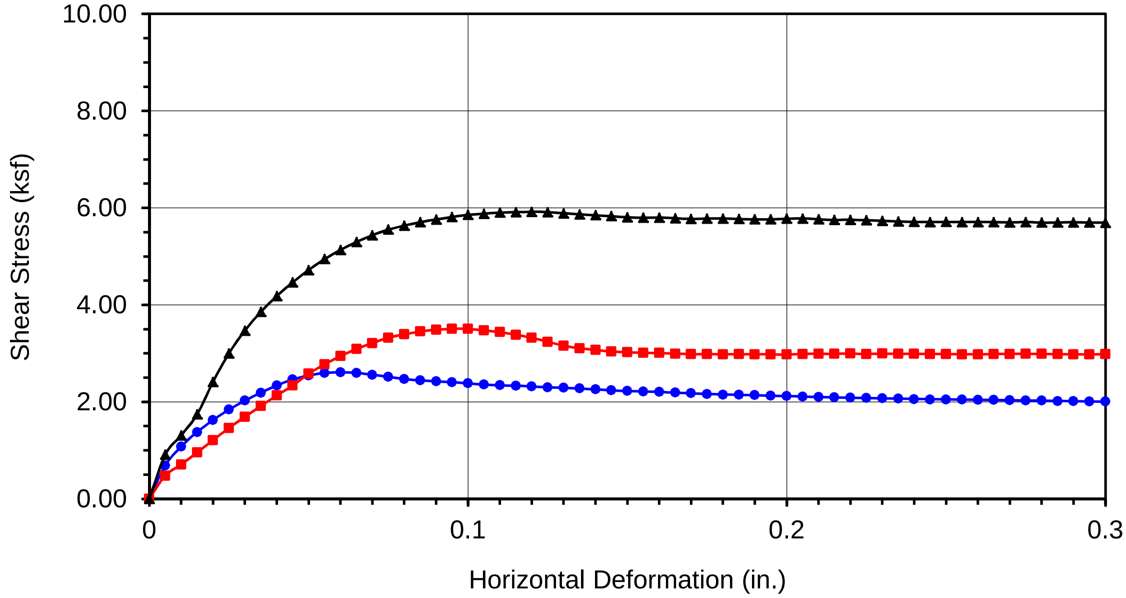
Consolidated Drained - ASTM D 3080

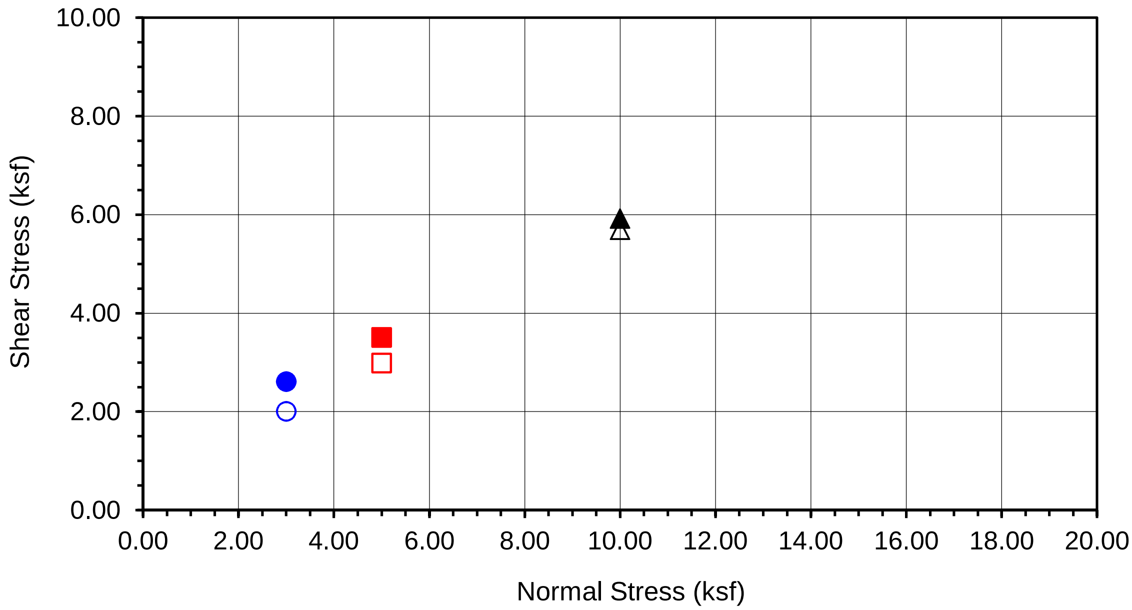
Leighton

Project Name:	1201 S. Grand Avenue Geotechnical Investigation	Tested By:	G. Bathala	Date:	03/14/18
Project No.:	15083A	Checked By:	J. Ward	Date:	03/29/18
Boring No.:	B-2	Sample Type:	C		
Sample No.:	10a	Depth (ft.):	50-50.5		

Soil Identification: Light olive brown sandy lean clay s(CL)

Sample Diameter(in):	2.415	2.415	2.415
Sample Thickness(in.):	1.000	1.000	1.000
Weight of Sample + ring(gm):	200.96	199.96	203.77
Weight of Ring(gm):	45.58	42.78	45.92
Before Shearing			
Weight of Wet Sample+Cont.(gm):	177.87	177.87	177.87
Weight of Dry Sample+Cont.(gm):	162.79	162.79	162.79
Weight of Container(gm):	55.80	55.80	55.80
Vertical Rdg.(in): Initial	0.2769	0.2473	0.0000
Vertical Rdg.(in): Final	0.2891	0.2681	-0.0418
After Shearing			
Weight of Wet Sample+Cont.(gm):	218.48	212.41	227.08
Weight of Dry Sample+Cont.(gm):	193.09	187.57	203.45
Weight of Container(gm):	61.49	55.75	71.81
Specific Gravity (Assumed):	2.70	2.70	2.70
Water Density(pcf):	62.43	62.43	62.43





Boring No.	**B-2**
Sample No.	**10a**
Depth (ft)	**50-50.5**

Sample Type:

C

Soil Identification:

Light olive brown sandy lean clay s(CL)

Normal Stress (kip/ft²)	3.000	5.000	10.000
Peak Shear Stress (kip/ft²)	● 2.609	■ 3.505	▲ 5.920
Shear Stress @ End of Test (ksf)	○ 2.006	□ 2.987	△ 5.693
Deformation Rate (in./min.)	0.0017	0.0017	0.0017
Initial Sample Height (in.)	1.000	1.000	1.000
Diameter (in.)	2.415	2.415	2.415
Initial Moisture Content (%)	14.09	14.09	14.09
Dry Density (pcf)	113.3	114.6	115.1
Saturation (%)	77.9	80.8	81.8
Soil Height Before Shearing (in.)	0.9878	0.9792	0.9582
Final Moisture Content (%)	19.3	18.8	18.0

Leighton

DIRECT SHEAR TEST RESULTS

Consolidated Drained - ASTM D 3080

Project No.: 15083A

1201 S. Grand Avenue Geotechnical Investigation

03-18

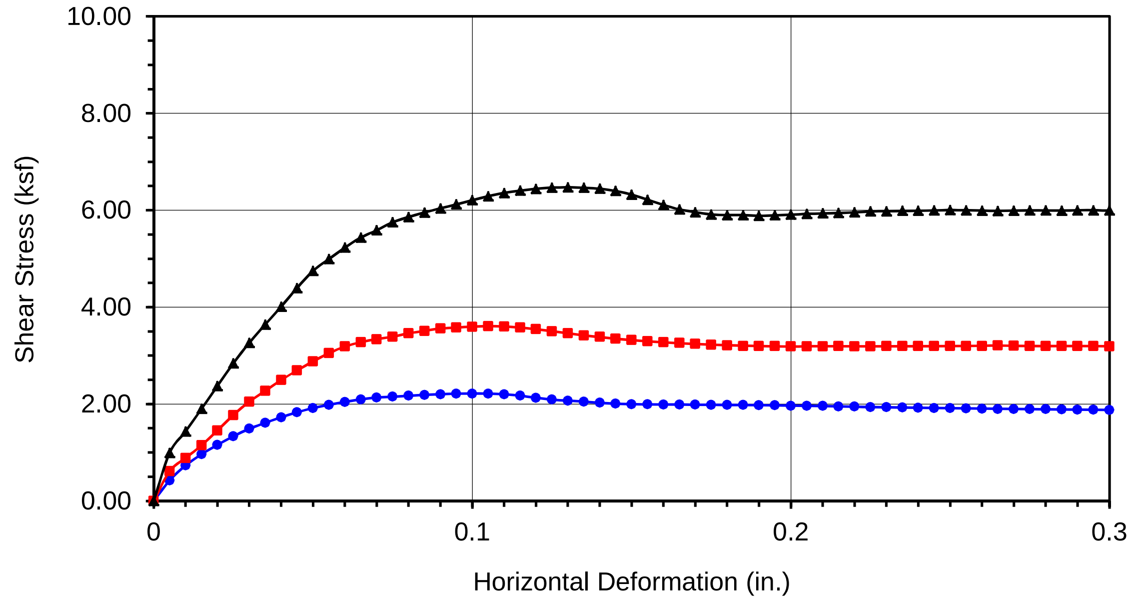
DIRECT SHEAR TEST

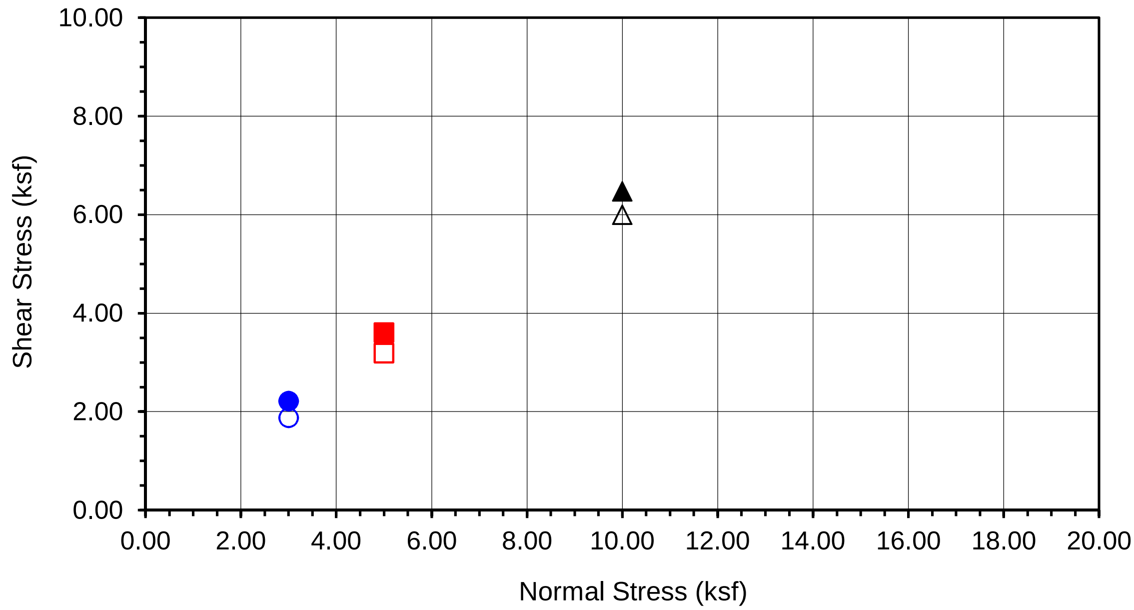
Consolidated Drained - ASTM D 3080

Project Name:	1201 S. Grand Avenue Geotechnical Investigation	Tested By:	R. Manning	Date:	03/15/18
Project No.:	15083A	Checked By:	J. Ward	Date:	03/29/18
Boring No.:	B-2	Sample Type:	C		
Sample No.:	10b	Depth (ft.):	50.5-51		

Soil Identification: Yellowish brown sandy lean clay s(CL)

Sample Diameter(in):	2.415	2.415	2.415
Sample Thickness(in.):	1.000	1.000	1.000
Weight of Sample + ring(gm):	194.12	199.72	199.49
Weight of Ring(gm):	40.65	43.81	42.47
Before Shearing			
Weight of Wet Sample+Cont.(gm):	191.83	191.83	191.83
Weight of Dry Sample+Cont.(gm):	167.60	167.60	167.60
Weight of Container(gm):	39.29	39.29	39.29
Vertical Rdg.(in): Initial	0.0000	0.2624	0.2382
Vertical Rdg.(in): Final	-0.0316	0.2924	0.2864
After Shearing			
Weight of Wet Sample+Cont.(gm):	215.02	193.28	238.28
Weight of Dry Sample+Cont.(gm):	188.73	168.30	214.15
Weight of Container(gm):	60.94	37.70	82.57
Specific Gravity (Assumed):	2.70	2.70	2.70
Water Density(pcf):	62.43	62.43	62.43





Boring No.	**B-2**
Sample No.	**10b**
Depth (ft)	**50.5-51**

Sample Type:

C

Soil Identification:

Yellowish brown sandy lean clay s(CL)

Normal Stress (kip/ft²)	3.000	5.000	10.000
Peak Shear Stress (kip/ft²)	● 2.216	■ 3.606	▲ 6.473
Shear Stress @ End of Test (ksf)	○ 1.877	□ 3.191	△ 5.995
Deformation Rate (in./min.)	0.0017	0.0017	0.0017
Initial Sample Height (in.)	1.000	1.000	1.000
Diameter (in.)	2.415	2.415	2.415
Initial Moisture Content (%)	18.88	18.88	18.88
Dry Density (pcf)	107.4	109.1	109.8
Saturation (%)	89.4	93.5	95.4
Soil Height Before Shearing (in.)	0.9684	0.9700	0.9518
Final Moisture Content (%)	20.6	19.1	18.3

Leighton

DIRECT SHEAR TEST RESULTS

Consolidated Drained - ASTM D 3080

Project No.: 15083A

1201 S. Grand Avenue Geotechnical Investigation

03-18

DIRECT SHEAR TEST

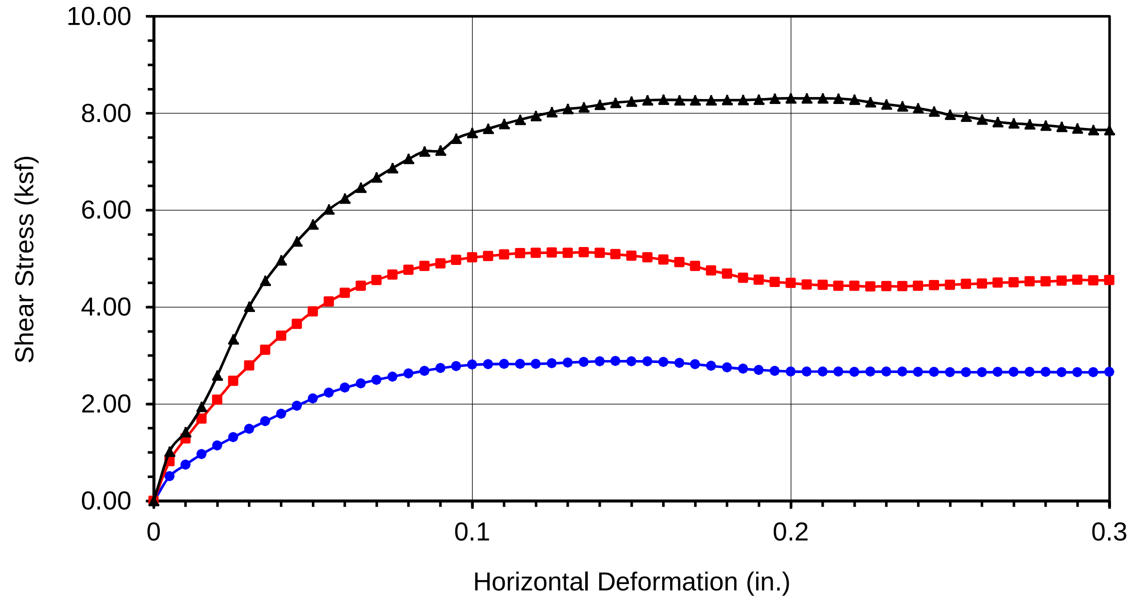
Consolidated Drained - ASTM D 3080

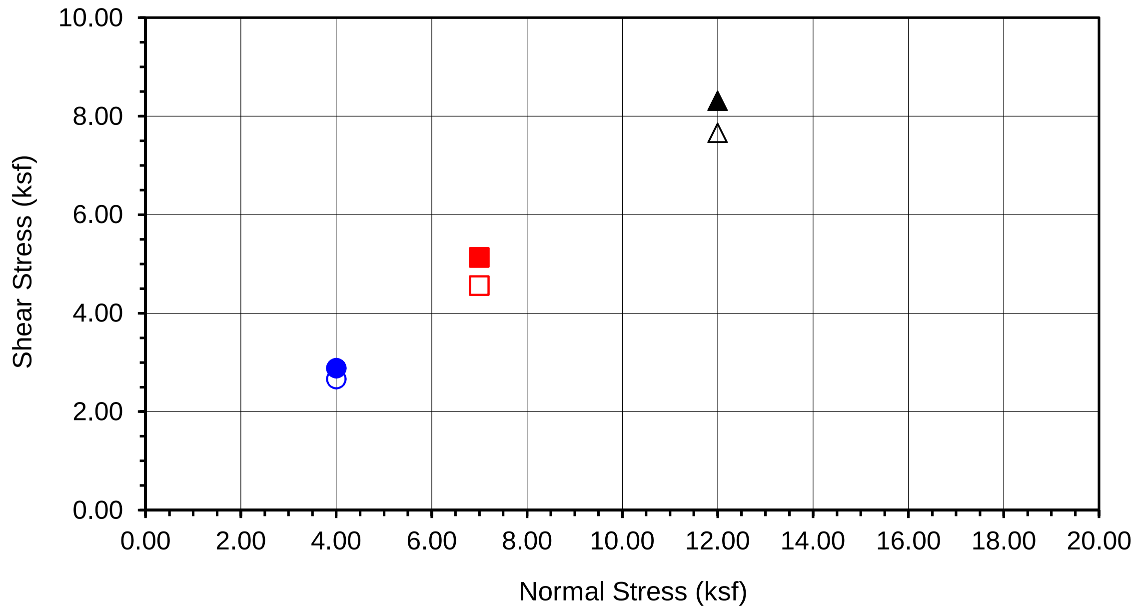
Leighton

Project Name:	1201 S. Grand Avenue Geotechnical Investigation	Tested By:	R. Manning	Date:	03/15/18
Project No.:	15083A	Checked By:	J. Ward	Date:	03/29/18
Boring No.:	B-2	Sample Type:	C		
Sample No.:	11b	Depth (ft.):	60.5-61		

Soil Identification: Dark yellowish brown sandy lean clay s(CL)

Sample Diameter(in):	2.415	2.415	2.415
Sample Thickness(in.):	1.000	1.000	1.000
Weight of Sample + ring(gm):	203.39	203.75	202.15
Weight of Ring(gm):	45.89	45.60	42.49
Before Shearing			
Weight of Wet Sample+Cont.(gm):	548.47	548.47	548.47
Weight of Dry Sample+Cont.(gm):	485.44	485.44	485.44
Weight of Container(gm):	82.54	82.54	82.54
Vertical Rdg.(in): Initial	0.0000	0.3205	0.2908
Vertical Rdg.(in): Final	-0.0332	0.3617	0.3359
After Shearing			
Weight of Wet Sample+Cont.(gm):	212.59	211.68	216.78
Weight of Dry Sample+Cont.(gm):	188.61	189.30	196.31
Weight of Container(gm):	57.77	57.18	60.20
Specific Gravity (Assumed):	2.70	2.70	2.70
Water Density(pcf):	62.43	62.43	62.43





Boring No.	**B-2**
Sample No.	**11b**
Depth (ft)	**60.5-61**

Sample Type:

C

Soil Identification:
Dark yellowish brown sandy lean clay s(CL)

Normal Stress (kip/ft²)	4.000	7.000	12.000
Peak Shear Stress (kip/ft²)	● 2.886	■ 5.131	▲ 8.312
Shear Stress @ End of Test (ksf)	○ 2.660	□ 4.558	△ 7.658
Deformation Rate (in./min.)	0.0017	0.0017	0.0017
Initial Sample Height (in.)	1.000	1.000	1.000
Diameter (in.)	2.415	2.415	2.415
Initial Moisture Content (%)	15.64	15.64	15.64
Dry Density (pcf)	113.3	113.7	114.8
Saturation (%)	86.5	87.6	90.2
Soil Height Before Shearing (in.)	0.9668	0.9588	0.9549
Final Moisture Content (%)	18.3	16.9	15.0

Leighton

DIRECT SHEAR TEST RESULTS

Consolidated Drained - ASTM D 3080

Project No.: 15083A

1201 S. Grand Avenue Geotechnical Investigation

03-18

Boring No.	B-1	B-2	B-2	B-2	B-2	B-2	B-2	B-2
Sample No.	13b	8b	9b	10a	10b	11a	12a	14b
Depth (ft.)	81-81.5	40.3-40.8	45.5-46	50-50.5	50.5-51	60-60.5	70-70.3	100.5-101
Sample Type	C	C	C	C	C	C	C	C
Soil Identification	Yellowish brown silty sand (SM)	Olive brown well-graded sand with silt and gravel (SW-SM)g	Yellowish brown silty sand (SM)	Light olive brown sandy lean clay s(CL)	Yellowish brown sandy lean clay s(CL)	Dark yellowish brown lean clay with sand (CL)s	Yellowish brown sandy silt s(ML)	Yellowish brown silty clay with sand (CL-ML)s
Pocket Penetrometer (tons/ft^2)	N/A	N/A	2.50	4.25/>4.50	>4.50	>4.5	2.50	>4.5
Weight Soil + Rings / Tube (g)	850.00	1106.58	1086.80	1181.90	1205.40	1184.10	943.00	1159.40
Weight of Rings / Tube (g)	210.35	275.30	274.75	273.61	275.89	275.13	274.87	276.37
Average Length (in.)	5.000	6.175	6.234	5.910	6.020	5.869	4.310	6.130
Average Diameter (in.)	2.415	2.387	2.387	2.387	2.387	2.387	2.387	2.387
Wet. Wt. of Soil + Cont. (g)	325.91	227.74	189.78	177.87	191.83	701.77	668.13	575.79
Dry Wt. of Soil + Cont. (g)	318.95	220.46	178.41	162.79	167.60	625.41	539.40	463.40
Weight of Container (g)	63.37	57.20	57.57	55.80	39.29	218.30	0.00	99.69
Container No.								
Wet Density	106.4	114.6	110.9	130.8	131.4	131.8	132.0	122.6
Moisture Content (%)	**2.7**	**4.5**	**9.4**	**14.1**	**18.9**	**18.8**	**23.9**	**30.9**
Dry Density (pcf)	**103.6**	**109.7**	**101.4**	**114.7**	**110.6**	**111.0**	**106.5**	**93.7**
Degree of Saturation (%)	11.7	22.4	38.3	81.0	97.2	97.7	110.7	104.4

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MOISTURE & DENSITY of SOILS

ASTM D 2216 & ASTM D 2937

Project Name: 1201 S. Grand Avenue Geotechical Investigation

Project No.: 15083A

Tested By: RMM/GB Date: 03/16/18

MOISTURE CONTENT
ASTM D 2216

Leighton

Project Name: **1201 S. Grand Avenue Geotechnical Investigation**
Project No.: **15083A**

Tested By: O. Figueroa
Date: 03/13/18
Checked By: J. Ward
Date: 03/30/18

Boring No.	B-1	B-1	B-1	B-1	B-1
Sample No.	2	4	6	8	9a
Depth (ft)	10-11.5	20-21.5	30-31.5	40-41.5	45-46
Sample Type	S	S	S	S	S
Sample Description	Brown poorly-graded sand with silt & gravel (SP-SM)g	Brown well-graded sand with silt & gravel (SW-SM)g	Dark brown silty clay with sand (CL-ML)s	Brown well-graded sand with silt & gravel (SW-SM)g	Olive brown sandy lean clay s(CL)
Wt. wet soil + container (g)	351.7	908.8	366.9	672.4	455.6
Wt. dry soil + container (g)	346.3	877.3	322.6	644.2	406.1
Weight of container (g)	51.1	82.7	39.2	79.1	77.4
Moisture Content (%)	**1.8**	**4.0**	**15.6**	**5.0**	**15.1**

Boring No.	B-1	B-1	B-1	B-1	B-1
Sample No.	11	12	14	15	16
Depth (ft)	60-61.5	70-71.5	90-91.5	100-101.5	110-111.5
Sample Type	S	S	S	S	S
Sample Description	Dark brown fat clay with sand (CH)s	Brown poorly-graded sand with silt (SP-SM)	Brown poorly-graded sand with silt & gravel (SP-SM)g	Olive brown silt with sand (ML)s	Dark brown poorly-graded sand with silt & gravel (SP-SM)g
Wt. wet soil + container (g)	605.3	251.9	300.2	1032.2	288.0
Wt. dry soil + container (g)	503.2	244.0	291.9	858.4	274.9
Weight of container (g)	77.8	38.8	62.7	108.6	38.8
Moisture Content (%)	**24.0**	**3.8**	**3.6**	**23.2**	**5.5**

MOISTURE CONTENT
ASTM D 2216

Leighton

Project Name: **1201 S. Grand Avenue Geotechnical Investigation**
Project No.: **15083A**

Tested By: O. Figueroa
Date: 03/13/18
Checked By: J. Ward
Date: 03/30/18

Boring No.	B-1	B-1	B-1	B-2	B-2
Sample No.	17	18	19	2	4
Depth (ft)	120-121.5	135-136.5	150-151.5	10-11.5	20-21.5
Sample Type	S	S	S	S	S
Sample Description	Brown poorly-graded sand with silt & gravel (SP-SM)g	Light olive brown poorly-graded sand with silt (SP-SM)	Yellowish brown poorly-graded sand with silt (SP-SM)	Brown poorly-graded sand with silt & gravel (SP-SM)g	Brown poorly-graded sand with silt & gravel (SP-SM)g
Wt. wet soil + container (g)	491.3	492.3	239.1	343.7	573.3
Wt. dry soil + container (g)	466.7	463.3	227.3	338.2	551.0
Weight of container (g)	82.6	76.7	38.3	39.8	76.1
Moisture Content (%)	**6.4**	**7.5**	**6.2**	**1.8**	**4.7**

Boring No.	B-2	B-2			
Sample No.	6	13			
Depth (ft)	30-31.5	80-80.4			
Sample Type	S	C			
Sample Description	Olive brown poorly-graded sand with silt & gravel (SP-SM)g	Olive brown poorly-graded sand with silt & gravel (SP-SM)g			
Wt. wet soil + container (g)	429.1	715.6			
Wt. dry soil + container (g)	412.9	682.1			
Weight of container (g)	77.2	74.9			
Moisture Content (%)	**4.8**	**5.5**			

Leighton

ATTERBERG LIMITS

ASTM D 4318

Project Name:	1201 S. Grand Avenue Geotechnical Investigation	Tested By:	R. Manning	Date:	03/16/18
Project No. :	15083A	Input By:	J. Ward	Date:	03/28/18
Boring No.:	B-1	Checked By:	J. Ward		
Sample No.:	9a & 9b combined	Depth (ft.)	45-46.5		
Soil Identification:	Olive brown sandy lean clay s(CL)				

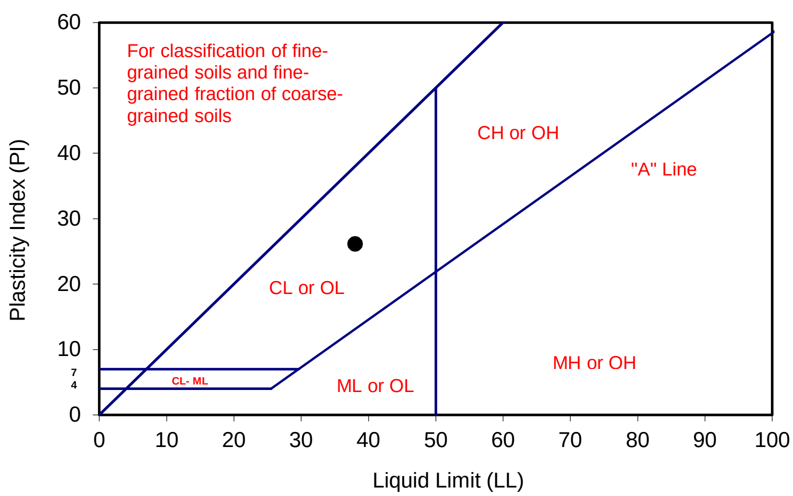
TEST	PLASTIC LIMIT		LIQUID LIMIT			
NO.	1	2	1	2	3	4
Number of Blows [N]			34	23	17	
Wet Wt. of Soil + Cont. (g)	21.12	21.37	27.14	26.22	26.87	
Dry Wt. of Soil + Cont. (g)	20.08	20.28	23.53	22.78	23.14	
Wt. of Container (g)	11.30	11.05	13.65	13.76	13.67	
Moisture Content (%) [Wn]	11.85	11.81	36.54	38.14	39.39	

Liquid Limit	**38**
Plastic Limit	**12**
Plasticity Index	**26**
Classification	**CL**

PI at "A" - Line = 0.73(LL-20) 13.14

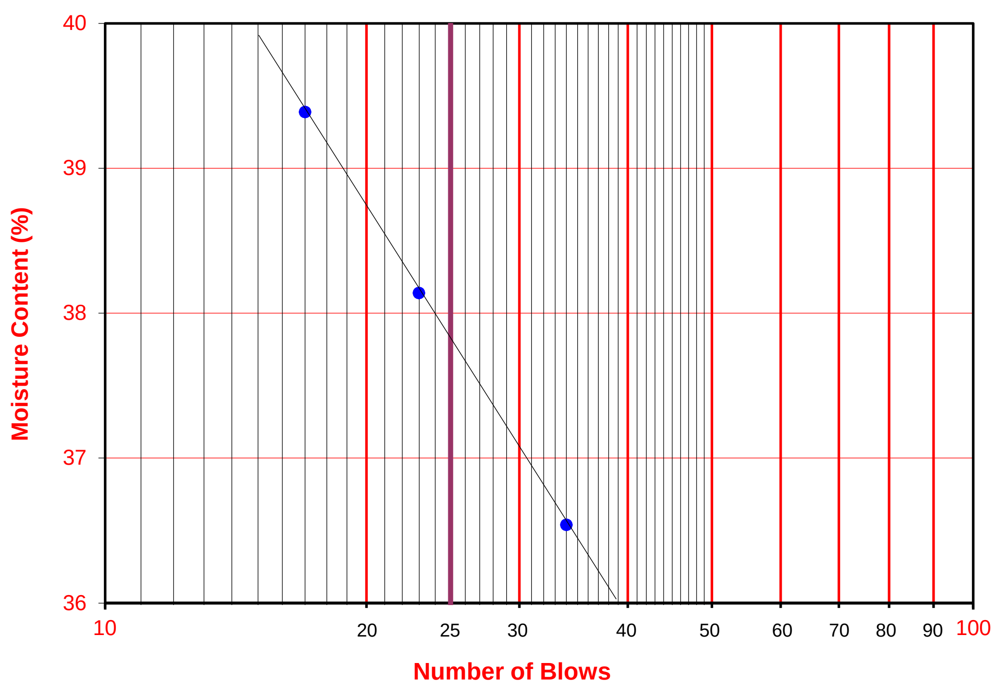
One - Point Liquid Limit Calculation

$LL = Wn(N/25)^{0.121}$



PROCEDURES USED

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



ATTERBERG LIMITS

ASTM D 4318

Project Name:	1201 S. Grand Avenue Geotechnical Investigation	Tested By:	R. Manning	Date:	03/14/18
Project No. :	15083A	Input By:	J. Ward	Date:	03/28/18
Boring No.:	B-1	Checked By:	J. Ward		
Sample No.:	11	Depth (ft.)	60-61.5		
Soil Identification:	Dark brown fat clay with sand (CH)s				

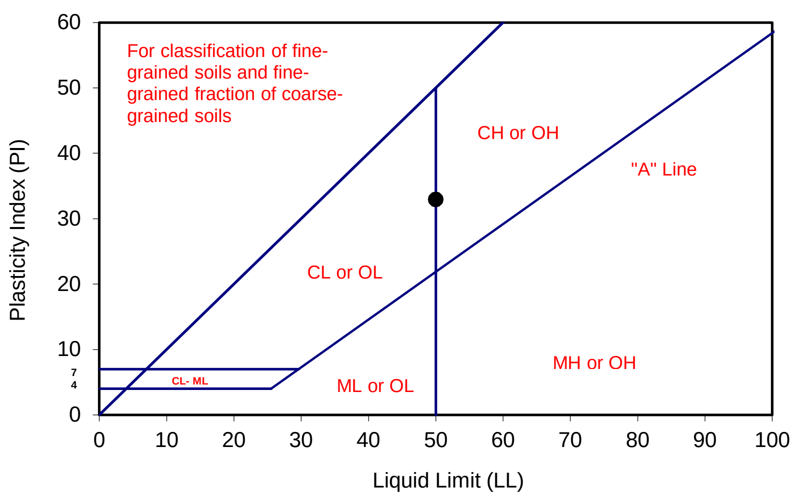
TEST	PLASTIC LIMIT		LIQUID LIMIT			
NO.	1	2	1	2	3	4
Number of Blows [N]			32	25	20	
Wet Wt. of Soil + Cont. (g)	17.84	17.97	25.06	26.09	26.82	
Dry Wt. of Soil + Cont. (g)	16.89	17.02	21.38	21.96	22.38	
Wt. of Container (g)	11.30	11.46	13.71	13.66	13.76	
Moisture Content (%) [Wn]	16.99	17.09	47.98	49.76	51.51	

Liquid Limit	**50**
Plastic Limit	**17**
Plasticity Index	**33**
Classification	**CH**

PI at "A" - Line = 0.73(LL-20) 21.9

One - Point Liquid Limit Calculation

$LL = Wn(N/25)^{0.121}$



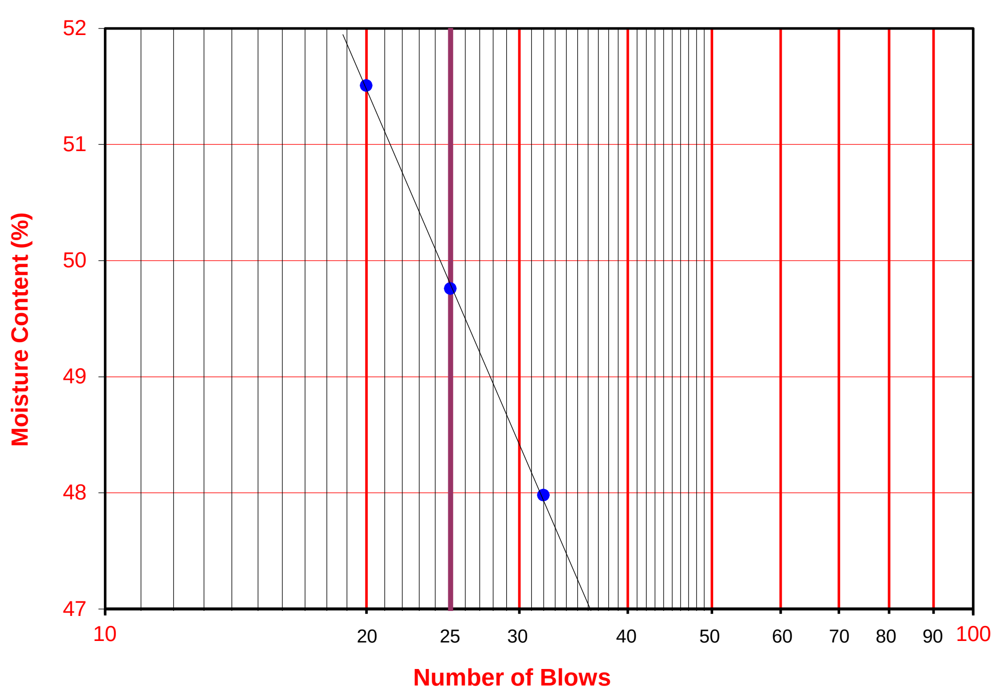
PROCEDURES USED

☐ Wet Preparation
Multipoint - Wet

X Dry Preparation
Multipoint - Dry

X Procedure A
Multipoint Test

☐ Procedure B
One-point Test



ATTERBERG LIMITS

ASTM D 4318

Leighton

Project Name:	1201 S. Grand Avenue Geotechnical Investigation	Tested By:	R. Manning	Date:	03/16/18
Project No. :	15083A	Input By:	J. Ward	Date:	03/28/18
Boring No.:	B-2	Checked By:	J. Ward		
Sample No.:	9b	Depth (ft.)	45.5-46		
Soil Identification:	Yellowish brown silty sand (SM)				

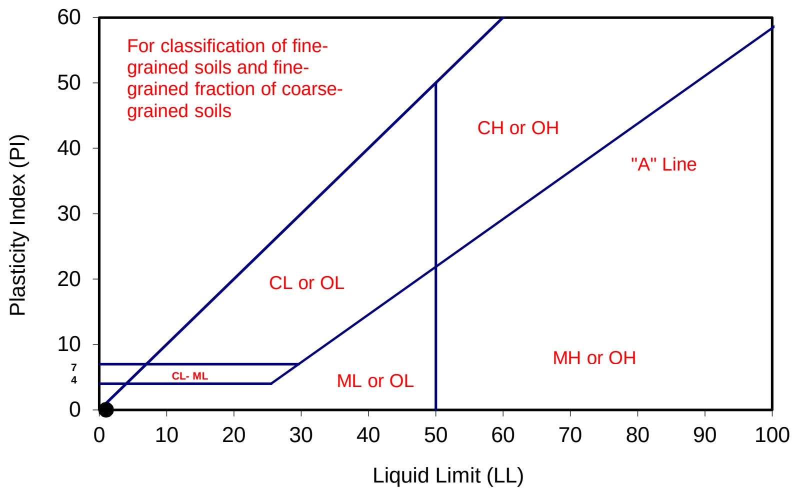
TEST	PLASTIC LIMIT		LIQUID LIMIT			
NO.	1	2	1	2	3	4
Number of Blows [N]			7			
Wet Wt. of Soil + Cont. (g)	**Cannot be rolled:**		28.81	**Cannot get more than 7 blows:**		
Dry Wt. of Soil + Cont. (g)	**NonPlastic**		25.65	**NonPlastic**		
Wt. of Container (g)			13.68			
Moisture Content (%) [Wn]			26.40			

Liquid Limit	**NP**
Plastic Limit	**NP**
Plasticity Index	**NP**
Classification	**NP**

PI at "A" - Line = 0.73(LL-20) =

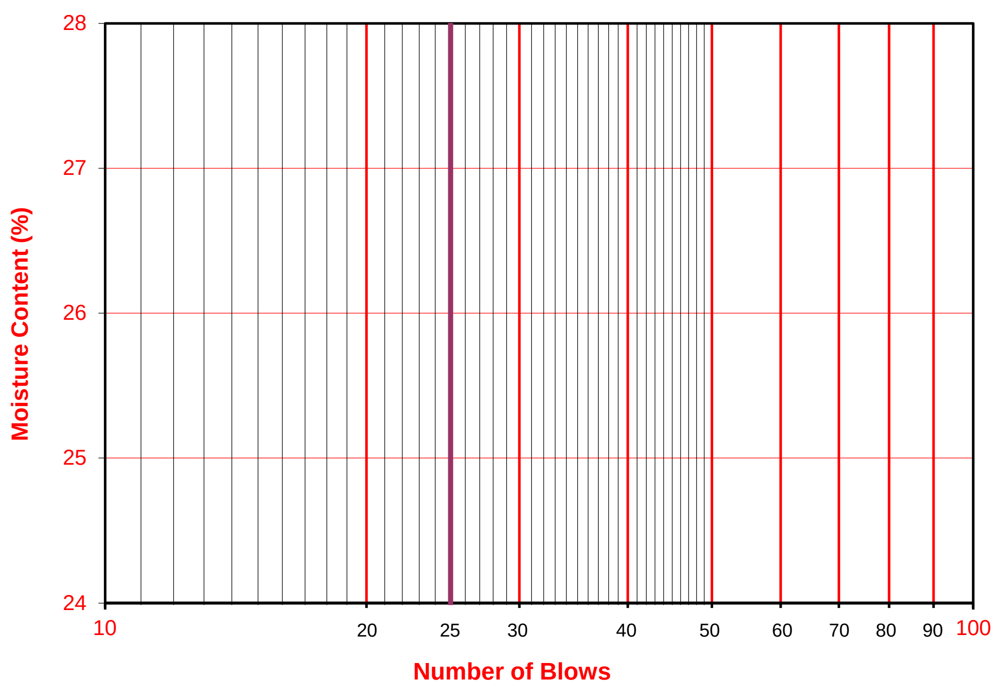
One - Point Liquid Limit Calculation

$LL = Wn(N/25)^{0.121}$



PROCEDURES USED

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



ATTERBERG LIMITS

ASTM D 4318

Project Name:	1201 S. Grand Avenue Geotechnical Investigation	Tested By:	G. Bathala	Date:	03/23/18
Project No. :	15083A	Input By:	J. Ward	Date:	03/28/18
Boring No.:	B-2	Checked By:	J. Ward		
Sample No.:	10a	Depth (ft.)	50-50.5		
Soil Identification:	Light olive brown sandy lean clay s(CL)				

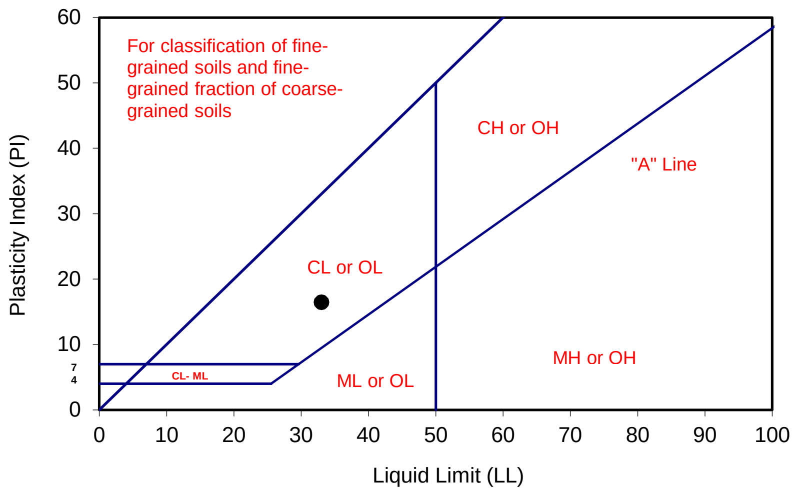
TEST NO.	PLASTIC LIMIT 1	PLASTIC LIMIT 2	LIQUID LIMIT 1	LIQUID LIMIT 2	LIQUID LIMIT 3	LIQUID LIMIT 4
Number of Blows [N]			32	25	18	
Wet Wt. of Soil + Cont. (g)	20.74	21.90	23.56	22.84	23.28	
Dry Wt. of Soil + Cont. (g)	19.37	20.46	20.57	20.06	20.16	
Wt. of Container (g)	11.11	11.71	11.30	11.71	11.28	
Moisture Content (%) [Wn]	16.59	16.46	32.25	33.29	35.14	

Liquid Limit	**33**
Plastic Limit	**17**
Plasticity Index	**16**
Classification	**CL**

PI at "A" - Line = 0.73(LL-20) 9.49

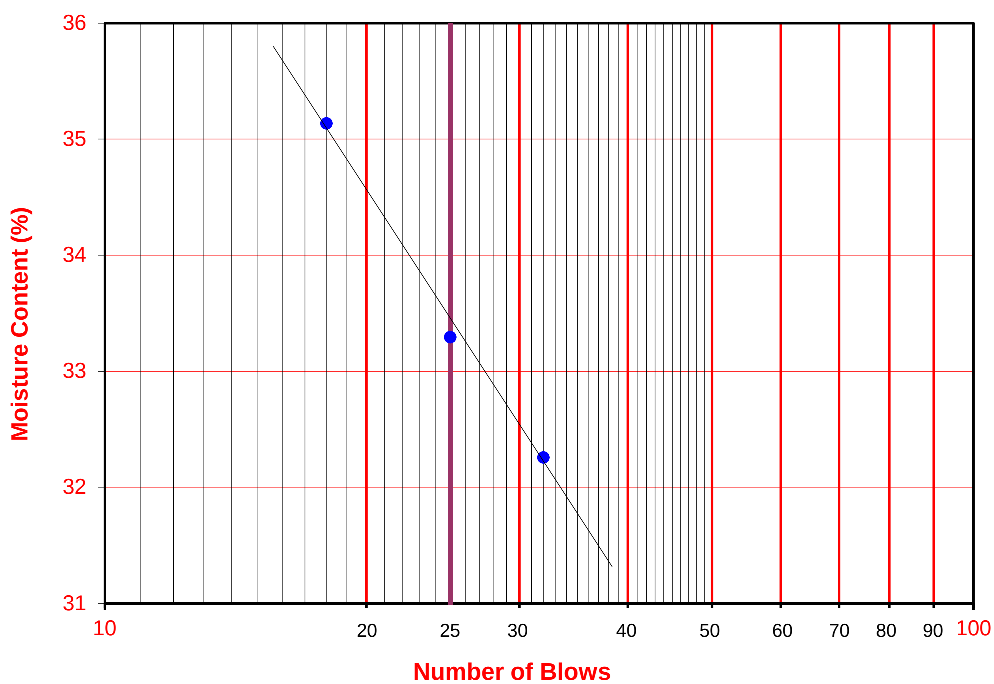
One - Point Liquid Limit Calculation

$LL = Wn(N/25)^{0.121}$



PROCEDURES USED

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



ATTERBERG LIMITS

ASTM D 4318

Project Name:	1201 S. Grand Avenue Geotechnical Investigation	Tested By:	R. Manning	Date:	03/20/18
Project No. :	15083A	Input By:	J. Ward	Date:	03/28/18
Boring No.:	B-2	Checked By:	J. Ward		
Sample No.:	10b	Depth (ft.)	50.5-51		
Soil Identification:	Yellowish brown sandy lean clay s(CL)				

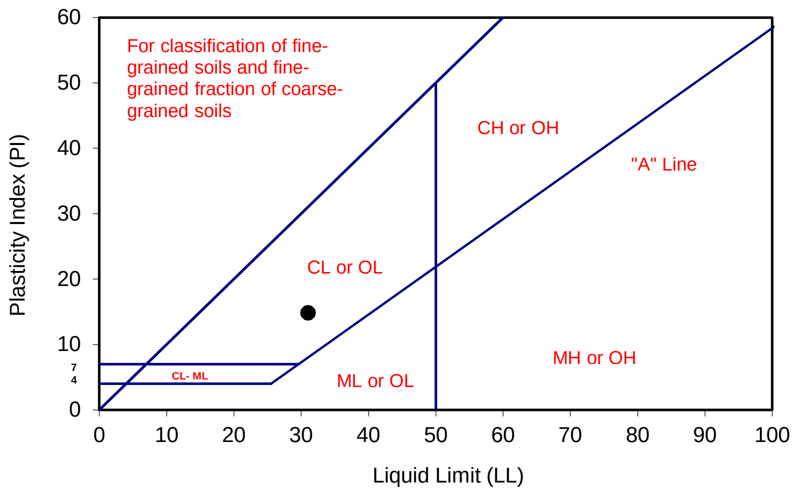
TEST	PLASTIC LIMIT		LIQUID LIMIT			
NO.	1	2	1	2	3	4
Number of Blows [N]			30	23	18	
Wet Wt. of Soil + Cont. (g)	19.38	19.76	28.26	29.08	29.09	
Dry Wt. of Soil + Cont. (g)	18.24	18.60	24.85	25.37	25.26	
Wt. of Container (g)	11.14	11.46	13.65	13.67	13.65	
Moisture Content (%) [Wn]	16.06	16.25	30.45	31.71	32.99	

Liquid Limit	**31**
Plastic Limit	**16**
Plasticity Index	**15**
Classification	**CL**

PI at "A" - Line = 0.73(LL-20) 8.03

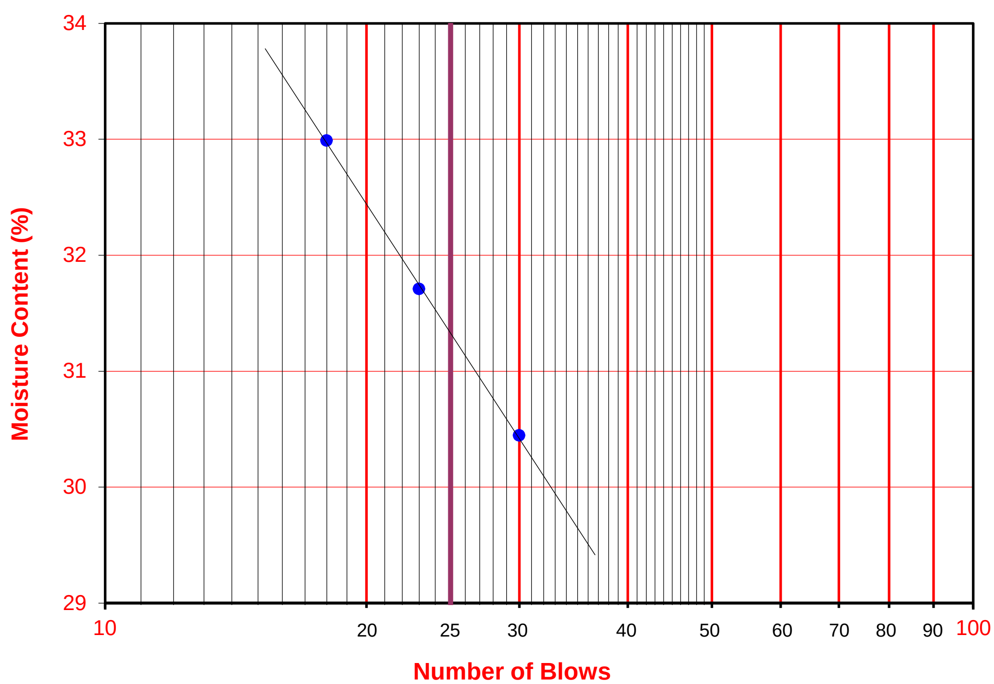
One - Point Liquid Limit Calculation

$LL = Wn(N/25)^{0.121}$



PROCEDURES USED

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



ATTERBERG LIMITS

ASTM D 4318

Project Name:	1201 S. Grand Avenue Geotechnical Investigation	Tested By:	R. Manning	Date:	03/20/18
Project No. :	15083A	Input By:	J. Ward	Date:	03/28/18
Boring No.:	B-2	Checked By:	J. Ward		
Sample No.:	11b	Depth (ft.)	60.5-61		

Soil Identification: Dark yellowish brown sandy lean clay s(CL)

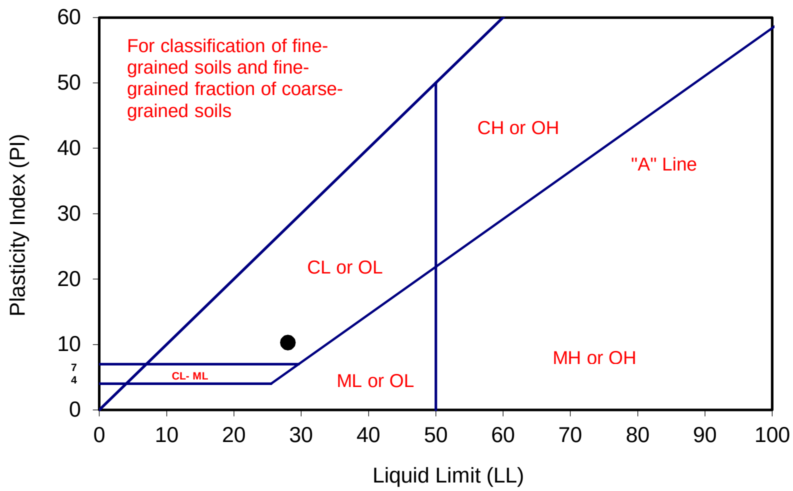
TEST NO.	PLASTIC LIMIT 1	PLASTIC LIMIT 2	LIQUID LIMIT 1	LIQUID LIMIT 2	LIQUID LIMIT 3	LIQUID LIMIT 4
Number of Blows [N]			34	26	19	
Wet Wt. of Soil + Cont. (g)	17.99	17.76	29.19	29.71	27.77	
Dry Wt. of Soil + Cont. (g)	17.06	16.85	25.90	26.21	24.55	
Wt. of Container (g)	11.77	11.75	13.67	13.78	13.63	
Moisture Content (%) [Wn]	17.58	17.84	26.90	28.16	29.49	

Liquid Limit	**28**
Plastic Limit	**18**
Plasticity Index	**10**
Classification	**CL**

PI at "A" - Line = 0.73(LL-20) 5.84

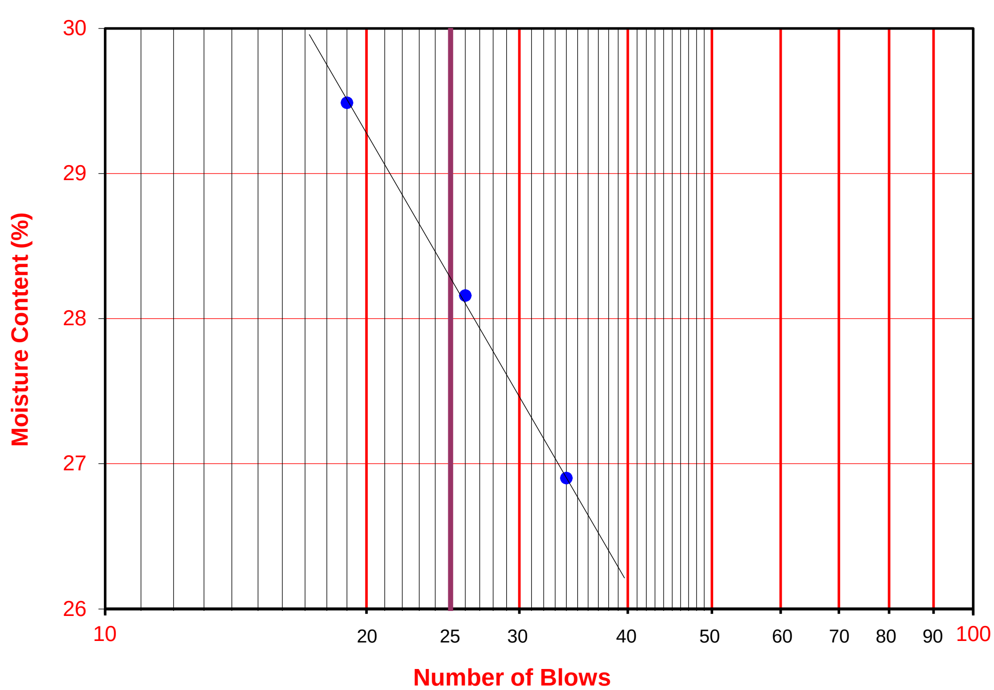
One - Point Liquid Limit Calculation

$LL = Wn(N/25)^{0.121}$



PROCEDURES USED

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



Leighton

PARTICLE-SIZE ANALYSIS OF SOILS

ASTM D 422

Project Name: 1201 S. Grand Avenue Geotechnical Investigation
Tested By: G. Berdy
Date: 03/15/18

Project No.: 15083A
Data Input By: J. Ward
Date: 03/28/18

Boring No.: B-1

Sample No.: 4
Depth (feet): 20-21.5

Soil Identification: Olive brown well-graded sand with silt and gravel (SW-SM)g

		Soil Type
% Gravel	40	(SW-SM)g
% Sand	49	
% Fines	11	

			Moisture Content of Total Air-Dry Soil	Moisture Content of Air-Dry Soil Passing #10	After Hydrometer & Wet Sieve ret. in #200 Sieve
Specific Gravity (Assumed)	2.70	Wt.of Air-Dry Soil + Cont.(g)	0.00	75.34	
Correction for Specific Gravity	0.99	Dry Wt. of Soil + Cont. (g)	0.00	75.25	158.98
Wt.of Air-Dry Soil + Cont. (g)	880.29	Wt. of Container No.___ (g)	1.00	57.72	77.30
Wt. of Container	82.65	Moisture Content (%)	0.00	0.51	
Dry Wt. of Soil (g)	797.64	Wt. of Dry Soil (g)			81.68

Coarse Sieve

U.S. Sieve	Cumulative Wt. Of Dry Soil Retained (g)	% Passing
3"	0.00	100.0
1½"	0.00	100.0
3/4"	83.23	89.6
3/8"	204.52	74.4
No. 4	316.37	60.3
No. 10	437.28	45.2
Pan		

Sieve after Hydrometer & Wet Sieve

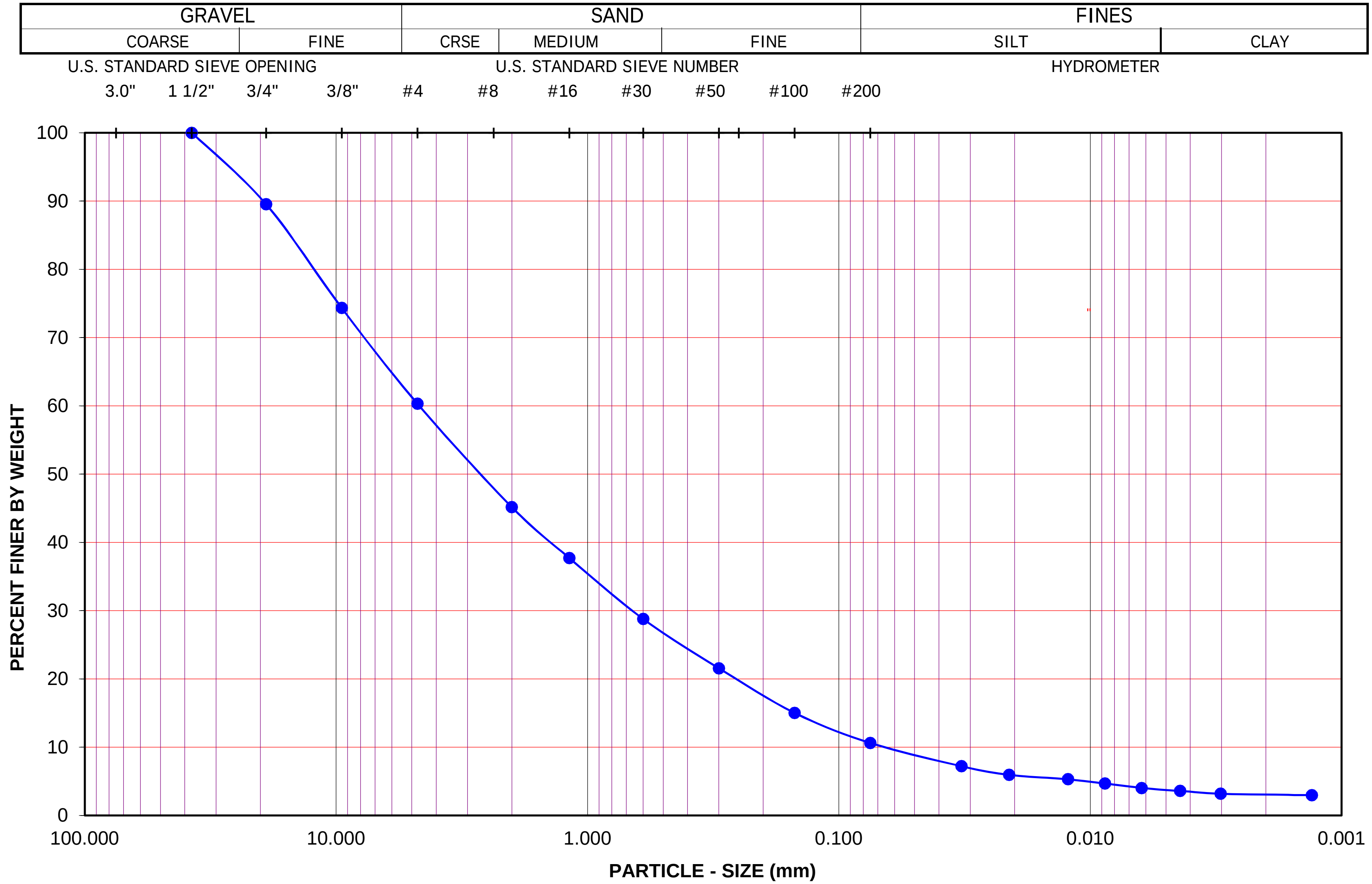
U.S. Sieve Size	Cumulative Wt. Of Dry Soil Retained (g)	% Passing	% Total Sample
No. 10	0.00	100.0	45.2
No. 16	17.40	83.5	37.7
No. 30	38.21	63.7	28.8
No. 50	55.02	47.8	21.6
No. 100	70.25	33.3	15.0
No. 200	80.53	23.5	10.6
Pan			

Hydrometer

Wt. of Air-Dry Soil (g) 105.85
Wt. of Dry Soil (g) 105.31

Deflocculant 125 cc of 4% Solution

Date	Time	Elapsed Time (min)	Water Temperature (°C)	Composite Correction 152H	Actual Hydrometer Readings	% Total Sample (%)	Soil Particle Diameter (mm)
19-Mar-18	9:38	0		9.0			
	9:40	2	21.3	9.0	26.0	7.2	0.0325
	9:43	5	21.3	9.0	23.0	6.0	0.0210
	9:53	15	21.3	9.0	21.5	5.3	0.0123
	10:08	30	21.4	9.0	20.0	4.7	0.0087
	10:38	60	21.6	9.0	18.5	4.0	0.0062
	11:38	120	22.2	9.0	17.5	3.6	0.0044
	13:48	250	23.0	9.0	16.5	3.2	0.0030
20-Mar-18	9:38	1440	20.9	9.0	16.0	3.0	0.0013



Project Name:	1201 S. Grand Avenue Geotechnical Investigation		
Project No.:	15083A		
Boring No.:	B-1	Sample No.:	4
Depth (feet):	20-21.5	Soil Type :	(SW-SM)g
Soil Identification:	Olive brown well-graded sand with silt and gravel (SW-SM)g		

Leighton

PARTICLE - SIZE DISTRIBUTION ASTM D 422

GR:SA:FI : (%) 40 : 49 : 11

Mar-18

Leighton

PARTICLE-SIZE ANALYSIS OF SOILS

ASTM D 422

Project Name: 1201 S. Grand Avenue Geotechnical Investigation
Tested By: G. Berdy
Date: 03/15/18

Project No.: 15083A
Data Input By: J. Ward
Date: 03/28/18

Boring No.: B-1

Sample No.: 15
Depth (feet): 100-101.5

Soil Identification: Olive brown silt with sand (ML)s

		Soil Type
% Gravel	**0**	**(ML)s**
% Sand	**26**	
% Fines	**74**	

			Moisture Content of Total Air-Dry Soil	Moisture Content of Air-Dry Soil Passing #10	After Hydrometer & Wet Sieve ret. in #200 Sieve
Specific Gravity (Assumed)	2.70	Wt.of Air-Dry Soil + Cont.(g)	0.00	70.72	
Correction for Specific Gravity	0.99	Dry Wt. of Soil + Cont. (g)	0.00	70.56	94.79
Wt.of Air-Dry Soil + Cont. (g)	872.23	Wt. of Container No.___ (g)	1.00	58.31	79.42
Wt. of Container	108.65	Moisture Content (%)	0.00	1.31	
Dry Wt. of Soil (g)	763.58	Wt. of Dry Soil (g)			15.37

Coarse Sieve

U.S. Sieve	Cumulative Wt. Of Dry Soil Retained (g)	% Passing
3"	0.00	100.0
1½"	0.00	100.0
3/4"	0.00	100.0
3/8"	1.66	99.8
No. 4	2.25	99.7
No. 10	3.41	99.6
Pan		

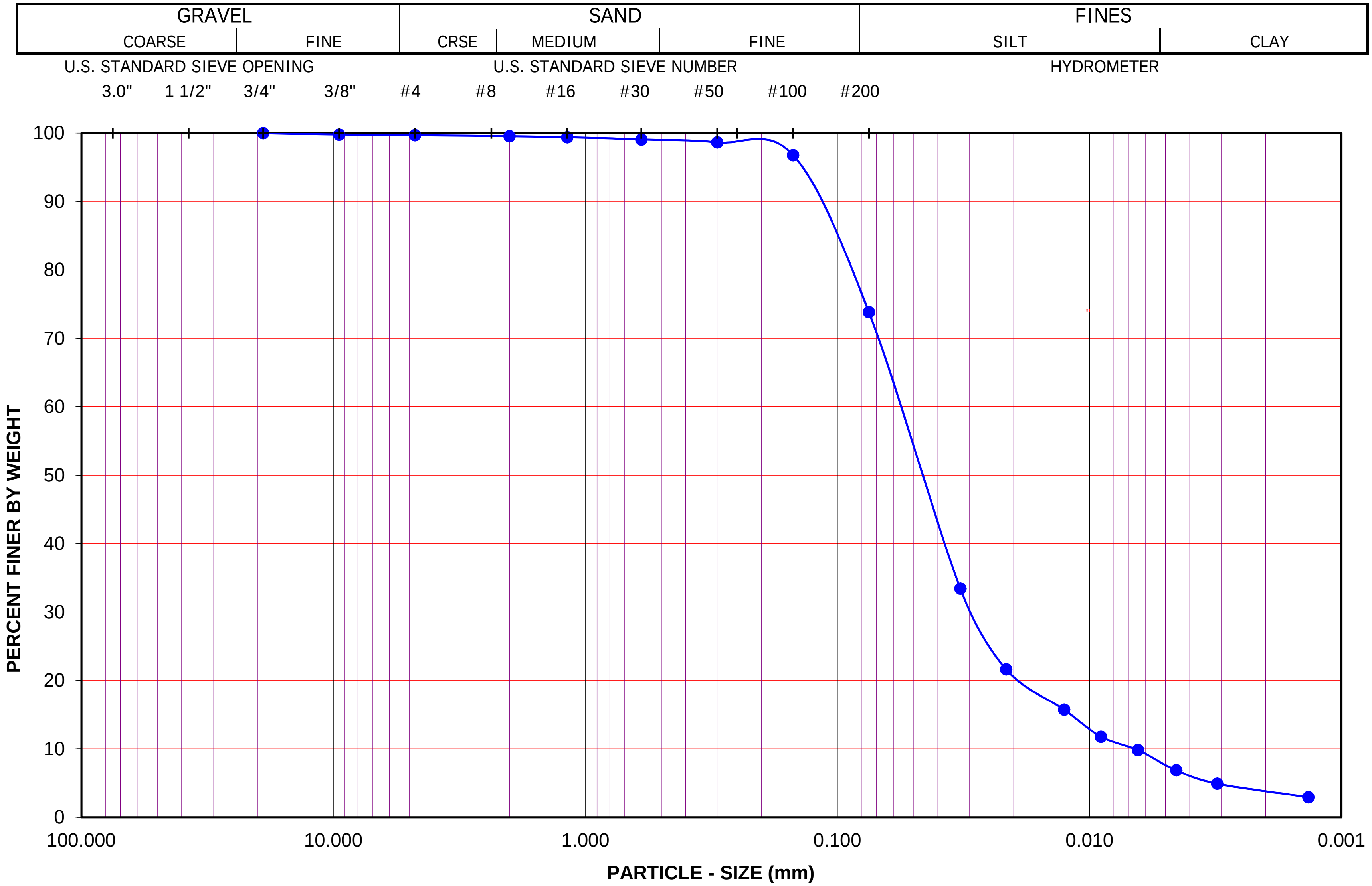
Sieve after Hydrometer & Wet Sieve

U.S. Sieve Size	Cumulative Wt. Of Dry Soil Retained (g)	% Passing	% Total Sample
No. 10	0.00	100.0	99.6
No. 16	0.08	99.8	99.4
No. 30	0.24	99.5	99.1
No. 50	0.46	99.1	98.6
No. 100	1.39	97.2	96.8
No. 200	12.97	74.2	73.8
Pan			

Hydrometer
Wt. of Air-Dry Soil (g) 50.87
Wt. of Dry Soil (g) 50.21

Deflocculant 125 cc of 4% Solution

Date	Time	Elapsed Time (min)	Water Temperature (°C)	Composite Correction 152H	Actual Hydrometer Readings	% Total Sample (%)	Soil Particle Diameter (mm)
19-Mar-18	9:30	0		9.0			
	9:32	2	21.4	9.0	26.0	33.4	0.0325
	9:35	5	21.4	9.0	20.0	21.6	0.0214
	9:45	15	21.4	9.0	17.0	15.7	0.0126
	10:00	30	21.4	9.0	15.0	11.8	0.0090
	10:30	60	21.6	9.0	14.0	9.8	0.0064
	11:30	120	22.0	9.0	12.5	6.9	0.0045
	13:40	250	23.0	9.0	11.5	4.9	0.0031
20-Mar-18	9:30	1440	20.7	9.0	10.5	3.0	0.0014



Project Name: 1201 S. Grand Avenue Geotechnical Investigation

Project No.: 15083A

Boring No.: B-1

Sample No.: 15

Depth (feet): 100-101.5

Soil Type : (ML)s

Soil Identification: Olive brown silt with sand (ML)s

Leighton

PARTICLE - SIZE DISTRIBUTION ASTM D 422

GR:SA:FI : (%) 0 : 26 : 74

Mar-18

PARTICLE-SIZE ANALYSIS OF SOILS

ASTM D 422

Project Name: 1201 S. Grand Avenue Geotechnical Investigation
Tested By: G. Berdy Date: 03/16/18
Project No.: 15083A
Data Input By: J. Ward Date: 03/28/18
Boring No.: B-2
Sample No.: 8b Depth (feet): 40.3-40.8
Soil Identification: Olive brown well-graded sand with silt and gravel (SW-SM)g

		Soil Type
% Gravel	**24**	**(SW-SM)g**
% Sand	**67**	
% Fines	**9**	

			Moisture Content of Total Air-Dry Soil	Moisture Content of Air-Dry Soil Passing #10	After Hydrometer & Wet Sieve ret. in #200 Sieve
Specific Gravity (Assumed)	2.70	Wt.of Air-Dry Soil + Cont.(g)	0.00	79.18	
Correction for Specific Gravity	0.99	Dry Wt. of Soil + Cont. (g)	0.00	79.17	164.97
Wt.of Air-Dry Soil + Cont. (g)	697.45	Wt. of Container No.___ (g)	1.00	57.71	74.27
Wt. of Container	77.77	Moisture Content (%)	0.00	0.05	
Dry Wt. of Soil (g)	619.68	Wt. of Dry Soil (g)			90.70

Coarse Sieve

U.S. Sieve	Cumulative Wt. Of Dry Soil Retained (g)	% Passing
3"	0.00	100.0
1½"	0.00	100.0
3/4"	20.14	96.7
3/8"	63.97	89.7
No. 4	150.75	75.7
No. 10	264.27	57.4
Pan		

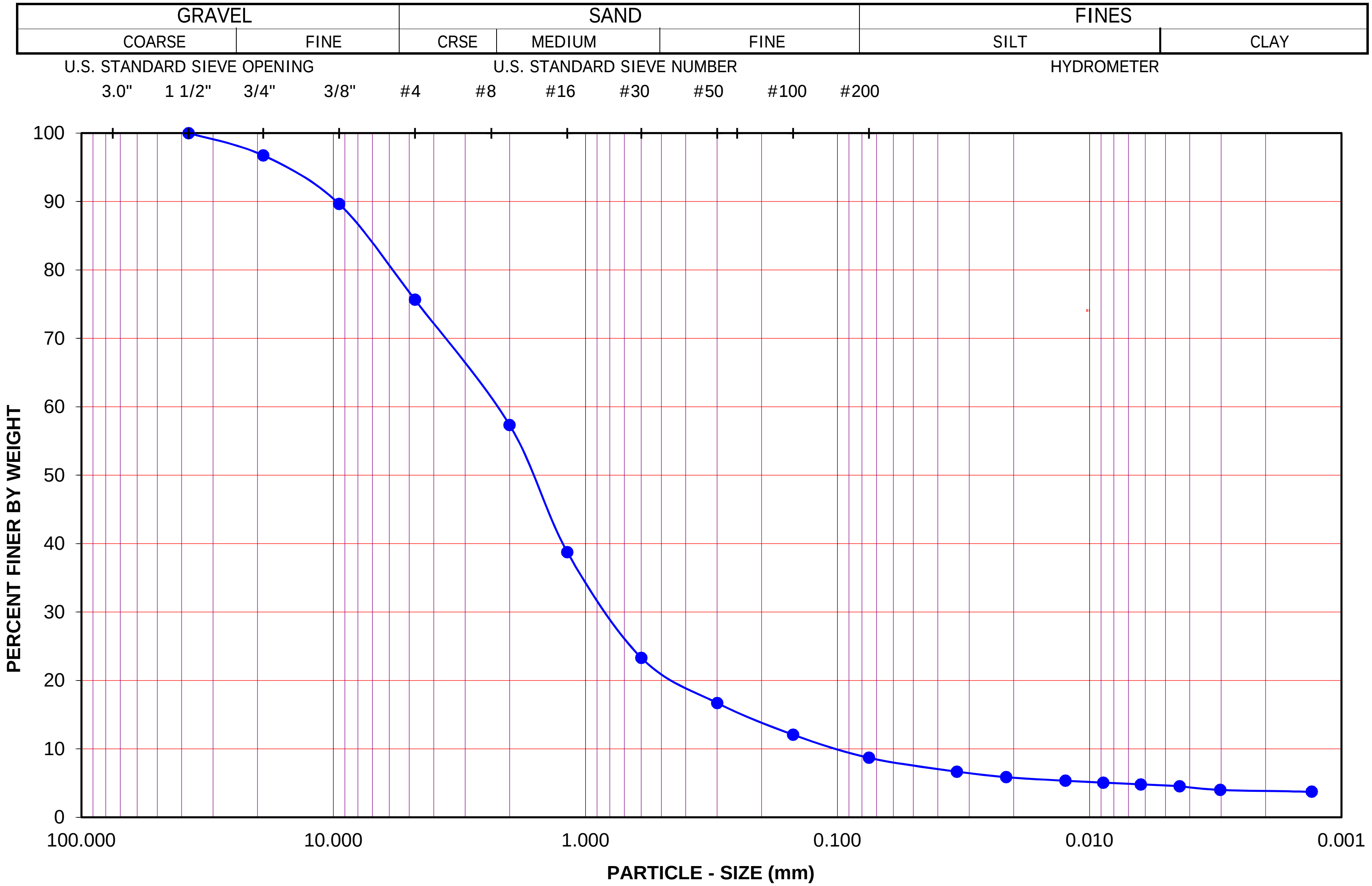
Sieve after Hydrometer & Wet Sieve

U.S. Sieve Size	Cumulative Wt. Of Dry Soil Retained (g)	% Passing	% Total Sample
No. 10	0.00	100.0	57.4
No. 16	34.42	67.6	38.8
No. 30	63.06	40.7	23.3
No. 50	75.27	29.2	16.7
No. 100	83.86	21.1	12.1
No. 200	90.12	15.2	8.7
Pan			

Hydrometer Wt. of Air-Dry Soil (g) 106.31 Wt. of Dry Soil (g) 106.26

Deflocculant 125 cc of 4% Solution

Date	Time	Elapsed Time (min)	Water Temperature (°C)	Composite Correction 152H	Actual Hydrometer Readings	% Total Sample (%)	Soil Particle Diameter (mm)
19-Mar-18	9:34	0		9.0			
	9:36	2	21.3	9.0	21.5	6.7	0.0336
	9:39	5	21.3	9.0	20.0	5.9	0.0214
	9:49	15	21.3	9.0	19.0	5.4	0.0125
	10:04	30	21.3	9.0	18.5	5.1	0.0088
	10:34	60	21.6	9.0	18.0	4.8	0.0063
	11:34	120	22.1	9.0	17.5	4.6	0.0044
	13:44	250	23.0	9.0	16.5	4.0	0.0030
20-Mar-18	9:34	1440	20.9	9.0	16.0	3.7	0.0013



Project Name: 1201 S. Grand Avenue Geotechnical Investigation

Project No.: 15083A

Boring No.: B-2

Sample No.: 8b

Depth (feet): 40.3-40.8

Soil Type : (SW-SM)g

Leighton

PARTICLE - SIZE DISTRIBUTION ASTM D 422

Soil Identification: Olive brown well-graded sand with silt and gravel (SW-SM)g

GR:SA:FI : (%) **24 : 67 : 9**

Mar-18

PARTICLE-SIZE DISTRIBUTION (GRADATION) of SOILS USING SIEVE ANALYSIS

ASTM D 6913

Leighton

Project Name:	1201 S. Grand Avenue Geotechnical Investigation	Tested By:	O. Figueroa	Date:	03/14/18
Project No.:	15083A	Checked By:	J. Ward	Date:	03/28/18
Boring No.:	B-1	Depth (feet):	40-41.5		
Sample No.:	8				
Soil Identification:	Brown well-graded sand with silt and gravel (SW-SM)g				

		Moisture Content of Total Air - Dry Soil	
Container No.:	742	Wt. of Air-Dry Soil + Cont. (g)	0.0
Wt. of Air-Dried Soil + Cont.(g)	644.2	Wt. of Dry Soil + Cont. (g)	0.0
Wt. of Container (g)	79.1	Wt. of Container No.______ (g)	1.0
Dry Wt. of Soil (g)	565.1	Moisture Content (%)	0.0

After Wet Sieve	Container No.	742
	Wt. of Dry Soil + Container (g)	595.0
	Wt. of Container (g)	79.1
	Dry Wt. of Soil Retained on # 200 Sieve (g)	515.9

U. S. Sieve Size (in.)	U. S. Sieve Size (mm.)	Cumulative Weight Dry Soil Retained (g)	Percent Passing (%)
1 1/2"	37.5		
1"	25.0	0.0	100.0
3/4"	19.0	18.3	96.8
1/2"	12.5	31.2	94.5
3/8"	9.5	40.5	92.8
#4	4.75	89.4	84.2
#8	2.36	147.4	73.9
#16	1.18	231.5	59.0
#30	0.600	337.0	40.4
#50	0.300	423.5	25.1
#100	0.150	481.9	14.7
#200	0.075	513.4	9.1
PAN			

GRAVEL: **16 %**

SAND: **75 %**

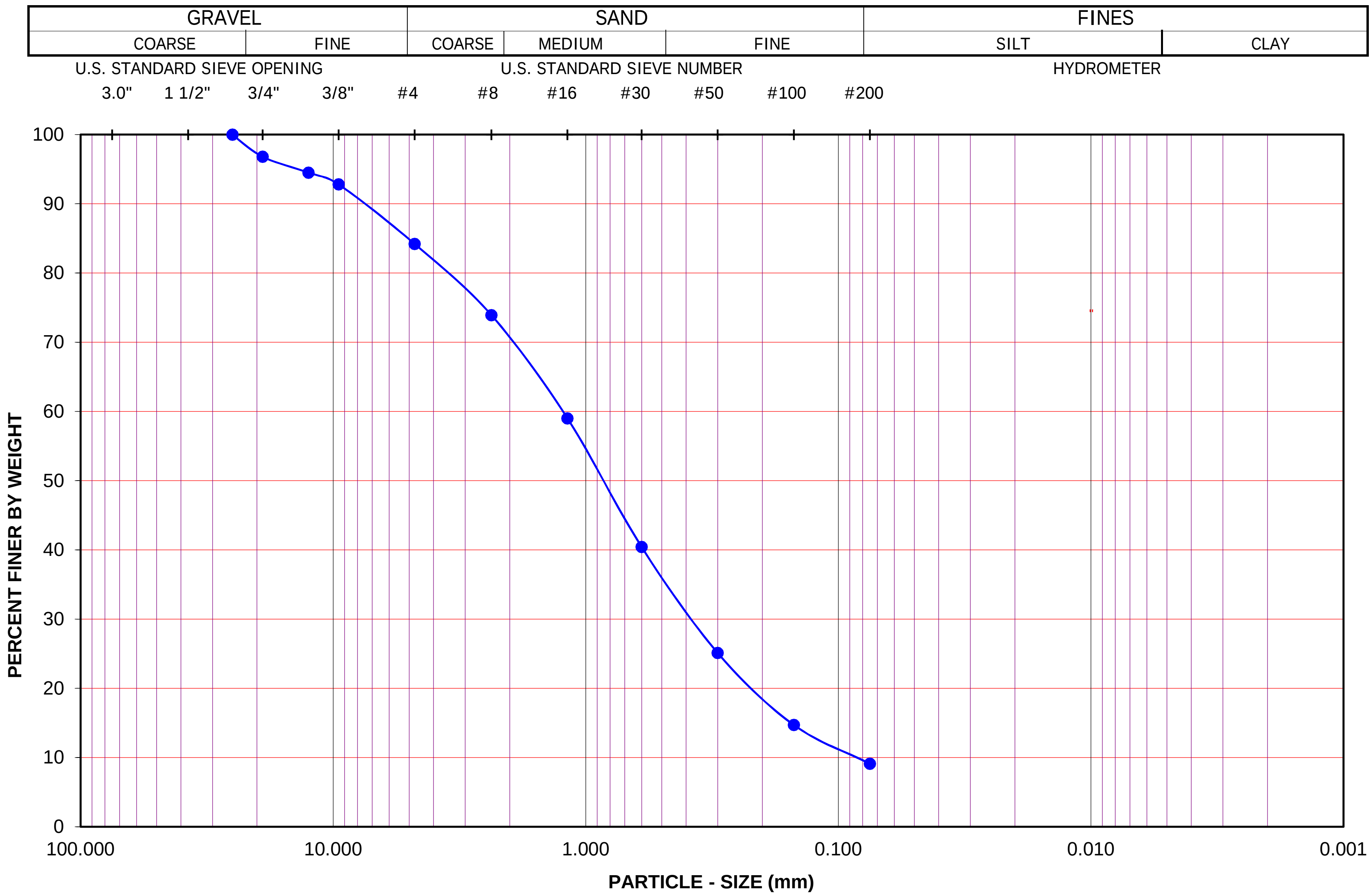
FINES: **9 %**

GROUP SYMBOL: **(SW-SM)g**

Cu = D60/D10 = 15.48

Cc = $(D30)^2/(D60*D10)$ = 1.39

Remarks:



Project Name: 1201 S. Grand Avenue Geotechnical Investigation

Project No.: 15083A

Leighton

PARTICLE - SIZE DISTRIBUTION ASTM D 6913

Boring No.: B-1

Sample No.: 8

Depth (feet): 40-41.5

Soil Type : (SW-SM)g

Soil Identification: Brown well-graded sand with silt and gravel (SW-SM)g

GR:SA:FI : (%) 16 : 75 : 9

Mar-18

PARTICLE-SIZE DISTRIBUTION (GRADATION) of SOILS USING SIEVE ANALYSIS

ASTM D 6913

Leighton

Project Name:	1201 S. Grand Avenue Geotechnical Investigation	Tested By:	G. Bathala	Date:	03/16/18
Project No.:	15083A	Checked By:	J. Ward	Date:	03/28/18
Boring No.:	B-2	Depth (feet):	60-60.5		
Sample No.:	11a				
Soil Identification:	Dark yellowish brown fat clay with sand (CH)s				

		Moisture Content of Total Air - Dry Soil	
Container No.:	DR	Wt. of Air-Dry Soil + Cont. (g)	0.0
Wt. of Air-Dried Soil + Cont.(g)	625.4	Wt. of Dry Soil + Cont. (g)	0.0
Wt. of Container (g)	218.3	Wt. of Container No.______ (g)	1.0
Dry Wt. of Soil (g)	407.1	Moisture Content (%)	0.0

After Wet Sieve	Container No.	DR
	Wt. of Dry Soil + Container (g)	337.8
	Wt. of Container (g)	218.3
	Dry Wt. of Soil Retained on # 200 Sieve (g)	119.5

U. S. Sieve Size (in.)	U. S. Sieve Size (mm.)	Cumulative Weight Dry Soil Retained (g)	Percent Passing (%)
1 1/2"	37.5		
1"	25.0		
3/4"	19.0		
1/2"	12.5		
3/8"	9.5		
#4	4.75	0.0	100.0
#8	2.36	0.9	99.8
#16	1.18	4.2	99.0
#30	0.600	13.5	96.7
#50	0.300	29.2	92.8
#100	0.150	56.8	86.0
#200	0.075	112.4	72.4
PAN			

GRAVEL: **0 %**

SAND: **28 %**

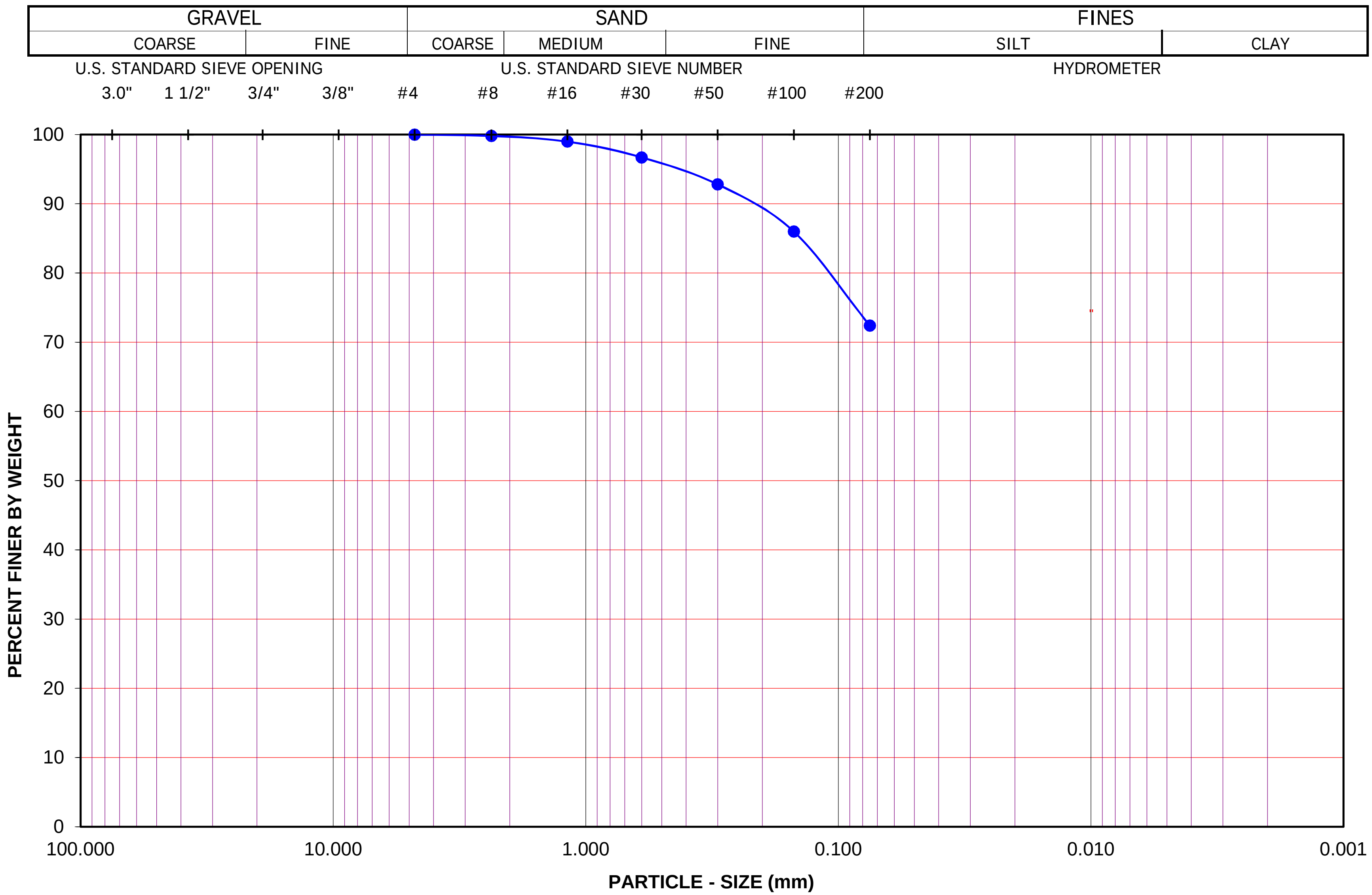
FINES: **72 %**

GROUP SYMBOL: **(CH)s**

Cu = D60/D10 = ________

Cc = $(D30)^2/(D60*D10)$ = ________

Remarks:



Project Name: 1201 S. Grand Avenue Geotechnical Investigation

Project No.: 15083A

Boring No.: B-2

Sample No.: 11a

Depth (feet): 60-60.5

Soil Type : (CH)s

Soil Identification: Dark yellowish brown fat clay with sand (CH)s

Leighton

PARTICLE - SIZE DISTRIBUTION ASTM D 6913

GR:SA:FI : (%) 0 : 28 : 72

Mar-18

PARTICLE-SIZE DISTRIBUTION (GRADATION) of SOILS USING SIEVE ANALYSIS

ASTM D 6913

Leighton

Project Name:	1201 S. Grand Avenue Geotechnical Investigation	Tested By: G. Bathala	Date:	03/16/18
Project No.:	15083A	Checked By: J. Ward	Date:	03/28/18
Boring No.:	B-2	Depth (feet): 60-60.5		
Sample No.:	11a			
Soil Identification:	Dark yellowish brown lean clay with sand (CL)s			

		Moisture Content of Total Air - Dry Soil	
Container No.:	DR	Wt. of Air-Dry Soil + Cont. (g)	0.0
Wt. of Air-Dried Soil + Cont.(g)	625.4	Wt. of Dry Soil + Cont. (g)	0.0
Wt. of Container (g)	218.3	Wt. of Container No.______ (g)	1.0
Dry Wt. of Soil (g)	407.1	Moisture Content (%)	0.0

After Wet Sieve	Container No.	DR
	Wt. of Dry Soil + Container (g)	337.8
	Wt. of Container (g)	218.3
	Dry Wt. of Soil Retained on # 200 Sieve (g)	119.5

U. S. Sieve Size (in.)	U. S. Sieve Size (mm.)	Cumulative Weight Dry Soil Retained (g)	Percent Passing (%)
1 1/2"	37.5		
1"	25.0		
3/4"	19.0		
1/2"	12.5		
3/8"	9.5		
#4	4.75	0.0	100.0
#8	2.36	0.9	99.8
#16	1.18	4.2	99.0
#30	0.600	13.5	96.7
#50	0.300	29.2	92.8
#100	0.150	56.8	86.0
#200	0.075	112.4	72.4
PAN			

GRAVEL: **0 %**

SAND: **28 %**

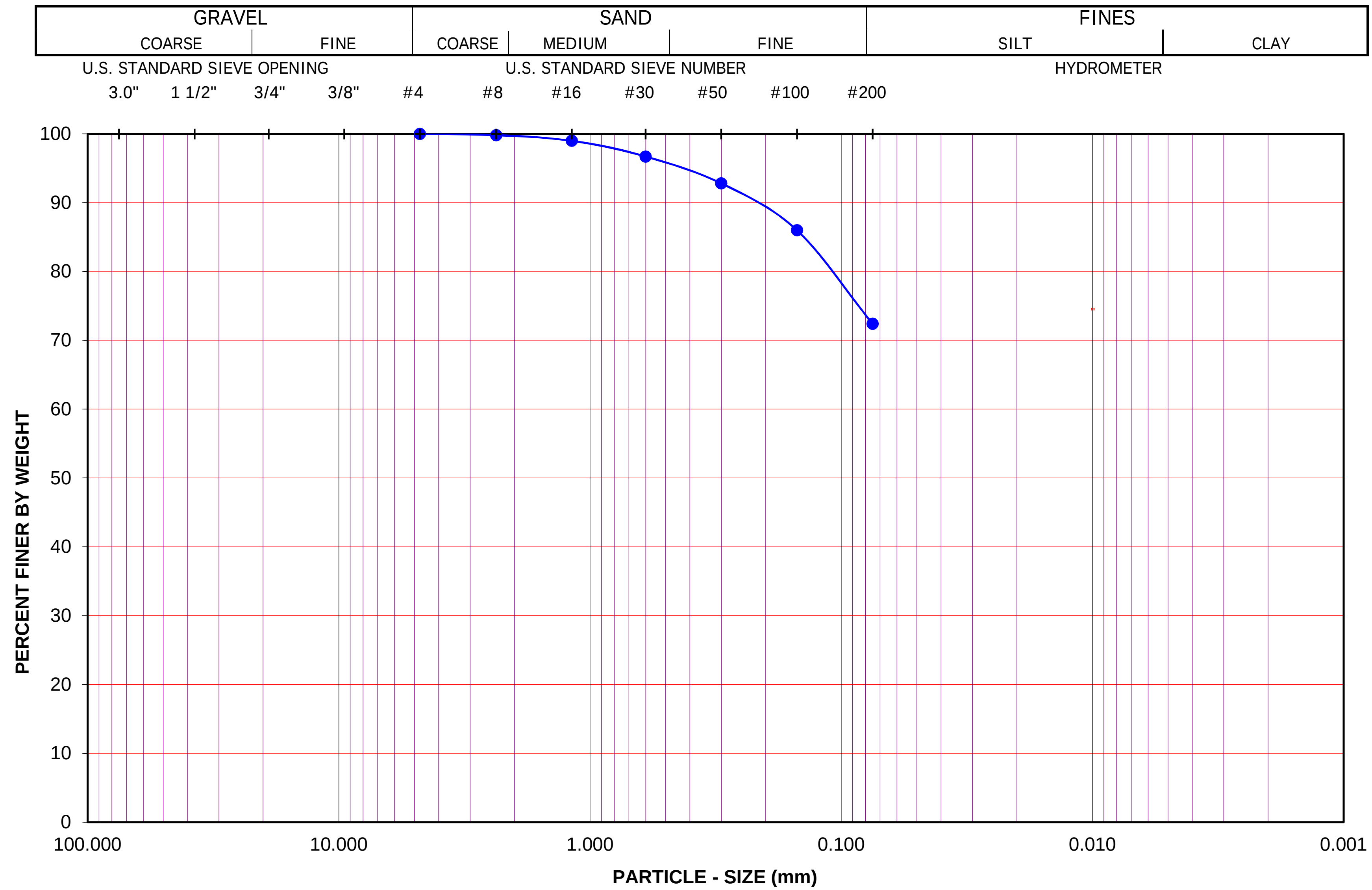
FINES: **72 %**

GROUP SYMBOL: **(CL)s**

Cu = D60/D10 = ______

Cc = $(D30)^2/(D60*D10)$ = ______

Remarks:



Project Name: 1201 S. Grand Avenue Geotechnical Investigation

Project No.: 15083A

Boring No.: B-2

Sample No.: 11a

Depth (feet): 60-60.5

Soil Type : (CL)s

Soil Identification: Dark yellowish brown lean clay with sand (CL)s

Leighton

PARTICLE - SIZE DISTRIBUTION ASTM D 6913

GR:SA:FI : (%) 0 : 28 : 72

Mar-18

PARTICLE-SIZE DISTRIBUTION (GRADATION) of SOILS USING SIEVE ANALYSIS

ASTM D 6913

Leighton

Project Name:	1201 S. Grand Avenue Geotechnical Investigation	Tested By:	R. Manning	Date:	03/16/18
Project No.:	15083A	Checked By:	J. Ward	Date:	03/28/18
Boring No.:	B-2	Depth (feet):	60.5-61		
Sample No.:	11b				
Soil Identification:	Dark yellowish brown sandy lean clay s(CL)				

		Moisture Content of Total Air - Dry Soil	
Container No.:	HA	Wt. of Air-Dry Soil + Cont. (g)	0.0
Wt. of Air-Dried Soil + Cont.(g)	649.2	Wt. of Dry Soil + Cont. (g)	0.0
Wt. of Container (g)	246.3	Wt. of Container No.______ (g)	1.0
Dry Wt. of Soil (g)	402.9	Moisture Content (%)	0.0

After Wet Sieve	Container No.	HA
	Wt. of Dry Soil + Container (g)	438.5
	Wt. of Container (g)	246.3
	Dry Wt. of Soil Retained on # 200 Sieve (g)	192.2

U. S. Sieve Size (in.)	U. S. Sieve Size (mm.)	Cumulative Weight Dry Soil Retained (g)	Percent Passing (%)
1 1/2"	37.5		
1"	25.0		
3/4"	19.0		
1/2"	12.5		
3/8"	9.5	0.0	100.0
#4	4.75	0.2	100.0
#8	2.36	3.0	99.3
#16	1.18	12.8	96.8
#30	0.600	36.1	91.0
#50	0.300	72.5	82.0
#100	0.150	121.3	69.9
#200	0.075	189.1	53.1
PAN			

GRAVEL: **0 %**

SAND: **47 %**

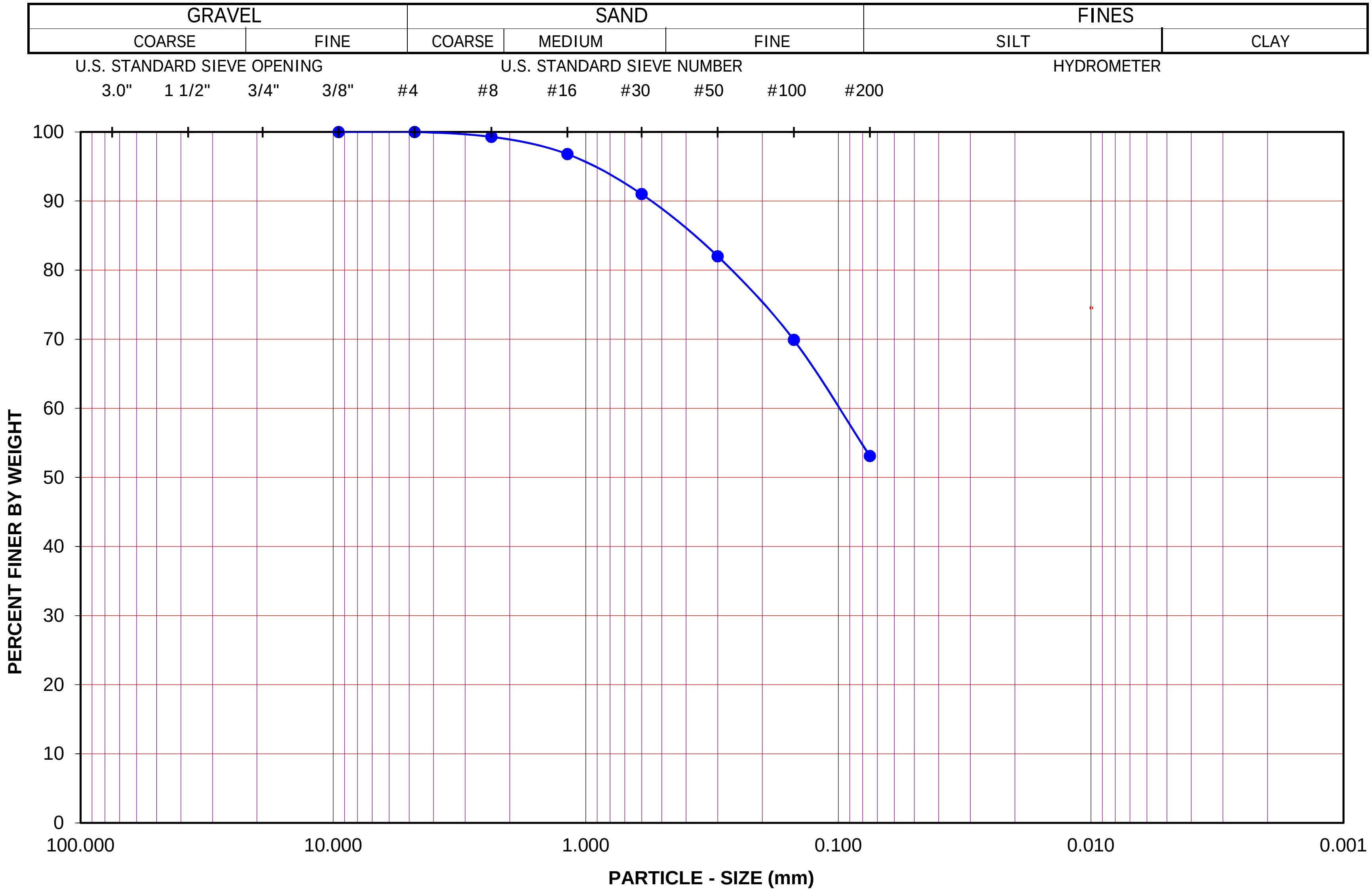
FINES: **53 %**

GROUP SYMBOL: **s(CH)**

Cu = D60/D10 = ______

Cc = $(D30)^2/(D60*D10)$ = ______

Remarks: ______



Project Name: 1201 S. Grand Avenue Geotechnical Investigation

Project No.: 15083A

Boring No.: B-2

Sample No.: 11b

Depth (feet): 60.5-61

Soil Type : s(CH)

Soil Identification: Dark yellowish brown sandy lean clay s(CL)

Leighton

PARTICLE - SIZE DISTRIBUTION ASTM D 6913

GR:SA:FI : (%) 0 : 47 : 53

Mar-18

Leighton

PARTICLE-SIZE DISTRIBUTION (GRADATION) of SOILS USING SIEVE ANALYSIS
ASTM D 6913

Project Name:	1201 S. Grand Avenue Geotechnical Investigation	Tested By:	R. Manning	Date:	03/20/18
Project No.:	15083A	Checked By:	J. Ward	Date:	03/28/18
Boring No.:	B-2	Depth (feet):	70-70.3		
Sample No.:	12a				
Soil Identification:	Yellowish brown sandy silt s(ML)				

		Moisture Content of Total Air - Dry Soil	
Container No.:	P-41	Wt. of Air-Dry Soil + Cont. (g)	0.0
Wt. of Air-Dried Soil + Cont.(g)	632.8	Wt. of Dry Soil + Cont. (g)	0.0
Wt. of Container (g)	93.4	Wt. of Container No.______ (g)	1.0
Dry Wt. of Soil (g)	539.4	Moisture Content (%)	0.0

After Wet Sieve	Container No.	P-41
	Wt. of Dry Soil + Container (g)	265.4
	Wt. of Container (g)	93.4
	Dry Wt. of Soil Retained on # 200 Sieve (g)	172.0

U. S. Sieve Size (in.)	U. S. Sieve Size (mm.)	Cumulative Weight Dry Soil Retained (g)	Percent Passing (%)
1 1/2"	37.5		
1"	25.0		
3/4"	19.0		
1/2"	12.5		
3/8"	9.5	0.0	100.0
#4	4.75	0.7	99.9
#8	2.36	2.4	99.6
#16	1.18	5.3	99.0
#30	0.600	12.9	97.6
#50	0.300	36.9	93.2
#100	0.150	82.8	84.6
#200	0.075	167.9	68.9
PAN			

GRAVEL: **0 %**

SAND: **31 %**

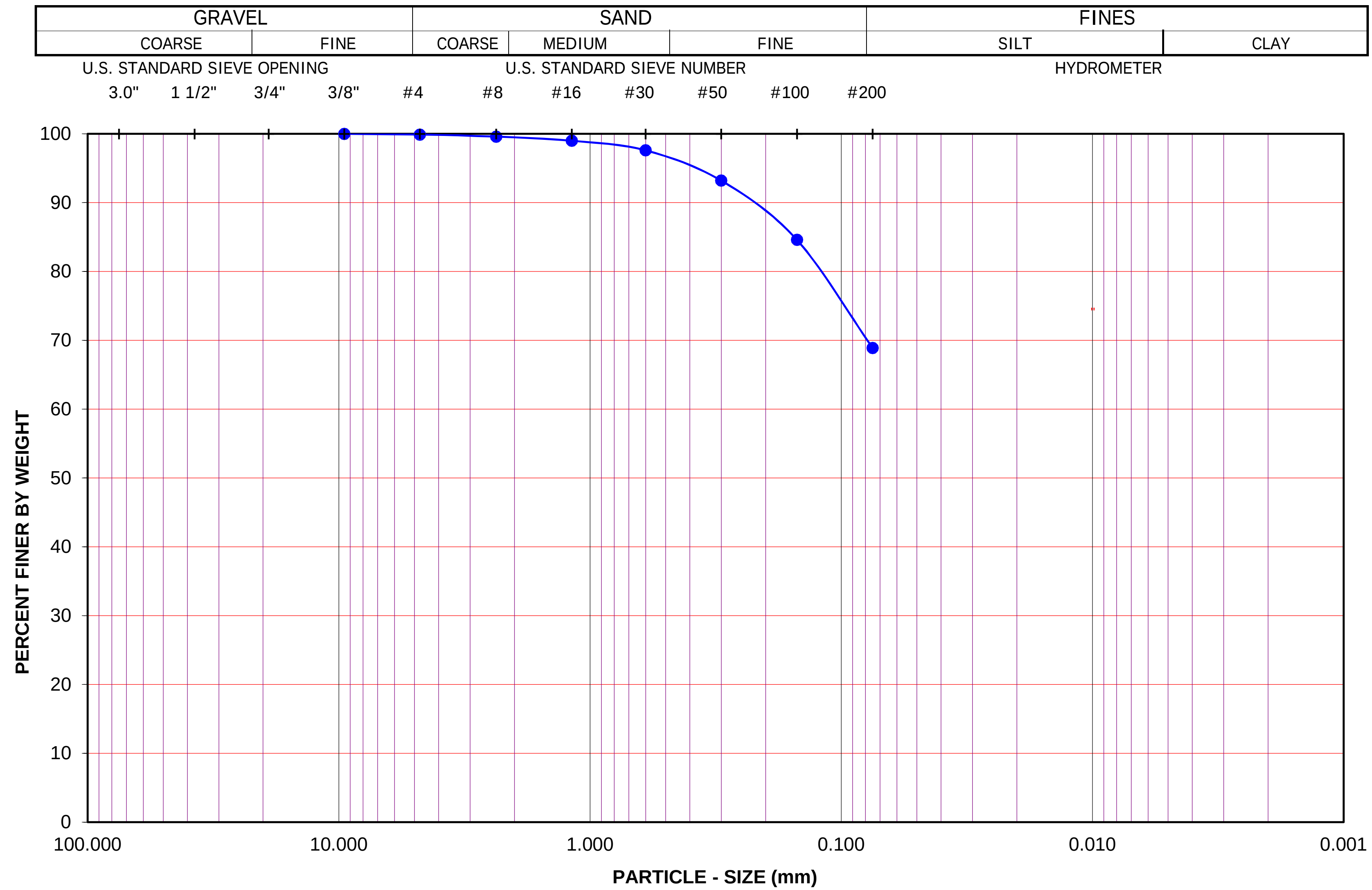
FINES: **69 %**

GROUP SYMBOL: **s(ML)**

Cu = D60/D10 = ______

Cc = (D30)²/(D60*D10) = ______

Remarks:



Project Name: 1201 S. Grand Avenue Geotechnical Investigation

Project No.: 15083A

Leighton

PARTICLE - SIZE DISTRIBUTION ASTM D 6913

Boring No.: B-2

Sample No.: 12a

Depth (feet): 70-70.3

Soil Type : s(ML)

Soil Identification: Yellowish brown sandy silt s(ML)

GR:SA:FI : (%) 0 : 31 : 69

Mar-18

APPENDIX C

GROUND-MOTION EVALUATION

Appendix C - Ground Motion Evaluation

TABLE OF CONTENTS

1. INTRODUCTION

This appendix presents the ground-motion evaluation for the proposed development that includes construction of a high-rise tower to be located on 1201 S. Grand Avenue, Los Angeles, California (located on Figure C-1). The presented results include recommended site-specific response spectra and proposed seed acceleration time histories.

1.1 Project Description

The planned tower will be in a parcel of land bound by a 20-ft wide public alley to the northwest, South Grand Ave to the southeast, West 12th Street to the northeast, and an existing 1-story building at 1225 S. Grand to the southwest (as shown on Figure C-1). The proposed site for the development is currently partially occupied by a surface parking lot towards the southwest side, and partially occupied by an existing warehouse building at 1201 S. Grand Avenue on the northeast side. The existing warehouse building is three stories high and, based on LA City records, was built in 1948; this building will be demolished and replaced by the currently proposed development.

Based on the Entitlement design plan set provided to us by the project's Architects (MVE Partners) dated 05/05/2020 and our discussions, we understand that the development will include a mixed-use tower 40 stories tall, attaining a total height of 462'06'' (428'06" at the roof deck level). We further understand that the residential tower will be surrounded by a podium and parking structure, which will have 2 to 3 subterranean levels and 8 levels of above-ground structures. The bottom of the foundations for both the tower and podium structures is anticipated to be at about 30 to 40 feet below the existing ground surface.

We further understand that the dynamic characteristics of the proposed tower are still under development; we expect the first period to be around 4½ seconds, to be confirmed once the project's Structural Engineer of Record (SEOR) is selected. For presentation purposes, seismic source deaggregation, and acceleration time history record selection, a horizontal period of 4.0 seconds has been selected to represent the key period range of the interest; however, the ground motions are selected such that they present reliable spectral ordinates up to a period of 10 seconds.

1.2 Seismic Design Approach

We understand that the structural design for this structure is being carried out in conformance with the ASCE 7-16 provisions (including Supplement 1 effective December 12, 2018), using the performance-based design procedure as specified by the "An Alternative Procedure for Seismic Analysis and Design of Tall Buildings Located in the Los Angeles Region" document published by Los Angeles Tall Buildings Structural Design Council (LATBSDC), dated June 8, 2017, with the 2018 Supplements (LATBSDS, 2018).

To fulfill the seismic design requirements, the following site-specific response spectra are developed herein:

- A "Maximum Considered Event" uniform hazard spectrum with risk-targeted, maximum-rotated ordinates at 5% damping; also known as a site-specific MCE_R response spectrum (corresponding to a 1% probability of collapse in a 50-year period; i.e., a modified 2,475-year return period spectrum);

- A "Design Basis earthquake (DBE)" uniform hazard spectrum maximum-rotated ordinates at 5% damping; also known as a site-specific DRS response spectrum (corresponding to 2/3 of the site specific MCER response spectrum);
- A "Service-Level Earthquake" uniform hazard spectrum with average horizontal spectral ordinates at 1.74% damping, based on Section 3.4.4 in the 2018 LATBSDC guidelines (corresponding to a 50% probability of exceedance in a 30-year period; i.e., a 43-year return period).

These spectra will be used in the Collapse Prevention and Serviceability Evaluation for the structure under seismic loads. We understand that the Collapse Prevention Evaluation requires the development of earthquake time histories scaled or spectrally matched to the site-specific Maximum Considered Earthquake (MCE_R) response spectrum. To comply with the time history selection and modification requirements of the design standard (Section 16.2.1.1 of ASCE 7-16), eleven (11) pairs of spectrum-compatible time histories are developed. This represents a modification with respect to ASCE 7-10 (the previous standard), which required the development of only 7 (seven) pairs. The selection of the seed time histories and the proposed approach to spectrally modify the acceleration records to be used in the Collapse Prevention analyses are presented in this report. The Serviceability Evaluation will be performed following a spectral analysis approach using the Service Level Earthquake (SLE) spectrum; as such, the development of time histories for the SLE spectrum is not necessary.

We understand that our input on ground-motion aspects of the project is requested to further develop the structural design concepts for the project. We also understand that the results of our work would be reviewed by a Peer Review Panel as a part of the Performance-Based Design approach. As such, both the selected seed acceleration time histories and the approach to their spectral modification should be reviewed and approved by the Peer Review Panel prior to finalizing the project. The final design time histories will be documented in the final report upon receiving review comments by the structural engineering team and the Peer Review Panel.

1.3 Subsurface Conditions

In performing this work, we have reviewed the available subsurface information. This included site investigations completed by GeoPentech in 2015 for a nearby project (1229 S. Grand Avenue), and results of the investigations performed for this site, including two borings, one of which subsequently instrumented as down hole, and two geophysical lines performed in the alley and along the West 12th Street sidewalk.

The available data indicates that the subsurface materials are composed of fill materials overlying stiff/dense alluvial sandy silts, silty clays, and gravelly to clean sands.

Ground motions herein are developed for a hypothetical outcrop (horizon) at the towers' foundation level (see Section 3.2), characterized by a time-averaged shear-wave velocity in the upper 30 m of material below the foundation level (V_{S30}) of 1,700 ft/s (518 m/s).

If the site location or site conditions change appreciably, the ground motions presented herein would need to be re-evaluated.

2. ASCE 7-16 CODE-BASED VALUES

Given the site latitude and longitude (located near 34°2'24.23"N, 118°15'49.51"W) and site shear wave velocity (discussed below), mapped seismic hazard values were queried from the USGS online seismic design map application at https://earthquake.usgs.gov/ws/designmaps/asce7-16.html based on the mapped 2015 National Earthquake Hazards Reduction Program (NEHRP) factors. *These values are superseded in this report by the site-specific values presented in this Appendix but are provided here for completeness*.

The mapped S_S and S_1 for the site are 1.937 g and 0.688 g, respectively. As discussed in more detail in Section 3.2 of this report, the shear wave velocity data recently collected by GeoPentech at the project site indicates a V_{S30} value of 1,700 ft/s (518 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). Using the ASCE 7-16 standard, the mapped design parameters for a Site Class C, ***Risk Category I, II, or III*** structure at this location yield a ***Seismic Design Category D***.

Based on this information, the general procedure ground motion analysis carried out in accordance with Chapter 16A of the 2019 CBC and ASCE 7-16 results in general design spectral acceleration parameters S_{DS} and S_{D1} of 1.550 g and 0.642 g, respectively.

3. SOURCE, SITE AND GROUND-MOTION CHARACTERIZATION

Probabilistic and Deterministic Seismic Hazard Analyses (PSHA and DSHA, respectively) involve the characterization of seismic sources, the attenuation of the seismic energy through the transmission paths, and the local site conditions. Seismic sources pertinent to the seismic hazards of the site are characterized based on geologic information. The effects of transmission paths and local site conditions are incorporated through the use of ground-motion prediction equations – GMPEs (also known as attenuation relationships), which provide the variation in peak horizontal acceleration or spectral acceleration with distance and other predictive parameters for a given local site condition. Key information on seismic sources, site conditions, and attenuation relationships used in this study are summarized below.

3.1 Seismic Sources

The site is located within a seismically active region of southern California, as evidenced by Quaternary faulting and historic earthquakes. The locations of Quaternary-active surface-rupturing faults mapped by the US Geological Survey (USGS, 2010) and instrumentally-recorded earthquakes (Hauksson et al., 2012) relative to the project site are shown on Figure C-2a. The closest Late Quaternary (within the last 15,000 years) surface fault ruptures occurred on the Hollywood Fault (about 9 km north of the site) and the Newport-Inglewood Onshore/Beverly Hills Lineament system (about 9 km west). The 1994 Northridge earthquake and the 1987 Whittier Narrows earthquake were approximately 32 and 16 kilometers north and east of the site, respectively. Based on the PEER (2014) database, the Northridge earthquake generated ground motions on the order of 0.14 g (peak ground acceleration, PGA) and 13 cm/s (peak ground velocity, PGV) at the Pico & Sentous recording station about ¾ km northwest of the site. No recording station data for the 1987 Whittier Narrows earthquake near the subject site are in the PEER (2014) database.

The Seismic Source Characterization (SSC) model used for this project is based on the characterization used by the USGS to develop the 2008 and 2014 versions of National Seismic Hazard Maps (NSHM; Petersen et al., 2008, 2014; and USGS, 2009). The recently completed Uniform California Earthquake Rupture Forecast version 3 (UCERF3) efforts (WGCEP, 2013a,b) updated previous characterizations of several faults in the state and added many new sources. 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. The locations of the seismic sources relative to the project site are shown on the fault map on Figure C-2b. The best-estimate parameters (including maximum magnitude, closest distance, slip rate, and style of faulting) for these seismic sources are summarized in Table C-1. All faults shown on Figure C-2b and listed in Table C-1 were included in the PSHA. In addition to the discrete seismic sources presented on Table C-1, background seismicity that is consistent with the gridded seismicity used in the NSHM calculation was also used in the PSHA.

3.2 Site Seismic Data

The 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 m of the site (V_{S30}) is the primary parameter used to approximate soil non-linearity in the ground-motion models.

At the beginning of March 2018, during a preliminary phase of the same project, GeoPentech performed several field investigations, including drilling two borings (B-1 and B-2), completing one downhole seismic survey in boring B-1 at the location of the proposed tower, and conducting two geophysical lines (18-1 and 18-2) for refraction based on ambient noise (MASW). The approximate locations of the current investigations are shown on Figure C-3a. Past field investigations completed for neighboring project sites are also shown in Figure C-3a. The two borings for this project (B-1 and B-2) were drilled to depths of approximately 150 and 100 feet, respectively, below existing grade. Further details of the field investigation are available in the geotechnical investigation report for this project.

As the proposed development includes 2 to 3 subterranean basement levels, ground motions herein are specified at a hypothetical outcrop (horizon) 35 feet below the ground surface. This hypothetical outcrop is assumed to be representative of the site conditions; as such, the shear-wave velocity measurements between 35- and 135-feet below the ground surface have been used to define the site-specific V_{S30}. Using the shear-wave velocity measurements collected at the site, the estimate of 1,700 ft/s was used as the site V_{S30} over the depth range of interest at a hypothetical outcrop 35 feet below the ground surface. The design velocity profile along with supporting field measurements is shown on Figure C-3b. This value of V_{S30} (1,700 ft/s) corresponds to Site Class C in ASCE 7-16. The site-specific measurements that support this V_{S30} calculation followed the procedures outlined in Chapter 20 of ASCE 7-16. More details on the measurements and calculations are in Appendix A of the Geotechnical Investigation Report for the subject site.

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. The approximate depths to these interfaces were estimated to be 320 meters and 2.6 km, respectively. These estimates were based on the SCEC Community Velocity Model (CVM-S4) by Magistrale et al. (2000 and 2012), are consistent with our understanding of the Los Angeles Basin at the site, and are in general agreement with values previously used for projects in the vicinity of downtown Los Angeles.

3.3 Attenuation Relationships

Seismic shaking is estimated using empirical ground motion attenuation relationships and calculated as the spectral acceleration (SA) for a given period. Calculated values represent the average horizontal component considering 5% damping. All five of the Next Generation Attenuation West 2 (NGA W2) ground motion attenuation models were used in the PSHA: Abrahamson et al., 2014; Boore et al., 2014; Campbell and Bozorgnia, 2014; Chiou and Youngs, 2014; and Idriss (2014). Each of the attenuation relationships was assigned an equal weight of 1/5 to approximately address the “modeling” part of the epistemic uncertainty. Because the site is located on the hanging-wall side of the Puente Hills (both alternatives) and the Compton reverse faults, applicable hanging-wall flags have been implemented when applying the attenuation relationships.

4. PROBABILISTIC SEISMIC HAZARD ANALYSIS

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 Event (i.e., the MCE_R) based on 5% spectral damping. The PSHA evaluation was performed using the version number 43.b of the computer program Hazard (Abrahamson, 2013). The hazard engine and inputs are fully consistent with what implemented for the PSHA analyses of the adjacent project site located at 1229 S. Grand Ave (GeoPentech, Inc, 2015).

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 horizontal peak ground acceleration (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., the hazard at a specified return period) over a range of spectral periods.

Source Contribution at Short Periods: Figure C-4 presents seismic hazard curves for PGA. The total hazard (solid black line) and the contributions of various seismic sources to the total seismic hazard are shown on the figure. Reference lines are provided to mark the 2,475-yr ARP (which represents a 2% probability of exceedance in 50 years) and other key ARPs. The Elysian Park (Upper) Fault controls the PGA hazard for ARPs longer than about 200 years. At the 2,475-yr ARP, the Elysian Park Fault contributes about 34% of the total PGA hazard. The combined Puente Hills sources (i.e., the Puente Hills, Puente Hills–LA, Puente Hills–Santa Fe Springs, and Puente Hills–Coyote Hills faults) are the second-highest contributor at the 2,475-yr ARP, producing about 28% of the total PGA hazard. The combined Compton sources (i.e., both SSC alternatives) contribute 10% of the 2,475-yr PGA hazard. The Raymond Fault contributes about 8% of the ground motions at the same hazard level, and the Santa Monica–Hollywood–Anacapa-Dume fault system and Newport-Inglewood Onshore Fault contribute about 6% and 4%, respectively. Background seismicity produces about 5% of the PGA hazard at the 2,475-yr ARP, and the other sources collectively contribute about 5% of the PGA hazard at the 2,475-yr ARP.

Source Contribution at Long Periods: Figure C-5 presents similar seismic hazard curves for a period of 4.0-seconds. The 4.0-second hazard is dominated by the San Andreas Fault System for ARPs shorter than about 2,000 years. Beyond an ARP of about 2,000 years, the combined Puente Hills sources control the 4.0-second hazard. At the 2,475-yr ARP, the combined Puente Hills sources contribute about 17% of the hazard. The San Andreas Fault System and the Elysian Park Fault each generate about 16% of the ground-motion hazard at the 2,475-yr ARP. Contributions from other faults at the 2,475-yr hazard level are tabulated on Figure C-5. Background seismicity only produces about 1% of the hazard, and the other sources collectively contribute about 11% of the 4.0-second hazard at the 2,475-yr ARP, with the values tabulated on Figure C-5.

Hazard Deaggregation: Figure C-6 presents the deaggregation at average return periods of 43 and 2,475 years for PGA. The PGA deaggregation for the 43-yr ARP (Figure C-6, top) shows the hazard is from M_W 5 to 8 events, distributed across a wide range of distances, and generated by a range of predicted ground motion intensities (i.e., a range of epsilon values). At the 2,475-yr ARP (Figure C-6, bottom), the majority of the hazard is from M_W 5.5 to 7.5 events within 20 km of the project site. This hazard is dominated by characteristic M_W 6 to 7 earthquakes on several faults within 5 to 10 km of the site, producing median and above median (epsilons between 0 and 2+) ground motions. The characteristic event on the Puente Hills–LA Fault (M_W 6.8±0.2) produces median to 99th percentile ground motions about 5 km away from the site.

Figure C-7 presents the deaggregation at average return periods of 43 and 2,475 years for the 4.0-second period. At the 43-yr ARP (Figure C-7, top), the hazard distribution is bimodal: M_W 6 to 7.5 earthquakes on faults within 5 to 15 km of the site contribute significantly to the hazard, as do M_W 6.5 to 8.5 events on faults 30 to 75 km from the site. Most of the shaking is from 5th to 84th percentile ground shaking (epsilons between -2 and 1), although there is some contribution from distant 84th

percentile motions. In fact, the effect of distant, large-magnitude capable, high-slip rate sources such as the San Andreas (57 km away) and the San Jacinto (71 km away) systems is visible in the spike related to the M-R bins with magnitude between 7 and 8.0 located 50 to 75 km away from the site The deaggregation for the 2,475-yr ARP at the 4.0-second period (Figure C-7, bottom) is generally similar to the 43-yr ARP, but with higher epsilon ground motions and more relative contributions from the closer fault sources. The high-epsilon ground motions (about 50t^{h} to 94th percentile) from characteristic (M_W 8.2±0.2) earthquakes on the San Andreas Fault are responsible for the large contributions within the 75 to 100 km distance bin. As discussed in more detail in Section 7.1 below, the deaggregations for the 4.0-second spectral period for the 2,475-yr ARP were used as the basis for the selection of representative seed earthquake acceleration time histories.

Probabilistic-Based Response 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 ranging from 43-yr ARP to 10,000-yr ARP on Figure C-8. The 2,475-yr ordinates at 5% damping are also tabulated on Table C-2 in Column 3.

The probabilistic MCE_R spectrum, which represents the maximum-rotated, risk-targeted ordinates per ASCE 7-16, is shown on Figure C-9a. The ordinates are tabulated on Table C-2 in Column 6. 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 collapse in 50 years. The adjustment between average-horizontal and maximum-rotated components is based on the period-specific ratios in Shahi and Baker (2014a). 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 0.902 is specified for periods 0.2-second and shorter and a scale factor of 0.900 is used for periods of 1.0-second and longer; scale factors for periods between 0.2- and 1.0-second are linearly interpolated. The incorporation of both of these scale factors is reflected in the modified probabilistic MCE_R spectrum on Figure C-9a, and the process of developing the probabilistic MCE_R spectral ordinates is shown on Table C-2 in Columns 3 through 6.

The SLE response spectrum, which represents the 43-year ARP uniform hazard spectrum, is also shown on Figure C-9a. The SLE response spectrum represents a 50% probability of exceedance in 30 years with 1.74% damping. Details bearing on the development of the site-specific SLE spectrum are discussed in Section 6.1 below.

5. DETERMINISTIC SEISMIC HAZARD ANALYSIS

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 S_S 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 F_a 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. The table below lists the key contributors to the DSHA ground motions, as well as the fault parameters used in the analysis.

Seismic Source	Moment Magnitude (M_W)	Closest Distance to Site (km)	Style of Faulting
Puente Hills (LA) Fault	6.8	4	Reverse
Puente Hills (Alt.1) Fault	7.0	5	Reverse
Elysian Park Fault	6.5	5	Reverse
Newport-Inglewood Onshore System	7.2	10	Strike-Slip
Compton Fault	7.3	14	Reverse
San Andreas System	8.2	57	Strike-Slip

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 C-9b. 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 above (i.e., for the probabilistic MCE_R development).

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 C-9b, the Puente Hills (LA) case controls the deterministic MCE_R spectrum between PGA and about 2.0-seconds, the Puente Hills (Alt. 1) case controls at about 3.0 and 4.0 seconds, and the Newport Inglewood Connected case controls at the longer spectral periods. The code-based deterministic minimum attains smaller spectral amplitudes as compared to the 84^{th} percentile of DSHA scenarios throughout the period range.

The deterministic MCE_R spectral ordinates are tabulated in Table C-2 in Column 10, and the process of developing the deterministic MCE_R spectral ordinates is shown on Table 2 in Columns 7 through 10.

6. SITE-SPECIFIC RESPONSE SPECTRA

In accordance with the 2018 LATBDC, site-specific response spectra have been developed for two hazard levels: Service-Level Earthquake (SLE) spectrum, which is represented by a 1.74% damped Uniform Hazard Spectrum reflecting ground motions with a 50% probability of exceedance in 30 years (43 year return period); and the MCE_R response spectrum, developed in accordance with the requirements of Section 21.2 of ASCE 7-16. The site-specific Design Basis earthquake, also known as the Design Response Spectrum (DRS), is also provided for the design of non-structural components; this has been developed in accordance with the requirements of Section 21.3 of ASCE 7-16. The development of these spectra is discussed below.

6.1 Site-Specific SLE Response Spectrum

The SLE response spectrum, which represents the 43-year ARP uniform hazard spectrum, is shown on Figure C-9a. The SLE response spectrum represents a 50% probability of exceedance in 30 years with 1.74% damping. The target equivalent viscous damping β of 1.74% is obtained from Equation 1 in Section 3.4.4 of the 2018 LATBSDC provision accounting for the height of the building from grade to top of the roof.

The 43-year ARP uniform hazard spectrum ordinates were converted from 5% damping (as predicted by the GMPEs in the hazard calculation) to 1.74% 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 the 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 shown on Figure C-9a and tabulated in Table C-3, column 9. The process of developing the SLE ordinates is shown in Table C-3, columns 7 through 9.

6.2 Site-Specific MCE_R Spectrum

Figure C-10a shows the final development of the site-specific MCE_R. As shown on Figure C-10a, the deterministic MCE_R spectral ordinates exceed the probabilistic MCE_R ones across the full range of periods, except at spectral periods above 7.5 seconds where the deterministic and probabilistic MCE_R spectra attain very similar spectral amplitude. The recommended MCE_R is based on the lesser of the deterministic MCE_R and the probabilistic MCE_R response spectra, which are defined as the 5% damped acceleration response spectrum. As a check, the MCE_R is constrained to be no less than the 80% of code-based (ASCE 7-16, Ch. 11) risk-targeted, maximum considered earthquake general ground-motion spectrum.

The 80% of code-based general spectrum controls the site-specific MCE_R at periods longer than 4.0 seconds, in addition to a narrow interval at short periods around 0.05 seconds; the probabilistic MCE_R controls the site-specific MCE_R at the remaining spectral periods.

The final recommended site-specific MCE_R spectral ordinates are tabulated in Table C-2 in Column 12, and the process of developing the site-specific MCE_R spectral ordinates is shown on Table C-2, Columns 6, 10, 11 and 12.

6.3 Directionality in the MCE_R Response Spectrum

Based on the updated definitions in Chapter 11 of ASCE 7-16, 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 $M_W \geq 7$. The project site falls into this category, therefore the emphasis on capturing the potential effects has been in the time history development stage. As noted below, this remains an important consideration and directivity effects are captured in the time history development by selecting an appropriate number of seeds that have pulse-like characteristics, and matching them in a way that preserves the naturally occurring polarized response. For the purpose of developing time history matching targets, the MCE_R response spectrum is provided for both the maximum orientation direction and for the average orientation direction. The final site-specific MCE_R spectrum, as computed, represents the maximum rotated component of ground motion (i.e. RotD100), which is assumed to be equivalent to the FN component. The average orientation component (RotD50) is obtained by "un-rotating" the maximum components using the same Shahi and Baker (2014a) rotation factors discussed before, and represents the average horizontal (AH) component of ground motion.

The final site-specific MCE_R spectra are shown for the RotD100 and RotD50 components on Figure C-10b and tabulated in Table C-2, Columns 12 and 13.

6.4 Site-Specific Design Response Spectrum

The Design Response Spectrum (DRS) was then developed as 2/3 of the site-specific MCER (but no less than the code-based minimum, which is defined as 80% of the code-based spectrum using ASCE 7-16, Sections 11.4.5 and 11.5.6). The DRS development is shown on Figure C-11, where the code-based minimum defines the DRS between about 0.03 and 0.075 seconds and at the spectral periods longer than about 3.0-seconds. At other spectral periods, the DRS is defined by 2/3 of the site-specific MCE_R. The final recommended DRS is shown highlighted on Figure C-11, and the spectral ordinates are tabulated in Table C-3, Column 6. The process of developing the DRS ordinates is shown in Table C-3 in Columns 3 through 6.

Using ASCE 7-16, Section 21.4, the site-specific seismic design parameters are defined as follows:

- S_{DS} = 1.603g, based on 90% of the spectral acceleration at a period of 0.2 seconds;
- S_{D1} = 0.654, based on the spectral acceleration at a period of 1.0 second;

- S_{MS} = 2.405g, based on 1.5 times S_{DS};
- S_{M1} = 0.981g, based on 1.5 times S_{D1}.

To apply these ground motions in the nonlinear response history analysis, seed time histories have been selected (discussed below) and will be spectrally modified to be consistent with the MCE_R response spectrum demands.

7. ACCELERATION TIME HISTORY ANALYSIS

7.1 Seed Time History Selection

The selection of seed time histories for the nonlinear response analysis was carried out to identify records which have similar magnitude and closest distance to the events that control the hazard in the period range of interest at the MCE_R hazard level. The time history screening procedure implemented in this project adheres to the requirements in Section 16.2.2 of ASCE 7-16 with LATBSDC's exceptions. A total of eleven sets of two-component time histories are selected based on the deaggregation of the 4.0-second hazard at the 2,475-yr ARP (Figure C-7). Although Figure C-7 shows that the hazard has contribution from distant large magnitude events, the results of the individual sources contribution to the 4.0-second hazard at the 2,475-yr ARP in Figure C-7 suggest that the local events, representing the near-field earthquakes, cumulatively contribute significantly to the total hazard. Accordingly, seven local records with magnitude above 6.0 and closest distances between 0 and 20 km have been selected, and four distant records with magnitudes greater than 7.0 and closest distances between 30 and 80 km have been selected.

Screening for Appropriate Magnitude and Distance: The records initially considered for time history analysis included all 31,336 records in the PEER NGA-West2 Ground Motion Database (Ancheta et al., 2014). This database contains records from 599 shallow crustal earthquakes with magnitudes ranging from 3.0 to 7.9 and closest distances ranging from 0.05 to 1,533 km. Figure C-12 shows the PEER Ground Motion Records for which the Joyner-Boore Distance metric is available, plotted by this distance and the magnitude (with the additional constraint of M_W >5). Because both local moderate magnitude events and distant large magnitude events contribute to the hazard, local records with magnitudes between 6.0 and 7.5 with closest distances within 20 km of the site and distant records with magnitudes greater than 7.0 and closest distances between 30 and 80 km have been screened in for consideration. This magnitude-distance screening identified a subset of 452 records from 56 earthquakes for the local event and 326 records from 19 earthquakes for the distant event. To select recordings from sites with reasonably similar local site conditions, recordings from hard rock (Site Class A) and soft soil (Site Class E and F) sites were eliminated from consideration. Recordings from other events were also eliminated based on the following conditions: (1) presence of only one horizontal component recording; (2) time series not presently available for download at the PEER online database (PEER, 2014); (3) incompatible style-of-faulting with respect to the main contributors to the hazard (i.e. normal faulting events were not considered); (4) potential misclassification of intraslab earthquakes within subduction zones as shallow crustal seismicity events; (5) aftershocks after main seismic sequence.

Screening for Longest Usable Period and PGA Scaling: The local and distant subsets identified on Figure C-12 were then further reduced to identify the most appropriate records for spectral matching. Although the fundamental period of the tower has not yet been finalized, based on the structural engineer's initial guidance, it can currently be estimated to be around 4.5 seconds. Accordingly, because spectral matching is performed over the period range of 0.2 to 2 times the fundamental period of the structure (ASCE 7-16, Section 16.2.3.1), records with a longest usable period shorter than 10.0-seconds were eliminated from consideration. Finally, records requiring a scale factor to match the target PGA greater than approximately 5 for the local events and approximately 10 for the distant events were also screened out, so as to avoid excessive scaling-up of the ground motions. Note that as an exception, some of the Denali records have distant scale factors greater than 10. These specific records were retained because of their ample low frequency content, which is desirable for spectrally matching to long periods. This screening process is illustrated on Figures C-13a and C-13b for the local and distant events, respectively. At the end of this initial screening process, a total of 77 candidate seed time histories from 8 earthquakes remain for the distant event and of 95 candidate seed time histories from 18 earthquakes remain for the local event.

Design Local Event - Screening for Peak Ground Velocity (PGV): To further refine the selection for the local events, the next screening was developed to identify records with a Peak Ground Velocity (PGV) similar to the PGV for the controlling event at the site. PGV is used as a metric to try to capture records that have appropriate velocity pulses due to the proximity of the site to the local sources. Because of the correlation between PGA and PGV, a modified PGV for each record was determined after scaling the record to the target MCE_R PGA. The scaled PGVs were then compared to one of the design local events (for the PGV application, a magnitude 7.0 at a distance of 5 km). Using the NGA-West2 ground-motion models, the PGV for the design local event would be expected to have a 50^{th} percentile value of about 56 cm/s, an 84^{th} percentile of about 102 cm/s, and a 95^{th} percentile of about 186 cm/s. Based on the epsilon range for the design event (i.e., the deaggregation plots) and observed distribution of the data within the PEER database, the PGVs of the most desirable records for analysis would fall between the 84th and 95th percentile PGV.

Design Local Event - Screening for Goodness of Fit (GOF): To improve the selection for the local events, an additional screening was implemented aimed to identify records with spectral shapes similar to the target spectrum. To address spectral shape, a goodness-of-fit (GOF) was calculated between the target spectrum and the geometric mean of the horizontal components of the as-recorded seed time histories, scaled to the target PGA. The GOF was calculated as the Sum of the Square of Errors (SSE) in natural log units between the scaled seed time history and the target response spectrum. Thus, the records with the smallest GOF have as-recorded shapes closest to the target spectrum.

Design Local Event - Screening for Significant Duration (SD): To complete the selection for the local events, the final screening attempted to select records with significant durations (SD) similar to the controlling local event at the site. Selecting records with an appropriate duration is believed to be an important consideration, as significant duration may have an effect on the median predicted response in nonlinear dynamic response analysis. To address significant duration, the time in seconds between the 5th and 95th percentile Arias Intensity ($D_{a5\text{-}95}$) was estimated using the recently published Afshari and Stewart (2016) procedure. Using this model, the $D_{a5\text{-}95}$ for the design local event (M_W 7.0 at 5.0 km) with unspecified style-of-faulting would have a 50th percentile significant duration of about 10 seconds, a 16th percentile of about 7 seconds, and an 84th percentile of about 15 seconds. Recordings with durations between the 16th and 84th percentiles were given preference to avoid heavily scaling-up low intensity, long duration records to the target, which would produce an unrealistically long duration of high intensity shaking. It is worth noticing that the other widely used $D_{a5\text{-}95}$ model by Kempton and Stewart (2006) would have yield to somewhat higher durations for the same target event, ranging between 9 second to 24 seconds for the 16th to 85th percentiles. Given the large spread between the two $D_{a5\text{-}95}$ models, the record selection tried to encompass a wide range of durations within these boundaries (i.e. 7 to 24 seconds).

Design Local Event – Record Selection: The PGV, GOF and SD values were jointly considered for the final screening step applied to the design local event are shown on Figure C-14a and C-14b, grouped by earthquake, with the design local event PGV and SD percentiles shown. From these plots, seven local events that fall within the target PGV and SD ranges and have low GOFs were selected. Care was taken to select only one set of recordings from a given earthquake to broaden the characteristics of the recorded events in the analysis. The selected records and their key characteristics are listed in Table C-4.

As shown in Table C-4, the as-recorded orientation for one out of the seven ground motions for the design local event already corresponds to the FN and FP direction (± 10 degrees) with respect to the causative fault. Additionally, five ground motions are classified as pulse-like time histories, with pulse period ranging between about 1.4 to 6.3 seconds. The relative occurrence of the pulse-like motions as compared to the total amount of selected recordings for the design local event is informed by the pulse probability model by Shahi and Baker (2014b) applied to the hazard significant sources within 15 km from the project site, assuming hypocenter position for these calculations taken so as to maximize forward directivity effects.

Design Distant Event – Final Screening and Record Selection: To further refine the selection for the distant events, the final screening attempted to select records with significant durations (SD) similar to the controlling distant event at the site. Using the Afshari and Stewart (2016) model, the $D_{a5\text{-}95}$ for the design distant event (M_W 7.9 at 57 km) would have a 50th percentile significant duration of about 57 seconds, a 16th percentile of about 38 seconds, and a 5th percentile of 26 seconds. Recordings with durations between the 5th and 50th percentiles were given preference to avoid heavily scaling-up low intensity, long duration records to the target, which would produce an unrealistically long duration of high intensity shaking. It is worth noticing that, for the same design event M-R combination, the Kempton and Stewart (2006) duration model widely used for other similar projects in downtown LA would have yielded lower $D_{a5\text{-}95}$ estimates, ranging from about 24.5 seconds for the 16th percentile to about 39 seconds for the 50th percentile. Finally, the GOF (described above) was also used for the distant event screening to identify records having spectral shapes closest to the target spectrum after PGA scaling but prior to additional spectral modification.

The SD and GOF values considered for the final screening step applied to the design distant event are shown on Figure C-15, grouped by earthquake, with the design distant event SD percentiles according to Afshari and Stewart (2016) shown. From this plot, four distant events were selected. As with the selection for the design local event seeds, care was taken to pick only one set of recordings from a given earthquake to broaden the characteristics of the recorded events in the analysis. The selected records and their key characteristics are also listed in Table C-4.

The response spectra for the two horizontal components for the selected seven seeds representing the design local event are shown on Figure C-16a. The same spectra scaled to match the target MCE_R PGA are also plotted against the target response spectra on Figure C-16a. The response spectra for the two horizontal components for the selected four seeds representing the design local event are shown on Figure C-16b. The same spectra scaled to match the target MCE_R PGA are also plotted against the target response spectra on Figure C-16b.

The acceleration, velocity, and displacement plots for each set of two components are shown for visual inspection on Figures C-17 through C-27, respectively.

7.2 Proposed Spectral Modification of Time Histories

Spectral modification of the eleven (11) selected time history pairs will be performed after the Structural Engineer of Record and Peer Review Panel review the selected time histories described herein. This report will then be updated to include horizontal time histories spectrally modified so that the RotD100 spectrum resulting from the two modified horizontal component of ground motion is consistent with the target MCE_R.

The scaling and spectral modification will be completed to obtain reasonable agreement between the RotD100 spectra of the recorded time history and the appropriate MCE_R spectrum within a period range defined by ASCE 7-16, Section 16.2.3.1, as 0.2 to 2 times the fundamental period of the structure (with the understanding that the lower bound may be modified so to capture the periods needed for 90% mass participation in both directions of the building).

The proposed spectral modification approach is a simplified version of the Hybrid approach (CTBUH - Golesorkhi et al., 2017) stemming from the work of Mazzoni et al. (2012). The suggested method utilizes a clever procedure to scale and to apply small modification to the as-recorded spectral intensities of the seed time histories, while displaying a key advantage that the suite of final ground motions is likely to preserve the original non-stationary characteristics (of particular importance for the polarized motions with clear pulses in the velocity trace), and tends to maintain the natural inter-period correlation (which would be completely altered following a tight spectral matching procedure where spectral "peaks" and "troughs" are smoothly filled up). Additionally, such procedure retains a level of variability among the modified time histories, without the excessive scaling often observed when uniform scaling is applied as the sole modification approach. After computing record-specific, component-specific new targets through the hybrid procedure described above, each recording from the eleven pairs of time histories was matched to their respective target spectra using the program RSPMATCH (Abrahamson, 1992) as improved in 2010 (Al-Atik and Abrahamson, 2010). The program iteratively arrives at a time history with a tight spectral match by adding tapered cosine wavelets to the seed time history in the time domain.

After spectral modification, the average of the resulting RotD100 spectra of the two horizontal components will be checked to confirm that the intensity at the conditioned periods exceeds the MCE_R spectral values at those periods, as required by Chapter 16 of ASCE 7-16.

8. LIMITATIONS

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 exploration.

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.

9. REFERENCES

Abrahamson, N.A. (2018). RspMatch updates study, personal communication.

Abrahamson, N.A. (2013), personal communication. Haz43b FORTRAN Executable.

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.

Afshari K., Stewart J.P. (2016). Physically parameterized prediction equations for significant duration in active crustal regions, Earthquake Spectra, vol. 32, pp. 2057–2081

Al-Atik, L., and Abrahamson, N.A. (2010). An Improved Method for Nonstationary Spectral Matching, Earthquake Spectra, vol. 26 No. 3, pp. 601–617.

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

American Society of Civil Engineers (ASCE) (2018). Supplement 1 to the ASCE Standard ASCE/SEI 7-16, Minimum Design Loads for Buildings and Other Structures, effective December 12, 2018, 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.

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 2015 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.

Golesorkhi, R., Joseph, L., Klemencic, R., Shook, D. & Vilse, J. (2017). Performance-Based Seismic Design for Tall Buildings: An output of the CTBUH Performance-Based Seismic Design Working Group. Chicago, Council on Tall Building and Urban Habitat.

GeoPentech (2015). 1233 South Grand Avenue Geotechnical Report, prepared for City Century, 23 pp plus Figures, Tables, and Appendices, dated December 10, 2015

Hauksson, E., Yang, W., and Shearer, P.M. (2012). 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.

Kempton, J.J., and Stewart, J.P. (2006). Prediction equations for significant duration of earthquake ground motions considering site and near-source effects, Earthquake Spectra, vol. 22 No. 4, pp. 985-1013.

Los Angeles Tall Buildings Structural Design Council – LATBSDC (2018). An Alternative Procedure for Seismic Analysis and Design of Tall Buildings Located in the Los Angeles Region – 2017 Edition with 2018 Supplements, dated March 20, 2018.

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.

Mazzoni, S., Hachem, M., and Sinclair (2012). An Improved Approach for Ground Motion Suite Selection and Modification for Use in Response History Analysis, 15th World Conference on Earthquake Engineering, Lisboa, 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.

Rezaeian, S., Y. Bozorgnia, I.M. Idriss, K. Campbell, N. Abrahamson, and W. Silva (2012). Spectral Damping Scaling Factors for Shallow Crustal Earthquakes in Active Tectonic Regions. PEER Report 2012/01. Pacific Earthquake Engineering Research Center, California, USA

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

Shahi, S.K. and Baker, J.W. (2014b). An Efficient Algorithm to Identify Strong-Velocity Pulses in Multicomponent Ground Motions, Bulletin of the Seismological Society of America, Vol. 104, No. 5, pp. 2456–2466, October 2014, doi: 10.1785/0120130191

Southern California Earthquake Center (SCEC) (2014). Significant Earthquakes and Faults: Historical Earthquakes & Significant Faults in Southern CA, available at [http://www.data.scec.org/significant/index.html] accessed August 2014.

United States Geological Survey (USGS) (2010). Quaternary fault and fold database for the United States, updated November 2010, available at [http//earthquakes.usgs.gov/regional/qfaults/].

USGS (2009). Harmsen, S., personal communication, October 2009.

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.

WGCEP (2008). The Uniform California Earthquake Rupture Forecast, Version 2 (UCERF 2), prepared by 2007 Working Group on California Earthquake Probabilities: US Geological Survey Open File Report 2007-1437 and California Geological Survey Special Report 203, [http://pubs.usgs.gov/of/2007/1497/].

TABLE C-1
CHARACTERIZATION[1] OF FAULTS SIGNIFICANT TO THE
1201 SOUTH GRAND AVENUE PROJECT

Fault Name	Style of Faulting[2]	Maximum Magnitude (Mw)	Slip Rate (mm/yr)	Closest Rupture Distance From Site (km)
Puente Hills (LA)	RV	6.8	0.9	4
Puente Hills	RV	7	0.9	5
Elysian Park (Upper)	RV	6.5	1.9	5
Hollywood	OBL	6.5	0.9	9
Newport-Inglewood	SS	7.2	1.0	10
Raymond	OBL	6.6	2.0	11
Compton	RV	7.3	0.9	14
Verdugo	RV	6.8	0.4	15
Santa Monica	OBL	6.6	1.0	15
Puente Hills (Santa Fe Springs)	RV	6.4	0.9	20
Sierra Madre	RV	7.2	2.0	22
Elsinore - Whittier[3]	SS	7.0	4.2	22
Puente Hills (Coyote Hills)	RV	6.7	0.9	24
Palos Verdes	SS	7.4	3.0	26
Malibu Coast	OBL	6.9	0.3	26
Anaheim	RV	6.3	0.2	28
Sierra Madre (San Fernando)	RV	6.5	2.0	29
Anacapa-Dume	OBL	7.1	0.4	29
Redondo Canyon	RV	6.6	0.4	29
Northridge	RV	6.9	1.5	30
Northridge Hills	RV	6.8	1.3	30
Clamshell-Sawpit	RV	6.4	0.4	31
Mission Hills	RV	6.3	1.3	31
Santa Susana East (connector)	RV	6.2	6.0	31
San Gabriel	OBL	7.3	0.4	33
Santa Susana	RV	6.9	6.0	34
San Jose	OBL	6.5	0.4	38
Peralta Hills	RV	6.4	0.4	41

Fault Name	Style of Faulting[2]	Maximum Magnitude (Mw)	Slip Rate (mm/yr)	Closest Rupture Distance From Site (km)
Holser	RV	6.7	0.4	44
San Pedro Basin	SS	7.1	1.0	44
Chino	OBL	6.7	1.0	45
Cucamonga	RV	6.8	1.5	51
San Joaquin Hills	RV	6.8	0.6	51
Simi-Santa Rosa	OBL	6.8	0.6	52
Oak Ridge (Onshore)	RV	7.1	4.0	56
Del Valle	RV	6.2	0.4	56
San Andreas[3]	SS	8.2	29.0	57
Newport-Inglewood Offshore	SS	7	1.0	62
Fontana	SS	6.6	0.4	64
Malibu Coast (Extension)	OBL	6.9	0.3	64
San Cayetano	RV	7.1	6.0	65
San Diego Trough North	SS	7.3	2.0	70
San Jacinto[3]	SS	7.9	6.0	71
Santa Cruz-Catalina Ridge	OBL	7.4	1.0	73
Sisar	RV	6.8	0.4	74
Oceanside Blind Thrust	RV	7.2	1.0	75
Cleghorn	SS	6.7	0.5	80
Ventura-Pitas Point	OBL	7.1	1.6	81
Santa Ynez (East)	SS	7.2	2.0	83
Santa Cruz Island	SS	7.2	0.6	94
Channel Islands Thrust	RV	7.2	1.5	95
Oak Ridge (Offshore)	RV	6.9	3.0	96
Mission Ridge-Arroyo Parida-Santa Ana	RV	7.0	0.9	97
North Frontal (West)	RV	7.1	0.1	97
San Clemente	SS	7.5	1.8	98
Coronado Bank	SS	7.4	1.8	100

Notes:

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

(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 C-2
SITE-SPECIFIC MCE_R DEVELOPMENT CALCULATION SHEET
1201 SOUTH GRAND AVENUE PROJECT

Column 1	Column 2	Column 3	Column 4	Column 5	Column 6	Column 7	Column 8	Column 9	Column 10	Column 11	Column 12
		2475-yr UHS (PSHA)	*Risk Collapse Scaling Factors*	*Max. Orientation Scaling Factors*	*Probabilistic MCE_R*	*84th %tile DSHA*	*Max. Direction 84th %tile DSHA*	*Code-Based Detetermistic Minimum MCE_R*	*Deterministic MCE_R*	*Code Minimum MCE_R*	*Final Site-Specific MCE_R*
Period	***Frequency***	*RotD50*		*RotD50*	*RotD100*	*RotD50*	*RotD100*	*RotD100*	*RotD100*	*RotD100*	*RotD100*
(sec)	(Hz)	(g)	-	-	(g)	(g)	(g)	(g)	(g)	(g)	(g)
0.010	100	0.986	0.902	1.190	1.058	2.428	2.889	0.730	1.172	0.878	1.058
0.020	50	1.049	0.902	1.190	1.126	1.011	1.203	0.750	1.203	1.013	1.126
0.030	33	1.089	0.902	1.190	1.168	1.091	1.299	0.809	1.299	1.148	1.168
0.050	20	1.312	0.902	1.190	1.408	1.288	1.532	0.955	1.532	1.417	1.417
0.075	13	1.689	0.902	1.190	1.813	1.604	1.909	1.190	1.909	1.754	1.813
0.100	10	2.021	0.902	1.190	2.170	1.869	2.224	1.386	2.224	1.860	2.170
0.150	6.67	2.330	0.902	1.200	2.523	2.147	2.577	1.606	2.577	1.860	2.523
0.200	5.00	2.448	0.902	1.210	2.672	2.341	2.832	1.765	2.832	1.860	2.672
0.250	4.00	2.392	0.902	1.220	2.632	2.368	2.889	1.800	2.889	1.860	2.632
0.300	3.33	2.220	0.902	1.220	2.443	2.260	2.758	1.718	2.758	1.860	2.443
0.400	2.50	1.905	0.902	1.230	2.113	1.981	2.437	1.518	2.437	1.860	2.113
0.500	2.00	1.655	0.901	1.230	1.835	1.734	2.132	1.329	2.132	1.541	1.835
0.750	1.33	1.169	0.901	1.240	1.306	1.236	1.533	0.955	1.533	1.027	1.306
1.000	1.00	0.879	0.900	1.240	0.981	0.914	1.134	0.706	1.134	0.771	0.981
1.500	0.67	0.537	0.900	1.240	0.599	0.555	0.688	0.429	0.688	0.514	0.599
2.000	0.50	0.378	0.900	1.240	0.422	0.380	0.471	0.293	0.471	0.385	0.422
3.000	0.33	0.230	0.900	1.250	0.258	0.235	0.294	0.183	0.294	0.257	0.258
4.000	0.25	0.158	0.900	1.260	0.179	0.157	0.198	0.124	0.198	0.193	0.193
5.000	0.20	0.126	0.900	1.260	0.143	0.125	0.157	0.098	0.157	0.154	0.154
7.500	0.13	0.080	0.900	1.280	0.093	0.070	0.090	0.056	0.090	0.103	0.103
10.000	0.10	0.051	0.900	1.290	0.059	0.042	0.055	0.034	0.055	0.062	0.062

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; GMRotI50 and RotD50 are produced by NGA West 1 and West2, respectively.
Column 4	= Site-specific risk coefficient (CR) from USGS.
Column 5	= Scale factor to obtain maximum-oriented spectral acceleration; from Shahi and Baker (2014).
Column 6	= Probabilistic risk-targeted, maximum considered earthquake ground motion spectral ordinates in units of g for 5% damping.
Column 7	= 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 8	= Deterministic, maximum considered earthquake ground motion spectral ordinates in units of g for 5% damping.
Column 9	= 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 10	= Deterministic maximum considered earthquake ground motion spectral ordinates in units of g for 5% damping; maximum value from Columns 8 and 9.
Column 11	= 80% of code-based (ASCE 7-16, Ch. 11) risk-targeted, maximum considered earthquake ground motion spectral ordinates in units of g for 5% damping.
Column 12	= Final risk-targeted, maximum considered earthquake ground motion spectral ordinates in units of g for 5% damping; minimum value from Columns 6 and 10, but no less than Column 11.
Column 13	= Final horizontal average risk-targeted, maximum considered earthquake ground-motion spectral ordinates in units of g for 5% damping; value from Column 12 divided by Column 5.

TABLE C-3
SITE-SPECIFIC DBE AND SLE DEVELOPMENT CALCULATION SHEET
1201 SOUTH GRAND AVENUE PROJECT

Column 1	Column 2	Column 3	Column 4	Column 5	Column 6	Column 7	Column 8	Column 9
		Code-Based DRS	*80% of Code-Based DRS*	*2/3 of MCE_R*	*Final Site-Specific DRS*	*43-yr UHS (PSHA)*	*Damping Scaling Factors*	*SLE ($\beta = 1.7\%$)*
Period	***Frequency***	*RotD100*	*RotD100*	*RotD100*	*RotD100*	*RotD50*		*RotD50*
(sec)	(Hz)	(g)	(g)	(g)	(g)	(g)	(g)	(g)
0.010	100	0.732	0.586	0.705	0.705	0.178	0.999	0.178
0.020	50	0.844	0.676	0.751	0.751	0.189	1.008	0.190
0.030	33	0.957	0.765	0.779	0.779	0.195	1.037	0.203
0.050	20	1.181	0.945	0.945	0.945	0.237	1.118	0.265
0.075	13	1.461	1.169	1.209	1.209	0.302	1.216	0.367
0.100	10	1.550	1.240	1.446	1.446	0.358	1.288	0.461
0.150	6.67	1.550	1.240	1.682	1.682	0.412	1.345	0.554
0.200	5.00	1.550	1.240	1.781	1.781	0.415	1.367	0.567
0.250	4.00	1.550	1.240	1.755	1.755	0.394	1.366	0.537
0.300	3.33	1.550	1.240	1.628	1.628	0.364	1.371	0.499
0.400	2.50	1.550	1.240	1.409	1.409	0.305	1.372	0.418
0.500	2.00	1.284	1.027	1.223	1.223	0.259	1.371	0.355
0.750	1.33	0.856	0.685	0.871	0.871	0.176	1.359	0.239
1.000	1.00	0.642	0.514	0.654	0.654	0.128	1.355	0.174
1.500	0.67	0.428	0.342	0.399	0.399	0.078	1.347	0.105
2.000	0.50	0.321	0.257	0.281	0.281	0.055	1.331	0.073
3.000	0.33	0.214	0.171	0.172	0.172	0.033	1.322	0.044
4.000	0.25	0.161	0.128	0.128	0.128	0.023	1.303	0.030
5.000	0.20	0.128	0.103	0.103	0.103	0.018	1.291	0.023
7.500	0.13	0.086	0.068	0.068	0.068	0.010	1.258	0.013
10.000	0.10	0.051	0.041	0.041	0.041	0.006	1.188	0.007

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. 11) design ground motion spectral ordinates in units of g for 5% damping.
Column 4	= Code-based (ASCE 7-16, Ch. 21) minimum design ground motion spectral ordinates in units of g for 5% damping; 80% of the value in Column 3.
Column 5	= Minimum Design Earthquake (DE) ground motion spectral ordinates in units of g for 5% damping; 2/3 of the MCE_R.
Column 6	= Final design ground motion spectral ordinates in units of g for 5% damping; maximum value from Columns 4 and 5.
Column 7	= Mean uniform hazard spectral ordinates for 43- yr average return period in units of g for 5% damping.
Column 8	= Damping Scaling Factor used to convert spectral ordinates from 5% damping to 1.7% damping; developed per Rezaeian et al. (2012).
Column 9	= Service-Level Earthquake ground motion spectral ordinates in units of g for 1.7% damping; developed per Rezaeian et al. (2012).

TABLE C-4
CHARACTERISTICS OF GROUND MOTION RECORDS SELECTED FOR SPECTRAL MATCHING
1201 SOUTH GRAND AVENUE PROJECT

Local Event Seed Time History Records

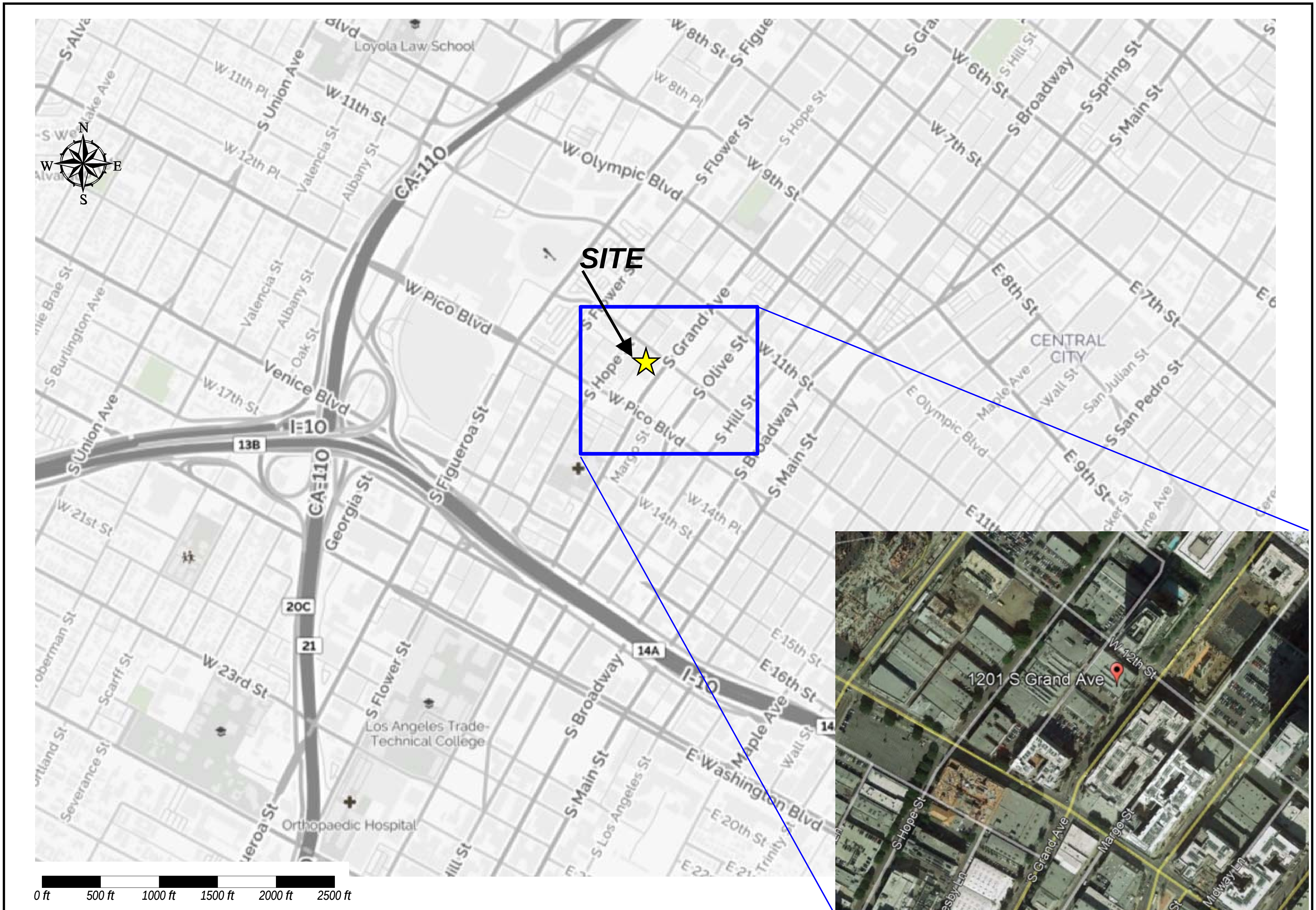
Analysis Record No.	*Earthquake Name*	*Station Name*	*PEER NGA Record No.*	*H1* (deg)	*H2* (deg)	*Date*	*Earthquake Magnitude*	*Rupture Mechanism*	*Closest Distance* (km)	*NEHRP Site Class/V_{S30}* (m/s)	*PGA* (g)	*D_{a5-95}* (sec)	*T_L* (sec)	*T_P* (sec)
GM1	*Imperial Valley-06*	*El Centro Differential Array*	*184*	*270*	*0*	10/15/1979	6.53	SS	5.09	D / 202	0.437	7.0	34.78	6.265
GM2	*Loma Prieta*	*Gilroy Array #2*	*766*	*0*	*90*	10/18/1989	6.93	RV/OBL	11.07	D / 271	0.357	11.0	13.33	1.729
GM3	*Chuetsu-oki, Japan*	*Joetsu Kakizakiku Kakizaki*	*4847*	*0*	*90*	07/16/2007	6.8	RV	11.94	C / 383	0.424	20.3	11.43	1.4
GM4	*Bam, Iran*	*Bam*	*4040*	*278 (FP)*	*8 (FN)*	12/26/2003	6.6	SS	1.7	C / 487	0.738	9.6	16.00	2.023
GM5	*Niigata, Japan*	*NIGH11*	*4228*	*0*	*90*	10/23/2004	6.63	RV	8.93	C / 375	0.508	12.2	20.00	1.799
GM6	*Hector Mine*	*Hector*	*1787*	*0*	*90*	10/16/1999	7.13	SS	11.66	C / 726	0.311	11.7	26.67	N/A
GM7	*Iwate, Japan*	*MYGH02*	5678	*0*	*90*	06/13/2008	6.9	RV	11.1	D / 399	0.252	9.5	26.67	N/A

Distant Event Seed Time History Records

GM8	*El Mayor-Cucapah*	*Bonds Corner*	*5969*	*0*	*90*	04/04/2010	7.2	SS	32.9	D / 223	0.240	38.3	16.00	N/A
GM9	*Kocaeli, Turkey*	*Atakoy*	*1149*	*0*	*90*	08/17/1999	7.51	SS	58.3	D / 310	0.121	36.1	26.67	N/A
GM10	*Chi-Chi, Taiwan*	*CHY088*	*1236*	*90*	*0*	09/29/1999	7.62	RV/OBL	37.5	C / 319	0.184	33.8	20.00	N/A
GM11	*Denali, Alaska*	*Carlo (temp)*	*2107*	*90*	*0*	11/03/2002	7.9	SS	50.9	C / 399	0.090	24.3	12.82	N/A

Earthquake Characteristic Key

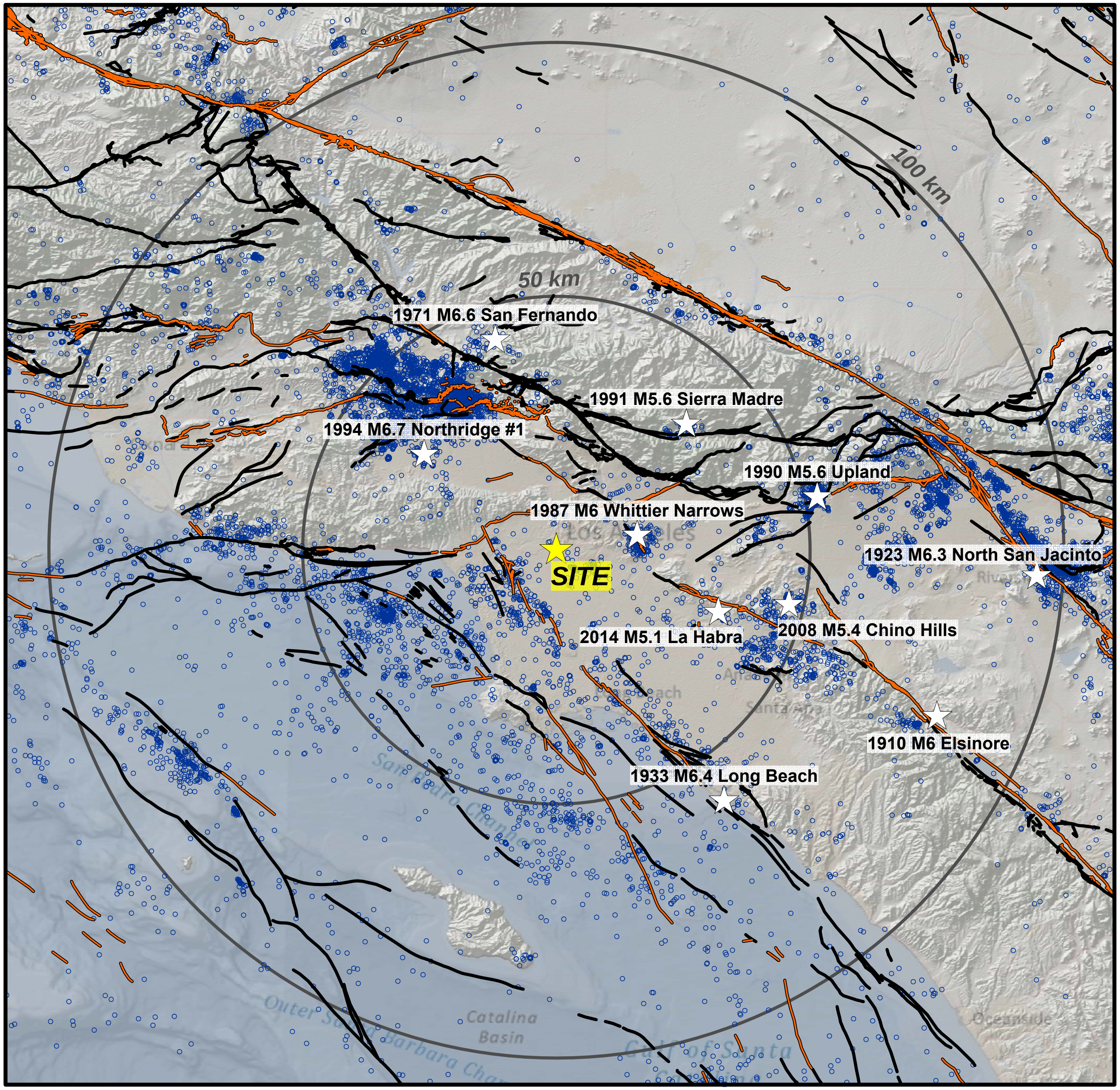
Earthquake Name	= The common name of earthquake; usually includes the name of the general area or country where earthquake occurred.
Station Name	= The unique name of strong-motion station.
PEER NGA Record No.	= An arbitrary unique number assigned to each strong-motion record in the NGA database for identification purposes.
H1	= The orientation of the H1 component, if orientation is within 5 degrees of fault normal or fault parallel, denoted with (FN) or (FP).
H2	= The orientation of the H2 component, if orientation is within 5 degrees of fault normal or fault parallel, denoted with (FN) or (FP).
Date	= Date of earthquake.
Earthquake Magnitude	= Moment magnitude of earthquake.
Rupture Mechanism	= Mechanism based on rake angle, SS = Strike-slip, RV = Reverse, RV/OBL = Reverse-Oblique, NML = Normal.
Closest Distance	= Closest distance from the recording site to the ruptured area (km).
NEHRP Site Class/V_{S30}	= The preferred NEHRP site class determined based on the preferred VS30 values (m/s).
PGA	= Peak ground acceleration of the selected record (g).
D_{a5-95}	= Significant duration of the selected record as defined by the 5th to 95th percentile of Arias intensity (sec); geometric mean of significant duration from two components listed.
T_L	= Longest usable period, inverse of lowest usable frequency indicated by PEER; minimum of two components listed (sec).
T_P	= Pulse period of component of record with maximum peak-to-peak velocity (sec); N/A if no pulse is classified in record.



SITE LOCATION MAP

Date: MAY 2020	Project No.: 15083A	Project: 1201 S. GRAND AVENUE PROJECT	Figure C-1

GeoPentech



Legend

USGS Quaternary Fault & Fold Database [1]

Age of Most Recent Displacement

< 15,000 years

< 1,600,000 years

Seismicity [2]

M ≥ 2.0

Historic Earthquake

Notes:

1. Fault traces are from USGS Quaternary Fault and Fold Database (USGS, 2010).
2. Seismicity (hollow blue dots) is from Hauksson et al. (2012) catalog ("HYS" catalog). Catalog includes all instrumentally-recorded events in southern Calfornia from 01/01/1981 through 06/30/2011. Only M ≥ 2.0 events are shown here. Significant post-1900 earthquakes identified by name (white stars) are from the Southern California Earthquake Center (SCEC) online database.

Approximate Scale

N ***0 km*** ***20 km*** ***40 km*** ***60 km***

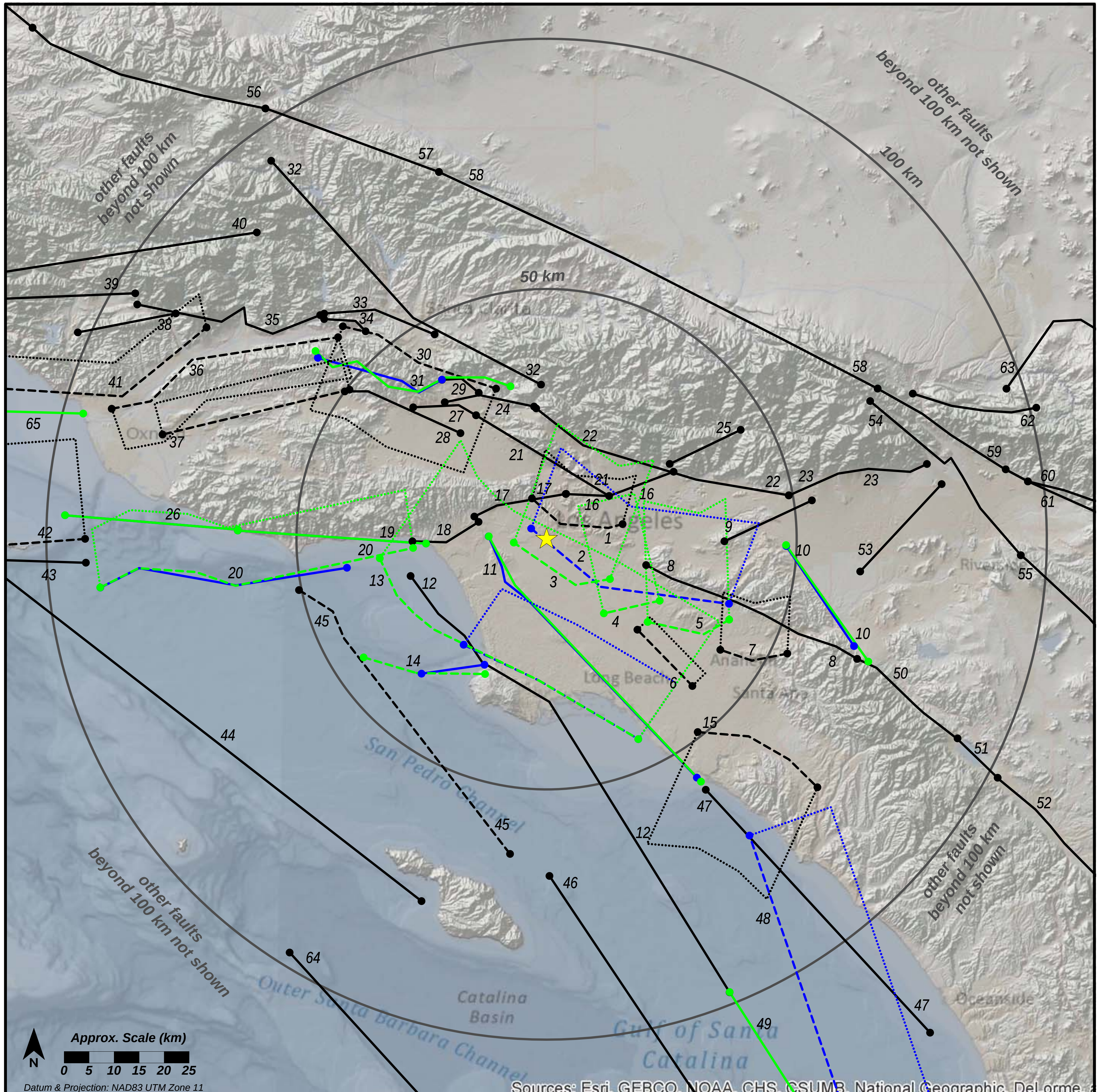
Datum & Projection: NAD83 UTM Zone 11

REGIONAL FAULT & SEISMICITY MAP

Date: JUN 2015	Project No.: 15083A	Project: 1201 S. GRAND AVENUE PROJECT	Figure C-2a

GeoPentech

QFFD & Seismicity Map.srf



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
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	Coronado Bank
50	Elsinore - Glen Ivy
51	Elsinore - Temecula/Glen Ivy Stepover
52	Elsinore - Temecula
53	Fontana
54	San Jacinto - San Bernardino Valley
55	San Jacinto - San Jacinto Valley
56	San Andreas - Big Bend
57	San Andreas - North Mojave
58	San Andreas - South Mojave
59	San Andreas - North San Bernardino
60	San Andreas (North Branch Mill Creek)
61	San Andreas - South San Bernardino
62	Cleghorn
63	North Frontal (West)
64	San Clemente
65	Oak Ridge (Offshore)

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. All fault traces based on UCERF3 (WGCEP, 2013a) except for "Type A" faults (San Andreas, San Jacinto, Elsinore); Type A faults based on UCERF2 (WGCEP, 2008; USGS, 2009). 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 0.05 mm/yr are shown. Slip rates are preferred values from UCERF3 (WGCEP, 2013b). Only Type A faults outside 100 km are shown.
3. Fault Models 1 & 2 based on UCERF3 (WGCEP, 2013a,b). Seismic source characterization geometries for non-Type A faults are generally as shown in WGCEP (2013a,b) and slip rates are in WGCEP (2013b). Magnitude-frequency distributions are composite characteristic with maximum magnitude calculated from Shaw (2009) regression. Type A faults characterized as documented in WGCEP (2008) and 2008 NSHM (Petersen et al., 2008).

Figure 2b - Haz Code Faults.srf

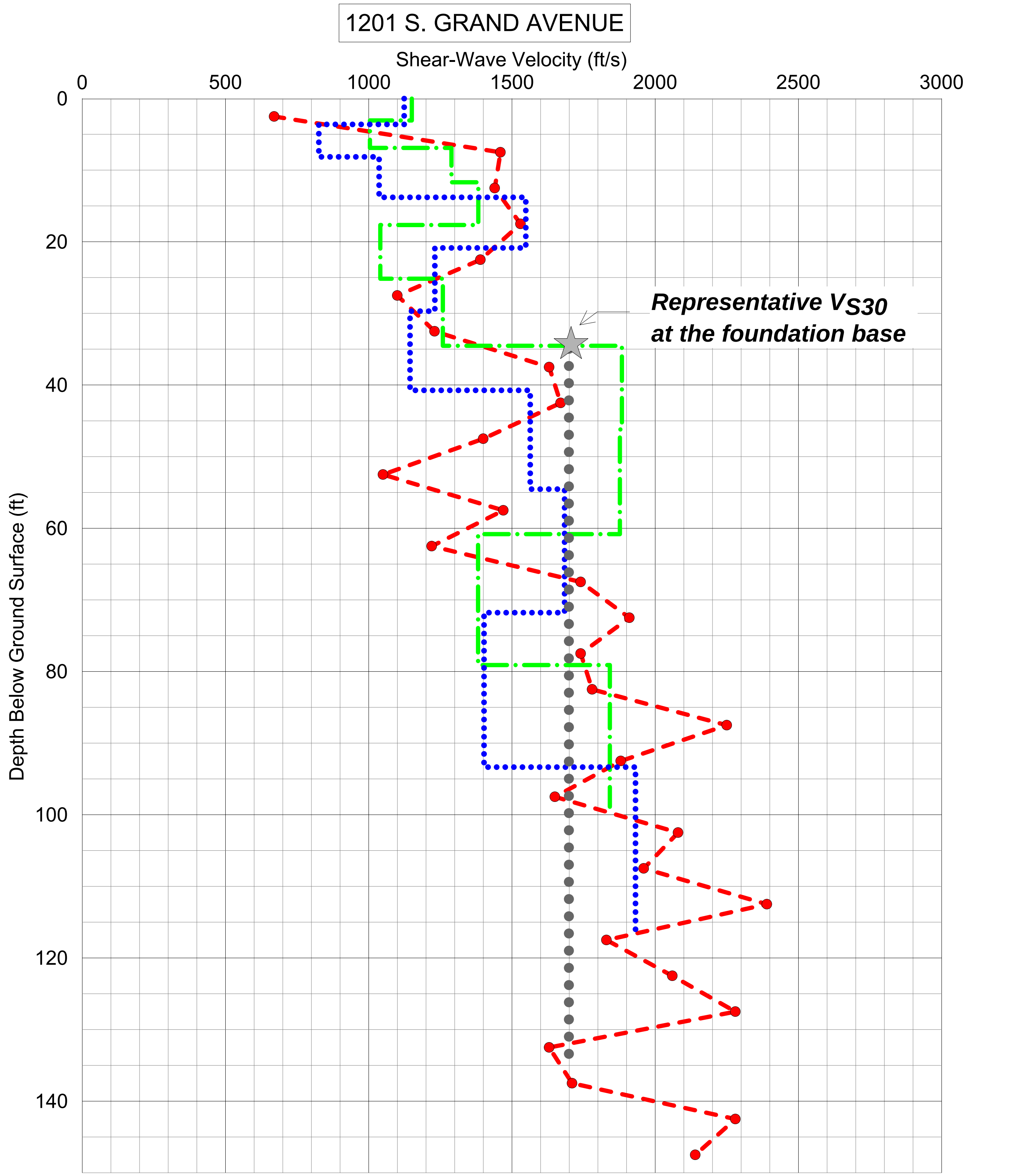
Date: JUN 2015	Project No.: 15083A	Project: 1201 S. GRAND AVENUE PROJECT	SIMPLIFIED FAULT MAP FOR PSHA	Figure C-2b

GeoPentech

FIELD EXPLORATION LOCATIONS

Date: MAR 2018	Project No.: 15083A	Project: 1201 S. GRAND AVENUE PROJECT	Figure C-3a

GeoPentech



LEGEND

SW18-1 B1 - Seismic Downhole (Vs30[35-135']= 1,700 ft/s)

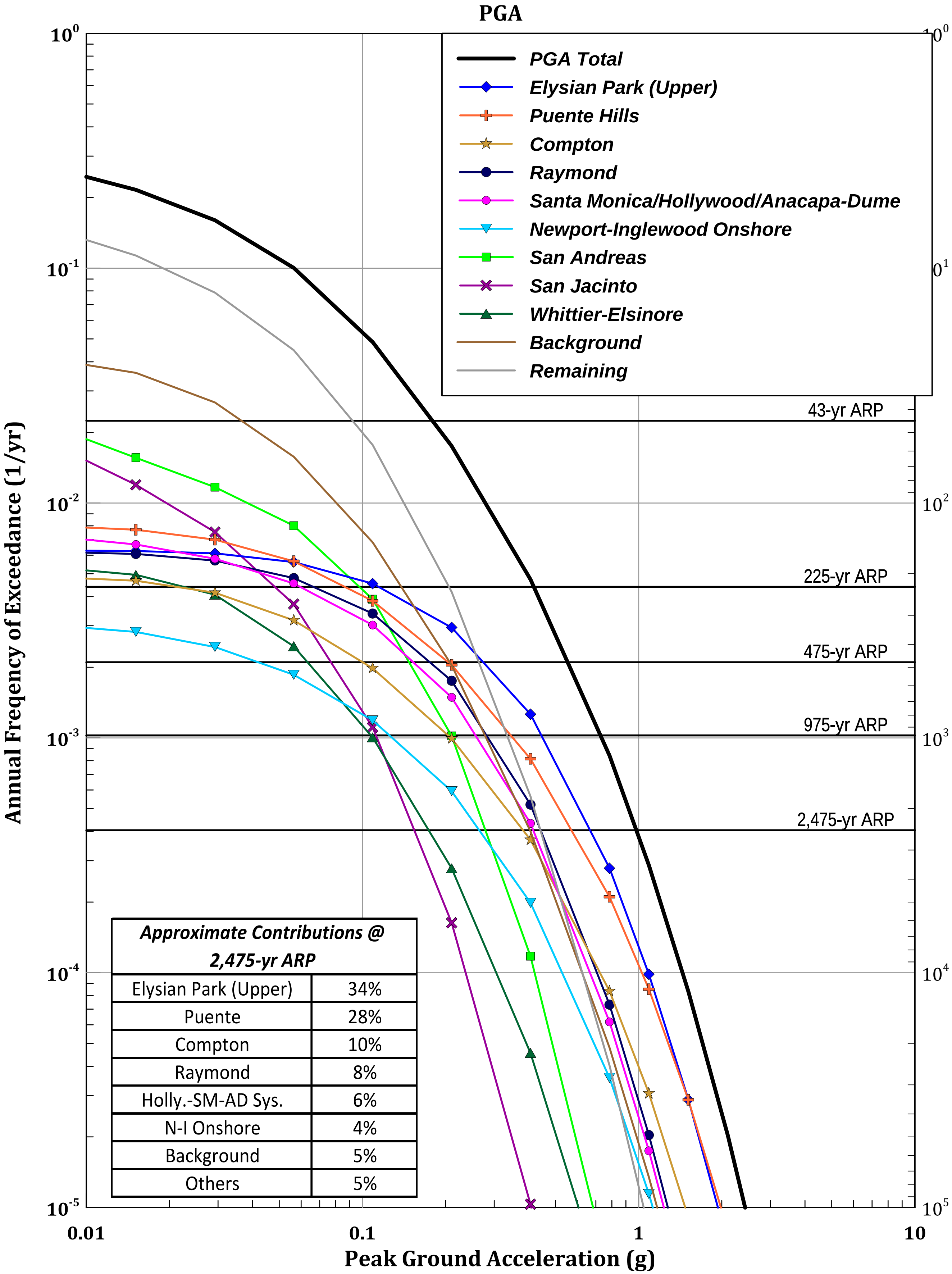
SW18-2

NOTE: V_{S30} shown in legend was calculated below building foundation level (35 ft, bgs)

SHEAR-WAVE VELOCITY PROFILE SUMMARY

Project No.: 15083A	Project: 1201 S. GRAND AVENUE PROJECT	Date: MAR 2018	Figure C-3b

GeoPentech

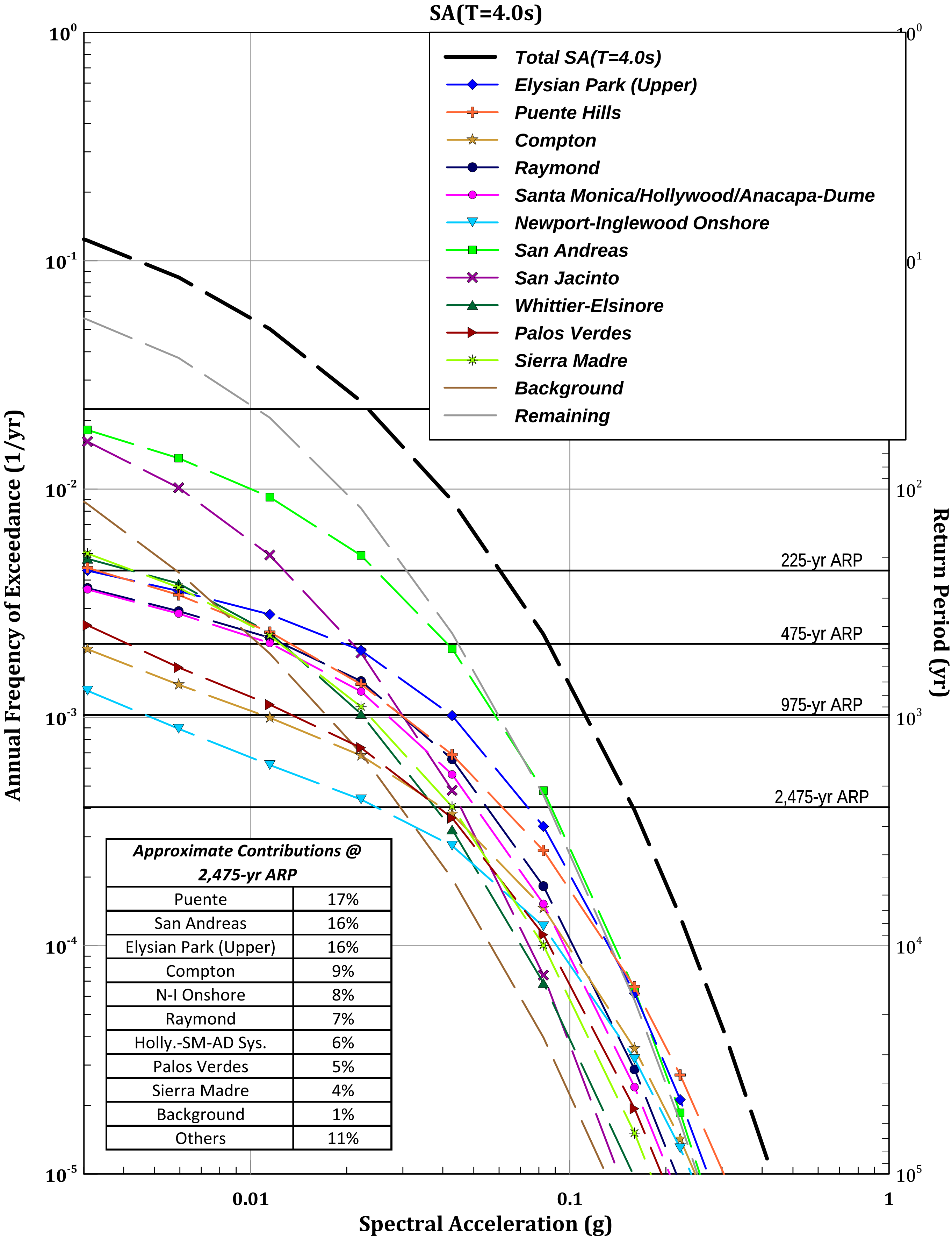


Approximate Contributions @ 2,475-yr ARP	
Elysian Park (Upper)	34%
Puente	28%
Compton	10%
Raymond	8%
Holly.-SM-AD Sys.	6%
N-I Onshore	4%
Background	5%
Others	5%

SOURCE CONTRIBUTIONS TO THE TOTAL HAZARD AT PGA

Project No.: 15083A	Project: 1201 S. GRAND AVENUE PROJECT	Date: MAR 2018	Figure C-4

GeoPentech

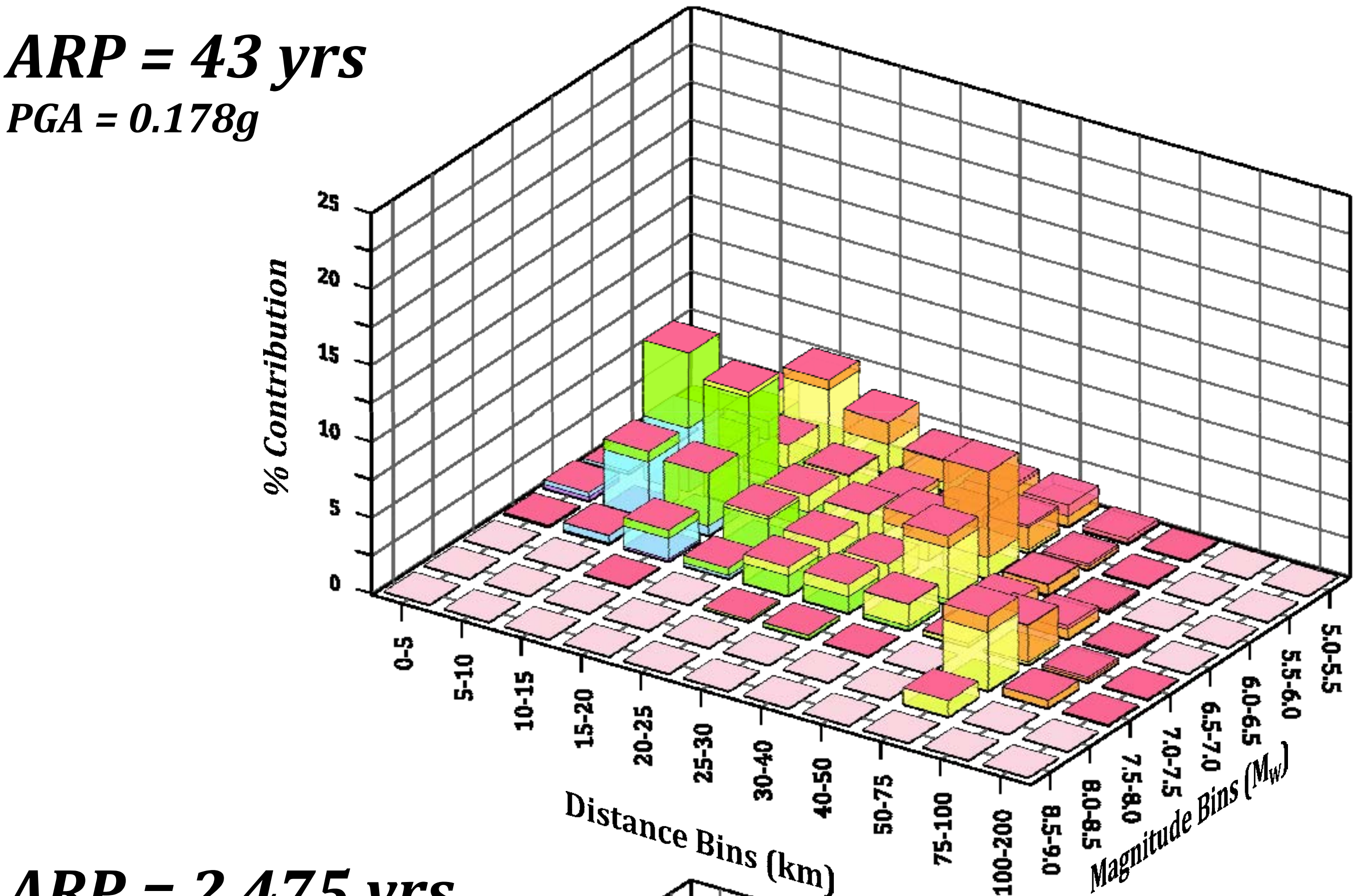


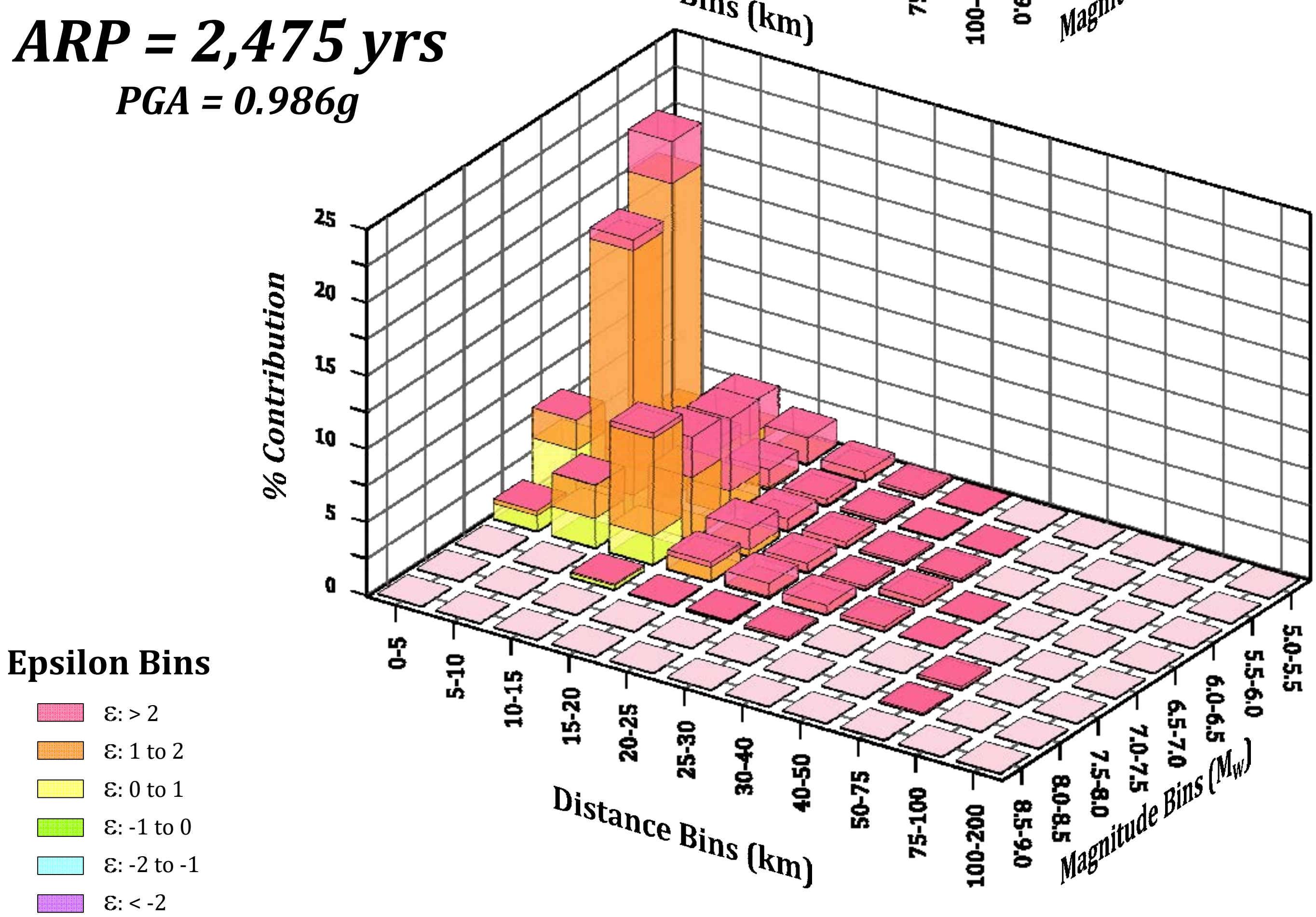
Approximate Contributions @ 2,475-yr ARP	
Puente	17%
San Andreas	16%
Elysian Park (Upper)	16%
Compton	9%
N-I Onshore	8%
Raymond	7%
Holly.-SM-AD Sys.	6%
Palos Verdes	5%
Sierra Madre	4%
Background	1%
Others	11%

SOURCE CONTRIBUTIONS TO THE TOTAL HAZARD AT 4.0-SECOND SPECTRAL PERIOD

Project No.: 15083A	Project: 1201 S. GRAND AVENUE PROJECT	Date: MAR 2018	Figure C-5

GeoPentech

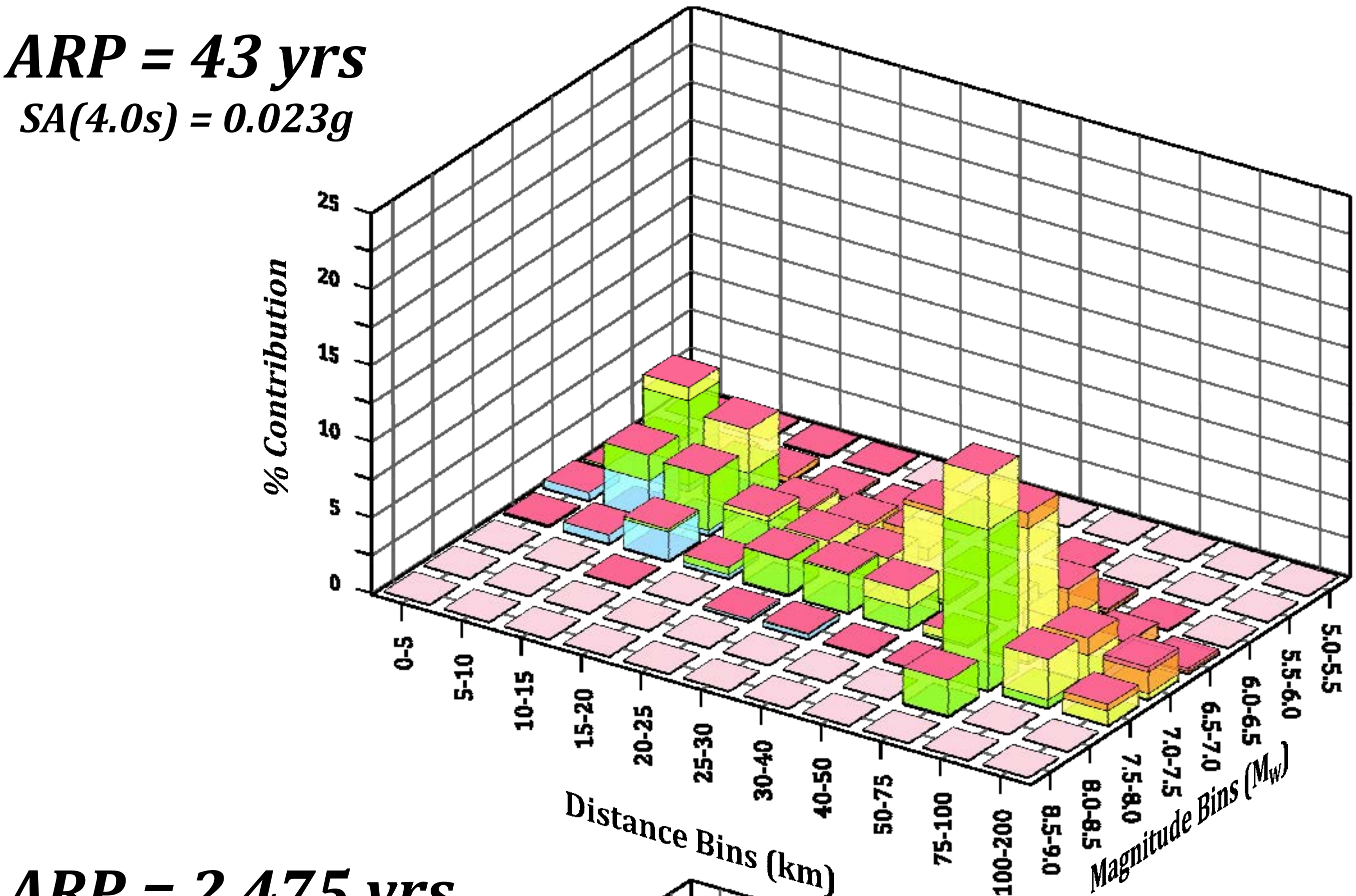


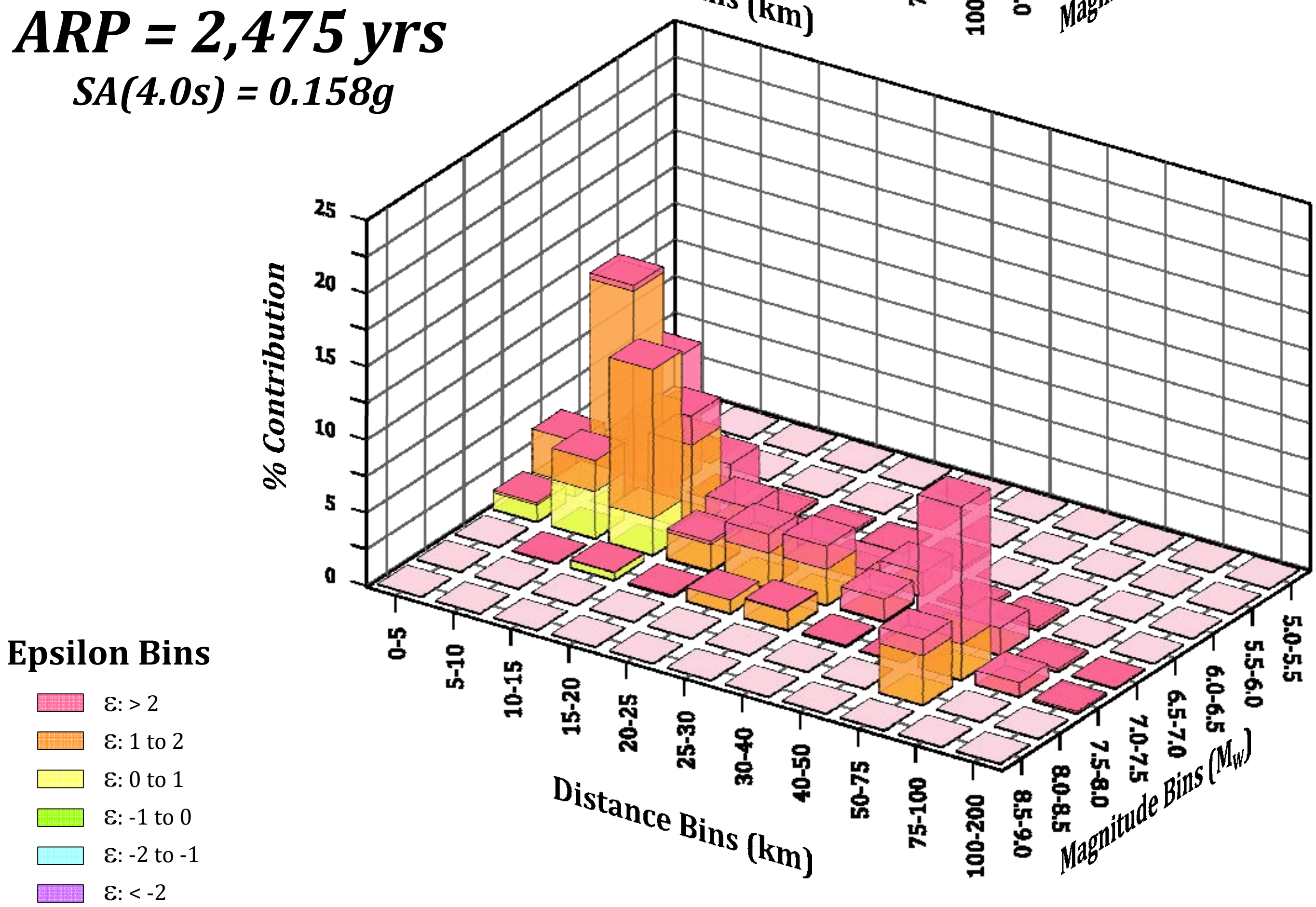


HAZARD DEAGGREGATION FOR PGA

Project No.: 15083A	Project: 1201 S. GRAND AVENUE PROJECT	Date: MAR 2018	Figure C-6

GeoPentech

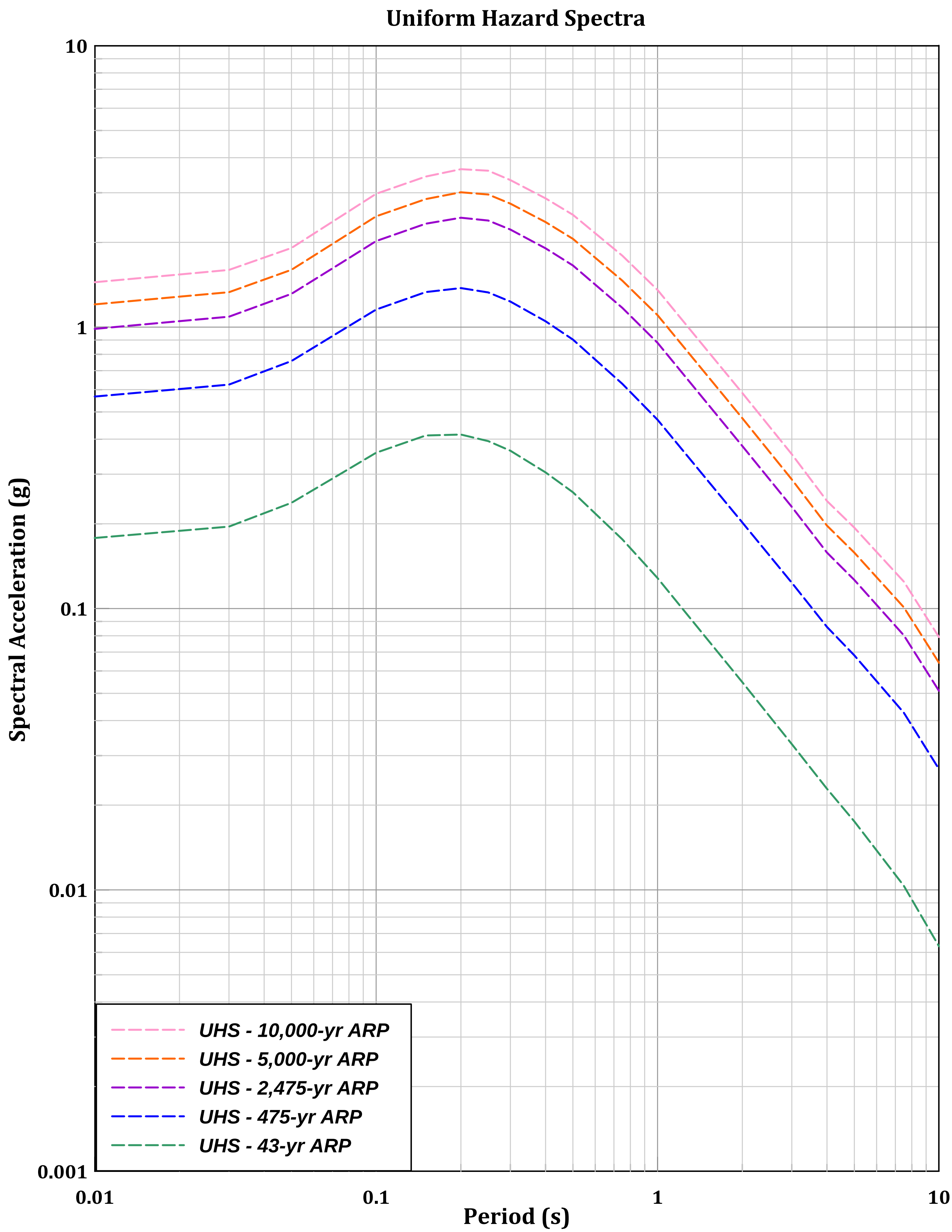




HAZARD DEAGGREGATION FOR 4.0-SECOND SPECTRAL PERIOD

Project No.: 15083A	Project: 1201 S. GRAND AVENUE PROJECT	Date: MAR 2018	Figure C-7

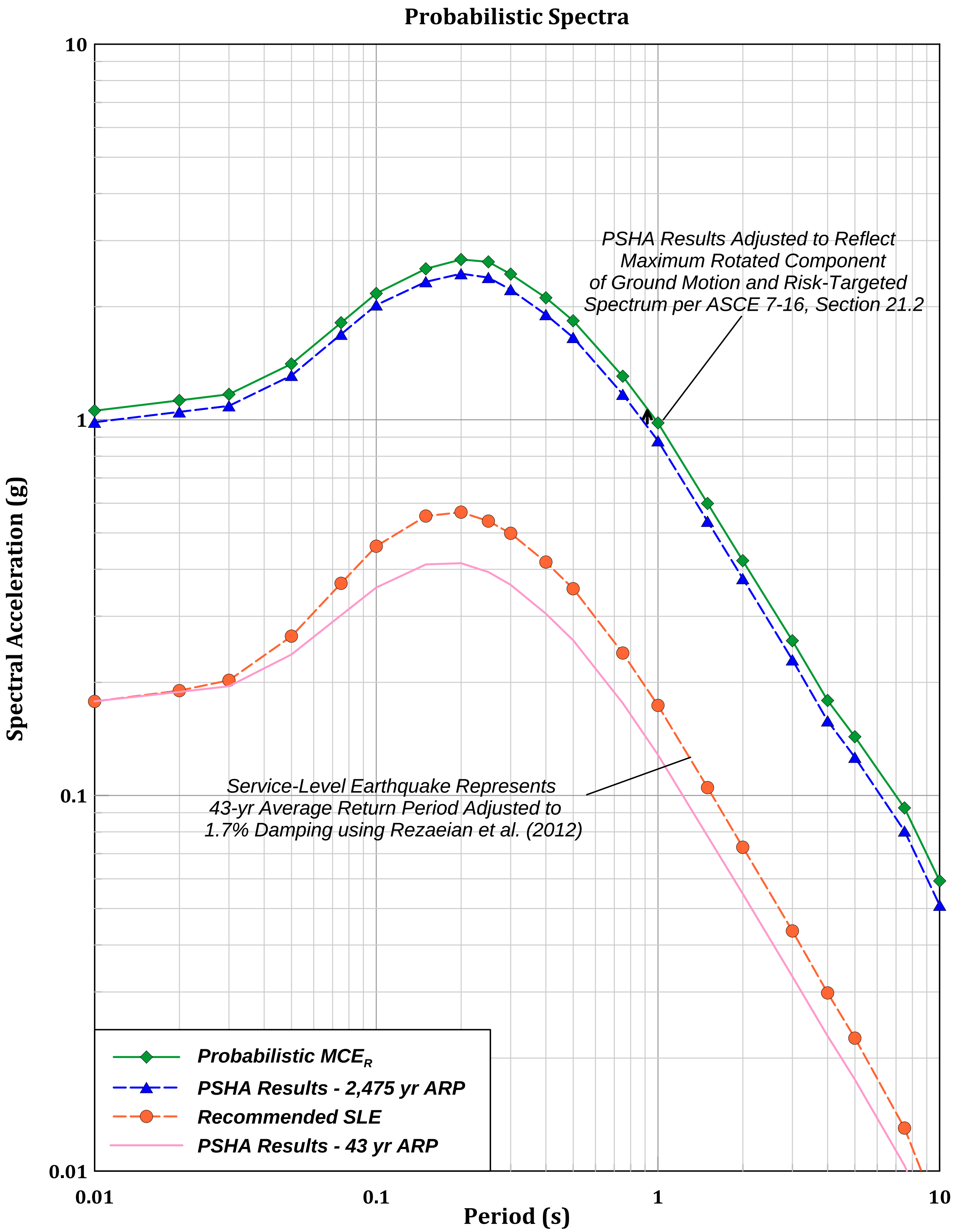
GeoPentech



Note: Spectra represent average horizontal components at 5% damping.

UNIFORM HAZARD SPECTRA			
Project No.: 15083A	Project: 1201 S. GRAND AVENUE PROJECT	Date: MAR 2018	Figure C-8

GeoPentech

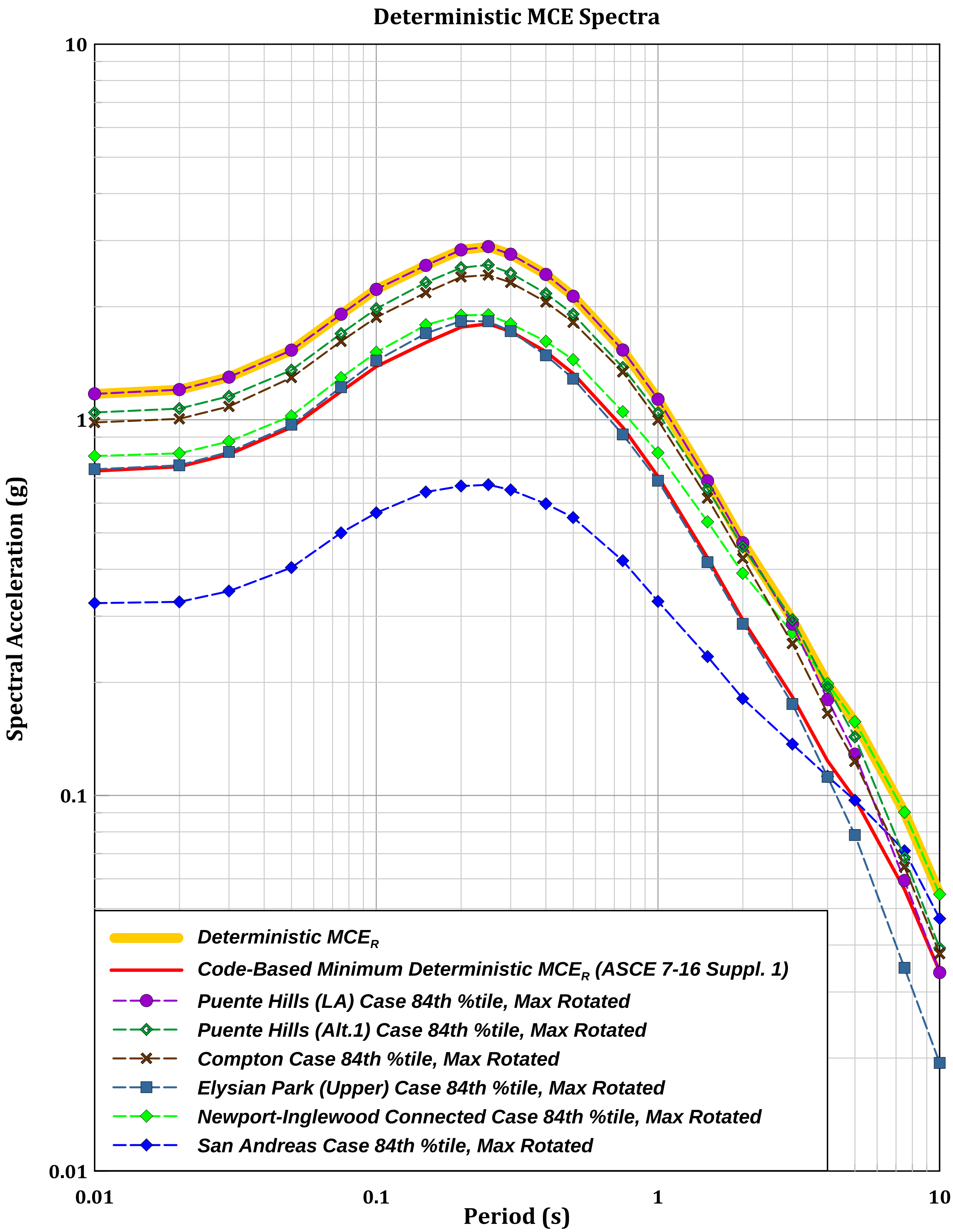


Note: All spectra are for Damping (β) = 5.0% unless otherwise noted.

PROBABILISTIC SPECTRA

Project No.: 15083A	Project: 1201 S. GRAND AVENUE PROJECT	Date: MAY 2020	Figure C-9a

GeoPentech

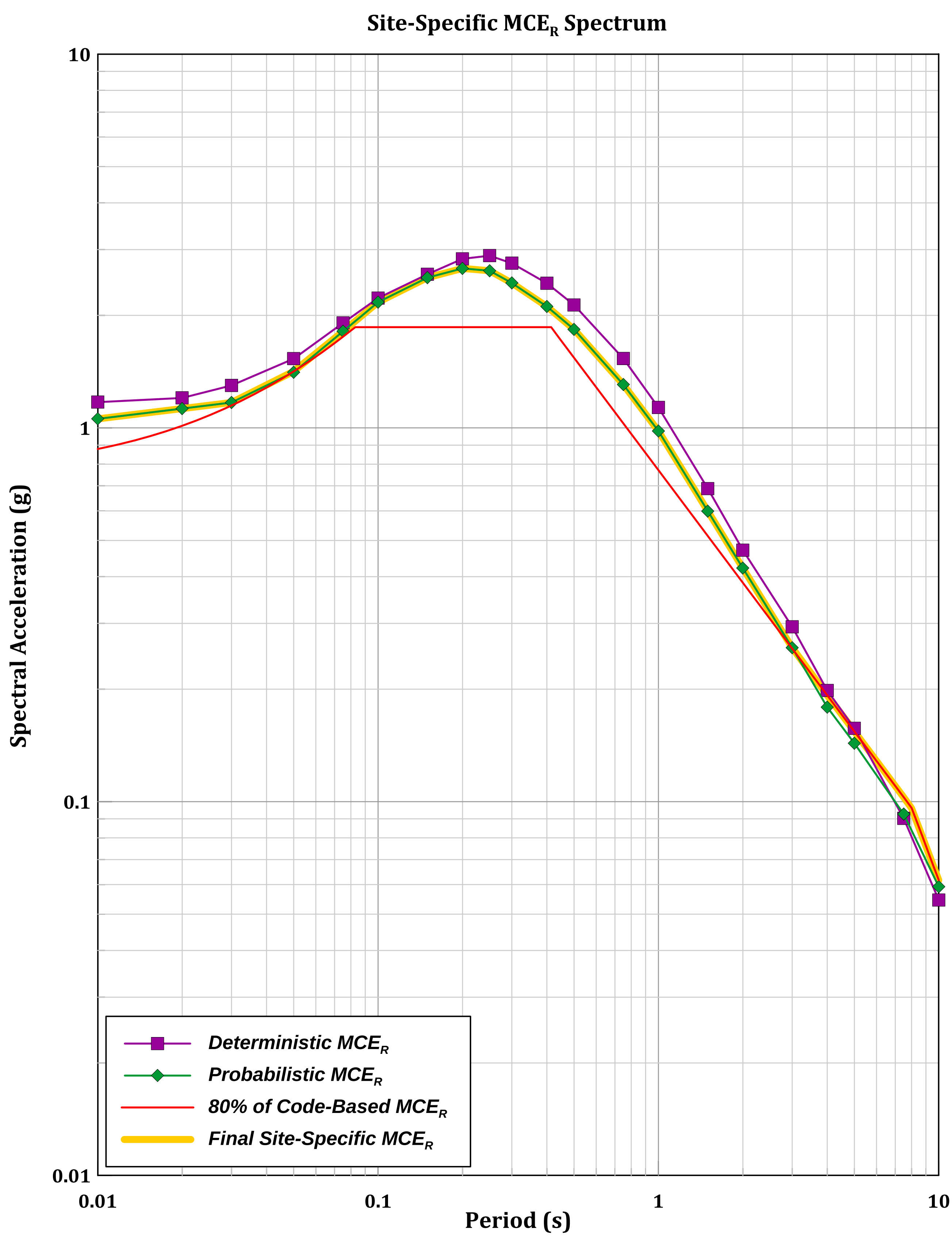


Note: All spectra are for Damping (β) = 5.0%

DETERMINISTIC MCE$_R$ SPECTRUM

Project No.: 15083A	Project: 1201 S. GRAND AVENUE PROJECT	Date: MAY 2020	Figure C-9b

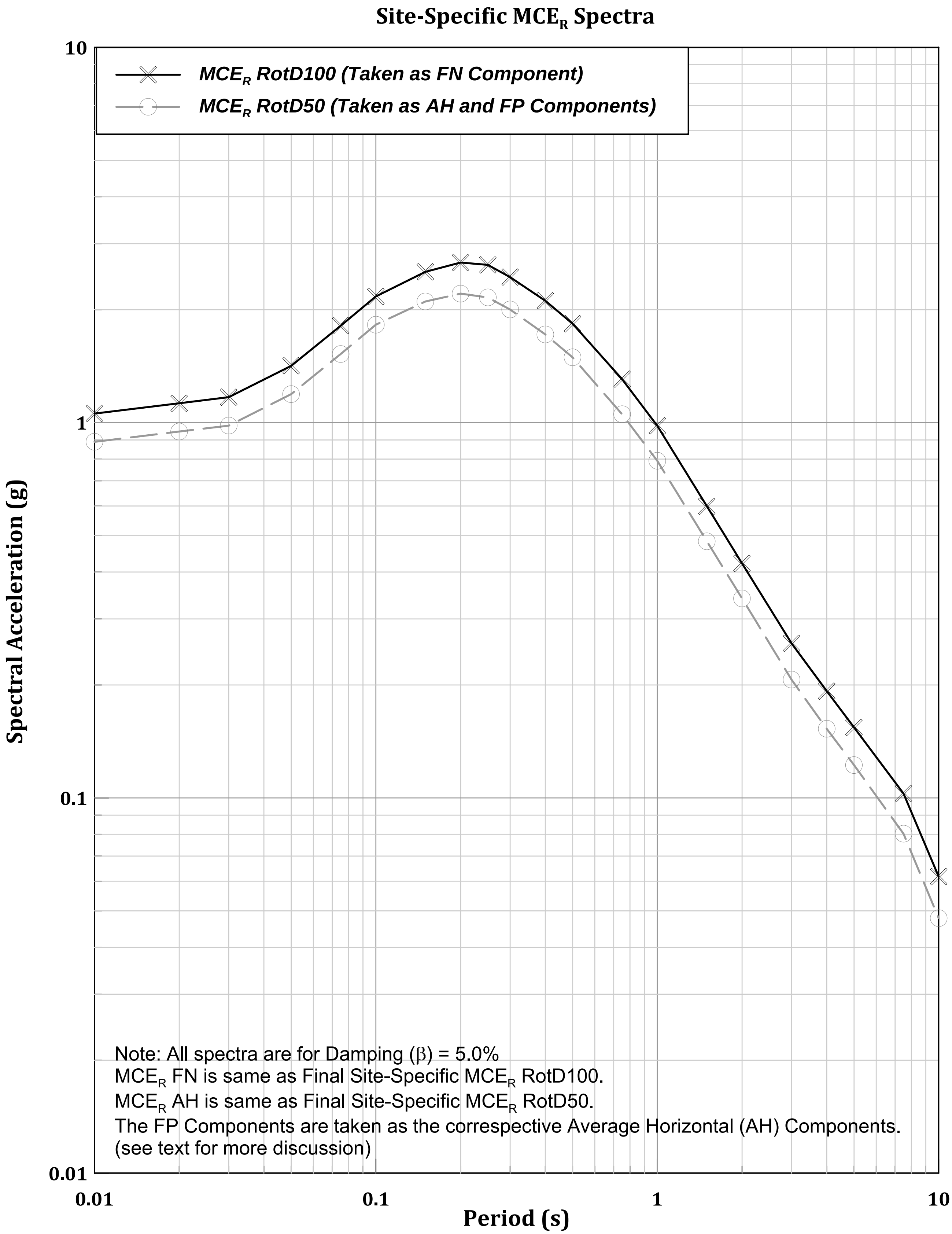
GeoPentech



Note: All spectra are for Damping (β) = 5.0%

SITE-SPECIFIC MCE_R SPECTRUM			
Project No.: 15083A	Project: 1201 S. GRAND AVENUE PROJECT	Date: MAY 2020	Figure C-10a

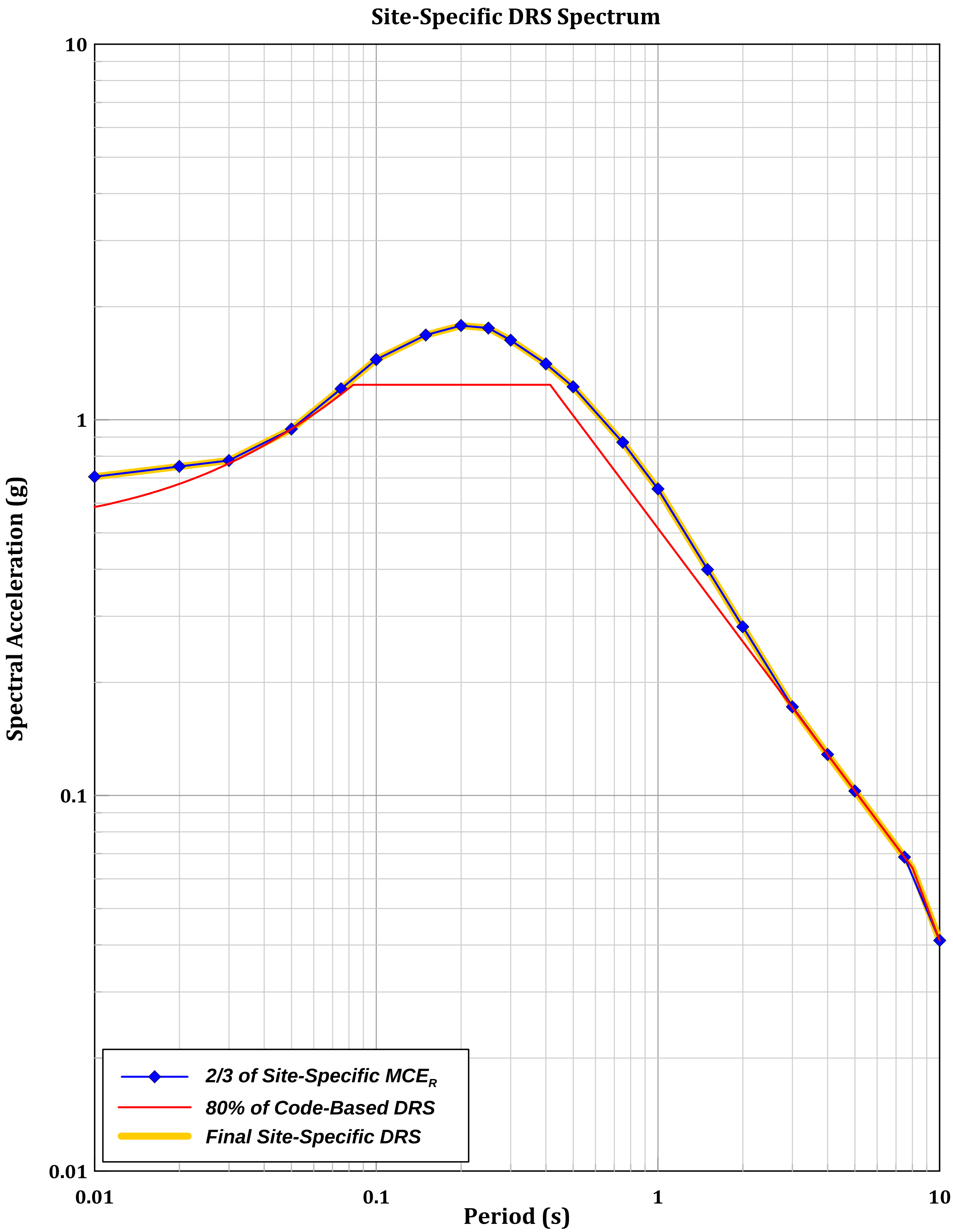
GeoPentech



MCE_R SPECTRA FOR DIFFERENT COMPONENT ORIENTATION

Project No.: 15083A	Project: 1201 S. GRAND AVENUE PROJECT	Date: MAY 2020	Figure C-10b

GeoPentech

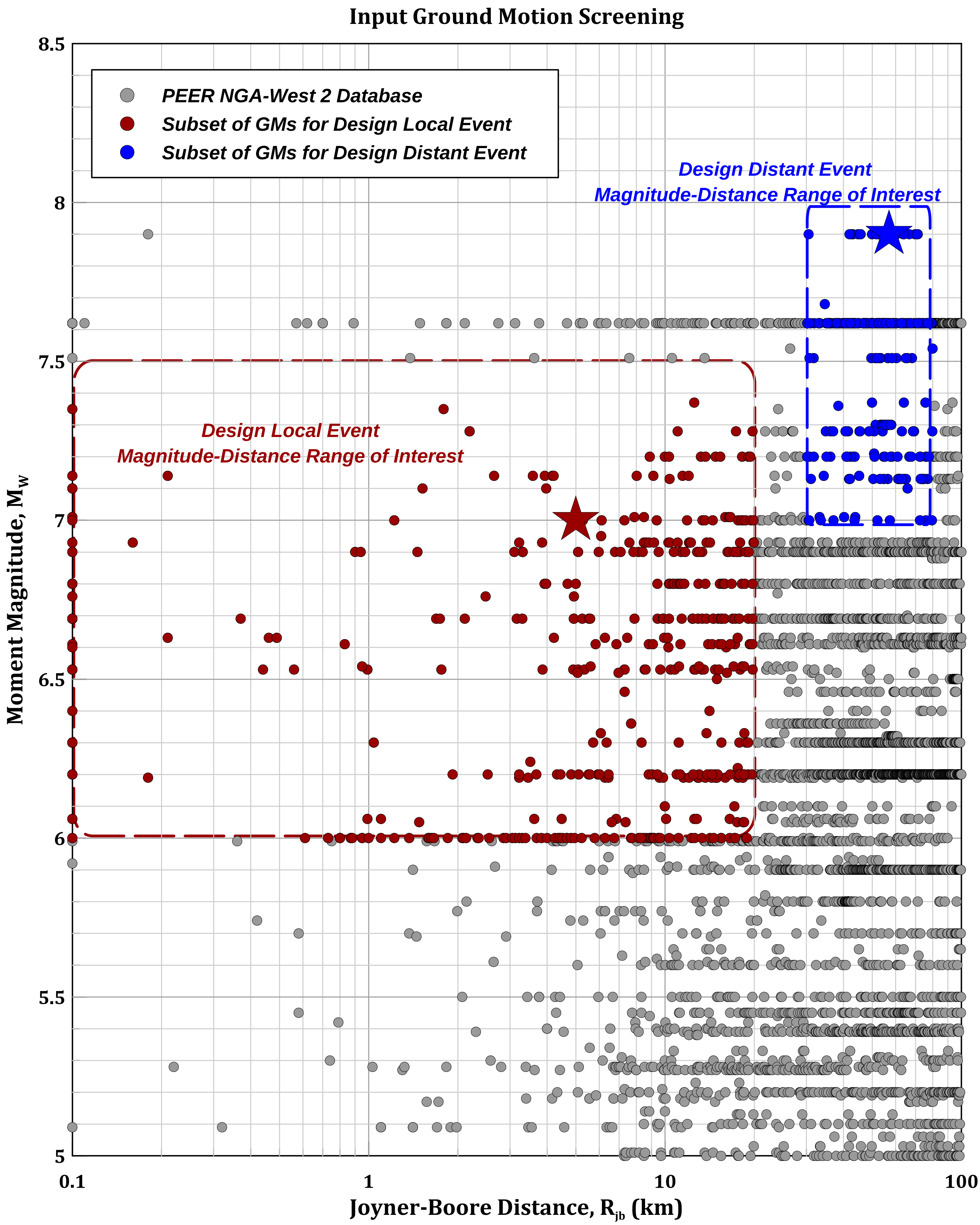


Note: All spectra are for Damping (β) = 5.0%

SITE-SPECIFIC DRS SPECTRUM

Project No.: 15083A	Project: 1201 S. GRAND AVENUE PROJECT	Date: MAY 2020	Figure C-11

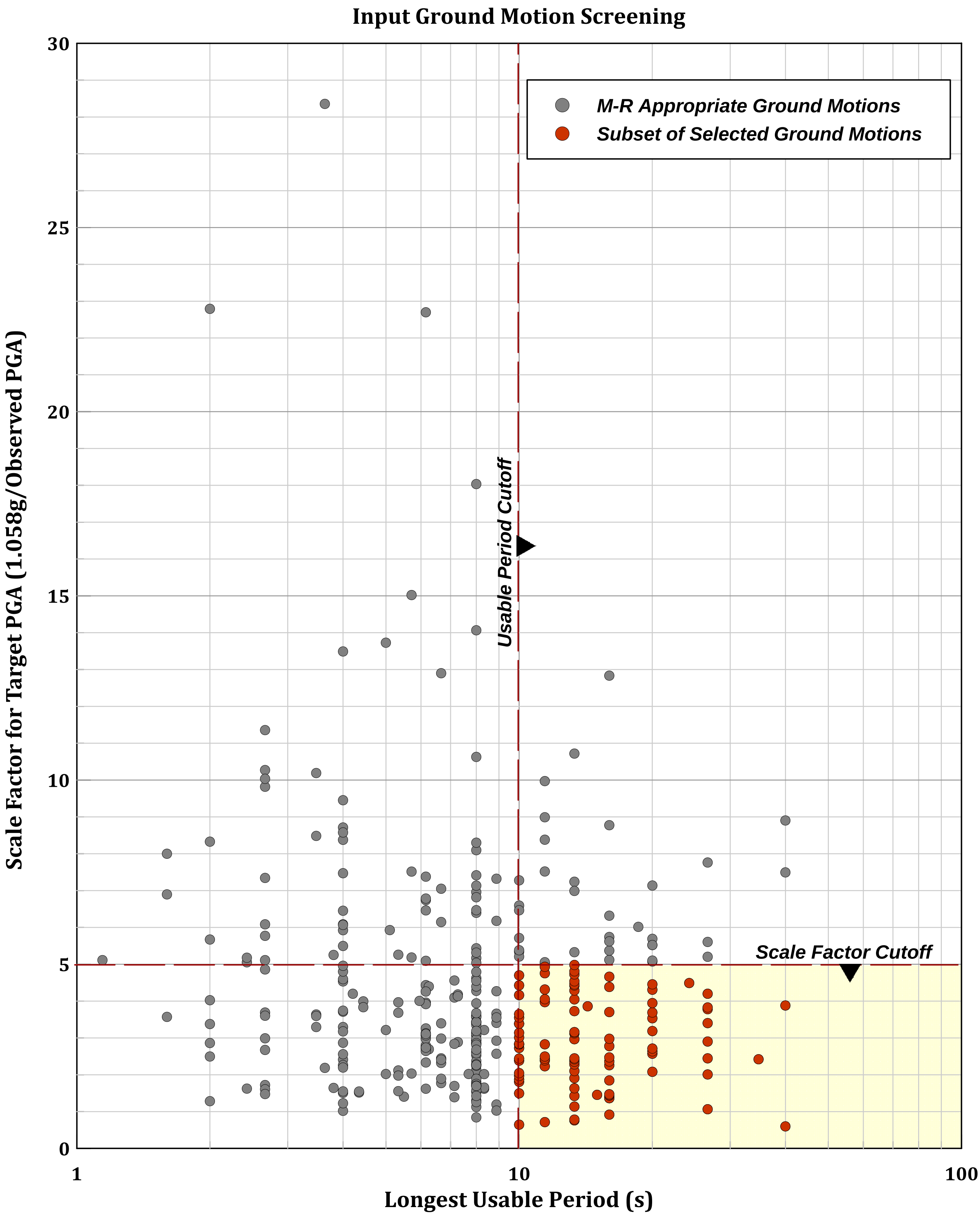
GeoPentech



MAGNITUDE AND DISTANCE SCREENING

Project No.: 15083A	Project: 1201 S. GRAND AVENUE PROJECT	Date: MAR 2018	Figure C-12

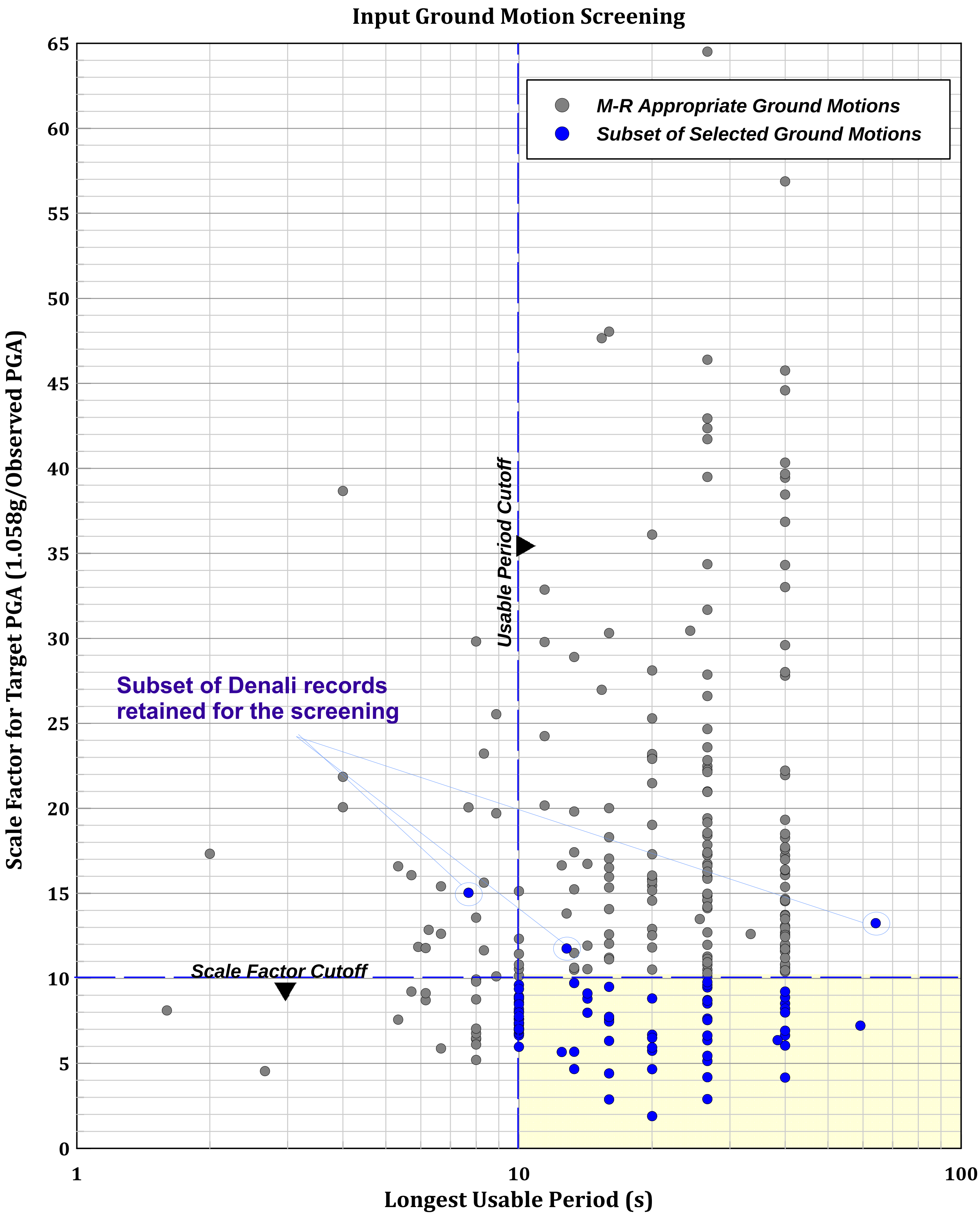
GeoPentech



LONGEST USABLE PERIOD AND SCALE FACTOR SCREENING FOR DESIGN LOCAL EVENT

Project No.: 15083A	Project: 1201 S. GRAND AVENUE PROJECT	Date: MAY 2020	Figure C-13a

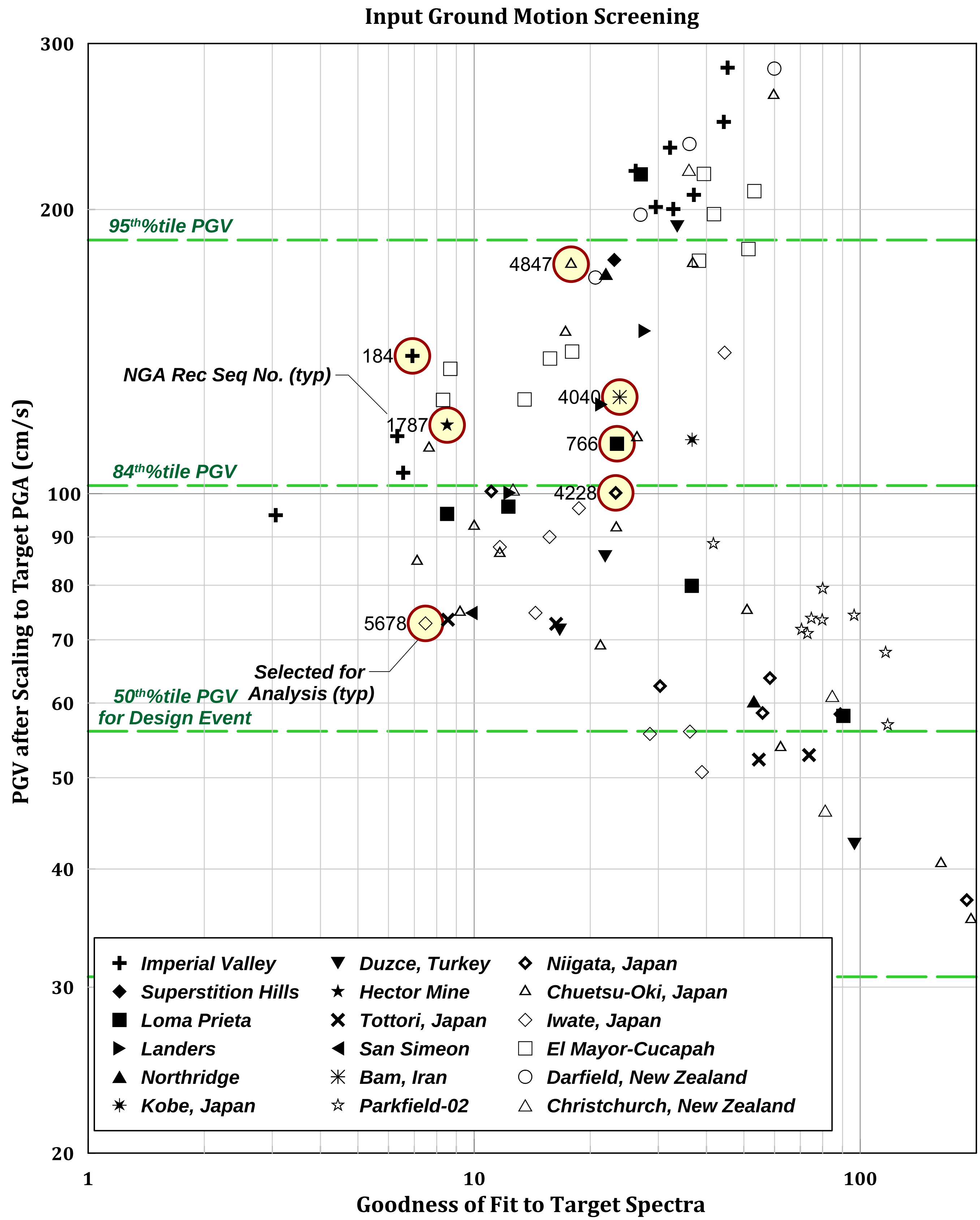
GeoPentech



LONGEST USABLE PERIOD AND SCALE FACTOR SCREENING FOR DESIGN DISTANT EVENT

Project No.: 15083A	Project: 1201 S. GRAND AVENUE PROJECT	Date: MAY 2020	Figure C-13b

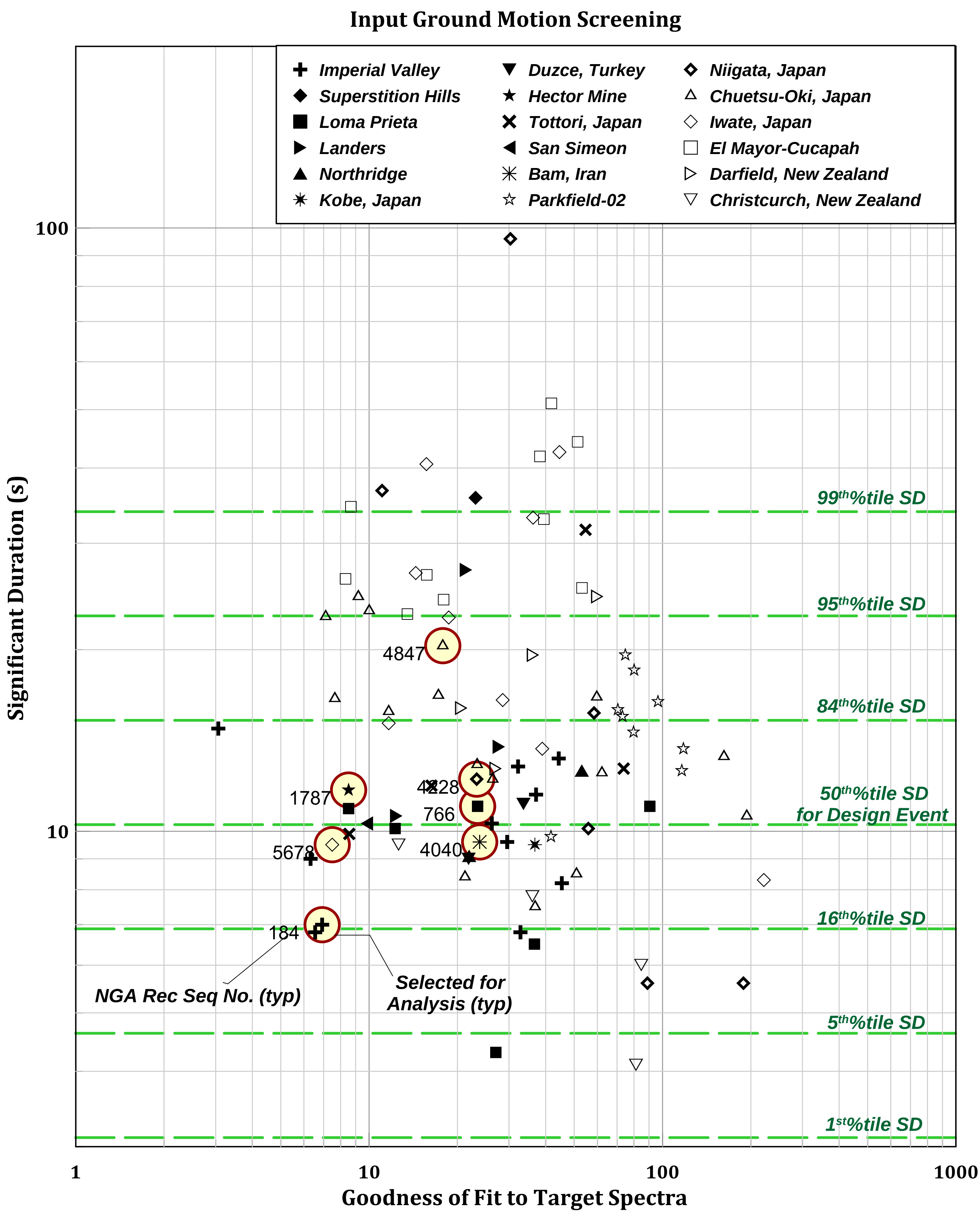
GeoPentech



PGV AND GOODNESS OF FIT SCREENING FOR DESIGN LOCAL EVENT

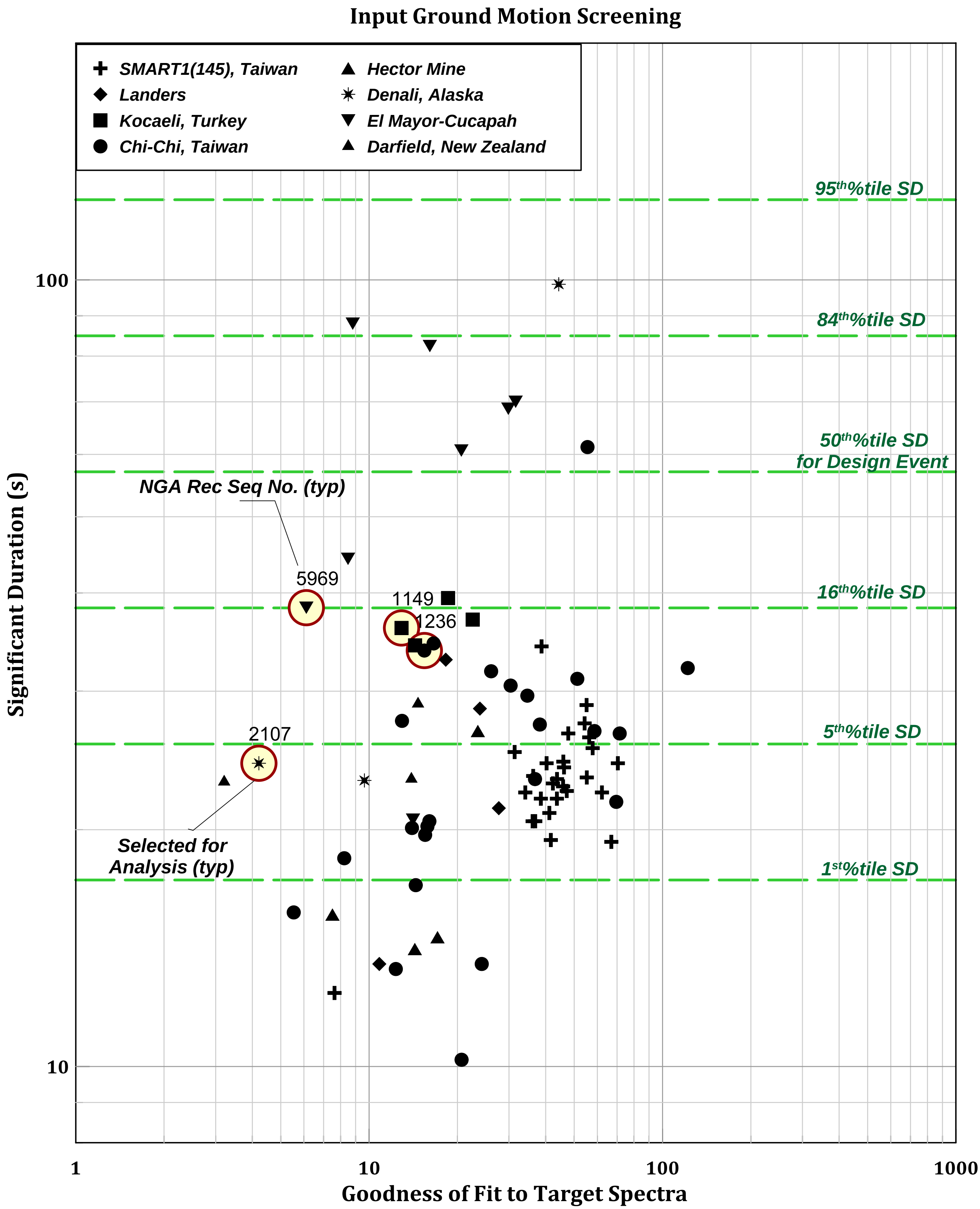
Project No.: 15083A	Project: 1201 S. GRAND AVENUE PROJECT	Date: MAY 2020	Figure C-14a

GeoPentech



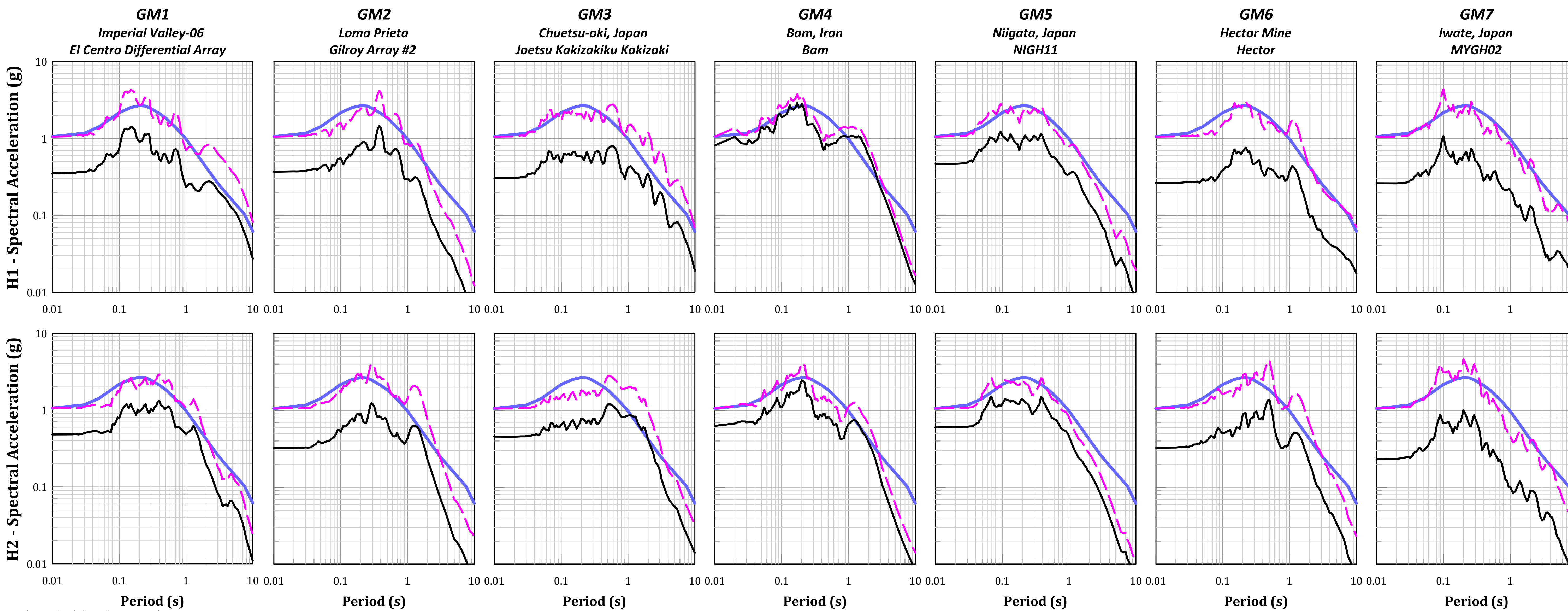
SIGNIFICANT DURATION AND GOODNESS OF FIT SCREENING FOR DESIGN LOCAL EVENT			
Project No.: 15083A	Project: 1201 S. GRAND AVENUE PROJECT	Date: MAY 2020	Figure C-14b

GeoPentech



SIGNIFICANT DURATION AND GOODNESS OF FIT SCREENING FOR DESIGN DISTANT EVENT			
Project No.: 15083A	Project: 1201 S. GRAND AVENUE PROJECT	Date: MAY 2020	Figure C-15

GeoPentech



Local Event Seed Time History Records

Analysis Record No.	Earthquake Name	Station Name	PEER NGA Record No.	H1 (deg)	H2 (deg)	Date	Earthquake Magnitude	Rupture Mechanism	Closest Distance (km)	NEHRP Site Class/V_{S30} (m/s)	PGA (g)	D_{a5-95} (sec)	T_L (sec)	T_P (sec)
GM1	Imperial Valley-06	El Centro Differential Array	184	270	0	10/15/1979	6.53	SS	5.09	D / 202	0.437	7.0	34.78	6.265
GM2	Loma Prieta	Gilroy Array #2	766	0	90	10/18/1989	6.93	RV/OBL	11.07	D / 271	0.357	11.0	13.33	1.729
GM3	Chuetsu-oki, Japan	Joetsu Kakizakiku Kakizaki	4847	0	90	07/16/2007	6.8	RV	11.94	C / 383	0.424	20.3	11.43	1.4
GM4	Bam, Iran	Bam	4040	278 (FP)	8 (FN)	12/26/2003	6.6	SS	1.7	C / 487	0.738	9.6	16.00	2.023
GM5	Niigata, Japan	NIGH11	4228	0	90	10/23/2004	6.63	RV	8.93	C / 375	0.508	12.2	20.00	1.799
GM6	Hector Mine	Hector	1787	0	90	10/16/1999	7.13	SS	11.66	C / 726	0.311	11.7	26.67	N/A
GM7	Iwate, Japan	MYGH02	5678	0	90	06/13/2008	6.9	RV	11.1	D / 399	0.252	9.5	26.67	N/A

Target Site Specific MCE_R Spectrum - RotD100 Component
As-Recorded TH
As-Recorded TH, Scaled to Target PGA

Earthquake Characteristic Key

Earthquake Name	= The common name of earthquake; usually includes the name of the general area or country where earthquake occurred.
Station Name	= The unique name of strong-motion station.
PEER NGA Record No.	= An arbitrary unique number assigned to each strong-motion record in the NGA database for identification purposes.
H1	= The orientation of the H1 component, if orientation is within 5 degrees of fault normal or fault parallel, denoted with (FN) or (FP).
H2	= The orientation of the H2 component, if orientation is within 5 degrees of fault normal or fault parallel, denoted with (FN) or (FP).
Date	= Date of earthquake.
Earthquake Magnitude	= Moment magnitude of earthquake.
Rupture Mechanism	= Mechanism based on rake angle, SS = Strike-slip, RV = Reverse, RV/OBL = Reverse-Oblique, NML = Normal.
Closest Distance	= Closest distance from the recording site to the ruptured area (km).
NEHRP Site Class/V_{S30}	= The preferred NEHRP site class determined based on the preferred VS30 values (m/s).
PGA	= Peak ground acceleration of the selected record (g).
D_{a5-95}	= Significant duration of the selected record as defined by the 5th to 95th percentile of Arias intensity (sec); geometric mean of significant duration from two components listed.
T_L	= Longest usable period, inverse of lowest usable frequency indicated by PEER; minimum of two components listed (sec).
T_P	= Pulse period of component of record with maximum peak-to-peak velocity (sec); N/A if no pulse is classified in record.

DESIGN LOCAL EVENT SEED TIME HISTORIES

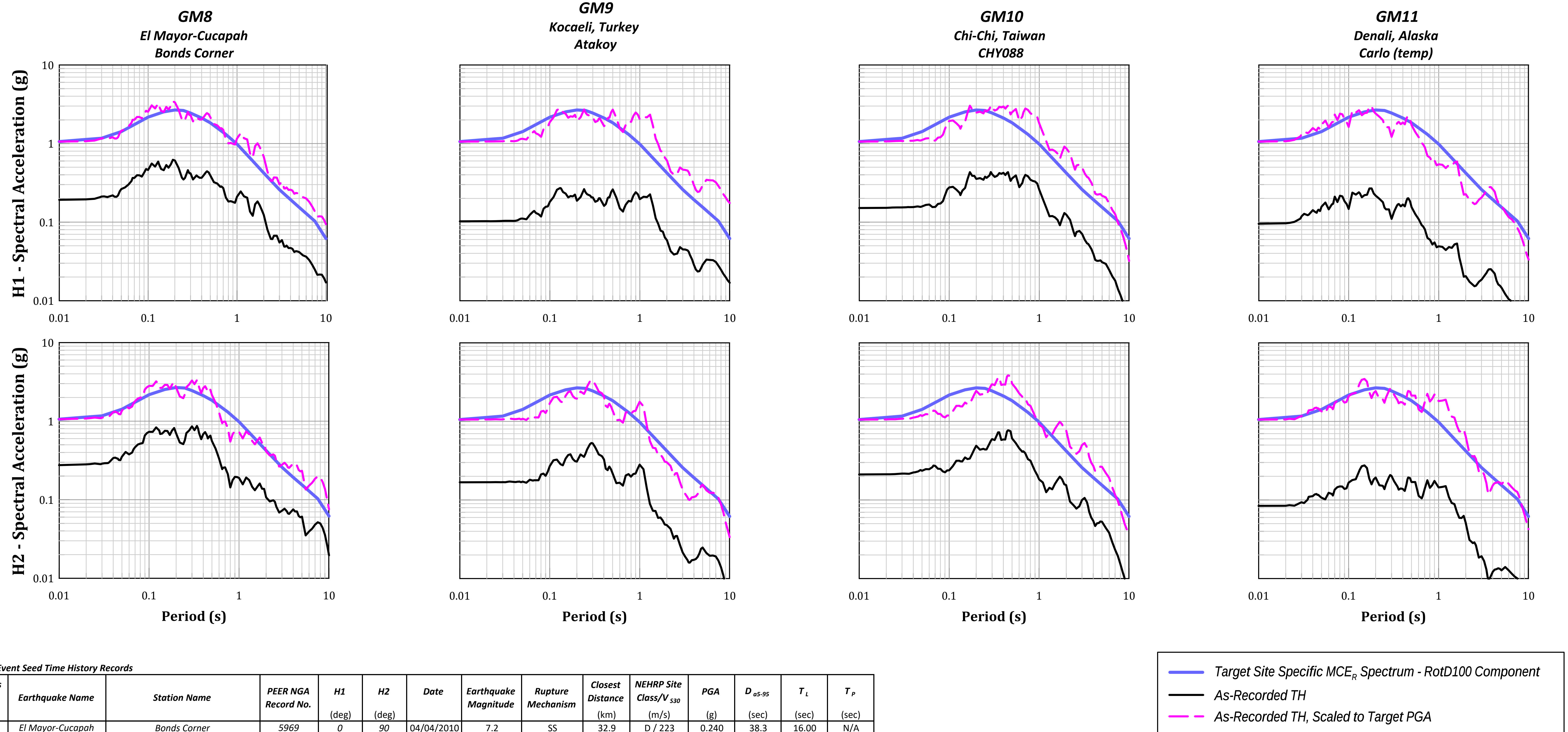
Project: 1201 S. GRAND AVENUE PROJECT

Project No.: 15083A

Date: MAY 2020

Figure C-16a

GeoPentech



Distant Event Seed Time History Records

Analysis Record No.	Earthquake Name	Station Name	PEER NGA Record No.	H1 (deg)	H2 (deg)	Date	Earthquake Magnitude	Rupture Mechanism	Closest Distance (km)	NEHRP Site Class/V_{S30} (m/s)	PGA (g)	D_{a5-95} (sec)	T_L (sec)	T_P (sec)
GM8	El Mayor-Cucapah	Bonds Corner	5969	0	90	04/04/2010	7.2	SS	32.9	D / 223	0.240	38.3	16.00	N/A
GM9	Kocaeli, Turkey	Atakoy	1149	0	90	08/17/1999	7.51	SS	58.3	D / 310	0.121	36.1	26.67	N/A
GM10	Chi-Chi, Taiwan	CHY088	1236	90	0	09/29/1999	7.62	RV/OBL	37.5	C / 319	0.184	33.8	20.00	N/A
GM11	Denali, Alaska	Carlo (temp)	2107	90	0	11/03/2002	7.9	SS	50.9	C / 399	0.090	24.3	12.82	N/A

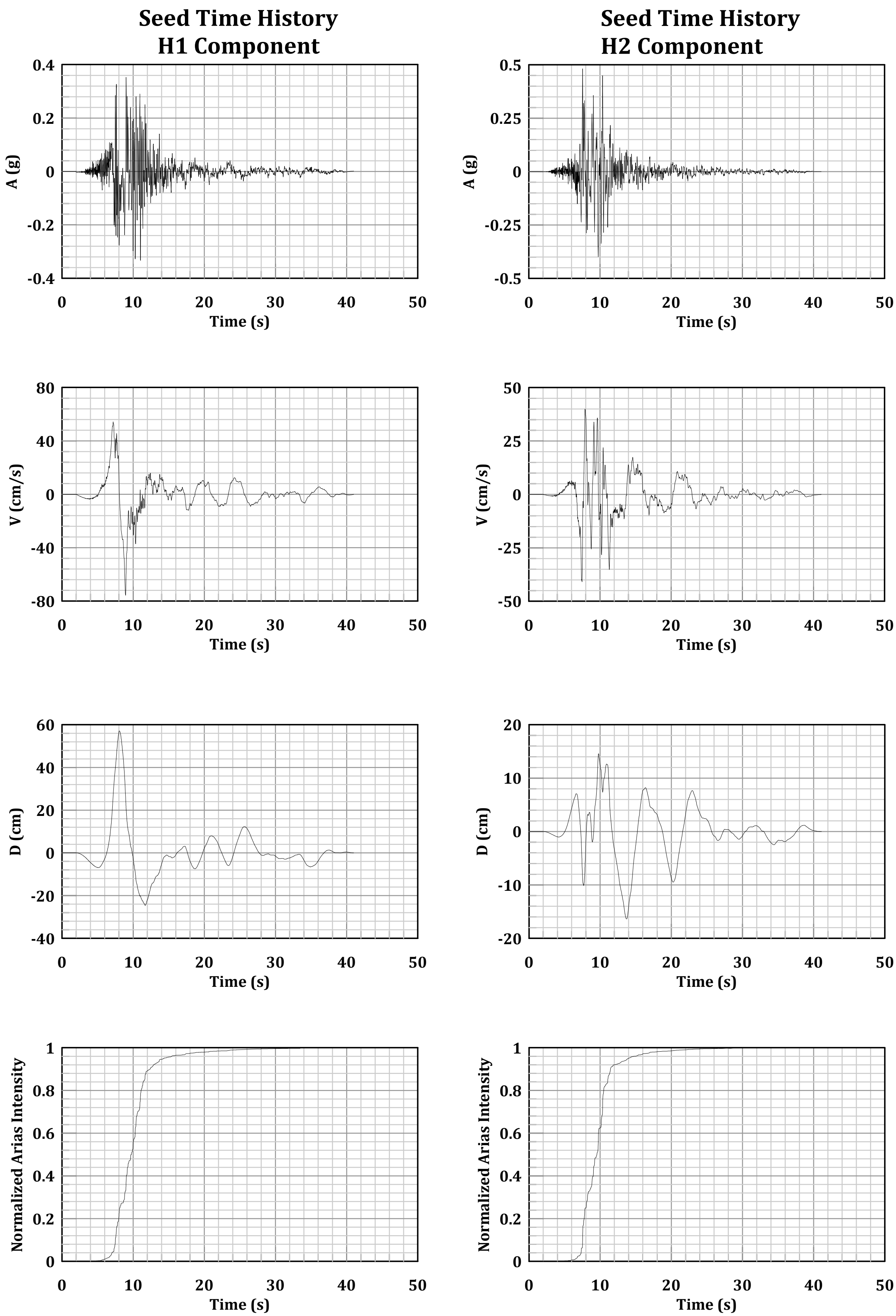
Earthquake Characteristic Key

Earthquake Name	= The common name of earthquake; usually includes the name of the general area or country where earthquake occurred.
Station Name	= The unique name of strong-motion station.
PEER NGA Record No.	= An arbitrary unique number assigned to each strong-motion record in the NGA database for identification purposes.
H1	= The orientation of the H1 component, if orientation is within 5 degrees of fault normal or fault parallel, denoted with (FN) or (FP).
H2	= The orientation of the H2 component, if orientation is within 5 degrees of fault normal or fault parallel, denoted with (FN) or (FP).
Date	= Date of earthquake.
Earthquake Magnitude	= Moment magnitude of earthquake.
Rupture Mechanism	= Mechanism based on rake angle, SS = Strike-slip, RV = Reverse, RV/OBL = Reverse-Oblique, NML = Normal.
Closest Distance	= Closest distance from the recording site to the ruptured area (km).
NEHRP Site Class/V_{S30}	= The preferred NEHRP site class determined based on the preferred VS30 values (m/s).
PGA	= Peak ground acceleration of the selected record (g).
D_{a5-95}	= Significant duration of the selected record as defined by the 5th to 95th percentile of Arias intensity (sec); geometric mean of significant duration from two components listed.
T_L	= Longest usable period, inverse of lowest usable frequency indicated by PEER; minimum of two components listed (sec).
T_P	= Pulse period of component of record with maximum peak-to-peak velocity (sec); N/A if no pulse is classified in record.

DESIGN DISTANT EVENT SEED TIME HISTORIES

Project: 1201 S. GRAND AVENUE PROJECT		Figure C-16b
Project No.: 15083A	Date: MAY 2020	

GeoPentech



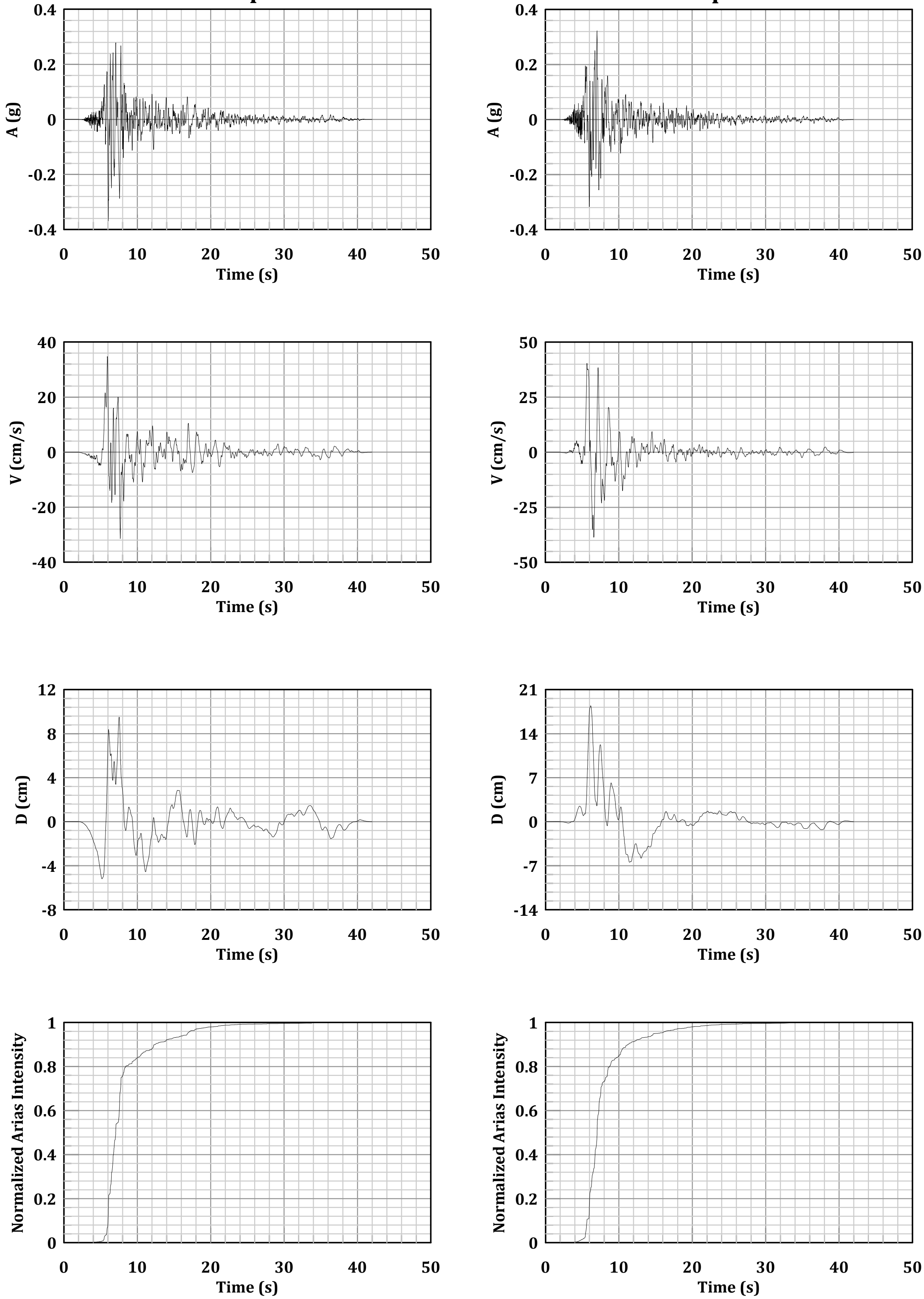
Seed GM1: Imperial Valley-06, El Centro Differential Array

Project No.: 15083A	Project: 1201 S. GRAND AVENUE PROJECT	Date: MAY 2020	Figure C-17

GeoPentech

Seed Time History
H1 Component

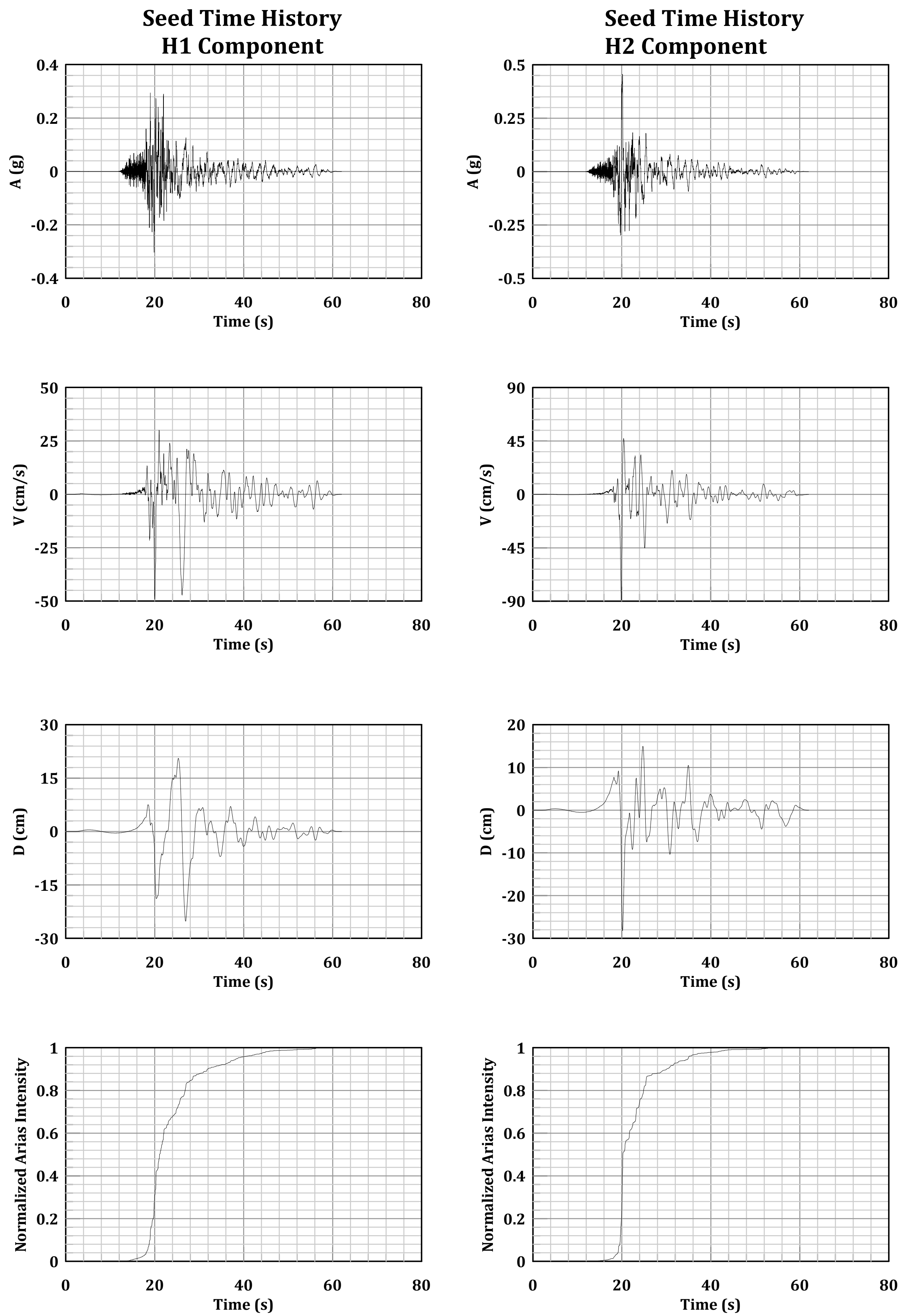
Seed Time History
H2 Component



Seed GM2: Loma Prieta, Gilroy Array #2

Project No.: 15083A	Project: 1201 S. GRAND AVENUE PROJECT	Date: MAY 2020	Figure C-18

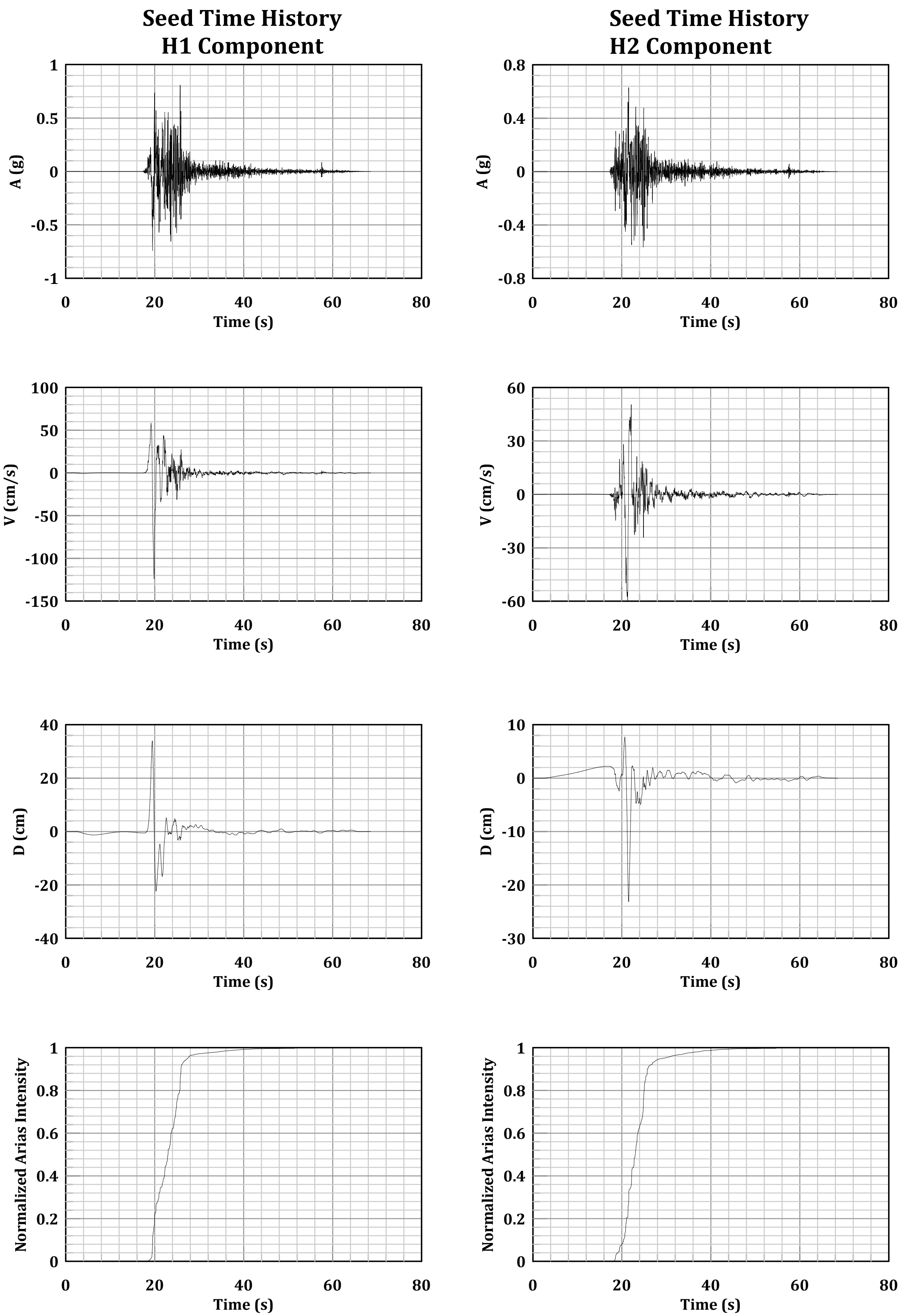
GeoPentech



Seed GM3: Chuetsu-oki - Japan, Joetsu Kakizakiku Kakizaki

Project No.: 15083A	Project: 1201 S. GRAND AVENUE PROJECT	Date: MAY 2020	Figure C-19

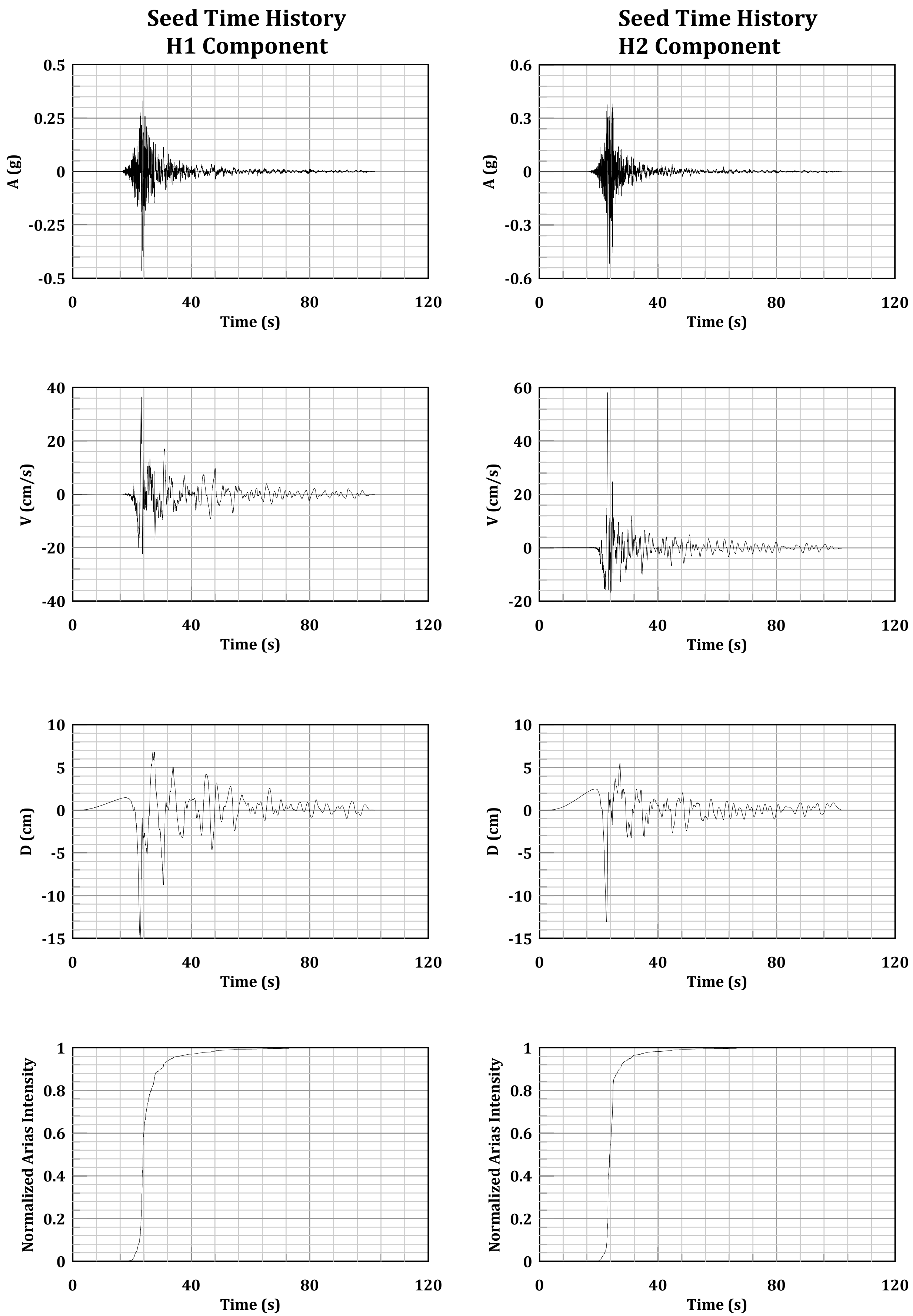
GeoPentech



Seed GM4: Bam - Iran, Bam

Project No.: 15083A	Project: 1201 S. GRAND AVENUE PROJECT	Date: MAY 2020	Figure C-20

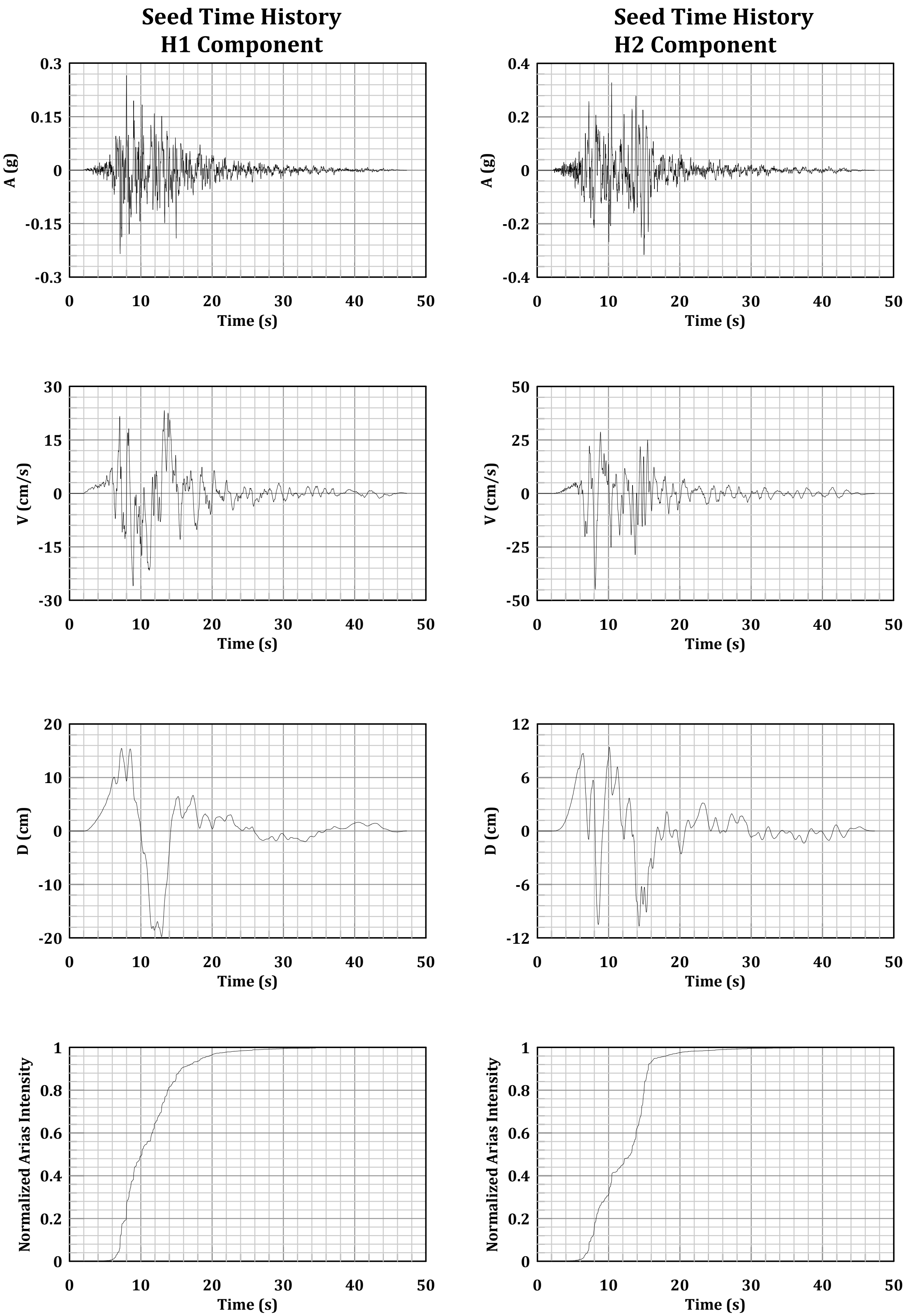
GeoPentech



Seed GM5: Niigata - Japan, NIGH11

Project No.: 15083A	Project: 1201 S. GRAND AVENUE PROJECT	Date: MAY 2020	Figure C-21

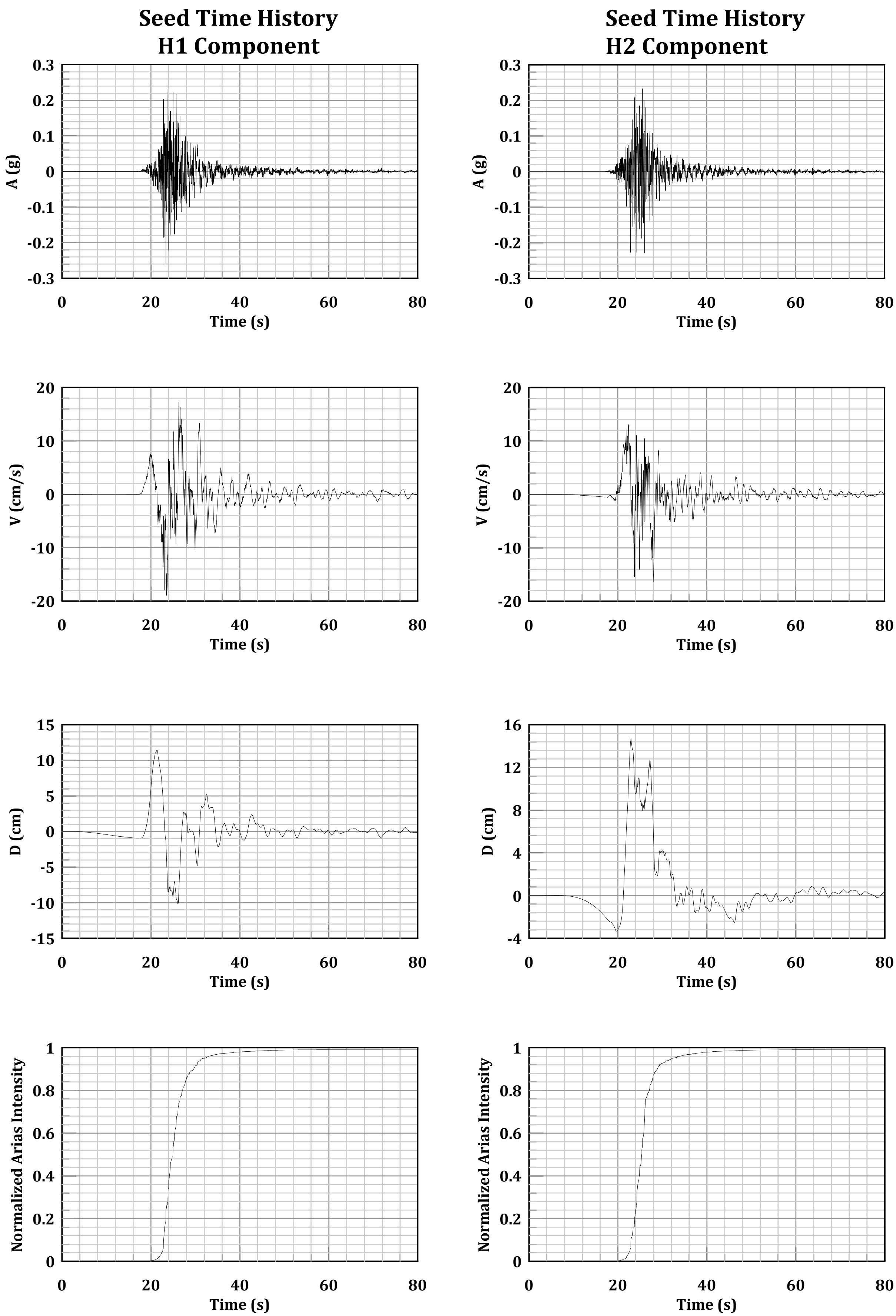
GeoPentech



Seed GM6: Hector Mine, Hector

Project No.: 15083A	Project: 1201 S. GRAND AVENUE PROJECT	Date: MAY 2020	Figure C-22

GeoPentech



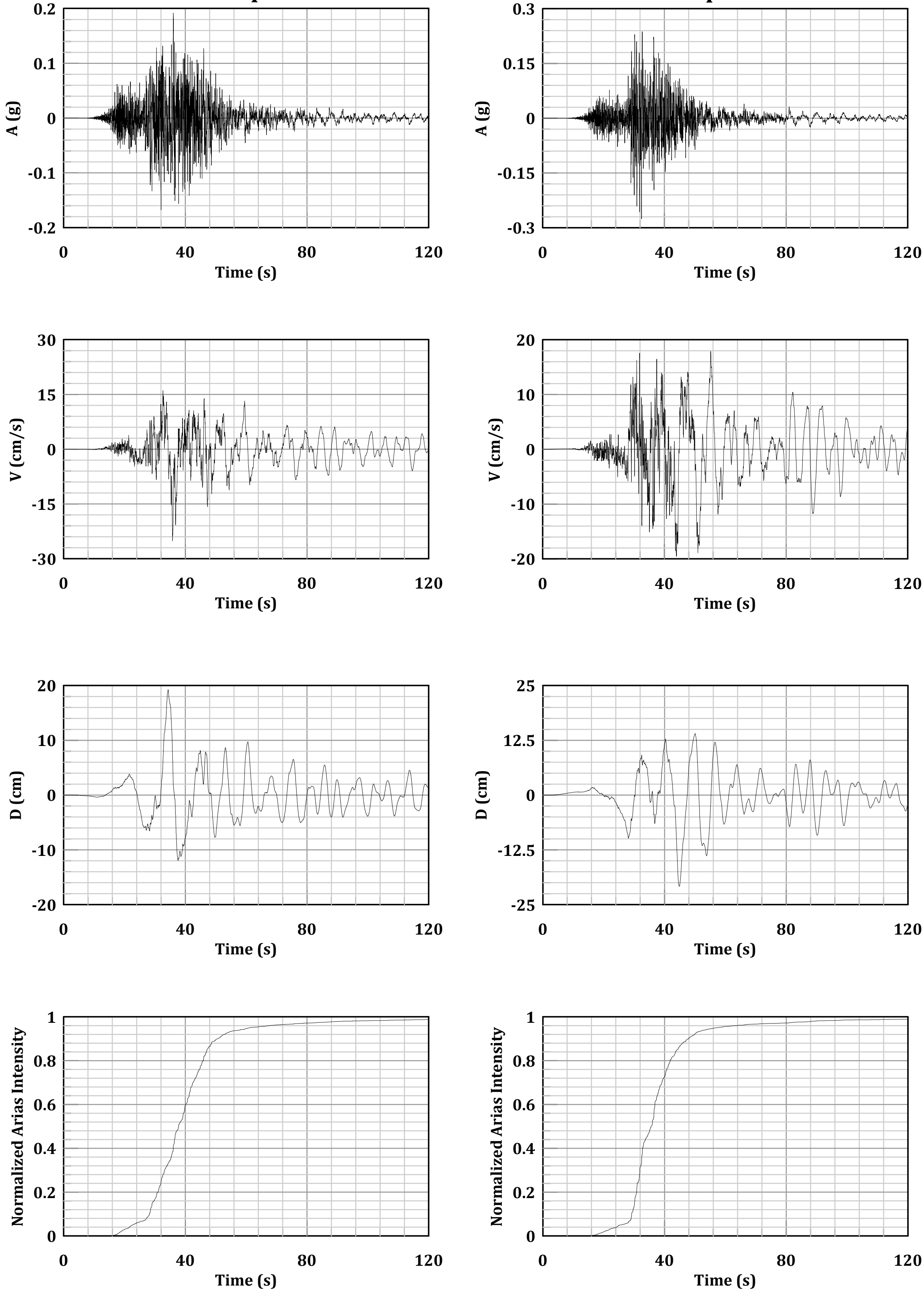
Seed GM7: Iwate - Japan, MYGH02

Project No.: 15083A	Project: 1201 S. GRAND AVENUE PROJECT	Date: MAY 2020	Figure C-23

GeoPentech

Seed Time History H1 Component

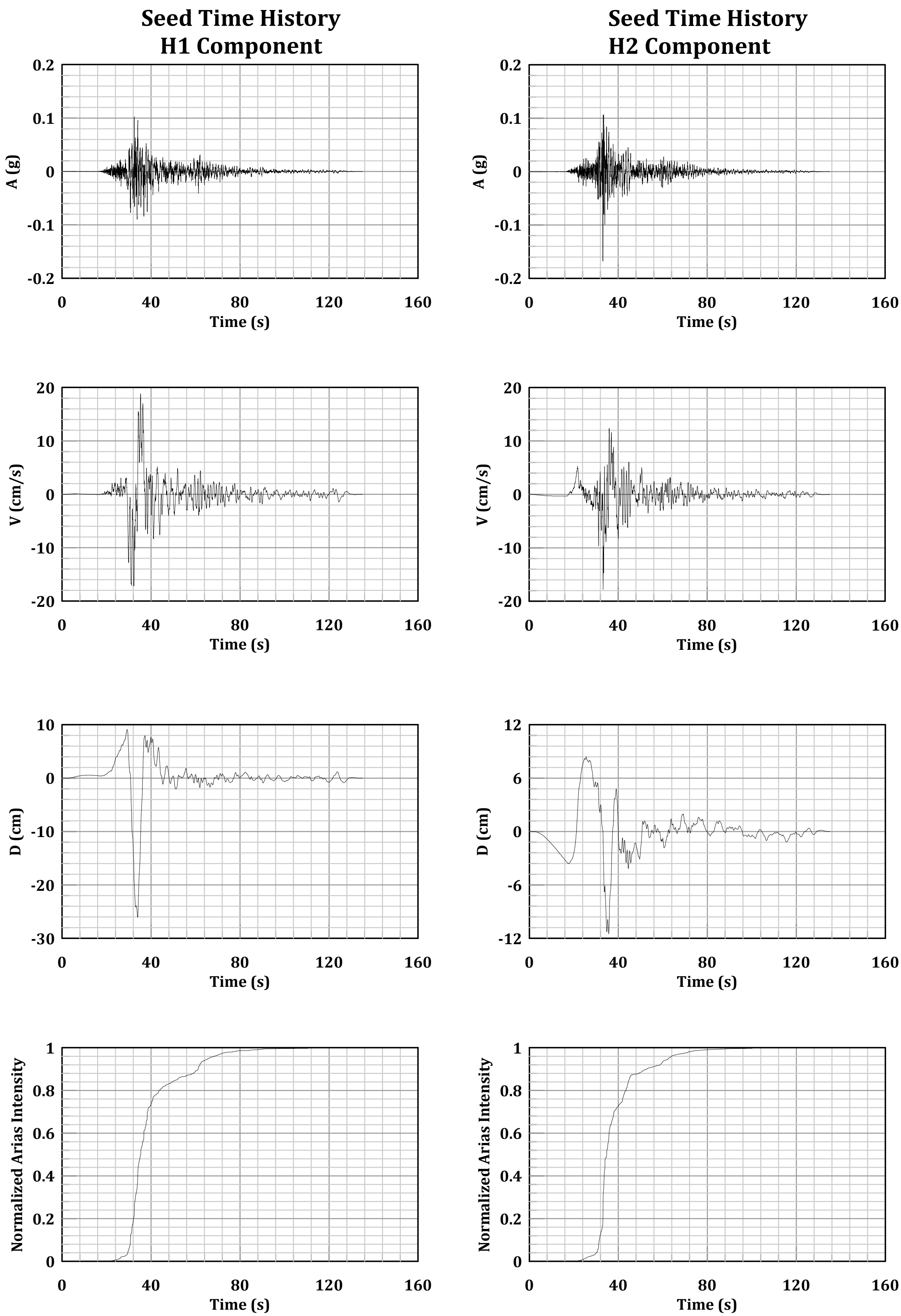
Seed Time History H2 Component



Seed GM8: El Mayor-Cucapah, Bonds Corner

Project No.: 15083A	Project: 1201 S. GRAND AVENUE PROJECT	Date: MAY 2020	Figure C-24

GeoPentech

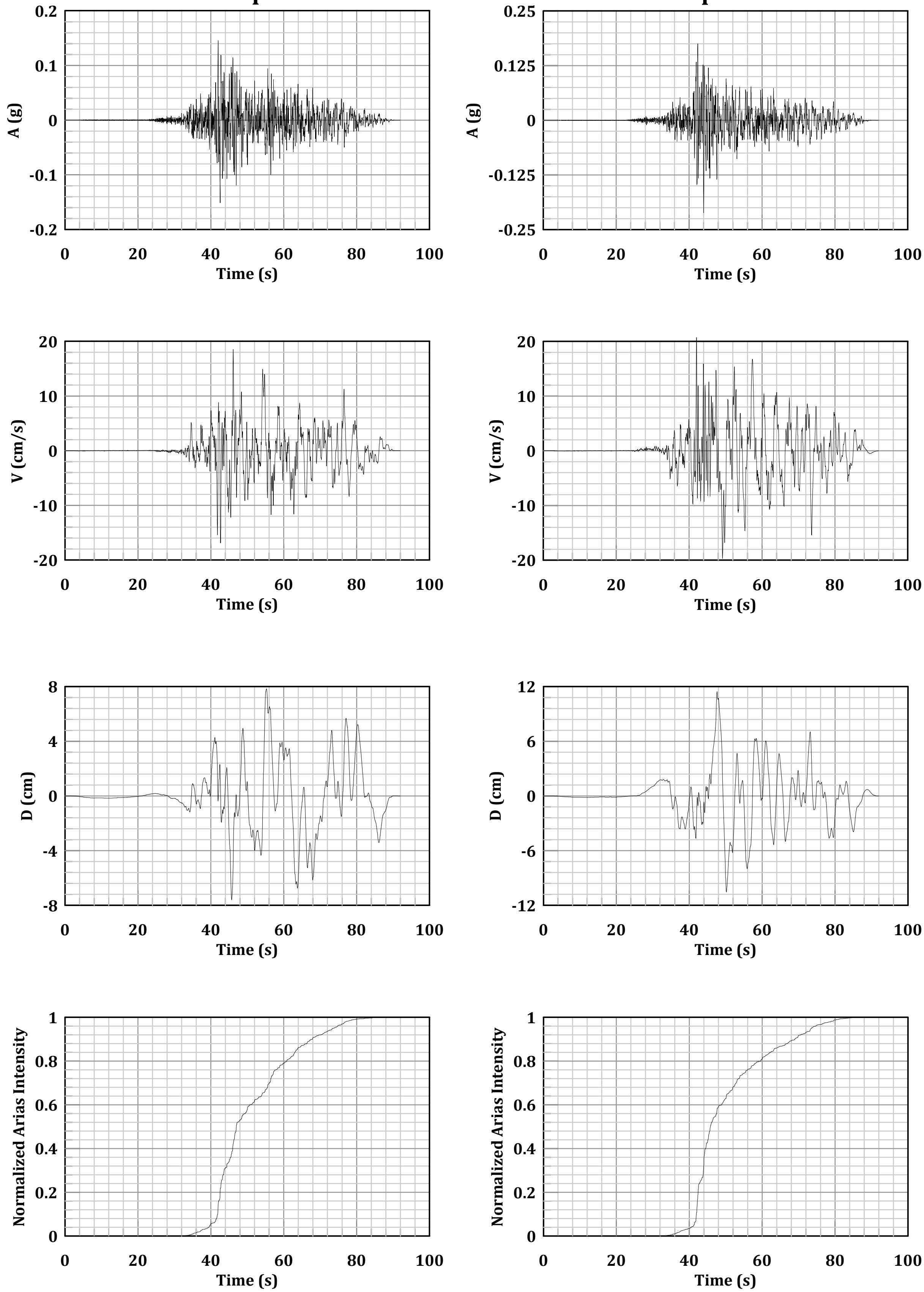


Seed GM9: Kocaeli - Turkey, Atakoy

Project No.: 15083A	Project: 1201 S. GRAND AVENUE PROJECT	Date: MAY 2020	Figure C-25

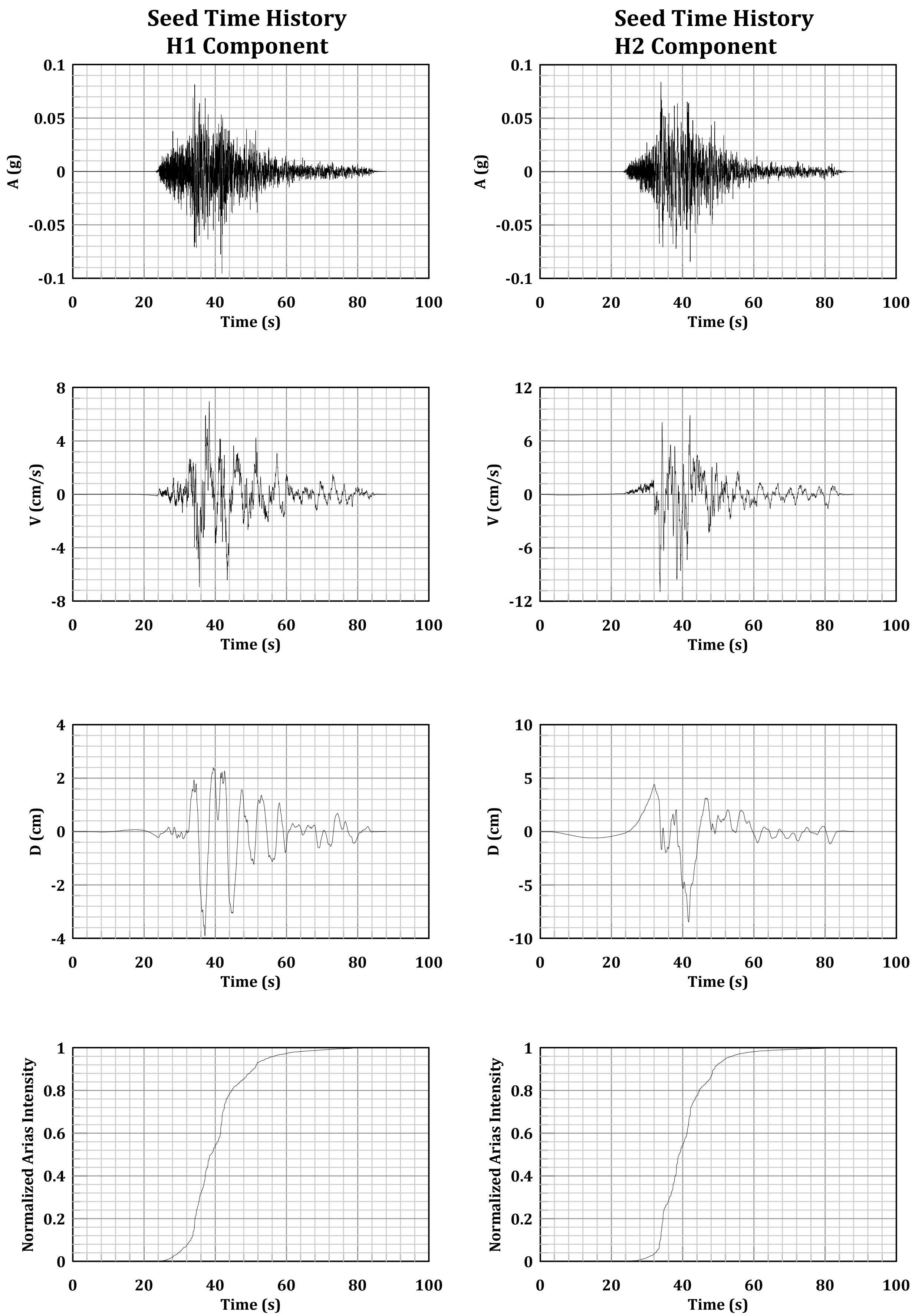
GeoPentech

Seed Time History H1 Component | **Seed Time History H2 Component**



Seed GM10: Chi-Chi - Taiwan, CHY088			
Project No.: 15083A	Project: 1201 S. GRAND AVENUE PROJECT	Date: MAY 2020	Figure C-26

GeoPentech



Seed GM11: Denali - Alaska, Carlo (temp)

Project No.: 15083A	Project: 1201 S. GRAND AVENUE PROJECT	Date: MAY 2020	Figure C-27

GeoPentech