Appendix IS-3

Geotechnical Engineering Investigation

Geotechnologies, Inc.
Consulting Geotechnical Engineers

439 Western Avenue
Glendale, California 91201-2837
818.240.9600 • Fax 818.240.9675

March 19, 2018
Revised March 19, 2020
File Number 21194

Chait Company Architects
7306 Coldwater Canyon Avenue, Unit 12
North Hollywood, California 91605

Attention: Michael Chait

Subject: Geotechnical Engineering Investigation
Proposed Office Building
12575 Beatrice Street, Los Angeles, California

Ladies and Gentlemen:

This letter transmits the Geotechnical Engineering Investigation for the subject property prepared by Geotechnologies, Inc. This report provides geotechnical recommendations for the development of the site, including earthwork, seismic design, retaining walls, excavations, and foundation design.

A Preliminary Geotechnical Engineering Investigation report was previously prepared by this firm on April 4, 2016. The preliminary report was prepared based on the results of four Cone Penetration Test Soundings (CPTs) performed at the subject site. The preliminary report was submitted to the LADBS Grading Division for review. Subsequently, the LADBS Grading Division prepared a Soils Report Review Letter (Log # 97201), dated March 23, 2017, requesting a comprehensive investigation be performed at the subject site, which would include geotechnical borings, laboratory testing, liquefaction analysis, and foundation analysis. A copy of the review letter by the Grading Division is included in the Appendix of this report for reference.

In order to comply with the LADBS requirements, three geotechnical borings and laboratory testing were performed as part of this current investigation. The results of the prior CPT analyses have been incorporated into the finding and analyses of this report. The recommendations presented in this report shall supersede those presented previously in the Preliminary Geotechnical Engineering Investigation report. The subsurface conditions described herein have been projected from subsurface exploration and laboratory testing. The exploration and testing presented in this report should in no way be construed to reflect any variations which may occur between the exploration locations or which may result from changes in subsurface conditions.

Should you have any questions please contact this office.

Respectfully submitted,
GEOTECHNOLOGIES, INC.

REGISTERED PROFESSIONAL ENGINEER
STANLEY S'U KEI TANG
No 56178
Exp 12/31/20
CIVIL
STATE OF CALIFORNIA

STANLEY S. TANG
R.C.E. 56178

Distribution: (2) Addressee
(2) City of Los Angeles

Email to: [michael@chaitco.com]

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GEOTECHNICAL ENGINEERING INVESTIGATION
PROPOSED OFFICE BUILDING
12575 BEATRICE STREET
LOS ANGELES, CALIFORNIA

INTRODUCTION

This report presents the results of the geotechnical engineering investigation performed on the subject property. The purpose of this investigation was to identify the distribution and engineering properties of the earth materials underlying the site, and to provide geotechnical recommendations for the design of the proposed development.

A Preliminary Geotechnical Engineering Investigation report was previously prepared by this firm on April 4, 2016. The preliminary report was prepared based on the results of four Cone Penetration Test Soundings (CPTs) performed at the subject site. The preliminary report was submitted to the LADBS Grading Division for review. Subsequently, the LADBS Grading Division prepared a Soils Report Review Letter (Log # 97201), dated March 23, 2017, requesting a comprehensive investigation be performed at the subject site, which would include geotechnical borings, laboratory testing, liquefaction analysis, and foundation analysis. A copy of the review letter by the Grading Division is included in the Appendix of this report for reference.

In order to comply with the LADBS requirements, three geotechnical borings and laboratory testing were performed as part of this current investigation. The results of the prior CPT analyses have been incorporated into the finding and analyses of this report. The recommendations presented in this report shall supersede those presented previously in the Preliminary Geotechnical Engineering Investigation report.

PROPOSED DEVELOPMENT

Information concerning the proposed development was furnished by the client. The site is proposed to be developed with a 4 to 5-story office building, which will be constructed over 3 above grade parking levels and 2 subterranean parking levels. It is anticipated that the proposed subterranean levels will extend on the order of 20 feet below the existing site grade. Based on the latest design plans, the finished floor elevation at the ground floor level will be at approximately 23.0 feet above Mean Sea Level (MSL), and the finished floor elevation of the B2 subterranean parking level will be at approximately 4.0 feet above MSL.

Maximum column gravity loads are estimated to be on the order of 1,700 kips. Maximum wall gravity loads are estimated to be between 24 kips per lineal foot. It is anticipated that excavation on the order of 25 feet will be required for the proposed subterranean levels and foundation elements.

Any changes in the design of the project or location of any structure, as outlined in this report, should be reviewed by this office. The recommendations contained in this report should not be considered valid until reviewed and modified or reaffirmed, in writing, subsequent to such review.

SITE CONDITIONS

The property is located at 12575 Beatrice Street, in the City of Los Angeles, California. The project site consists of a rectangular shaped lot, and is bounded by adjacent properties to the north and to the east, and by Beatrice Street to the south and by Jandy Place to the west. The site is currently developed with an existing office building, garage, and associated parking lots. The existing structures will be demolished as part of the proposed development.

Based on available survey by AJA Surveying, the current site elevation varies approximately between 22.0 and 27.0 feet above MSL. Drainage across the site is by sheetflow to the city streets. The vegetation on the site consists of isolated trees, and planters. The neighboring development consists primarily of commercial and residential structures.

GEOTECHNICAL EXPLORATION

FIELD EXPLORATION

The site was explored between March 17, 2016, and December 20, 2017, by excavating three exploratory borings, and performing four Cone Penetration Test Soundings (CPTs). The exploratory borings varied between 80 and 120 feet in depth below the existing site grade. The borings were excavated with the aid of a rotary wash drill rig, equipped with an automatic hammer, and using 5-inch diameter hollowstem augers. The exploration locations are shown on the Plot Plan and the geologic materials encountered are logged on Plates A-1 through A-3.

The CPT soundings were advanced to refusal, which generally occurred at depths between 53½ and 56¼ feet below the existing site grade. CPT-04 encountered refusal at a depth of 4 feet below the existing site grade, possibly due to buried utility lines. The CPT sounding locations are shown on the Plot Plan and interpretations of the geologic materials encountered are provided in the enclosed CPT Sounding Data Logs in the Appendix.

Geologic Materials

Fill materials underlying the subject site consist primarily of sandy to silty clays, with mixtures of sandy silts and silty sands. The fill materials are dark brown to dark gray in color, moist to very moist, medium firm to stiff, medium dense, fine grained. Fill thickness on the order of 12½ feet was encountered in the exploratory borings.

The upper native soils consist of stratified younger alluvial soil layers of silts, clays, silty sands, and gravelly sands. The upper native soils are dark brown to grayish brown in color, moist to very moist to wet, fine to coarse grained, with occasional gravel.

Older alluvium was generally encountered below a depth of 55 to 57½ feet below the existing site grade. The older alluvium consists of sands to gravelly sands, which are gray to dark gray in color, wet, dense to very dense, fine to coarse grained, with varying amount of gravel and cobbles. All of the CPT soundings, except for CPT-04, encountered refusal within the Older Alluvium. More detailed soil profiles may be obtained from individual boring and CPT logs presented in the Appendix of this report.

Groundwater

Groundwater was encountered at depths between 22½ and 30 feet below the existing site grade during exploration. Review of the Hazard Zone Report of the Venice 7½-Minute Quadrangle (CDMG, 1998, Revised 2006) indicates the historic high groundwater level for the subject site was approximately 7 feet below the ground surface.

It should be noted that the site elevations for this vicinity of Playa Del Rey had been raised by past grading activities. It has been the policy of the Los Angeles Department of Building and Safety (LADBS) to establish the historic high groundwater surface elevation in the Playa Vista area to be at an elevation of 9.0 feet above MSL, which corresponds to an approximate depth of 15 feet below the existing ground surface.

Fluctuations in the level of groundwater may occur due to variations in rainfall, temperature, and other factors not evident at the time of the measurements reported herein. Fluctuations also may occur across the site. High groundwater levels can result in changed conditions.

SEISMIC EVALUATION

REGIONAL GEOLOGIC SETTING

The subject property is located in the northern portion of the Peninsular Ranges Geomorphic Province. The Peninsular Ranges are characterized by northwest-trending blocks of mountain ridges and sediment-floored valleys. The dominant geologic structural features are northwest trending fault zones that either die out to the northwest or terminate at east-trending reverse faults that form the southern margin of the Transverse Ranges.

REGIONAL FAULTING

Based on criteria established by the California Division of Mines and Geology (CDMG) now called California Geologic Survey (CGS), faults may be categorized as active, potentially active, or inactive. Active faults are those which show evidence of surface displacement within the last 11,000 years (Holocene-age). Potentially-active faults are those that show evidence of most recent surface displacement within the last 1.6 million years (Quaternary-age). Faults showing no evidence of surface displacement within the last 1.6 million years are considered inactive for most purposes, with the exception of design of some critical structures.

Buried thrust faults are faults without a surface expression but are a significant source of seismic activity. They are typically broadly defined based on the analysis of seismic wave recordings of hundreds of small and large earthquakes in the southern California area. Due to the buried nature of these thrust faults, their existence is usually not known until they produce an earthquake. The risk for surface rupture potential of these buried thrust faults is inferred to be low (Leighton, 1990). However, the seismic risk of these buried structures in terms of recurrence and maximum potential magnitude is not well established. Therefore, the potential for surface rupture on these surface-verging splays at magnitudes higher than 6.0 cannot be precluded.

SEISMIC HAZARDS AND DESIGN CONSIDERATIONS

The primary geologic hazard at the site is moderate to strong ground motion (acceleration) caused by an earthquake on any of the local or regional faults. The potential for other earthquake-induced hazards was also evaluated including surface rupture, liquefaction, dynamic settlement, inundation and landsliding.

Surface Rupture

In 1972, the Alquist-Priolo Special Studies Zones Act (now known as the Alquist-Priolo Earthquake Fault Zoning Act) was passed into law. The Act defines "active" and "potentially active" faults utilizing the same aging criteria as that used by California Geological Survey (CGS). However, established state policy has been to zone only those faults which have direct evidence of movement within the last 11,000 years. It is this recency of fault movement that the CGS considers as a characteristic for faults that have a relatively high potential for ground rupture in the future.

CGS policy is to delineate a boundary from 200 to 500 feet wide on each side of the known fault trace based on the location precision, the complexity, or the regional significance of the fault. If a site lies within an Earthquake Fault Zone, a geologic fault rupture investigation must be performed that demonstrates that the proposed building site is not threatened by surface displacement from the fault before development permits may be issued.

Ground rupture is defined as surface displacement which occurs along the surface trace of the causative fault during an earthquake. Based on research of available literature and results of site reconnaissance, no known active faults or potentially active faults underlie the subject site. In addition, the subject site is not located within an Alquist-Priolo Earthquake Fault Zone. Based on these considerations, the potential for surface ground rupture at the subject site is considered low.

Liquefaction

Liquefaction is a phenomenon in which saturated silty to cohesionless soils below the groundwater table are subject to a temporary loss of strength due to the buildup of excess pore pressure during cyclic loading conditions such as those induced by an earthquake. Liquefaction-related effects include loss of bearing strength, amplified ground oscillations, lateral spreading, and flow failures.

The Seismic Hazards Maps of the State of California (CDMG, 1999), classifies the site as part of the potentially "Liquefiable" area. This determination is based on groundwater depth records, soil type and distance to a fault capable of producing a substantial earthquake.

Site-specific liquefaction analyses were performed following the Recommended Procedures for Implementation of the California Geologic Survey Special Publication 117A, Guidelines for Analyzing and Mitigating Seismic Hazards in California (CGS, 2008), and the EERI Monograph (MNO-12) by Idriss and Boulanger (2008).

Liquefaction analyses were performed utilizing the Standard Penetration Test data and the laboratory testing of the soils samples collected from the exploratory borings, and supplemented by the Cone Penetration Test (CPT) soundings data. CPT Sounding Number 1 was performed adjacent to Boring Number 2 for the purpose of comparison and correlation of soil data.

The enclosed SPT liquefaction analyses were performed using a spreadsheet developed based on Idriss and Boulanger (2008). This semi-empirical method is based on a correlation between measured values of Standard Penetration Test (SPT) resistance and field performance data.

The Cone Penetration Test data was analyzed utilizing a spreadsheet program developed based on the published article, "Evaluating Cyclic Liquefaction Potential Using the Cone Penetration Test" (P.K. Robertson and C.E. Wride, 1998), to estimate the grain size characteristics directly from the CPT data and to incorporate the interpreted results into evaluating the resistance to cyclic loading.

The enclosed liquefaction analyses were analyzed using the modal magnitude and peak ground motion for the project site. A modal magnitude (M_W) of 6.7 is obtained using the USGS Probabilistic Seismic Hazard Deaggregation program (USGS, 2014).

Downhole seismic velocity measurement was performed by GeoPentech within boring B1, which was excavated to a depth of 120 feet below the existing site grade. According to the seismic survey, an average shearwave velocity (V_{S30}) of 850 feet/second was measured between 0 and 100 feet. This shearwave velocity measurement corresponds to a site classification for seismic design of Site Class D ($600 < V_{S30} < 1{,}200$ feet/sec). Using the ASCE 7 Hazard Tool website (https://asce7hazardtool.online/), a code-based peak ground acceleration (PGA_M) of 0.88g was obtained.

A Site-Specific Ground Motion Development Report was prepared by GeoPentech. The ground motion report indicated that the site-specific ground surface MCE_R spectral acceleration of 0.806g at a period of 0.01-second may be used in lieu of the code-based value for the purpose of liquefaction evaluation. However, for the purpose of conservatism, this firm has elected to use the higher PGA_M of 0.88g, which was obtained from the ASCE 7 Hazard Tool, for the enclosed liquefaction evaluation.

It has been the policy of the Los Angeles Department of Building and Safety (LADBS) to establish the historic high water surface elevation in the Playa Vista area to be at an elevation of 9.0 feet above MSL, which corresponds to an approximate depth of 15 feet below the existing ground surface. This historically highest groundwater level was conservatively utilized for the enclosed liquefaction analyses.

The enclosed SPT liquefaction analyses were performed based on blowcount data collected from the three exploratory borings, B1 through B3. Standard Penetration Test (SPT) data were collected at 5-foot intervals for all three borings. Alternating California Modified Ring Samples were collected in between the SPT data in order to collect relatively undisturbed soil samples for testing and analyses. Samples of the collected materials were conveyed to the laboratory for testing and analysis. Fines content, as defined by percentage passing the #200 sieve, were utilized for the fines correction factor in computing the corrected blowcount. In addition, Atterberg Limit tests were performed for the underlying samples and the results are presented in Plates F-1 through F-3 of this report.

According to the SP117A (which referenced papers by Bray and Sancio, 2006), soils having a Plastic Index greater than 18, or a moisture content not greater than 80% of the liquid limit, are considered to be not susceptible to liquefaction. Therefore, where the results of Atterberg Limits testing showed a Plastic Index greater than 18, the soils would be considered non-liquefiable, and the analysis of these clayey soil layers was turned off in the liquefaction susceptibility column.

Both the SPT and CPT liquefaction analyses indicate that the underlying soils would be liquefiable under the MCE ground motions.

Dynamic Settlement

Seismically-induced settlement can be an effect related to earthquake ground motion. Such settlements are typically most damaging when the settlements are differential in nature across the length of structures. Total seismic-induced liquefaction settlement, between 1.09 inches to 3.77 inches, is anticipated to occur as a result of liquefaction. The following table presents the results of the liquefaction settlement obtained from the analyses.

Exploration Point	Liquefiable Zones (generalized profile)	Total Liquefaction Settlement (inches)
B1	27.5'-37.5' 50'-57.5'	3.77"
B2	30'-32.5' 40'-45'	2.02"
B3	30'-32.5' 42.5'-47.5' 50'-55'	2.74"
CPT-01	27'-35' (Stratified Layers) 39'-42.5' (Stratified Layers) 43'-47.5' (Stratified Layers)	1.09"
CPT-02	16'-17' 29'-40' (Stratified Layers) 42'-46' 47'-48.5' 51'-55' (Stratified Layers)	2.33"
CPT-03	26.5'-27.5' 33'-33.5' 39'-41' 43.5'-53' (Stratified Layers)	1.40"

It should be noted, due to the inherent limitation of the borehole sampling methodology (which the SPT blowcount data were collected at 5-foot intervals), numerous thin, granular, liquefiable layers could be mischaracterized or missed by the sampling procedure. Reliance on the SPT blowcount data could also overestimate the thickness of the potentially liquefiable layers due to sampling frequency. One of the advantages of the Cone Penetration Test (CPT) is its repeatability and reliability, and its ability to provide a relatively continuous profiling of the underlying soils. The CPT method is extremely helpful especially in highly stratified soil conditions.

Surface Manifestation

It has been shown in recent studies by O'Rourke and Pease (1997) and Youd and Garris (1995), building upon work by Ishihara (1985), that the visible effects of liquefaction on the ground surface are only manifested if the relative and absolute thicknesses of liquefiable soils to overlying non-liquefiable surface material fall within a certain range. On the subject site, the relative thicknesses of liquefiable soils to overlying non-liquefiable surface material fall well outside the bounds within which surface effects of liquefaction have been observed during past earthquakes. As a result, the likelihood that surface effects of liquefaction would occur on the subject site would be considered very low to non-existent. Therefore, it is the opinion of Geotechnologies, Inc. that, should liquefaction occur within the potentially liquefiable zones, there would be a negligible effect on the proposed structures.

Lateral Spreading

Lateral spreading is the most pervasive type of liquefaction-induced ground failure. During lateral spread, blocks of mostly intact, surficial soil displace downslope or towards a free face along a shear zone that has formed within the liquefied sediment. According to the procedure provided by Bartlett, Hansen, and Youd, "Revised Multilinear Regression Equations for Prediction of Lateral Spread Displacement", ASCE, Journal of Geotechnical Engineering, Vol. 128, No. 12, December 2002, when the saturated cohesionless sediments with $(N_1)_{60} > 15$, significant displacement is not likely for $M < 8$ earthquakes.

The saturated cohesionless sediments underlying the subject site have corrected $(N_1)_{60}$ value greater than 15. The modal earthquake magnitude which contributes the majority of the ground motion to the site is 6.7. Therefore, the potential for lateral spread is considered to be remote for the subject site.

Tsunamis, Seiches and Flooding

Tsunamis are large ocean waves generated by sudden water displacement caused by a submarine earthquake, landslide, or volcanic eruption. Review of the County of Los Angeles Flood and

Inundation Hazards Map, Leighton (1990), indicates the site does not lie within the mapped tsunami inundation boundaries.

Seiches are oscillations generated in enclosed bodies of water which can be caused by ground shaking associated with an earthquake. No major water-retaining structures are located immediately up gradient from the project site. Therefore, the risk of flooding from a seismically-induced seiche is considered to be remote.

According to the County of Los Angeles General Plan (Leighton, 1990), the site is located within the potential inundation boundaries of several upgradient reservoirs, should any of the dams retaining these reservoirs fail during a major earthquake. A determination of whether a higher site elevation would remove the site from the potential inundation zones is beyond the scope of this investigation.

Landsliding

The probability of seismically-induced landslides occurring on the site is considered to be low due to the general lack of elevation difference slope geometry across or adjacent to the site.

CONCLUSIONS AND RECOMMENDATIONS

Based upon the exploration, laboratory testing, and research, it is the finding of Geotechnologies, Inc. that construction of the proposed structure is considered feasible from a geotechnical engineering standpoint provided the advice and recommendations presented herein are followed and implemented during construction.

On the order of 12½ feet of existing fill materials was encountered in the exploratory borings. Due to the highly variable nature of the underlying fill materials, the existing fill are considered to be unsuitable for support of the proposed foundations, floor slabs, or additional fill.

Groundwater was encountered at depths between 22½ and 30 feet below the existing site grade during exploration. The upper native soils consist of younger alluvial deposits to approximate depths between 55 and 57½ feet below the existing site grade. The younger alluvial deposits comprise primarily of highly expansive clay soils with stratified layers of medium dense silty sands to sands. Based on the enclosed liquefaction analyses, these thin granular younger alluvial deposits are potentially liquefiable during the MCE level ground motion with estimated total seismic settlement between 1.09 and 3.77 inches.

Very dense Older Alluvium, consisting of sands and gravelly sands, was encountered generally below a depth of 55 to 57½ feet below the existing site grade. Due to the liquefaction potential of the younger alluvial deposits, it is recommended that the proposed structure be supported on a pile foundation system bearing in the underlying Older Alluvium.

The use of driven pre-cast concrete piles for support of the proposed structures is not recommended due to noise and vibration concerns impacting the existing and neighboring developments. It is recommended that the proposed structure be supported on a system of Auger Cast Displacement Piles (ACDP). A summary of pile design recommendations is provided in the "Foundation Design" section below. No predrilling is allowed. The proposed floor slab shall be designed as a structural slab, deriving support entirely from the foundation piles.

Prior to installation of the production piles, an indicator test pile program must be performed. Indicator test pile program shall include additional CPT soundings, Gamma-Gamma tests (GDL), low strain Pile Integrity Tests (PIT), and static pile load tests. In addition, one test pile shall be exhumed to examine the pile integrity.

Based on available survey by AJA Surveying, the current site elevation varies approximately between 22.0 and 27.0 feet above MSL. It is the policy of the Los Angeles Department of Building and Safety (LADBS) for the historic high water surface elevation in the Playa Vista area to be at an elevation of 9.0 feet above MSL. Based on the latest design plans, the finished floor elevation of the B2 subterranean parking level will be at approximately 4.0 feet above MSL, which corresponds to 5 feet below the historically highest groundwater level. Since the lowest subterranean level will extend below above the historically highest groundwater level, it is recommended that the proposed structure be designed for hydrostatic pressure.

Due to the anticipated liquefaction potential, it is recommended that buried utilities and drain lines be equipped with flexible or swing joints to allow for differential vertical displacements.

The validity of the conclusions and design recommendations presented herein is dependent upon review of the geotechnical aspects of the proposed construction by this firm. The subsurface conditions described herein have been projected from explorations on the site as indicated and should in no way be construed to reflect any variations which may occur between these explorations or which may result from changes in subsurface conditions. Any changes in the design or location of any structure, as outlined in this report, should be reviewed by this office. The recommendations contained herein should not be considered valid until reviewed and modified or reaffirmed subsequent to such review.

SEISMIC DESIGN CONSIDERATIONS

Seismic Velocity Measurements

Downhole seismic velocity measurements were performed by GeoPentech within boring B1, which was excavated to a depth of 120 feet below the existing site grade. According to the seismic survey, an average shearwave velocity (V_{S30}) of 850 feet/second was measured between 0 and 100 feet. An average shearwave velocity (V_{S30}) of 950 feet/second was measured between

20 and 120 feet. These velocities correspond to a site classification for seismic design of Site Class D (600 < V_{S30} <1,200 feet/sec). A copy of the GeoPentech's Ground Motion Development Report, dated June 15, 2018, is presented in the Appendix of this report.

2019 California Building Code Seismic Parameters

According to Table 20.3-1 presented in ASCE 7-16, the subject site is classified as Site Class F due to the liquefiable nature of the underlying soils. According to Section 20.3.1 (site class definition for Site Class F) found in Chapter 20, titled "Site Classification Procedure for Seismic Design", ASCE 7-10, Minimum Design Loads for Buildings and Other Structures, an exception is provided under Site Classification F.

EXCEPTION: *For structures having fundamental periods of vibration equal to or less than 0.5 s, site-response analysis is not required to determine spectral accelerations for liquefiable soils. Rather, a site class is may be determined in accordance with Section 20.3 and the corresponding values of F_a and F_v determined from Tables 11.4-1 and 11.4-2. (This can be C, D or E)*

The following code based seismic parameters may be utilized for the design of structures with fundamental period of vibration equal to or less than 0.5 seconds. Due to the building period, it is likely that the code based design parameters will be superseded by the Site-Specific Design Response Spectrum analysis, however, the code based seismic parameters are presented herein for completeness. Based on the shearwave velocity measurement (V_{S30}), the subject site may be classified as Site Class D, which corresponds to a "Stiff Soil" Profile, in accordance with the ASCE 7 standard. This information and the site coordinates were input into the USGS U.S. Seismic Design Maps tool (Version 3.1.0) to calculate the ground motions for the site.

2019 CALIFORNIA BUILDING CODE SEISMIC PARAMETERS	
Site Class	D
Mapped Spectral Acceleration at Short Periods (S_S)	1.871g
Site Coefficient (F_a)	1.0
Maximum Considered Earthquake Spectral Response for Short Periods (S_{MS})	1.871g
Five-Percent Damped Design Spectral Response Acceleration at Short Periods (S_{DS})	1.247g
Mapped Spectral Acceleration at One-Second Period (S_1)	0.660g
Site Coefficient (F_v)	1.7*
Maximum Considered Earthquake Spectral Response for One-Second Period (S_{M1})	1.122g*
Five-Percent Damped Design Spectral Response Acceleration for One-Second Period (S_{D1})	0.748g*

ASCE 7-16 Site-Specific Design Response Spectrum Analysis

The structure's fundamental period of vibration is anticipated to be greater than 0.5 seconds (which will need to be confirmed by the project structural engineer). Therefore, a site-specific ground motion evaluation is required in conformance with the ASCE 7-16 and the 2019 California Building Code. A site-specific ground motion evaluation was completed by GeoPentech as part of this investigation. Tables 3 and 4 of the GeoPentech report provide the site-specific Surface MCE_R Spectrum and the Surface Design Response Spectrums (DRS). A more detailed discussion of the ground motion evaluation methodology and assumptions is provided in the Ground Motion Development Report by GeoPentech, dated March 18, 2020. A copy of GeoPentech's report is presented in the Appendix. Following the ASCE 7-16, Section 21.4, the site-specific design acceleration parameters are summarized in the following table.

- S_{DS} = 1.238 g, based on 90% of the spectral acceleration at a period of 0.3-seconds
- S_{D1} = 0.820 g, based on the site V_{S30} and T*Sa at a period of 1.5-second
- S_{MS} = 1.856 g, based on 1.5 times S_{DS}
- S_{M1} = 1.230 g, based on 1.5 times S_{D1}

SITE SPECIFIC DESIGN ACCELERATION PARAMETERS	
Seismic Parameters	ASCE 7-16 Site Specific Site Class D
S_{MS}	1.856g
S_{M1}	1.230g
S_{DS}	1.238g
S_{D1}	0.820g

In addition, a peak ground acceleration (PGA_M) of 0.806g was obtained from the site-specific spectral development by GeoPentech, which could be utilized for the enclosed liquefaction analyses. However, for the purpose of conservatism, this firm has elected to use the higher PGA_M of 0.88g, which was obtained from the ASCE 7 Hazard Tool, for the enclosed liquefaction evaluation.

FILL SOILS

On the order of 12½ feet of existing fill materials was encountered in the exploratory borings. Excavation of the proposed subterranean level will remove the existing fill materials from the project site. Due to the highly variable nature of the underlying fill materials, the existing fill are considered to be unsuitable for support of the proposed foundations, floor slabs, or additional fill. This material and any fill generated during demolition should be penetrated by the proposed pile foundation system.

EXPANSIVE SOILS

The onsite geologic materials are in the low to high expansion range. The Expansion Index was found to be between 35 and 95 for bulk samples remolded to 90 percent of the laboratory maximum density. Recommended reinforcing is noted in the “Slabs-on-Grade" section of this report.

SOIL CORROSION POTENTIAL

The results of soil corrosion potential testing performed by HDR, Inc. indicate that the electrical resistivities of the soils were in the moderately to severely corrosive categories in the as-received moisture conditions and at saturation. Soil pH values of the samples ranged between 7.4 and 7.6, indicating mildly alkaline condition. The soluble salt content of the samples ranged from moderate to high. Nitrate was detected in low concentrations. Ammonium concentration was high enough to be aggressive to copper. Sulfate content is considered negligible.

In summary, the soils are classified as severely corrosive to ferrous metals, aggressive to copper, and sulfate attack on concrete is negligible. Detailed results, discussion of results and recommended mitigating measures are provided within the report by HDR, Inc. resented herein. Any questions regarding the results of the soil corrosion report should be addressed to HDR, Inc.

METHANE ZONES

According to the NavigateLA website, the site is located within a Methane Zone as designated by the City. A qualified methane consultant should be retained to consider the requirements and implications of the City’s Methane Zone designation. A copy of the Methane Zone Map is enclosed herein.

GRADING GUIDELINES

The following grading guidelines may be utilized for any miscellaneous site grading which may be required as part of the proposed development.

Site Preparation

- A thorough search should be made for possible underground utilities and/or structures. Any existing or abandoned utilities or structures located within the footprint of the proposed grading should be removed or relocated as appropriate.

- All vegetation and soft or disturbed geologic materials should be removed from the areas to receive controlled fill. All existing fill materials and any disturbed geologic materials resulting from grading operations shall be completely removed and properly recompacted prior to foundation excavation.

- Any vegetation or associated root system located within the footprint of the proposed structures should be removed during grading.

- Subsequent to the indicated removals, the exposed grade shall be scarified to a depth of six inches, moistened to optimum moisture content, and recompacted in excess of the minimum required comparative density.

- The excavated areas shall be observed by the geotechnical engineer prior to placing compacted fill.

Compaction

The City of Los Angeles Department of Building and Safety requires a minimum 90 percent of the maximum density, except for cohesionless soils having less than 15 percent finer than 0.005 millimeters, which shall be compacted to a minimum 95 percent of the maximum density in accordance with the most recent revision of the Los Angeles Building Code.

All fill should be mechanically compacted in layers not more than 8 inches thick. All fill shall be compacted to at least 90 percent (or 95 percent for cohesionless soils having less than 15 percent finer than 0.005 millimeters) of the maximum laboratory density for the materials used. The maximum density shall be determined by the laboratory operated by Geotechnologies, Inc. using the test method described in the most recent revision of ASTM D 1557.

Field observation and testing shall be performed by a representative of the geotechnical engineer during grading to assist the contractor in obtaining the required degree of compaction and the proper moisture content. Where compaction is less than required, additional compactive effort shall be made with adjustment of the moisture content, as necessary, until a minimum of 90 percent (or 95 percent for cohesionless soils having less than 15 percent finer than 0.005 millimeters) compaction is obtained.

Acceptable Materials

The excavated onsite materials are considered satisfactory for reuse in the controlled fills as long as any debris and/or organic matter is removed. Any imported materials shall be observed and tested by the representative of the geotechnical engineer prior to use in fill areas. Imported materials should contain sufficient fines so as to be relatively impermeable and result in a stable subgrade when compacted. Any required import materials should consist of geologic materials with an expansion index of less than 90. The water-soluble sulfate content of the import materials should be less than 0.1% percentage by weight.

Imported materials should be free from chemical or organic substances which could affect the proposed development. A competent professional should be retained in order to test imported materials and address environmental issues and organic substances which might affect the proposed development.

Utility Trench Backfill

Utility trenches should be backfilled with controlled fill. The utility should be bedded with clean sands at least one foot over the crown. The remainder of the backfill may be onsite soil compacted to 90 percent (or 95 percent for cohesionless soils having less than 15 percent finer than 0.005 millimeters) of the laboratory maximum density. Utility trench backfill should be tested by representatives of this firm in accordance with the most recent revision of ASTM D-1557.

Wet Subgrade Soils

It is anticipated that the subgrade soils will be well above the optimum moisture content. Therefore, the excavated material to be placed as compacted fill, and the materials exposed at the bottom of excavated plane may require significant drying and aeration prior to recompaction.

The subgrade soils should be expected to be wet, soft, and prone to pumping under operation of construction equipment. The placement of a mat of crushed rock over the bottom of the excavations will most likely be necessary to stabilize and protect the subgrade soils from pumping under construction traffic and to create a firm working surface.

A representative of this office should observe the subgrade as it becomes exposed so that the recommendations provided herein may be revised or reaffirmed as necessary. It is recommended the subgrade be protected and/or stabilized as it becomes exposed.

Protection or stabilization of the subgrade may be accomplished by placement of a minimum one-foot thick layer of angular 1 to 3-inch crushed rocks. The crushed rock should be placed and vibrated to a dense state as the subgrade becomes exposed. The elevation at the bottom of excavation will require adjustment to provide space for the mat of crushed rock. The client should be aware that subgrade stabilization is a trial and error process. There is no way to accurately predict the amount of rock that will be required to adequately stabilize the bottoms. The mat of rock may be several feet thick. A representative of this firm should be on site during stabilization efforts in order to assist the contractor in obtaining a stabilized bottom.

Rubber tire construction equipment shall not be attempted to operate directly on the subgrade soils prior to placing the stabilization rock. Direct operation of rubber tire equipment on soft subgrade soils will likely result in excessive disturbance to the soils, and will result in a delay to the construction schedule. In either case, it is recommended track mounted equipment be utilized. Extreme care should be utilized to place crushed rock as the subgrade becomes exposed.

Due to the anticipated heavy weight of the pile drilling machines, it is recommended the pile contractor observe and evaluate the subgrade conditions as it becomes exposed in order to evaluate its suitability for support of the drilling equipment. Other stabilization methods (such as soil cement mixing or mud mats) may also be suitable for treatment of the subgrade.

Shrinkage

Shrinkage results when a volume of soil removed at one density is compacted to a higher density. A shrinkage factor between 5 and 15 percent should be anticipated when excavating and recompacting the existing fill and underlying native geologic materials on the site to an average comparative compaction of 92 percent.

Weather Related Grading Considerations

When rain is forecast all fill that has been spread and awaits compaction shall be properly compacted prior to stopping work for the day or prior to stopping due to inclement weather.

These fills, once compacted, shall have the surface sloped to drain to an area where water can be removed.

Temporary drainage devices should be installed to collect and transfer excess water to the street in non-erosive drainage devices. Drainage should not be allowed to pond anywhere on the site, and especially not against any foundation or retaining wall. Drainage should not be allowed to flow uncontrolled over any descending slope.

Work may start again, after a period of rainfall, once the site has been reviewed by a representative of this office. Any soils saturated by the rain shall be removed and aerated so that the moisture content will fall within three percent of the optimum moisture content.

Surface materials previously compacted before the rain shall be scarified, brought to the proper moisture content and recompacted prior to placing additional fill, if considered necessary by a representative of this firm.

Geotechnical Observations and Testing During Grading

Geotechnical observations and testing during grading are considered to be a continuation of the geotechnical investigation. It is critical that the geotechnical aspects of the project be reviewed by representatives of Geotechnologies, Inc. during the construction process. Compliance with the design concepts, specifications or recommendations during construction requires review by this firm during the course of construction. Any fill which is placed should be observed, tested, and verified if used for engineered purposes. Please advise this office at least twenty-four hours prior to any required site visit.

FOUNDATION DESIGN – AUGER CAST DISPLACEMENT PILES

Auger Cast Displacement Piles (ACDP)

The use of driven pre-cast concrete piles for support of the proposed structures is not recommended due to noise and vibration concerns impacting the existing and neighboring developments. Therefore, it is recommended that the proposed structure be supported on a system of Auger Cast Displacement Piles (ACDP). The proposed floor slab shall be designed as a structural slab, deriving support entirely from the foundation piles.

The ACDP piles are full displacement piles, installed by using a closed tip displacement tool connected with a forward flight auger below and a reverse auger above. The proposed piles shall be a minimum of 16 inches in diameter, and shall be drilled to penetrate through all fill, and the upper native soils, and bear a minimum of 3 pile diameters into the underlying Older Alluvium (consisting of very dense sands and gravelly sands).

The elevation of the Older Alluvium varies across the site. Once the project design achieves more definition with foundation gridlines, it is recommended that additional CPTs and borings be performed at the site prior to performing the indicator pile program to better define the elevation of the Older Alluvium.

A net allowable axial capacity of 200 kips (with a minimum safety factor of 2) may be utilized for design using the 16-inch diameter ACDP piles, bearing in the Older Alluvium. An ultimate axial capacity (static and seismic) of 570 kips with pile head deflection of less than 1.0 inch may be assumed for the ADCP piles. The ultimate capacity includes a downdrag force of 170 kips, as a result of the potentially liquefiable soils.

(Ultimate Axial Capacity – Downdrag Forces) / Safety Factor of 2 = Net Allowable Axial Capacity
570 kips (ultimate) – 170 kips (downdrag) / 2 = 200 kips (allowable)

A summary of the pile recommendations is presented below, and a more detailed specification is presented in the Appendix of this report.

- Minimum pile diameter shall be 16 inches.
- No predrilling will be allowed.
- Piles shall extend to a minimum depth of 55 feet below the existing site grade, and shall be embedded a minimum of 3 pile diameters into the older alluvial soils (consisting of very dense sands, and gravelly sands), whichever is greater.
- Recommended Net Allowable Axial Compression Capacity of 200 kips (with a safety factor of 2).
- Recommended Allowable Axial Tension Capacity of 100 kips (50 percent of the allowable axial compression capacity).
- Recommended Lateral Capacity Charts provided at the end of the report may be utilized for free head and fixed head conditions, with a maximum 0.5 inch lateral deflection.
- Piles in groups should be spaced at least 3 diameters on center. If the piles are so spaced, no reduction in the downward or upward capacities need be considered due to group action.
- Settlement of pile foundations is anticipated to be less than 1 inch.
- An indicator test pile program shall be performed at the project site prior to production pile, to verify the pile design capacities. All pile load tests shall be performed in accordance with ASTM D1143M. The test piles shall be sacrificial and shall not be utilized for foundation support.
- Low Strain Pile Integrity Tests (PIT) shall be performed on a minimum of 10 percent of the production piles to verify the structural integrity of the piles.

Lateral Design for Pile Foundation

Lateral loads may be resisted by the piles in contact with the underlying soils. Maximum recommended allowable lateral capacities for 0.5-inch deflection for single, isolated, fixed-head and free-head piles are presented in the Appendix. No factors of safety have been applied to the lateral load values calculated to induce 0.5-inch lateral deflection.

Single isolated piles may be classified as piles spaced at or greater than 8 widths on center. For pile groups where piles will be spaced closer than 8 diameters on center in the direction of loading, the following reduction factor may be utilized to determine the allowable lateral pile capacities to maintain a 0.5-inch pile deflection.

Pile Spacing	Percentage of Lateral Passive Resistance
7B	70%
6B	55%
5B	45%
4B	38%
3B	33%

Where B is the diameter of the proposed piles

Lateral capacities provided are for drilled, cast-in-place concrete piles, penetrating the materials encountered during the course of this investigation. Assumed as part of these lateral capacity calculations are a concrete modulus of elasticity of at least 3,000,000 pounds per square inch.

A one-third increase may be used for transient loading such as wind or seismic forces. The capacities presented are based on the strength of the soils. The compressive and tensile strength of the pile sections should be checked to verify the structural capacity of the piles.

Settlement

The maximum settlement of pile-supported foundations is not expected to exceed 1 inch. Differential settlement is not expected to exceed ½ inch.

Piling Equipment

The piling equipment used for the project shall conform to the specifications below.

- *Piling Rig* – The contractor shall use equipment of adequate torque, crowd force, and power, to achieve the design tip elevation. As a minimum, the piling rig shall be capable of providing a minimum torque of 150,000 ft-lbs, and 25 tons of down crowd thrust.

- *Automated Monitoring Equipment* – The drilling rig shall be equipped with an automated monitoring equipment (AME) designed to monitor the pile installation process. During the drilling process, the AME shall record auger depth, drill torque, and elapsed time. During the grouting process, the AME shall record the auger depth, grout pressure, and elapsed time.

- *Displacement Tool* – The drilling tool shall consist of a minimum 10-inch diameter drill stem, 16-inch diameter displacement element, connected with a forward flight auger below and a reverse flight auger above. The diameter of the flights of both augers shall be the same as that of the diameter of the displacement element.

- *Grouting Equipment* – A grout port shall be located near the tip of the displacement auger. A continuous system of grout mixing, pumping, and agitating equipment shall be utilized. Equipment shall be maintained in good working order to maintain a continuous flow of concrete during auger withdrawal. The grout pump shall be capable of developing displacement pressures of 250-psi.

Pile Installation Procedures

The following installation procedures may be followed to install the ACDP.

1. Contractor is responsible for using equipment of adequate torque, crowd, and power to achieve the design tip elevation. The piling rig and displacement tool used for the production pile installation shall be of identical design to that used for the indicator pile test program.

2. The forward flight auger is advanced until it reaches the design tip elevation. The grout port in the displacement tool shall be closed with a plug that prevents soil and/or water from entering the hollow shaft while the displacement tool is advanced into the ground.

3. The displacement element and the reverse flight auger displace the soil cuttings laterally into the wall of the shaft and create a smooth walled shaft with diameter equivalent to the displacement element (both test piles and production piles shall be a minimum of 16 inches in diameter).

4. A minimum delivery pressure of 250 psi plus the hydraulic pressure developed by the grout column in the drill stem shall be applied to create the pile. The operator shall maintain positive rotation of the displacement auger continuously throughout the grouting process until the displacement element is completely retracted from the ground.

5. The piling rig shall be equipped with automated monitoring equipment (AME) to record the auger depth, drill torque, grout pressure, and elapsed time. All recorded data shall be provided for review.

6. Once the grouted pile shaft is filled with concrete, the steel reinforcing cage shall be inserted into the concrete pile. All reinforcing elements are fitted with centralizers or clip spacers.

Indicator Test Pile Program

An indicator pile test program must be performed and approved by the City of Los Angeles prior to installation of the production piles. The number of test piles shall be equivalent to a minimum of 2 test piles, or 1 percent of the production piles for the proposed structure, whichever is greater. All pile load tests shall be performed in accordance with ASTM D1143 to verify the pile design capacities. The test piles and reaction piles shall be considered sacrificial and shall not be utilized for foundation support of the proposed buildings.

Additional foundation piles may be necessary if the actual load tests do not meet the recommended allowable loads.

- Load tests shall be performed on sacrificial test piles in accordance with ASTM D1143M. The design load shall be held until the measured creep does not exceed 0.01 inch per hour. Piles with a settlement rate exceeding 0.01 inch/hour under the design load during a pile test will be rejected.

- Pile load tests shall be performed to a minimum load equivalent to the ultimate capacity of 570 kips.

- Test piles and reaction piles shall be sacrificial and shall not be incorporated as foundation piles. Sacrificial test piles and reaction piles shall be cut off 3 feet below the finished grade and abandoned in place following the completion of the testing program.

- Gamma-Gamma density logging (GDL) and Low Strain Pile Integrity Tests (PIT) shall be performed on all test piles and reaction piles. GDL shall be performed in accordance with Caltrans CT 233. PIT shall be performed in accordance with ASTM D5882.

- One test pile shall be exhumed from the ground to physically examine the pile integrity.

- Results of the pile load testing will be submitted as a summary letter to the LADBS Grading Division for review and approval.

Geotechnical Inspections

During pile installation, a City of Los Angeles Deputy Grading Inspector shall record and maintain data for each pile, including the following:

- Pile Number
- Installed pile length
- Auger torque vs. depth
- Head pressure inside the tremie pipe vs. depth
- Drilling rate vs. depth
- Concrete volume vs. depth
- Unanticipated site conditions if any

Non-Destructive Testing

None-destructive testing methods shall be employed to evaluate the integrity of the piles installed to provide quality control and assurance of the pile construction method.

- Gamma-Gamma density logging (GDL) and Low Strain Pile Integrity Tests (PIT) shall be performed on all test piles and reaction piles. GDL shall be performed in accordance with Caltrans CT 233. PIT shall be performed in accordance with ASTM D5882.

- Low Strain Pile Integrity Tests (PIT) shall be performed on 10 percent of the production piles.

- If any PIT test indicates a discontinuity within a tested pile, that pile shall be evaluated by the geotechnical and structural engineers. Unsatisfactory piles may be abandoned in place and shall be replaced with replacement piles.

Miscellaneous Foundations

Foundations for small miscellaneous outlying structures, such as property line fence walls, planters, exterior canopies, exterior staircases and ramps, and trash enclosures, which will not be tied-in to the proposed structure may be supported on conventional foundations bearing in compacted fill. impractical

Up to 12½ feet of existing fill was encountered during exploration. Records of certification of the existing fill could not be found during research of available records at the City of Los Angeles. Due to the depth of the existing fill, removal and recompaction of all existing fill materials would be unfeasible and cost prohibitive. The client should be aware that removal of all existing fill in the area of small miscellaneous outlying structures is not required, however, small outlying structures constructed in this manner will most likely have a shorter design life and increased maintenance costs, and may potentially be damaged and will require replacement should liquefaction occurs during a major seismic event. In addition, the City will require a modification request for placement of compacted fill over existing uncertified fill, and the use of existing uncertified fill for support of foundations for small miscellaneous outlying structures.

It is recommended that existing fill materials be removed and recompacted to a minimum depth of 2 feet below the bottom of the proposed footings for small outlying miscellaneous structures. Additional removal and recompaction may be necessary if additional loose or soft soils are encountered during grading.

Continuous wall footings may be designed for a bearing value of 1,500 pounds per square foot, and should be a minimum of 12 inches in width, 24 inches in depth below the lowest adjacent grade and 24 inches into the recommended bearing material. No bearing value increases are recommended. All continuous foundations should be reinforced with a minimum of four #4 steel bars. Two should be placed near the top of the foundation, and two should be placed near the bottom.

Since the recommended bearing capacity is a net value, the weight of concrete in the foundations may be taken as 50 pounds per cubic foot and the weight of the soil backfill may be neglected when determining the downward load on the foundations.

Resistance to lateral loading may be provided by friction acting at the base of foundations and by passive earth pressure. An allowable coefficient of friction of 0.3 may be used with the dead load forces.

Passive geologic pressure for the sides of foundations poured against undisturbed or recompacted soil may be computed as an equivalent fluid having a density of 100 pounds per cubic foot with a maximum earth pressure of 1,500 pounds per square foot. The passive and friction components may be combined for lateral resistance without reduction. A one-third increase in the passive value may be used for short duration loading such as wind or seismic forces.

RETAINING WALL DESIGN

It is anticipated that the proposed subterranean level will extend on the order of 20 feet below the existing site grade. Based on the latest design plans, the finished floor elevation of the lowest B2 subterranean parking level will be at approximately 4.0 feet above MSL, which corresponds to 5 feet below the historically highest groundwater level. Since the lowest subterranean level will extend below above the historically highest groundwater level, it is recommended that the proposed structure be designed for hydrostatic pressure.

Cantilever retaining walls supporting a level backslope may be designed utilizing a triangular distribution of active earth pressure. Restrained retaining walls may be designed utilizing a triangular distribution of at-rest earth pressure. Retaining walls may be designed utilizing the following table:

Height of Retaining Wall (feet)	Cantilever Retaining Wall Triangular Distribution of Active Earth Pressure with Hydrostatic Pressure (pcf)	Restrained Retaining Wall Triangular Distribution of At-Rest Earth Pressure with Hydrostatic Pressure (pcf)
Up to 25 feet	80 pcf	105 pcf

The lateral earth pressures recommended above for retaining walls assume that the proposed retaining walls will be designed for full hydrostatic pressure based on the ground surface, and a permanent drainage system behind the retaining walls will be eliminated. Additional active pressure should be added for a surcharge condition due to sloping ground, vehicular traffic or adjacent structures.

Small miscellaneous site cantilever retaining walls (such as property line walls, ramps, and planters), up to 5 feet in height, may be designed for a triangular distribution of active earth pressure of 35 pcf. This wall pressure assumes that a permanent drainage system will be installed so that external water pressure will not be developed against the walls. Miscellaneous structures may be supported on conventional foundations following the recommendations provided in the "Miscellaneous Foundation" section above.

The upper ten feet of the retaining wall adjacent to streets, driveways or parking areas 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 walls due to normal street traffic. If the traffic is kept back at least ten feet from the retaining walls, the traffic surcharge may be neglected. Foundations may be designed using the allowable bearing capacities, friction, and passive earth pressure found in the "Foundation Design" section above.

Dynamic (Seismic) Earth Pressure

Retaining walls exceeding 6 feet in height shall be designed to resist the additional earth pressure caused by seismic ground shaking. A triangular pressure distribution should be utilized for the additional seismic loads, with an equivalent fluid pressure of 25 pounds per cubic foot. The seismic earth pressure should be combined with the lateral active earth pressure for analyses of restrained basement walls under seismic loading condition.

Surcharge from Adjacent Structures

As indicated herein, additional active pressure should be added for a surcharge condition due to sloping ground, vehicular traffic or adjacent structures for retaining walls and shoring design.

The following surcharge equation provided in the LADBS Information Bulletin Document No. P/BC 2008-83, may be utilized to determine the surcharge loads on basement walls and shoring system for existing structures located within the 1:1 (h:v) surcharge influence zone of the excavation and basement.

Resultant lateral force: $R = (0.3*P*h^2)/(x^2+h^2)$

Location of lateral resultant: $d = x*[(x^2/h^2+1)*\tan^{-1}(h/x)-(x/h)]$

where:

R	=	resultant lateral force measured in pounds per foot of wall width.
P	=	resultant surcharge loads of continuous or isolated footings measured in pounds per foot of length parallel to the wall.
x	=	distance of resultant load from back face of wall measured in feet.
h	=	depth below point of application of surcharge loading to top of wall footing measured in feet.
d	=	depth of lateral resultant below point of application of surcharge loading measure in feet.
$\tan^{-1}(h/x)$	=	the angle in radians whose tangent is equal to h/x.

The structural engineer and shoring engineer may use this equation to determine the surcharge loads based on the loading of the adjacent structures located within the surcharge influence zone.

Waterproofing

Moisture effecting retaining walls is one of the most common post construction complaints. Poorly applied or omitted waterproofing can lead to efflorescence or standing water inside the building. Efflorescence is a process in which a powdery substance is produced on the surface of the concrete by the evaporation of water. The white powder usually consists of soluble salts such as gypsum, calcite, or common salt. Efflorescence is common to retaining walls and does not affect their strength or integrity.

It is recommended that retaining walls be waterproofed. Waterproofing design and inspection of its installation is not the responsibility of the geotechnical engineer. A qualified waterproofing consultant should be retained in order to recommend a product or method which would provide protection to below grade walls.

Retaining Wall Drainage

Unless the retaining walls are structurally designed for hydrostatic pressure, all retaining walls shall be provided with a subdrain in order to minimize the potential for future hydrostatic pressure buildup behind the proposed retaining walls. Subdrains may consist of four-inch diameter perforated pipes, placed with perforations facing down. The pipe shall be encased in at least one-foot of gravel around the pipe. The gravel may consist of three-quarter inch to one inch crushed rocks.

Where retaining walls are to be constructed adjacent to property lines or shoring system, there is usually not enough space for placement of a standard perforated pipe and gravel drainage system. As an alternative to the recommended perforated drain pipe and gravel system, 2-inch diameter weepholes with 1 cubic foot of gravel pockets may be placed at the 8 feet on center along the base of the wall. The gravel may consist of three-quarter inch to one inch crushed rocks. A collector is placed within the gravel which directs collected waters through the wall to a sump or standard pipe and gravel system constructed under the slab.

A compacted fill blanket or other seal shall be provided at the surface. Retaining walls may be backfilled with gravel adjacent to the wall to within 2 feet of the ground surface. The onsite earth materials are acceptable for use as retaining wall backfill as long as they are compacted to a minimum of 90 percent (or 95 percent for cohesionless soils having less than 15 percent finer than 0.005 millimeters) of the maximum density as determined by the latest revision of ASTM D 1557.

Certain types of subdrain pipe are not acceptable to the various municipal agencies, it is recommended that prior to purchasing subdrainage pipe, the type and brand is cleared with the proper municipal agencies. Subdrainage pipes should outlet to an acceptable location.

Retaining Wall Backfill

Any required backfill should be mechanically compacted in layers not more than 8 inches thick, to at least 90 percent (or 95 percent for cohesionless soils having less than 15 percent finer than 0.005 millimeters) of the maximum density obtainable by the latest revision of ASTM D 1557 method of compaction. Flooding should not be permitted. Proper compaction of the backfill will be necessary to reduce settlement of overlying walks and paving. Some settlement of required backfill should be anticipated, and any utilities supported therein should be designed to accept differential settlement, particularly at the points of entry to the structure.

Proper compaction of the backfill will be necessary to reduce settlement of overlying walks and paving. Some settlement of required backfill should be anticipated, and any utilities supported therein should be designed to accept differential settlement, particularly at the points of entry to the structure.

TEMPORARY EXCAVATIONS

It is anticipated that excavations on the order of 20 to 25 feet in vertical height will be required for the proposed subterranean levels, pile caps, and grade beams. The excavations are expected to expose fill and dense native soils, which are suitable for vertical excavations up to 5 feet where not surcharged by adjacent traffic or structures.

Surcharged excavations are currently not anticipated. Should the design or location of any structures, as outlined in this report, be changed or altered, the recommendations contained herein should not be considered valid until reviewed and modified or reaffirmed subsequent to such review.

Where sufficient space is available, temporary unsurcharged embankments could be sloped back without shoring. Excavations over 5 feet in height should may be excavated at a uniform 1:1

(h:v) slope gradient in its entirety to a maximum height of 15 feet. A uniform sloped excavation does not have a vertical component.

Where sloped embankments are utilized, the tops of the slopes should be barricaded to prevent vehicles and storage loads within seven feet of the tops of the slopes. If the temporary construction embankments are to be maintained during the rainy season, berms are suggested along the tops of the slopes where necessary to prevent runoff water from entering the excavation and eroding the slope faces. The soils exposed in the cut slopes should be inspected during excavation by personnel from this office so that modifications of the slopes can be made if variations in the soil conditions occur.

Excavation Observations

It is critical that the soils exposed in the cut slopes are observed by a representative of Geotechnologies, Inc. during excavation so that modifications of the slopes can be made if variations in the geologic material conditions occur. Many building officials require that temporary excavations should be made during the continuous observations of the geotechnical engineer. All excavations should be stabilized within 30 days of initial excavation.

SHORING

The following information on the design and installation of the shoring is as complete as possible at this time. It is suggested that a review of the final shoring plans and specifications be made by this office prior to bidding or negotiating with a shoring contractor.

The recommended method of shoring consists of steel soldier piles, placed in drilled holes and backfilled with concrete. The soldier piles may be designed as cantilevers or laterally braced utilizing drilled tie-back anchors or raker braces.

Soldier Piles

Drilled cast-in-place soldier piles should be placed no closer than 2½ diameters on center. The minimum diameter of the piles is 18 inches. Structural concrete should be used for the soldier piles below the excavation; lean-mix concrete may be employed above that level. As an alternative, lean-mix concrete may be used throughout the pile where the reinforcing consists of a wideflange section. The slurry must be of sufficient strength to impart the lateral bearing pressure developed by the wideflange section to the earth materials. For soldier pile design purposes, an allowable passive value for the earth materials below the bottom plane of excavation may be assumed to be 230 pounds per square foot per foot of depth, up to a maximum of 2,300 pounds per square foot. This assumes a saturated condition. To develop the full lateral value, provisions should be implemented to assure firm contact between the soldier piles and the undisturbed earth materials.

The frictional resistance between the soldier piles and retained earth material may be used to resist the vertical component of the anchor load. The coefficient of friction may be taken as 0.25 based on uniform contact between the steel beam and lean-mix concrete and retained earth. The portion of soldier piles below the plane of excavation may also be employed to resist the downward loads. The downward capacity may be determined using a frictional resistance of 200 pounds per square foot. The minimum depth of embedment for shoring piles is 7 feet below the bottom of excavated plane for restrained shoring system, and 10 feet below the bottom of excavated plane for cantilever shoring system.

Groundwater was encountered at depths between 22½ and 30 feet below the existing site grade during exploration. Caving of the saturated earth materials below the groundwater level should be expected to occur during drilling of piles. Casing or polymer drilling fluid will most likely be required during drilling in order to maintain open shafts. If casing is used, extreme care should be employed so that the pile is not pulled apart as the casing is withdrawn. At no time should the distance between the surface of the concrete and the bottom of the casing be less than 5 feet.

Depending on the draw down level associated with the future dewatering program, it is anticipated that the proposed piles will likely encounter water. Piles placed below the water level will require the use of a tremie to place the concrete into the bottom of the hole. A tremie shall consist of a water-tight tube having a diameter of not less than 6 inches with a hopper at the top. The tube shall be equipped with a device that will close the discharge end and prevent water from entering the tube while it is being charged with concrete. The tremie shall be supported so as to permit free movement of the discharge end over the entire top surface of the work and to permit rapid lowering when necessary to retard or stop the flow of concrete. The discharge end shall be closed at the start of the work to prevent water entering the tube and shall be entirely sealed at all times, except when the concrete is being placed. The tremie tube shall be kept full of concrete. The flow shall be continuous until the work is completed and the resulting concrete seal shall be monolithic and homogeneous. The tip of the tremie tube shall always be kept about five feet below the surface of the concrete and definite steps and safeguards should be taken to insure that the tip of the tremie tube is never raised above the surface of the concrete.

A special concrete mix should be used for concrete to be placed below water. The design shall provide for concrete with a strength of 1,000 psi over the initial job specification. An admixture that reduces the problem of segregation of paste/aggregates and dilution of paste shall be included. The slump shall be commensurate to any research report for the admixture, provided that it shall also be the minimum for a reasonable consistency for placing when water is present.

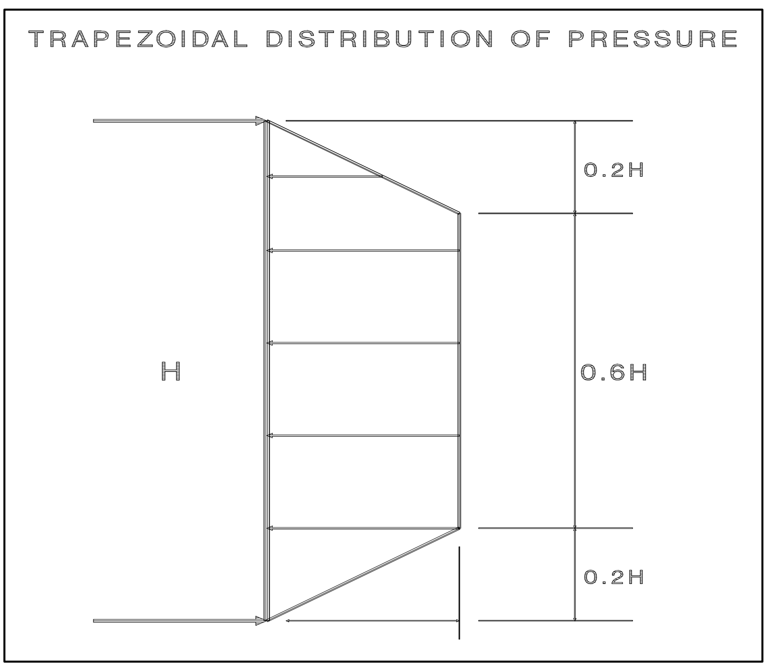
<u>Lagging</u>

At this time, it is anticipated that most or all of the excavation will require continuous lagging. It is recommended that the exposed soils be observed by a representative of the geotechnical engineer to verify the cohesive nature of the earth materials, and determine whether any lagging may be omitted.

Soldier piles and anchors should be designed for the full anticipated pressures. Due to arching in the earth materials, the pressure on the lagging will be less. It is recommended that the lagging be designed for the full design pressure but be limited to a maximum of 400 pounds per square foot.

Lateral Pressures

A triangular distribution of lateral earth pressure should be utilized for the design of a cantilever shoring system. A trapezoidal distribution of lateral earth pressure (as shown in the diagram below) would be appropriate where shoring is to be restrained at the top by tie backs or raker braces. The lateral pressures provided below assume temporary dewatering will be maintained during the use of the shoring system, and hydrostatic forces will not develop on the shoring.



Pressures for the design of cantilevered and restrained shoring supporting level back slopes are presented in the following table.

Height of Shoring (feet)	**Cantilever Shoring System Equivalent Fluid Pressure (pcf) Triangular Distribution of Pressure**	**Restrained Shoring System Lateral Earth Pressure (psf)* Trapezoidal Distribution of Pressure**
Up to 25 feet	48 pcf	30H psf

*Where H is the height of the shoring in feet.

Where a combination of sloped embankment and shoring is utilized, the pressure will be greater and must be determined for each combination.

Surcharge from Adjacent Traffic or Structures

Additional active pressures should be applied where the shoring will be surcharged by adjacent traffic or structures. Traffic and/or structure surcharge pressures should be determined in accordance with the "Retaining Wall Design" section of this report.

Tieback Anchor Design and Installation

Tieback anchors may be used to resist lateral loads. Friction anchors are recommended. For design purposes, it may be assumed that the active wedge is defined by a plane drawn 35 degrees with the vertical through the bottom plane of the excavation. Friction anchors should extend a minimum of 20 feet beyond the potentially active wedge.

Tieback anchors may be installed between 20 and 40 degrees below the horizontal. Caving may occur within granular materials or in shafts drilled below the groundwater level. Measures should be implemented to handle caving materials, including the use of drill casing during drilling. Where caving occurs the following provisions should be implemented in order to minimize such caving. The anchor shafts should be filled with concrete by pumping from the tip out, and the concrete should extend from the tip of the anchor to the active wedge. In order to minimize the chances of caving, it is recommended that the portion of the anchor shaft within the active wedge be backfilled with sand before testing the anchor. This portion of the shaft should be filled tightly and flush with the face of the excavation. The sand backfill should be placed by pumping; the sand may contain a small amount of cement to facilitate pumping.

Depending on the techniques utilized, and the experience of the contractor performing the installation, it is anticipated that a skin friction of 1,250 pounds per square foot could be utilized for post-grouted anchors, provided the design does not rely on end-bearing plates to provide the necessary capacity. It is anticipated that multiple grouting stages will be required for post-grouted anchors. Only the frictional resistance developed beyond the active wedge should be utilized in resisting lateral loads. Anchors should be placed at least 6 feet on center to be considered isolated.

Tieback Anchor Testing

At least 10 percent of the anchors should be selected for “Quick”, 200 percent tests. It is recommended that at least three anchors be selected for 24-hour, 200 percent tests. It is recommended that the 24-hour tests be performed prior to installation of additional tiebacks. The purpose of the 200 percent tests is to verify the friction value assumed in design. The anchors should be tested to develop twice the assumed friction value. Where satisfactory tests are not achieved on these initial anchors, the anchor diameter and/or length should be increased until satisfactory test results are obtained.

The total deflection during the 24-hour 200 percent test should not exceed 12 inches. During the 24-hour tests, the anchor deflection should not exceed 0.75 inches measured after the 200 percent test load is applied.

For the "quick" 200 percent tests, the 200 percent test load should be maintained for 30 minutes. The total deflection of the anchor during the 200 percent quick tests should not exceed 12 inches; the deflection after the 200 percent load has been applied should not exceed 0.25 inch during the 30-minute period.

All of the remaining anchors should be tested to at least 150 percent of design load. The total deflection during the 150 percent test should not exceed 12 inches. The rate of creep under the 150 percent test load 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 anchor should be locked-off at the design load. This should be verified by rechecking the load in the anchor. The load should be within 10 percent of the design load. Where satisfactory tests are not attained, the anchor diameter and/or length should be increased or additional anchors installed until satisfactory test results are obtained. Where post-grouted anchors are utilized, additional post-grouting may be required. The installation and testing of the anchors should be observed by a representative of the soils engineer.

Deflection

It is difficult to accurately predict the amount of deflection of a shored embankment. It should be realized that some deflection will occur. Where there are structures within a 1:1 plane drawn upward from the bottom of the excavation, it is recommended that the shoring be designed for a maximum deflection of ½-inch at the top of the shored embankment. Where there are not structures within a 1:1 projection from the bottom of the excavation, it is recommended the shoring be designed for a maximum deflection of 1 inch. If greater deflection occurs during construction, additional bracing may be necessary to minimize settlement of adjacent buildings and streets.

Pre-Construction Survey

Prior to shoring installation and excavation, it is recommended the adjacent improvements be surveyed to provide a documented record of their condition. Such a survey would aid in the resolution of any disputes that may arise concerning damage to adjacent facilities caused by the proposed construction.

Monitoring

Because of the depth of the excavations, some means of monitoring the performance of the shoring system is suggested. The monitoring should consist of periodic surveying of the lateral and vertical locations of the tops of all soldier piles and the lateral movement along the entire lengths of selected soldier piles.

Shoring Observations

It is critical that the installation of shoring is observed by a representative of this office. Many local agencies require that shoring installation be performed under the continuous observation of the geotechnical engineer. The observations are made so that modifications of the recommendations can be made if variations in the earth material or groundwater conditions occur. Also, the observations will allow for a report to be prepared on the installation of shoring for the use of the local building official.

SLABS-ON-GRADE

Interior Building Floor Slab

The proposed building floor slabs shall be designed as structural slabs deriving support from the pile foundation system. Any geologic materials loosened or over-excavated should be wasted from the site or properly compacted to 90 percent (or 95 percent for cohesionless soils having less than 15 percent finer than 0.005 millimeters) of the maximum dry density.

Hydrostatic Considerations for Interior Building Floor Slabs

Where constructed below the historic high groundwater elevation, interior building floor slabs shall be waterproofed and designed to withstand the hydrostatic uplift pressure based on the historic high water elevation of 9.0 feet above MSL. The uplift pressure to be used in design should be 62.4(H) pounds per square foot, where "H" is the height of the height of the historic high water level above the bottom of the building floor slab in feet. It is recommended a qualified waterproofing consultant be retained in order to provide waterproofing recommendations for the proposed project.

Outdoor Concrete Flatwork

Outdoor concrete flatwork should be a minimum of 4 inches in thickness, and should be reinforced with a minimum of #3 steel bars on 12-inch centers each way. Outdoor concrete flatwork should be cast over undisturbed natural geologic materials or properly controlled fill materials. Any geologic materials loosened or over-excavated should be wasted from the site or properly compacted to 90 percent (or 95 percent for cohesionless soils having less than 15 percent finer than 0.005 millimeters) of the maximum dry density.

Design of Slabs That Receive Moisture-Sensitive Floor Coverings

Geotechnologies, Inc. does not practice in the field of moisture vapor transmission evaluation and mitigation. Therefore, it is recommended that a qualified consultant be engaged to evaluate the general and specific moisture vapor transmission paths and any impact on the proposed construction. The qualified consultant should provide recommendations for mitigation of potential adverse impacts of moisture vapor transmission on various components of the structure.

It is recommended that the floor slabs in the lowest subterranean level should be waterproofed. A qualified waterproofing consultant should be retained in order to recommend a product or method which would provide protection for concrete slabs-on-grade.

All concrete slabs-on-grade should be supported on vapor retarder. The design of the slab and the installation of the vapor retarder should comply with the most recent revisions of ASTM E 1643 and ASTM E 1745. The vapor retarder should comply with ASTM E 1745 Class A requirements.

Where a vapor retarder is used, a low-slump concrete should be used to minimize possible curling of the slabs. The barrier can be covered with a layer of trimmable, compactible, granular fill, where it is thought to be beneficial. See ACI 302.2R-32, Chapter 7 for information on the placement of vapor retarders and the use of a fill layer.

Concrete Crack Control

The recommendations presented in this report are intended to reduce the potential for cracking of concrete slabs-on-grade due to settlement. However even where these recommendations have been implemented, foundations, stucco walls and concrete slabs-on-grade may display some cracking due to minor soil movement and/or concrete shrinkage. The occurrence of concrete cracking may be reduced and/or controlled by limiting the slump of the concrete used, proper concrete placement and curing, and by placement of crack control joints at reasonable intervals, in particular, where re-entrant slab corners occur.

For standard control of concrete cracking, a maximum crack control joint spacing of 10 feet should not be exceeded. Lesser spacings would provide greater crack control. Joints at curves and angle points are recommended. The crack control joints should be installed as soon as practical following concrete placement. Crack control joints should extend a minimum depth of one-fourth the slab thickness. Construction joints should be designed by a structural engineer.

Complete removal of the existing fill soils beneath outdoor flatwork such as walkways or patio areas, is not required, however, due to the rigid nature of concrete, some cracking, a shorter design life and increased maintenance costs should be anticipated. In order to provide uniform support beneath the flatwork it is recommended that a minimum of 12 inches of the exposed subgrade beneath the flatwork be scarified and recompacted to 90 percent (or 95 percent for cohesionless soils having less than 15 percent finer than 0.005 millimeters) relative compaction.

PAVEMENTS

Prior to placing paving, the existing grade should be scarified to a depth of 12 inches, moistened as required to obtain optimum moisture content, and recompacted to 95 percent of the maximum density as determined by the most recent revision of ASTM D 1557. The client should be aware that removal of all existing fill in the area of new paving is not required, however, pavement constructed in this manner will most likely have a shorter design life and increased maintenance costs. The following pavement sections are recommended:

Service	Asphalt Pavement Thickness Inches	Base Course Inches
Passenger Cars	3	4
Moderate Truck	4	6
Heavy Truck	6	9

A subgrade modulus of 100 pounds per cubic inch may be assumed for design of concrete paving. Concrete paving for passenger cars and moderate truck traffic shall be a minimum of 6 inches in thickness, and shall be underlain by 4 inches of aggregate base. Concrete paving for heavy truck traffic shall be a minimum of 7½ inches in thickness, and shall be underlain by 6 inches of aggregate base. For standard crack control maximum expansion joint spacing of 10 feet should not be exceeded. Lesser spacings would provide greater crack control. Joints at curves and angle points are recommended.

Aggregate base should be compacted to a minimum of 95 percent of the most recent revision of ASTM D 1557 laboratory maximum dry density. Base materials should conform to Sections 200-2.2 or 200-2.4 of the "Standard Specifications for Public Works Construction", (Green Book), latest edition.

SITE DRAINAGE

Proper surface drainage is critical to the future performance of the project. Saturation of a soil can cause it to lose internal shear strength and increase its compressibility, resulting in a change in the designed engineering properties. Proper site drainage should be maintained at all times.

All site drainage should be collected and transferred to the street in non-erosive drainage devices. The proposed structure should be provided with roof drainage. Discharge from downspouts, roof drains and scuppers should not be permitted on unprotected soils within five feet of the building perimeter. Drainage should not be allowed to pond anywhere on the site, and especially not against any foundation or retaining wall. Drainage should not be allowed to flow uncontrolled over any descending slope. Planters which are located within a distance equal to the depth of a retaining wall should be sealed to prevent moisture adversely affecting the wall. Planters which are located within five feet of a foundation should be sealed to prevent moisture affecting the earth materials supporting the foundation.

STORMWATER DISPOSAL

Regulatory agencies have been requiring the disposal of a certain amount of stormwater generated on a site by infiltration into the site soils. Increasing the moisture content of a soil can cause it to lose internal shear strength and increase its compressibility, resulting in a change in the designed engineering properties. This means that any overlying structure, including buildings, pavements and concrete flatwork, could sustain damage due to saturation of the subgrade soils. Structures serviced by subterranean levels could be adversely impacted by stormwater disposal by increasing the design fluid pressures on retaining walls and causing leaks in the walls. Proper site drainage is critical to the performance of any structure in the built environment.

Due to the liquefaction potential, the depth of fill materials, and the historically highest groundwater level, infiltration of stormwater is considered to be unfeasible for the subject site.

DESIGN REVIEW

Engineering of the proposed project should not begin until approval of the geotechnical report by the Building Official is obtained in writing. Significant changes in the geotechnical recommendations may result during the building department review process.

It is recommended that the geotechnical aspects of the project be reviewed by this firm during the design process. This review provides assistance to the design team by providing specific recommendations for particular cases, as well as review of the proposed construction to evaluate whether the intent of the recommendations presented herein are satisfied.

CONSTRUCTION MONITORING

Geotechnical observations and testing during construction are considered to be a continuation of the geotechnical investigation. It is critical that this firm review the geotechnical aspects of the project during the construction process. Compliance with the design concepts, specifications or recommendations during construction requires review by this firm during the course of construction. All foundations should be observed by a representative of this firm prior to placing concrete or steel. Any fill which is placed should be observed, tested, and verified if used for engineered purposes. Please advise Geotechnologies, Inc. at least twenty-four hours prior to any required site visit.

If conditions encountered during construction appear to differ from those disclosed herein, notify Geotechnologies, Inc. immediately so the need for modifications may be considered in a timely manner.

It is the responsibility of the contractor to ensure that all excavations and trenches are properly sloped or shored. All temporary excavations should be cut and maintained in accordance with applicable OSHA rules and regulations.

EXCAVATION CHARACTERISTICS

The exploration performed for this investigation is limited to the geotechnical excavations described. Direct exploration of the entire site would not be economically feasible. The owner, design team and contractor must understand that differing excavation and drilling conditions may be encountered based on boulders, gravel, oversize materials, groundwater and many other conditions. Fill materials, especially when they were placed without benefit of modern grading codes, regularly contain materials which could impede efficient grading and drilling. Southern California sedimentary bedrock is known to contain variable layers which reflect differences in depositional environment. Such layers may include abundant gravel, cobbles and boulders. Similarly, bedrock can contain concretions. oncretions are typically lenticular and follow the bedding. They are formed by mineral deposits. Concretions can be very hard. Excavation and drilling in these areas may require full size equipment and coring capability. The contractor should be familiar with the site and the geologic materials in the vicinity.

CLOSURE AND LIMITATIONS

The purpose of this report is to aid in the design and completion of the described project. Implementation of the advice presented in this report is intended to reduce certain risks associated with construction projects. The professional opinions and geotechnical advice contained in this report are sought because of special skill in engineering and geology and were prepared in accordance with generally accepted geotechnical engineering practice. Geotechnologies, Inc. has a duty to exercise the ordinary skill and competence of members of the engineering profession. Those who hire Geotechnologies, Inc. are not justified in expecting infallibility, but can expect reasonable professional care and competence.

The scope of the geotechnical services provided did not include any environmental site assessment for the presence or absence of organic substances, hazardous/toxic materials in the soil, surface water, groundwater, or atmosphere, or the presence of wetlands.

Proper compaction is necessary to reduce settlement of overlying improvements. Some settlement of compacted fill should be anticipated. Any utilities supported therein should be designed to accept differential settlement. Differential settlement should also be considered at the points of entry to the structure.

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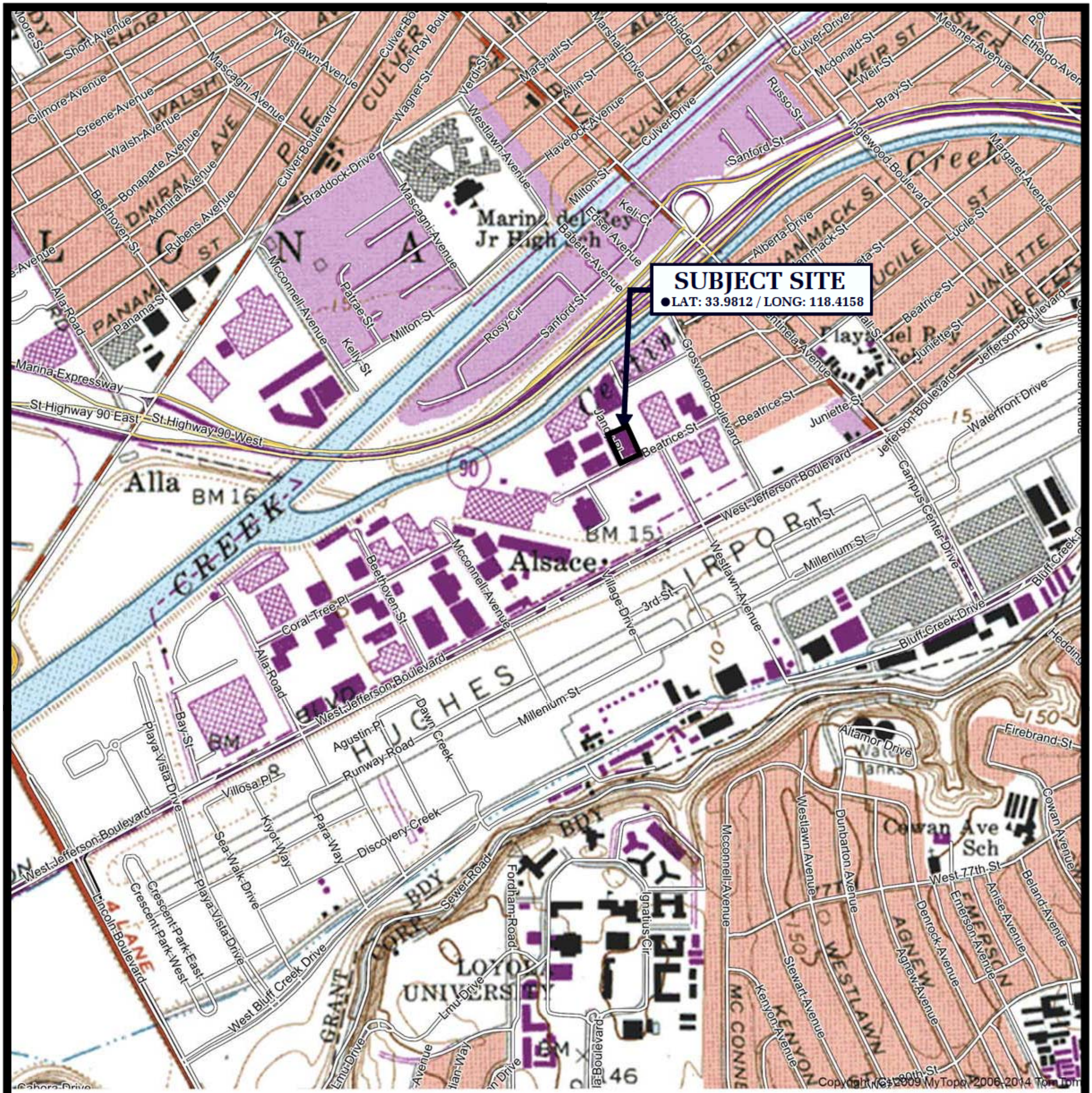


SCALE 1:12000

0.0 0.1 0.2 0.3 0.4 0.5 0.6
MILES

0 1000 2000 3000
FFFT

N

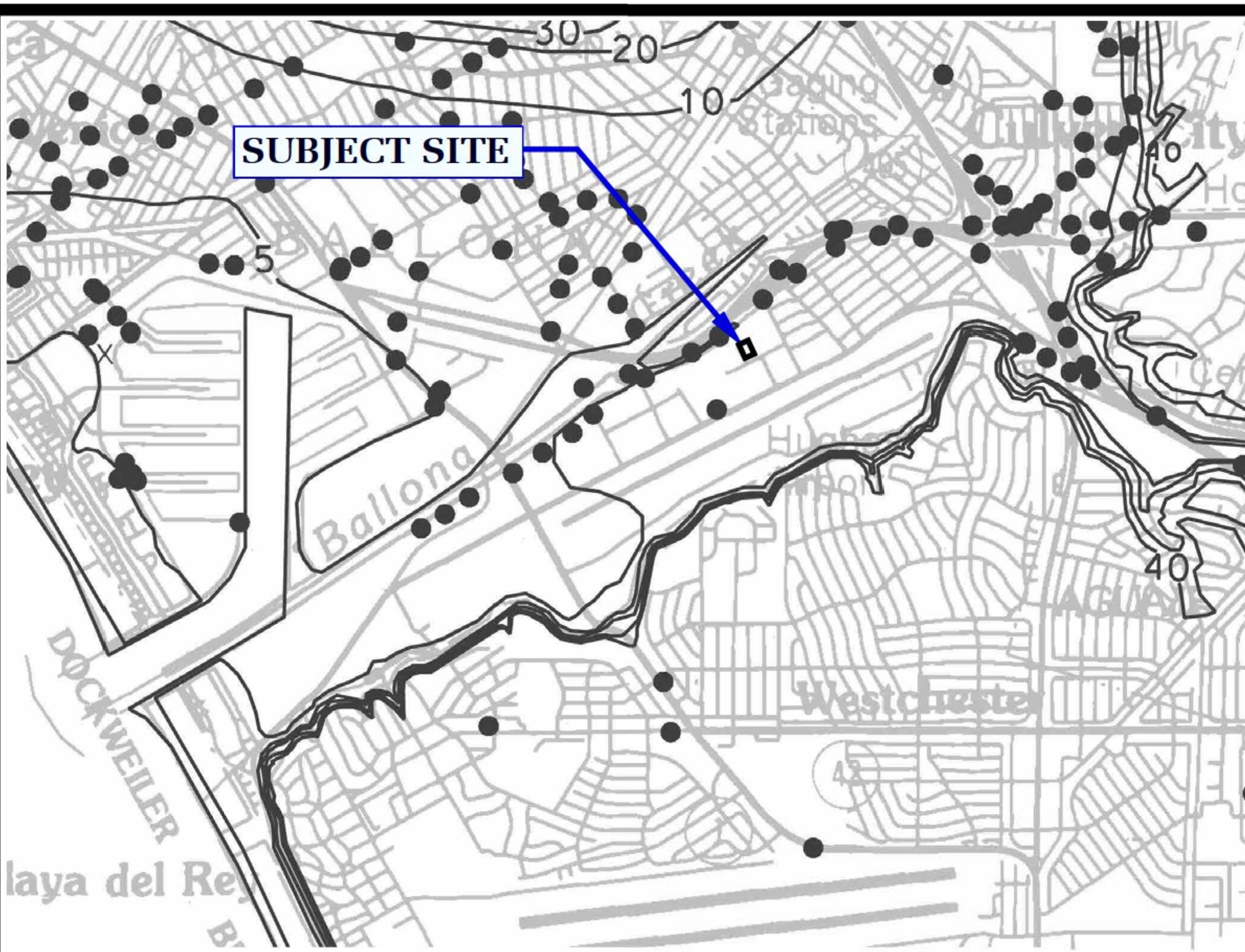
REFERENCE: U.S.G.S. TOPOGRAPHIC MAPS, 7.5 MINUTE SERIES, VENICE, CA QUADRANGLE

VICINITY MAP

Geotechnologies, Inc.
Consulting Geotechnical Engineers

CHAIT COMPANY

FILE NO. 21194



ONE MILE

SCALE

30 Depth to groundwater in feet

REFERENCE: CDMG, SEISMIC HAZARD ZONE REPORT, 036
VENICE 7.5 - MINUTE QUADRANGLE, LOS ANGELES COUNTY, CALIFORNIA (1998, REVISED 2006)

HISTORICALLY HIGHEST GROUNDWATER LEVELS

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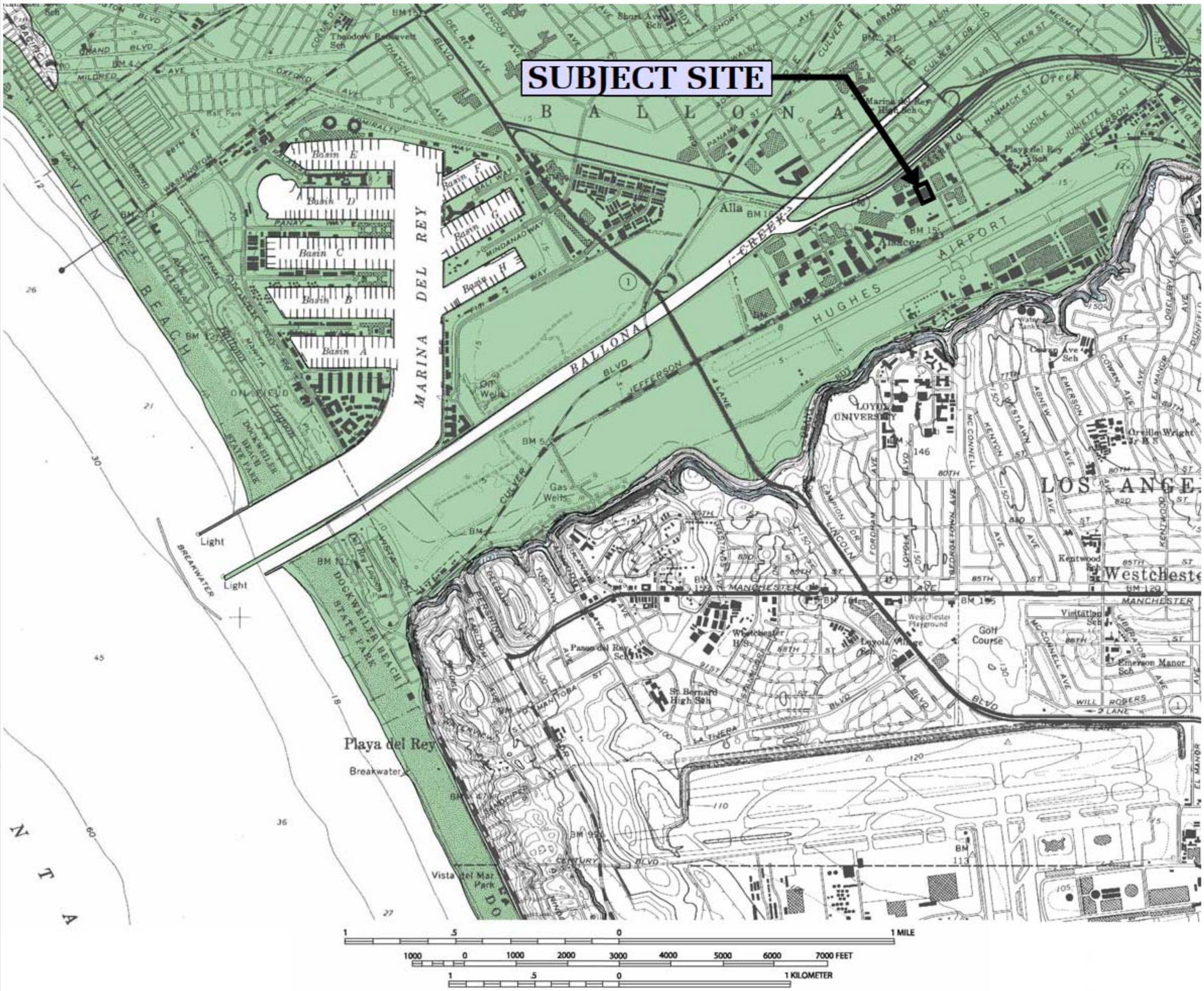

LIQUEFACTION AREA

REFERENCE: SEISMIC HAZARD ZONES, VENICE QUADRANGLE OFFICIAL MAP (CDMG, 1999)

SEISMIC HAZARD ZONE MAP

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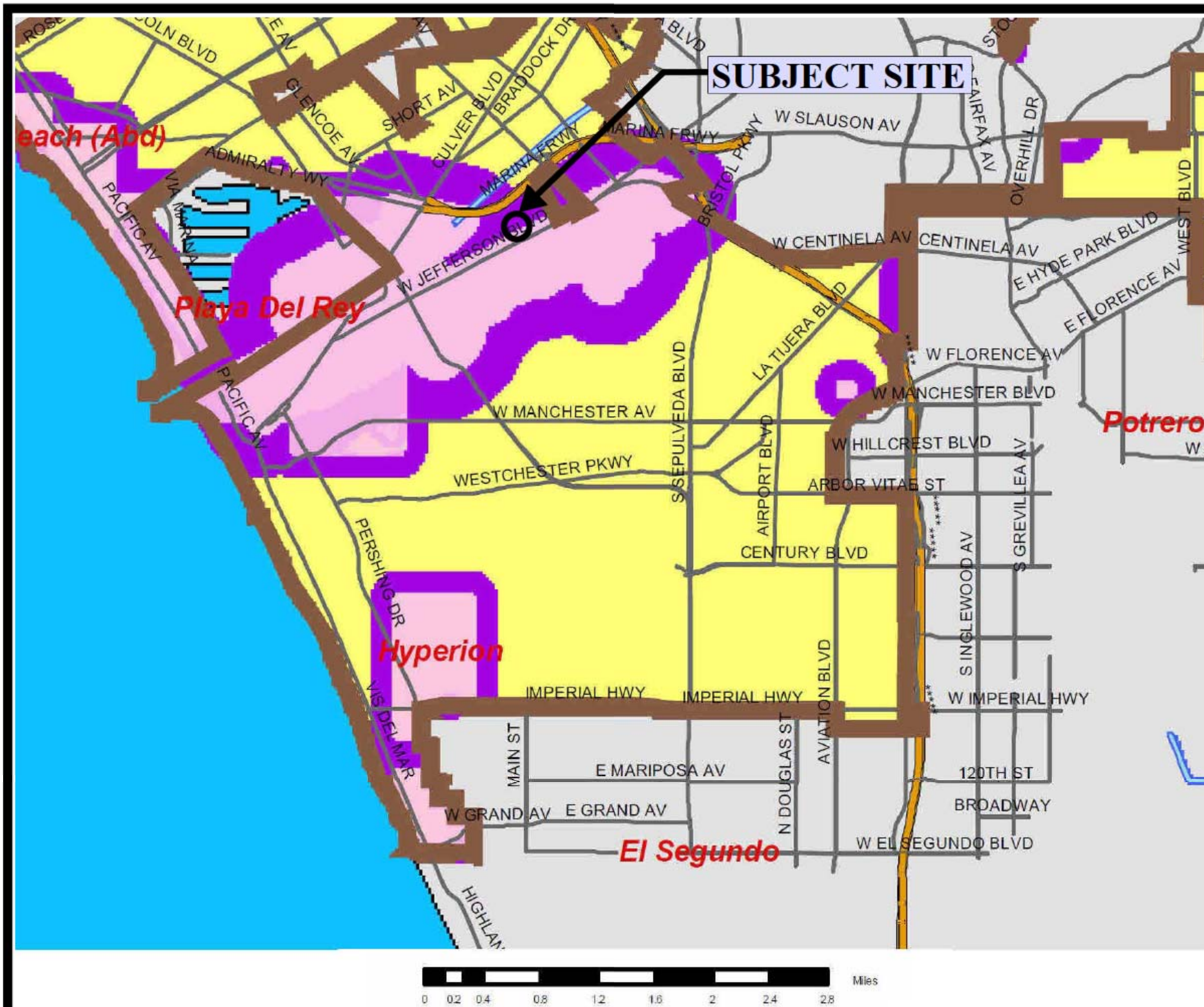
FILE NO. 21194



Methane Zone

Methane Buffer Zone

Council District Boundary

REFERENCE: GIS Mapping, Bureau of Engineering, Department fo Public Works - 09/24/03

METHANE ZONE RISK MAP

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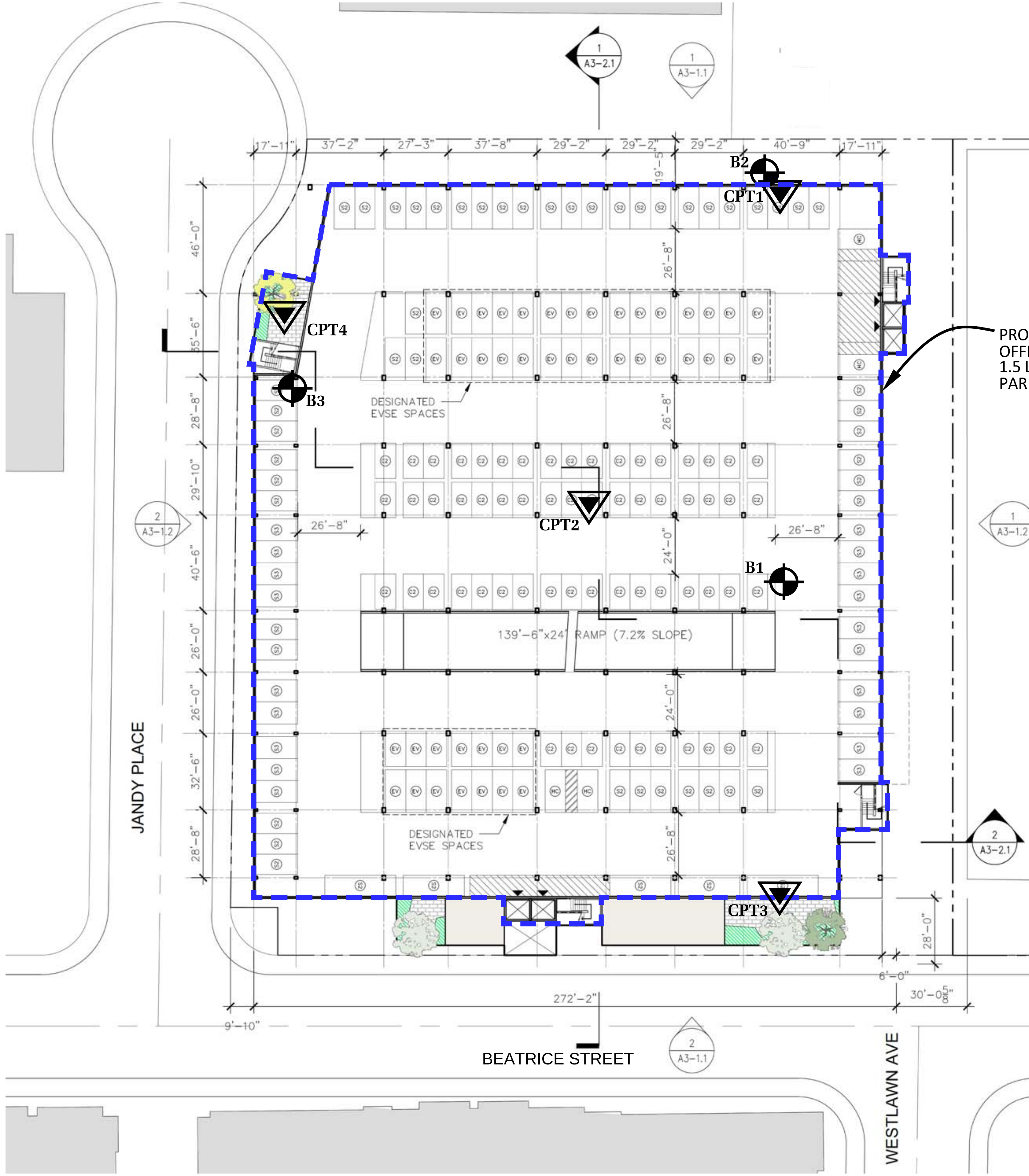
FILE NO. 21194



PROPOSED 6- TO 8-STORY OFFICE BUILDING OVER 1.5 LEVELS OF SUBTERRANEAN PARKING GARAGE

SCALE IN FEET

0 30 50 100

LEGEND

B3 LOCATION & NUMBER OF BORING

CPT4 LOCATION & NUMBER OF CONE PENETROMETER TEST PERFORMED BY GEOTECHNOLOGIES, INC.

LIMITS OF SUBTERRANEAN PARKING LEVEL

REFERENCE: FLOOR PLAN PARKING LEVEL 2 PROVIDED BY CLIENT DATED FEBRUARY 22, 2017

PLOT PLAN

Geotechnologies, Inc.
Consulting Geotechnical Engineers

CHAIT COMPANY ARCHITECTS
12575 BEATRICE ST., LOS ANGELES

FILE No. 21194	DRAWN BY: TC
DATE: March 2018	

BORING LOG NUMBER 1

Chait Company Architects **Date: 12/18/17**

File No. 21194 **Method: Used 5-inch diameter Rotary Wash Drill Rig**

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Asphalt for Parking
				0 --		4-inch Asphalt over 4-inch Base
				-		
				1 --		FILL: Sandy Clay, dark brown, moist, stiff
				-		
				2 --		
				-		
				3 --		Sandy Clay, dark and gray, moist, medium firm to stiff
				-		
				4 --		
				-		
5	12	26.2	SPT	5 --		
				-		
				6 --		
				-		
				7 --		
7.5	18	14.4	115.3	-		
				8 --		Sandy Silt, dark gray, moist, stiff
				-		
				9 --		
				-		
10	6	27.1	SPT	10 --		
				-		Silty Clay, dark gray, moist, medium firm to stiff
				11 --		
				-		
				12 --		
12.5	13	30.7	93.1	-		
				13 --	CH	Silty Clay, dark gray, very moist, stiff
				-		
				14 --		
				-		
15	6	40.2	SPT	15 --		
				-		Silty Clay, dark gray, very moist, soft to medium firm
				16 --		
				-		
				17 --		
17.5	9	29.1	93.3	-		
				18 --		
				-		
				19 --		
				-		
20	5	32.3	SPT	20 --		
				-		
				21 --		
				-		
				22 --		
22.5	12	30.7	87.4	-		
				23 --	CL	Silty Clay, dark gray, moist, medium firm
				-		
				24 --		
				-		
25	8	29.8	SPT	25 --		
				-		

BORING LOG NUMBER 1

Chait Company Architects

File No. 21194

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				26 --		
				27 --		
27.5	12	28.9	95.8	-		
				28 --	ML	Sandy Silt, dark gray, very moist, stiff
				29 --		
30	8	31.3	SPT	30 --		
				31 --		
				32 --		
32.5	26	23.4	103.0	-		
				33 --	SC	Clayey Sand, dark gray, wet, medium dense, fine grained
				34 --		
35	14	23.7	SPT	35 --		
				36 --		
				37 --		
37.5	53	16.9	112.6	-		
				38 --	SP/SW	Sand to Gravelly Sand, gray, wet, dense, fine to coarse grained
				39 --		
40	35	16.9	SPT	40 --		
				41 --		
				42 --		
42.5	43	12.9	112.1	-		
				43 --		
				44 --		
45	24	15.1	SPT	45 --		
				-	SP	Sand, dark gray, wet, medium dense, fine to medium grained, occasional gravel
				46 --		
				47 --		
47.5	22	25.2	96.2	-		
				48 --	SC/ML	Clayey Sand to Sandy Silt, dark gray, wet, medium dense to medium firm, fine grained
				49 --		
50	19	27.1	SPT	50 --		

BORING LOG NUMBER 1

Chait Company Architects

File No. 21194

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				51 --		
				52 --		
52.5	19	22.6	99.6	-		
				53 --	ML	Sandy Silt, dark gray, wet, medium firm, fine grained
				54 --		
55	11	34.2	SPT	55 --		
				-	SC	Clayey Sand, dark gray, wet, medium dense, fine grained
				56 --		
				57 --		
57.5	44	22.2	101.7	-		
				58 --	SP	Sand, dark gray, wet, dense, fine grained
				59 --		
60	73	15.5	SPT	60 --		
				61 --		
				62 --		
62.5	84	7.2	129.1	-		
				63 --	SW	Gravelly Sand, gray, wet, very dense, fine to coarse grained
				64 --		
65	91	9.0	SPT	65 --		
				-	SP	Sand, dark gray, wet, very dense, fine to medium grained, occasional gravel
				66 --		
				67 --		
67.5	39 50/4"	11.1	120.1	-		
				68 --		
				69 --		
70	80	19.6	SPT	70 --		
				-		Sand, dark gray, wet, very dense, fine grained
				71 --		
				72 --		
72.5	41 50/3"	17.9	108.4	-		
				73 --		
				74 --		
75	83	17.3	SPT	75 --		
				-		

BORING LOG NUMBER 1

Chait Company Architects

File No. 21194

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				-		
				76 --		
				-		
				77 --		
77.5	42	20.7	108.4	-		
	50/3"			78 --		Sand, gray, wet, very dense, fine grained
				-		
				79 --		
				-		
80	84	15.6	SPT	80 --		
				-		
				81 --		
				-		
				82 --		
82.5	40	17.7	112.7	-		
	50/3"			83 --		Sand, dark gray, wet, very dense, fine to medium grained
				-		
				84 --		
				-		
85	65	10.6	SPT	85 --		
				-		Sand, gray, wet, dense, fine grained
				86 --		
				-		
				87 --		
87.5	35	16.9	112.9	-		
	50/4"			88 --		Sand, gray to dark gray, wet, very dense, fine to medium grained
				-		
				89 --		
				-		
90	81	17.0	SPT	90 --		
				-		
				91 --		
				-		
				92 --		
92.5	39	16.0	113.9	-		
	50/3"			93 --		Sand, gray, wet, very dense, fine grained
				-		
				94 --		
				-		
95	71	18.9	SPT	95 --		
				-		
				96 --		
				-		
				97 --		
97.5	30	16.5	106.1	-		
	50/5"			98 --		
				-		
				99 --		
				-		
100	62	16.0	SPT	100 --		
				-		

BORING LOG NUMBER 1

Chait Company Architects

File No. 21194

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				-		
				101 --		
				-		
				102 --		
102.5	29	16.1	114.8	-		
	50/5"			103 --		Sand, gray, wet, very dense, fine grained
				-		
				104 --		
				-		
105	79	19.8	SPT	105 --		
				-		
				106 --		
				-		
				107 --		
107.5	40	20.5	106.3	-		
	50/3"			108 --		
				-		
				109 --		
				-		
110	34	14.6	SPT	110 --		
	50/5"			-		Sand, gray, wet, very dense, fine grained
				111 --		
				-		
				112 --		
112.5	100/9"	13.0	121.0	-		
				113 --		Sand, gray, wet, very dense, fine to medium grained
				-		
				114 --		
				-		
115	43	15.0	SPT	115 --		
	50/5.5"			-		
				116 --		
				-		
				117 --		
117.5	100/10"	14.0	121.2	-		
				118 --		
				-		
				119 --		
				-		
120	90	23.8	SPT	120 --		
				-		Total Depth 120 feet
				121 --		Water at 22½ feet
				-		Fill to 12½ feet
				122 --		
				-		
				123 --		NOTE: The stratification lines represent the approximate
				-		boundary between earth types; the transition may be gradual.
				124 --		
				-		Used 5-inch diameter Rotary Wash Drill Rig
				125 --		
				-		

BORING LOG NUMBER 2

Chait Company Architects **Date: 12/15/17**

File No. 21194 **Method: 5-inch diameter Rotary Wash Drill Rig**

km/ae

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Asphalt for Parking
				0 --		4-inch Asphalt over 4-inch Base
				-		
				1 --		FILL: Silty Sand to Sandy Silt, dark brown, moist, medium dense, fine grained, stiff
				-		
				2 --		
2.5	28	14.3	120.3	-		
				3 --		
				-		
				4 --		
				-		
5	17	19.6	SPT	5 --		
				-		Sandy Clay, dark brown, moist, medium firm to stiff, fine grained
				6 --		
				-		
				7 --		
7.5	19	16.7	112.7	-		
				8 --		
				-		
				9 --		
				-		
10	11	28.2	SPT	10 --		
				-		Silty Clay, gray, very moist, medium firm to stiff, fine grained
				11 --		
				-		
				12 --		
12.5	13	24.0	100.1	-		
				13 --	CL	Sandy Clay, gray to yellowish brown, very moist, stiff
				-		
				14 --		
				-		
15	6	45.9	SPT	15 --		
				-	CH	Silty Clay, dark gray, very moist, medium firm
				16 --		
				-		
				17 --		
17.5	8	41.0	80.7	-		
				18 --		
				-		
				19 --		
				-		
20	6	29.5	SPT	20 --		
				-		
				21 --		
				-		
				22 --		
22.5	11	27.1	97.4	-		
				23 --	CL	Sandy Clay, gray, very moist, soft to medium firm
				-		
				24 --		
				-		
25	5	31.2	SPT	25 --		
				-		

BORING LOG NUMBER 2

Chait Company Architects

File No. 21194

km/ae

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				26 --		
				27 --		
27.5	12	26.4	98.9	-		
				28 --		Sandy Clay, dark gray, moist, soft to medium firm
				29 --		
30	5	31.6	SPT	30 --		
				31 --		
				32 --		
32.5	30	23.4	101.1	-		
				33 --	SM/ML	Silty Sand to Sandy Silt, gray, wet, medium dense to stiff, fine grained
				34 --		
35	27	24.3	SPT	35 --		
				36 --		
				37 --		
37.5	52	8.3	128.4	-		
				38 --	SP/SW	Sand to Gravelly Sand, dark gray, wet, dense, fine to coarse grained
				39 --		
40	14	14.8	SPT	40 --		
				-	SM	Silty Sand, gray and dark brown, wet, medium dense, fine to coarse grained, with occasional gravel
				41 --		
				42 --		
42.5	23	16.9	116.8	-		
				43 --		
				44 --		
45	43	14.7	SPT	45 --		
				-	SP/SM	Sand to Silty Sand, gray to dark gray, wet, dense, fine to coarse grained, occasional gravel
				46 --		
				47 --		
47.5	57	9.3	132.1	-		
				48 --	SW	Gravelly Sand, gray, wet, dense, fine to coarse grained
				49 --		
50	21	21.4	SPT	50 --		
				-	SM	Silty Sand, gray to dark gray, wet, dense, fine to medium grained

BORING LOG NUMBER 2

Chait Company Architects

File No. 21194

km/ae

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				51 --		
				52 --		
52.5	18	45.9	77.1	-		
				53 --	CH	Silty Clay, dark gray, moist, stiff
				54 --		
55	36	21.8	SPT	55 --		
				-	SP	Sand, dark brown to light gray, wet, dense, fine to medium grained
				56 --		
				57 --		
57.5	86	9.2	123.3	-		
				58 --		
				59 --		
60	38	12.9	SPT	60 --		
				-		Sand, dark gray, wet, dense, fine to medium grained, with occasional cobbles
				61 --		
				62 --		
62.5	91	8.8	129.4	-		
				63 --		
				64 --		
65	71	15.2	SPT	65 --		
				-	SW	Sand to Gravelly Sand, dark to yellowish brown, wet, very dense, fine to medium grained, occasional cobbles
				66 --		
				67 --		
67.5	36 50/4"	14.1	112.5	-		
				68 --		
				69 --		
70	30 50/5"	13.1	SPT	70 --		
				-		Sand, gray to dark gray, wet, very dense, fine to medium grained, with gravel and cobbles
				71 --		
				72 --		
72.5	45 50/3"	20.0	105.8	-		
				73 --		
				74 --		
75	83	16.4	SPT	75 --		
				-		

BORING LOG NUMBER 2

Chait Company Architects

File No. 21194

km/ae

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				76		
				77		
77.5	53 50/2"	10.0	129.1	78		Sand, gray, wet, very dense, fine grained
				79		
80	31 50/5"	16.5	SPT	80		
				81		Total Depth 80 feet Water at 24 feet Fill to 12½ feet
				82		
				83		NOTE: The stratification lines represent the approximate boundary between earth types; the transition may be gradual.
				84		Used 5-inch diameter Rotary Wash Drill Rig
				85		
				86		
				87		
				88		
				89		
				90		
				91		
				92		
				93		
				94		
				95		
				96		
				97		
				98		
				99		
				100		

BORING LOG NUMBER 3

Chait Company Architects — **Date: 12/20/17**

File No. 21194 — **Method: 8-inch diameter Hollow Stem Auger**

km/ae

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Asphalt for Parking
				0		4-inch Asphalt over 4-inch Base
				1		FILL: Sandy Silt, dark brown, moist, stiff
				2		
2.5	25	11.3	117.9			
				3		Clayey Sand, dark grayish brown, moist, medium dense, fine grained
				4		
5	23	20.5	SPT	5		
				6		Sandy Silt to Silty Sand, dark gray, moist, stiff to medium dense, fine grained
				7		
7.5	26	15.7	115.2			
				8		Silty Sand to Sandy Clay, gray to dark gray, moist, medium dense to medium firm, fine grained
				9		
10	13	17.0	SPT	10		
						Sandy Clay, dark gray, moist, medium firm to stiff
				11		
				12		
12.5	14	32.6	84.8			
				13	CH	Silty Clay, dark brown, very moist, soft to medium firm
				14		
15	6	40.4	SPT	15		
				16		
				17		
17.5	5	44.6	77.4			
				18		Silty Clay, dark gray, very moist, soft
				19		
20	3	33.3	SPT	20		
				21		
				22		
22.5	9	32.4	88.9			
				23		
				24		
25	5	30.1	SPT	25		
					CL	Sandy Clay, dark gray, moist, soft to medium firm

BORING LOG NUMBER 3

Chait Company Architects

File No. 21194

km/ae

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				- 26 -- - 27 --		
27.5	18	28.5	96.8	- 28 -- - 29 -- -		Sandy Clay, dark gray, very moist, stiff
30	16	23.2	SPT	30 -- - 31 -- - 32 --	SM	Silty Sand, dark gray, wet, medium dense, fine grained
32.5	38	14.4	115.7	- 33 -- - 34 -- -	SP/SW	Sand to Gravelly Sand, dark to yellowish brown, wet, dense, fine to coarse grained
35	30	12.5	SPT	35 -- - 36 -- - 37 --		
37.5	51	10.5	127.5	- 38 -- - 39 -- -		Sand to Gravelly Sand, dark gray, wet, dense, fine to coarse grained
40	33	11.8	SPT	40 -- - 41 -- - 42 --		
42.5	24	19.7	109.5	- 43 -- - 44 -- -	SM	Silty Sand, dark gray, wet, medium dense, fine grained
45	14	24.5	SPT	45 -- - 46 -- - 47 --		
47.5	57	9.8	124.9	- 48 -- - 49 -- -	SP	Sand, gray to dark gray, wet, dense, fine to medium grained
50	14	23.6	SPT	50 -- -	CL	Sandy Clay, dark gray, wet, medium firm, fine grained

BORING LOG NUMBER 3

Chait Company Architects

File No. 21194

km/ae

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				-		
				51 --		
				-		
				52 --		
52.5	27	30.7	91.8	-		
				53 --		Sandy Clay, dark gray, wet, firm to stiff, fine grained
				-		
				54 --		
				-		
55	25	21.1	SPT	55 --		
				-	SP	Sand, gray to dark gray, wet, medium dense, fine grained
				56 --		
				-		
				57 --		
57.5	49	19.8	91.2	-		
				58 --		
				-		
				59 --		
				-		
60	24	17.3	SPT	60 --		
				-		
				61 --		
				-		
				62 --		
62.5	41	7.8	131.5	-		
				63 --		Sand, gray to dark gray, wet, dense, fine to medium grained
				-		
				64 --		
				-		
65	41	12.3	SPT	65 --		
	50/5"			-	SW	Sand to Gravelly Sand, gray to dark gray, wet, very dense, fine
				66 --		to medium grained, occasional cobbles
				-		
				67 --		
67.5	38	14.1	119.2	-		
	50/3"			68 --		
				-		
				69 --		
				-		
70	81	13.7	SPT	70 --		
				-		
				71 --		
				-		
				72 --		
72.5	41	21.2	107.6	-		
	50/3"			73 --		Sand to Gravelly Sand, gray to dark gray, wet, very dense, fine
				-		to medium grained, occasional gravel and cobbles
				74 --		
				-		
75	79	20.9	SPT	75 --		
				-		

BORING LOG NUMBER 3

Chait Company Architects

File No. 21194

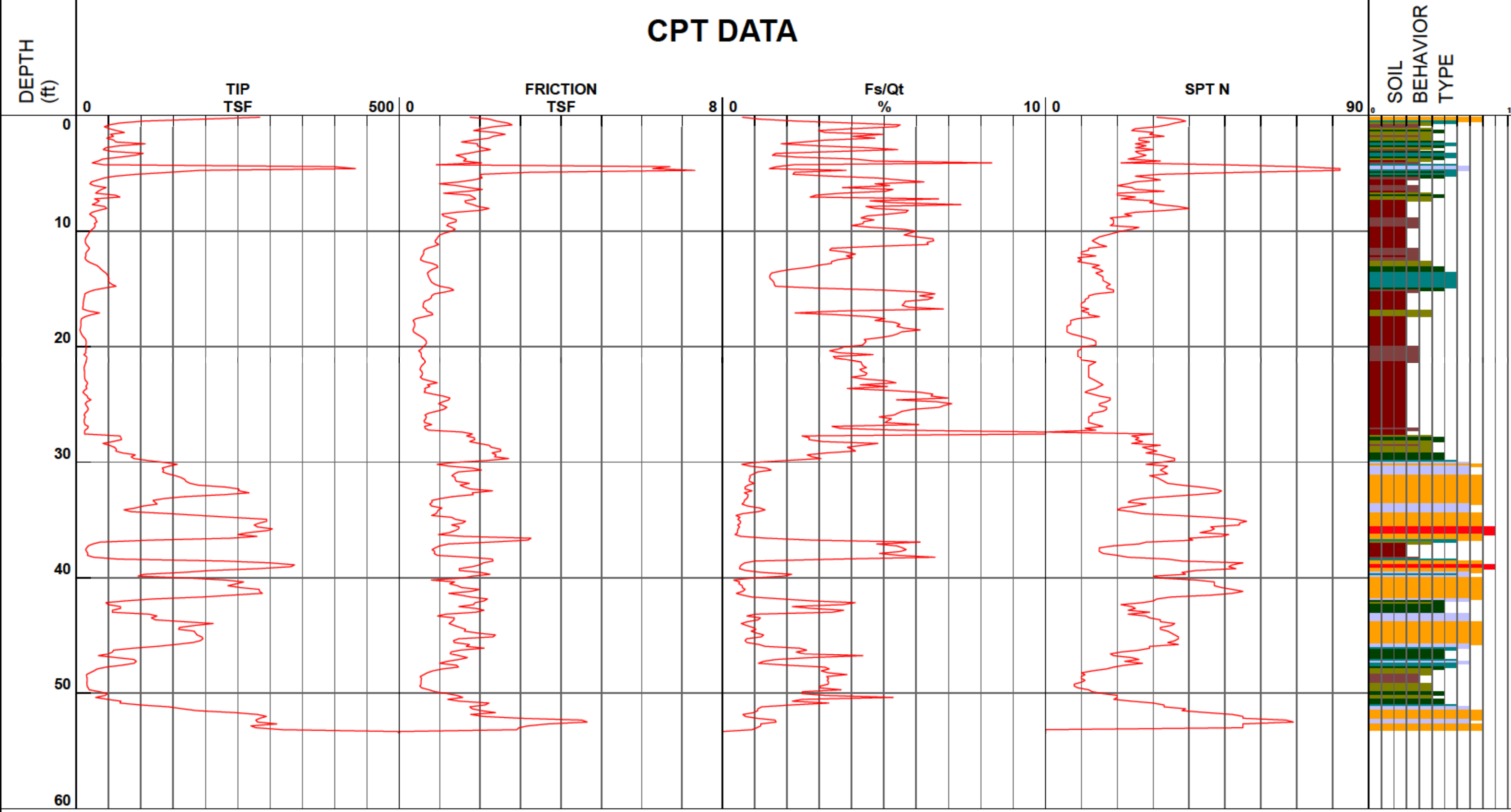
km/ae

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				- 76 -- - 77 --		
77.5	37 50/3"	17.9	108.9	- 78 -- - 79 -- -		Gravelly Sand, gray, very dense, fine to medium grained, with cobbles
80	40 50/5"	19.0	SPT	80 -- - 81 -- - 82 -- - 83 -- - 84 -- - 85 -- - 86 -- - 87 -- - 88 -- - 89 -- - 90 -- - 91 -- - 92 -- - 93 -- - 94 -- - 95 -- - 96 -- - 97 -- - 98 -- - 99 -- - 100 -- -		Total Depth 80 feet Water at 30 feet Fill to 12½ feet NOTE: The stratification lines represent the approximate boundary between earth types; the transition may be gradual. Used 5-inch diameter Rotary Wash Drill Rig

Geotechnologies Inc

Project	New Beatrice West Development	Operator	RC-DG	Filename	SDF(244).cpt
Job Number	21194	Cone Number	DDG1333	GPS	
Hole Number	CPT-01	Date and Time	3/17/2016 9:32:37 AM	Maximum Depth	53.48 ft
EST GW Depth During Test		20.00 ft			

Net Area Ratio .8



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

Cone Size 10cm squared

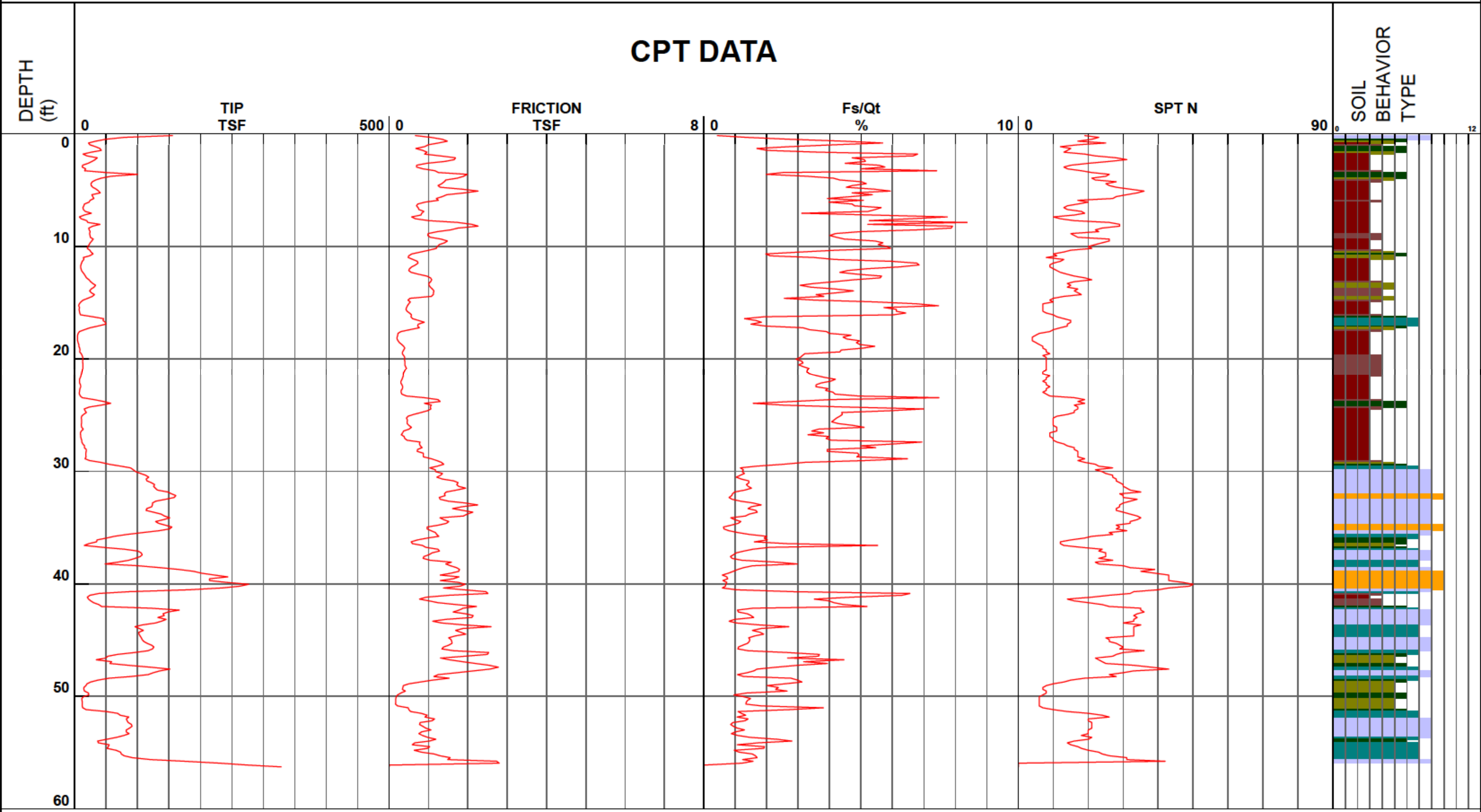
S*Soil behavior type and SPT based on data from UBC-1983

Geotechnologies Inc

Project	New Beatrice West Development	Operator	RC-DG	Filename	SDF(246).cpt
Job Number	21194	Cone Number	DDG1333	GPS	
Hole Number	CPT-02	Date and Time	3/17/2016 11:45:53 AM	Maximum Depth	56.27 ft
EST GW Depth During Test		20.00 ft			

Net Area Ratio .8

CPT DATA



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

Cone Size 10cm squared

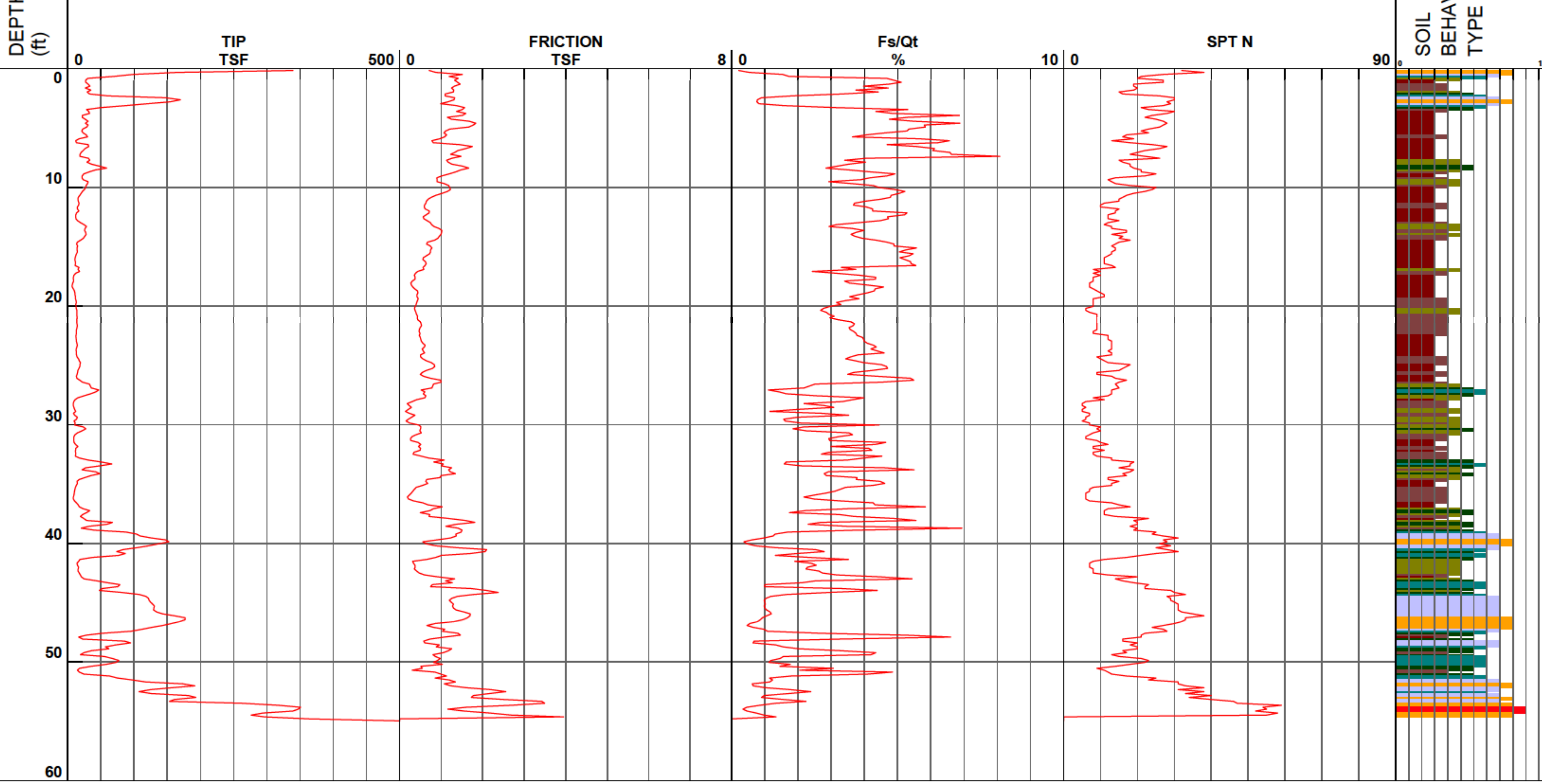
S*Soil behavior type and SPT based on data from UBC-1983

Geotechnologies Inc

Project	New Beatrice West Development	Operator	RC-DG	Filename	SDF(243).cpt
Job Number	21194	Cone Number	DDG1333	GPS	
Hole Number	CPT-03	Date and Time	3/17/2016 8:40:28 AM	Maximum Depth	54.95 ft
EST GW Depth During Test		20.00 ft			

Net Area Ratio .8

CPT DATA



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

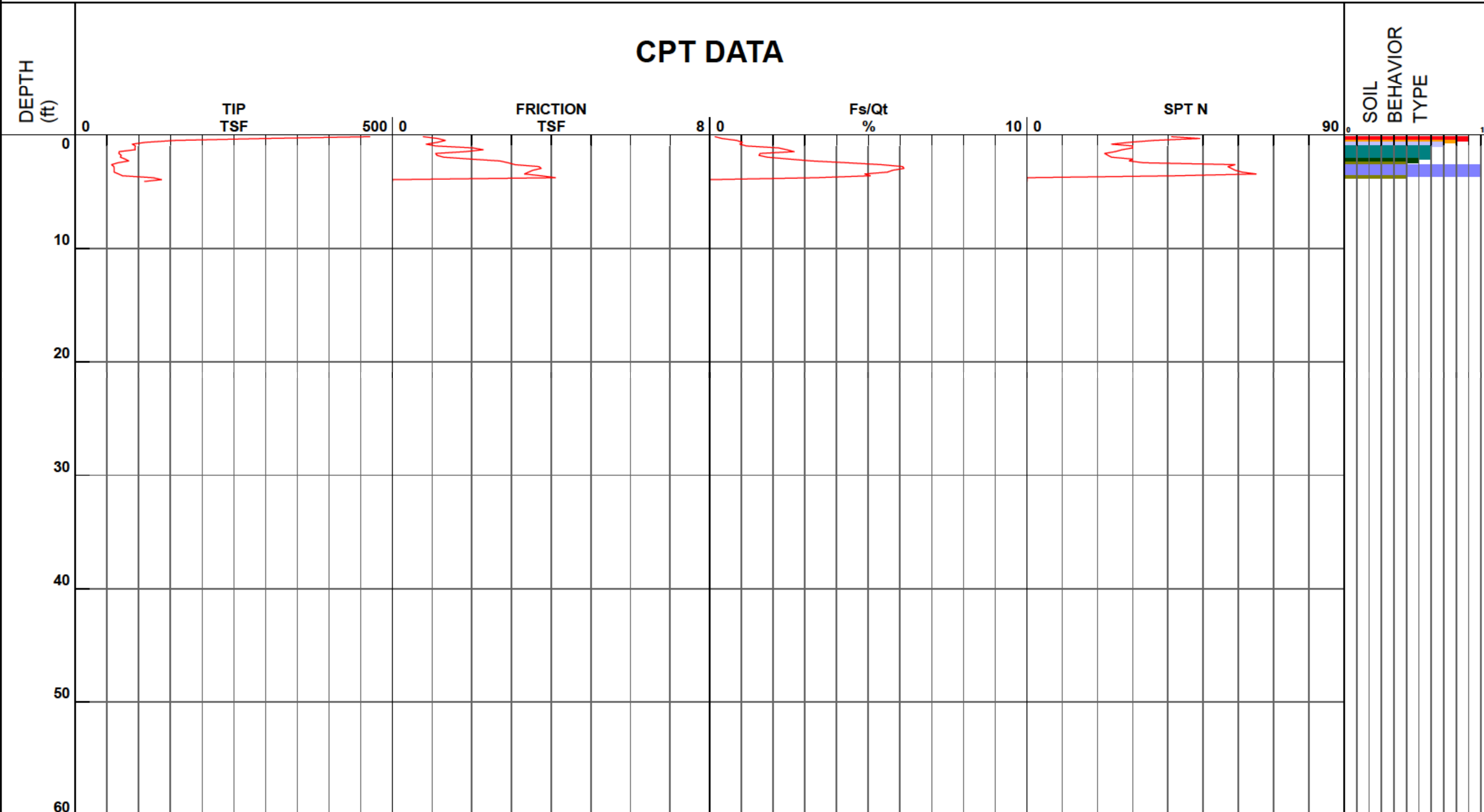
Cone Size 10cm squared

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

Geotechnologies Inc

Project	New Beatrice West Development	Operator	RC-DG	Filename	SDF(245).cpt
Job Number	21194	Cone Number	DDG1333	GPS	
Hole Number	CPT-04	Date and Time	3/17/2016 10:52:32 AM	Maximum Depth	4.10 ft
EST GW Depth During Test		20.00 ft			

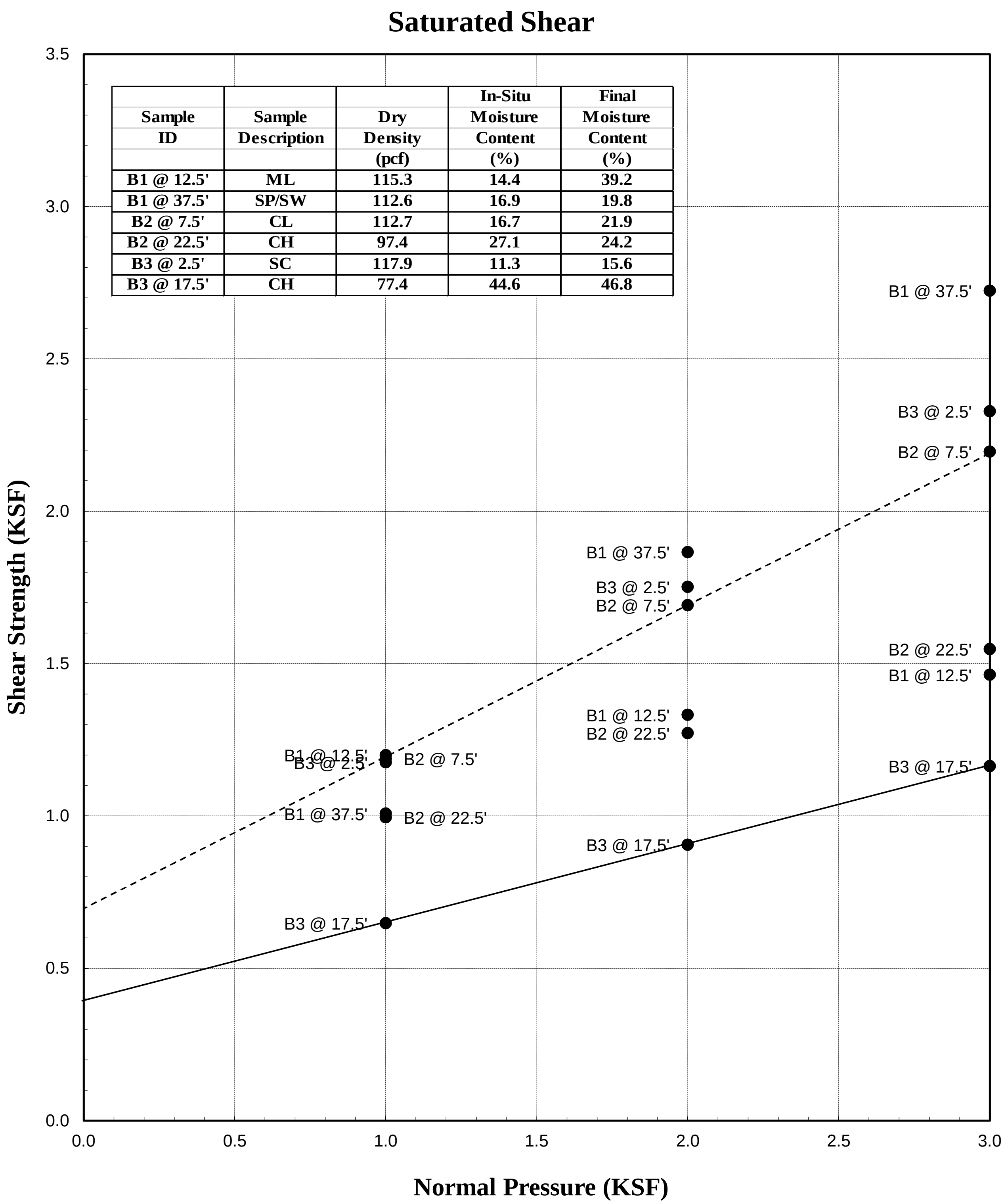
Net Area Ratio .8



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

Cone Size 10cm squared

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



Sample ID	Sample Description	Dry Density (pcf)	In-Situ Moisture Content (%)	Final Moisture Content (%)
B1 @ 12.5'	ML	115.3	14.4	39.2
B1 @ 37.5'	SP/SW	112.6	16.9	19.8
B2 @ 7.5'	CL	112.7	16.7	21.9
B2 @ 22.5'	CH	97.4	27.1	24.2
B3 @ 2.5'	SC	117.9	11.3	15.6
B3 @ 17.5'	CH	77.4	44.6	46.8

Exist Fill: ϕ: 26.5 degrees
c: 685.0 psf

Alluvium: ϕ: 14.5 degrees
c: 390.0 psf

SHEAR TEST DIAGRAM

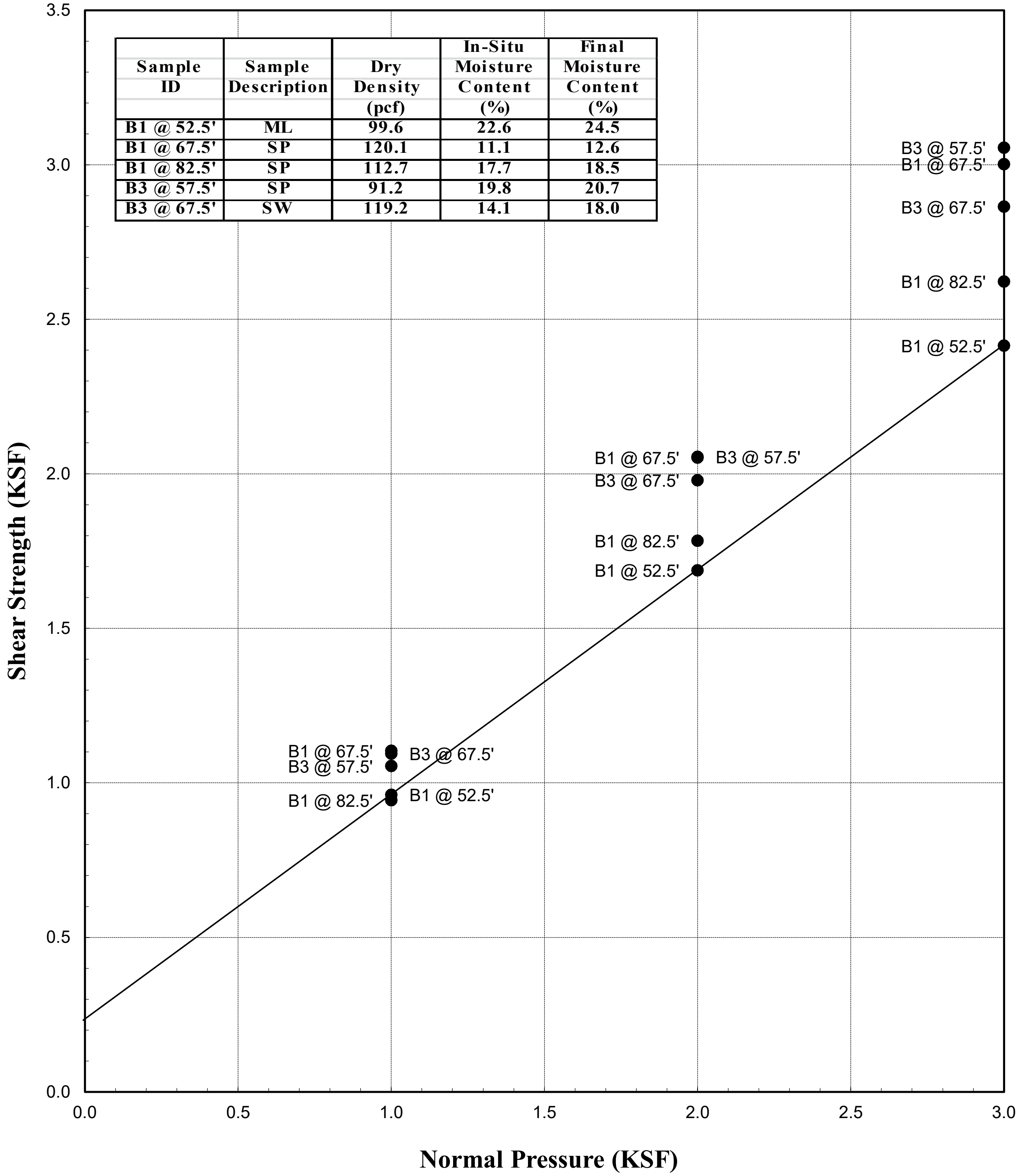
Geotechnologies, Inc.
Consulting Geotechnical Engineers

PROJECT: CHAIT COMPANY

FILE NO.: 21194

PLATE: B-1

Saturated Shear



Sample ID	Sample Description	Dry Density (pcf)	In-Situ Moisture Content (%)	Final Moisture Content (%)
B1 @ 52.5'	ML	99.6	22.6	24.5
B1 @ 67.5'	SP	120.1	11.1	12.6
B1 @ 82.5'	SP	112.7	17.7	18.5
B3 @ 57.5'	SP	91.2	19.8	20.7
B3 @ 67.5'	SW	119.2	14.1	18.0

ϕ: 36.0 degrees
c: 235.0 psf

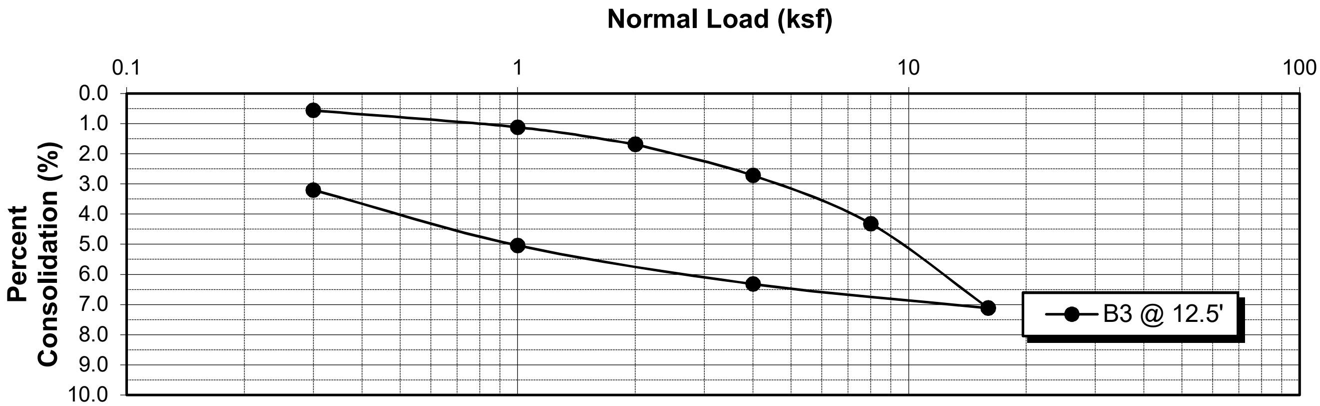
SHEAR TEST DIAGRAM

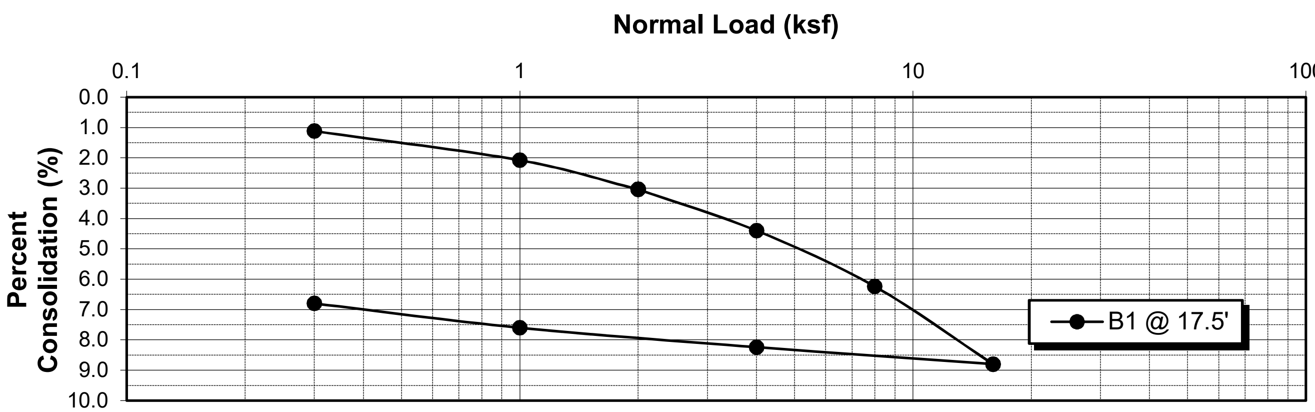
Geotechnologies, Inc.
Consulting Geotechnical Engineers

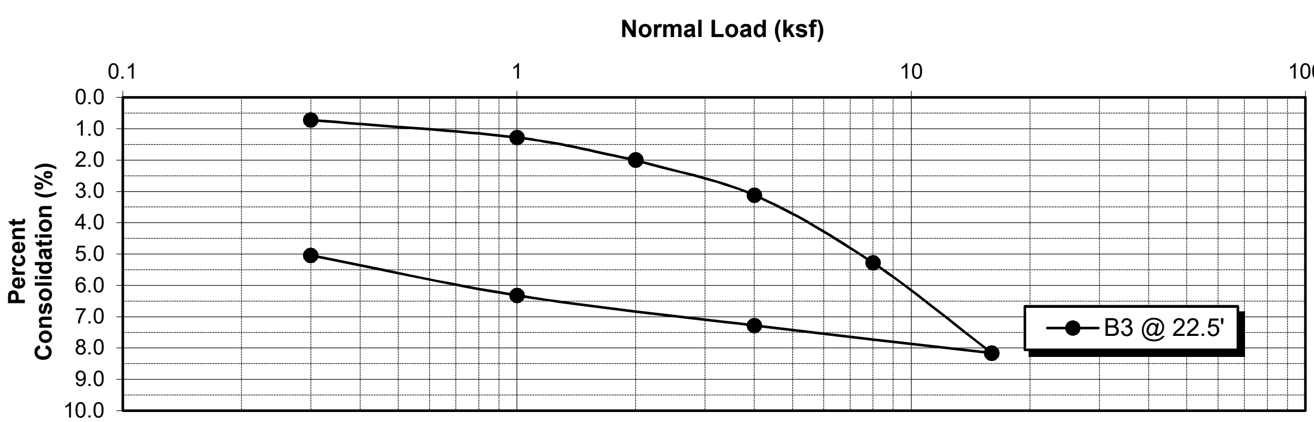
PROJECT: CHAIT COMPANY

FILE NO.: 21194

PLATE: B-2







Water added at 2 KSF

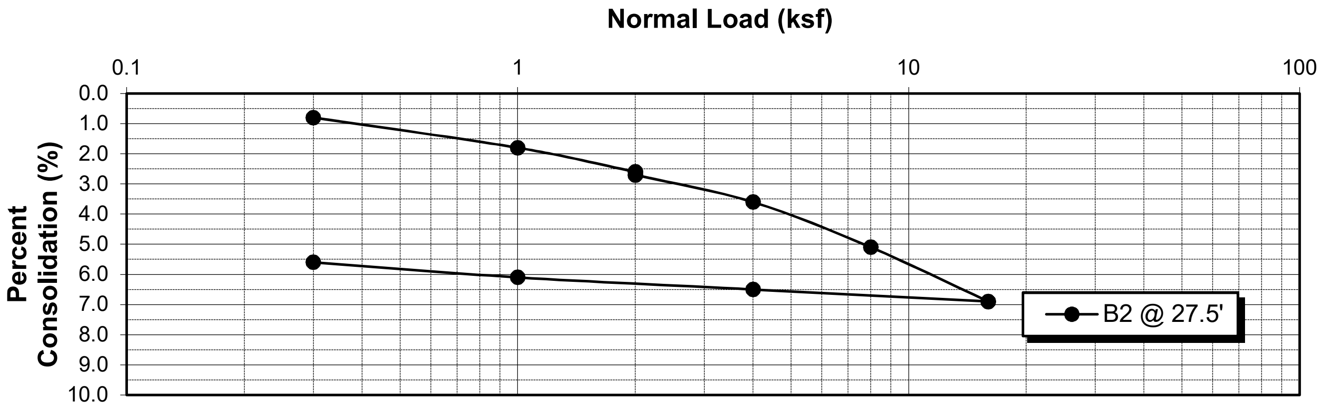
CONSOLIDATION

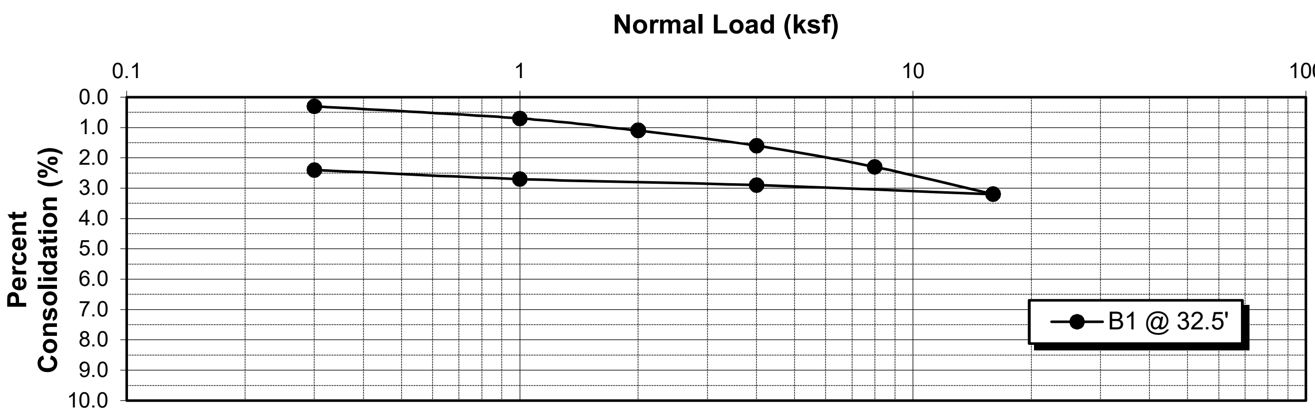
Geotechnologies, Inc.
CONSULTING GEOTECHNICAL ENGINEERS

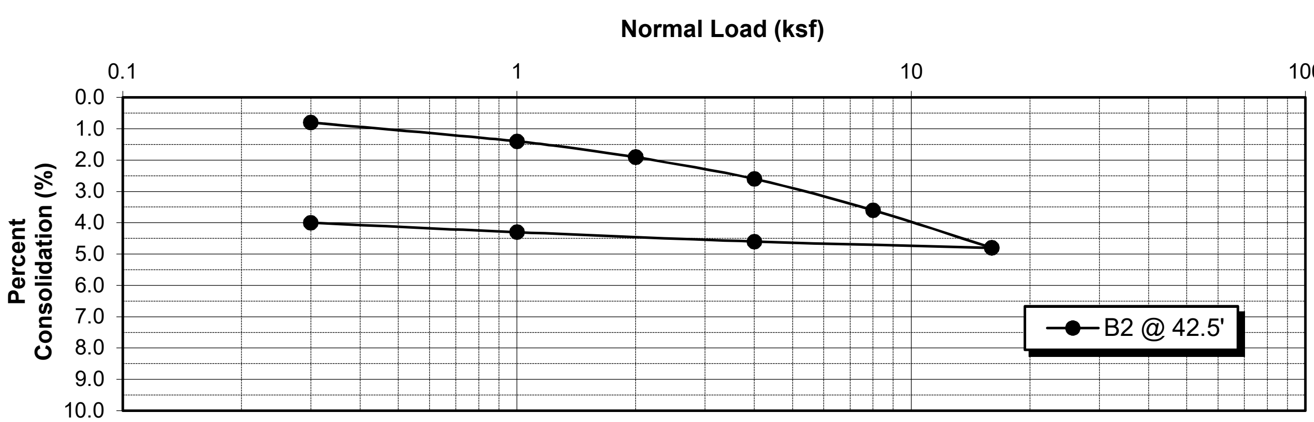
PROJECT: CHAIT COMPANY

FILE NO. 21194

PLATE: C-1







Water added at 2 KSF

CONSOLIDATION

Geotechnologies, Inc.

CONSULTING GEOTECHNICAL ENGINEERS

PROJECT: CHAIT COMPANY	
FILE NO. 21194	PLATE: C-2

ASTM D-1557

SAMPLE	B1 @ 1-5'	B3 @ 1-5'
SOIL TYPE:	SM/CL	SC/CL
MAXIMUM DENSITY pcf.	129.0	121.0
OPTIMUM MOISTURE %	10.0	13.5
PERCENT FINER THAN 0.005MM %	<15%	>15%

ASTM D 4829

SAMPLE	B1 @ 1-5'	B3 @ 1-5'
SOIL TYPE:	SM/CL	SC/CL
EXPANSION INDEX UBC STANDARD 18-2	35	95
EXPANSION CHARACTER	LOW	HIGH

SULFATE CONTENT

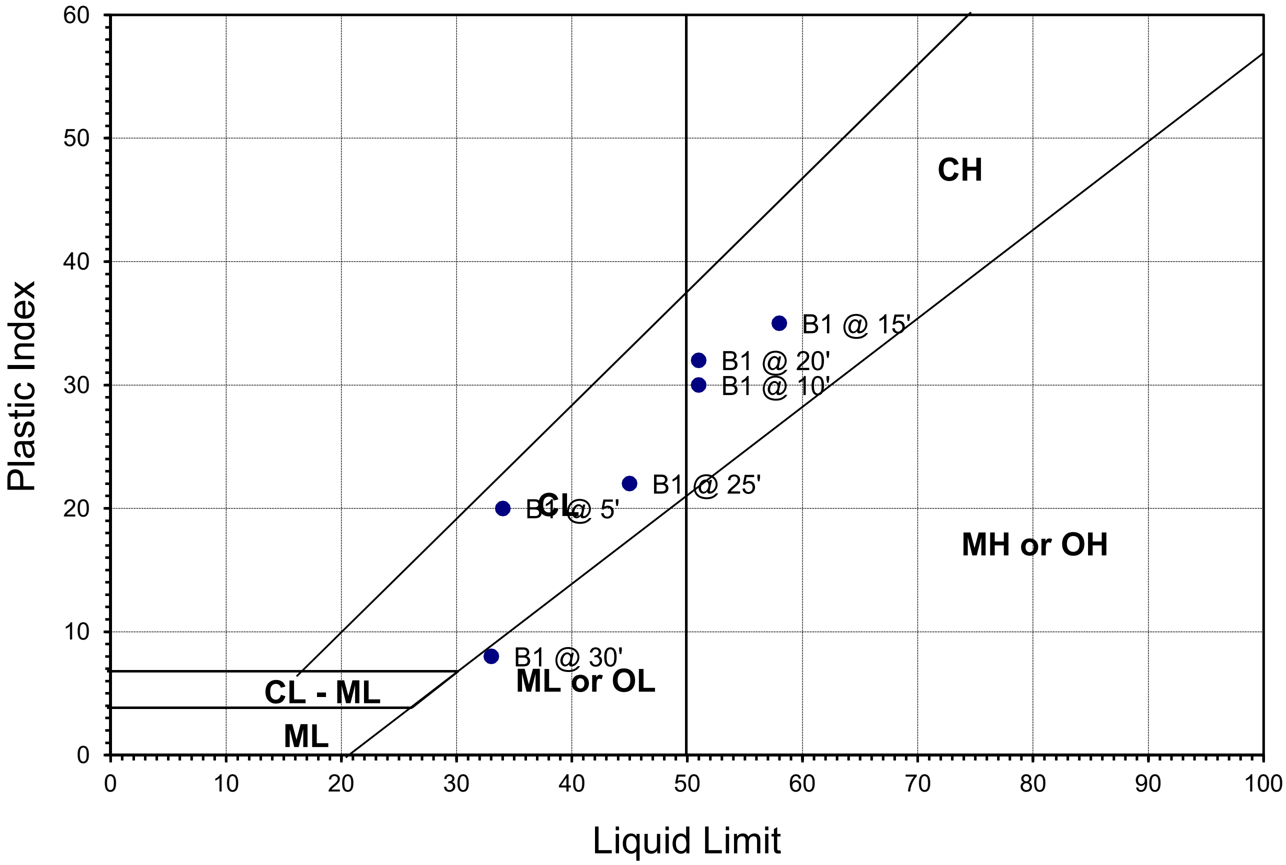
SAMPLE	B1 @ 1-5'	B3 @ 1-5'
SULFATE CONTENT: (ppm)	<250	<250

COMPACTION/EXPANSION/SULFATE DATA SHEET

Geotechnologies, Inc.
Consulting Geotechnical Engineers

CHAIT COMPANY ARCHITECTS

FILE NO. 21194 | PLATE: D



Sample ID	Descriptions	Passing #200	Liquid Limit	Plastic Limit	Plastic Index
B1 @ 5'	CL	62.6	34.0	14.0	20.0
B1 @ 10'	CH	84.8	51.0	21.0	30.0
B1 @ 15'	CH	89.7	58.0	23.0	35.0
B1 @ 20'	CH	81.2	51.0	19.0	32.0
B1 @ 25'	CL	80.2	45.0	23.0	22.0
B1 @ 30'	ML	79.8	33.0	25.0	8.0
B1 @ 35'	SC	31.1			
B1 @ 40'	SW	14.2			
B1 @ 50'	SC/ML	41.5			
B1 @ 55'	SC	46.7			

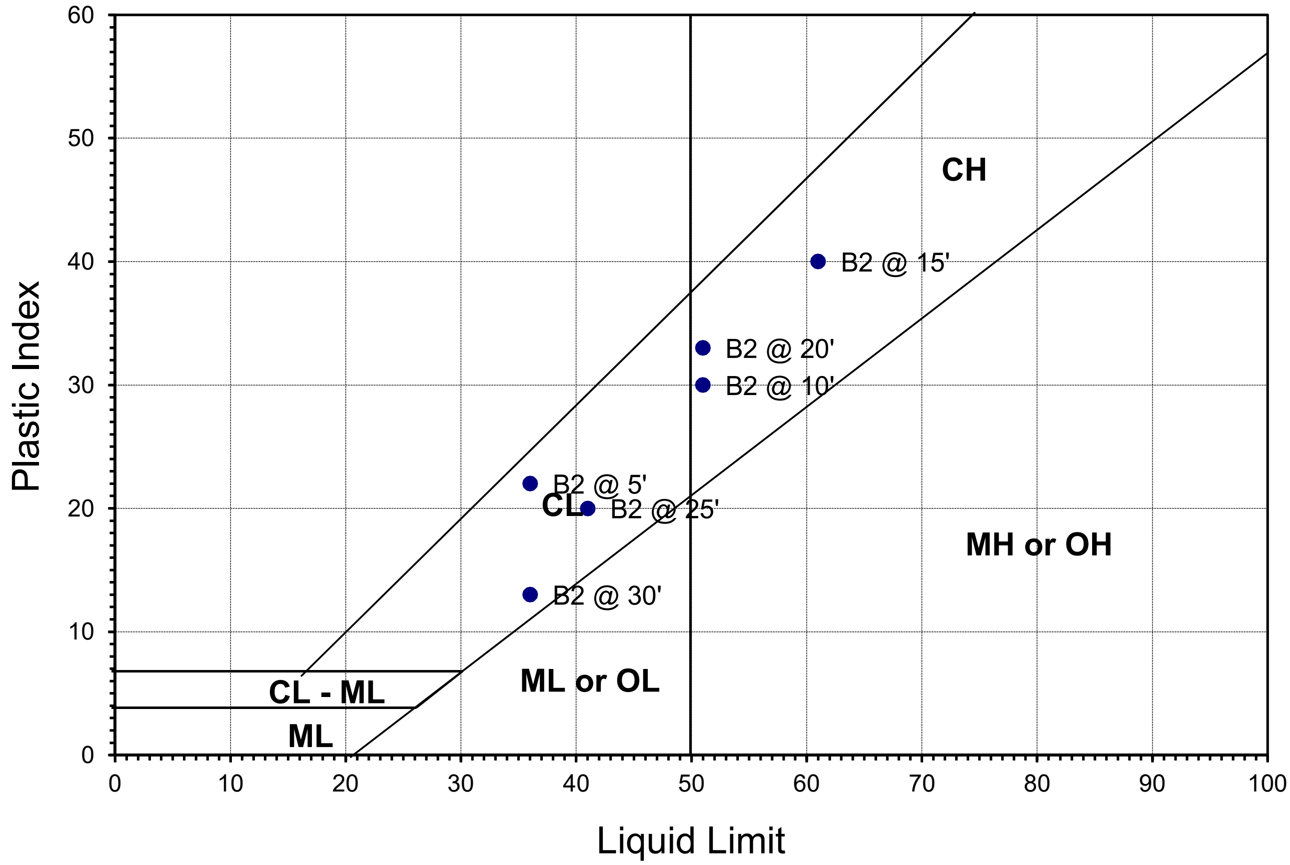
ATTERBERG LIMITS

Geotechnologies, Inc.
CONSULTING GEOTECHNICAL ENGINEERS

PROJECT: CHAIT COMPANY

FILE NO. 21194

PLATE: F-1



Sample ID	Descriptions	Passing #200	Liquid Limit	Plastic Limit	Plastic Index
B2 @ 5'	CL	59.2	36.0	14.0	22.0
B2 @ 10'	CH	80.3	51.0	21.0	30.0
B2 @ 15'	CH	93.5	61.0	21.0	40.0
B2 @ 20'	CH	78.6	51.0	18.0	33.0
B2 @ 25'	CL	91.3	41.0	21.0	20.0
B2 @ 30'	CL	82.8	36.0	23.0	13.0
B2 @ 40'	SM	23.0			
B2 @ 50'	SM	32.1			

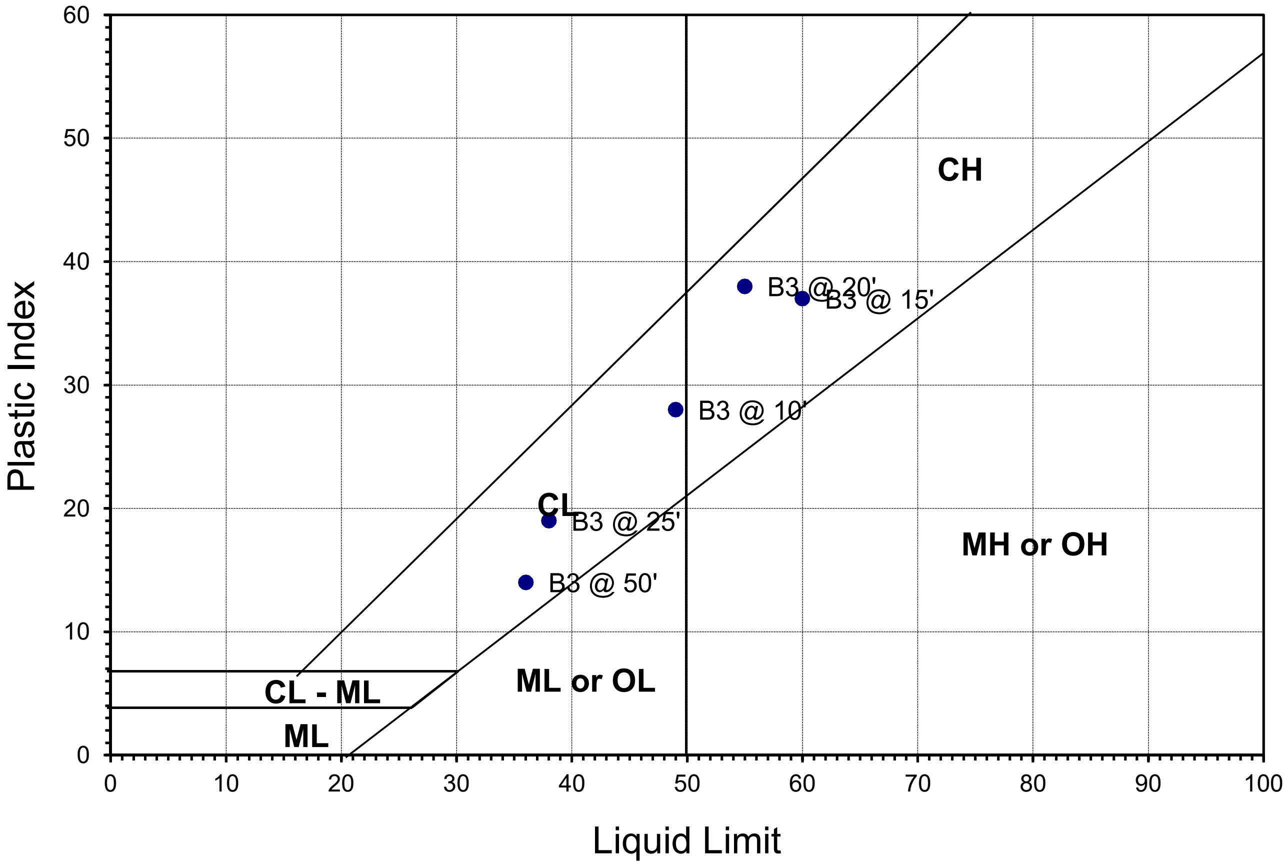
ATTERBERG LIMITS

Geotechnologies, Inc.
CONSULTING GEOTECHNICAL ENGINEERS

PROJECT: CHAIT COMPANY

FILE NO. 21194

PLATE: F-2



Sample ID	Descriptions	Passing #200	Liquid Limit	Plastic Limit	Plastic Index
B3 @ 10'	CL	51.6	49.0	21.0	28.0
B3 @ 15'	CH	78.8	60.0	23.0	37.0
B3 @ 20'	CH	85.7	55.0	17.0	38.0
B3 @ 25'	CL	88.7	38.0	19.0	19.0
B3 @ 30'	SM	30.7			
B3 @ 45'	SM	17.4			
B3 @ 50'	CL	53.0	36.0	22.0	14.0
B3 @ 55'	SP	9.6			
B3 @ 60'	SP	5.5			

ATTERBERG LIMITS

Geotechnologies, Inc.
CONSULTING GEOTECHNICAL ENGINEERS

PROJECT: CHAIT COMPANY

FILE NO. 21194 | PLATE: F-3

Geotechnologies, Inc.

Project: Chait Company
File No.: 21194
Description: Liquefaction Analysis
Boring Numb 1

LIQUEFACTION EVALUATION (Idriss & Boulanger, EERI NO 12)

EARTHQUAKE INFORMATION:

Earthquake Magnitude (M):	6.7
Peak Ground Horizontal Acceleration, PGA (g):	0.88
Calculated Mag.Wtg.Factor:	1.234

GROUNDWATER INFORMATION:

Current Groundwater Level (ft):	22.5
Historically Highest Groundwater Level* (ft):	15.0
Unit Weight of Water (pcf):	62.4

* Based on California Geological Survey Seismic Hazard Evaluation Report

BOREHOLE AND SAMPLER INFORMATION:

Borehole Diameter (inches):	5
SPT Sampler with room for Liner (Y/N):	Y

LIQUEFACTION BOUNDARY:

Plastic Index Cut Off (PI):	18
Minimum Liquefaction FS:	1

Depth to Base Layer (feet)	Total Unit Weight (pcf)	Current Water Level (feet)	Historical Water Level (feet)	Field SPT Blowcount N	Depth of SPT Blowcount (feet)	Fines Content #200 Sieve (%)	Plastic Index (PI)	Vetical Stress σ_{vc}, (psf)	Effective Vert. Stress σ_{vc}', (psf)	Fines Corrected $(N_1)_{60-cs}$	Stress Reduction Coeff, r_d	Cyclic Shear Ratio CSR	Cyclic Resistance Ratio (CRR)	Factor of Safety CRR/CSR (F.S.)	Liquefaction Settlment ΔS_i (inches)
1	132.0	Unsaturated	Unsaturated	12	5	62.6	20	132.0	132.0	32.4	1.00	0.574	0.936	Non-Liq.	0.00
2	132.0	Unsaturated	Unsaturated	12	5	62.6	20	264.0	264.0	32.4	1.00	0.572	0.936	Non-Liq.	0.00
3	132.0	Unsaturated	Unsaturated	12	5	62.6	20	396.0	396.0	32.4	1.00	0.571	0.936	Non-Liq.	0.00
4	132.0	Unsaturated	Unsaturated	12	5	62.6	20	528.0	528.0	32.4	0.99	0.568	0.936	Non-Liq.	0.00
5	132.0	Unsaturated	Unsaturated	12	5	62.6	20	660.0	660.0	34.6	0.99	0.566	1.376	Non-Liq.	0.00
6	132.0	Unsaturated	Unsaturated	12	5	62.6	20	792.0	792.0	32.4	0.99	0.564	0.935	Non-Liq.	0.00
7	132.0	Unsaturated	Unsaturated	12	5	62.6	20	924.0	924.0	30.2	0.98	0.562	0.679	Non-Liq.	0.00
8	132.0	Unsaturated	Unsaturated	12	5	62.6	20	1056.0	1056.0	28.5	0.98	0.559	0.550	Non-Liq.	0.00
9	132.0	Unsaturated	Unsaturated	12	5	62.6	20	1188.0	1188.0	28.5	0.97	0.557	0.549	Non-Liq.	0.00
10	132.0	Unsaturated	Unsaturated	12	5	62.6	20	1320.0	1320.0	27.2	0.97	0.554	0.474	Non-Liq.	0.00
11	132.0	Unsaturated	Unsaturated	6	10	84.8	30	1452.0	1452.0	15.5	0.96	0.552	0.206	Non-Liq.	0.00
12	132.0	Unsaturated	Unsaturated	6	10	84.8	30	1584.0	1584.0	15.0	0.96	0.549	0.199	Non-Liq.	0.00
13	121.7	Unsaturated	Unsaturated	6	15	89.7	35	1705.7	1705.7	14.6	0.96	0.546	0.193	Non-Liq.	0.00
14	121.7	Unsaturated	Unsaturated	6	15	89.7	35	1827.4	1827.4	14.3	0.95	0.543	0.188	Non-Liq.	0.00
15	121.7	Unsaturated	Unsaturated	6	15	89.7	35	1949.1	1949.1	14.9	0.95	0.541	0.194	Non-Liq.	0.00
16	121.7	Unsaturated	Saturated	6	15	89.7	35	2070.8	2008.4	14.8	0.94	0.554	0.191	Non-Liq.	0.00
17	121.7	Unsaturated	Saturated	6	15	89.7	35	2192.5	2067.7	14.6	0.93	0.567	0.189	Non-Liq.	0.00
18	120.4	Unsaturated	Saturated	6	15	89.7	35	2312.9	2125.7	14.5	0.93	0.578	0.187	Non-Liq.	0.00
19	120.4	Unsaturated	Saturated	6	15	89.7	35	2433.3	2183.7	14.4	0.92	0.589	0.185	Non-Liq.	0.00
20	120.4	Unsaturated	Saturated	6	15	89.7	35	2553.7	2241.7	14.2	0.92	0.599	0.184	Non-Liq.	0.00
21	120.4	Unsaturated	Saturated	5	20	81.2	32	2674.1	2299.7	12.7	0.91	0.607	0.168	Non-Liq.	0.00
22	120.4	Unsaturated	Saturated	5	20	81.2	32	2794.5	2357.7	12.6	0.91	0.615	0.167	Non-Liq.	0.00
23	114.3	Saturated	Saturated	8	25	80.2	22	2908.8	2409.6	16.8	0.90	0.623	0.209	Non-Liq.	0.00
24	114.3	Saturated	Saturated	8	25	80.2	22	3023.1	2461.5	16.6	0.90	0.629	0.207	Non-Liq.	0.00
25	114.3	Saturated	Saturated	8	25	80.2	22	3137.4	2513.4	16.5	0.89	0.636	0.205	Non-Liq.	0.00
26	114.3	Saturated	Saturated	8	25	80.2	22	3251.7	2565.3	16.4	0.88	0.641	0.203	Non-Liq.	0.00
27	114.3	Saturated	Saturated	8	25	80.2	22	3366.0	2617.2	16.3	0.88	0.646	0.201	Non-Liq.	0.00
28	123.5	Saturated	Saturated	8	30	79.8	8	3489.5	2678.3	16.7	0.87	0.650	0.206	0.3	0.32
29	123.5	Saturated	Saturated	8	30	79.8	8	3613.0	2739.4	16.6	0.87	0.654	0.204	0.3	0.32
30	123.5	Saturated	Saturated	8	30	79.8	8	3736.5	2800.5	16.5	0.86	0.657	0.202	0.3	0.32
31	123.5	Saturated	Saturated	8	30	79.8	8	3860.0	2861.6	16.4	0.85	0.659	0.200	0.3	0.32
32	123.5	Saturated	Saturated	8	30	79.8	8	3983.5	2922.7	16.3	0.85	0.661	0.198	0.3	0.33
33	127.1	Saturated	Saturated	14	35	31.1	0	4110.6	2987.4	25.8	0.84	0.663	0.360	0.5	0.22
34	127.1	Saturated	Saturated	14	35	31.1	0	4237.7	3052.1	25.6	0.84	0.664	0.353	0.5	0.22
35	127.1	Saturated	Saturated	14	35	31.1	0	4364.8	3116.8	25.4	0.83	0.664	0.346	0.5	0.22
36	127.1	Saturated	Saturated	14	35	31.1	0	4491.9	3181.5	25.2	0.82	0.665	0.339	0.5	0.23
37	127.1	Saturated	Saturated	14	35	31.1	0	4619.0	3246.2	25.0	0.82	0.665	0.333	0.5	0.23
38	131.6	Saturated	Saturated	35	40	14.2	0	4750.6	3315.4	61.0	0.81	0.665	2.000	3.0	0.00
39	131.6	Saturated	Saturated	35	40	14.2	0	4882.2	3384.6	60.7	0.80	0.664	2.000	3.0	0.00
40	131.6	Saturated	Saturated	35	40	14.2	0	5013.8	3453.8	60.4	0.80	0.663	2.000	3.0	0.00
41	131.6	Saturated	Saturated	35	40	14.2	0	5145.4	3523.0	60.1	0.79	0.662	2.000	3.0	0.00
42	131.6	Saturated	Saturated	35	40	14.2	0	5277.0	3592.2	59.8	0.79	0.661	2.000	3.0	0.00
43	126.6	Saturated	Saturated	35	40	14.2	0	5403.6	3656.4	59.6	0.78	0.659	2.000	3.0	0.00
44	126.6	Saturated	Saturated	35	40	14.2	0	5530.2	3720.6	59.3	0.77	0.658	2.000	3.0	0.00
45	126.6	Saturated	Saturated	35	40	14.2	0	5656.8	3784.8	59.1	0.77	0.656	2.000	3.0	0.00
46	126.6	Saturated	Saturated	24	45	0.0	0	5783.4	3849.0	37.1	0.76	0.655	1.831	2.8	0.00
47	126.6	Saturated	Saturated	24	45	0.0	0	5910.0	3913.2	36.9	0.76	0.653	1.721	2.6	0.00
48	120.5	Saturated	Saturated	19	50	41.5	0	6030.5	3971.3	32.1	0.75	0.651	0.692	1.1	0.00
49	120.5	Saturated	Saturated	19	50	41.5	0	6151.0	4029.4	31.9	0.74	0.649	0.672	1.0	0.00
50	120.5	Saturated	Saturated	19	50	41.5	0	6271.5	4087.5	31.8	0.74	0.647	0.654	1.0	0.00
51	120.5	Saturated	Saturated	19	50	41.5	0	6392.0	4145.6	31.6	0.73	0.645	0.637	1.0	0.08
52	120.5	Saturated	Saturated	19	50	41.5	0	6512.5	4203.7	31.4	0.73	0.643	0.621	1.0	0.08
53	122.2	Saturated	Saturated	19	50	41.5	0	6634.7	4263.5	31.3	0.72	0.641	0.606	0.9	0.09
54	122.2	Saturated	Saturated	19	50	41.5	0	6756.9	4323.3	31.1	0.71	0.639	0.591	0.9	0.09
55	122.2	Saturated	Saturated	19	50	41.5	0	6879.1	4383.1	31.0	0.71	0.636	0.577	0.9	0.09
56	122.2	Saturated	Saturated	11	55	46.7	0	7001.3	4442.9	18.0	0.70	0.634	0.206	0.3	0.30
57	122.2	Saturated	Saturated	11	55	46.7	0	7123.5	4502.7	17.9	0.70	0.631	0.205	0.3	0.30
58	124.3	Saturated	Saturated	73	60	0.0	0	7247.8	4564.6	111.3	0.69	0.628	1.907	3.0	0.00
59	124.3	Saturated	Saturated	73	60	0.0	0	7372.1	4626.5	111.0	0.69	0.626	1.897	3.0	0.00
60	124.3	Saturated	Saturated	73	60	0.0	0	7496.4	4688.4	110.6	0.68	0.623	1.887	3.0	0.00
61	124.3	Saturated	Saturated	73	60	0.0	0	7620.7	4750.3	110.2	0.68	0.620	1.877	3.0	0.00
62	124.3	Saturated	Saturated	73	60	0.0	0	7745.0	4812.2	109.8	0.67	0.617	1.868	3.0	0.00
63	138.3	Saturated	Saturated	73	60	0.0	0	7883.3	4888.1	109.4	0.67	0.614	1.857	3.0	0.00
64	138.3	Saturated	Saturated	73	60	0.0	0	8021.6	4964.0	108.9	0.66	0.610	1.845	3.0	0.00
65	138.3	Saturated	Saturated	73	60	0.0	0	8159.9	5039.9	108.5	0.65	0.606	1.834	3.0	0.00
66	138.3	Saturated	Saturated	91	65	0.0	0	8298.2	5115.8	134.7	0.65	0.603	1.823	3.0	0.00
67	138.3	Saturated	Saturated	91	65	0.0	0	8436.5	5191.7	134.2	0.64	0.600	1.813	3.0	0.00
68	133.4	Saturated	Saturated	91	65	0.0	0	8569.9	5262.7	133.7	0.64	0.596	1.803	3.0	0.00
69	133.4	Saturated	Saturated	91	65	0.0	0	8703.3	5333.7	133.2	0.64	0.593	1.793	3.0	0.00
70	133.4	Saturated	Saturated	91	65	0.0	0	8836.7	5404.7	132.8	0.63	0.590	1.783	3.0	0.00
71	133.4	Saturated	Saturated	80	70	0.0	0	8970.1	5475.7	116.3	0.63	0.587	1.774	3.0	0.00
72	133.4	Saturated	Saturated	80	70	0.0	0	9103.5	5546.7	115.9	0.62	0.584	1.765	3.0	0.00
73	127.8	Saturated	Saturated	80	70	0.0	0	9231.3	5612.1	115.6	0.62	0.581	1.756	3.0	0.00
74	127.8	Saturated	Saturated	80	70	0.0	0	9359.1	5677.5	115.2	0.61	0.578	1.748	3.0	0.00
75	127.8	Saturated	Saturated	80	70	0.0	0	9486.9	5742.9	114.9	0.61	0.575	1.739	3.0	0.00
76	127.8	Saturated	Saturated	83	75	0.0	0	9614.7	5808.3	118.8	0.60	0.572	1.731	3.0	0.00
77	127.8	Saturated	Saturated	83	75	0.0	0	9742.5	5873.7	118.5	0.60	0.570	1.723	3.0	0.00
78	130.7	Saturated	Saturated	83	75	0.0	0	9873.2	5942.0	118.1	0.60	0.567	1.714	3.0	0.00
79	130.7	Saturated	Saturated	83	75	0.0	0	10003.9	6010.3	117.8	0.59	0.564	1.706	3.0	0.00
80	130.7	Saturated	Saturated	83	75	0.0	0	10134.6	6078.6	117.4	0.59	0.562	1.698	3.0	0.00
81	130.7	Saturated	Saturated	84	80	0.0	0	10265.3	6146.9	118.5	0.59	0.559	1.690	3.0	0.00
82	130.7	Saturated	Saturated	84	80	0.0	0	10396.0	6215.2	118.1	0.58	0.556	1.682	3.0	0.00
83	132.6	Saturated	Saturated	84	80	0.0	0	10528.6	6285.4	117.8	0.58	0.554	1.673	3.0	0.00
84	132.6	Saturated	Saturated	84	80	0.0	0	10661.2	6355.6	117.4	0.57	0.551	1.665	3.0	0.00
85	132.6	Saturated	Saturated	84	80	0.0	0	10793.8	6425.8	117.1	0.57	0.549	1.657	3.0	0.00
86	132.6	Saturated	Saturated	65	85	0.0	0	10926.4	6496.0	90.4	0.57	0.547	1.649	3.0	0.00
87	132.6	Saturated	Saturated	65	85	0.0	0	11059.0	6566.2	90.1	0.56	0.544	1.642	3.0	0.00
88	132.0	Saturated	Saturated	65	85	0.0	0	11191.0	6635.8	89.8	0.56	0.542	1.634	3.0	0.00
89	132.0	Saturated	Saturated	65	85	0.0	0	11323.0	6705.4	89.6	0.56	0.540	1.626	3.0	0.00
90	132.0	Saturated	Saturated	65	85	0.0	0	11455.0	6775.0	89.4	0.56	0.538	1.619	3.0	0.00
91	132.0	Saturated	Saturated	81	90	0.0	0	11587.0	6844.6	111.1	0.55	0.536	1.611	3.0	0.00
92	132.0	Saturated	Saturated	81	90	0.0	0	11719.0	6914.2	110.8	0.55	0.534	1.604	3.0	0.00
93	132.1	Saturated	Saturated	81	90	0.0	0	11851.1	6983.9	110.5	0.55	0.532	1.597	3.0	0.00
94	132.1	Saturated	Saturated	81	90	0.0	0	11983.2	7053.6	110.2	0.55	0.530	1.589	3.0	0.00
95	132.1	Saturated	Saturated	81	90	0.0	0	12115.3	7123.3	109.9	0.54	0.529	1.582	3.0	0.00
96	132.1	Saturated	Saturated	71	95	0.0	0	12247.4	7193.0	96.1	0.54	0.527	1.575	3.0	0.00
97	132.1	Saturated	Saturated	71	95	0.0	0	12379.5	7262.7	95.8	0.54	0.526	1.568	3.0	0.00
98	123.6	Saturated	Saturated	71	95	0.0	0	12503.1	7323.9	95.6	0.54	0.524	1.562	3.0	0.00
99	123.6	Saturated	Saturated	71	95	0.0	0	12626.7	7385.1	95.4	0.54	0.523	1.556	3.0	0.00
100	123.6	Saturated	Saturated	71	95	0.0	0	12750.3	7446.3	95.2	0.53	0.522	1.550	3.0	0.00
											Total Liquefaction Settlement, S =			**3.77 inches**	

Geotechnologies, Inc.

Project: Chait Company
File No.: 21194
Description: Liquefaction Analysis
Boring Numbe 2

LIQUEFACTION EVALUATION (Idriss & Boulanger, EERI NO 12)

EARTHQUAKE INFORMATION:

Earthquake Magnitude (M):	6.7
Peak Ground Horizontal Acceleration, PGA (g):	0.88
Calculated Mag.Wtg.Factor:	1.234
GROUNDWATER INFORMATION:	
Current Groundwater Level (ft):	24.0
Historically Highest Groundwater Level* (ft):	15.0
Unit Weight of Water (pcf):	62.4

* Based on California Geological Survey Seismic Hazard Evaluation Report

BOREHOLE AND SAMPLER INFORMATION:

Borehole Diameter (inches):	5
SPT Sampler with room for Liner (Y/N):	Y
LIQUEFACTION BOUNDARY:	
Plastic Index Cut Off (PI):	18
Minimum Liquefaction FS:	1

Depth to Base Layer (feet)	Total Unit Weight (pcf)	Current Water Level (feet)	Historical Water Level (feet)	Field SPT Blowcount N	Depth of SPT Blowcount (feet)	Fines Content #200 Sieve (%)	Plastic Index (PI)	Vetical Stress σ_{vc}, (psf)	Effective Vert. Stress σ_{vc}', (psf)	Fines Corrected $(N_1)_{60-cs}$	Stress Reduction Coeff, r_d	Cyclic Shear Ratio CSR	Cyclic Resistance Ratio (CRR)	Factor of Safety CRR/CSR (F.S.)	Liquefaction Settlment ΔS_i (inches)
1	137.4	Unsaturated	Unsaturated	17	5	59.2	22	137.4	137.4	46.1	1.00	0.574	2.000	Non-Liq.	0.00
2	137.4	Unsaturated	Unsaturated	17	5	59.2	22	274.8	274.8	46.1	1.00	0.572	2.000	Non-Liq.	0.00
3	137.4	Unsaturated	Unsaturated	17	5	59.2	22	412.2	412.2	46.1	1.00	0.571	2.000	Non-Liq.	0.00
4	137.4	Unsaturated	Unsaturated	17	5	59.2	22	549.6	549.6	46.1	0.99	0.568	2.000	Non-Liq.	0.00
5	137.4	Unsaturated	Unsaturated	17	5	59.2	22	687.0	687.0	45.3	0.99	0.566	2.000	Non-Liq.	0.00
6	137.4	Unsaturated	Unsaturated	17	5	59.2	22	824.4	824.4	42.5	0.99	0.564	2.000	Non-Liq.	0.00
7	137.4	Unsaturated	Unsaturated	17	5	59.2	22	961.8	961.8	39.9	0.98	0.562	2.000	Non-Liq.	0.00
8	131.5	Unsaturated	Unsaturated	17	5	59.2	22	1093.3	1093.3	37.8	0.98	0.559	2.000	Non-Liq.	0.00
9	131.5	Unsaturated	Unsaturated	17	5	59.2	22	1224.8	1224.8	38.2	0.97	0.557	2.000	Non-Liq.	0.00
10	131.5	Unsaturated	Unsaturated	17	5	59.2	22	1356.3	1356.3	36.6	0.97	0.554	2.000	Non-Liq.	0.00
11	131.5	Unsaturated	Unsaturated	11	10	80.3	30	1487.8	1487.8	24.0	0.96	0.552	0.349	Non-Liq.	0.00
12	131.5	Unsaturated	Unsaturated	11	10	80.3	30	1619.3	1619.3	23.2	0.96	0.549	0.324	Non-Liq.	0.00
13	124.1	Unsaturated	Unsaturated	11	10	80.3	30	1743.4	1743.4	22.5	0.96	0.546	0.305	Non-Liq.	0.00
14	124.1	Unsaturated	Unsaturated	11	10	80.3	30	1867.5	1867.5	21.8	0.95	0.543	0.290	Non-Liq.	0.00
15	124.1	Unsaturated	Unsaturated	11	10	80.3	30	1991.6	1991.6	23.4	0.95	0.541	0.319	Non-Liq.	0.00
16	124.1	Unsaturated	Saturated	6	15	93.5	40	2115.7	2053.3	14.7	0.94	0.554	0.190	Non-Liq.	0.00
17	124.1	Unsaturated	Saturated	6	15	93.5	40	2239.8	2115.0	14.5	0.93	0.566	0.188	Non-Liq.	0.00
18	113.8	Unsaturated	Saturated	6	15	93.5	40	2353.6	2166.4	14.4	0.93	0.578	0.186	Non-Liq.	0.00
19	113.8	Unsaturated	Saturated	6	15	93.5	40	2467.4	2217.8	14.3	0.92	0.588	0.184	Non-Liq.	0.00
20	113.8	Unsaturated	Saturated	6	15	93.5	40	2581.2	2269.2	14.2	0.92	0.598	0.183	Non-Liq.	0.00
21	113.8	Unsaturated	Saturated	6	20	78.6	33	2695.0	2320.6	14.1	0.91	0.607	0.182	Non-Liq.	0.00
22	113.8	Unsaturated	Saturated	6	20	78.6	33	2808.8	2372.0	14.0	0.91	0.615	0.180	Non-Liq.	0.00
23	123.8	Unsaturated	Saturated	5	25	91.3	20	2932.6	2433.4	12.4	0.90	0.622	0.165	Non-Liq.	0.00
24	123.8	Unsaturated	Saturated	5	25	91.3	20	3056.4	2494.8	12.3	0.90	0.628	0.164	Non-Liq.	0.00
25	123.8	Saturated	Saturated	5	25	91.3	20	3180.2	2556.2	12.2	0.89	0.633	0.163	Non-Liq.	0.00
26	123.8	Saturated	Saturated	5	25	91.3	20	3304.0	2617.6	12.1	0.88	0.638	0.161	Non-Liq.	0.00
27	123.8	Saturated	Saturated	5	25	91.3	20	3427.8	2679.0	12.1	0.88	0.643	0.160	Non-Liq.	0.00
28	125.1	Saturated	Saturated	5	25	91.3	20	3552.9	2741.7	12.3	0.87	0.647	0.162	Non-Liq.	0.00
29	125.1	Saturated	Saturated	5	25	91.3	20	3678.0	2804.4	12.2	0.87	0.650	0.161	Non-Liq.	0.00
30	125.1	Saturated	Saturated	5	25	91.3	20	3803.1	2867.1	12.2	0.86	0.653	0.160	Non-Liq.	0.00
31	125.1	Saturated	Saturated	5	30	82.8	13	3928.2	2929.8	12.1	0.85	0.655	0.159	0.2	0.40
32	125.1	Saturated	Saturated	5	30	82.8	13	4053.3	2992.5	12.0	0.85	0.657	0.158	0.2	0.40
33	124.7	Saturated	Saturated	27	35	0.0	13	4178.0	3054.8	45.8	0.84	0.659	2.000	3.0	0.00
34	124.7	Saturated	Saturated	27	35	0.0	13	4302.7	3117.1	45.5	0.84	0.660	2.000	3.0	0.00
35	124.7	Saturated	Saturated	27	35	0.0	13	4427.4	3179.4	45.3	0.83	0.661	2.000	3.0	0.00
36	124.7	Saturated	Saturated	27	35	0.0	0	4552.1	3241.7	45.1	0.82	0.661	2.000	3.0	0.00
37	124.7	Saturated	Saturated	27	35	0.0	0	4676.8	3304.0	44.8	0.82	0.662	2.000	3.0	0.00
38	139.1	Saturated	Saturated	27	35	0.0	0	4815.9	3380.7	44.6	0.81	0.661	2.000	3.0	0.00
39	139.1	Saturated	Saturated	27	35	0.0	0	4955.0	3457.4	44.3	0.80	0.660	2.000	3.0	0.00
40	139.1	Saturated	Saturated	27	35	0.0	0	5094.1	3534.1	44.1	0.80	0.658	2.000	3.0	0.00
41	139.1	Saturated	Saturated	14	40	23.0	0	5233.2	3610.8	23.5	0.79	0.657	0.292	0.4	0.24
42	139.1	Saturated	Saturated	14	40	23.0	0	5372.3	3687.5	23.3	0.79	0.655	0.288	0.4	0.24
43	136.6	Saturated	Saturated	14	40	23.0	0	5508.9	3761.7	23.1	0.78	0.653	0.283	0.4	0.24
44	136.6	Saturated	Saturated	14	40	23.0	0	5645.5	3835.9	22.9	0.77	0.651	0.279	0.4	0.25
45	136.6	Saturated	Saturated	14	40	23.0	0	5782.1	3910.1	22.8	0.77	0.649	0.275	0.4	0.25
46	136.6	Saturated	Saturated	43	45	0.0	0	5918.7	3984.3	68.0	0.76	0.647	2.000	3.1	0.00
47	136.6	Saturated	Saturated	43	45	0.0	0	6055.3	4058.5	67.6	0.76	0.645	1.992	3.1	0.00
48	144.0	Saturated	Saturated	43	45	0.0	0	6199.3	4140.1	67.3	0.75	0.642	1.978	3.1	0.00
49	144.0	Saturated	Saturated	43	45	0.0	0	6343.3	4221.7	66.9	0.74	0.639	1.963	3.1	0.00
50	144.0	Saturated	Saturated	43	45	0.0	0	6487.3	4303.3	66.6	0.74	0.636	1.949	3.1	0.00
51	144.0	Saturated	Saturated	21	50	32.1	0	6631.3	4384.9	34.6	0.73	0.633	1.029	1.6	0.00
52	144.0	Saturated	Saturated	21	50	32.1	0	6775.3	4466.5	34.4	0.73	0.630	0.982	1.6	0.00
53	112.5	Saturated	Saturated	21	50	32.1	0	6887.8	4516.6	34.3	0.72	0.628	0.955	1.5	0.00
54	112.5	Saturated	Saturated	21	50	32.1	0	7000.3	4566.7	34.1	0.71	0.626	0.930	1.5	0.00
55	112.5	Saturated	Saturated	21	50	32.1	0	7112.8	4616.8	34.0	0.71	0.624	0.906	1.5	0.00
56	112.5	Saturated	Saturated	36	55	0.0	0	7225.3	4666.9	54.6	0.70	0.622	1.890	3.0	0.00
57	112.5	Saturated	Saturated	36	55	0.0	0	7337.8	4717.0	54.4	0.70	0.620	1.883	3.0	0.00
58	134.7	Saturated	Saturated	36	55	0.0	0	7472.5	4789.3	54.2	0.69	0.617	1.872	3.0	0.00
59	134.7	Saturated	Saturated	36	55	0.0	0	7607.2	4861.6	54.0	0.69	0.614	1.861	3.0	0.00
60	134.7	Saturated	Saturated	36	55	0.0	0	7741.9	4933.9	53.8	0.68	0.611	1.850	3.0	0.00
61	134.7	Saturated	Saturated	38	60	0.0	0	7876.6	5006.2	56.6	0.68	0.608	1.839	3.0	0.00
62	134.7	Saturated	Saturated	38	60	0.0	0	8011.3	5078.5	56.4	0.67	0.605	1.829	3.0	0.00
63	140.8	Saturated	Saturated	38	60	0.0	0	8152.1	5156.9	56.1	0.67	0.601	1.818	3.0	0.00
64	140.8	Saturated	Saturated	38	60	0.0	0	8292.9	5235.3	55.9	0.66	0.598	1.807	3.0	0.00
65	140.8	Saturated	Saturated	38	60	0.0	0	8433.7	5313.7	55.7	0.65	0.595	1.796	3.0	0.00
66	140.8	Saturated	Saturated	71	65	0.0	0	8574.5	5392.1	103.6	0.65	0.591	1.785	3.0	0.00
67	140.8	Saturated	Saturated	71	65	0.0	0	8715.3	5470.5	103.3	0.64	0.588	1.775	3.0	0.00
68	128.4	Saturated	Saturated	71	65	0.0	0	8843.7	5536.5	102.9	0.64	0.585	1.766	3.0	0.00
69	128.4	Saturated	Saturated	71	65	0.0	0	8972.1	5602.5	102.6	0.64	0.582	1.757	3.0	0.00
70	128.4	Saturated	Saturated	71	65	0.0	0	9100.5	5668.5	102.3	0.63	0.579	1.749	3.0	0.00
71	128.4	Saturated	Saturated	90	70	0.0	0	9228.9	5734.5	129.3	0.63	0.576	1.740	3.0	0.00
72	128.4	Saturated	Saturated	90	70	0.0	0	9357.3	5800.5	128.9	0.62	0.574	1.732	3.0	0.00
73	127.0	Saturated	Saturated	90	70	0.0	0	9484.3	5865.1	128.5	0.62	0.571	1.724	3.0	0.00
74	127.0	Saturated	Saturated	90	70	0.0	0	9611.3	5929.7	128.1	0.61	0.568	1.716	3.0	0.00
75	127.0	Saturated	Saturated	90	70	0.0	0	9738.3	5994.3	127.8	0.61	0.566	1.708	3.0	0.00
76	127.0	Saturated	Saturated	83	75	0.0	0	9865.3	6058.9	117.5	0.60	0.563	1.700	3.0	0.00
77	127.0	Saturated	Saturated	83	75	0.0	0	9992.3	6123.5	117.2	0.60	0.560	1.692	3.0	0.00
78	142.0	Saturated	Saturated	83	75	0.0	0	10134.3	6203.1	116.8	0.60	0.557	1.683	3.0	0.00
79	142.0	Saturated	Saturated	83	75	0.0	0	10276.3	6282.7	116.4	0.59	0.554	1.674	3.0	0.00
80	142.0	Saturated	Saturated	83	75	0.0	0	10418.3	6362.3	116.0	0.59	0.552	1.665	3.0	0.00
											Total Liquefaction Settlement, S =			**2.02 inches**	

Geotechnologies, Inc.

Project: Chait Company
File No.: 21194
Description: Liquefaction Analysis
Boring Numbe 3

LIQUEFACTION EVALUATION (Idriss & Boulanger, EERI NO 12)

EARTHQUAKE INFORMATION:

Earthquake Magnitude (M):	6.7
Peak Ground Horizontal Acceleration, PGA (g):	0.88
Calculated Mag.Wtg.Factor:	1.234

GROUNDWATER INFORMATION:

Current Groundwater Level (ft):	30.0
Historically Highest Groundwater Level* (ft):	15.0
Unit Weight of Water (pcf):	62.4

* Based on California Geological Survey Seismic Hazard Evaluation Report

BOREHOLE AND SAMPLER INFORMATION:

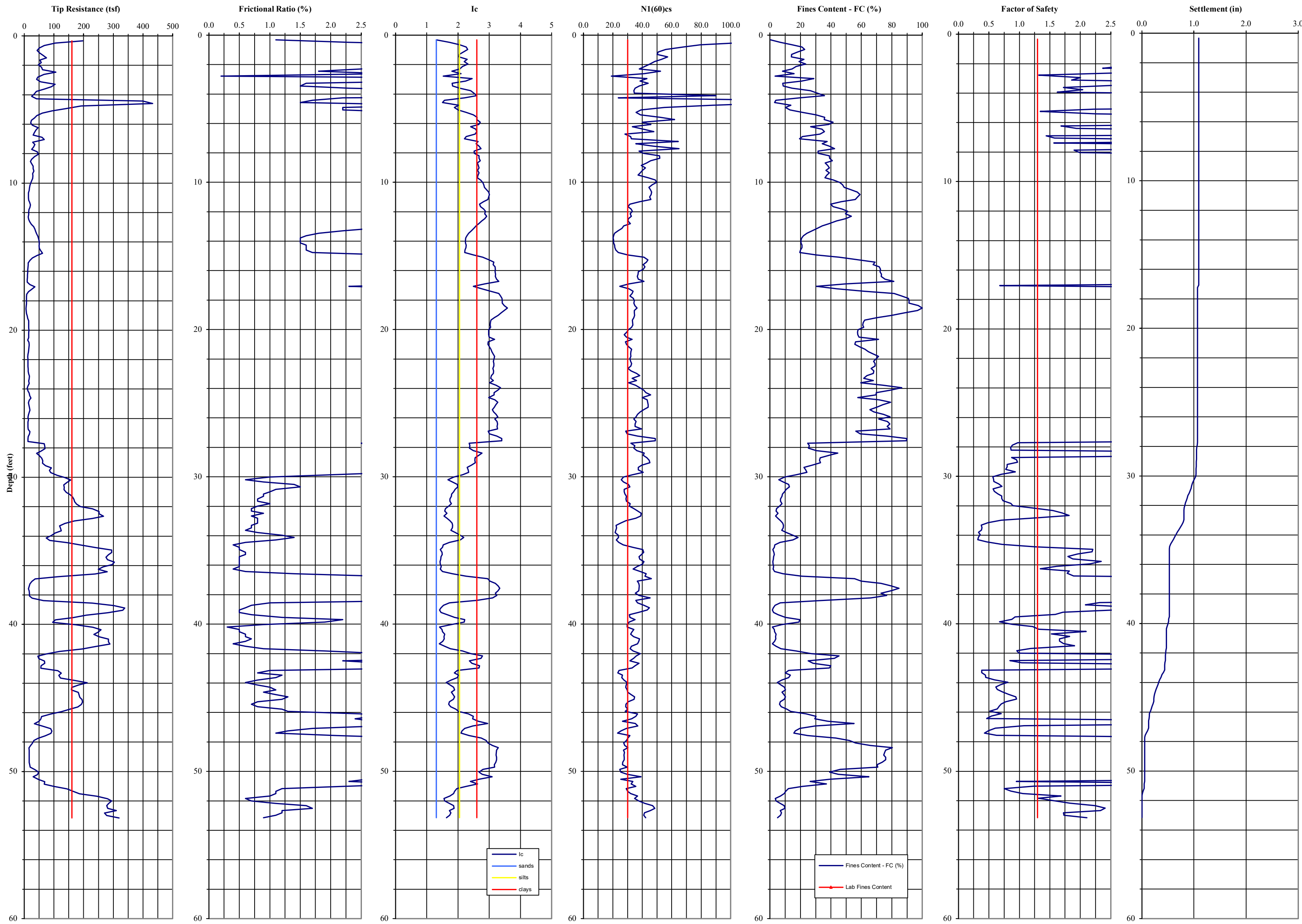
Borehole Diameter (inches):	5
SPT Sampler with room for Liner (Y/N):	Y

LIQUEFACTION BOUNDARY:

Plastic Index Cut Off (PI):	18
Minimum Liquefaction FS:	1

Depth to Base Layer (feet)	Total Unit Weight (pcf)	Current Water Level (feet)	Historical Water Level (feet)	Field SPT Blowcount N	Depth of SPT Blowcount (feet)	Fines Content #200 Sieve (%)	Plastic Index (PI)	Vetical Stress σ_{vc}, (psf)	Effective Vert. Stress σ_{vc}', (psf)	Fines Corrected $(N_1)_{60\text{-}cs}$	Stress Reduction Coeff, r_d	Cyclic Shear Ratio CSR	Cyclic Resistance Ratio (CRR)	Factor of Safety CRR/CSR (F.S.)	Liquefaction Settlment ΔS_i (inches)
1	131.3	Unsaturated	Unsaturated	23	5	0.0	0	131.3	131.3	54.8	1.00	0.574	2.000	Non-Liq.	0.00
2	131.3	Unsaturated	Unsaturated	23	5	0.0	0	262.6	262.6	54.8	1.00	0.572	2.000	Non-Liq.	0.00
3	131.3	Unsaturated	Unsaturated	23	5	0.0	0	393.9	393.9	54.8	1.00	0.571	2.000	Non-Liq.	0.00
4	131.3	Unsaturated	Unsaturated	23	5	0.0	0	525.2	525.2	52.3	0.99	0.568	2.000	Non-Liq.	0.00
5	131.3	Unsaturated	Unsaturated	23	5	0.0	0	656.5	656.5	50.8	0.99	0.566	2.000	Non-Liq.	0.00
6	131.3	Unsaturated	Unsaturated	23	5	0.0	0	787.8	787.8	47.8	0.99	0.564	2.000	Non-Liq.	0.00
7	131.3	Unsaturated	Unsaturated	23	5	0.0	0	919.1	919.1	45.4	0.98	0.562	2.000	Non-Liq.	0.00
8	133.2	Unsaturated	Unsaturated	23	5	0.0	0	1052.3	1052.3	43.4	0.98	0.559	2.000	Non-Liq.	0.00
9	133.2	Unsaturated	Unsaturated	23	5	0.0	0	1185.5	1185.5	43.9	0.97	0.557	2.000	Non-Liq.	0.00
10	133.2	Unsaturated	Unsaturated	23	5	0.0	0	1318.7	1318.7	42.5	0.97	0.554	2.000	Non-Liq.	0.00
11	133.2	Unsaturated	Unsaturated	13	10	51.6	28	1451.9	1451.9	28.0	0.96	0.552	0.507	Non-Liq.	0.00
12	133.2	Unsaturated	Unsaturated	13	10	51.6	28	1585.1	1585.1	27.0	0.96	0.549	0.449	Non-Liq.	0.00
13	112.4	Unsaturated	Unsaturated	6	15	78.8	37	1697.5	1697.5	14.7	0.96	0.546	0.194	Non-Liq.	0.00
14	112.4	Unsaturated	Unsaturated	6	15	78.8	37	1809.9	1809.9	14.3	0.95	0.543	0.189	Non-Liq.	0.00
15	112.4	Unsaturated	Unsaturated	6	15	78.8	37	1922.3	1922.3	15.0	0.95	0.541	0.195	Non-Liq.	0.00
16	112.4	Unsaturated	Saturated	6	15	78.8	37	2034.7	1972.3	14.9	0.94	0.555	0.193	Non-Liq.	0.00
17	112.4	Unsaturated	Saturated	6	15	78.8	37	2147.1	2022.3	14.8	0.93	0.568	0.191	Non-Liq.	0.00
18	111.9	Unsaturated	Saturated	6	15	78.8	37	2259.0	2071.8	14.7	0.93	0.580	0.189	Non-Liq.	0.00
19	111.9	Unsaturated	Saturated	6	15	78.8	37	2370.9	2121.3	14.5	0.92	0.591	0.188	Non-Liq.	0.00
20	111.9	Unsaturated	Saturated	6	15	78.8	37	2482.8	2170.8	14.4	0.92	0.601	0.186	Non-Liq.	0.00
21	111.9	Unsaturated	Saturated	3	20	85.7	38	2594.7	2220.3	9.9	0.91	0.610	0.144	Non-Liq.	0.00
22	111.9	Unsaturated	Saturated	3	20	85.7	38	2706.6	2269.8	9.8	0.91	0.619	0.143	Non-Liq.	0.00
23	117.7	Unsaturated	Saturated	3	20	85.7	38	2824.3	2325.1	9.8	0.90	0.627	0.142	Non-Liq.	0.00
24	117.7	Unsaturated	Saturated	3	20	85.7	38	2942.0	2380.4	9.7	0.90	0.633	0.142	Non-Liq.	0.00
25	117.7	Unsaturated	Saturated	3	20	85.7	38	3059.7	2435.7	9.6	0.89	0.640	0.141	Non-Liq.	0.00
26	117.7	Unsaturated	Saturated	5	25	88.7	19	3177.4	2491.0	12.3	0.88	0.645	0.164	Non-Liq.	0.00
27	117.7	Unsaturated	Saturated	5	25	88.7	19	3295.1	2546.3	12.3	0.88	0.650	0.163	Non-Liq.	0.00
28	124.4	Unsaturated	Saturated	5	25	88.7	19	3419.5	2608.3	12.5	0.87	0.654	0.165	Non-Liq.	0.00
29	124.4	Unsaturated	Saturated	5	25	88.7	19	3543.9	2670.3	12.4	0.87	0.658	0.164	Non-Liq.	0.00
30	124.4	Unsaturated	Saturated	5	25	88.7	19	3668.3	2732.3	12.3	0.86	0.661	0.162	Non-Liq.	0.00
31	124.4	Saturated	Saturated	16	30	30.7	0	3792.7	2794.3	30.2	0.85	0.663	0.580	0.9	0.11
32	124.4	Saturated	Saturated	16	30	30.7	0	3917.1	2856.3	30.0	0.85	0.665	0.559	0.8	0.11
33	132.4	Saturated	Saturated	30	35	0.0	0	4049.5	2926.3	51.4	0.84	0.666	2.000	3.0	0.00
34	132.4	Saturated	Saturated	30	35	0.0	0	4181.9	2996.3	51.1	0.84	0.667	2.000	3.0	0.00
35	132.4	Saturated	Saturated	30	35	0.0	0	4314.3	3066.3	50.8	0.83	0.668	2.000	3.0	0.00
36	132.4	Saturated	Saturated	30	35	0.0	0	4446.7	3136.3	50.5	0.82	0.668	2.000	3.0	0.00
37	132.4	Saturated	Saturated	30	35	0.0	0	4579.1	3206.3	50.2	0.82	0.668	2.000	3.0	0.00
38	140.8	Saturated	Saturated	30	35	0.0	0	4719.9	3284.7	49.9	0.81	0.667	2.000	3.0	0.00
39	140.8	Saturated	Saturated	30	35	0.0	0	4860.7	3363.1	49.6	0.80	0.665	2.000	3.0	0.00
40	140.8	Saturated	Saturated	30	35	0.0	0	5001.5	3441.5	49.3	0.80	0.664	2.000	3.0	0.00
41	140.8	Saturated	Saturated	33	40	0.0	0	5142.3	3519.9	53.9	0.79	0.662	2.000	3.0	0.00
42	140.8	Saturated	Saturated	33	40	0.0	0	5283.1	3598.3	53.6	0.79	0.660	2.000	3.0	0.00
43	131.1	Saturated	Saturated	14	45	17.4	0	5414.2	3667.0	22.4	0.78	0.659	0.272	0.4	0.25
44	131.1	Saturated	Saturated	14	45	17.4	0	5545.3	3735.7	22.2	0.77	0.657	0.268	0.4	0.25
45	131.1	Saturated	Saturated	14	45	17.4	0	5676.4	3804.4	22.1	0.77	0.655	0.265	0.4	0.25
46	131.1	Saturated	Saturated	14	45	17.4	0	5807.5	3873.1	21.9	0.76	0.653	0.261	0.4	0.26
47	131.1	Saturated	Saturated	14	45	17.4	0	5938.6	3941.8	21.8	0.76	0.651	0.258	0.4	0.26
48	137.2	Saturated	Saturated	25	55	0.0	0	6075.8	4016.6	38.8	0.75	0.649	2.000	3.1	0.00
49	137.2	Saturated	Saturated	25	55	0.0	0	6213.0	4091.4	38.5	0.74	0.646	1.986	3.1	0.00
50	137.2	Saturated	Saturated	25	55	0.0	0	6350.2	4166.2	38.2	0.74	0.643	1.973	3.1	0.00
51	137.2	Saturated	Saturated	14	50	53.0	14	6487.4	4241.0	22.8	0.73	0.640	0.272	0.4	0.25
52	137.2	Saturated	Saturated	14	50	53.0	14	6624.6	4315.8	22.7	0.73	0.637	0.269	0.4	0.25
53	120.0	Saturated	Saturated	14	50	53.0	14	6744.6	4373.4	22.6	0.72	0.635	0.266	0.4	0.25
54	120.0	Saturated	Saturated	14	50	53.0	14	6864.6	4431.0	22.4	0.71	0.633	0.264	0.4	0.25
55	120.0	Saturated	Saturated	14	50	53.0	14	6984.6	4488.6	22.3	0.71	0.631	0.262	0.4	0.25
56	120.0	Saturated	Saturated	25	55	9.6	14	7104.6	4546.2	38.0	0.70	0.628	1.909	3.0	0.00
57	120.0	Saturated	Saturated	25	55	9.6	14	7224.6	4603.8	37.8	0.70	0.626	1.900	3.0	0.00
58	109.2	Saturated	Saturated	25	55	9.6	14	7333.8	4650.6	37.7	0.69	0.624	1.893	3.0	0.00
59	109.2	Saturated	Saturated	25	55	9.6	14	7443.0	4697.4	37.6	0.69	0.622	1.886	3.0	0.00
60	109.2	Saturated	Saturated	25	55	9.6	14	7552.2	4744.2	37.4	0.68	0.620	1.834	3.0	0.00
61	109.2	Saturated	Saturated	24	60	5.5	14	7661.4	4791.0	34.2	0.68	0.618	0.924	1.5	0.00
62	109.2	Saturated	Saturated	24	60	5.5	14	7770.6	4837.8	34.1	0.67	0.616	0.901	1.5	0.00
63	141.8	Saturated	Saturated	24	60	5.5	14	7912.4	4917.2	33.9	0.67	0.612	0.865	1.4	0.00
64	141.8	Saturated	Saturated	24	60	5.5	14	8054.2	4996.6	33.7	0.66	0.608	0.831	1.4	0.00
65	141.8	Saturated	Saturated	24	60	5.5	14	8196.0	5076.0	33.5	0.65	0.605	0.800	1.3	0.00
66	141.8	Saturated	Saturated	100	65	0.0	14	8337.8	5155.4	147.7	0.65	0.601	1.818	3.0	0.00
67	141.8	Saturated	Saturated	100	65	0.0	14	8479.6	5234.8	147.1	0.64	0.598	1.807	3.0	0.00
68	136.0	Saturated	Saturated	100	65	0.0	14	8615.6	5308.4	146.6	0.64	0.594	1.797	3.0	0.00
69	136.0	Saturated	Saturated	100	65	0.0	14	8751.6	5382.0	146.1	0.64	0.591	1.787	3.0	0.00
70	136.0	Saturated	Saturated	100	65	0.0	14	8887.6	5455.6	145.5	0.63	0.588	1.777	3.0	0.00
71	136.0	Saturated	Saturated	81	70	0.0	14	9023.6	5529.2	117.5	0.63	0.585	1.767	3.0	0.00
72	136.0	Saturated	Saturated	81	70	0.0	14	9159.6	5602.8	117.1	0.62	0.581	1.757	3.0	0.00
73	130.4	Saturated	Saturated	81	70	0.0	14	9290.0	5670.8	116.7	0.62	0.578	1.748	3.0	0.00
74	130.4	Saturated	Saturated	81	70	0.0	14	9420.4	5738.8	116.3	0.61	0.576	1.740	3.0	0.00
75	130.4	Saturated	Saturated	81	70	0.0	14	9550.8	5806.8	116.0	0.61	0.573	1.731	3.0	0.00
76	130.4	Saturated	Saturated	79	75	0.0	14	9681.2	5874.8	112.8	0.60	0.570	1.723	3.0	0.00
77	130.4	Saturated	Saturated	79	75	0.0	14	9811.6	5942.8	112.4	0.60	0.567	1.714	3.0	0.00
78	128.4	Saturated	Saturated	79	75	0.0	14	9940.0	6008.8	112.1	0.60	0.564	1.706	3.0	0.00
79	128.4	Saturated	Saturated	79	75	0.0	14	10068.4	6074.8	111.8	0.59	0.562	1.698	3.0	0.00
80	128.4	Saturated	Saturated	79	75	0.0	14	10196.8	6140.8	111.4	0.59	0.559	1.690	3.0	0.00

Total Liquefaction Settlement, S = 2.74 inches



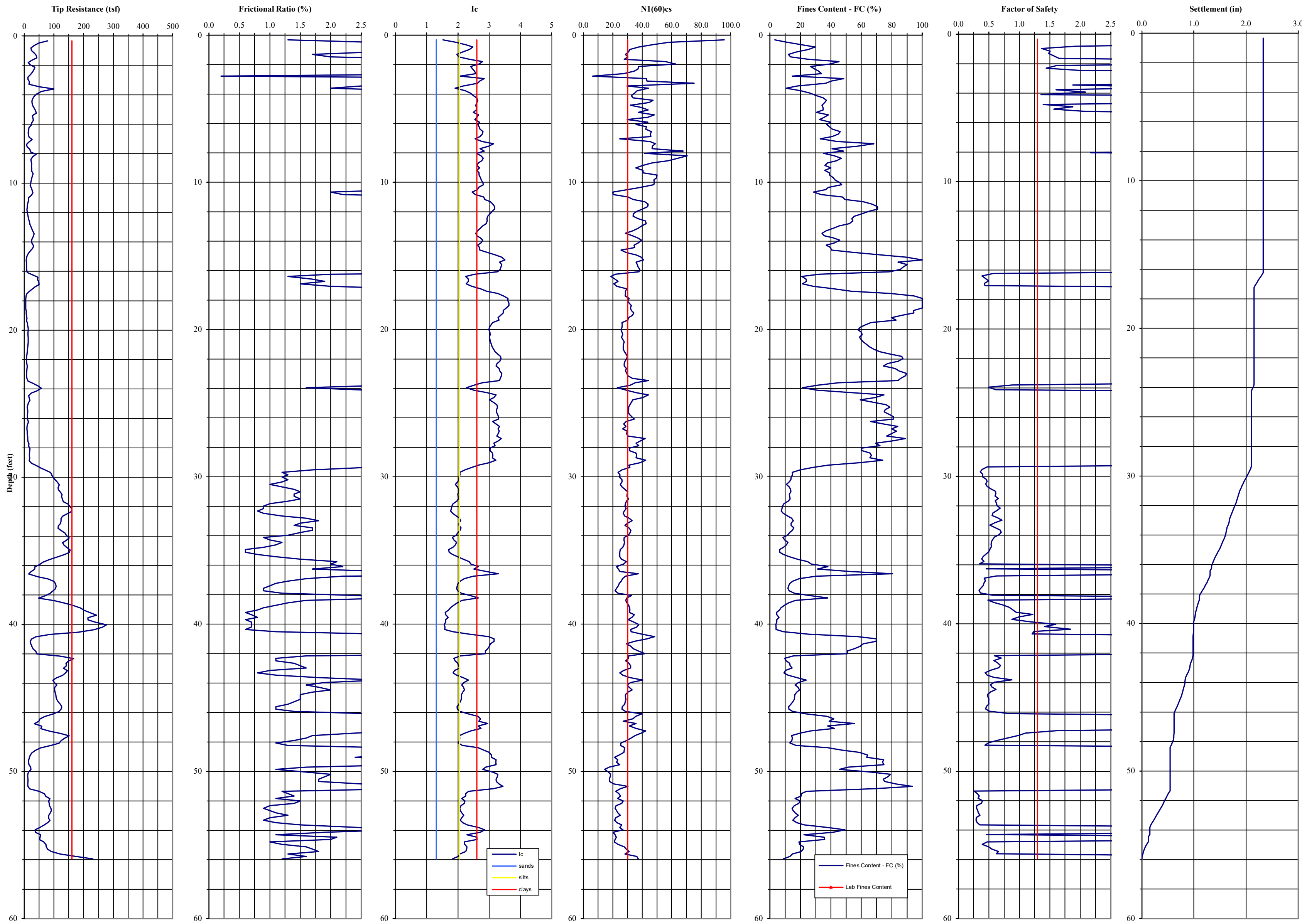
Geotechnologies, Inc.

Client: Chait Company

File No.: 21194

CPT Sounding No.: **CPT-01**

Magnitude (M_w) =	6.7	Cumulative Liquefaction Settlement =	1.09 inches
Peak Ground Acceleration (g) =	0.88 g	Depth to Historic High Water (feet) =	15.0 feet



Geotechnologies, Inc.

Client: Chait Company

File No.: 21194

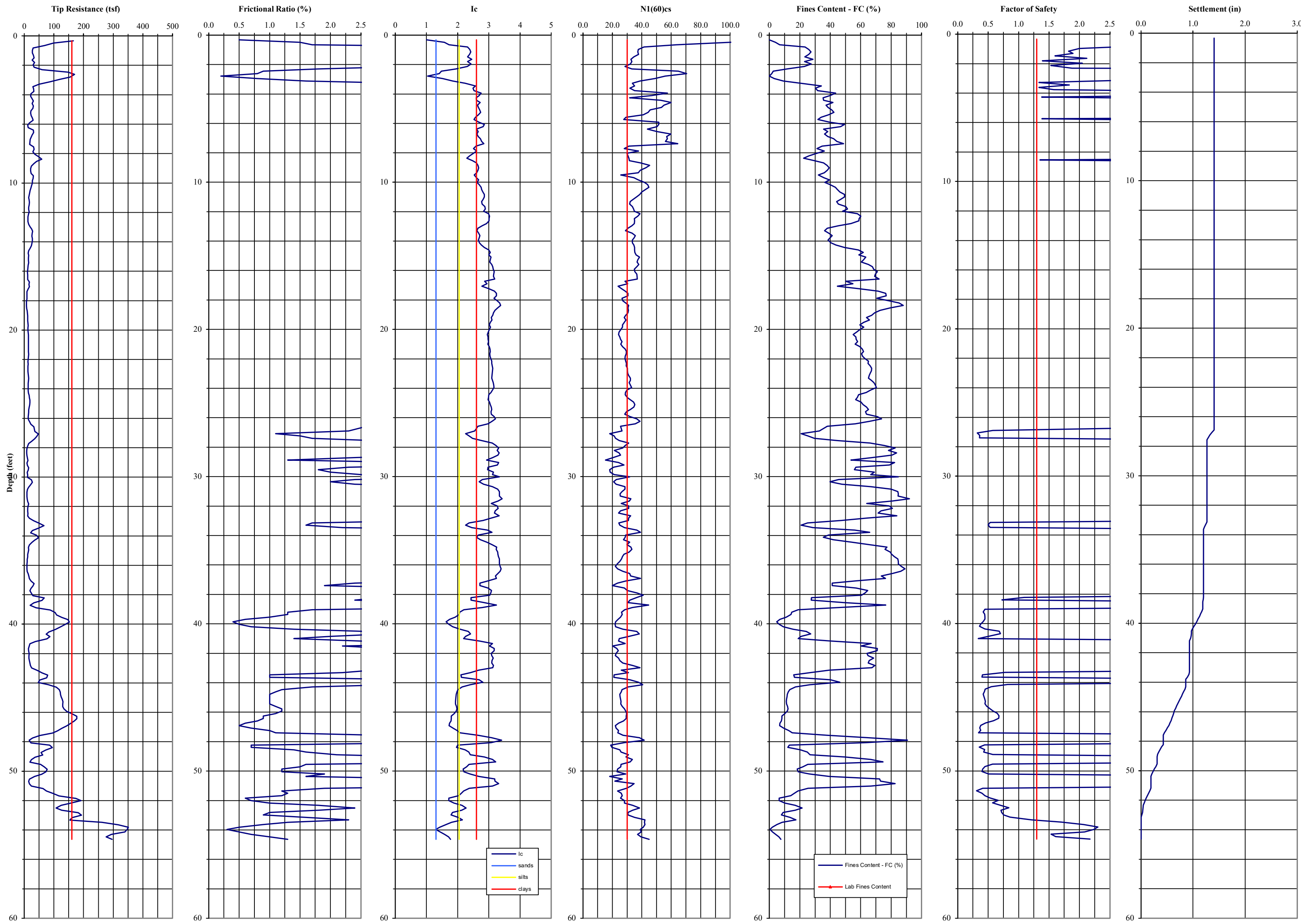
CPT Sounding No.: CPT-02

Magnitude (M_w) = 6.7

Peak Ground Acceleration (g) = 0.88 g

Cumulative Liquefaction Settlement = 2.33 inches

Depth to Historic High Water (feet) = 15.0 feet



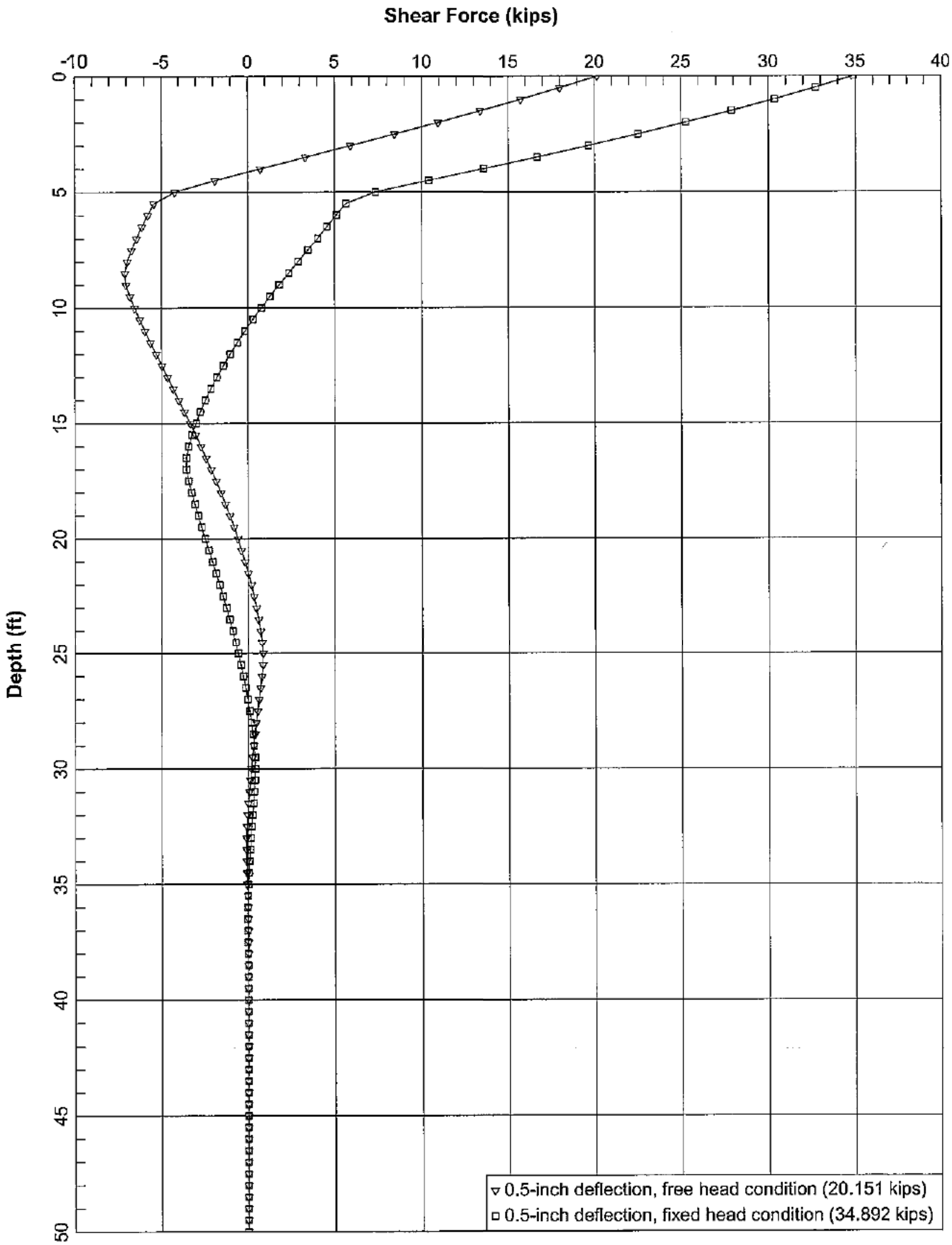
Geotechnologies, Inc.

Client: Chait Company

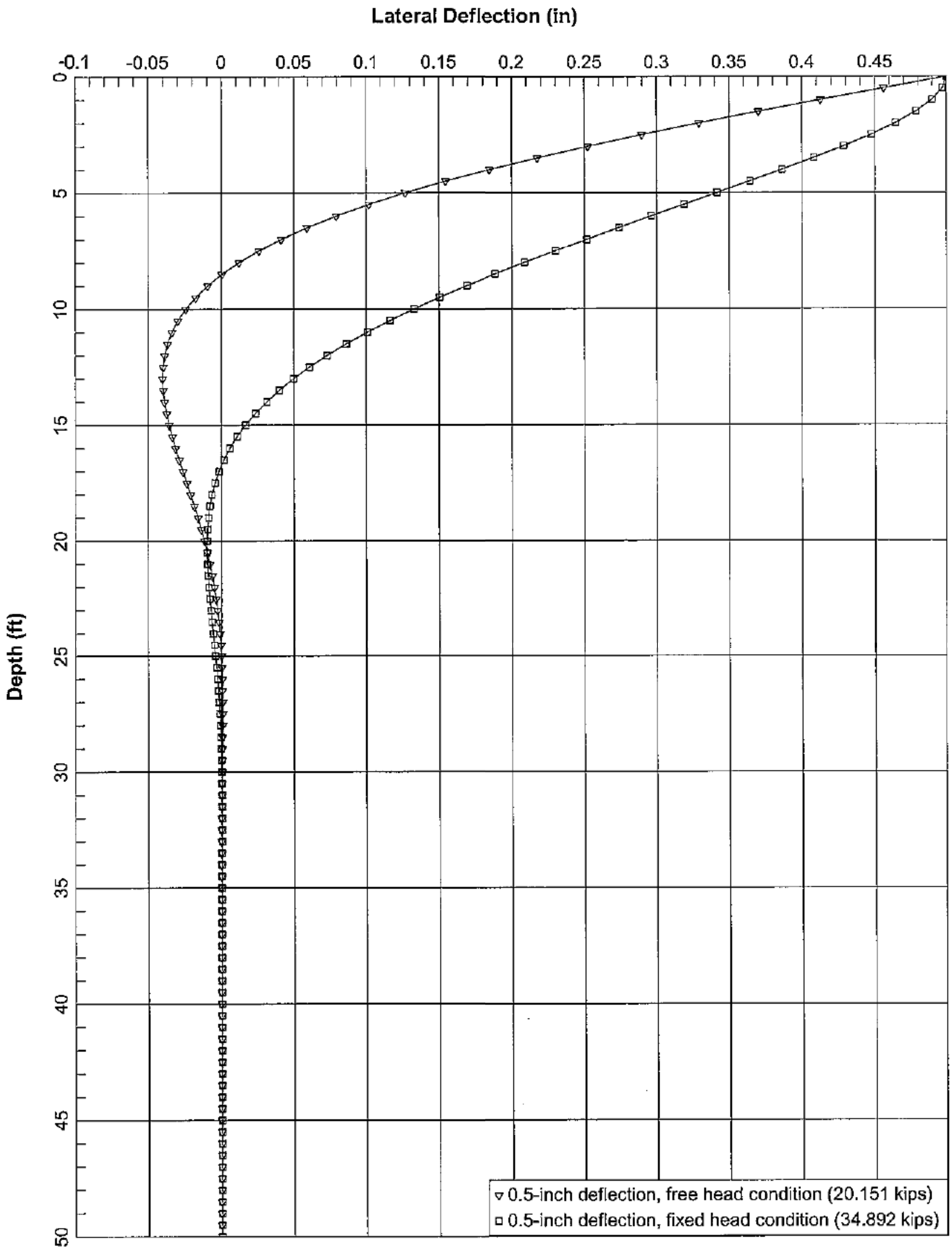
File No.: 21194

CPT Sounding No.: CPT-03

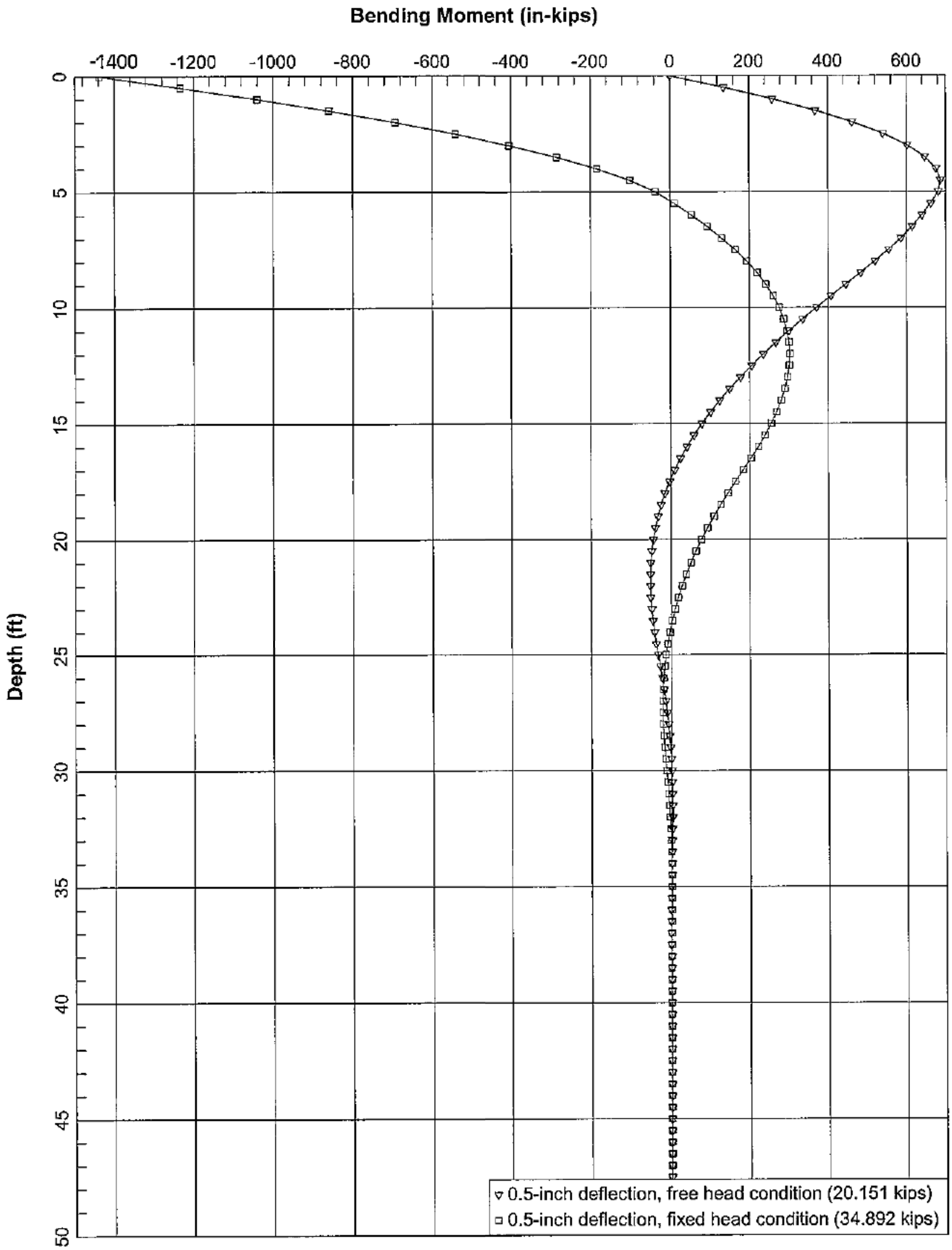
Magnitude (M_w) =	6.7	Cumulative Liquefaction Settlement =	1.40 inches
Peak Ground Acceleration (g) =	0.88 g	Depth to Historic High Water (feet) =	15.0 feet



File No. 21194, 16-inch diameter pile



File No. 21194, 16-inch diameter pile



File No. 21194, 16-inch diameter pile

12575 BEATRICE STREET
GROUND-MOTION DEVELOPMENT
LOS ANGELES, CALIFORNIA

Prepared for
Mr. Stan Tang
Geotechnologies, Inc.
439 Western Avenue
Glendale, California 91201

March 18, 2020

Prepared by
GeoPentech
5251 California Avenue, Suite 210
Irvine, CA 92617
(714) 796-9100
Fax (714) 796-9191
Web Site: www.geopentech.com

GeoPentech

March 18, 2020
Project No.: 17025A

Mr. Stan Tang
Geotechnologies, Inc.
439 Western Avenue
Glendale, California 91201

Subject: Ground-Motion Development Report
Proposed Playa Vista Campus Development at
12575 Beatrice Street
Los Angeles, California

Dear Mr. Tang:

In general accordance with the provisions of our agreement for professional services, we have completed a ground-motion evaluation for the subject project and have documented our findings in the accompanying draft report. This report supersedes the one submitted on June 15, 2018, and contains the recommended response spectra to be used in the design and analysis of the subject project.

We trust that this report meets the present needs of the project. If you should have any questions, please contact us.

Very truly yours,

Andrew Dinsick, PE
Associate

Carola Di Alessandro, Ph.D.
Project Professional

Steve Duke, CEG, PGp, CHg
Associate

TABLE OF CONTENTS

1. INTRODUCTION

This report presents the site-specific ground-motion evaluation for the proposed development located at 12575 Beatrice Street, at the corner of Beatrice Street and Jandy Place (Figure 1) in Playa Vista, California. We understand that the proposed structure will consist of a 8-story, 135-foot tall office structure comprised of two wings, rising above a commercial and parking podium, with 1.5 level of subterranean parking.

The site is located within a State Liquefaction Hazard Zone, and we understand that Geotechnologies, Inc. considers the site to be potentially liquefiable. Accordingly, it is also our understanding that pile foundations are being utilized to mitigate the liquefaction hazard. The foundation design is being completed by Geotechnologies and the preparation of a detailed foundation design is currently in-progress.

We understand that the design for this structure is being carried out in conformance with the 2016 California Building Code (CBC 2016) and ASCE 7-16 requirements (including Supplement 1 effective December 12, 2018). Furthermore, because of the deep foundation mitigation measures, the Site Class designation will be based on the shear-wave velocity measurements, and site response analyses are not needed. To fulfill the seismic design requirements, the following ground surface 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 Level” uniform hazard spectrum with risk-targeted, maximum-rotated ordinates at 5% damping (corresponding to 2/3 of the MCE_R response spectrum)

In preparing this report, site-specific shear-wave velocity measurements were collected at the site by GeoPentech in Boring B-1 (which was drilled and logged by Geotechnologies) and used in this analysis. The results of the shear-wave velocity measurements are discussed in more detail below in Section 3.2 and are also included herein as Appendix A. We also reviewed the boring log from Boring B-1 by Geotechnologies (reproduced herein at the end of Appendix A). Note that if the site location or site conditions change appreciably, the ground motions presented herein would need to be re-evaluated.

2. CODE-BASED VALUES

Given the site latitude and longitude (located near 33°58'51.78"N, 118°24'57.11"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/. 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 about 950 ft/s (290 m/s) for outcropping conditions at the foundation level approximately 20-feet below existing grade. This V_{S30} value corresponds to site classification for seismic design of ***Site Class D*** (600 < V_{S30} < 1,200 ft/s). Using the ASCE 7-16 standard, the mapped design parameters for a Site Class D, ***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 1.247 g and 0.748 g, respectively. These values are superseded by the site-specific values presented in this report but are provided here for completeness.

3. SOURCE, SITE AND GROUND-MOTION CHARACTERIZATION

Probabilistic and Deterministic Seismic Hazard Analyses (PSHA and DSHA, respectively) involve the characterization of seismic sources, transmission paths for seismic energy, and the local site conditions. Seismic sources pertinent to ground-motion hazards at the site are characterized based on geologic information. The effects of transmission paths and local site conditions are incorporated through the use of attenuation relationships (also known as ground-motion prediction equations – GMPEs), which provide the variation in peak horizontal acceleration or spectral acceleration with distance for a given local site condition. Key information on seismic sources, site conditions, and attenuation relationships used in this study is 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 3a. Figure 3a also shows estimated epicenters of historic earthquakes prior to instrumentation.

As shown on Figure 3a, the 1971 San Fernando earthquake epicenter was roughly 48 km north of the subject site, and the 1994 Northridge epicenter was approximately 30 km northwest. Other noticeable earthquakes such as the 2009 Inglewood and 1987 Whittier events occurred about 8 km south and 31 km east of the site.

Based on recordings in the PEER (2014) database from few stations about 2.5 km around the subject site, the Northridge earthquake generated ground motions on the order of 0.2 g (peak ground acceleration, PGA) and 19 cm/s (peak ground velocity, PGV). Data from the 1987 Whittier earthquake shows motions of about 0.045 g PGA and 2.5 cm/s PGV.

The closest recent surface ruptures are located approximately 5½ km east and 8 km north of site and occurred on the Newport Inglewood and Hollywood faults, respectively. The 1987 Whittier earthquake also generated surface rupture about 31 km east of the site. The 1994 Northridge earthquake occurred on a deep blind thrust fault and did not rupture the ground surface. Two late quaternary (<130 ka) inferred faults are very close to the project site: in fact, the site is located approximately 1 and 3 km away from the inferred traces of the Charnock and Overland faults, as mapped by Jennings (1994). These faults were not included in the PSHA analyses (discussed more in detail below) because several recent focused studies (Davis, 2000a and 2000b, among others) indicated absence of evident activity.

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 3b. The best-estimate parameters (including maximum magnitude, closest distance, slip rate, and style of faulting) for these seismic sources are summarized in Table 1. All faults shown on Figure 3b and listed in Table 1 were included in the PSHA. In addition to the discrete seismic sources presented in Table 1, background seismicity that is consistent with the gridded seismicity used in the NSHM calculation was also used in the PSHA. Specific scenarios evaluated for the DSHA are presented in Table 2.

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 meters of the site (V_{S30}) is the primary parameter used to approximate soil non-linearity in the ground motion models. Shear-wave velocity measurements, plotted on Figure 4, were collected by GeoPentech and are discussed in more detail in Appendix A.

It is our understanding that Geotechnologies has identified the potential for liquefiable soils at the site. At this time, the proposed structure is planned to be founded on piles; therefore, it is our understanding that any potential liquefaction hazard at the site will be mitigated by founding the proposed structures on piles. Accordingly, the seismic hazard analysis will be performed for outcropping V_{S30} conditions corresponding to Site Class D at the proposed basement slab level. As shown on Figure 4, an outcropping site-specific V_{S30} of 950 ft/s (290 m/s) was used for the hazard analysis. 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.

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 520 meters and 3.0 km, respectively. These estimates were based on the SCEC Community Velocity Model (CVM-S4) by Magistrale et al. (2000 and 2012) and are in general agreement with our understanding of the LA basin geometry in the vicinity of the project site.

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. Four of the 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); and Chiou and Youngs (2014). The Idriss (2014) model was not used as the site-specific V_{S30} measurement is outside the recommended range for the model. Each of the attenuation relationships was assigned an equal weight of 1/4 to approximately address the "modeling" part of the epistemic uncertainty.

Because the site is located on the hanging-wall side of the Compton and San Pedro Escarpment reverse faults, appropriate 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 for the site 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 current version number 45.2 of the computer program Hazard (Abrahamson, 2017). This program version has gone through validation effort being conducted by PEER.

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., a specified ARP) over a range of periods.

Figure 5a 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. At the 2,475-yr ARP (which represents a 2% probability of exceedance in 50 years), the combined Santa Monica, Hollywood and Anacapa-Dume fault system and the combined Compton sources (i.e., both SSC alternatives) are the main contributors to the PGA hazard, each contributing approximately 24% to the total PGA hazard. The Newport Inglewood and the background sources are also important contributors, producing about 14% and 13% of the 2,475-yr PGA hazard, respectively. The Palos Verdes fault generates about 8% of the 2,475-yr PGA hazard, and other sources collectively produce the remaining 17% of the 2,475-yr PGA hazard.

Figure 5b presents similar seismic hazard curves for the 1.0-second spectral period, which is estimated to be close to the softened fundamental period of the structure. The San Andreas fault controls the hazard at short return periods, i.e., shorter than about 300 years. At the 2,475-yr ARP, the combined Santa Monica, Hollywood and Anacapa-Dume fault system is the primary contributor, producing about 26% of the 2,475-yr PGA hazard. The Compton sources and the Newport Inglewood faults are also important contributors, producing about 21% and 16% of the 2,475-yr PGA hazard, respectively. The Palos Verdes fault contributes about 11% of the 2,475-yr PGA hazard, and the other sources collectively produce the remaining 26% of the 2,475-yr PGA hazard, with selected faults' contributions being tabulated on Figure 5b.

Figure 6 presents the deaggregation at average return periods of 43 and 2,475 years for PGA and for a period of 1.0-seconds. The 43-yr deaggregations at short period (top panel on the left side) indicates that the hazard is distributed over a broad range of distances (5 to 75 km) and magnitudes (M_W 5.0 to 8 events) with mostly median to 5th percentile ground motions (epsilons between 0 and -2). The 1.0-second, 43-yr deaggregation (bottom panel on the left side) show that the ground-motion hazard is mostly from M_W 6 to 8.0 events with a somewhat bimodal distance distribution; that is, most of the hazard comes from sources more than 50 km away, although a fair amount comes from sources in the intermediate distance. The spikes in the 50 to 75 km bin are from characteristic earthquakes on the San Andreas Fault System, whereas the hazard from other sources close to downtown LA shows around 30 km of distance. The 2,475-yr deaggregrations are shown on the on right half side of Figure 6. At PGA, most of the hazard is coming from M_W 6 to 7.5 earthquakes within 15 km of the site that generating mostly 50th to 95th percentile ground motions (epsilons between 0 and 2). The 1.0-second, 2,475-yr deaggregation (bottom panel on the right side) is quite similar to the deaggregation for the same ARP at PGA; however, some contribution is evident from very high epsilon ground motions produced by characteristic earthquakes on the San Andreas Fault System (M_W 8.2±0.2) about 69 km away from the site.

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 2,475-yr ARP on Figure 7. The 2,475-yr ordinates at 5% damping are also tabulated on Table 3 in Column 3.

The probabilistic MCE_R spectrum, which represents the maximum rotated, risk-targeted ordinates per ASCE 7-16, is shown on Figure 8. The ordinates are tabulated on Table 3 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 exceedance in 50 years. The adjustment between average horizontal and maximum rotated component is based on the period-specific ratios in Shahi and Baker (2014). The adjustment between the hazard and risk-targeted ordinates is based on the mapped ratios provided by ASCE 7-16 Method 1 (21.2.1.1). At the site latitude and longitude, a scale factor of 0.909 is specified for periods 0.2-second and shorter and a scale factor of 0.903 is used for periods of 1.0-second and longer; scale factors for periods between 0.2- and 1.0-second are linearly interpolated. Both of these scale factors are incorporated in the probabilistic MCE_R spectrum shown on Figure 8, and the process of developing the probabilistic MCE_R spectral ordinates is shown on Table 3 in Columns 3 through 6.

5. DETERMINISTIC SEISMIC HAZARD

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 Fa factor for Site Class D is 1.0, thus resulting in a threshold of 1.2 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, the several faults were evaluated for the DSHA. Table 2 lists the key contributors to the DSHA ground motions, as well as the fault parameters used in the analysis. The DSHA scenarios were evaluated using the ground-motion models and site parameters defined above in Section 3.

Predicted spectral amplitudes for each of these DSHA scenarios are shown on Figure 9. The DSHA ordinates reflect the 84^{th} percentile maximum rotated component of ground motion. The modification from the average horizontal component of ground motion to the maximum rotated component was performed using the same methodology described above for the development for the probabilistic MCE_R.

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 9, the Compton Fault and the San Pedro Escarpment cases present very similar spectral accelerations across all period range, with the San Pedro Escarpment controlling the short periods and the Compton Fault case slightly exceeding the San Pedro Escarpment case for periods above the spectral peak. At larger periods, i.e. above 3 seconds, the Newport Inglewood case controls the deterministic MCE_R spectrum. The deterministic MCE_R spectral ordinates are tabulated in Table 3 in Column 10, and the process of developing the deterministic MCE_R spectral ordinates is shown on Table 3 in Columns 7 through 10.

6. SITE-SPECIFIC RESPONSE SPECTRA

As this structure is being carried out in conformance with the 2016 California Building Code (CBC 2016) and ASCE 7-16 requirements, a "Maximum Considered Event" uniform hazard spectrum with risk-targeted, maximum-rotated ordinates at 5% damping was developed for the foundation level condition and is referred to as the final site-specific MCE_R response spectrum.

Figure 10 shows the development of the final site-specific MCE_R response spectrum. As stipulated in ASCE 7-16 Section 21.2.3, the MCE_R is based on the lesser of the deterministic MCE_R and the probabilistic MCE_R response spectra, which are both defined as the 5% damped acceleration response spectra. The deterministic MCE_R exceeds the probabilistic MCE_R across the full range of spectral periods, therefore the probabilistic MCE_R controls the site-specific MCE_R as shown on Figure 10. The final spectrum is then adjusted such that none of the spectral ordinates fall below 80% of the code-based MCE_R (also shown on Figure 10), as applicable. The final site-specific MCE_R spectrum is shown highlighted on Figure 10, and the spectral ordinates are tabulated in Table 3, Column 12. The process of developing the outcropping site-specific MCE_R spectral ordinates is shown in Table 3 in Columns 6 and 10 through 12.

The final site-specific ground surface Design Response Spectrum (DRS) was developed as 2/3 of site-specific ground surface MCE_R. The process of developing the DRS ordinates is shown in Table 4.

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

- S_{DS} = 1.238 g, based on 90% of the spectral acceleration at a period of 0.3-seconds
- S_{D1} = 0.820 g, based on the site V_{S30} and T*Sa at a period of 1.5-second
- S_{MS} = 1.856 g, based on 1.5 times S_{DS}
- S_{M1} = 1.230 g, based on 1.5 times S_{D1}

Lastly, the code-based peak ground acceleration PGA_M (MCE-level) from Section 11.8.3 of ASCE 7-16 requirements is 0.880 g for Site Class D. For the purpose of liquefaction evaluation, the site-specific ground surface MCE_R spectral acceleration at a period of 0.01-second can be used in lieu of the code-based value (cf. Table 3, Column 12). The magnitude for Magnitude Scaling Factors (MSFs) can be based on the mean magnitude from the 1.0-second spectral period hazard deaggregation in Figure 6 at the hazard level of interest (e.g., a M_W 7 at 10 km can be used for the MCE_R hazard level). To evaluate acceleration at depth for liquefaction evaluation purposes, we recommend using r_d reduction factors by Idriss and Boulanger (2008).

7. 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. This addendum addresses ground motion design only (i.e., response spectra) and does not evaluate any potential for surface rupture hazard, liquefaction, or other earthquake-related phenomena.

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.

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TABLE 1
CHARACTERIZATION[1] OF FAULTS SIGNIFICANT TO THE
12575 BEATRICE PLAYA VISTA GROUND-MOTION DEVELOPMENT PROJECT

Fault Name	Style of Faulting[2]	Maximum Magnitude (Mw)	Slip Rate (mm/yr)	Closest Rupture Distance From Site (km)
Newport-Inglewood	SS	7.2	1.2	5
Santa Monica	OBL	6.7	1.1	8
Puente Hills (LA)	RV	6.8	0.6	10
Compton	RV	7.3	0.8	10
San Pedro Escarpment	RV	7.1	0.2	10
SanVicente	RV	6.1	0.2	10
Hollywood	OBL	6.5	1.3	11
Palos Verdes	SS	7.4	2.3	11
Malibu Coast	OBL	6.9	0.5	11
North Salt Lake	RV	5.8	0.1	13
Anacapa-Dume	OBL	7.1	0.7	13
Puente Hills	RV	7.0	1.7	15
Elysian Park (Lower)	RV	6.8	0.1	15
Santa Monica Bay	RV	6.8	0.1	18
Elysian Park (Upper)	RV	6.5	1.4	19
Redondo Canyon	RV	6.6	0.4	19
Raymond	OBL	6.6	1.3	24
Verdugo	RV	6.8	0.6	27
Puente Hills (Santa Fe Springs)	RV	6.4	0.8	27
Northridge Hills	RV	6.8	1.3	28
Northridge	RV	6.9	1.5	28
San Pedro Basin	SS	7.1	1.1	29
Santa Susana East (connector)	RV	6.2	1.9	30
Mission Hills	RV	6.3	0.8	31
Sierra Madre	RV	7.2	1.5	33
Elsinore - Whittier[3]	SS	7	4.2	34
Sierra Madre (San Fernando)	RV	6.5	1.6	34
Anaheim	RV	6.3	0.1	35
Puente Hills (Coyote Hills)	RV	6.7	0.8	35
Santa Susana	RV	6.9	3.2	38
SanGabriel (Extension)	SS	7.1	0.5	39
San Gabriel	OBL	7.3	0.6	40
Holser	RV	6.7	0.5	43
Clamshell-Sawpit	RV	6.4	0.3	44
Simi-Santa Rosa	OBL	6.8	1.1	44
Malibu Coast (Extension)	OBL	6.9	0.8	48
San Jose	OBL	6.5	0.3	50
Richfield	RV	6.1	0.2	50
Oak Ridge (Onshore)	RV	7.1	2.6	50
Yorba Linda	RV	6.3	0.1	51
Peralta Hills	RV	6.4	0.4	51
Del Valle	RV	6.2	1.0	52
San Joaquin Hills	RV	6.8	0.5	55
Chino	OBL	6.7	0.9	56
Santa Cruz-Catalina Ridge	OBL	7.4	1.1	59
San Cayetano	RV	7.1	2.9	60
Sisar	RV	6.8	0.8	60
San Diego Trough North	SS	7.3	1.6	63
Newport-Inglewood Offshore	SS	7	1	63
Cucamonga	RV	6.8	1.7	64
San Andreas[3]	SS	8.2	29	69
Ventura-Pitas Point	OBL	7.1	1.5	73
Fontana	SS	6.6	0.3	76
Oceanside Blind Thrust	RV	7.2	0.7	76
Santa Ynez (East)	SS	7.2	1.5	78
Santa Cruz Island	OBL	7.2	0.85	78
Channel Islands Thrust	RV	7.2	1	78
Pine Mountain	RV	7.2	0.3	79
Oak Ridge (Offshore)	RV	6.9	1.7	81
SanClemente	SS	7.5	1.76	85
San Jacinto[3]	SS	7.9	6	86
Mission Ridge-Arroyo Parida-Santa Ana	RV	7	1.1	86
Red Mountain	RV	7.4	2.18	91
Channel Islands Western Deep Ramp	RV	7.2	0.41	92
Cleghorn	SS	6.7	0.45	94
Coronado Bank	SS	7.4	1.83	98
Big Pine (Central)	OBL	6.5	1	103
Garlock[3]	SS	7.5	6	104

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 2
DETERMINISTIC SEISMIC HAZARD ANALYSIS FAULT CHARACTERIZATION
12575 BEATRICE PLAYA VISTA GROUND-MOTION DEVELOPMENT PROJECT

Fault	Column 1	Column 2	Column 3	Column 4	Column 5	Column 6	Column 7	Column 8	Column 9	Column 10	Column 11	Column 12
	M_W	F_{RV}	F_N	F_{HW}	Z_{TOR}	Z_{BOT}	Dip	W	Z_{HYP}	R_{RUP}	R_{JB}	R_X
Newport-Inglewood Onshore	7.4	0	0	0	0	15	90	15.0	10.2	5.4	5.4	5.4
Compton	7.3	1	0	1	5.2	15	20	28.7	9.4	10.0	0	13
Elysian Park Upper	6.5	1	0	0	3	15	50	15.7	11.0	18.4	-18.2	-18.2
Palos Verdes	7.4	0	0	0	0	13.6	90	13.6	10.2	11	11	11
Puente Hills LA	6.8	1	0	0	2.1	15	27	28.4	7.8	9.6	-9	-9.4
Puente Hills Alt1	7	1	0	0	5	13	25	18.9	10.2	14.8	-14	-13.9
Whittier-Elsinore	7	0	0	0	0	15.5	75	16.0	10.2	33.9	33.9	33.9
Hwood-Santa Monica	7	1	0	0	0	17.3	70	18.4	10.2	7.8	7.8	-7.8
San Andreas	8.2	0	0	0	0	13.1	90	13.1	10.2	69.2	69.2	69.2
San Pedro Escarpment	7.1	1	0	1	1	12	20	32.2	5.2	9.9	0.0	25.8

Key

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

TABLE 3
SITE-SPECIFIC MCE_R DEVELOPMENT CALCULATION SHEET
12575 BEATRICE PLAYA VISTA GROUND-MOTION DEVELOPMENT 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 Deteterministic Minimum MCE_R*	*Deterministic MCE_R*	*Code Minimum MCE_R*	*Final Outcropping 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	0.745	0.909	1.190	0.806	2.946	3.506	0.468	1.367	0.674	0.806
0.020	50	0.756	0.909	1.190	0.818	1.160	1.381	0.473	1.381	0.749	0.818
0.030	33	0.779	0.909	1.190	0.842	1.199	1.426	0.488	1.426	0.824	0.842
0.050	20	0.902	0.909	1.190	0.976	1.378	1.639	0.561	1.639	0.973	0.976
0.075	13	1.132	0.909	1.190	1.225	1.657	1.971	0.675	1.971	1.161	1.225
0.100	10	1.328	0.909	1.190	1.437	1.904	2.265	0.775	2.265	1.348	1.437
0.150	6.67	1.565	0.909	1.200	1.707	2.211	2.654	0.908	2.654	1.497	1.707
0.200	5.00	1.712	0.909	1.210	1.883	2.493	3.017	1.032	3.017	1.497	1.883
0.250	4.00	1.802	0.909	1.220	1.997	2.645	3.227	1.104	3.227	1.497	1.997
0.300	3.33	1.861	0.908	1.220	2.063	2.806	3.423	1.172	3.423	1.497	2.063
0.400	2.50	1.813	0.908	1.230	2.024	2.851	3.506	1.200	3.506	1.497	2.024
0.500	2.00	1.699	0.907	1.230	1.895	2.649	3.258	1.115	3.258	1.497	1.895
0.750	1.33	1.351	0.905	1.240	1.516	2.112	2.619	0.896	2.619	1.197	1.516
1.000	1.00	1.091	0.903	1.240	1.221	1.631	2.022	0.692	2.022	0.898	1.221
1.500	0.67	0.732	0.903	1.240	0.820	1.031	1.279	0.438	1.279	0.598	0.820
2.000	0.50	0.533	0.903	1.240	0.597	0.700	0.868	0.297	0.868	0.449	0.597
3.000	0.33	0.323	0.903	1.250	0.364	0.403	0.503	0.172	0.503	0.299	0.364
4.000	0.25	0.216	0.903	1.260	0.246	0.283	0.357	0.122	0.357	0.224	0.246
5.000	0.20	0.158	0.903	1.260	0.180	0.208	0.262	0.090	0.262	0.180	0.180
7.500	0.13	0.089	0.903	1.280	0.103	0.103	0.132	0.045	0.132	0.120	0.120
10.000	0.10	0.057	0.903	1.290	0.066	0.060	0.077	0.026	0.077	0.072	0.072

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 (C_R) 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.

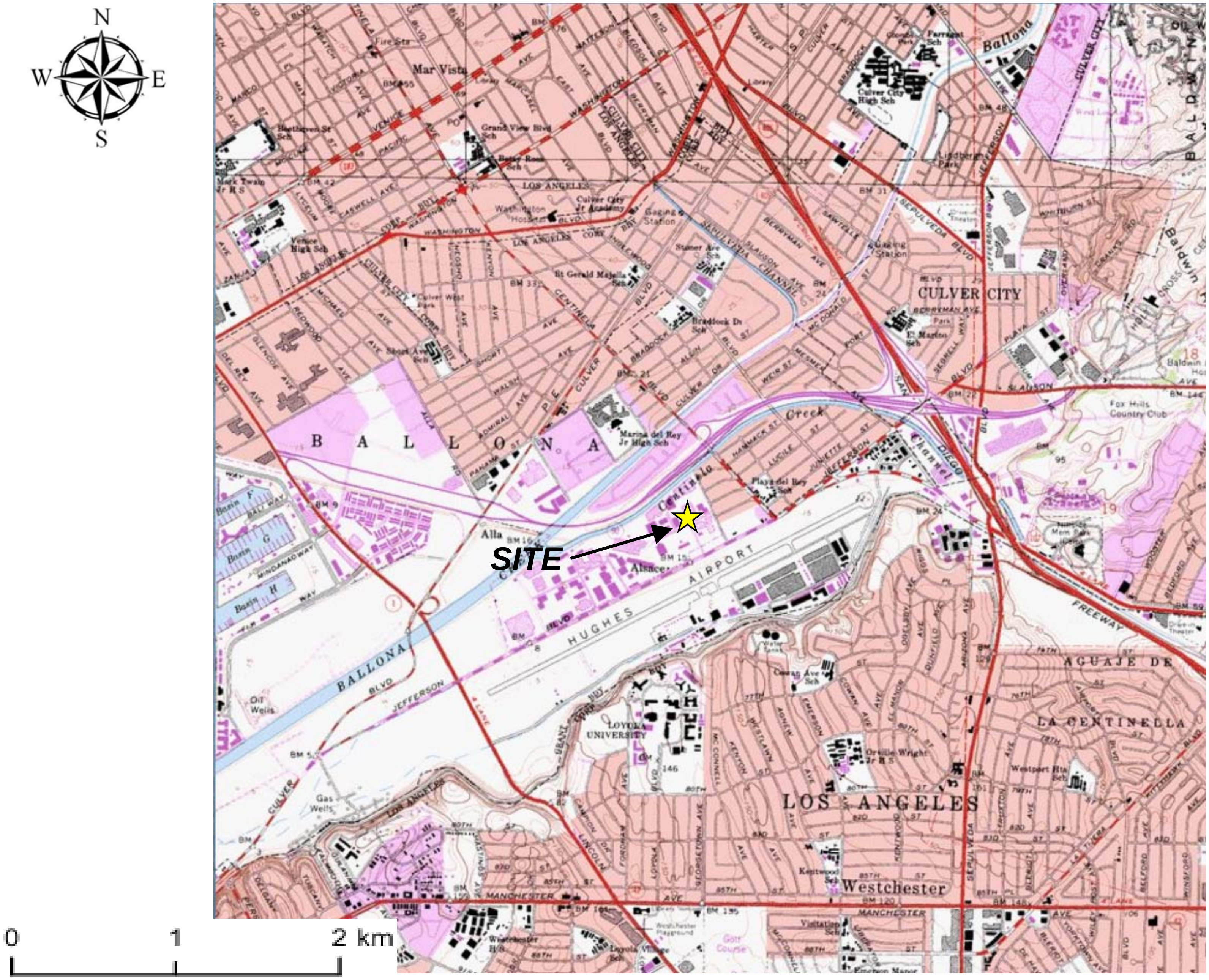
TABLE 4
SITE-SPECIFIC DRS DEVELOPMENT CALCULATION SHEET
12575 BEATRICE PLAYA VISTA GROUND-MOTION DEVELOPMENT PROJECT

Column 1	Column 2	Column 3	Column 4	Column 5	Column 6
		Code-Based DRS	*80% of Code-Based DRS*	*2/3 of MCE $_R$*	*Final Outcropping Site-Specific DRS*
Period	***Frequency***	*RotD100*	*RotD100*	*RotD100*	*RotD100*
(sec)	(Hz)	(g)	(g)	(g)	(g)
0.010	100	0.561	0.449	0.538	0.538
0.020	50	0.624	0.499	0.546	0.546
0.030	33	0.686	0.549	0.562	0.562
0.050	20	0.811	0.649	0.650	0.650
0.075	13	0.967	0.774	0.816	0.816
0.100	10	1.123	0.899	0.958	0.958
0.150	6.67	1.247	0.998	1.138	1.138
0.200	5.00	1.247	0.998	1.256	1.256
0.250	4.00	1.247	0.998	1.331	1.331
0.300	3.33	1.247	0.998	1.375	1.375
0.400	2.50	1.247	0.998	1.349	1.349
0.500	2.00	1.247	0.998	1.263	1.263
0.750	1.33	0.997	0.798	1.010	1.010
1.000	1.00	0.748	0.598	0.814	0.814
1.500	0.67	0.499	0.399	0.547	0.547
2.000	0.50	0.374	0.299	0.398	0.398
3.000	0.33	0.249	0.199	0.243	0.243
4.000	0.25	0.187	0.150	0.164	0.164
5.000	0.20	0.150	0.120	0.120	0.120
7.500	0.13	0.100	0.080	0.080	0.080
10.000	0.10	0.060	0.048	0.048	0.048

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

Key

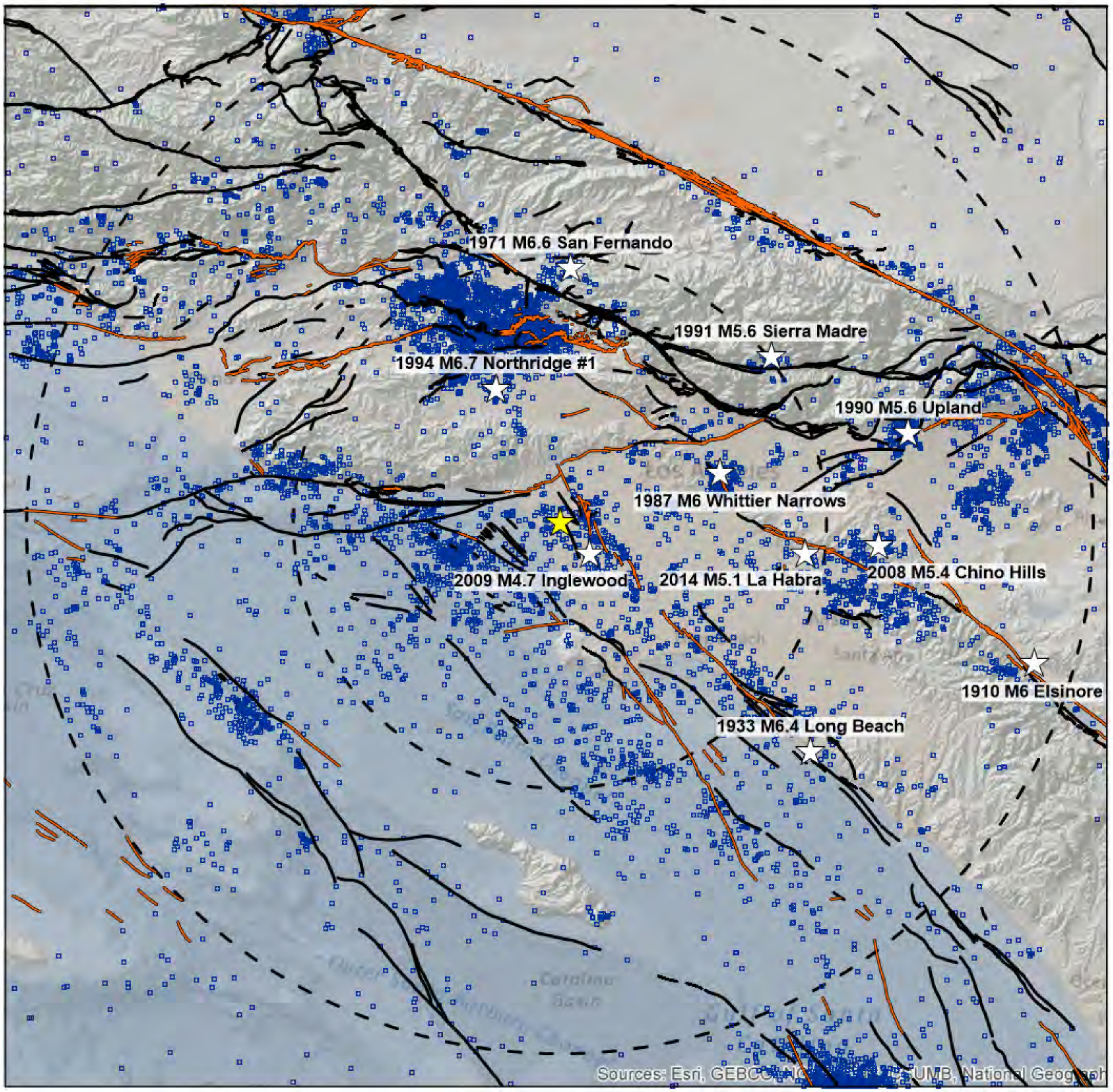
Column 1	= Spectral period in seconds.
Column 2	= Spectral frequency (inverse of spectral period) in Hertz.
Column 3	= Code-based (ASCE 7-16, Ch. 11) design ground-motion spectral ordinates in units of g for 5% damping.
Column 4	= Code-based (ASCE 7-16, Ch. 21) minimum design ground-motion spectral ordinates 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.



USGS
Topographic
Map:
Venice, CA

DETAILED SITE LOCATION MAP			
Date: FEB. 2018	Project No.: 17025A	Project: 12575 BEATRICE PLAYA VISTA GROUND-MOTION DEVELOPMENT	Figure 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

0 km 20 km 40 km 60 km

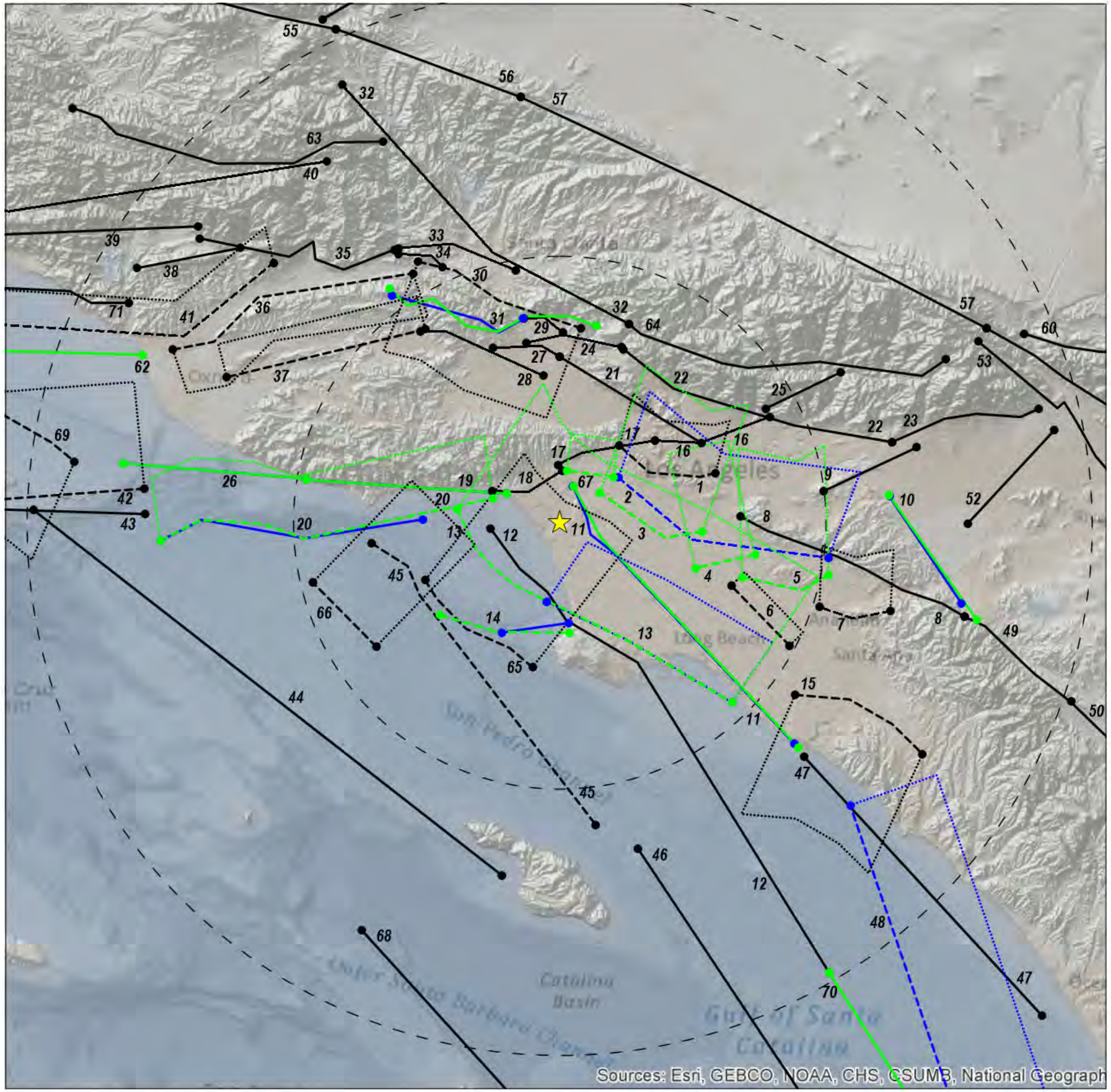
N

Datum & Projection: NAD83 UTM Zone 11

REGIONAL FAULT & SEISMICITY MAP

Date: FEB. 2018	Project No.: 17025A	Project: 12575 BEATRICE PLAYA VISTA GROUND-MOTION DEVELOPMENT	Figure 2

GeoPentech



N

Approximate Scale (km)

0 5 10 15 20 25 30 35 40 45 50

Datum & Projection: NAD83 UTM Zone 11

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)

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

Legend

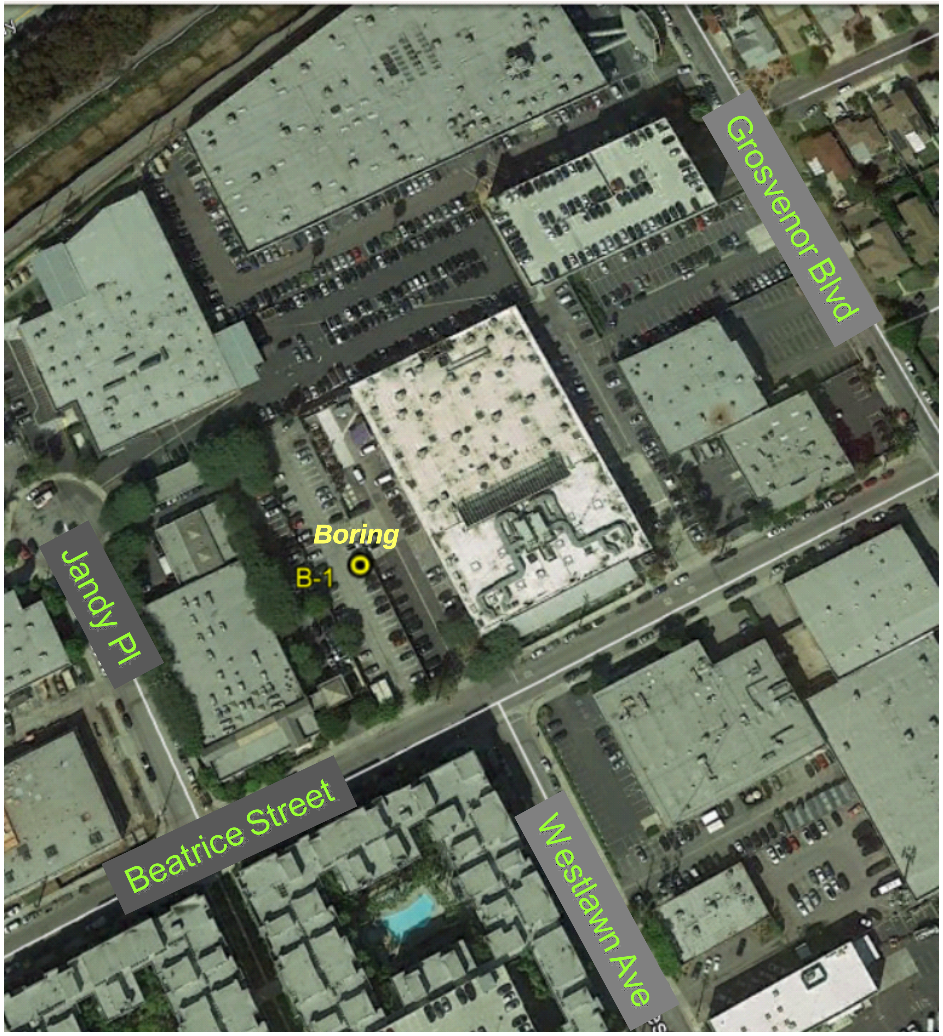
- *Surface Trace, Top of Fault (Both Fault Models)*
- *Blind Trace, Top of Fault (Both Fault Models)*
- *Blind Thrust Footprint (Both Fault Models)*
- *Surface Trace, Top of Fault (Fault Model 1)*
- *Blind Trace, Top of Fault (Fault Model 1)*
- *Blind Thrust Footprint (Fault Model 1)*
- *Surface Trace,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

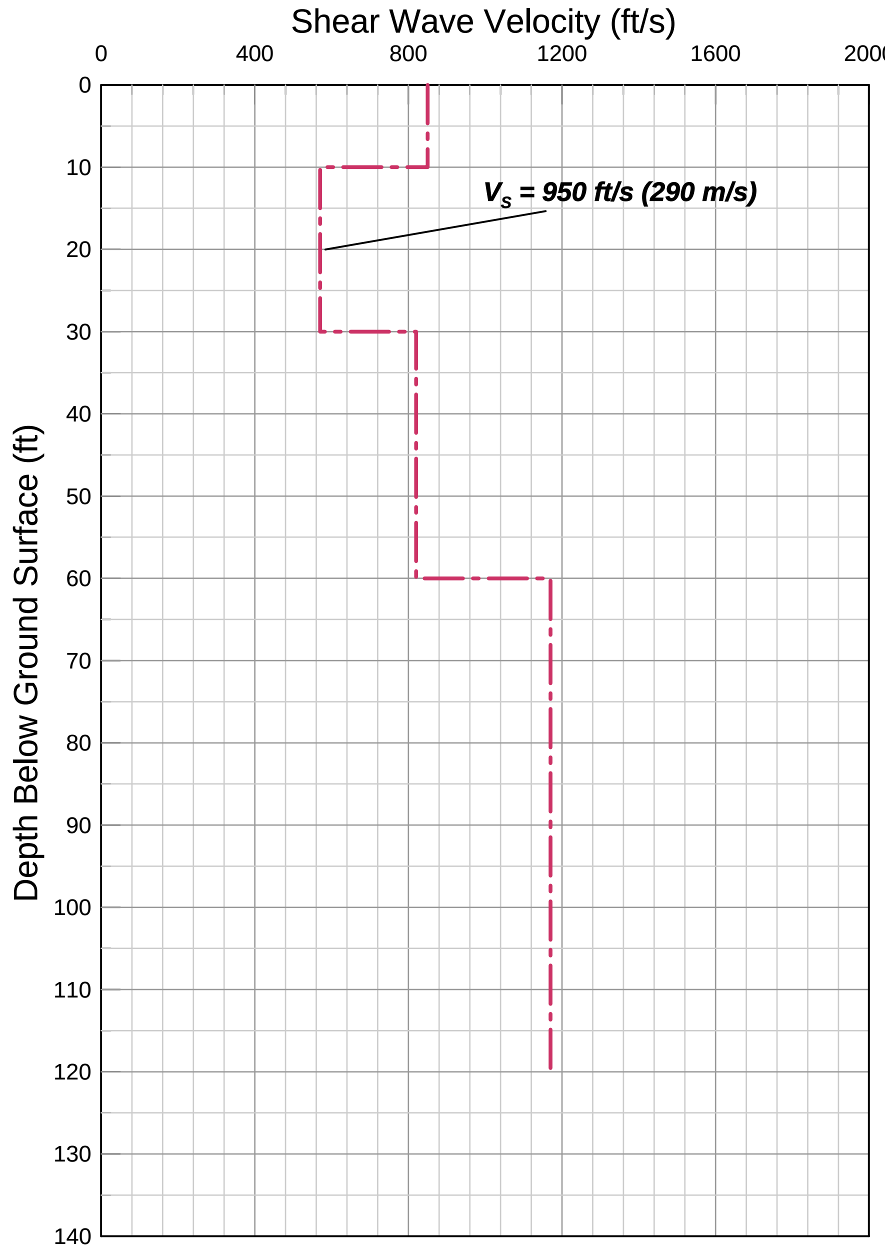
Notes:

1. 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, except for the following: Elysian Park (Lower), North Salt Lake, Richfield, and Yorba Linda (however, these faults are included in the PSHA). Slip rates are solution mean rates from UCERF3 (WGCEP, 2013a). 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 (2013a). Magnitude-frequency distributions approximate the SWUS WAACY model (GeoPentech, 2015) with characteristic magnitude calculated from Shaw (2009) regression. Type A faults characterized as documented in WGCEP (2008) and 2008 NSHM (Petersen et al., 2008).

SIMPLIFIED FAULT MAP FOR PSHA		
Project: 12575 BEATRICE GROUND MOTIONS		Figure 3
Project No.: 17025A	Date: FEB. 2018	

GeoPentech

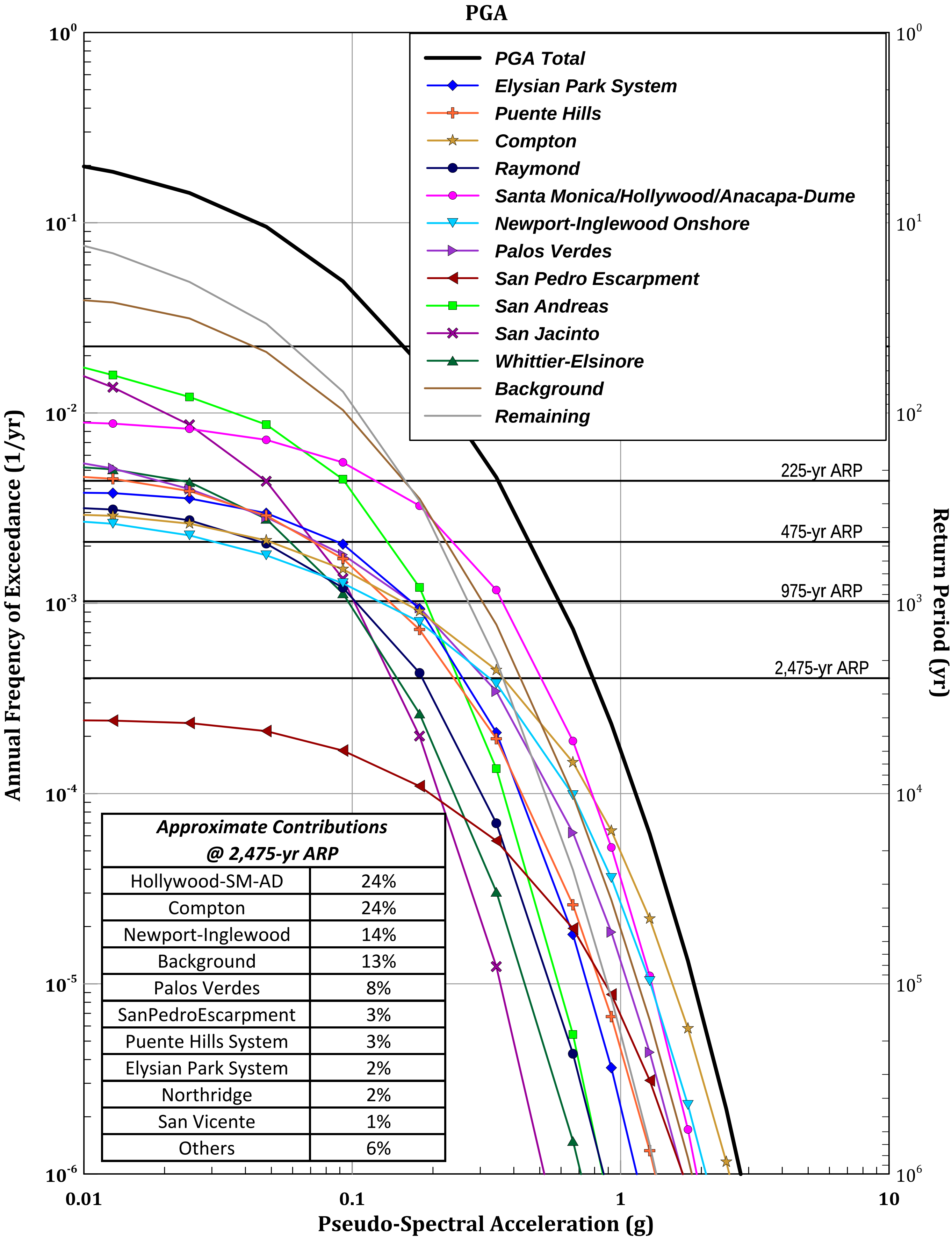




SITE SHEAR-WAVE VELOCITY

Date: FEB. 2016	Project No.: 17025A	Project: 12575 BEATRICE PLAYA VISTA GROUND-MOTION DEVELOPMENT	Figure 4

GeoPentech

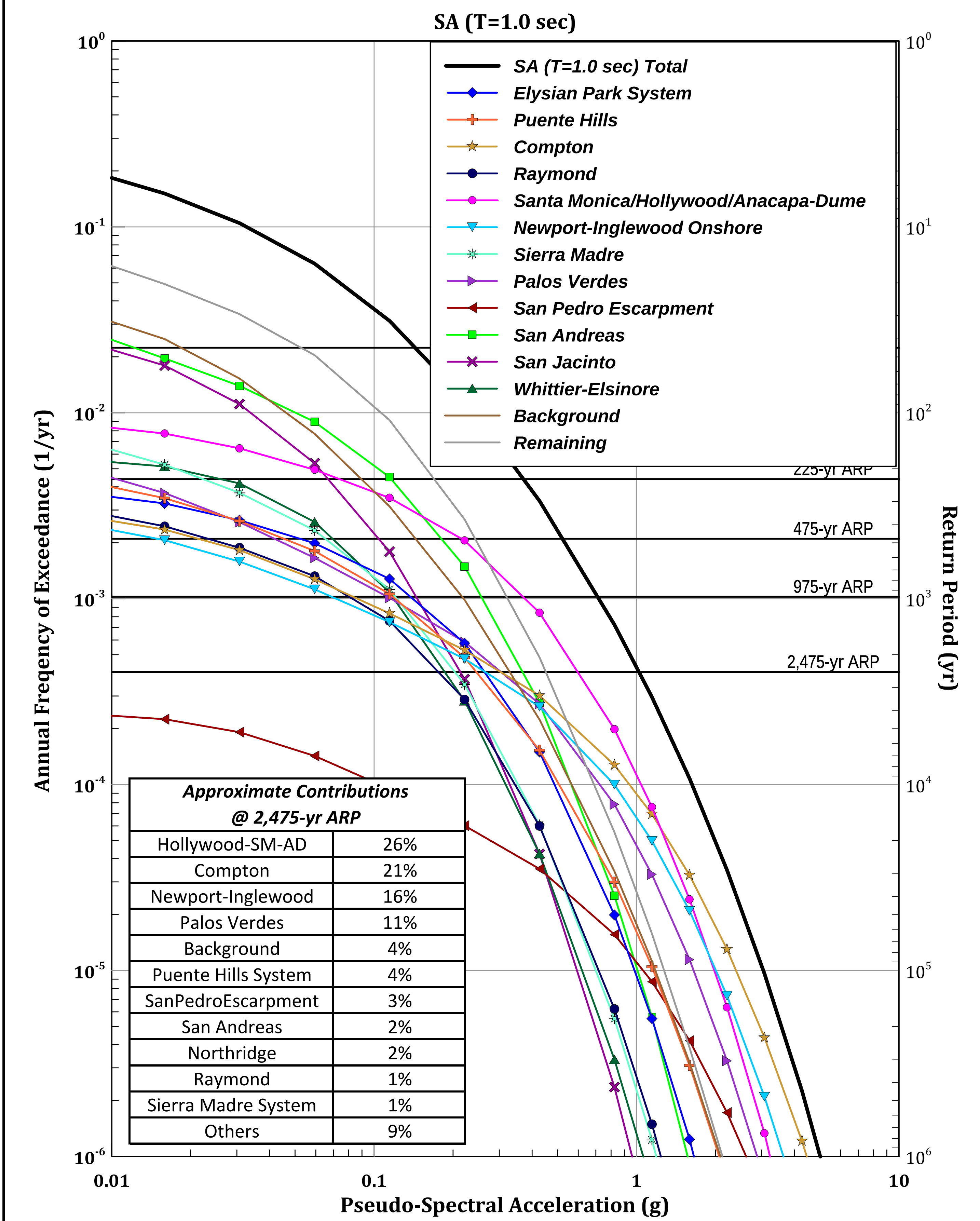


Approximate Contributions @ 2,475-yr ARP	
Hollywood-SM-AD	24%
Compton	24%
Newport-Inglewood	14%
Background	13%
Palos Verdes	8%
SanPedroEscarpment	3%
Puente Hills System	3%
Elysian Park System	2%
Northridge	2%
San Vicente	1%
Others	6%

SOURCE CONTRIBUTIONS TO THE TOTAL HAZARD AT PGA

Project No.: 17025A	Project: 12575 BEATRICE GROUND MOTIONS	Date: FEB. 2018	Figure 5a

GeoPentech

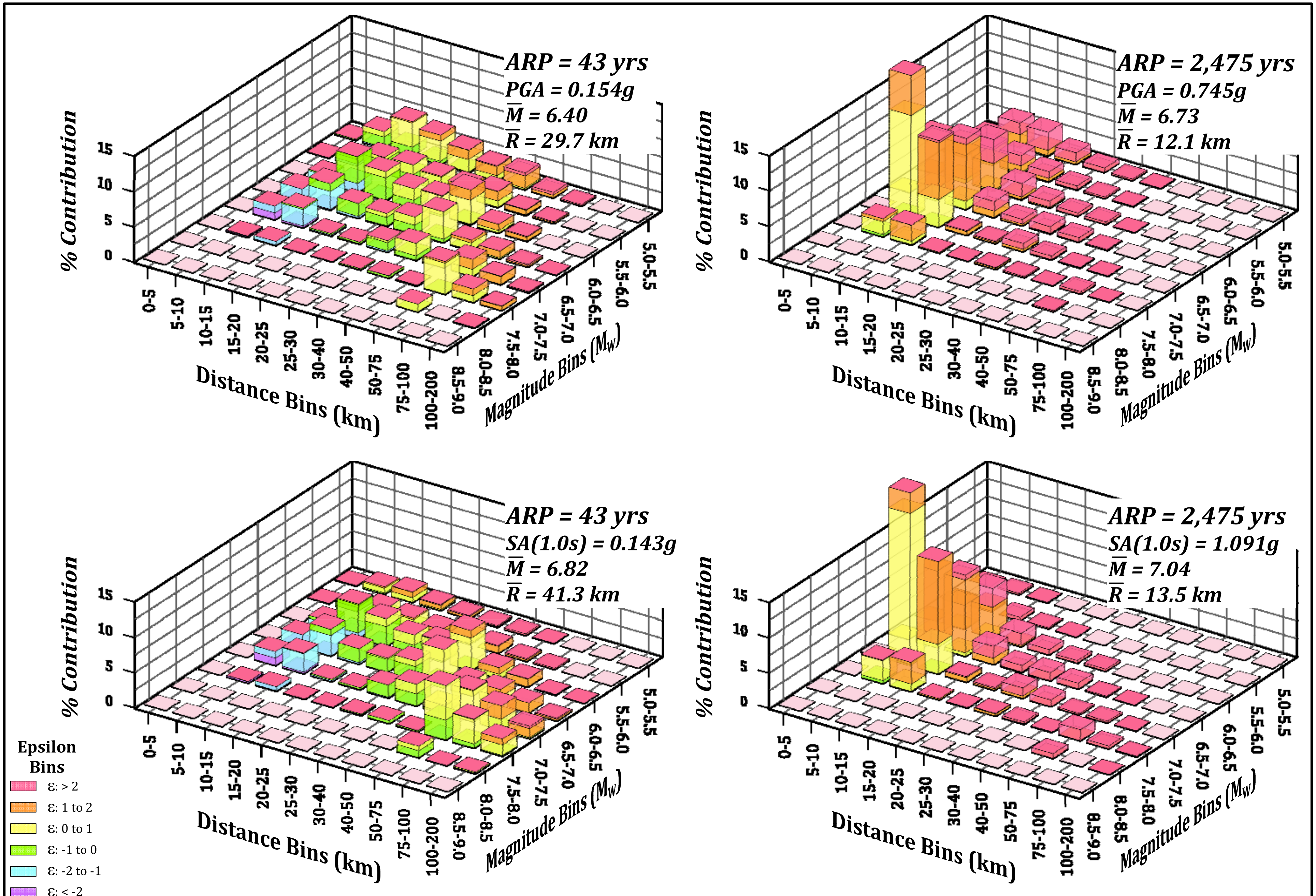


Approximate Contributions @ 2,475-yr ARP	
Hollywood-SM-AD	26%
Compton	21%
Newport-Inglewood	16%
Palos Verdes	11%
Background	4%
Puente Hills System	4%
SanPedroEscarpment	3%
San Andreas	2%
Northridge	2%
Raymond	1%
Sierra Madre System	1%
Others	9%

SOURCE CONTRIBUTIONS TO THE TOTAL HAZARD AT T=1.0 SEC

Project No.: 17025A	Project: 12575 BEATRICE GROUND MOTIONS	Date: FEB. 2018	Figure 5b

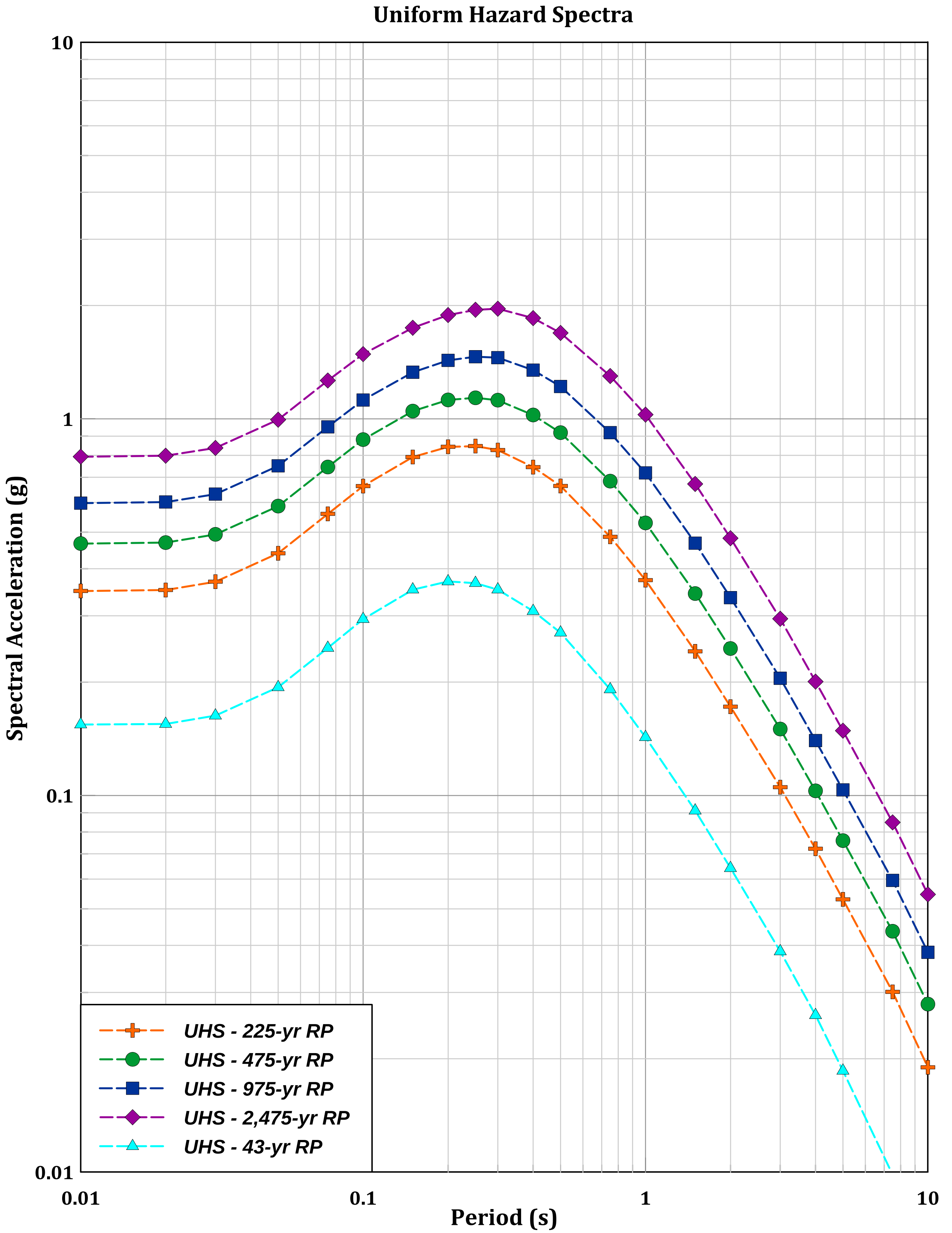
GeoPentech



HAZARD DEAGGREGATION FOR PGA & 1.0-SECOND SPECTRAL PERIOD

Project No.: 17025A	Date: MAR. 2020	Project: 12575 BEATRICE PLAYA VISTA GROUND-MOTION DEVELOPMENT	Figure 6

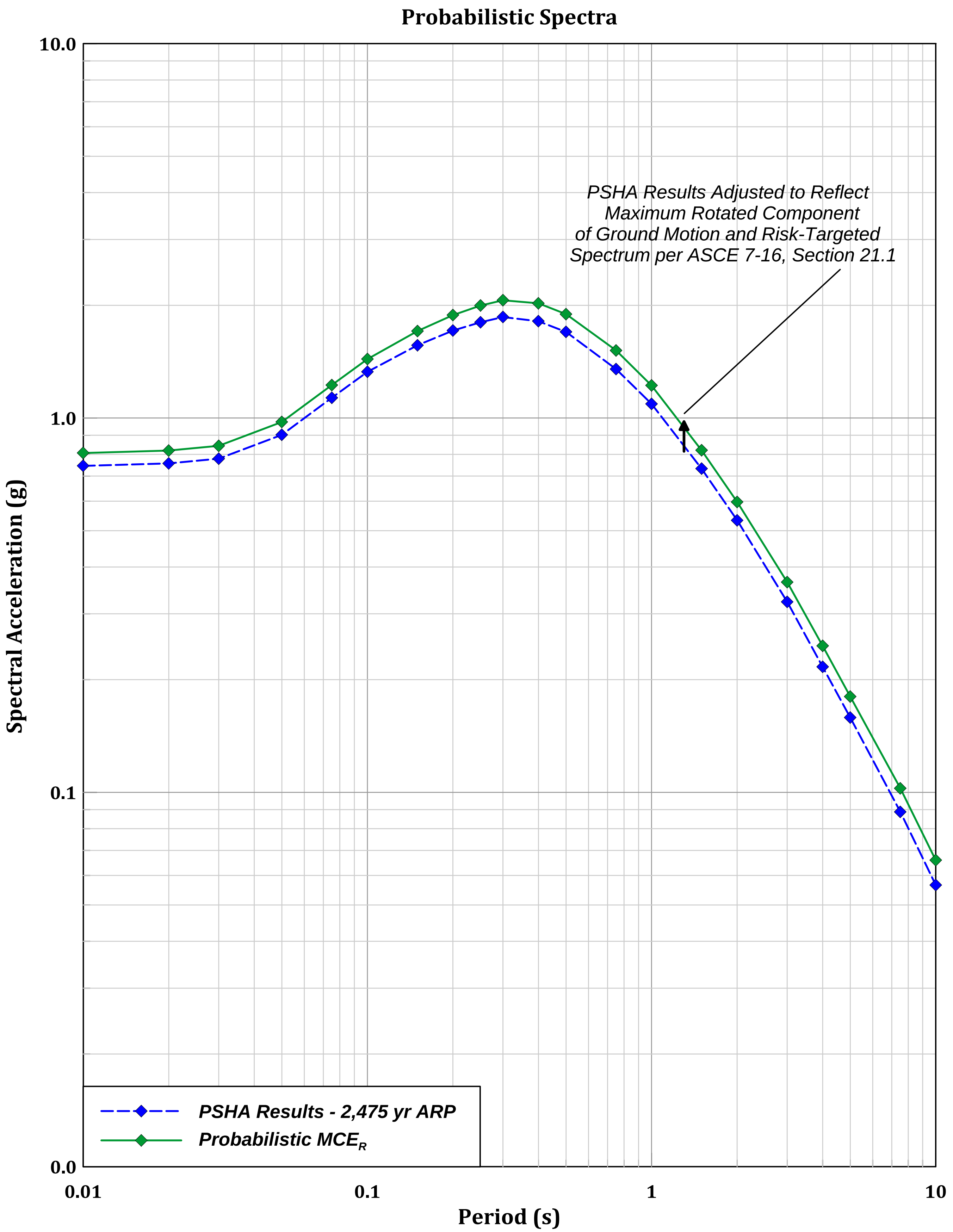
GeoPentech



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

UNIFORM HAZARD SPECTRA			
Project No.: 17025A	Project: 12575 BEATRICE GROUND MOTIONS	Date: FEB. 2018	Figure 7

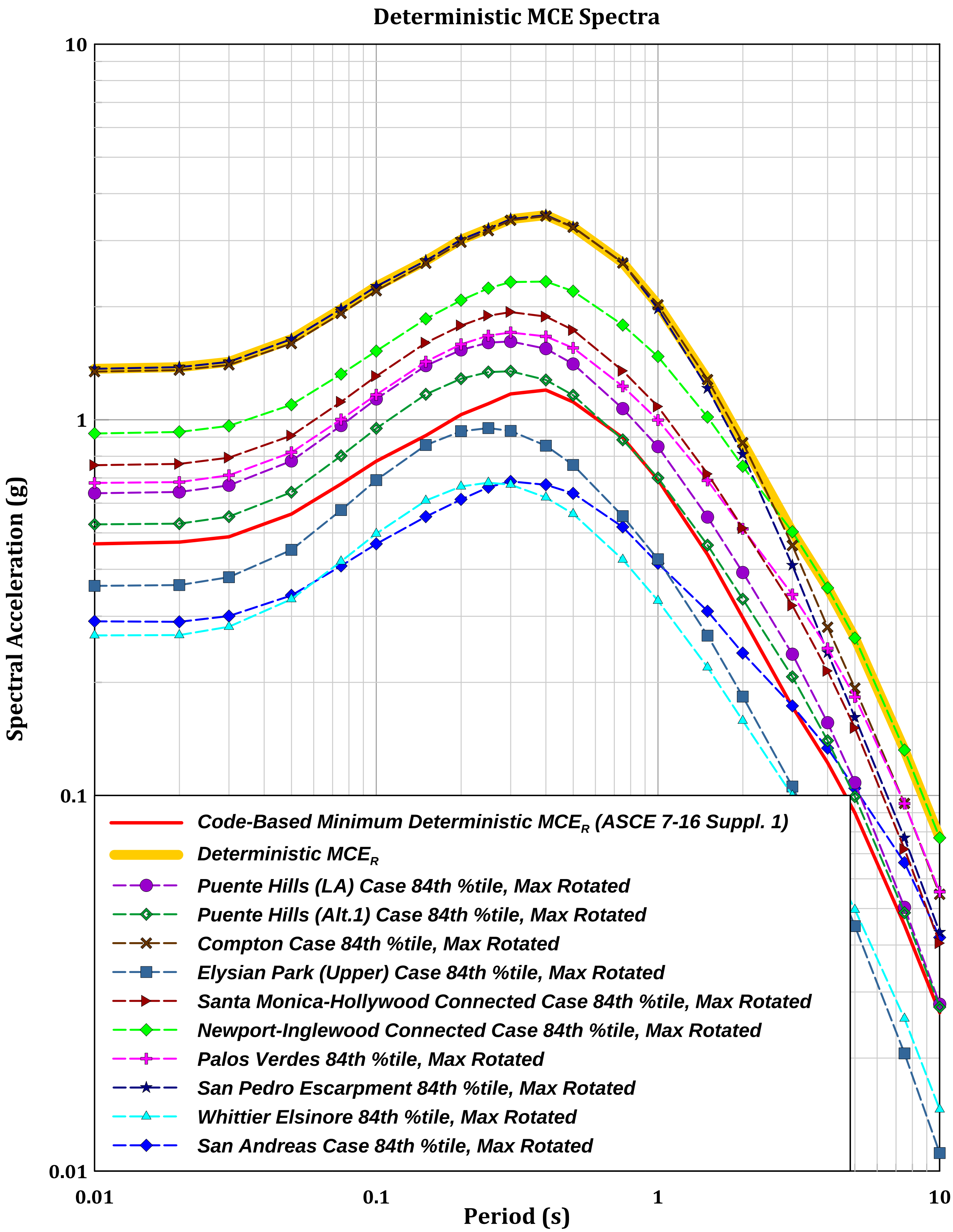
GeoPentech



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

PROBABILISTIC SPECTRA			
Project No.: 17025A	Project: 12575 BEATRICE GROUND MOTIONS	Date: MAR. 2020	Figure 8

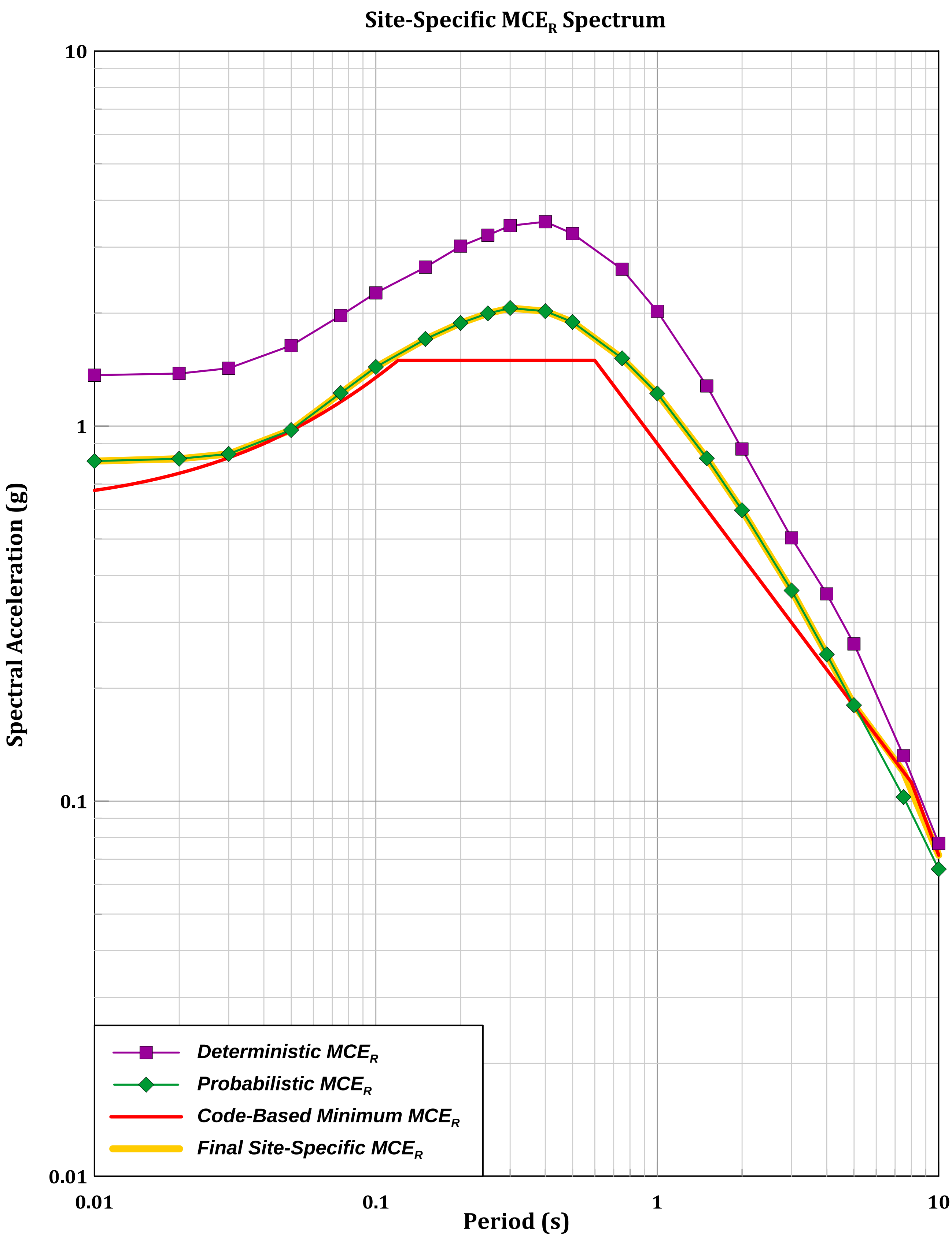
GeoPentech



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

DETERMINISTIC SPECTRA			
Project No.: 17025A	Project: 12575 BEATRICE GROUND MOTIONS	Date: MAR. 2020	Figure 9

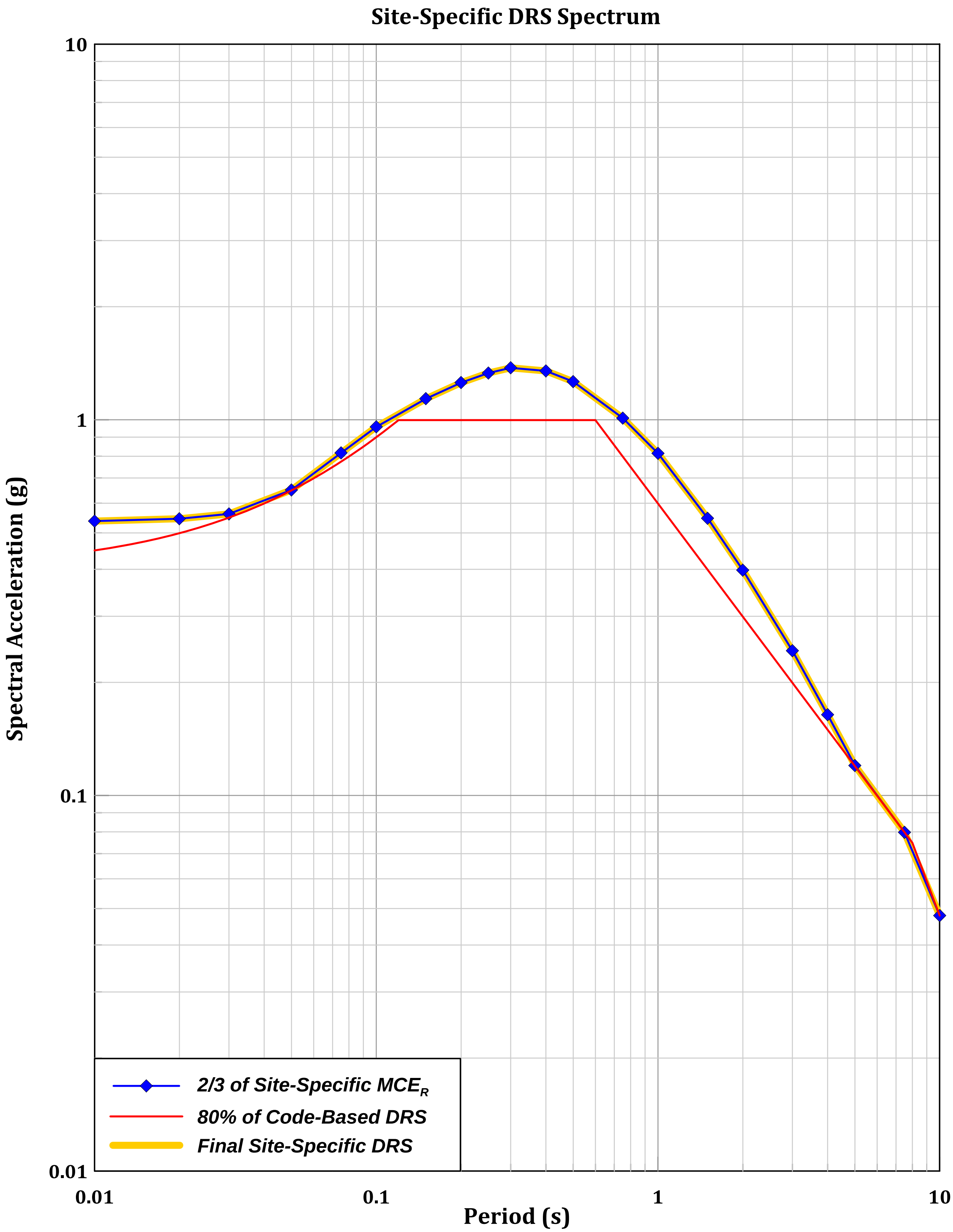
GeoPentech



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

SITE-SPECIFIC MCE_R SPECTRUM			
Project No.: 17025A	Project: 12575 BEATRICE GROUND MOTIONS	Date: MAR. 2020	Figure 10

GeoPentech



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

SITE-SPECIFIC DRS SPECTRUM

Project No.: 17025A	Project: 12575 BEATRICE GROUND MOTIONS	Date: MAR 2020	Figure 11

GeoPentech

APPENDIX A

Downhole Seismic Tests

APPENDIX A – Downhole Seismic Tests

This appendix presents the methods and results of the downhole seismic tests performed at the subject property. Downhole seismic tests were completed within Boring No. 1 on January 17, 2018 by GeoPentech. 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." The geophysical data were collected, processed, and interpreted by a California-licensed Professional Geophysicist (PGp).

Boring No. 1 was drilled and logged by Geotechnologies, Inc. on December 18, 2017, and a copy of the borehole log is included at the end of this appendix. Boring No. 1 was drilled with a 5-inch diameter bit using rotary wash drilling methods and a 2-inch diameter PVC casing was installed under the direction of Geotechnologies, Inc. as part of their geotechnical investigation. The annular space between the 5-inch diameter hole and 2-inch diameter casing was backfilled with bentonite-cement grout, which was assumed to be formulated to approximate the density of the surrounding geologic material and pumped in from the base of the borehole to completely fill the annular space.

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-inch wide by 4-inch high 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 (120 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 5.1.1.2. 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^{o} polarity change is noted to have occurred on the reverse hit. For analysis, a 15 Hz low-cut filter and 500 Hz high-cut filter was applied to the P waveforms, and a 15 Hz low-cut filter and 168 Hz high-cut filter were 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 No. 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 plot of the P and SH-wave travel-times as a function of depth showing the interpreted layer velocities; (3) a table of the calculated P and SH-wave interval velocities; (4) a table of the interpreted P and SH-wave layer velocities and depth ranges; and (5) a plot of the layer and interval velocity models as a function of depth.

Table A-1 below summarizes the interpreted P and SH layer velocities and depths shown on Figure A-1 for the various geologic units logged by Geotechnologies, Inc. in Boring No. 1. It is noted that groundwater was observed at a depth of approximately 22½ feet during drilling.

TABLE A-1
SUMMARY OF SH-WAVE AND P-WAVE VELOCITY LAYERS WITHIN BORING NO. 1

<table>
<tr><th>PREDOMINANT LITHOLOGY</th><th>Depth Range (ft)</th><th>SH-WAVE Velocity (ft/sec)</th><th>P-WAVE Velocity (ft/sec)</th></tr>
<tr><td>Medium firm to stiff, sandy Clay (CL) and sandy Silt (ML)
[Fill]</td><td>0 to 10</td><td>850</td><td>1,980</td></tr>
<tr><td>Soft to stiff, silty Clay (CH and CL)
[Alluvium]</td><td>10 to 25</td><td rowspan="2">570</td><td>1,450</td></tr>
<tr><td>Medium firm to stiff, silty Clay (CL) and sandy Silt (ML)
[Alluvium]</td><td>25 to 30</td><td rowspan="2">4,850</td></tr>
<tr><td>Medium dense to dense, Sand with some gravel (SC, SP and SW)
[Alluvium]</td><td>30 to 45</td><td rowspan="2">820</td></tr>
<tr><td>Medium firm sandy Silt (ML) and
Medium dense to dense, Sand (SC and SP)
[Alluvium]</td><td>45 to 60</td><td rowspan="2">5,600</td></tr>
<tr><td>Very dense, Sand (SP and SW)
[Alluvium]</td><td>60 to 120</td><td>1,170</td></tr>
</table>

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 Boring No. 1 was calculated between a depth of 0 to 100 feet and 20 to 120 feet. The results are summarized on Table A-2.

TABLE A-2
CALCULATED V_{S30} WITHIN BORING NO. 1

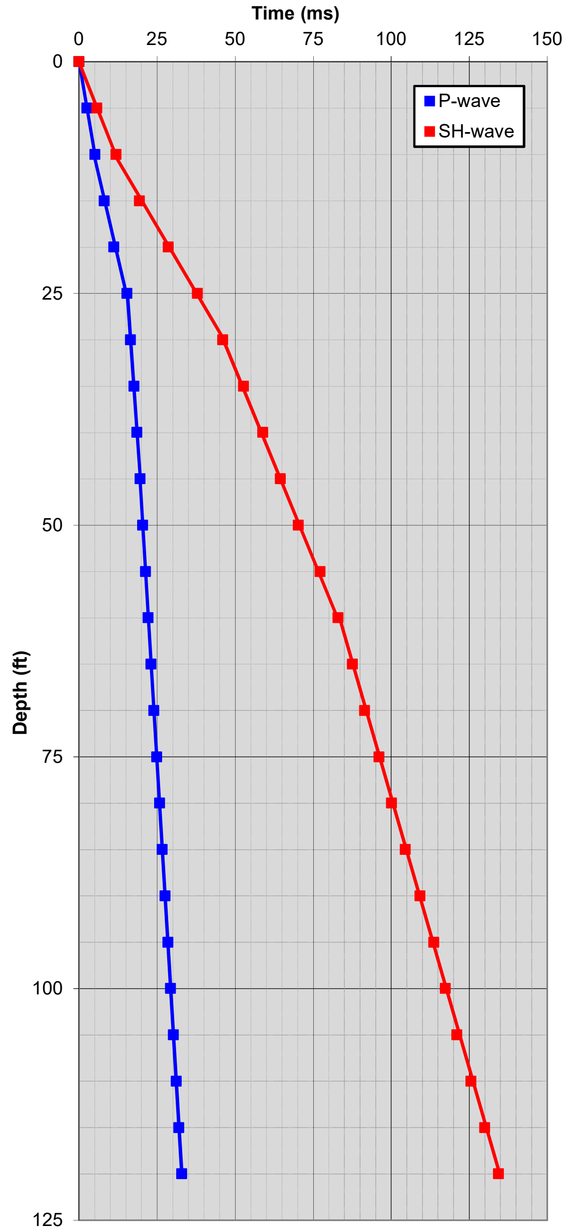
DEPTH RANGE (ft, below ground surface)	V_{S30} (ft/sec)
0 to 100	850
20 to 120	950

SEISMIC WAVE TRAVEL TIMES

Depth (ft)	P-time (ms)	P-layer	SH-wave (ms)	SH-layer
0	0	1	0	1
5	3.60	1	8.07	1
10	5.65	12	13.22	12
15	8.48	2	20.41	2
20	11.48	2	29.44	2
25	15.70	23	38.73	2
30	16.76	3	46.72	23
35	17.79	3	53.24	3
40	18.71	3	59.32	3
45	19.64	34	64.93	3
50	20.50	4	70.58	3
55	21.37	4	77.56	3
60	22.25	4	83.26	34
65	23.14	4	87.80	4
70	24.02	4	91.77	4
75	24.95	4	96.31	4
80	25.85	4	100.29	4
85	26.74	4	104.68	4
90	27.62	4	109.44	4
95	28.51	4	113.77	4
100	29.38	4	117.54	4
105	30.26	4	121.18	4
110	31.14	4	125.68	4
115	32.02	4	130.12	4
120	32.91	4	134.46	4

Source Offset (ft)
5

TRAVEL TIME PLOT



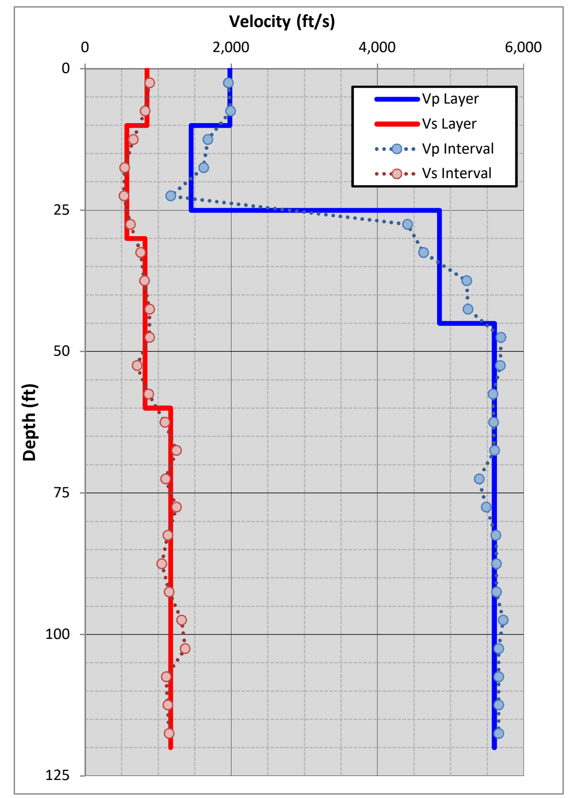
INTERVAL VELOCITES

Depth Range	P-Velocity (ft/s)	SH-Velocity (ft/s)
0 to 5	1,960	880
5 to 10	1,990	820
10 to 15	1,680	660
15 to 20	1,620	540
20 to 25	1,170	530
25 to 30	4,410	620
30 to 35	4,630	760
35 to 40	5,220	810
40 to 45	5,240	880
45 to 50	5,690	880
50 to 55	5,680	710
55 to 60	5,580	870
60 to 65	5,590	1,090
65 to 70	5,600	1,250
70 to 75	5,390	1,100
75 to 80	5,490	1,250
80 to 85	5,620	1,130
85 to 90	5,630	1,050
90 to 95	5,630	1,150
95 to 100	5,720	1,320
100 to 105	5,660	1,370
105 to 110	5,660	1,110
110 to 115	5,660	1,130
115 to 120	5,660	1,150

LAYER VELOCITES

Layer	P-Depth (ft)	P-Velocity (ft/s)	SH-Depth (ft)	SH-Velocity (ft/s)
1	0 to 10	1,980	0 to 10	850
2	10 to 25	1,450	10 to 30	570
3	25 to 45	4,850	30 to 60	820
4	45 to 120	5,600	60 to 120	1,170
5				
6				
7				
8				
9				
10				

VELOCITY MODEL



V_{S30} CALCULATION

Vs30 (ft/s)	Depth (ft)
850	0 to 100
950	20 to 120

GeoPentech

SEISMIC DOWNHOLE TEST RESULTS
BORING NUMBER 1

PROJECT: **12575 BEATRICE STREET**

PROJECT #: **17025A** | DATE: **JAN 2018** | FIGURE: **1**

BORING LOG NUMBER 1

Chait Company Architects **Date: 12/18/17**

File No. 21194 **Method: Used 5-inch diameter Rotary Wash Drill Rig**

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Asphalt for Parking
				0 --		4-inch Asphalt over 4-inch Base
				-		
				1 --		FILL: Sandy Clay, dark brown, moist, stiff
				-		
				2 --		
				-		
				3 --		Sandy Clay, dark and gray, moist, medium firm to stiff
				-		
				4 --		
				-		
5	12	26.2	SPT	5 --		
				-		
				6 --		
				-		
				7 --		
7.5	18	14.4	115.3	-		
				8 --		Sandy Silt, dark gray, moist, stiff
				-		
				9 --		
				-		
10	6	27.1	SPT	10 --		
				-		Silty Clay, dark gray, moist, medium firm to stiff
				11 --		
				-		
				12 --		
12.5	13	30.7	93.1	-		
				13 --	CH	Silty Clay, dark gray, very moist, stiff
				-		
				14 --		
				-		
15	6	40.2	SPT	15 --		
				-		Silty Clay, dark gray, very moist, soft to medium firm
				16 --		
				-		
				17 --		
17.5	9	29.1	93.3	-		
				18 --		
				-		
				19 --		
				-		
20	5	32.3	SPT	20 --		
				-		
				21 --		
				-		
				22 --		
22.5	12	30.7	87.4	-		
				23 --	CL	Silty Clay, dark gray, moist, medium firm
				-		
				24 --		
				-		
25	8	29.8	SPT	25 --		
				-		

BORING LOG NUMBER 1

Chait Company Architects

File No. 21194

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				-		
				26 --		
				-		
				27 --		
27.5	12	28.9	95.8	-		
				28 --	ML	Sandy Silt, dark gray, very moist, stiff
				-		
				29 --		
				-		
30	8	31.3	SPT	30 --		
				-		
				31 --		
				-		
				32 --		
32.5	26	23.4	103.0	-		
				33 --	SC	Clayey Sand, dark gray, wet, medium dense, fine grained
				-		
				34 --		
				-		
35	14	23.7	SPT	35 --		
				-		
				36 --		
				-		
				37 --		
37.5	53	16.9	112.6	-		
				38 --	SP/SW	Sand to Gravelly Sand, gray, wet, dense, fine to coarse grained
				-		
				39 --		
				-		
40	35	16.9	SPT	40 --		
				-		
				41 --		
				-		
				42 --		
42.5	43	12.9	112.1	-		
				43 --		
				-		
				44 --		
				-		
45	24	15.1	SPT	45 --		
				-	SP	Sand, dark gray, wet, medium dense, fine to medium grained, occasional gravel
				46 --		
				-		
				47 --		
47.5	22	25.2	96.2	-		
				48 --	SC/ML	Clayey Sand to Sandy Silt, dark gray, wet, medium dense to medium firm, fine grained
				-		
				49 --		
				-		
50	19	27.1	SPT	50 --		
				-		

BORING LOG NUMBER 1

Chait Company Architects

File No. 21194

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				-		
				51 --		
				-		
				52 --		
52.5	19	22.6	99.6	-		
				53 --	ML	Sandy Silt, dark gray, wet, medium firm, fine grained
				-		
				54 --		
				-		
55	11	34.2	SPT	55 --		
				-	SC	Clayey Sand, dark gray, wet, medium dense, fine grained
				56 --		
				-		
				57 --		
57.5	44	22.2	101.7	-		
				58 --	SP	Sand, dark gray, wet, dense, fine grained
				-		
				59 --		
				-		
60	73	15.5	SPT	60 --		
				-		
				61 --		
				-		
				62 --		
62.5	84	7.2	129.1	-		
				63 --	SW	Gravelly Sand, gray, wet, very dense, fine to coarse grained
				-		
				64 --		
				-		
65	91	9.0	SPT	65 --		
				-	SP	Sand, dark gray, wet, very dense, fine to medium grained,
				66 --		occasional gravel
				-		
				67 --		
67.5	39	11.1	120.1	-		
	50/4"			68 --		
				-		
				69 --		
				-		
70	80	19.6	SPT	70 --		
				-		Sand, dark gray, wet, very dense, fine grained
				71 --		
				-		
				72 --		
72.5	41	17.9	108.4	-		
	50/3"			73 --		
				-		
				74 --		
				-		
75	83	17.3	SPT	75 --		
				-		

BORING LOG NUMBER 1

Chait Company Architects

File No. 21194

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				-		
				76 --		
				-		
				77 --		
77.5	42	20.7	108.4	-		
	50/3"			78 --		Sand, gray, wet, very dense, fine grained
				-		
				79 --		
				-		
80	84	15.6	SPT	80 --		
				-		
				81 --		
				-		
				82 --		
82.5	40	17.7	112.7	-		
	50/3"			83 --		Sand, dark gray, wet, very dense, fine to medium grained
				-		
				84 --		
				-		
85	65	10.6	SPT	85 --		
				-		Sand, gray, wet, dense, fine grained
				86 --		
				-		
				87 --		
87.5	35	16.9	112.9	-		
	50/4"			88 --		Sand, gray to dark gray, wet, very dense, fine to medium grained
				-		
				89 --		
				-		
90	81	17.0	SPT	90 --		
				-		
				91 --		
				-		
				92 --		
92.5	39	16.0	113.9	-		
	50/3"			93 --		Sand, gray, wet, very dense, fine grained
				-		
				94 --		
				-		
95	71	18.9	SPT	95 --		
				-		
				96 --		
				-		
				97 --		
97.5	30	16.5	106.1	-		
	50/5"			98 --		
				-		
				99 --		
				-		
100	62	16.0	SPT	100 --		
				-		

BORING LOG NUMBER 1

Chait Company Architects

File No. 21194

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				-		
				101 --		
				-		
				102 --		
102.5	29	16.1	114.8	-		
	50/5"			103 --		Sand, gray, wet, very dense, fine grained
				-		
				104 --		
				-		
105	79	19.8	SPT	105 --		
				-		
				106 --		
				-		
				107 --		
107.5	40	20.5	106.3	-		
	50/3"			108 --		
				-		
				109 --		
				-		
110	34	14.6	SPT	110 --		
	50/5"			-		Sand, gray, wet, very dense, fine grained
				111 --		
				-		
				112 --		
112.5	100/9"	13.0	121.0	-		
				113 --		Sand, gray, wet, very dense, fine to medium grained
				-		
				114 --		
				-		
115	43	15.0	SPT	115 --		
	50/5.5"			-		
				116 --		
				-		
				117 --		
117.5	100/10"	14.0	121.2	-		
				118 --		
				-		
				119 --		
				-		
120	90	23.8	SPT	120 --		
				-		Total Depth 120 feet
				121 --		Water at 22½ feet
				-		Fill to 12½ feet
				122 --		
				-		
				123 --		NOTE: The stratification lines represent the approximate
				-		boundary between earth types; the transition may be gradual.
				124 --		
				-		Used 5-inch diameter Rotary Wash Drill Rig
				125 --		
				-		

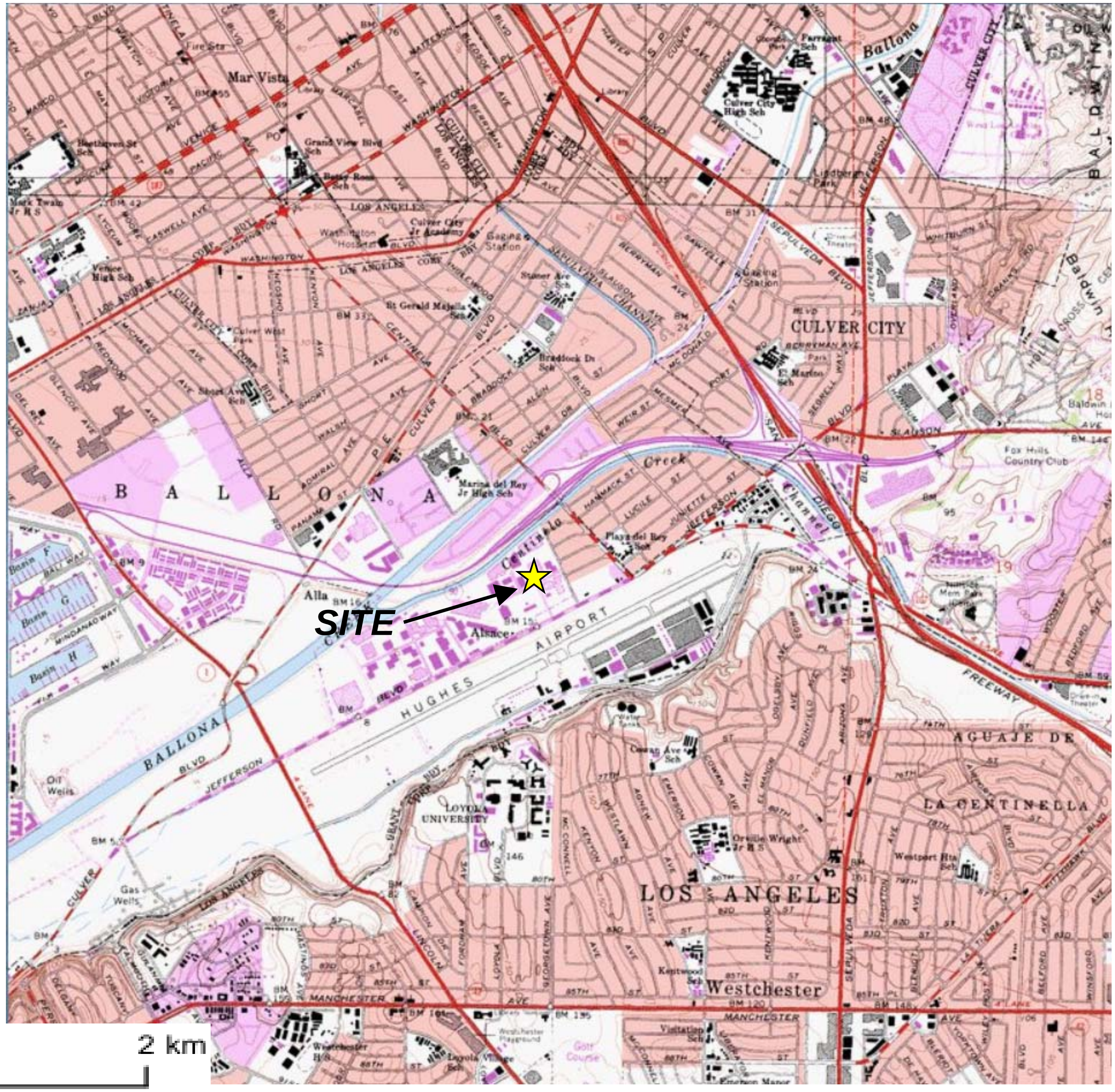
TABLE 5
FINAL SURFACE SITE-SPECIFIC SPECTRA DEVELOPMENT CALCULATION SHEET
12575 BEATRICE PLAYA VISTA GROUND-MOTION DEVELOPMENT PROJECT

Column 1	Column 2	Column 3	Column 4	Column 5	Column 6	**Column 7**	Column 8	Column 9	Column 10	**Column 11**
		Final Outcropping Site-Specific MCE $_R$	*Site Response Amplification Factors*	*Surface MCE $_R$*	*Code Minimum MCE $_R$ for Site Class E*	***Final Site-Specific Surface MCE $_R$***	*Code-Based DRS for Site Class E*	*80% of Code-Based DRS*	*2/3 of Final Site-Specific Surface MCE $_R$*	***Final Site-Specific Surface DRS***
Period	***Frequency***	*RotD100*		*RotD100*	*RotD100*	***RotD100***	*RotD100*	*RotD100*	*RotD100*	***RotD100***
(sec)	(Hz)	(g)	-	-	(g)	**(g)**	(g)	(g)	(g)	**(g)**
0.010	100	0.941	1.126	1.060	0.647	**1.060**	0.539	0.431	0.706	**0.706**
0.020	50	0.947	1.114	1.055	0.724	**1.055**	0.604	0.483	0.703	**0.703**
0.030	33	0.993	1.070	1.063	0.802	**1.063**	0.669	0.535	0.708	**0.708**
0.050	20	1.180	1.035	1.221	0.958	**1.221**	0.798	0.639	0.814	**0.814**
0.075	13	1.499	0.872	1.308	1.152	**1.308**	0.960	0.768	0.872	**0.872**
0.100	10	1.761	0.808	1.423	1.346	**1.423**	1.122	0.898	0.949	**0.949**
0.150	6.67	2.088	0.725	1.514	1.422	**1.514**	1.185	0.948	1.009	**1.009**
0.200	5.00	2.275	0.829	1.886	1.422	**1.886**	1.185	0.948	1.258	**1.258**
0.250	4.00	2.370	0.963	2.283	1.422	**2.283**	1.185	0.948	1.522	**1.522**
0.300	3.33	2.387	1.089	2.599	1.422	**2.599**	1.185	0.948	1.733	**1.733**
0.400	2.50	2.271	1.320	2.999	1.422	**2.999**	1.185	0.948	2.000	**2.000**
0.500	2.00	2.076	1.616	3.356	1.422	**3.356**	1.185	0.948	2.237	**2.237**
0.750	1.33	1.611	1.552	2.500	1.042	**2.500**	0.868	0.694	1.667	**1.667**
1.000	1.00	1.275	1.416	1.805	0.781	**1.805**	0.651	0.521	1.203	**1.203**
1.500	0.67	0.834	1.183	0.987	0.521	**0.987**	0.434	0.347	0.658	**0.658**
2.000	0.50	0.599	1.125	0.674	0.391	**0.674**	0.326	0.260	0.449	**0.449**
3.000	0.33	0.369	1.053	0.389	0.260	**0.389**	0.217	0.174	0.259	**0.259**
4.000	0.25	0.253	1.028	0.260	0.195	**0.260**	0.163	0.130	0.173	**0.173**
5.000	0.20	0.188	1.026	0.192	0.156	**0.192**	0.130	0.104	0.128	**0.128**
7.500	0.13	0.109	1.022	0.111	0.104	**0.111**	0.087	0.069	0.074	**0.074**
10.000	0.10	0.071	1.027	0.072	0.062	**0.072**	0.052	0.042	0.048	**0.048**

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	= Final risk-targeted, maximum considered earthquake (MCER) outcropping ground motion spectral ordinates in units of g for 5% damping; repeated from Table 3, Column 12.
Column 4	= Amplification factors between outcropping and ground surface using site-specific velocity profile and material properties.
Column 5	= Ground surface ground motion spectral ordinates in units of g for 5% damping; product of Columns 3 and 4.
Column 6	= 80% of code-based (ASCE 7-10, Ch. 11) risk-targeted, maximum considered earthquake ground motion spectral ordinates in units of g for 5% damping.
Column 7	= Final risk-targeted, maximum considered earthquake ground surface ground motion spectral ordinates in units of g for 5% damping; maximum value from Columns 5 and 6.
Column 8	= Code-based (ASCE 7-10, Ch. 11) design ground motion spectral ordinates in units of g for 5% damping.
Column 9	= Code-based (ASCE 7-10, Ch. 21) minimum design ground motion spectral ordinates in units of g for 5% damping; 80% of the value in Column 8.
Column 10	= Minimum Design Earthquake (DE) ground motion spectral ordinates in units of g for 5% damping; 2/3 of the final site-specific ground surface MCE_R in Column 7.
Column 11	= Final design ground surface ground motion spectral ordinates in units of g for 5% damping; maximum value from Columns 9 and 10.

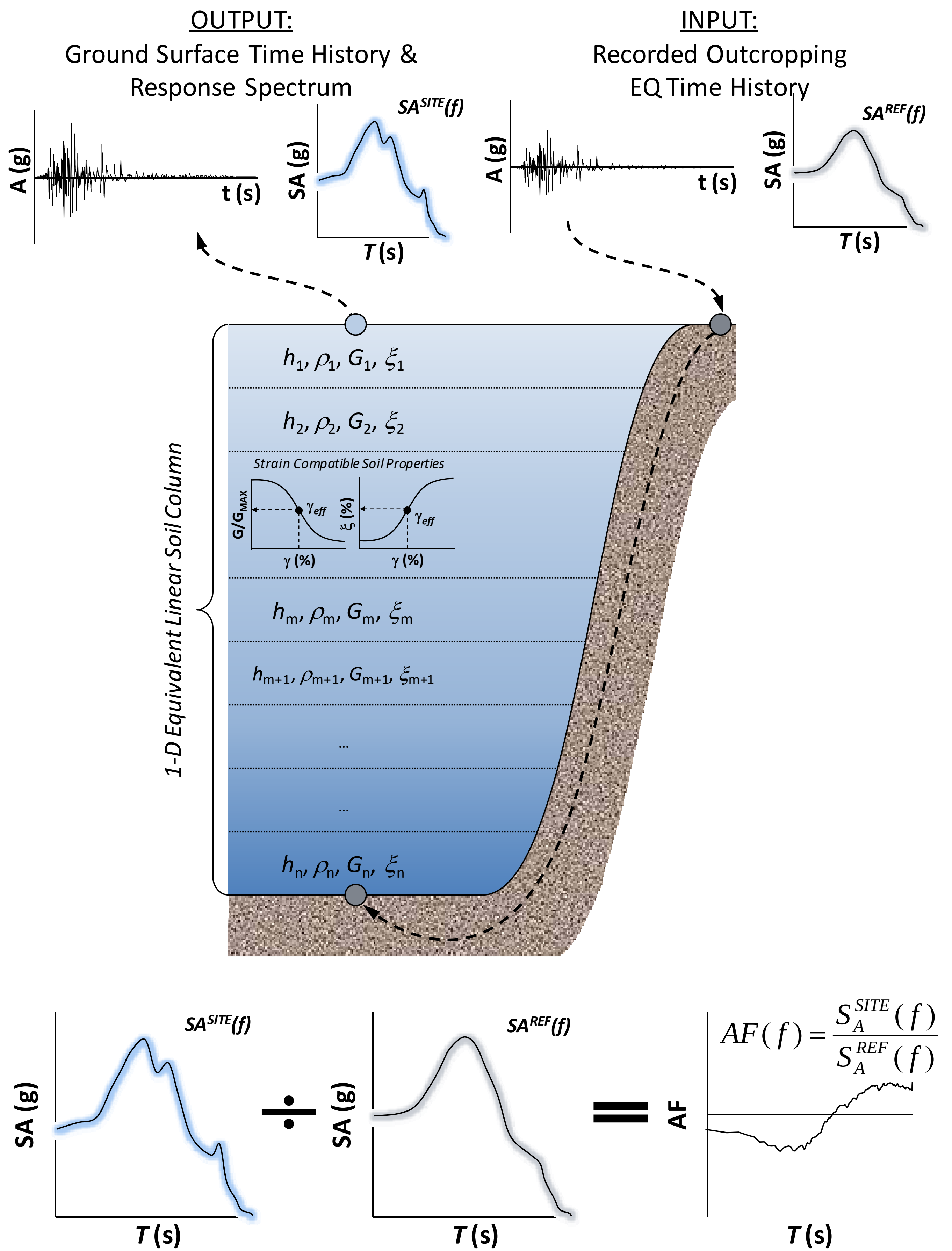


USGS Topographic Map: Venice, CA

DETAILED SITE LOCATION MAP

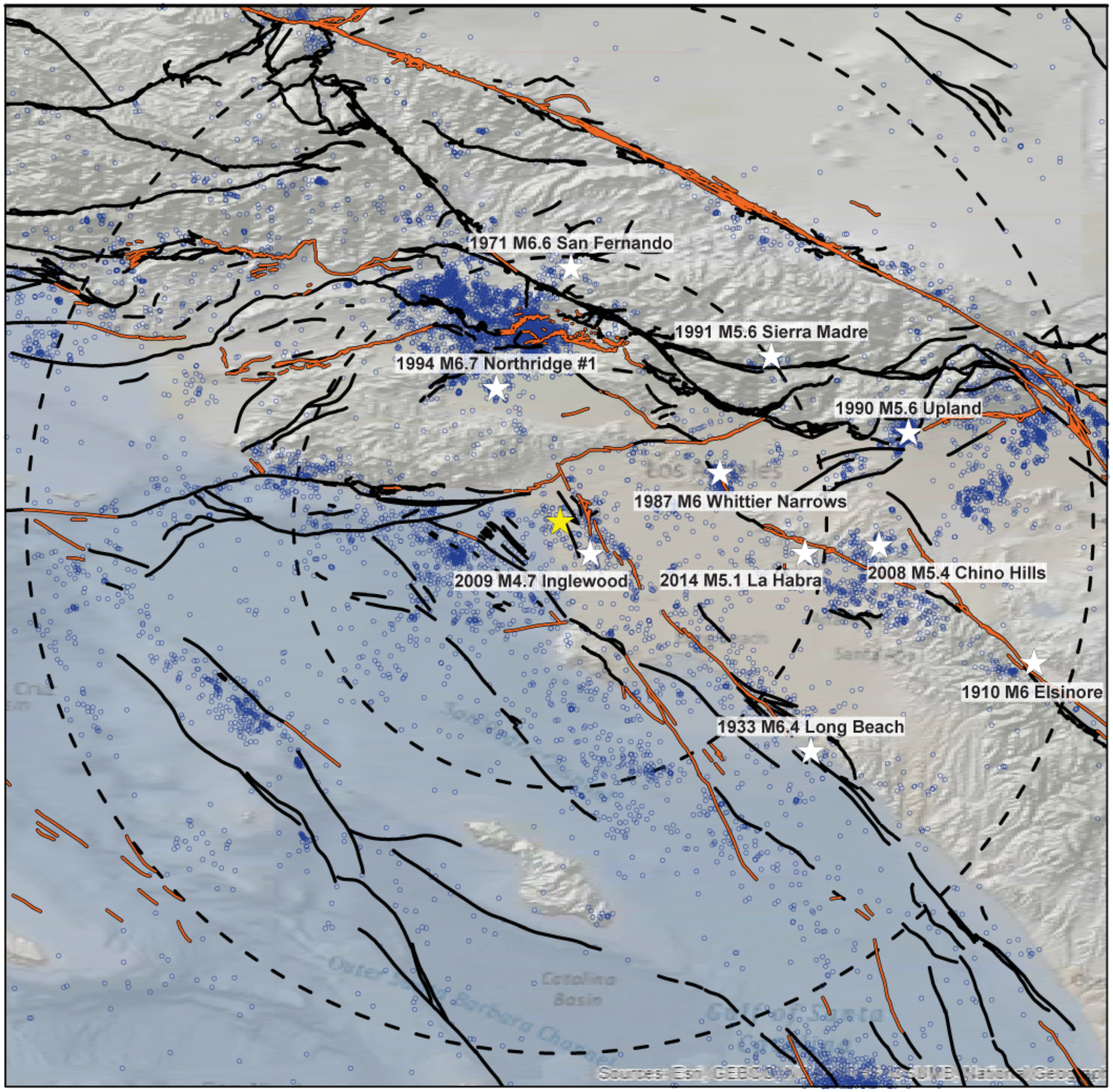
Date: FEB. 2018	Project No.: 17025A	Project: 12575 BEATRICE PLAYA VISTA GROUND-MOTION DEVELOPMENT	Figure 1

GeoPentech



SITE RESPONSE ANALYSIS APPROACH			
Project No.: 17025A	Project: 12575 BEATRICE GROUND MOTIONS	Date: FEB. 2018	Figure 2

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

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

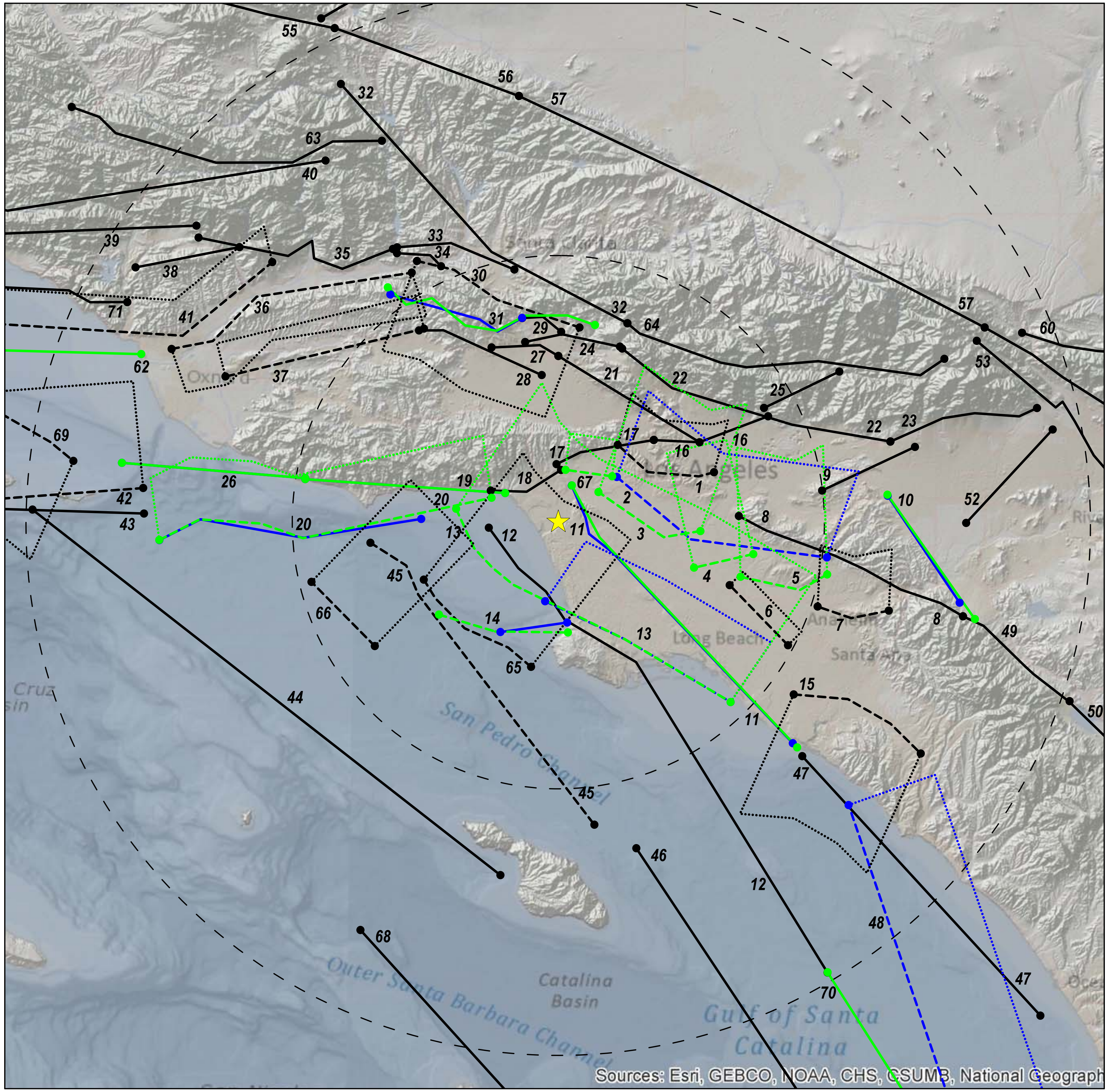
N

Datum & Projection: NAD83 UTM Zone 11

REGIONAL FAULT & SEISMICITY MAP

Date: FEB. 2018	Project No.: 17025A	Project: 12575 BEATRICE PLAYA VISTA GROUND-MOTION DEVELOPMENT	Figure 3a

GeoPentech



Approximate Scale (km)

0 5 10 15 20 25 30 35 40 45 50

N

Datum & Projection: NAD83 UTM Zone 11

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)

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

Legend

Surface Trace, Top of Fault (Both Fault Models)

Blind Trace, Top of Fault (Both Fault Models)

Blind Thrust Footprint (Both Fault Models)

Surface Trace, Top of Fault (Fault Model 1)

Blind Trace, Top of Fault (Fault Model 1)

Blind Thrust Footprint (Fault Model 1)

Surface Trace, Top of Fault (Fault Model 2)

Blind Trace, Top of Fault (Fault Model 2)

Blind Thrust Footprint (Fault Model 2)

Site

Notes:

1. 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, except for the following: Elysian Park (Lower), North Salt Lake, Richfield, and Yorba Linda (however, these faults are included in the PSHA). Slip rates are solution mean rates from UCERF3 (WGCEP, 2013a). 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 (2013a). Magnitude-frequency distributions approximate the SWUS WAACY model (GeoPentech, 2015) with characteristic magnitude calculated from Shaw (2009) regression. Type A faults characterized as documented in WGCEP (2008) and 2008 NSHM (Petersen et al., 2008).

SIMPLIFIED FAULT MAP FOR PSHA

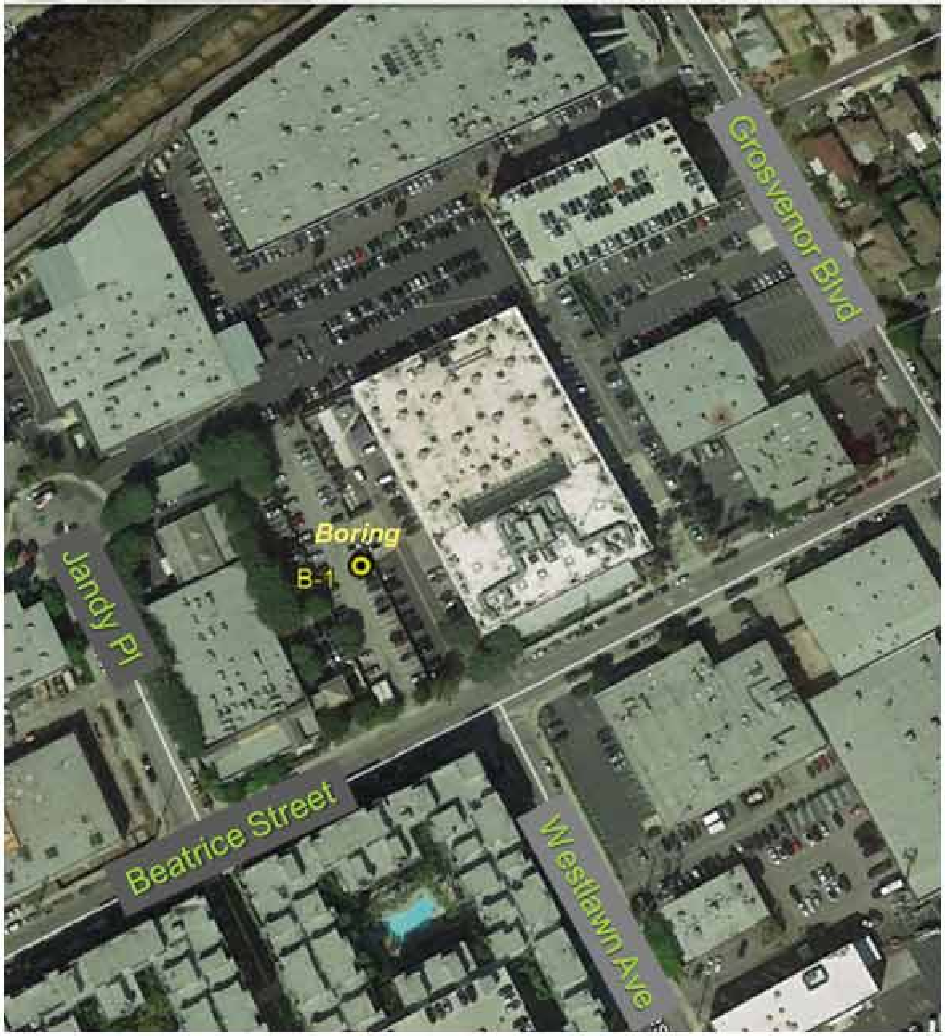
Project: 12575 BEATRICE GROUND MOTIONS

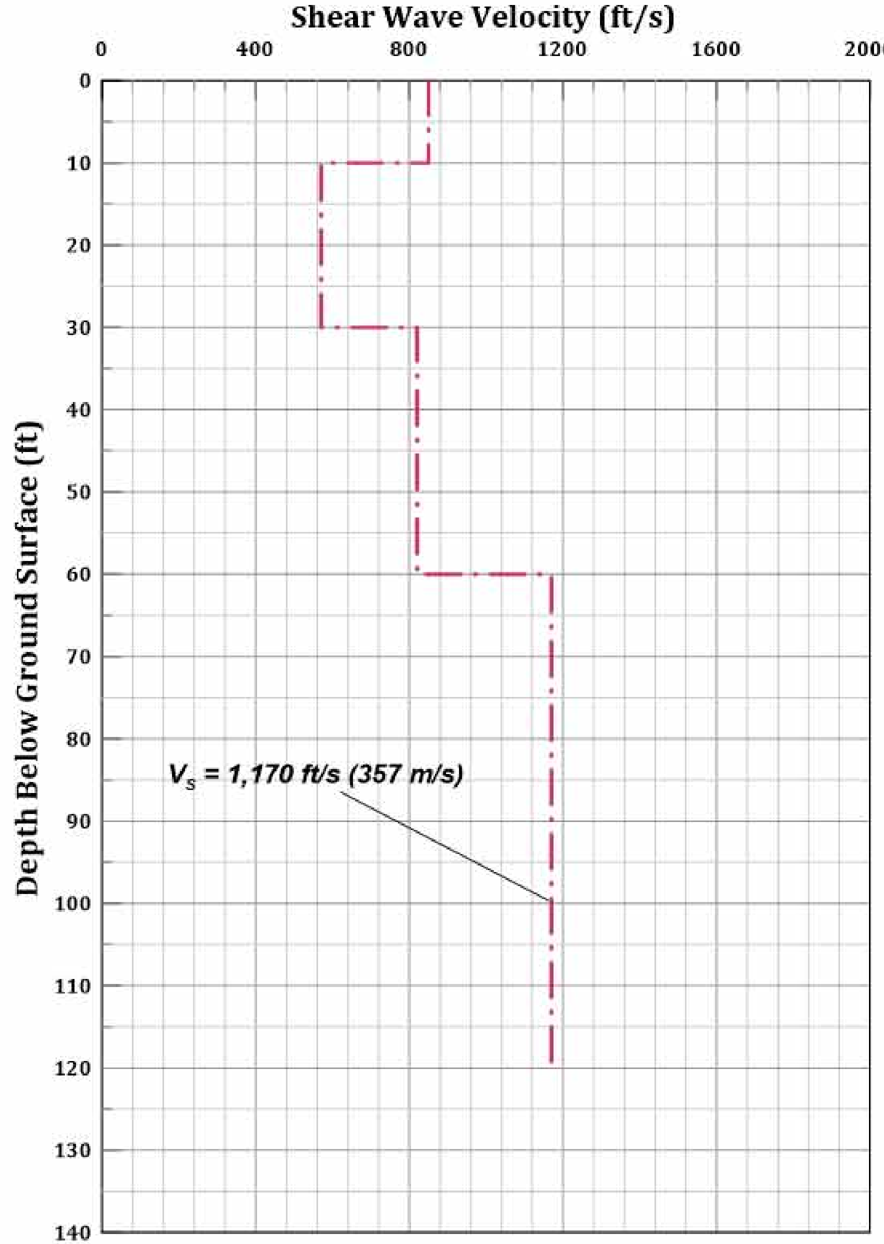
Project No.: 17025A

Date: FEB. 2018

Figure 3b

GeoPentech

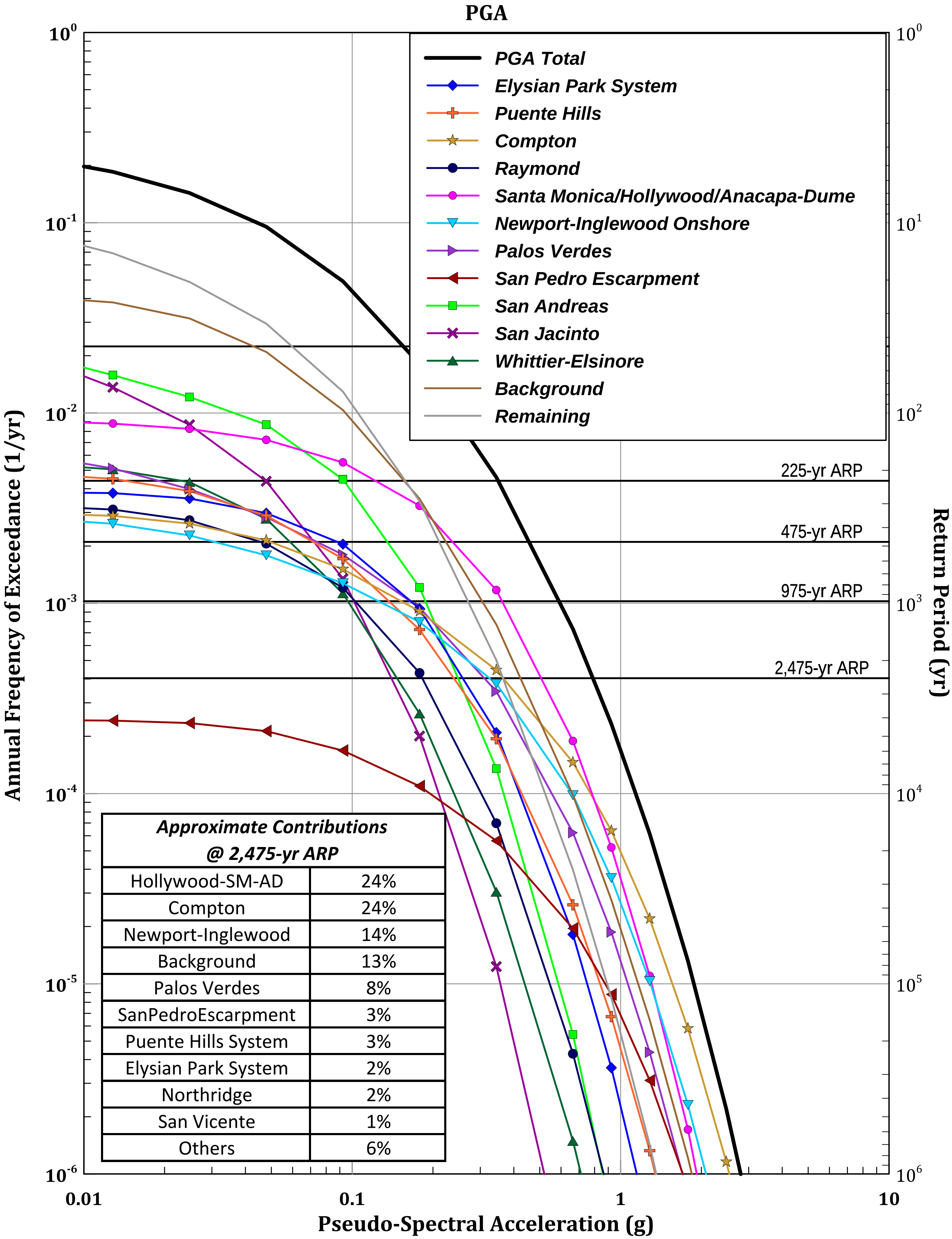




SITE SHEAR-WAVE VELOCITY

Date: FEB. 2016	Project No.: 17025A	Project: 12575 BEATRICE PLAYA VISTA GROUND-MOTION DEVELOPMENT	Figure 4

GeoPentech

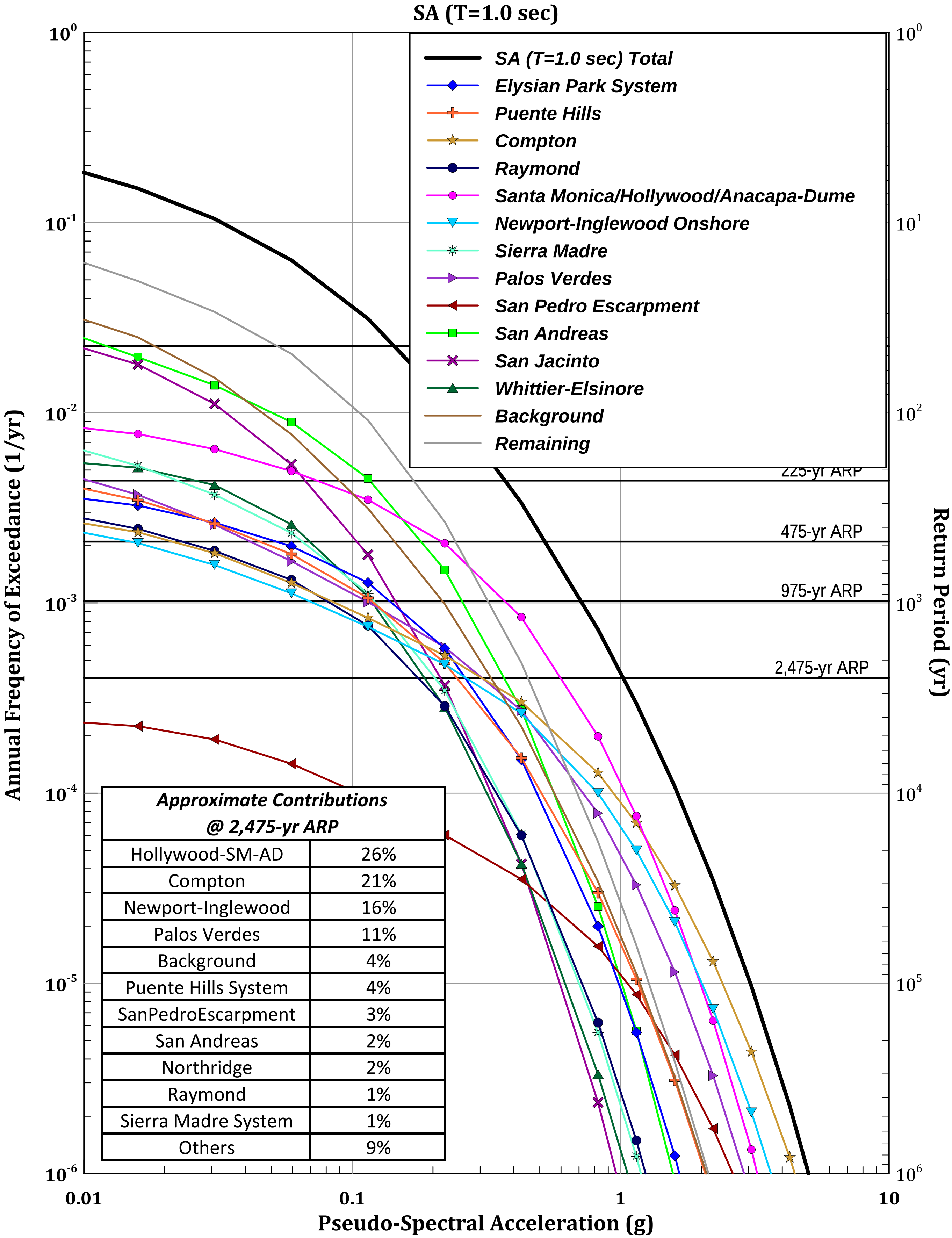


Approximate Contributions @ 2,475-yr ARP	
Hollywood-SM-AD	24%
Compton	24%
Newport-Inglewood	14%
Background	13%
Palos Verdes	8%
SanPedroEscarpment	3%
Puente Hills System	3%
Elysian Park System	2%
Northridge	2%
San Vicente	1%
Others	6%

SOURCE CONTRIBUTIONS TO THE TOTAL HAZARD AT PGA

Project No.: 17025A	Project: 12575 BEATRICE GROUND MOTIONS	Date: FEB. 2018	Figure 5a

GeoPentech

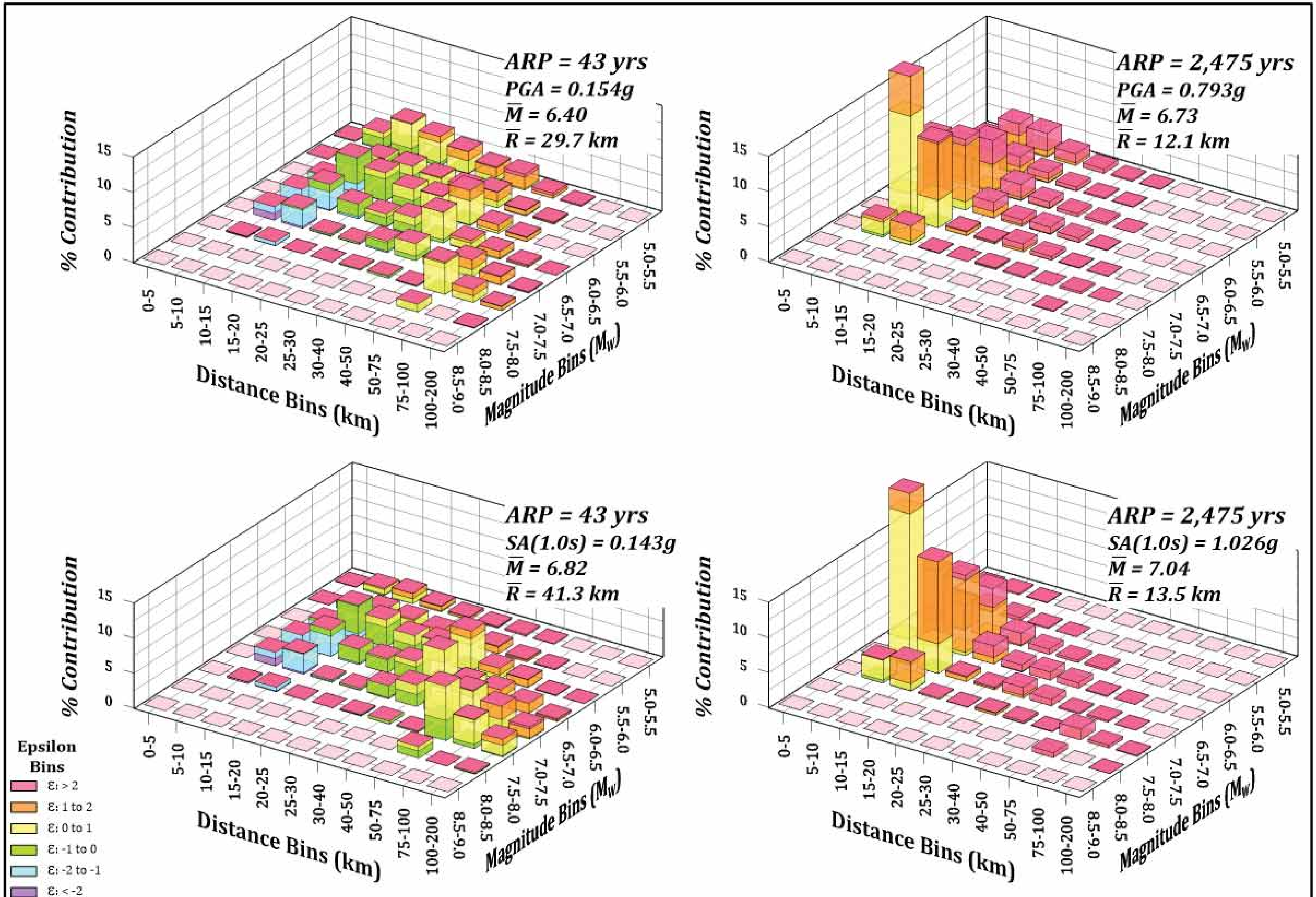


Approximate Contributions @ 2,475-yr ARP	
Hollywood-SM-AD	26%
Compton	21%
Newport-Inglewood	16%
Palos Verdes	11%
Background	4%
Puente Hills System	4%
SanPedroEscarpment	3%
San Andreas	2%
Northridge	2%
Raymond	1%
Sierra Madre System	1%
Others	9%

SOURCE CONTRIBUTIONS TO THE TOTAL HAZARD AT T=1.0 SEC

Project No.: 17025A	Project: 12575 BEATRICE GROUND MOTIONS	Date: FEB. 2018	Figure 5b

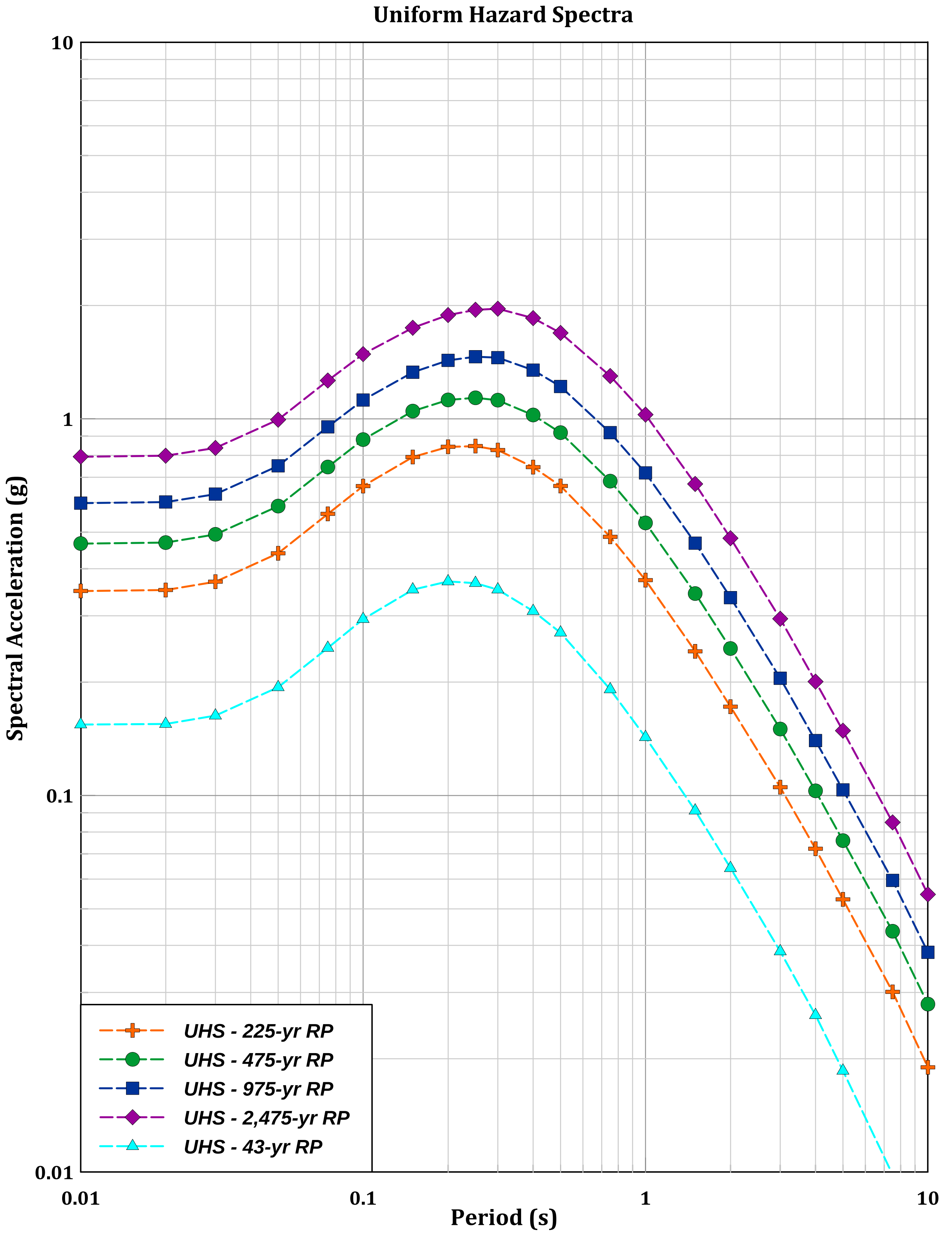
GeoPentech



HAZARD DEAGGREGATION FOR PGA & 1.0-SECOND SPECTRAL PERIOD

Project No.: 17025A	Date: FEB. 2018	Project: 12575 BEATRICE PLAYA VISTA GROUND-MOTION DEVELOPMENT	Figure 6

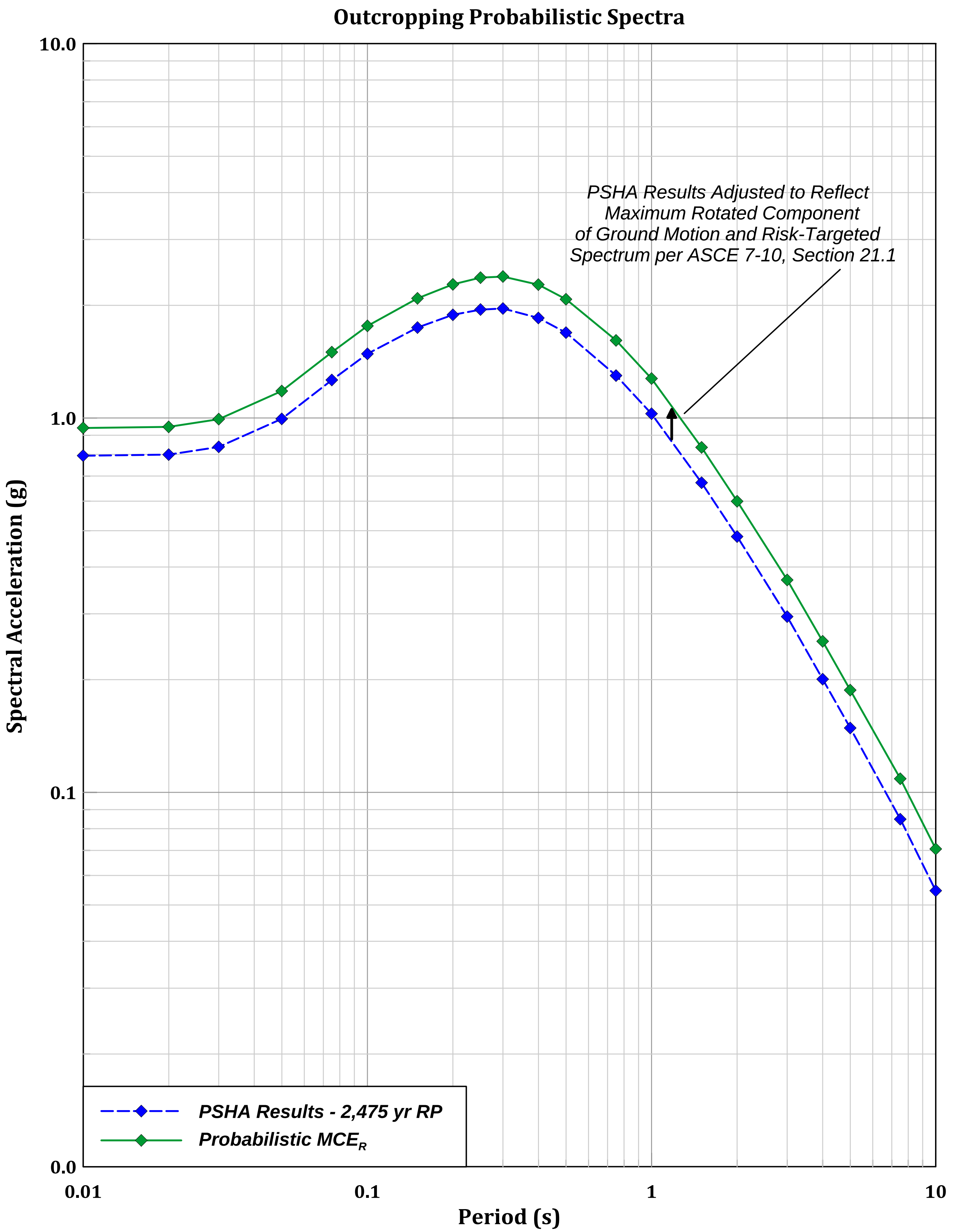
GeoPentech



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

UNIFORM HAZARD SPECTRA			
Project No.: 17025A	Project: 12575 BEATRICE GROUND MOTIONS	Date: FEB. 2018	Figure 7

GeoPentech

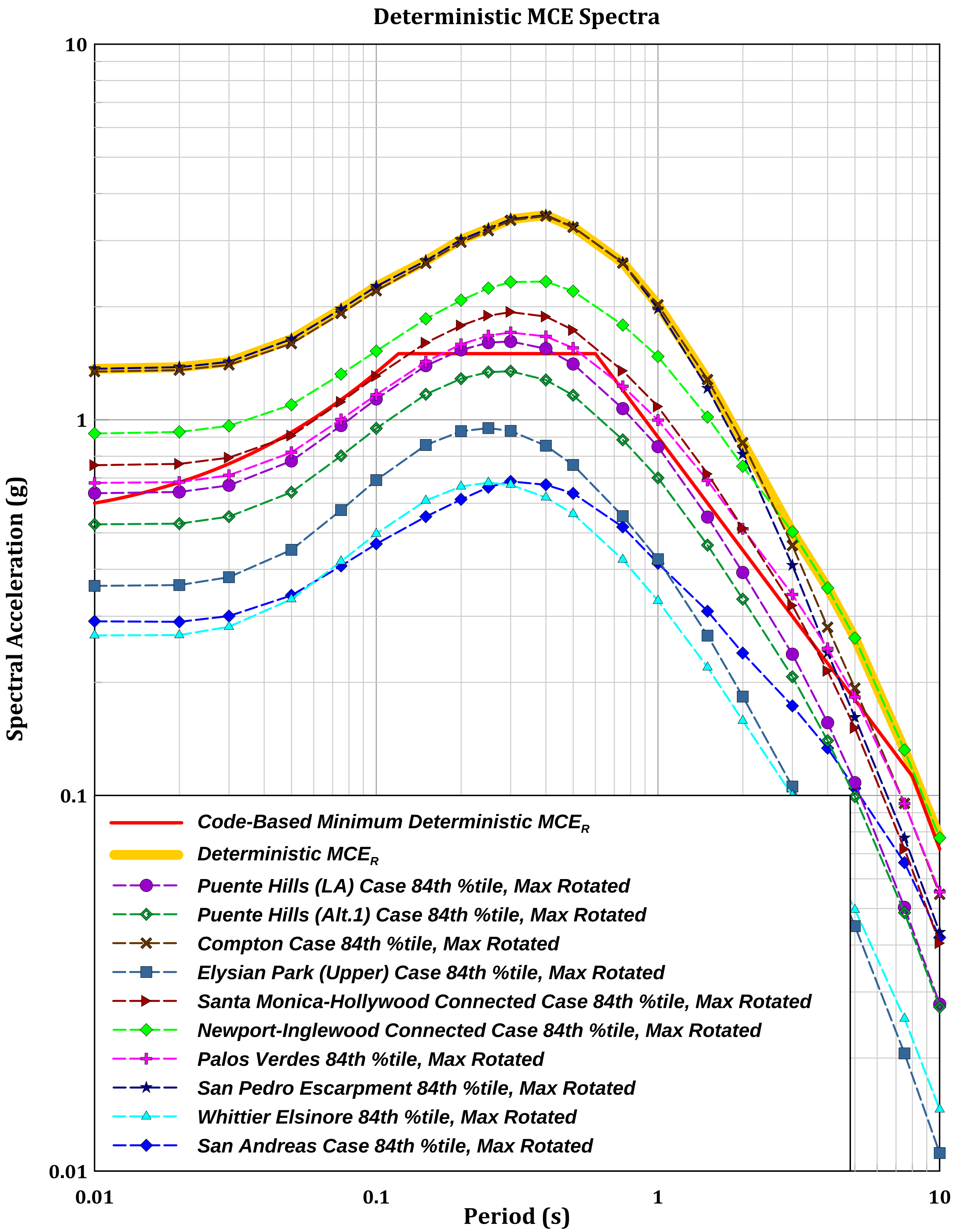


Note: All spectra are for Damping (β) = 5.0% unless otherwise indicated.

PROBABILISTIC SPECTRA FOR OUTCROPPING CONDITION (V_{S30} = 357 m/s)

Project No.: 17025A	Project: 12575 BEATRICE GROUND MOTIONS	Date: FEB. 2018	Figure 8

GeoPentech

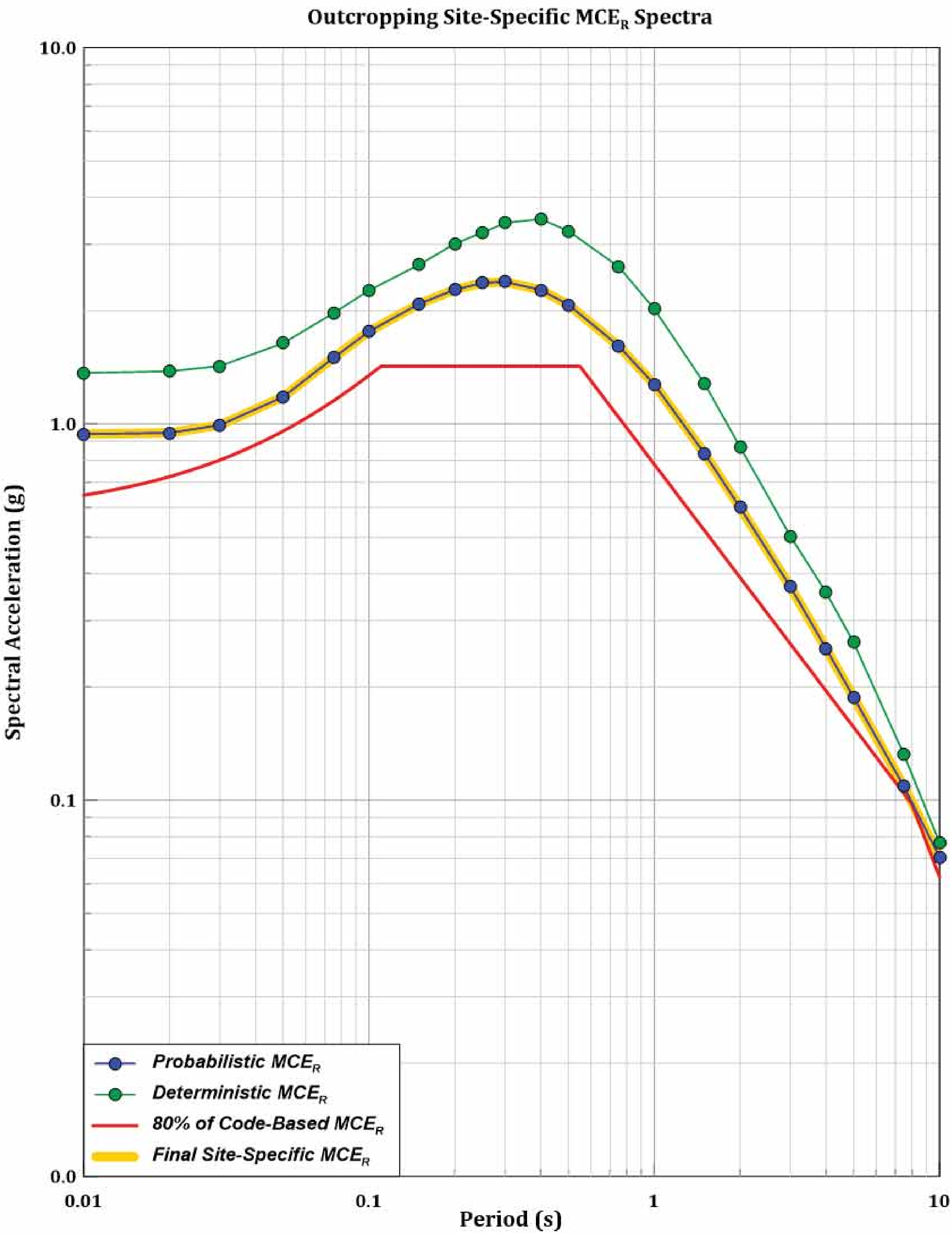


Note: All spectra are for Damping (β) = 5.0%

DETERMINISTIC SPECTRA FOR OUTCROPPING CONDITION (V_{S30} = 357 m/s)

Project No.: 17025A	Project: 12575 BEATRICE GROUND MOTIONS	Date: FEB. 2018	Figure 9

GeoPentech

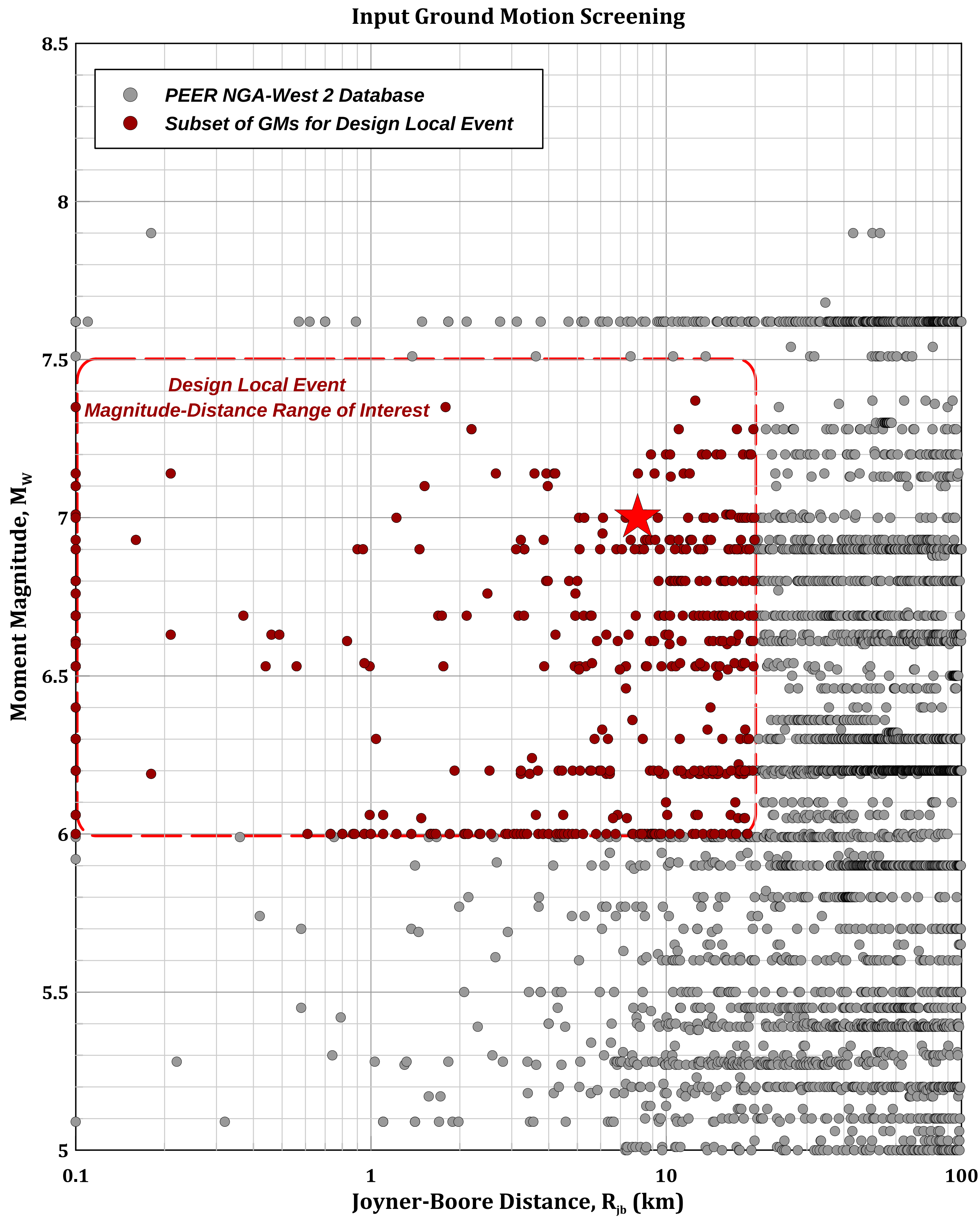


Note: All spectra are for Damping (β) = 5.0%.

DEVELOPMENT OF SITE-SPECIFIC MCE_R FOR OUTCROPPING CONDITION (V_{S30} = 357 m/s)

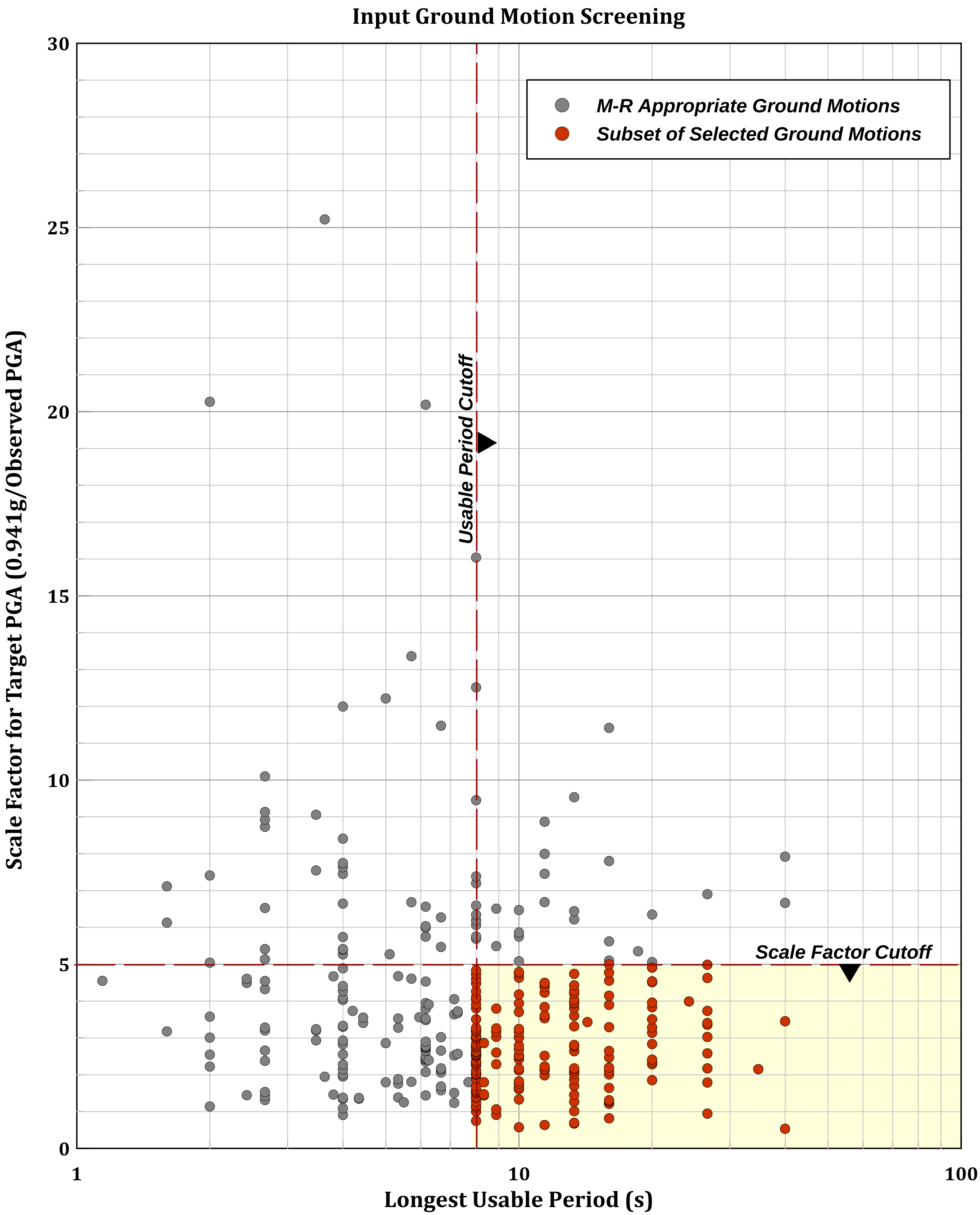
Project No.: 17025A	Project: 12575 BEATRICE GROUND MOTIONS	Date: FEB. 2018	Figure 10

GeoPentech



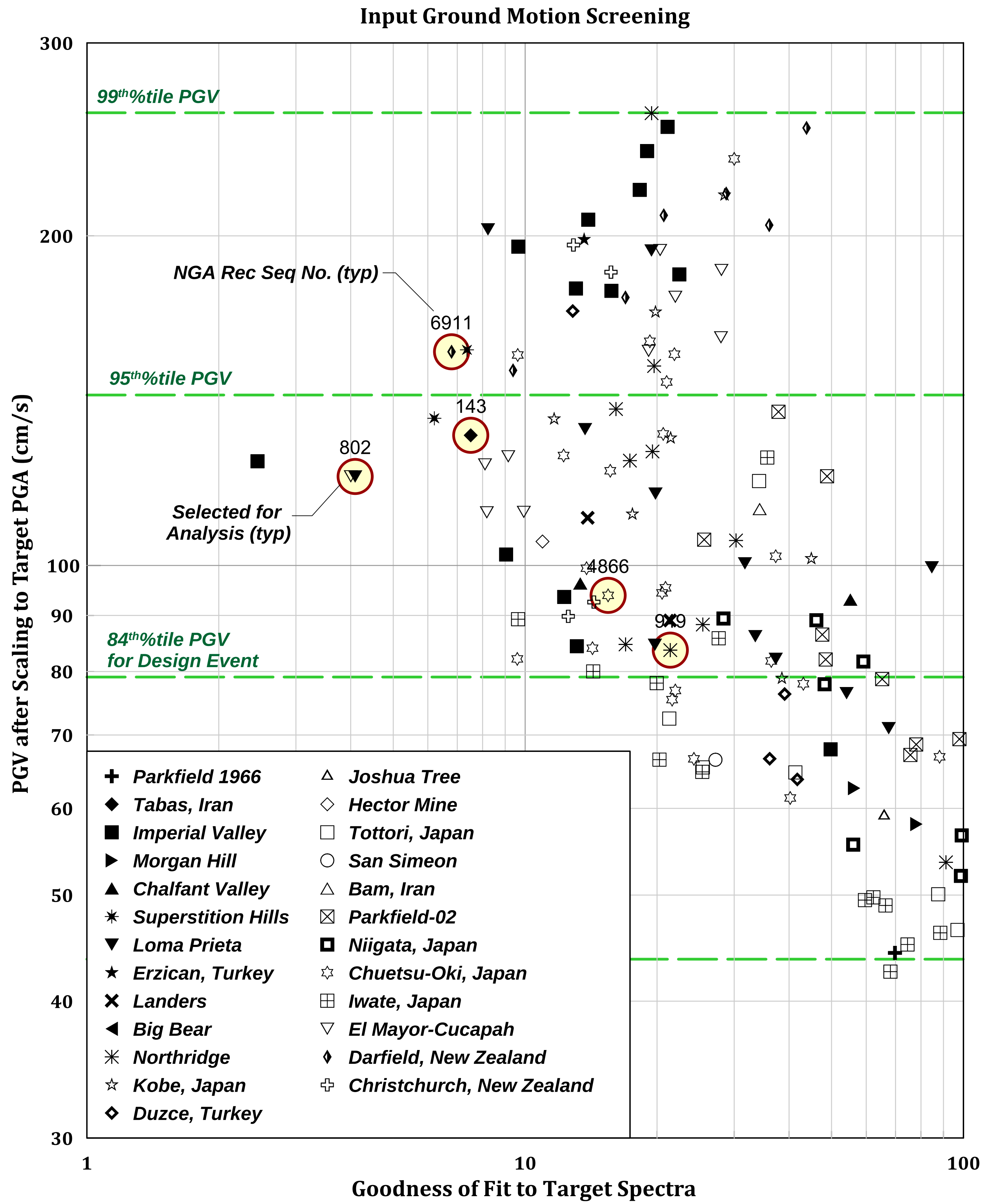
MAGNITUDE AND DISTANCE SCREENING			
Project No.: 17025A	Project: 12575 BEATRICE GROUND MOTIONS	Date: FEB. 2018	Figure 11

GeoPentech



LONGEST USABLE PERIOD AND SCALE FACTOR SCREENING FOR DESIGN LOCAL EVENT			
Project No.: 17025A	Project: 12575 BEATRICE GROUND MOTIONS	Date: FEB. 2018	Figure 12

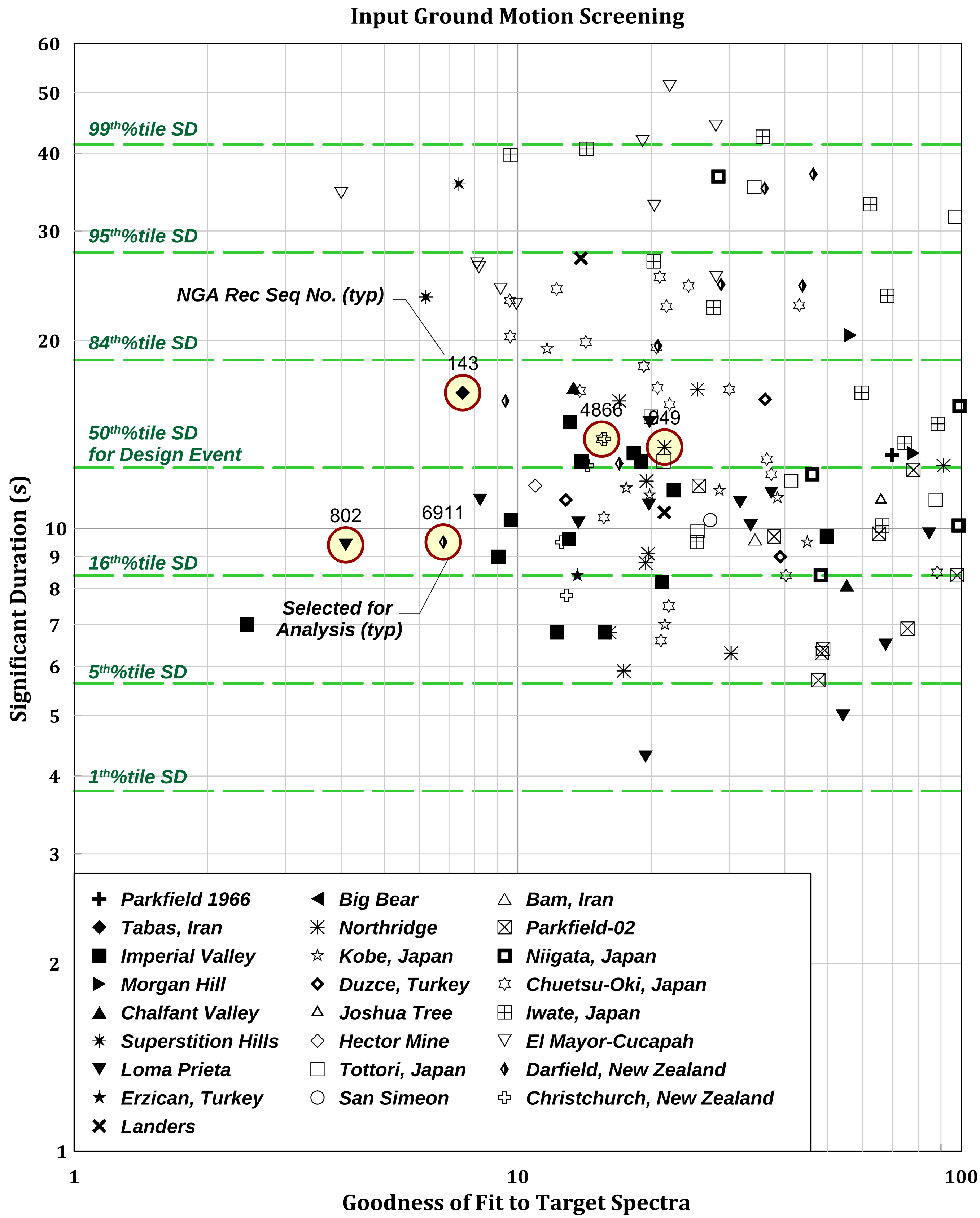
GeoPentech



PGV AND GOODNESS OF FIT SCREENING FOR DESIGN LOCAL EVENT

Project No.: 17025A	Project: 12575 BEATRICE GROUND MOTIONS	Date: FEB. 2018	Figure 13a

GeoPentech

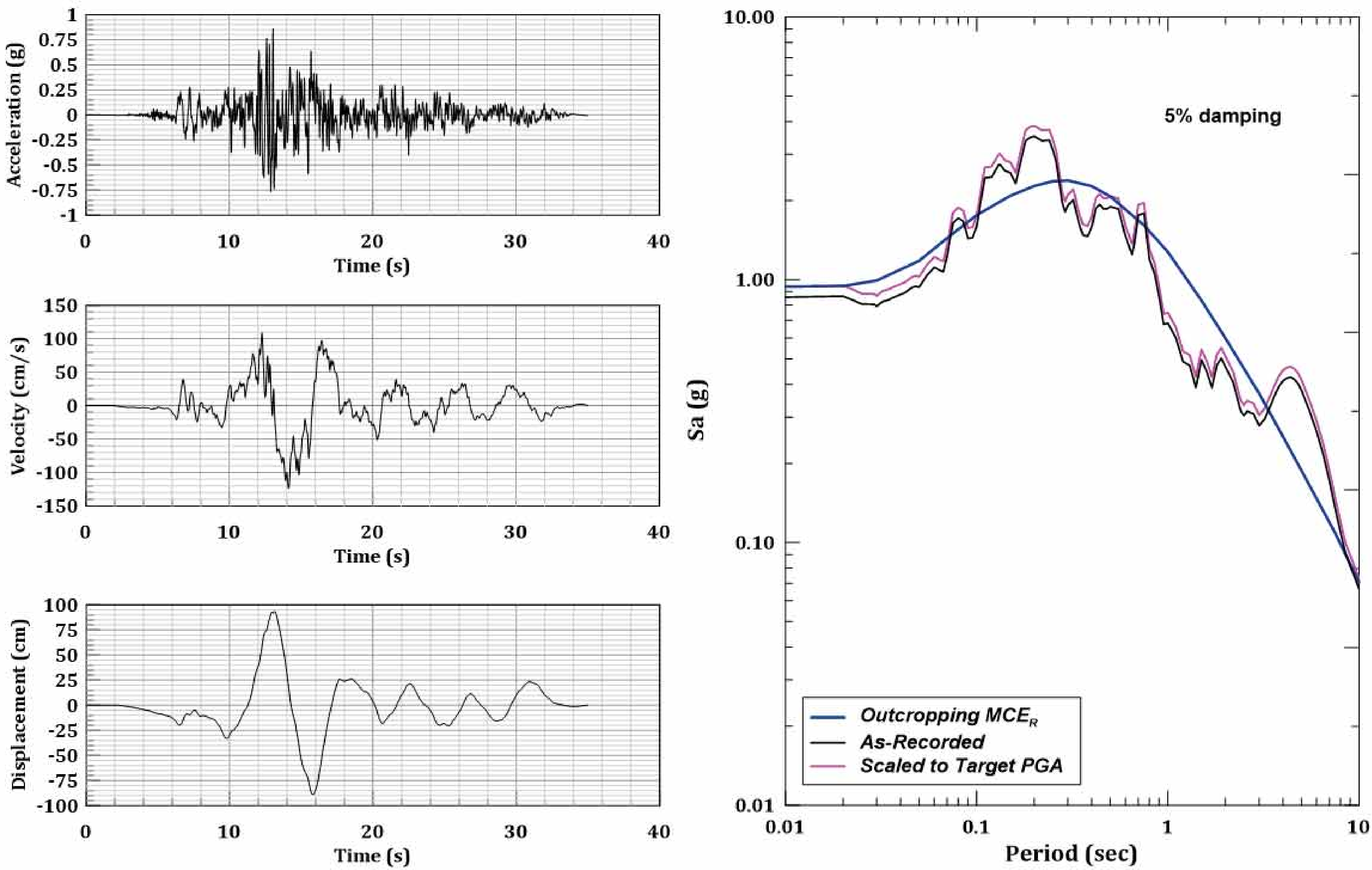


SIGNIFICANT DURATION AND GOODNESS OF FIT SCREENING FOR DESIGN LOCAL EVENT

Project No.: 17025A	Project: 12575 BEATRICE GROUND MOTIONS	Date: FEB. 2018	Figure 13b

GeoPentech

TABAS\TAB-T1.AT2

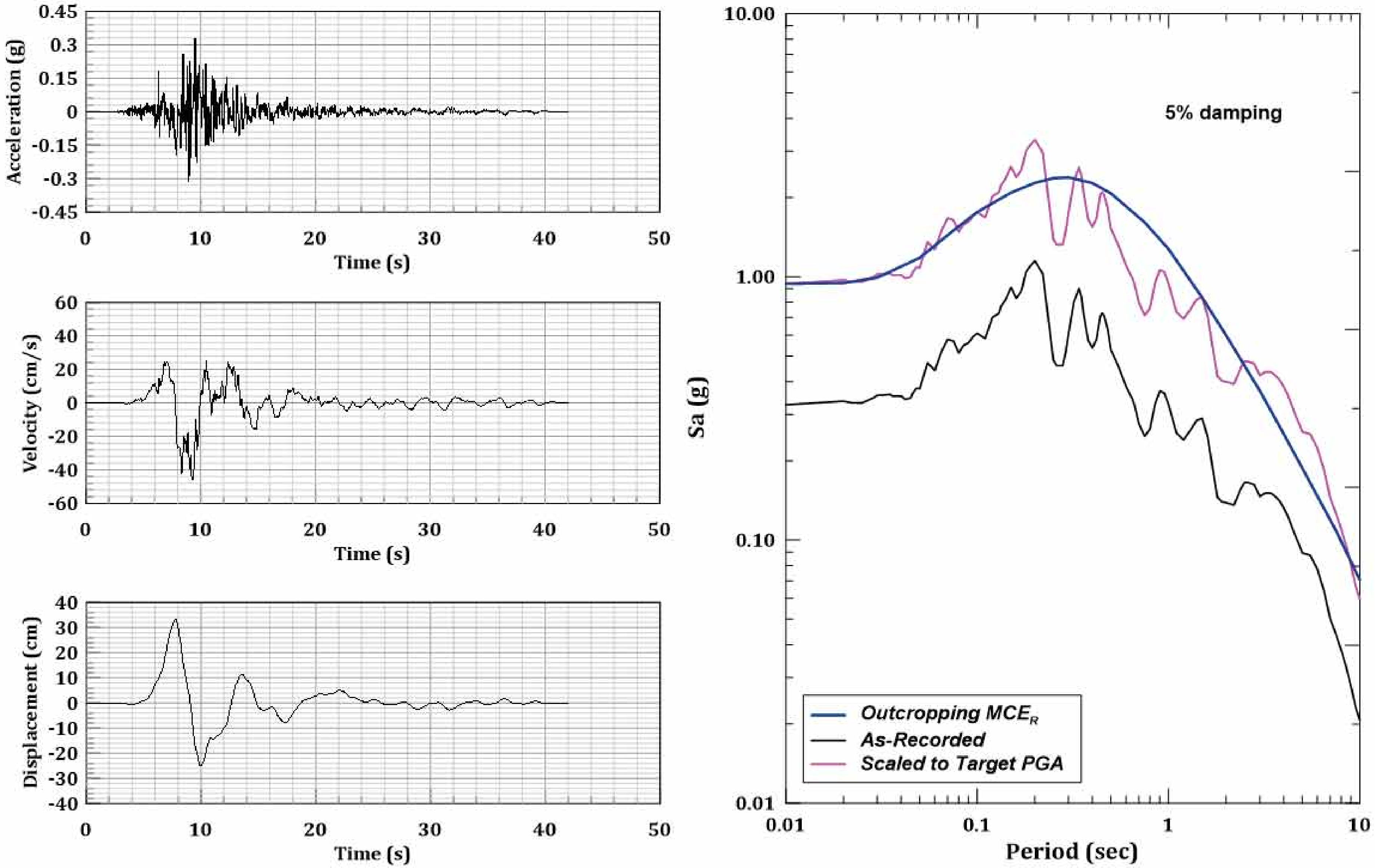


GM1 SEED

Date: FEB. 2018	Project No.: 17025A	Project: 12575 BEATRICE PLAYA VISTA GROUND-MOTION DEVELOPMENT	Figure 14

GeoPentech

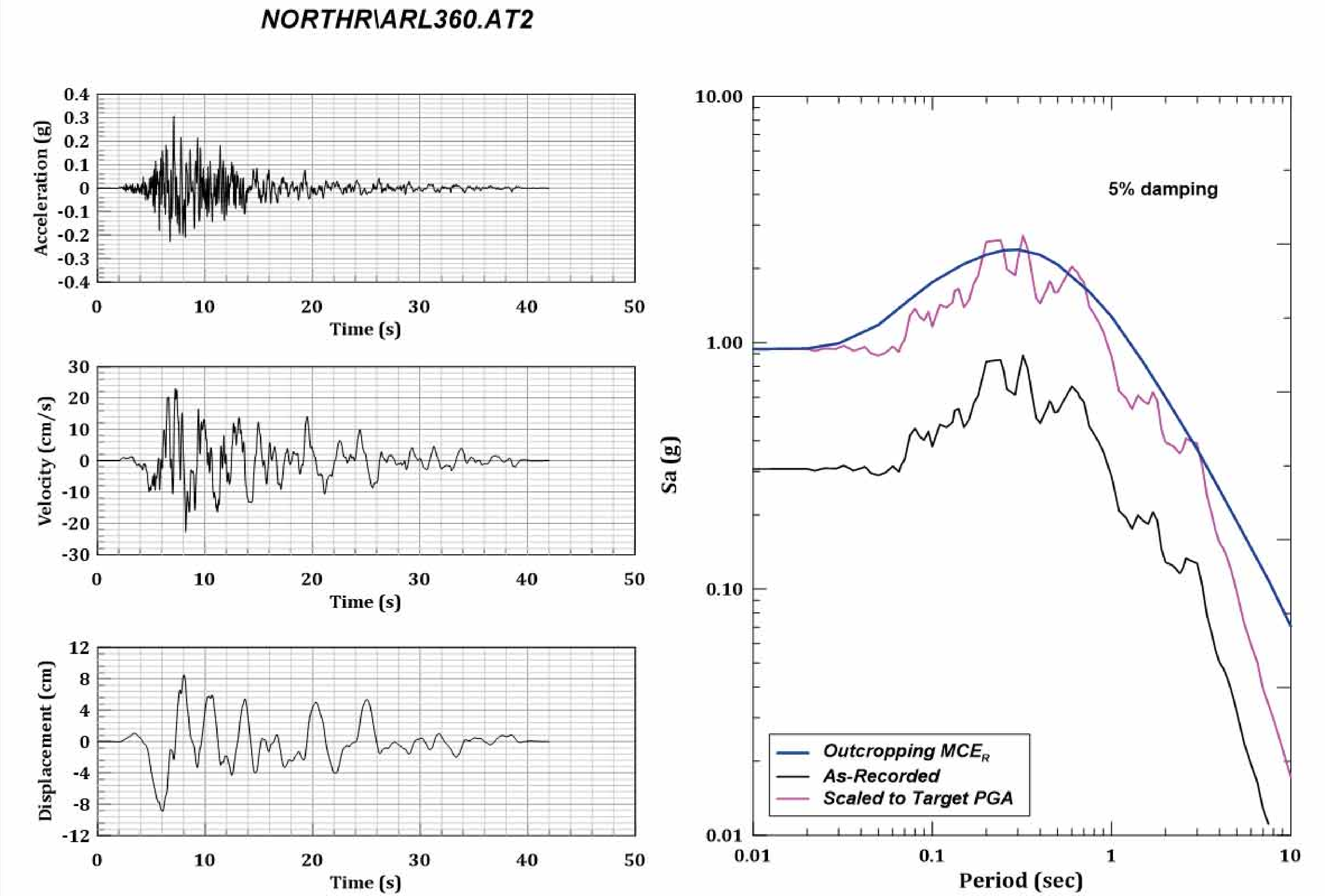
LOMAP\STG090.AT2



GM2 SEED

Date: FEB. 2018	Project No.: 17025A	Project: 12575 BEATRICE PLAYA VISTA GROUND-MOTION DEVELOPMENT	Figure 15

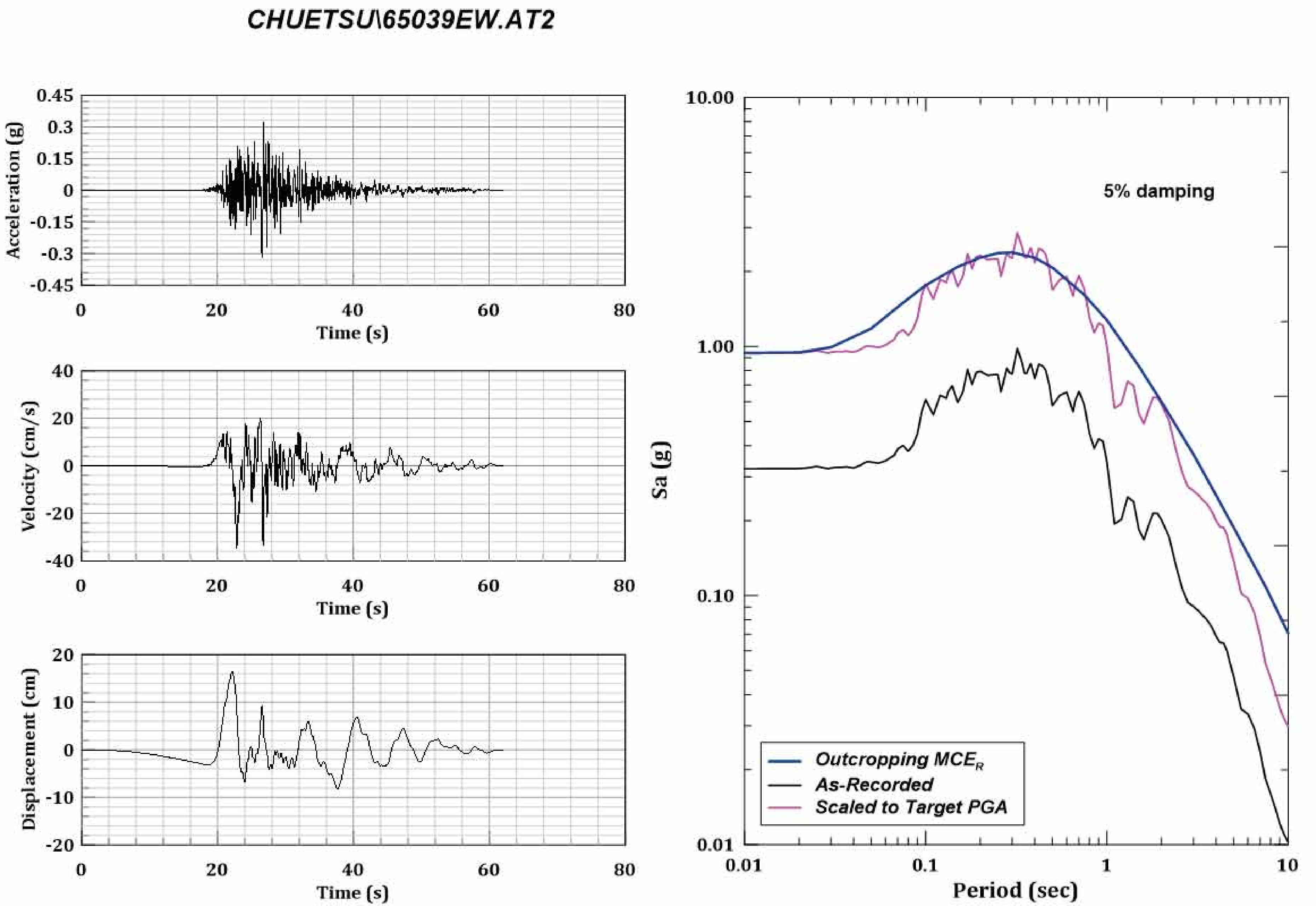
GeoPentech



GM3 SEED

Date: FEB. 2018	Project No.: 17025A	Project: 12575 BEATRICE PLAYA VISTA GROUND-MOTION DEVELOPMENT	Figure 16

GeoPentech

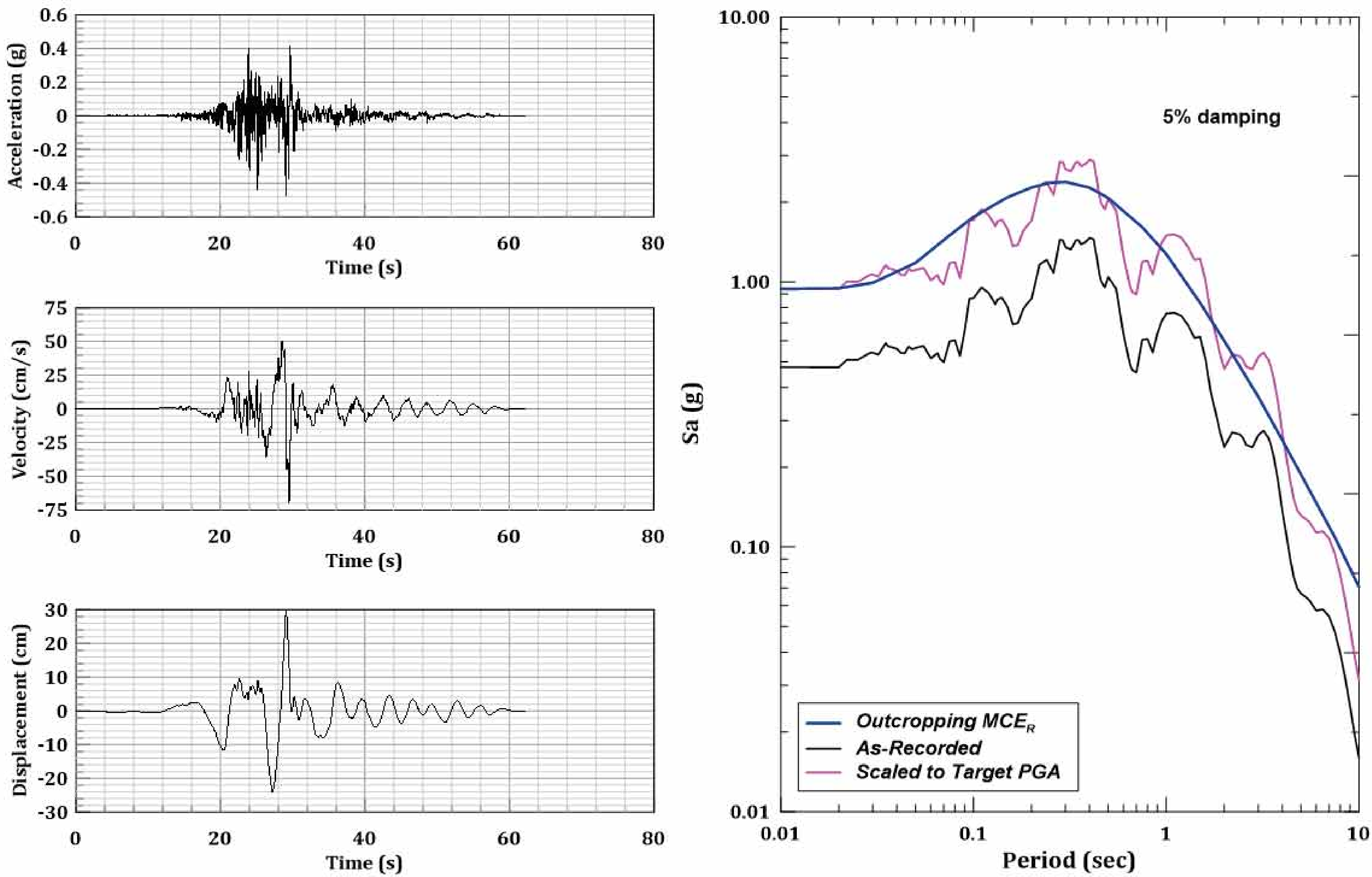


GM4 SEED

Date: FEB. 2018	Project No.: 17025A	Project: 12575 BEATRICE PLAYA VISTA GROUND-MOTION DEVELOPMENT	Figure 17

GeoPentech

DARFIELD\HORCS72E.AT2

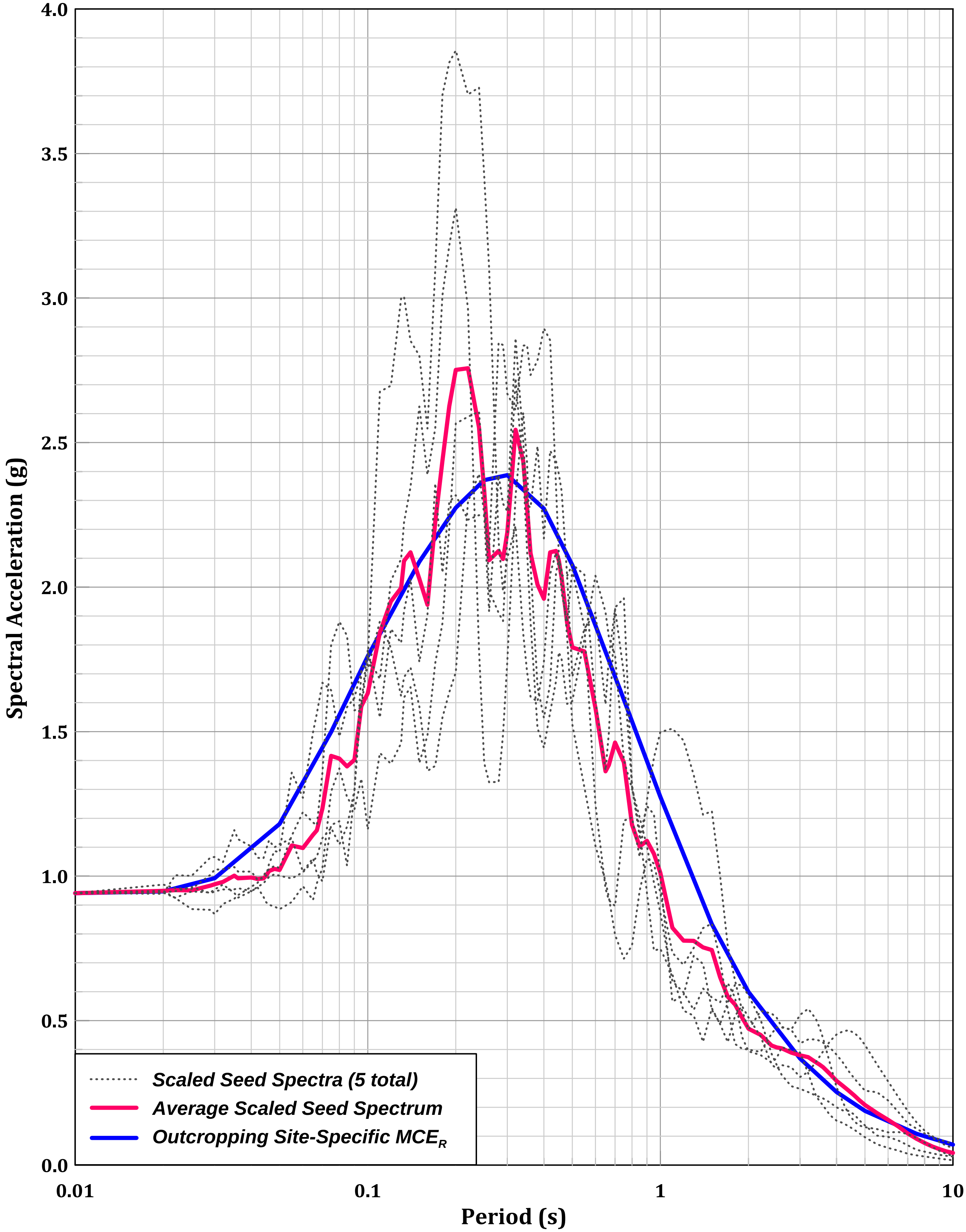


GM5 SEED

Date: FEB. 2018	Project No.: 17025A	Project: 12575 BEATRICE PLAYA VISTA GROUND-MOTION DEVELOPMENT	Figure 18

GeoPentech

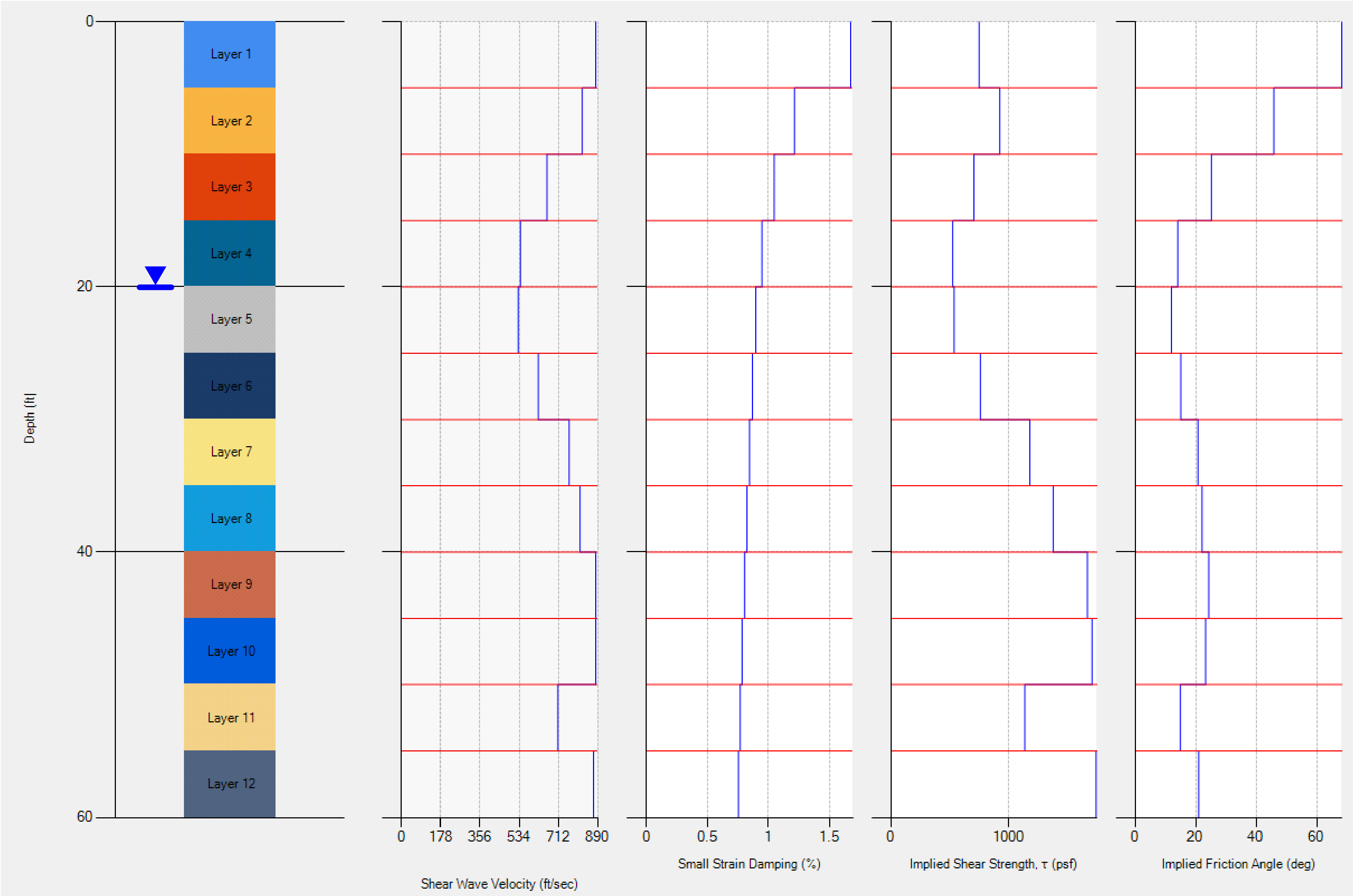
MCE$_R$-Scaled Seed Time History Spectra



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

MCE$_R$-SCALED SEED TIME HISTORY SPECTRA			
Project No.: 17025A	Project: 12575 BEATRICE GROUND MOTIONS	Date: FEB. 2018	Figure 19

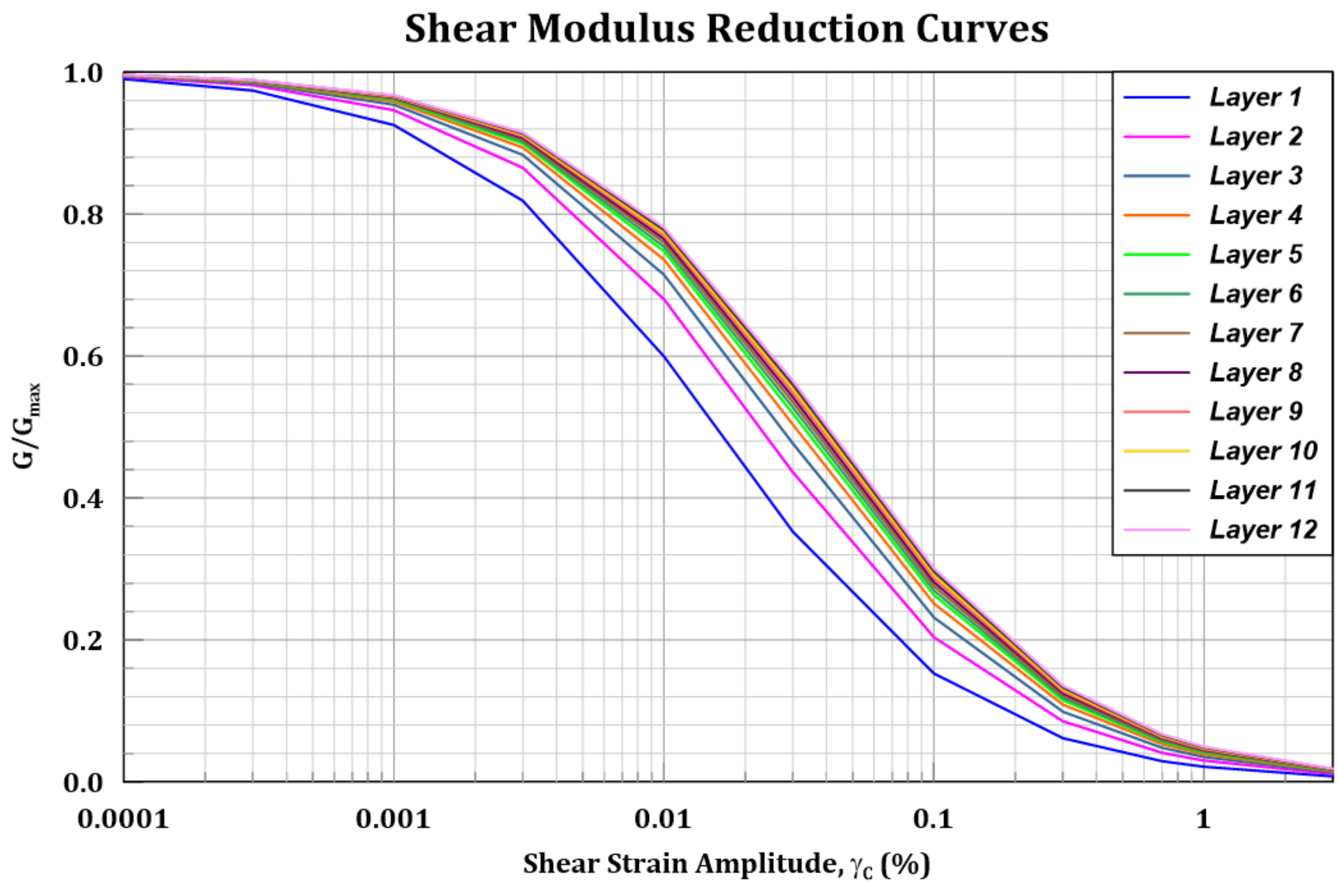
GeoPentech

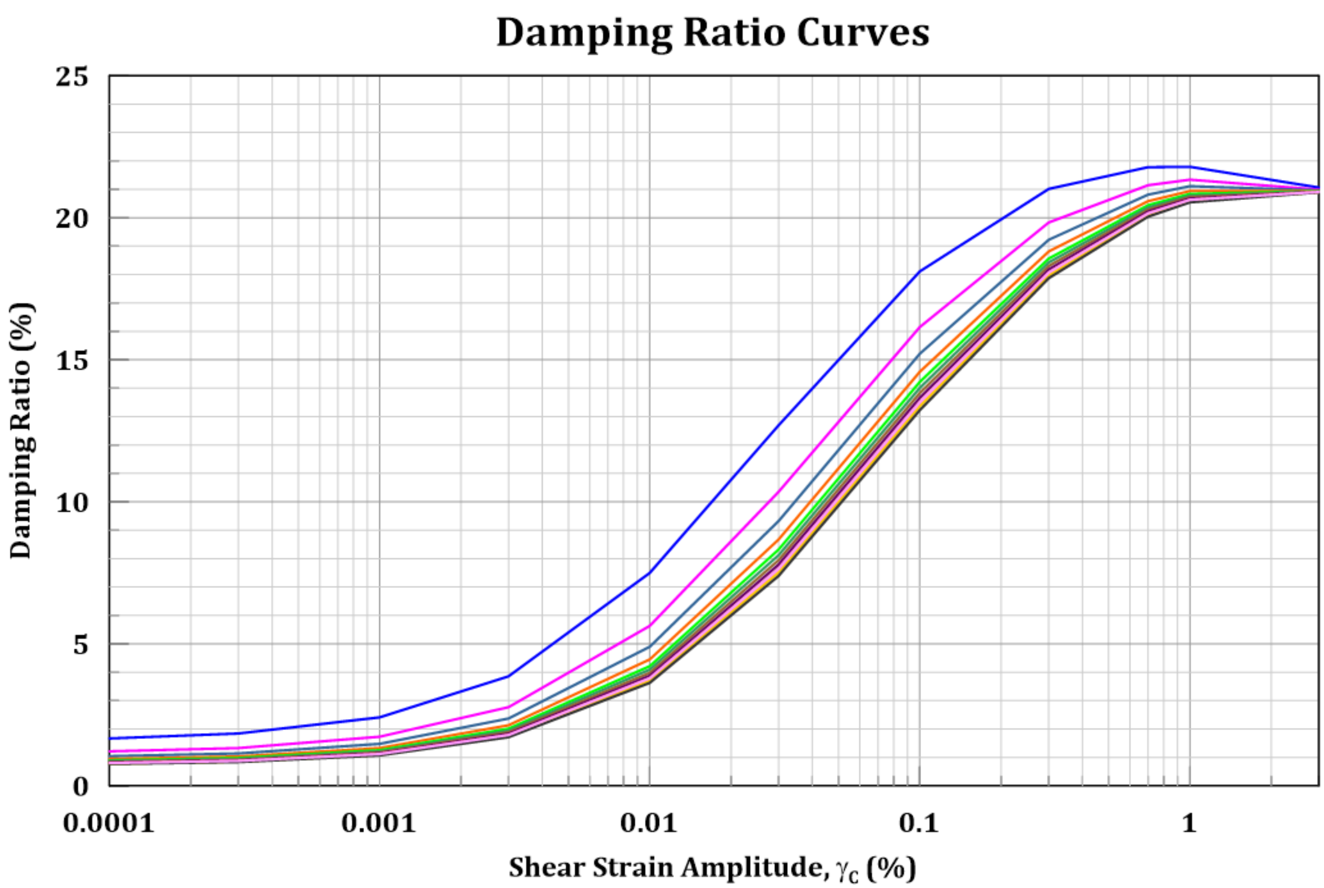


IDEALIZED SOIL COLUMN MODEL

Date: MAR 2018	Project No.: 17025A	Project: 12575 BEATRICE GROUND MOTIONS	Figure 20

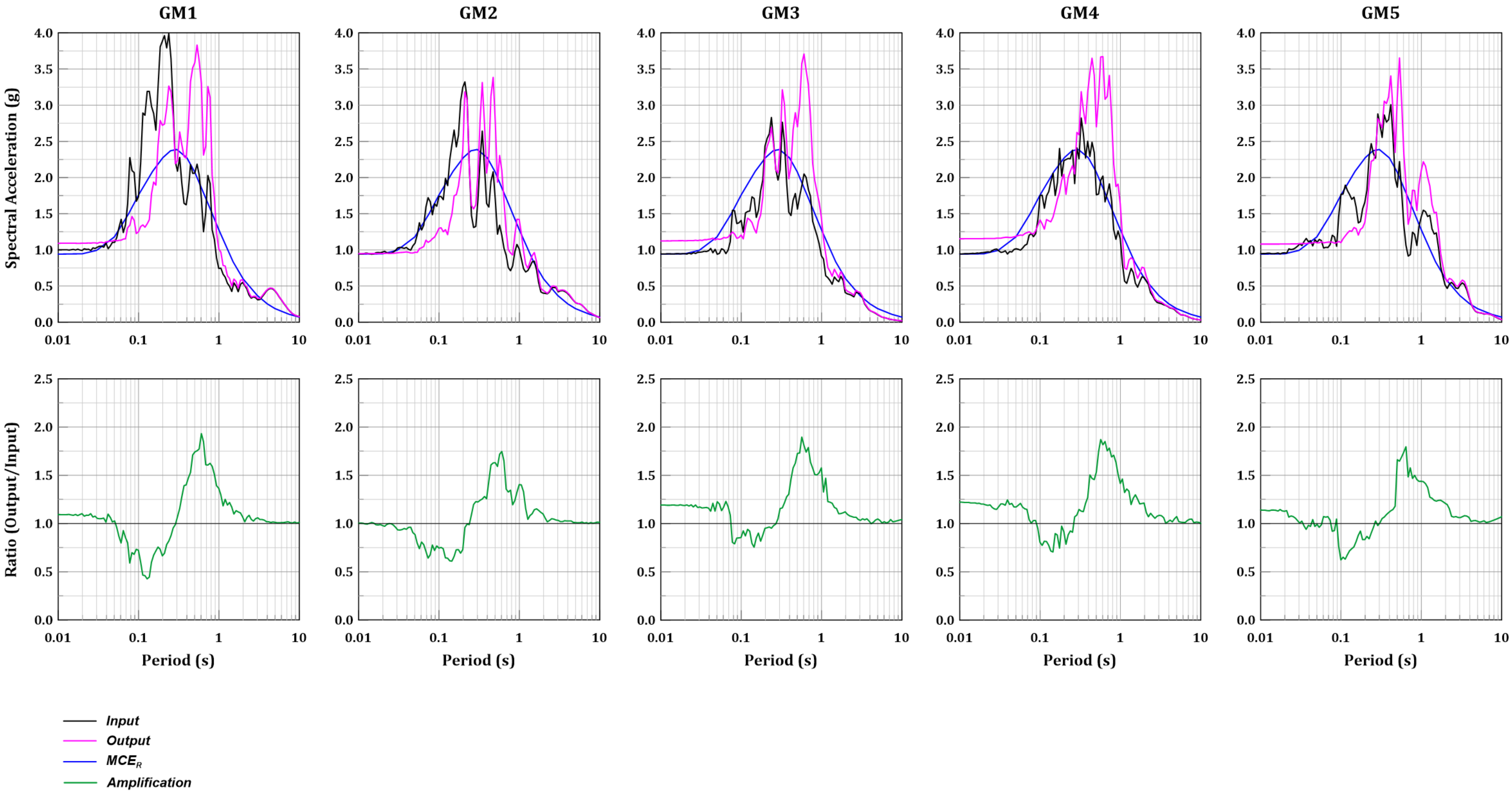
GeoPentech





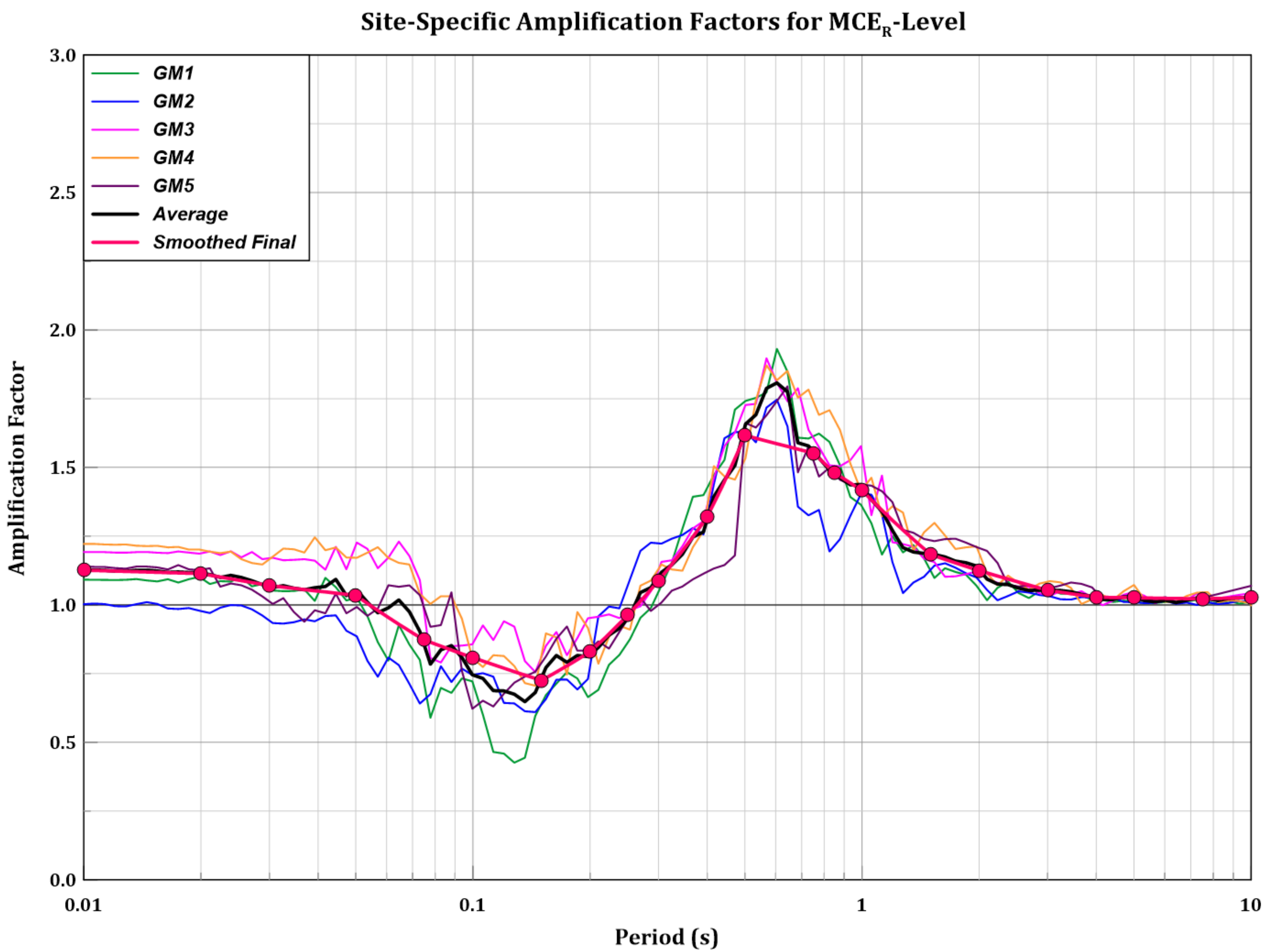
MODULUS REDUCTION AND DAMPING CURVES			
Project No.: 17025A	Project: 12575 BEATRICE GROUND MOTIONS	Date: MAR 2018	Figure 21

GeoPentech



SITE RESPONSE ANALYSIS RESULTS - MCE$_R$ LEVEL		
Project: 12575 BEATRICE GROUND MOTIONS		Figure 22
Project No.: 17025A	Date: MAR 2018	

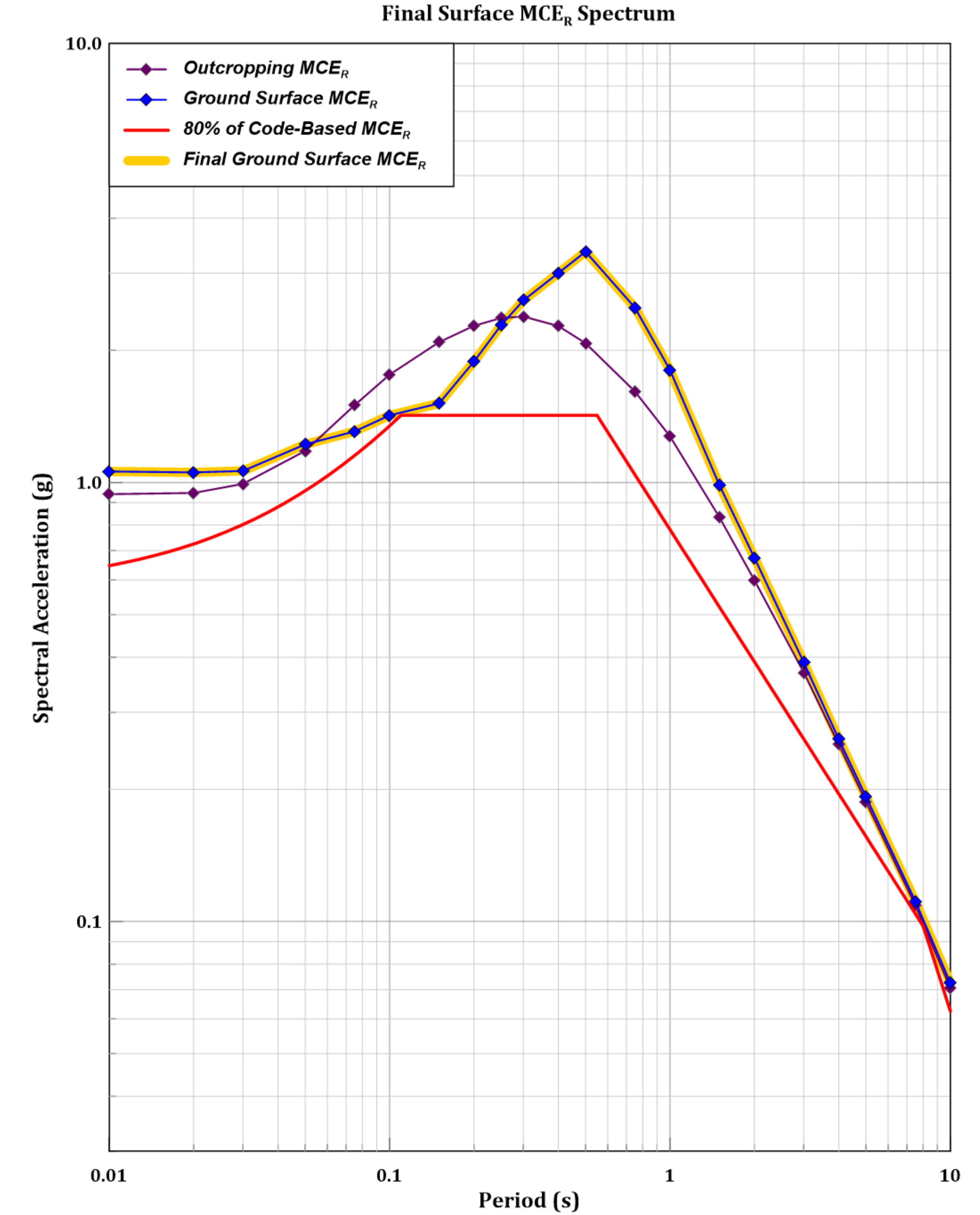
GeoPentech



SITE-SPECIFIC AMPLIFICATION FACTORS FOR MCE_R-LEVEL

Date: MAR 2018	Project No.: 17025A	Project: 12575 BEATRICE GROUND MOTIONS	Figure 23

GeoPentech

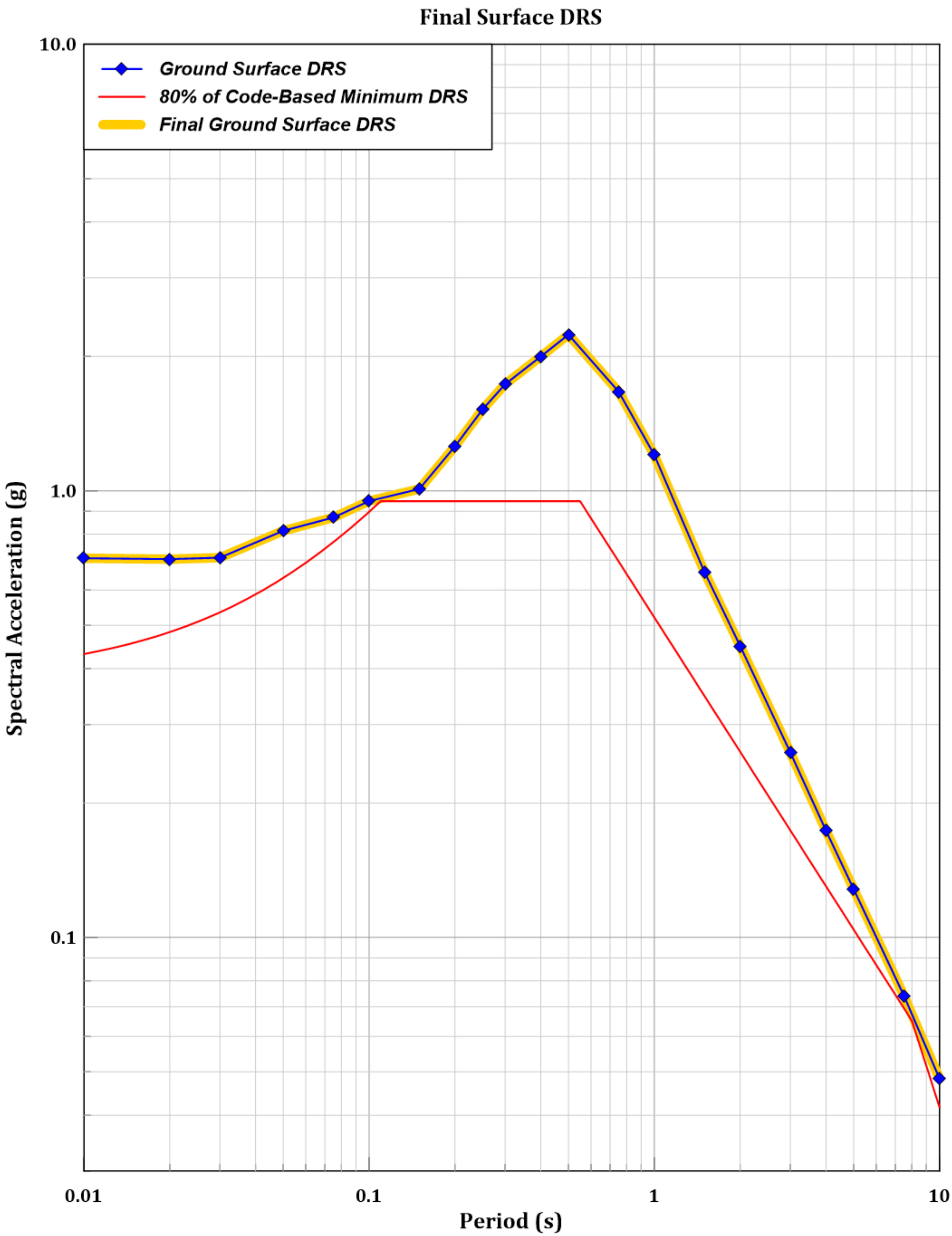


Note: All spectra are for Damping (β) = 5.0% unless otherwise indicated.

FINAL SURFACE MCE$_R$ SPECTRUM

Project No.: 17025A	Project: 12575 BEATRICE GROUND MOTIONS	Date: MAR 2018	Figure 24

GeoPentech



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

FINAL SURFACE DRS			
Project No.: 17025A	Project: 12575 BEATRICE GROUND MOTIONS	Date: MAR 2018	Figure 25

GeoPentech

Tuesday, April 03, 2018 via email: stang@geoteq.com

GEOTECHNOLOGIES, INC.
439 Western Ave.
Glendale, CA 91201

Attention: Mr. Stanley Tang

Re: Soil Corrosivity Study
Chait Company
Playa Del Rey, California
HDR #18-0198SCS, GI #21194

Introduction

Laboratory tests have been completed on three soil samples provided or the referenced project. The purpose of these tests was to determine whether the soils are likely to have deleterious effects on underground utility piping, hydraulic elevator cylinders, and concrete structures. HDR Engineering, Inc. (HDR) assumes that the samples provided are representative of the most corrosive soils at the site.

The proposed structure has 6 to 8 stories and 1.5 subterranean levels. The site is located at 12575 Beatrice Street in Playa Del Rey, California, and the water table is reportedly 24 to 30 feet deep.

The scope of this study is limited to a determination of soil corrosivity and general corrosion control recommendations for materials likely to be used for construction. HDR's recommendations do not constitute, and are not meant as a substitute for, design documents for the purpose of construction. If the architects and/or engineers desire more specific information, designs, specifications, or review of design, HDR will be happy to work with them as a separate phase of this project.

Laboratory Soil Corrosivity Tests

The electrical resistivity of each sample was measured in a soil box per ASTM G187 in its as-received condition and again after saturation with distilled water. Resistivities are at about their lowest value when the soil is saturated. The pH of the saturated samples was measured per CTM 643. A 5:1 water:soil extract from each sample was chemically analyzed for the major soluble salts commonly found in soil per ASTM D4327, ASTM D6919, and Standard Method 2320-B[1]. Laboratory test results are shown in the attached Table 1.

Soil Corrosivity

A major factor in determining soil corrosivity is electrical resistivity. The electrical resistivity of a soil is a measure of its resistance to the flow of electrical current. Corrosion of buried metal is an electrochemical process in which the amount of metal loss due to corrosion is directly proportional to the flow of electrical current (DC) from the metal into the soil. Corrosion currents, following Ohm's Law, are inversely proportional to soil resistivity. Lower electrical resistivities result from higher moisture and soluble salt contents and indicate corrosive soil.

A correlation between electrical resistivity and corrosivity toward ferrous metals is:[2]

Soil Resistivity in ohm-centimeters	Corrosivity Category
Greater than 10,000	Mildly Corrosive
2,001 to 10,000	Moderately Corrosive
1,001 to 2,000	Corrosive
0 to 1,000	Severely Corrosive

Other soil characteristics that may influence corrosivity towards metals are pH, soluble salt content, soil types, aeration, anaerobic conditions, and site drainage.

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[1] American Public Health Association (APHA). 2012. Standard Methods of Water and Wastewater. 22nd ed. American Public Health Association, American Water Works Association, Water Environment Federation publication. APHA, Washington D.C.

[2] Romanoff, Melvin. Underground Corrosion, NBS Circular 579. Reprinted by NACE. Houston, TX, 1989, pp. 166–167.

Electrical resistivities were in the moderately to severely corrosive categories with as-received moisture and at saturation. The as-received resistivities were at or near their saturated values.

Soil pH values varied from 7.4 to 7.6. This range is mildly alkaline.[3] These values do not particularly increase soil corrosivity.

The soluble salt content of the samples ranged from moderate to high. Bicarbonate, chloride, and sulfate salts were the primary constituents.

Nitrate as detected in low concentrations. The ammonium concentration in the sample from B-2 was high enough to be aggressive to copper.

Tests were not made for sulfide and oxidation-reduction (redox) potential because these samples did not exhibit characteristics typically associated with anaerobic conditions.

This soil is classified as severely corrosive to ferrous metals and aggressive to copper.

Corrosion Control Recommendations

The life of buried materials depends on thickness, strength, loads, construction details, soil moisture, etc., in addition to soil corrosivity, and is, therefore, difficult to predict. Of more practical value are corrosion control methods that will increase the life of materials that would be subject to significant corrosion.

The following recommendations are based on the soil conditions discussed in the Soil Corrosivity section above. Unless otherwise indicated, these recommendations apply to the entire site or alignment.

Steel Pipe

1. Underground steel pipe with rubber gasketed, mechanical, grooved end, or other nonconductive type joints should be bonded for electrical continuity. Electrical continuity is necessary for corrosion monitoring and cathodic protection.

2. Install corrosion monitoring test stations to facilitate corrosion monitoring and the application of cathodic protection:

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[3] *Romanoff, Melvin. Underground Corrosion, NBS Circular 579. Reprinted by NACE. Houston, TX, 1989, p. 8.*

a. At each end of the pipeline.

b. At each end of all casings.

c. Other locations as necessary so the interval between test stations does not exceed 1,200 feet.

3. To prevent dissimilar metal corrosion cells and to facilitate the application of cathodic protection, electrically isolate each buried steel pipeline per NACE SP0286 from:

   a. Dissimilar metals.

   b. Dissimilarly coated piping (cement-mortar vs. dielectric).

   c. Above ground steel pipe.

   d. All existing piping.

4. Choose one of the following corrosion control options:

   OPTION 1

   a. Apply a suitable dielectric coating intended for underground use such as:

      i. Polyurethane per AWWA C222 *or*

      ii. Extruded polyethylene per AWWA C215 *or*

      iii. A tape coating system per AWWA C214 *or*

      iv. Hot applied coal tar enamel per AWWA C203 *or*

      v. Fusion bonded epoxy per AWWA C213.

   b. Apply cathodic protection to steel piping as per NACE SP0169.

   OPTION 2

   a. As an alternative to dielectric coating and cathodic protection, apply a ¾-inch cement mortar coating per AWWA C205 or encase in concrete 3 inches thick, using any type of ASTM C150 cement. Joint bonds, test stations, and insulated joints are still recommended for this alternative.

NOTE: Some steel piping systems, such as for oil, gas, and high-pressure piping systems, have special corrosion and cathodic protection requirements that must be evaluated for each specific application.

Hydraulic Elevators

1. Choose one of the following corrosion control options for the hydraulic steel cylinders.

   OPTION 1

   a. Coat hydraulic elevator cylinders with a suitable dielectric coating intended for underground use such as:

      i. Polyurethane per AWWA C222 or

      ii. Extruded polyethylene per AWWA C215 or

      iii. A tape coating system per AWWA C214 or

      iv. Hot applied coal tar enamel per AWWA C203 or

      v. Fusion bonded epoxy per AWWA C213.

   b. Electrically insulate each cylinder from building metals by installing dielectric material between the piston platen and car, insulating the bolts, and installing an insulated joint in the oil line.

   c. Apply cathodic protection to hydraulic cylinders as per NACE SP0169.

   OPTION 2

   a. As an alternative to electrical insulation and cathodic protection, place each cylinder in a plastic casing with a plastic watertight seal at the bottom.

2. The elevator oil line should be placed above ground if possible but, if underground, should be protected by one of the following corrosion control options:

   OPTION 1

   a. Provide a bonded dielectric coating.

b. Electrically isolate the pipeline.

c. Apply cathodic protection to steel piping as per NACE SP0169.

OPTION 2

a. Place the oil line in a PVC casing pipe with solvent-welded joints and sealed at both ends to prevent contact with soil and moisture.

Ductile Iron Pipe

1. To prevent dissimilar metal corrosion cells and to facilitate the application of cathodic protection, electrically insulate underground iron pipe from dissimilar metals and from above ground iron pipe with insulating joints per NACE SP0286.

2. Bond all nonconductive type joints for electrical continuity. Electrical continuity is necessary for corrosion monitoring and cathodic protection.

3. Install corrosion monitoring test stations to facilitate corrosion monitoring and the application of cathodic protection:

   a. At each end of the pipeline.

   b. At each end of any casings.

   c. Other locations as necessary so the interval between test stations does not exceed 1,200 feet.

4. Choose one of the following corrosion control options:

   OPTION 1

   a. Apply a suitable coating intended for underground use such as:

      i. Polyethylene encasement per AWWA C105; *or*

      ii. Epoxy coating; *or*

      iii. Polyurethane; *or*

      iv. Wax tape.

NOTE: The thin factory-applied asphaltic coating applied to ductile iron pipe for transportation and aesthetic purposes does not constitute a corrosion control coating.

b. Apply cathodic protection to cast and ductile iron piping as per NACE SP0169.

OPTION 2

a. As an alternative to the coating systems described in Option 1 and cathodic protection, concrete encase all buried portions of metallic piping so that there is a minimum of 3 inches of concrete cover provided over and around surfaces of pipe, fittings, and valves using any type of ASTM C150 cement.

NOTE: Some iron piping systems, such as for fire water piping, have special corrosion and cathodic protection requirements that must be evaluated for each specific application.

Cast Iron Soil Pipe

1. Protect cast iron soil pipe with either a double wrap 4-mil or single wrap 8-mil polyethylene encasement per AWWA C105.
2. It is not necessary to bond the pipe joints or apply cathodic protection.
3. Provide 6 inches of clean sand backfill all around the pipe.

Clean Sand Backfill

1. HDR recommends the following parameters for clean sand backfill:
   a. Minimum saturated resistivity of no less than 3,000 ohm-cm; *and*
   b. pH between 6.0 and 8.0.
2. All backfill testing should be performed by a corrosion engineering laboratory.

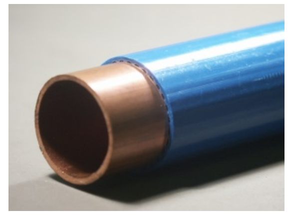
Copper Tubing

1. Electrically insulate underground copper pipe from dissimilar metals and from above ground copper pipe with insulating devices per NACE SP0286.
2. Electrically insulate cold water piping from hot water piping systems.

3. Protect buried copper tubing by one of the following measures:

    a. Prevention of soil contact. Soil contact may be prevented by placing the tubing above ground or encasing the tubing using PVC pipe with solvent-welded joints.

    b. Installation of a factory-coated copper pipe with a minimum 25-mil thickness such as Kamco's Aqua Shield™, Mueller's Streamline Protec™, or equal. The coating must be continuous with no cuts or defects.



    c. Installation of 12-mil polyethylene pipe wrapping tape with butyl rubber mastic over a suitable primer. Protect wrapped copper tubing by applying cathodic protection per NACE SP0169.

Plastic and Vitrified Clay Pipe

1. No special corrosion control measures are required for plastic and vitrified clay piping placed underground.

2. Protect all metallic fittings and valves with wax tape per AWWA C217, or with epoxy and appropriately sized cathodic protection per NACE SP0169.

All Pipe

1. On all pipes, appurtenances, and fittings not protected by cathodic protection, coat bare metal such as valves, bolts, flange joints, joint harnesses, and flexible couplings with wax tape per AWWA C217 after assembly.

2. Where metallic pipelines penetrate concrete structures such as building floors, vault walls, and thrust blocks use plastic sleeves, rubber seals, or other dielectric material to prevent pipe contact with the concrete and reinforcing steel.

Concrete Structures and Pipe

1. From a corrosion standpoint, any type of ASTM C150 cement may be used for concrete structures and pipe because the sulfate concentration is negligible, from 0 to 0.10 percent.[4,5,6]

2. Standard concrete cover over reinforcing steel may be used for concrete structures and pipe in contact with these soils due to the low chloride concentrations[7] found onsite. Limit the water-soluble chloride ion content in the concrete mix design to less than 0.3 percent by weight of cement.

3. Due to the deep reported groundwater at this site, cyclical or continual wetting of the subterranean levels is not anticipated. However, any contact between concrete structures and groundwater should be prevented.

Concrete Piles

Precast Concrete Piles

1. It is assumed that precast concrete piles will contain a minimum of 8 sacks of ASTM C150 Type V cement per cubic yard of concrete, a water/cement ratio not exceeding 0.45, and 2 inches of concrete cover. No further corrosion control measures are required for such piles.

2. If groundwater is present, solid steel lifting lugs are recommended to prevent groundwater from wicking into the pile interior. If wire rope lifting lugs are used, they should be carefully drilled out 1.5 inches deep and the hole filled with epoxy.

3. HDR understands that there may be no practical way to waterproof precast concrete piles. The concrete mix design for the piles should include supplementary cementitious admixtures to reduce permeability.

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[4] 2015 International Building Code (IBC) which refers to American Concrete Institute (ACI) 318-14 Table 19.3.2.1

[5] 2015 International Residential Code (IRC) which refers to American Concrete Institute (ACI) 318-14 Table 19.3.2.1

[6] 2016 California Building Code (CBC) which refers to American Concrete Institute (ACI) 318-14 Table 19.3.2.1

[7] Design Manual 303: Concrete Cylinder Pipe. Ameron. p.65

Steel Reinforced Cast in Place Concrete Piles

1. Protect steel reinforced cast-in-place and cast-in-drilled-hole concrete piles in accordance with the recommendations of the concrete structures section in this report.

2. HDR understands that there may be no practical way to waterproof cast in place concrete piles. The concrete mix design for the piles should include supplementary cementitious admixtures to reduce permeability.

Closure

The analysis and recommendations presented in this report are based upon data obtained from the laboratory samples. This report does not reflect variations that may occur across the site or due to the modifying effects of construction. If variations appear, HDR should be notified immediately so that further evaluation and supplemental recommendations can be provided.

HDR's services have been performed with the usual thoroughness and competence of the engineering profession. No other warranty or representation, either expressed or implied, is included or intended.

Please call if you have any questions.

Respectfully Submitted,
HDR Engineering, Inc.

04/03/2018

James Keegan

Greg Frost, PE

Enc: Table 1

Table 1 - Laboratory Tests on Soil Samples

Geotechnologies, Inc.
Chait Company
Your #21194, HDR Lab #18-0198SCS
27-Mar-18

Sample ID

			B1 @ 7.5'	B2 @ 17.5'	B3 @ 47.5'
Resistivity		**Units**			
as-received		ohm-cm	2,000	680	3,840
saturated		ohm-cm	1,400	680	2,840
pH			7.4	7.5	7.6
Electrical Conductivity		mS/cm	0.21	0.57	0.20
Chemical Analyses					
Cations					
calcium	Ca^{2+}	mg/kg	41	186	33
magnesium	Mg^{2+}	mg/kg	22	46	20
sodium	Na^{1+}	mg/kg	164	318	129
potassium	K^{1+}	mg/kg	6.7	23	13
Anions					
carbonate	CO_3^{2-}	mg/kg	ND	ND	ND
bicarbonate	HCO_3^{1-}	mg/kg	256	461	122
fluoride	F^{1-}	mg/kg	4.1	8.0	1.9
chloride	Cl^{1-}	mg/kg	44	189	83
sulfate	SO_4^{2-}	mg/kg	202	778	204
phosphate	PO_4^{3-}	mg/kg	4.1	ND	ND
Other Tests					
ammonium	NH_4^{1+}	mg/kg	ND	19	0.1
nitrate	NO_3^{1-}	mg/kg	1.4	6.4	1.4
sulfide	S^{2-}	qual	na	na	na
Redox		mV	na	na	na

Resistivity per ASTM G187, Cations per ASTM D6919, Anions per ASTM D4327, and Alkalinity per APHA 2320-B.
Electrical conductivity in millisiemens/cm and chemical analyses were made on a 1:5 soil-to-water extract.
mg/kg = milligrams per kilogram (parts per million) of dry soil.
Redox = oxidation-reduction potential in millivolts
ND = not detected
na = not analyzed

431 West Baseline Road · Claremont, CA 91711
Phone: 909.962.5485 · Fax: 909.626.3316

CITY OF LOS ANGELES

CALIFORNIA

ERIC GARCETTI
MAYOR

DEPARTMENT OF
BUILDING AND SAFETY
201 NORTH FIGUEROA STREET
LOS ANGELES, CA 90012

FRANK M. BUSH
GENERAL MANAGER
SUPERINTENDENT OF BUILDING

OSAMA YOUNAN, P.E.
EXECUTIVE OFFICER

SOILS REPORT REVIEW LETTER

March 23, 2017

LOG # 97201
SOILS/GEOLOGY FILE - 2
LIQ

Chait Company Architects
7306 Coldwater Canyon Avenue, Unit 12
North Hollywood, CA 91605

TRACT:	30549
LOT(S):	20-21
LOCATION:	12575 W. Beatrice St.

CURRENT REFERENCE REPORT/LETTER(S)	REPORT No.	DATE(S) OF DOCUMENT	PREPARED BY
Soils Report	21194	04/04/2016	Geotechnologies, Inc.

The Grading Division of the Department of Building and Safety has reviewed the referenced report that provides recommendations for the proposed 10-story office building over one level of subterranean parking.

The site is located in a designated liquefaction hazard zone as shown on the Seismic Hazard Zones map issued by the State of California.

Due to site access restrictions, no geotechnical borings were performed at this site. The consultants stated that "a comprehensive report shall be prepared when the site is available for exploration and the development plan achieves refinement".

The review of the subject report can not be completed at this time and will be continued upon submittal of an addendum to the report which shall include, but not be limited to, the following:

1. Perform comprehensive geotechnical investigation at this site including geotechnical borings, laboratory testing, liquefaction analysis, and foundation engineering analysis. Prepare a comprehensive geotechnical report and submit this report to the Department for review and approval. Note that the Department does not accept liquefaction analysis, foundation engineering analysis, site class classifications, and grading recommendations etc. solely based on Cone Penetration tests. Actual drilling, sampling, and laboratory testing shall be performed at this site.

The soils engineer shall prepare a report containing an itemized response to the review items indicated in this letter. If clarification concerning the review letter is necessary, the report review engineer may be contacted. Two copies of the response report, including one unbound wet-signed original for archiving purposes, a pdf-copy of the complete report in a CD or flash drive, and the appropriate fees will be required for submittal.

YING LIU
Geotechnical Engineer I

YL/yl
Log No. 97201
213-482-0480

cc: Geotechnologies, Inc., Project Consultant
WL District Office