13.0 GROUND MOTION DEVELOPMENT

13.1 Introduction

This section describes the development of earthquake response spectra and acceleration time histories for seismic analysis of the Tower. Time histories were developed for application in the non-linear structural analysis of the Tower performed by SGH. Figures and tables referenced in this section are provided in Attachment D and follow the naming convention "Figure D-X" and "Table D-X."

13.2 Methodology

Subsurface conditions at the project site consist of approximately 250 feet of soil overlying Franciscan Complex bedrock of varying quality. The existing Tower foundation system consists of piles extending approximately 75 to 85 feet bgs, with the bottom of the foundation mat at a depth of about 25 feet bgs. Our understanding is that the proposed PPU retrofit alternative currently under consideration consists of installing new additional piles along select areas of the foundation mat perimeter under the sidewalk. Based on discussions with the retrofit structural engineer, SGH, we understand that the non-linear structural model has been developed for time histories to be input at the bottom of the foundation mat (≈25 feet bgs).

In formulating our approach to developing the response spectra and time histories, we considered two aspects in addition to the typical methodology: 1) non-ergodic vs. ergodic treatment of site response within the hazard analysis, and 2) total depth of the site profile for site-response analysis. Both considerations stem from the potential for estimating inappropriately low spectral accelerations at long periods. The use of a "shallow" site-response profile does not allow the full wavelength of longer period motions to travel through the site-response profile. This would result in an underestimation of site response if longer wavelengths were present and not captured. The use of non-ergodic site response within the hazard calculation is one method for mitigating this effect (e.g., Stewart et al. 2017). Based on a review of Kamai et al. (2013),, we note that the presence of long-period spectral content due to site amplification is primarily observed in deep soil profiles corresponding to basin sites that are characteristic of Los Angeles and Seattle, where depth to rock often exceeds 500-1000 feet, rather than sites in San Francisco where the depth to rock is shallower (i.e., ≈ 250 feet bgs at the project site). Additionally, overlying soil thickness and depth to bedrock in the areas of San Francisco surrounding the project site can be described as highly variable and are not consistent with the subsurface conditions that lead to basin amplification. We have opted to extend the site profile to a greater depth to capture any long period spectral content that may be present at our site.

Ground motion prediction equations (GMPEs) are applicable for a range of the shear wave velocity parameter, V_{S30}, as specified by the GMPE developers and based on the underlying empirical data. V_{S30} represents the average shear wave velocity of the soil or rock over the upper 30 meters or 100 feet of a subsurface profile for analysis. The upper bound of V_{S30} applicability for the PEER NGA-West2 GMPEs, described further in Section 12.4.1.2, is between V_{S30} = 3281 feet per second (ft/s) (1000 meters per second [m/s]) and V_{S30} = 4921 ft/s (1500 m/s). Considering this constraint, and to allow for a deeper site profile for site-response analysis, we developed a deep shear wave velocity (V_S) profile and bedrock response spectra for the depth at which V_{S30} = 1200 m/s, which is within bedrock, rather than at the overlying soil and bedrock interface. This allows for a sufficiently deep profile to avoid potential underestimates of the long-period spectral content, while staying within the bounds of GMPE applicability. The bedrock response spectra are then propagated through the site profile with random-vibration theory (RVT) site response analysis. Foundation-level response spectra were obtained from the site-response analysis and served as the basis for time history development for the non-linear structural model. The process for time history development is outlined below:

1. Develop ASCE/SEI 7-16 site-specific MCE_R bedrock response spectra for a deep site profile with V_{S30} =1200 m/sec and Site Class B coefficients.
2. Perform site-response using the ASCE/SEI 7-16 site-specific MCE_R spectrum and site-specific 43-year return period spectrum.
3. Select time histories for the soft soil site conditions at foundation mat level.
4. Perform spectral matching.
5. Scale time histories to meet or exceed the ASCE/SEI 7-16 maximum-direction and Chapter 21 criteria.
6. Rotate time histories to fault-normal/fault-parallel direction.

13.3 Shear Wave Velocity Profile Development

The shear wave velocity profile for the project site was developed using two sources. The first was the near-surface V_S profile developed based on P-S logging V_S measurements extending 150-350 feet bgs from the nearby TTC (ARUP, 2010). The second was the Kamai et al. (2013) deep V_S profile for a V_{s30} = 160 m/s that extends several thousand feet bgs. Following discussions with Professor Kamai, the PEER profile considered for our model was shifted up to begin where the site-specific profile transitions to bedrock. Based on the applicable range of V_{S30} for the four PEER GMPEs selected, the depth corresponding to V_{S30} = 1200 m/s was chosen as the depth for which bedrock response spectra would be developed. V_{S30} = 1200 m/s corresponds to a Site Class B designation per ASCE/SEI 7-16. Depth to V_s of 2500 m/s ($Z_{2.5}$) was also estimated from the site-specific V_s profile, for use as GMPE inputs in the hazard analysis. The input of depth to V_S = 1000 m/s ($Z_{1.0}$) was shallower than the depth for which ground motions were developed and was therefore set to zero for GMPE inputs.

Figure D-1 shows the PEER V_S profile, the V_S profile from the east side of the TTC, and the development of the site-specific V_s profile from those two sources. Figure D-2 shows the site-specific V_s profile with the V_S parameters used in the GMPEs. Figure D-3 shows the V_S profile focusing on the upper 410 feet bgs used in site-response analysis.

GMPE Inputs from Site-Specific Profile: V_{S30} = 1200 m/s (3937 ft/s) at a depth of 410 feet

V_S = 2500 m/s (8202 ft/s) at a depth of 3460 feet ($Z_{2.5}$ = 3010 feet)

V_{S30} = 200 m/s at the mat foundation level (≈25 feet bgs; this will be used in the time history selection),

13.4 Bedrock Response Spectra Development

The bedrock-level acceleration response spectra for the project site were developed in accordance with the guidelines set forth by the Pacific Earthquake Engineering Research Center (PEER) Tall Buildings Initiative (TBI) report, Version 2.03, May 2017. The TBI report specifies that response spectra should be developed for two levels of ground shaking: the ASCE/SEI 7-16 Risk-Targeted Maximum Considered Earthquake (MCE_R) and the Service-Level Earthquake (43-year [yr] return period uniform hazard spectrum). Development of these spectra is described in the following sections. The response spectra developed for V_{S30} = 1200 m/s were used as the input response spectra for the site-response analysis.

13.4.1 Risk-Targeted Maximum Considered Earthquake (MCE_R)

The TBI adopts the MCE_R shaking level as defined in ASCE/SEI 7-16. Several response spectra were developed and compared to develop the final MCE_R response spectrum, including the:

1. code-based, map-based MCE_R spectrum for ASCE/SEI 7-16,
2. site-specific probabilistic spectra,
3. site-specific deterministic spectra,
4. code-based site-specific MCE_R probabilistic and deterministic spectra for design of new buildings,
5. and the code-based deterministic lower limit spectrum.

The process for developing the response spectra for the project site is described in the following subsections.

13.4.1.1 Code-Based Map-Based Spectra for ASCE/SEI 7-16

Response spectra parameters were established using mapped values from the 2014 USGS National Seismic Hazard Mapping Project (NSHMP) design maps (Petersen et al., 2014). These values were obtained from the USGS/FEMA-NEHRP U.S. Seismic Design Maps web service tool (https://earthquake.usgs.gov/ws/designmaps/asce7-16.htmlhttp://earthquake.usgs.gov/hazards/). The values were then modified to develop code-based map-based design and Maximum Considered Earthquake (MCE_R) response spectra following ASCE/SEI 7-16. The USGS tool provides risk-targeted, maximum response orientation, mapped spectral accelerations for the MCE hazard level for a reference site condition and the mapped long period transition period (T_L) for the site. Site response adjustment factors, F_a and F_v, were used to modify the response spectra for Site Class B site conditions (i.e., rock, 760 m/s < V_{S30} < 1500 m/s). The MCE short-period (S_S) and 1-second (S_1) spectral accelerations and T_L given by the NSHMP results are shown below, along with the values of F_a and F_v, from which we calculate S_{MS} and S_{M1} by applying the site coefficients to the NSHMP spectral acceleration values.

$$S_S = 1.5g \,;\, F_a = 0.9 \,;\, S_{MS} = 1.35g$$

$$S_1 = 0.6g \,;\, F_v = 0.8 \,;\, S_{M1} = 0.48g$$

$$T_L = 12 \text{ seconds}$$

Design spectral accelerations S_{DS} and S_{D1} are taken as two-thirds of the S_{MS} and S_{M1}. The design response spectrum (DRS) was developed in accordance with Section 11.4.6 of ASCE/SEI 7-16 as defined by the following equations:

$$S_a = S_{DS}\left(0.4 + 0.6\frac{T}{T_0}\right) \text{ for } T < T_0 \quad (1)$$

$$S_a = S_{DS} \text{ for } T_0 < T < T_S \quad (2)$$

$$S_a = \frac{S_{D1}}{T} \text{ for } T_S < T < T_L \quad (3)$$

$$S_a = \frac{S_{D1} * T_L}{T^2} \text{ for } T > T_L \quad (4)$$

The MCE_R is taken as 1.5 times the DRS at all periods. The map-based design and maximum considered earthquake response spectra for the Tower site are shown on Figure D-4 and tabulated in Table D-1.

13.4.1.2 Site-Specific Probabilistic Seismic Hazard Analysis

The probabilistic seismic hazard analysis (PSHA) was performed to characterize earthquake ground motions that may occur at the project site due to an earthquake in the region. The PSHA for this study generally follows the standard approach first developed by Cornell (1968) with the inclusion of parameter are randomization, and the consideration of epistemic uncertainty.

A Poisson process is used to compute how often a specified level of ground motion will be exceeded at a site. The PSHA computes the annual number of events that produce a ground motion parameter, Z, that exceeds a specified level, z. This number of events per year, ν, is also called the "annual frequency of exceedance," the inverse of which is called the "return period".

The calculation of the annual frequency of exceedance, ν, considers the rate of earthquakes of magnitudes 5 or greater, the rupture dimension of the earthquakes, the distance of the site relative to the earthquake, and the attenuation of the ground motion from the earthquake rupture to the site. The annual rate of exceedance of a ground motion test value, z, from a source, i, for a given earthquake that occurred on the source, i, is given by the equation:

$$\nu_i(Z > z) = N_i(E_i)P(Z > z|E_i)$$

where:

E_i is the given earthquake from source i, with a known magnitude and distance; and

$N_i(E_i)$ is the annual rate of the given earthquake per year from source i

The PSHA calculations are performed using the computer program Haz45.2 developed by Norm Abrahamson as modified to include UCERF3 (Slate, 2018). This program was validated as part of the Pacific Earthquake Engineering Research (PEER) center Probabilistic Seismic Hazard Analysis Code Verification Project (Hale et al., 2018).

13.4.1.3 Seismic Source Characterization

The seismic source characterization is based on the Uniform California Earthquake Rupture Forecast, Version 3 (UCERF3) (Field et al., 2013). The UCERF3 project was a multi-year project to provide an authoritative estimate of the magnitude, location and time-averaged frequency of potentially damaging earthquakes in California. This project addressed important issues such as: relaxing fault segmentation assumptions and to include multi-fault ruptures.

The UCERF3 model defines the long-term rate of all possible earthquake ruptures above a minimum magnitude of 5. Two alternative fault models, 3.1 and 3.2, give the spatial geometry of the larger, active faults throughout the region, with the alternative models representing the epistemic uncertainty in the fault system geometry. Each model comprises two types of sources: supraseismogenic sources who rupture dimensions are larger than the seismogenic depth occurring on explicitly modeled faults, and other earthquakes modeled as seismicity on a geographic grid. Supraseismogenic sources are modeled using fault segments with lengths that are approximately equal to the seismogenic depth. These fault segments are then linked to create larger fault ruptures. The two earthquake models have 253,706 and 305,709 unique "viable" ruptures respectively. The mean 3.1 and 3.2 fault models are implemented in this analysis. Input files for these models were provided via personal email communication with Kevin Milner (2018).

Figure 4 shows the location of the faults relative to the project site. The project location is approximately 14.6 km from the San Andreas fault system. Figures D-5 through D-8 show the recurrence rates for earthquakes on the San Andreas, San Gregorio, Calaveras and Hayward faults. The UCERF3 model allows very large ruptures which jump between fault systems which creates significantly larger magnitude earthquakes than are historically considered in the Bay Area. The implementation of UCERF3 considers these large ruptures, the effect of which are evident in the recurrence rates for the Hayward Fault. Ruptures to the north and south of the site that include segments of the Hayward fault are differentiated on Figure D-8 The ruptures to the south can continue all the way to the LA region and thus their magnitudes are significant. Due to the high activity rate of the San Andreas fault system, the close distance to the site, and the large magnitudes that rupture the San Andreas, this fault dominates the seismic hazard at this site for the annual probability levels considered in this analysis.

UCERF3 separates earthquakes into two main categories: supraseismogenic ruptures with length and width larger than the seismogenic depth occurring on explicitly modeled faults, and all other earthquakes, modeled as seismicity on a geographic grid. For each supraseismogenic rupture, the UCERF3 model provides the user with the associated magnitude, rate and multiple-plane geometry. The annual rate of exceedance of a ground motion test value, z, from the UCERF3 supraseismogenic sources is given by:

$$\nu(Z > z) = \sum_{N_{Supra}} N_i(E_i)P(Z > z|E_i)$$

where:

E_i is a given supraseismogenic rupture, with an associated magnitude and distance calculated from the given multiple-plane geometry;

$N_i(E_i)$ is the annual rate of the supraseimogenic rupture per year; and

N_{Supra} is the number of supraseimogenic ruptures with a magnitude greater than M_{min} and distance less than R_{max}

13.4.1.4 Rock Ground Motion Characterization

The rock ground motion is characterized by using a suite of four ground motion prediction equations (GMPEs) from the Next Generation Attenuation Relationships for Western US (NGA-West2) Project. The NGA-West2 models were developed as part of a multi-year effort to improve attenuation models for active tectonic regions such as California. This project addressed important issues such as: modeling of directionality, verification for recent small, moderate and large magnitude events, and evaluation of soil amplification factors.

The GMPEs selected for this analysis are Abrahamson, Silva and Kamai (2014), Boore et al. (2014), Campbell and Bozorgnia (2014) and Chiou and Youngs (2014). The GMPEs are given equal weight. The combined epistemic uncertainty in ground motion prediction and seismic source characterization is included in the PSHA by including an epistemic uncertainty factor of 2 on the ground motions.

The NGA-West 2 models use the average shear wave over the top thirty meters (Vs_{30}) as an index of site response. Basin response is included in the model by the depth to a shear wave velocity of 1,000 m/s ($Z_{1.0}$) and 2,500 m/s ($Z_{2.5}$). For this analysis the rock ground motion is characterized at a Vs_{30} of 1,200 m/s, $Z_{1.0}$ of 0.0 km, and $Z_{2.5}$ of 0.930 km. These depths are estimated based on the shear wave velocity profile in Section 12.3.

13.4.1.5 Probabilistic Seismic Hazard Analysis Results

Uniform hazard response spectra (UHS) for the 2%, 5%, and 20% probabilities of exceedance in 50 years, corresponding to equivalent 2,475-, 975-, and 225-year return periods, were developed and are included on Figure D-9 and in Table D-2. The UHS for the 50% probability of exceedance in 30 years (corresponding to a 43-year return period for the Service-Level Earthquake [SLE]) was also developed and is included on Figure D-9 and in Table D-2.

A deaggregation of the PSHA results was performed for the 2,475-year return period ground motion at 5 seconds, the results are shown on Figure D-10. The mean magnitude and distance for spectral acceleration at 5 seconds is 7.8 and 16 km, respectively.

13.4.1.6 Deterministic Seismic Hazard Analysis

The deterministic seismic hazard analysis (DSHA) scenario was selected based on the recurrence rates for supraseismogenic ruptures in UCERF3 that include rupture segments within 15 km of the project site, as shown on Figure D-11. The rupture segments within 15km of the site are shown in Figure 4. The deterministic magnitude is chosen by taking the magnitude with an annual frequency of exceedance that corresponds to the 2,475-year return period, for the site, a M_w 8.25 on the San Andreas fault, with a strike-slip mechanism. The surface trace of the San Andreas fault is located approximately 9.3 miles (14.6 kilometers) west of the project site, with a strike-slip faulting mechanism and vertical fault plane. Distance from the project site to the San Andreas fault was estimated using the Third California Earthquake Rupture Forecast (UCERF3) model (Field et al. 2014). Median and 84th percentile deterministic horizontal response spectra were developed for an M_W 8.25 earthquake rupturing the San Andreas fault as close as 9.3 miles (14.6 kilometers) from the project site using the same four 2014 NGA-West2 GMPEs and weights as used for the PSHA. Results of the DSHA are presented on Figure D-12 and in Table D-3.

13.4.1.7 Site-Specific Response Spectra for Design of New Buildings

Site-specific response spectra for new buildings were developed in accordance with ASCE/SEI 7-16. The risk-targeted maximum considered earthquake (MCE_R) ground motions were developed for the orientation of maximum horizontal response, and maximum demand factors were applied at all spectral periods to account for the maximum horizontal response. The maximum demand factors used for the site-specific analyses are included in Table D-4 and are as defined by Shahi and Baker (2013); these factors are generally consistent with those used for developing the NSHMP maps.

The site-specific probabilistic MCE_R was developed by taking the product of the PSHA 2,475-year return period response spectral ordinates, the risk coefficients, C_{RS} and C_{R1}, as determined from Figures 22-18 and 22-19 of ASCE/SEI 7-16 for the location of the site, and the maximum demand factors. The site-specific deterministic MCE_R was taken as the larger of the deterministic lower limit, as defined in ASCE/SEI 7-16 Section 21.2.2, and the controlling DSHA with the maximum demand factors. For the project site, the site-specific deterministic spectrum is taken as the deterministic lower limit at all periods except 4 seconds through 7.5 seconds, where it is controlled by the DSHA with maximum demand factors. The final site-specific MCE_R was then taken as the lesser of the site-specific probabilistic MCE_R and the site-specific deterministic MCE_R spectra. For the project site, the MCE_R spectral response ordinates are controlled by the probabilistic MCE_R for all periods. The site-specific probabilistic MCE_R, site-specific deterministic MCE_R, and final site-specific MCE_R spectra are presented on Figure D-13 and in Table D-4.

13.4.2 Service-Level Earthquake

TBI defines the Service-Level Earthquake (SLE) shaking level as the 43-year return period (50% probability of exceedance in 30 years), with ground motions obtained using probabilistic seismic hazard analysis. TBI descriptions of the SLE spectrum state that SLE ground motions are obtained "using probabilistic seismic hazard analysis (PSHA)" and that PSHA should be used to obtain the "uniform hazard spectra associated with SLE and 2475-year ground motions levels." TBI explicitly describes the MCE_R spectrum as "modified from the 2475-year ground motion level through application of risk coefficients and maximum direction coefficients." This implies that risk coefficients and maximum direction coefficients are not applied to the 43-year uniform hazard spectrum (UHS), thus we have not applied the maximum direction coefficients for this case.

The probabilistic seismic hazard assessment is described under the preceding MCE_R section of this report, with results shown on Figure D-9 and in Table D-2.

13.5 Site-Response Analysis

The following sections describe the site-response analysis at the Tower site performed for the MCE_R spectrum.

13.5.1 Soil Profile Development

The baseline site profile used in the site-response analysis is provided in Table D-5. This profile was developed based on available information from the geotechnical reports, subsurface investigations, laboratory testing, and related documentation for the Tower and nearby surrounding sites, as summarized in the introductory sections of this report. A preliminary analysis was conducted using this baseline site-response profile. Unit weights were selected to be consistent with the analyses described in the Section 9 - Settlement Evaluations, Section 10 - Lateral Earth Pressure Resistance, and Section 11 - Existing Pile Analyses sections of this report. Shear wave velocities were selected based on the V_S profile developed for the project site, described in Section12.3. Shear modulus reduction and damping ratio curves were selected to be consistent with the results of laboratory testing and the dynamic properties in the Kamai et al. (2013) report used to establish the deep shear wave velocity profile. Sensitivity analyses will be performed using alternative subsurface profiles at a future date.

13.5.2 Random Vibration Theory Analysis

An equivalent-linear free-field frequency-domain site-response analysis was conducted to propagate the input bedrock response spectrum to foundation mat level. This approach allows for the use of an input base motion response spectrum, rather than requiring development of time histories at the input base motion level where limited ground motion recordings are available. A suite of ground motion records corresponding to the soft soil conditions at the foundation level was then selected for spectral matching to the foundation-level spectrum. This approach allows for both a deeper site-response profile and more representative time histories for the non-linear structural model (i.e. based on recordings from soft soil sites).

The site-response analysis was performed using the random-vibratory theory (RVT) method in the open-source program Strata (v0.5.9) (Kottke, https://github.com/arkottke/strata). Strata uses RVT to compute the expected maximum site-response of a layered site, subjected to one-dimensional vertical propagation of earthquake ground motions. The input parameters for performing an RVT site-response analysis include: a layered soil system with shear wave velocity, shear modulus reduction and damping values defined by specified strain-dependent relationships, and an input motion represented by a Fourier amplitude spectrum (FAS) and ground motion duration (Rathje et al. 2005). The input FAS is back-calculated from an input acceleration response spectrum using the RVT inversion technique developed by Vucetic & Dobry (1991).

Soil profile properties have been described in the preceding sections of this report. The input acceleration response spectrum for Strata RVT analysis is the MCE_R bedrock spectrum, described previously, and implemented as an outcrop motion. A ground motion duration ($D_{5\text{-}75}$) of 20 seconds, estimated using the deterministic scenario and the prediction equations from Bommer et al. (2009) and Abrahamson and Silva (1996), was input along with the MCE_R spectrum. The analysis was performed with 30 randomized permutations of shear wave velocity and dynamic material properties. The randomization was performed to capture a range of peak responses that are associated with the uncertainty in the soil profiles.

13.5.3 Foundation-Level Response Spectra

The foundation-level MCE_R response spectrum resulting from site response analysis was extracted as a within motion at 25 feet bgs, corresponding to the base of the foundation mat in the structural model. Figure D-14 shows the input bedrock outcrop and resulting 25-foot depth within response spectra.

The same analysis was performed for the SLE, with a duration of 6 seconds. As with the MCE_R motions, the results of the SLE were extracted as within motions at 25 feet bgs. The input bedrock outcrop SLE and the 25-foot within response spectra are shown on Figure D-15 and in Table D-6 for comparison.

13.5.4 Recommended MCE_R Spectrum

ASCE/SEI 7-16 Chapter 21 specifies that for time history development that incorporates site response analysis, the ground surface response spectrum shall not be lower than the input base motion MCE_R response spectrum multiplied by the surface-to-base response spectral ratios calculated at each period from site-response analysis. Comparisons of this spectra and code spectra for a Site Class D indicate that the site response spectrum at foundation level underpredicts the short period spectral ordinates significantly. For design, ASCE/SEI 7-16 Section 21.3 defines the minimum design response spectrum shall be taken as 2/3 of the MCER, but not less than 80% of the design spectrum calculated in accordance with Section 11.4.5. We propose implementing a similar threshold for the MCE_R level where we compare 80% of the Section 11.4.5 MCE_R spectrum with the foundation level site response spectrum and do not go below the 80% threshold. Instead of using the MCE_R defined in Section 11.4.5 from ASCE/SEI 7-16, we recommend using the ASCE/SEI 7-10 provisions that do not include consideration of deep basin effects in the Site Class D site coefficient, F_v. In our opinion, consideration of deep basin effects is not appropriate for the underlying geologic characteristics of the location of this site in San Francisco. Our recommended MCE_R spectrum is presented in Table D-7 and shown on Figure D-16 which is taken as 80% of the ASCE/SEI 7-10 Site Class D spectrum at periods less than 2 seconds and at 7.5 and 10 seconds and as the foundation level site response spectrum at all other periods.

13.6 Development of Acceleration Time Histories

The following sections describe the selection and development of time histories for use in the dynamic structural analysis of the Tower. Acceleration time histories were developed for the MCE_R site response spectrum at foundation level (25 feet bgs).

13.6.1 Development of Time Histories in Accordance with ASCE/SEI 7-16

Seed time histories were selected from the Pacific Earthquake Engineering Research (PEER) Center NGA-West2 Ground Motion Database at their as-recorded orientations. As defined by ASCE/SEI 7-16 Chapter 16, spectral modification shall be performed either by amplitude scaling or spectrally-matching over the period range of interest (i.e., 0.2T to 1.5T). Amplitude scaling should occur such that the average of the maximum-direction spectra from all ground motions in the period range of interest shall not fall below 90% of the target response spectrum for any period within the period range of interest. Similarly defined by ASCE/SEI 7-16 Chapter 16, Section 16.2.3.3, spectral matching shall be performed over the period range of interest, such that the average of the maximum-direction spectra for the suite meets or exceeds 110% of the target spectrum over the period range of interest. As defined by ASCE/SEI 7-16 Chapter 21, Section 21.1.3, time history development that incorporates site response shall result in a ground surface response spectrum that is not lower than the base motion MCE_R response spectrum multiplied by the surface-to-base response spectral ratios obtained from the site response analysis.

For the Tower, the fundamental period of the structure (T) is estimated to be about 5.8 seconds in the model x-direction and about 4.1 seconds in the model y-direction, based on SGH structural evaluations. The lower bound period to achieve at least 90% mass participation is estimated to be 0.4 seconds, based on SGH structural evaluations. The upper bound period is estimated to be 8.7 seconds (based on 1.5T). The resulting period range of interest is estimated to be 0.4 seconds to 8.7 seconds for the SGH structural model.

13.6.2 Selection of Seed Time Histories

Eleven pairs of seed time histories for the target were selected from the PEER NGA-West2 database to roughly match the dominant characteristics of the deterministic scenario. The criteria used for selecting time histories were:

- Magnitude (M_w): 7.0-8.3;
- Rupture distance: 0 to 25 km;
- Style of Faulting: strike-slip, reverse and normal; and
- V_{S30}: 150 to 600 m/s.

Overall spectral shape, duration, and scaling factor were also considered for the selection of the final seed time histories. The eleven pairs of time histories selected for spectral matching to the target spectrum (and properties associated with the seed time histories) are listed in Table D-8.

13.6.3 Spectral Matching

Eleven pairs of two-component horizontal recorded ground motions were spectrally-matched to the target recommended horizontal foundation-level response spectrum. The time histories were spectrally-matched using RSPMatch (2018 release). RSPMatch was originally developed by Abrahamson (1992) as a modification to Lilhanand and Tseng (1988). The approach developed by Lilhanand and Tseng (1988) makes small wavelet adjustments to the time history; those adjustments are used to make non-stationary modifications to the seed time history. The introduction of non-stationary modifications allows for modification of the time history while maintaining non-stationary properties. Following the spectral matching, each time history was baseline corrected. Baseline correction is used to remove any long period drift that might be added to the time history during the spectral matching process. To perform the baseline correction, a polynomial is fit to the acceleration record such that the acceleration, velocity and displacement go to zero at the end of the record. Final response spectra are calculated once spectral matching is complete for each time history component.

The 2018 release of RSPMatch allows for the component to component variability of the two horizontal components to be maintained. This is accomplished within the program by calculating the ratio of each component to the geomean of the two components and multiplying this ratio by the target to achieve a component specific target. The geomean of the two matched components will then be equal to the target. The geomean of the two matched components are plotted with the target spectrum in Figure D-18. The response spectra for all matched time histories and their geomean is compared with the target in Figures D-19 through D-29.

13.6.4 Maximum-Direction Spectra

As described previously, ASCE/SEI 7-16 Chapter 16 specifies that for ground motion modification via amplitude-scaling or spectral matching, the average of the maximum-direction spectra must also be compared with the target spectrum over the period range of interest.

Each pair of spectrally-matched horizontal time histories was rotated to capture the maximum resultant response spectrum for that pair over any direction. The maximum resultant response spectrum for each time history pair was taken as the envelope of the resultant spectra from rotation, where each resultant spectrum was found from the pair of orthogonal horizontal time histories at that rotation increment. The maximum response values are calculated by rotating the time histories in one-degree increments and taking the maximum value for each period. Thus, the maximum response is no longer the product of a single direction, but rather the maximum spectral value for a given period, regardless of direction. The 11 resultant spectra were then scaled such that the geometric mean of the scaled 11 resultant spectra met or exceeded 110% of the target response spectrum, meeting the code-based maximum direction requirement. The suite of ground motions was scaled such that the average of the 11-resultant maximum-direction spectra, described previously, equals or exceeds 110% of the Chapter 21 envelope at each spectral ordinate.

A comparison of the matched scaled maximum component horizontal response spectra with the target spectrum is shown in Figure D-30. Appendices E through F show a comparison between the acceleration, velocity and displacement time histories for the as-recorded (scaled to the target peak ground acceleration) and matched time history records.

13.6.5 Fault-Normal and Fault-Parallel Rotation

ASCE/SEI 7-16 Section 16.2.4 specifies that for near-fault sites (as defined in ASCE/SEI 7-16 Section 11.4.1), horizontal ground motion pairs will be rotated to the fault-normal (FN) and fault-parallel (FP) directions of the causative fault and applied to the building at such orientation. Following matching and maximum-direction scaling, each matched scaled ground motion pair was rotated to FN/FP for the San Andreas fault (azimuth approximately 325 degrees). Preliminary FN/FP time histories were provided to SGH for use in the structural model. The direction of FN/FP for each pair of matched scaled ground motions was defined such that the variability of the direction of maximum component with response to FN/FP was appropriately sampled for a site 15km from a fault following the results of Watson-Lamprey and Boore (2007), rotation angles are included in Table D-9. Based on these results we expect the direction of maximum component with respect to the FN/FP component to be random.

13.7 Development of an Alternative Set of Acceleration Time Histories

The following sections describe the development of alternative time histories for use in the dynamic structural analysis of the Tower. Two retrofit alternatives have been proposed: 1) the SGH PPU option described at the beginning of this section, and 2) a Leslie E. Robertson & Associates (LERA) rock-pile alternative that consists of a large number of additional piles being installed through the foundation mat at locations across the existing Tower footprint and extending into the bedrock stratum at depth. Ground motions and time histories were developed by ENGEO for structural analysis of the LERA retrofit alternative, and the accompanying analysis memorandum (ENGEO, 2018) was provided to both retrofit teams in the interest of producing a viable retrofit solution for the Tower. The ground motion and time history development performed by ENGEO followed a different methodology than the approach described in the previous Ground Motion Development (Sections 12.1-12.7) sections of this report.

At the request of SGH, we compared the target spectrum for time history selection from the ENGEO analysis (Table 9.2-1, ENGEO 2018) with the target spectrum for time history selection from the analysis described previously in this report. This comparison is shown in Figure D-10 and demonstrates that the ENGEO target spectrum is significantly higher at a majority of spectral periods values between 0.01 and 10 seconds, with the 25-foot depth within spectrum exceeding the ENGEO target spectrum by about 10-15% over the periods of 1.8 seconds to 2.3 seconds. Based on the results of this comparison, and for comparison with the earlier analysis conducted by the LERA retrofit team, SGH requested that we perform spectral matching of the ENGEO time history suite to the ENGEO target spectrum, for application in the SGH structural model.

It is unclear from the ENGEO memorandum if ground motions and time histories were developed following ASCE/SEI 7-10 or ASCE/SEI 7-16 guidelines, and how the code-based criteria were incorporated in development of the target spectrum. We are working under the assumption that the target spectrum as presented in Table 9.2-1 (ENGEO 2018) has met all code-based requirements. We note that the ENGEO spectrum was developed for the ground surface, though it is our understanding that the SGH model will apply acceleration time histories at the base of the foundation mat (25 ft bgs). It is our understanding that SGH will apply the ENGEO-based time histories to their structural response model to assess an upper bound envelope of the potential ground shaking. Our time history development process for this exercise follows ASCE/SEI 7-16 Chapter 16, Section 16.2, as described in Section 12.6.3 above. The time histories were spectrally-matched using RSPMatch (2006 release) as they were not modified for the final report. Acceleration time histories were developed for the ENGEO target spectrum following the process outlined below.

13.7.1 Development of Alternative Time Histories in Accordance with ASCE/SEI 7-16

The unscaled seed time history suite selected by ENGEO (Table 9.3-1, ENGEO 2018) was downloaded from the Pacific Earthquake Engineering Research (PEER) Center NGA-West2 Ground Motion Database at the as-recorded orientations. Time history development was performed in accordance with the guidelines of ASCE/SEI 7-16 Chapter 16, as described previously in Section 12.6.3. Based on discussions with SGH, we have performed spectral matching to develop the alternative acceleration time histories for application in the structural model.

For the Tower, the fundamental period of the structure (T) is estimated to be about 5.8 seconds in the model x-direction and about 4.1 seconds in the model y-direction, based on SGH structural evaluations. The lower bound period to achieve at least 90% mass participation is estimated to be 0.4 seconds, based on SGH structural evaluations. The upper bound period is estimated to be 8.7 seconds (based on 1.5T). The resulting period range of interest is estimated to be 0.4 seconds to 8.7 seconds for the SGH structural model. As described in the ENGEO memorandum (Section 9.3, ENGEO 2018) the LERA structural model has a period range of interest of 0.3 seconds at the lower bound (based on 90% modal mass participation) to 9 seconds at the upper bound (based on 2T).

The alternative suite of seed time histories for spectral matching to the ENGEO target spectrum (and properties associated with the seed time histories) are listed in Table D-7.

13.7.2 Down-Sampling

To reduce the runtime of the structural model, select time history records in the ENGEO suite were down-sampled prior to spectral matching. As described in Dabaghi & Der Kiureghian (2014), "Down-sampling is a signal processing procedure that increases the initial sampling time step Δt_o of a signal to $\Delta tf = n\Delta t_o$, where n is usually an integer. Every n points are selected and the points in between are discarded." Down-sampling was performed at a time step of 0.01 seconds on select as-recorded time histories downloaded from the PEER database. Down-sampled time histories were only provided for those records where the process did not result in significant frequency content loss. Table D-8 provides the as-recorded and down-sampled timesteps and record lengths.

13.7.3 Down-Sampling Limitations

Down-sampling may result in the loss of high-frequency content, depending on the original frequency content of the ground motion and the as-recorded and down-sampled time steps. The period range of interest is taken as 0.2T to 1.5T of the structural period (T). Based on discussions with SGH, we understand that periods smaller than 0.1 second will have negligible effect on the response of the structure. Acceleration, velocity, and displacement response spectra were compared for the three components of each record to evaluate how well the truncated, down-sampled time histories approximate the as-recorded time histories. As expected, acceleration response spectra showed some high-frequency content loss and decreased amplitudes as a result of the down-sampling process, primarily in the 0.01-second to 0.1-second period range. Based on discussions with SGH, and comparisons between the spectra, down-sampling to a time step of 0.01 seconds was determined to be satisfactory for the purposes of this assessment. There may be additional ground motion effects or effects arising from interaction with the structural model resulting from the down-sampling process, beyond the response spectra comparisons that were considered.

13.7.4 Spectral Matching

Spectral matching was performed in accordance with the guidelines of ASCE/SEI 7-16 Chapter 16, as described previously in Section 12.6.3. Both horizontal components were spectrally-matched to the same horizontal target (Figure D-10).

13.7.5 Maximum-Direction Spectra

The mean of the 11 maximum-direction resultant spectra was compared with the target response spectrum to confirm that the geometric mean of the scaled 11 resultant spectra meets or exceeds 110% of the target response spectrum, meeting the code-based maximum direction requirement as described previously in Section 12.6.4.

A comparison of the matched maximum component horizontal response spectrum with the target spectrum is shown in Figure D-11. Attachments E through F show a comparison between the acceleration, velocity and displacement time histories for the as-recorded scaled to the target peak ground acceleration and matched time history records.

13.7.6 Fault-Normal and Fault-Parallel Rotation

Each matched ground motion pair was rotated to FN/FP for the San Andreas fault (azimuth approximately 325 degrees), as described previously in Section 12.6.6. A comparison of the FN/FP response spectra with the target spectrum is shown in Figure D-11. FN/FP time histories were provided to SGH for use in the structural model.

Table Number: **D-1**

CODE MAP-BASED DESIGN AND MCE_R LEVEL RESPONSE SPECTRA
Geotechnical Investigation for the Perimeter Pile Upgrade
City and County of San Francisco, California

T (sec)	Spectral Acceleration, S_A (g)	
	Map-Based Design Reponse Spectrum	Map-Based MCE_R Spectrum
0.01	0.436	0.654
0.02	0.512	0.768
0.03	0.588	0.882
0.05	0.740	1.110
0.075	0.900	1.350
0.1	0.900	1.350
0.15	0.900	1.350
0.2	0.900	1.350
0.25	0.900	1.350
0.3	0.900	1.350
0.4	0.800	1.200
0.5	0.640	0.960
0.75	0.427	0.640
1	0.320	0.480
1.5	0.213	0.320
2	0.160	0.240
3	0.107	0.160
4	0.080	0.120
5	0.064	0.096
7.5	0.043	0.064
10	0.032	0.048

Table Number: **D-2**

PROBABILISTIC UNIFORM HAZARD RESPONSE SPECTRA
Geotechnical Investigation for the Perimeter Pile Upgrade
City and County of San Francisco, California

T (sec)	Geomean Spectral Acceleration, S_A (g)		
	2% in 50 Year P_E 2475-year RP	10% in 50 Year P_E 475-year RP	50% in 50 Year P_E 43-year RP
0.01	0.697	0.365	0.062
0.02	0.717	0.374	0.065
0.03	0.827	0.429	0.080
0.05	1.160	0.595	0.113
0.075	1.495	0.760	0.139
0.1	1.627	0.831	0.153
0.15	1.654	0.852	0.160
0.2	1.503	0.772	0.149
0.25	1.327	0.685	0.134
0.3	1.181	0.605	0.121
0.4	0.975	0.499	0.101
0.5	0.825	0.415	0.071
0.75	0.598	0.298	0.037
1	0.458	0.215	0.024
1.5	0.317	0.144	0.013
2	0.243	0.115	0.009
3	0.181	0.071	0.006
4	0.146	0.046	0.004
5	0.123	0.053	0.006
7.5	0.083	0.033	0.003
10	0.056	0.022	0.002

Table Number: **D-3**

DETERMINISTIC RESPONSE SPECTRA
Geotechnical Investigation for the Perimeter Pile Upgrade
City and County of San Francisco, California

T (sec)	Spectral Acceleration, S_A (g)	
	Median M_W 8.25 at 5.5 km	84th Percentile M_W 8.25 at 5.5 km
0.01	0.241	0.435
0.02	0.247	0.447
0.03	0.276	0.506
0.05	0.361	0.680
0.075	0.449	0.866
0.1	0.486	0.942
0.15	0.502	0.958
0.2	0.466	0.877
0.25	0.419	0.783
0.3	0.375	0.705
0.4	0.315	0.597
0.5	0.269	0.517
0.75	0.194	0.382
1	0.151	0.301
1.5	0.109	0.219
2	0.088	0.177
3	0.066	0.133
4	0.055	0.109
5	0.046	0.092
7.5	0.030	0.059
10	0.020	0.039

Table Number: **D-4**

SITE-SPECIFIC RESPONSE SPECTRA FOR NEW BUILDINGS (ASCE/SEI 7-10)
Geotechnical Investigation for the Perimeter Pile Upgrade
City and County of San Francisco, California

T (sec)	Maximum Demand Factors[1]	Risk Coefficient C_R	Spectral Acceleration, S_A (g)					
			Probabilistic MCE_R (Section 21.2.1)	Deterministic Lower Limit (Section 21.2.2)	Deterministic MCE_R (Section 21.2.2)	Site-Specific MCE_R (Section 21.2.3)	80% of Map-Based Design Response Spectra (Section 11.4.5)	Site-Specific Design Response Spectra (Section 21.3)
0.01	1.192	0.934	0.776	0.540	0.540	0.540	0.349	0.360
0.02	1.191	0.934	0.797	0.654	0.654	0.654	0.410	0.436
0.03	1.188	0.934	0.918	0.768	0.768	0.768	0.470	0.512
0.05	1.187	0.934	1.286	0.996	0.996	0.996	0.592	0.664
0.075	1.187	0.934	1.657	1.350	1.350	1.350	0.720	0.900
0.1	1.186	0.934	1.803	1.350	1.350	1.350	0.720	0.900
0.15	1.196	0.934	1.847	1.350	1.350	1.350	0.720	0.900
0.2	1.204	0.934	1.690	1.350	1.350	1.350	0.720	0.900
0.25	1.213	0.933	1.501	1.350	1.350	1.350	0.720	0.900
0.3	1.217	0.932	1.339	1.350	1.350	1.339	0.720	0.893
0.4	1.227	0.929	1.112	1.200	1.200	1.112	0.640	0.741
0.5	1.228	0.927	0.939	0.960	0.960	0.939	0.512	0.626
0.75	1.236	0.921	0.681	0.640	0.640	0.640	0.341	0.427
1	1.239	0.915	0.520	0.480	0.480	0.480	0.256	0.320
1.5	1.236	0.915	0.359	0.320	0.320	0.320	0.171	0.213
2	1.24	0.915	0.276	0.240	0.240	0.240	0.128	0.160
3	1.247	0.915	0.206	0.160	0.166	0.166	0.085	0.111
4	1.257	0.915	0.168	0.120	0.137	0.137	0.064	0.091
5	1.264	0.915	0.142	0.096	0.116	0.116	0.051	0.077
7.5	1.284	0.915	0.097	0.064	0.076	0.076	0.034	0.051
10	1.29	0.915	0.067	0.048	0.051	0.051	0.026	0.034

Table Number: **D-5**

SITE-RESPONSE ANALYSIS - SOIL PROFILE PROPERTIES
Geotechnical Investigation for the Perimeter Pile Upgrade
City and County of San Francisco, California

Soil Type	Layer Thickness (ft)	Unit Weight (pcf)	Dynamic Properties		
			Shear Wave Velocity, V_s (ft/s)	G/G_{max} Curve	Damping Ratio Curve
Fill / Dune Sand	20	105	350	Vucetic & Dobry, PI = 30	Vucetic & Dobry, PI = 30
Upper (Young) Bay Mud	16	129	450	Seed & Idriss, Sand Mean	Seed & Idriss, Sand Mean
Upper Marine Sand	28	105	700	Vucetic & Dobry, PI = 15	Vucetic & Dobry, PI = 15
Lower (Young) Bay Mud	12	130	500	Seed & Idriss, Sand Mean	Seed & Idriss, Sand Mean
Lower Marine Sand	12	125	750	Vucetic & Dobry, PI = 30	Vucetic & Dobry, PI = 30
Old Bay Clay 1 - Crust	12	114	900	Vucetic & Dobry, PI = 30	Vucetic & Dobry, PI = 30
Old Bay Clay 1	68	125	700	Vucetic & Dobry, PI = 30	Vucetic & Dobry, PI = 30
Old Bay Clay 2 - Crust	12	123	1200	Vucetic & Dobry, PI = 30	Vucetic & Dobry, PI = 30
Old Bay Clay 2	60	148	1000	Vucetic & Dobry, PI = 50	Vucetic & Dobry, PI = 50
Rock 1	60	148	2500	Vucetic & Dobry, PI = 100	Vucetic & Dobry, PI = 100
Rock 2	108	148	3281	Vucetic & Dobry, PI = 200	Vucetic & Dobry, PI = 200
Rock 3	42	120	3904	Seed & Idriss, Sand Mean	Seed & Idriss, Sand Mean
Half-Space	---	---	3936	---	---

Table Number: **D-6**

SITE RESPONSE ANALYSIS FOR 43-YEAR RETURN PERIOD
Geotechnical Investigation for the Perimeter Pile Upgrade
City and County of San Francisco, California

T (sec)	Geomean Spectral Acceleration, S_A (g)	
	50% in 50 Year PE 43-year RP	Site-Specific
0.01	0.062	0.082
0.02	0.065	0.085
0.03	0.080	0.104
0.05	0.113	0.147
0.075	0.139	0.180
0.1	0.153	0.196
0.15	0.160	0.207
0.2	0.149	0.188
0.25	0.134	0.168
0.3	0.121	0.152
0.4	0.101	0.135
0.5	0.071	0.097
0.75	0.037	0.051
1	0.024	0.042
1.5	0.013	0.024
2	0.009	0.013
3	0.006	0.007
4	0.004	0.005
5	0.006	0.007
7.5	0.003	0.004
10	0.002	0.002

Table Number: **D-7**

RESPONSE SPECTRA COMPARISON
Geotechnical Investigation for the Perimeter Pile Upgrade
City and County of San Francisco, California

T (sec)	Spectral Acceleration, S_A (g)			
	Site Class D Map-Based MCE_R Spectrum	80% of the MCE_R Spectrum	Foundation Level MCE_R	Recommended MCE_R
0.01	0.675	0.540	0.179	0.540
0.02	0.750	0.600	0.192	0.600
0.03	0.825	0.660	0.190	0.660
0.05	0.975	0.780	0.189	0.780
0.075	1.163	0.930	0.192	0.930
0.1	1.350	1.080	0.196	1.080
0.15	1.500	1.200	0.220	1.200
0.2	1.500	1.200	0.269	1.200
0.25	1.500	1.200	0.342	1.200
0.3	1.500	1.200	0.385	1.200
0.4	1.500	1.200	0.431	1.200
0.5	1.500	1.200	0.406	1.200
0.75	1.200	0.960	0.383	0.960
1	0.900	0.720	0.356	0.720
1.5	0.600	0.480	0.349	0.480
2	0.450	0.360	0.372	0.372
3	0.300	0.240	0.259	0.259
4	0.225	0.180	0.187	0.187
5	0.180	0.144	0.138	0.144
7.5	0.120	0.096	0.094	0.096
10	0.090	0.072	0.068	0.072

Table Number: **D-8**

SEED TIME HISTORIES SELECTED FOR SCALING
Geotechnical Investigation for the Perimeter Pile Upgrade
City and County of San Francisco, California

PEER Record Number	Tp-Pulse Period (sec)	5-75% Duration (sec)	Arias Intensity (m/s)	Earthquake Name	Year	Station Name	Magnitude	Mechanism	R_{jb} (km)	R_{rup} (km)	V_{S30} (m/s)
173	4.5	5.3	0.7	Imperial Valley-06	1979	El Centro Array #10	6.5	Strike-slip	8.6	8.6	203
192	-	14	0.1	Imperial Valley-06	1979	Westmorland Fire Sta	6.5	Strike-slip	14.8	15.3	194
900	7.5	10.9	0.9	Landers	1992	Yermo Fire Station	7.3	Strike-slip	23.6	23.6	354
1155	-	25.8	0.5	Kocaeli Turkey	1999	Bursa Tofas	7.5	Strike-slip	60.4	60.4	290
1158	-	6.1	1.3	Kocaeli Turkey	1999	Duzce	7.5	Strike-slip	13.6	15.4	282
1176	4.9	7	1.3	Kocaeli Turkey	1999	Yarimca	7.5	Strike-slip	1.4	4.8	297
1193	6.7	13.8	1.8	Chi-Chi Taiwan	1999	CHY024	7.6	Reverse-oblique	9.6	9.6	428
1244	5.3	13.5	3	Chi-Chi Taiwan	1999	CHY101	7.6	Reverse-oblique	9.9	9.9	259
1476	5.3	16.5	0.8	Chi-Chi Taiwan	1999	TCU029	7.6	Reverse-oblique	28.0	28.0	407
2107	-	12.6	0.2	Denali Alaska	2002	Carlo (Temp.)	7.9	Strike-slip	49.9	50.9	399
6887	12.6	11	0.9	Darfield New Zealand	2010	Christchurch Botanical Gardens	7.0	Strike-slip	18.1	18.1	187

Table Number: **D-9**

TIME HISTORY ROTATION
Geotechnical Investigation for the Perimeter Pile Upgrade
City and County of San Francisco, California

PEER Record Number	Earthquake Name	Station Name	First Component (degrees[1])	Fault Normal (degrees[1])	Fault Parallel (degrees[1])
173	Imperial Valley-06	El Centro Array #10	50	69	159
192	Imperial Valley-06	Westmorland Fire Sta	90	91	181
900	Landers	Yermo Fire Station	270	129	219
1155	Kocaeli Turkey	Bursa Tofas	0	146	236
1158	Kocaeli Turkey	Duzce	180	162	252
1176	Kocaeli Turkey	Yarimca	60	179	269
1193	Chi-Chi Taiwan	CHY024	90	195	285
1244	Chi-Chi Taiwan	CHY101	90	211	301
1476	Chi-Chi Taiwan	TCU029	90	228	318
2107	Denali Alaska	Carlo (Temp.)	90	90	180
6887	Darfield New Zealand	Christchurch Botanical Gardens	271	260	350

Note: 1) Degrees with respect to North.

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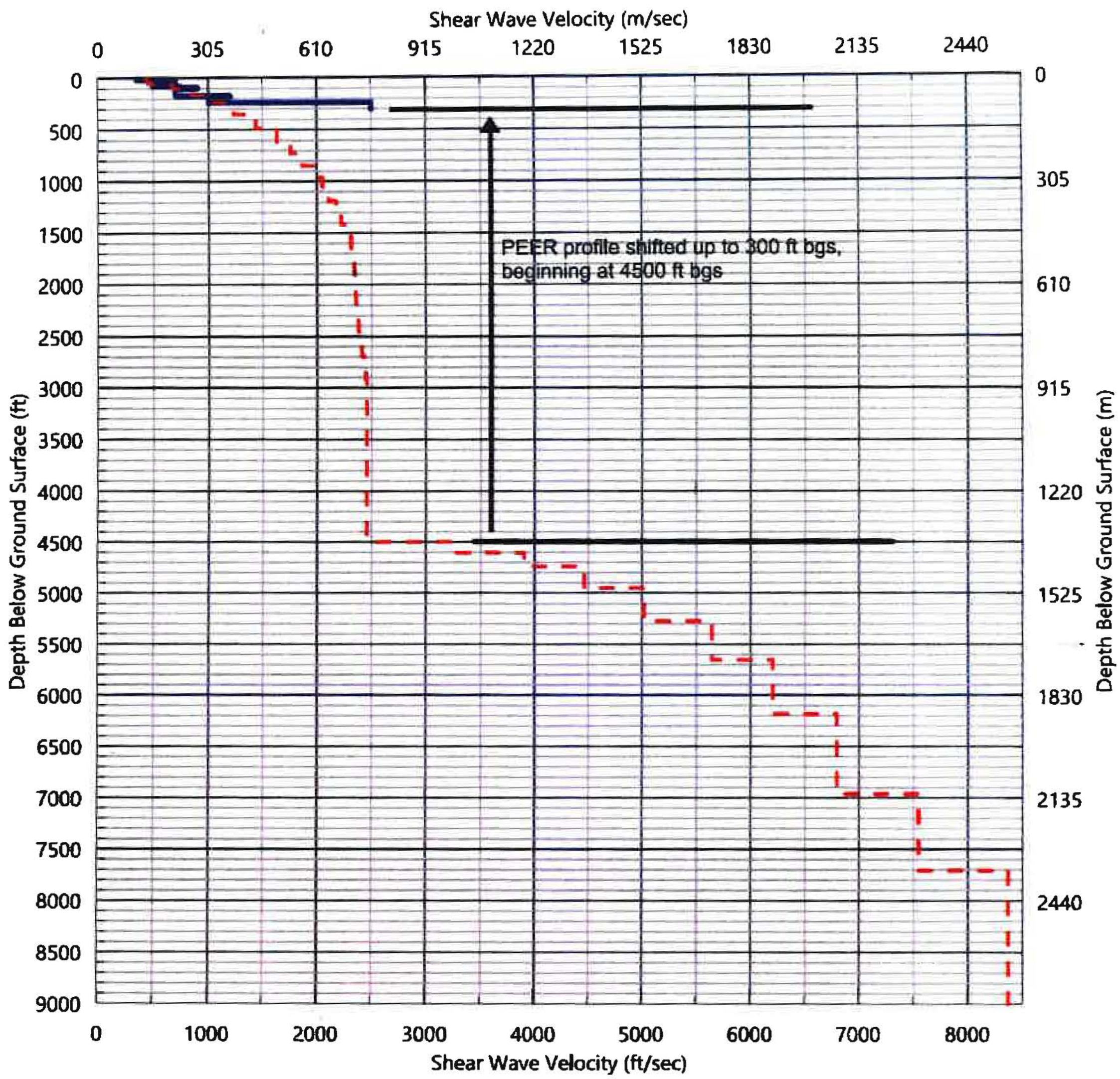

PEER V_S Profile ("BayMud3", $Z_{1.0}$ = 4500 ft)

Transbay Transit Center V_S Profile

Note:

1. PEER V_S profile from Kamai et al. 2013 (PEER Report 2013/12).
2. TTC V_S profile from ARUP 2010 (Plate 13).

Project No: 18-001.00	Date: 11/29/2018	Created By: DAM	Checked By: --	Figure No: **D-1**

SLATE GEOTECHNICAL CONSULTANTS

SHEAR WAVE VELOCITY PROFILES
PEER DEEP PROFILE & TTC NEAR-SURFACE PROFILE

Geotechnical Investigation for the Perimeter Pile Upgrade
City and County of San Francisco, California

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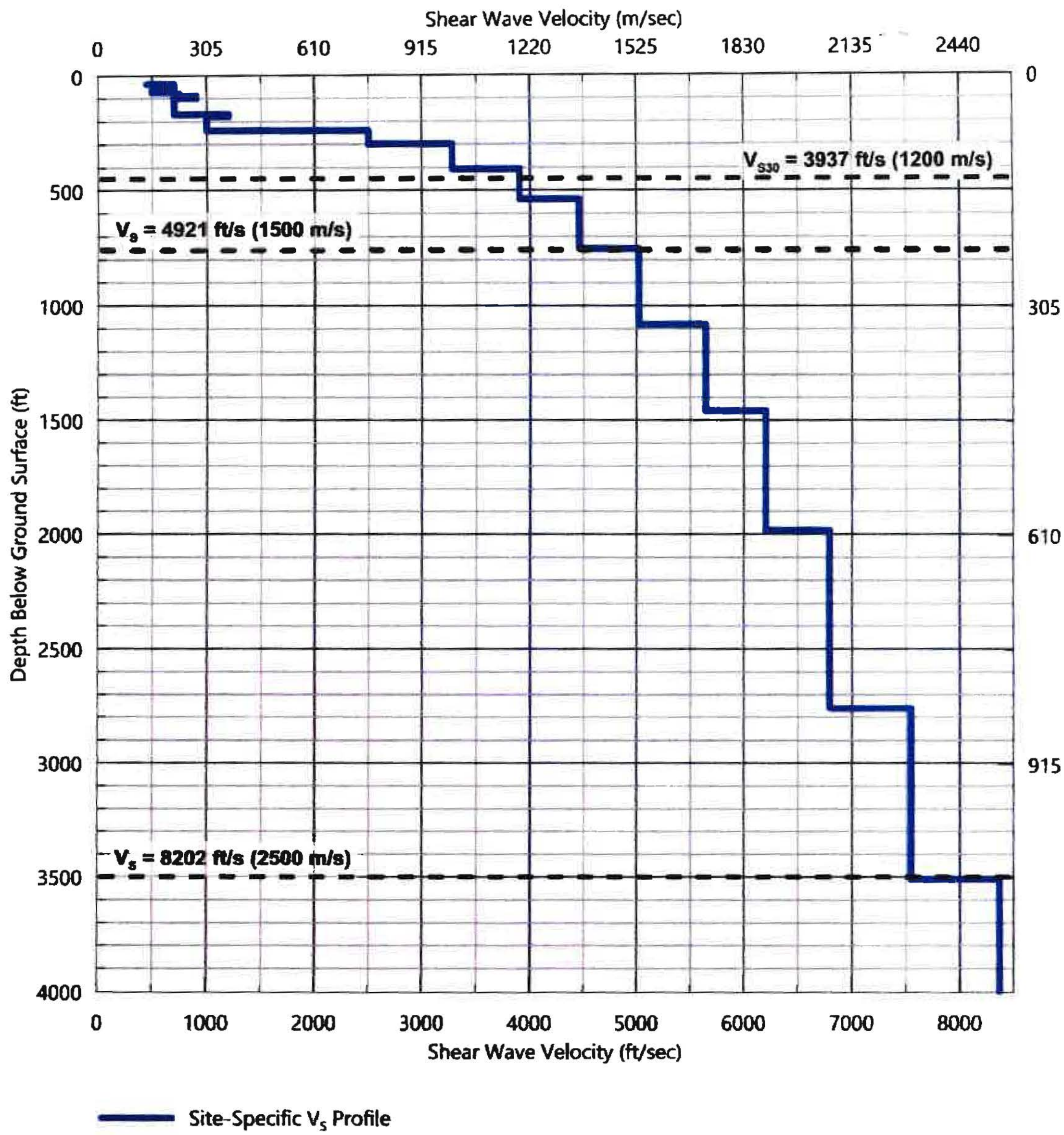

Project No: 18-001.00	Date: 11/29/2018	Created By: DAM	Checked By: --	Figure No: **D-2**

SLATE GEOTECHNICAL CONSULTANTS

SHEAR WAVE VELOCITY PROFILES
SITE SPECIFIC PROFILE DEVELOPED FOR ANALYSIS

Geotechnical Investigation for the Perimeter Pile Upgrade
City and County of San Francisco, California

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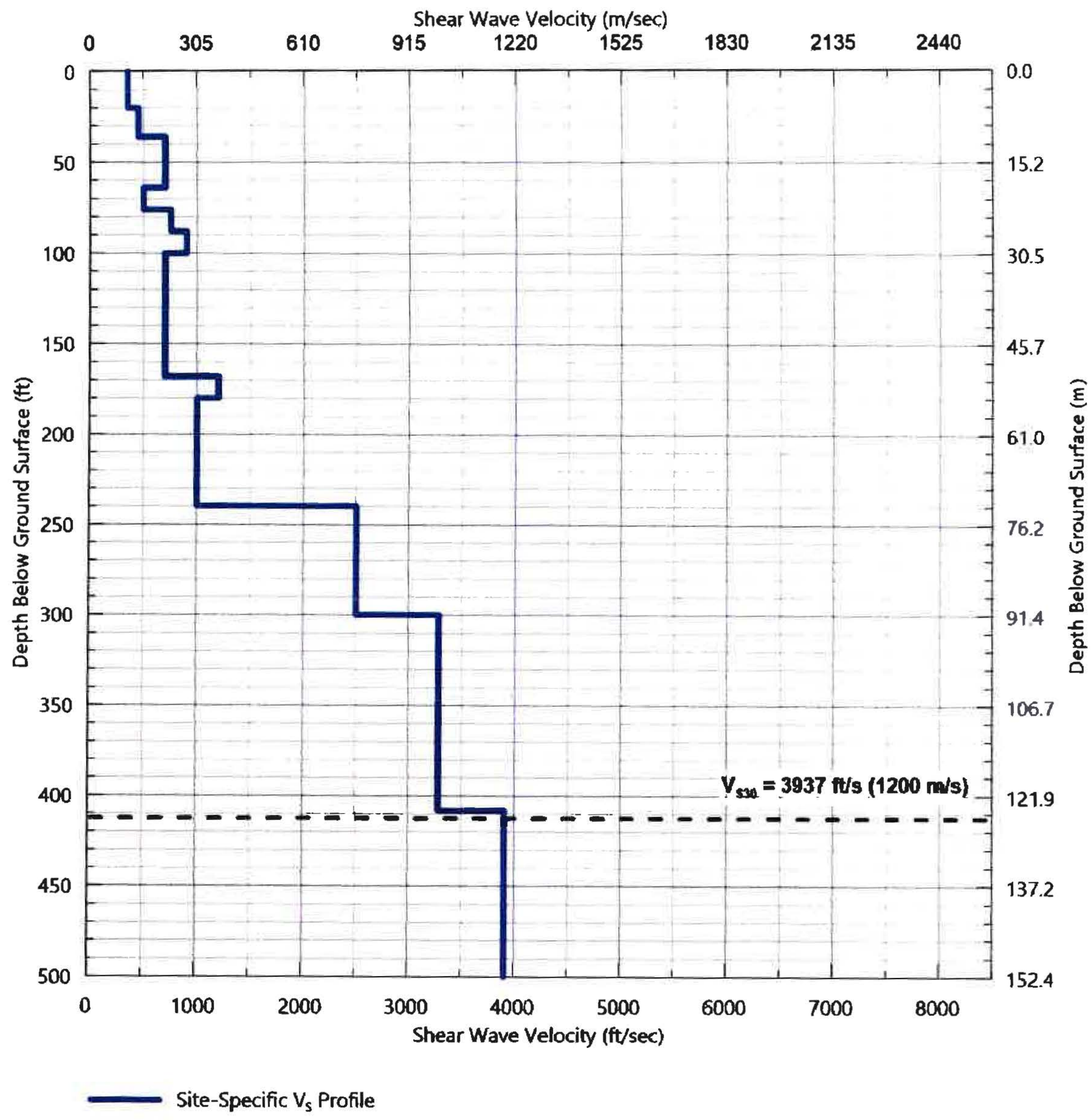

Project No: 18-001.00	Date: 11/29/2018	Created By: DAM	Checked By: --	Figure No: **D-3**

SLATE GEOTECHNICAL CONSULTANTS

SHEAR WAVE VELOCITY PROFILES
SITE SPECIFIC PROFILE DEVELOPED FOR ANALYSIS (0-500 FT DEPTH)

Geotechnical Investigation for the Perimeter Pile Upgrade
City and County of San Francisco, California

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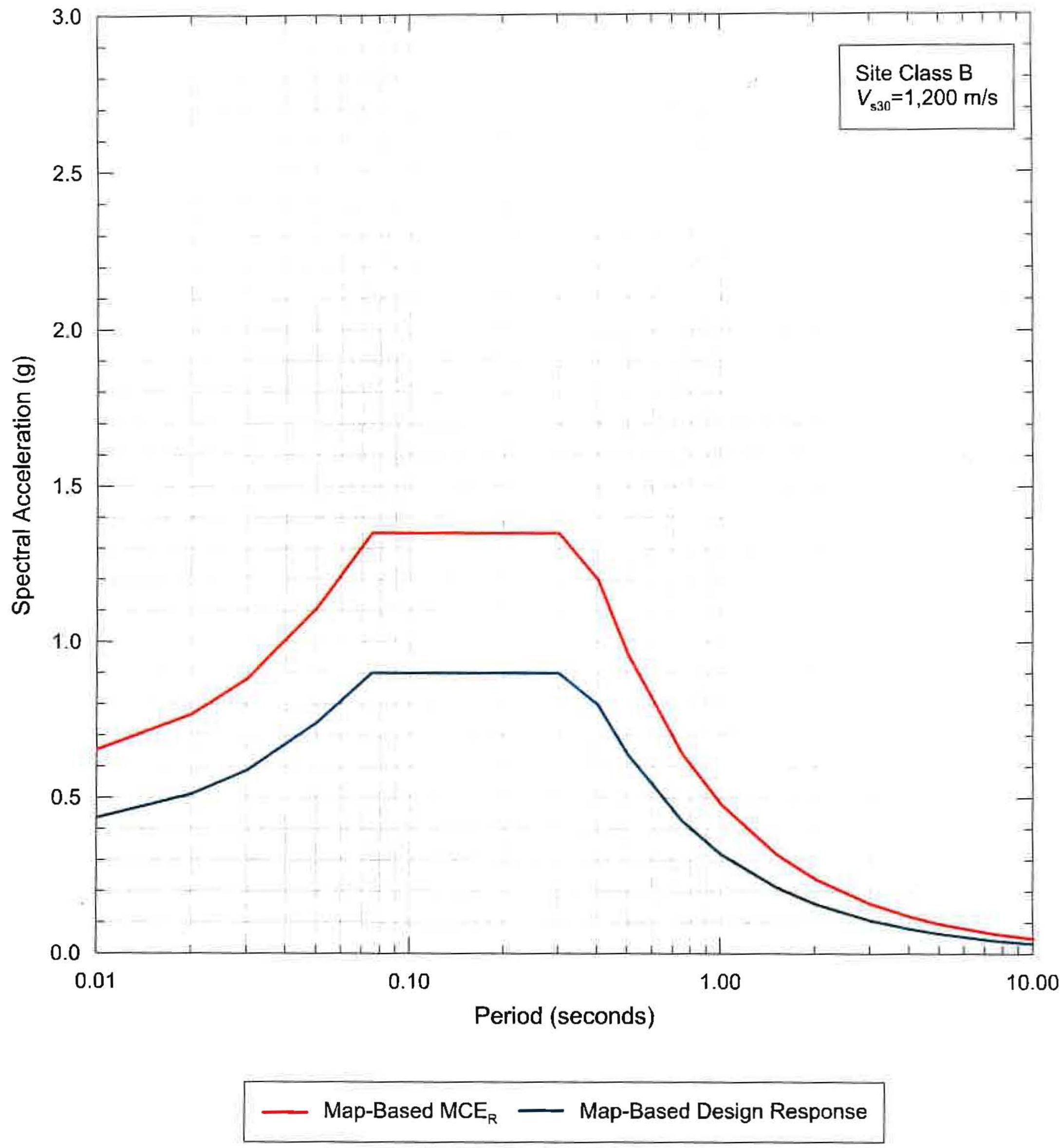

Note:
Spectra are damped 5%, except PGA

Project No: 18-001.00	Date: 11/29/2018	Created By: DAM	Checked By: --	Figure No: **D-4**

SLATE GEOTECHNICAL CONSULTANTS

CODE MAP-BASED DESIGN AND MCER LEVEL RESPONSE SPECTRA
DEVELOPED FOR INPUT BASE MOTION AT BEDROCK LEVEL

Geotechnical Investigation for the Perimeter Pile Upgrade
City and County of San Francisco, California

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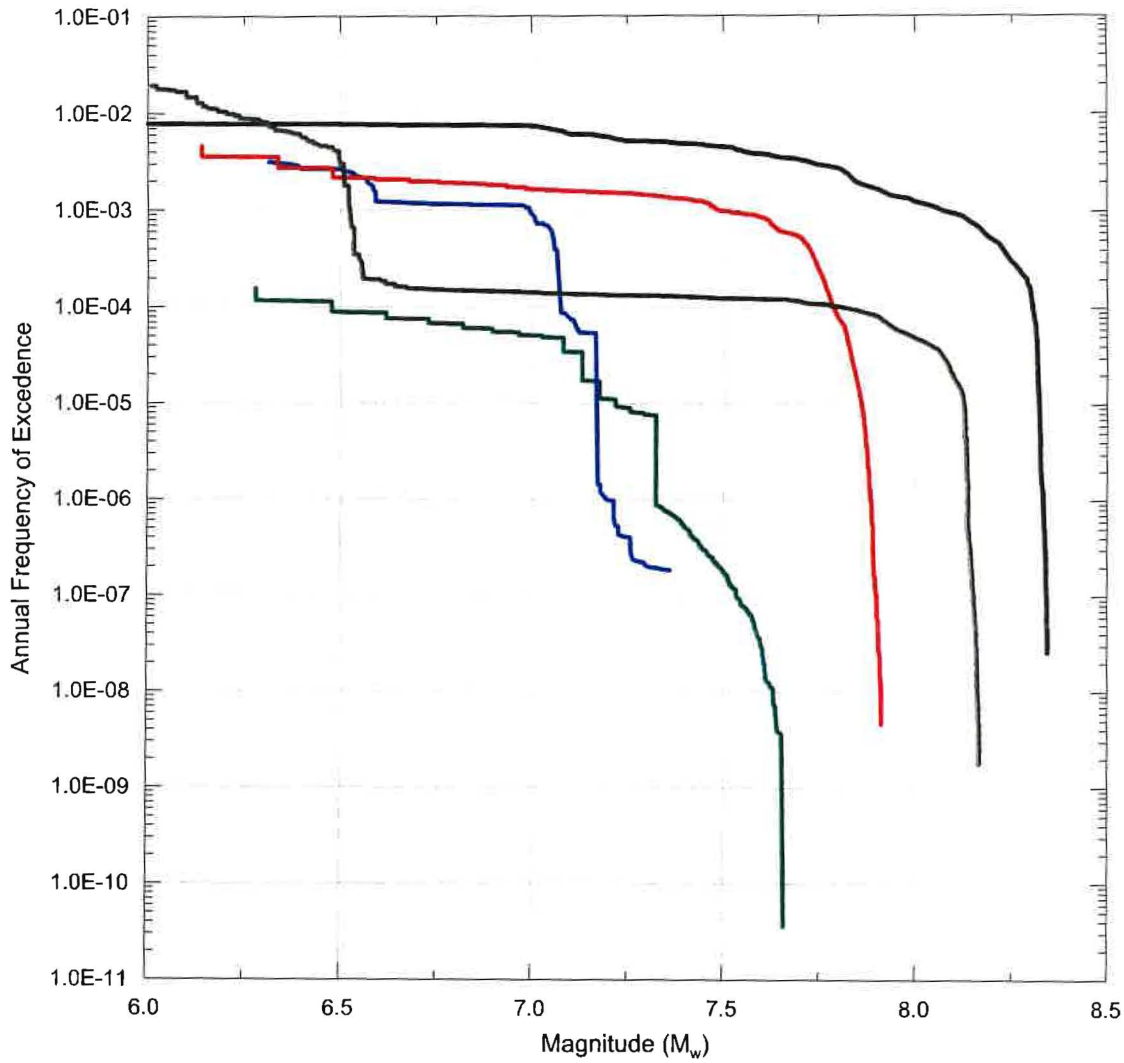
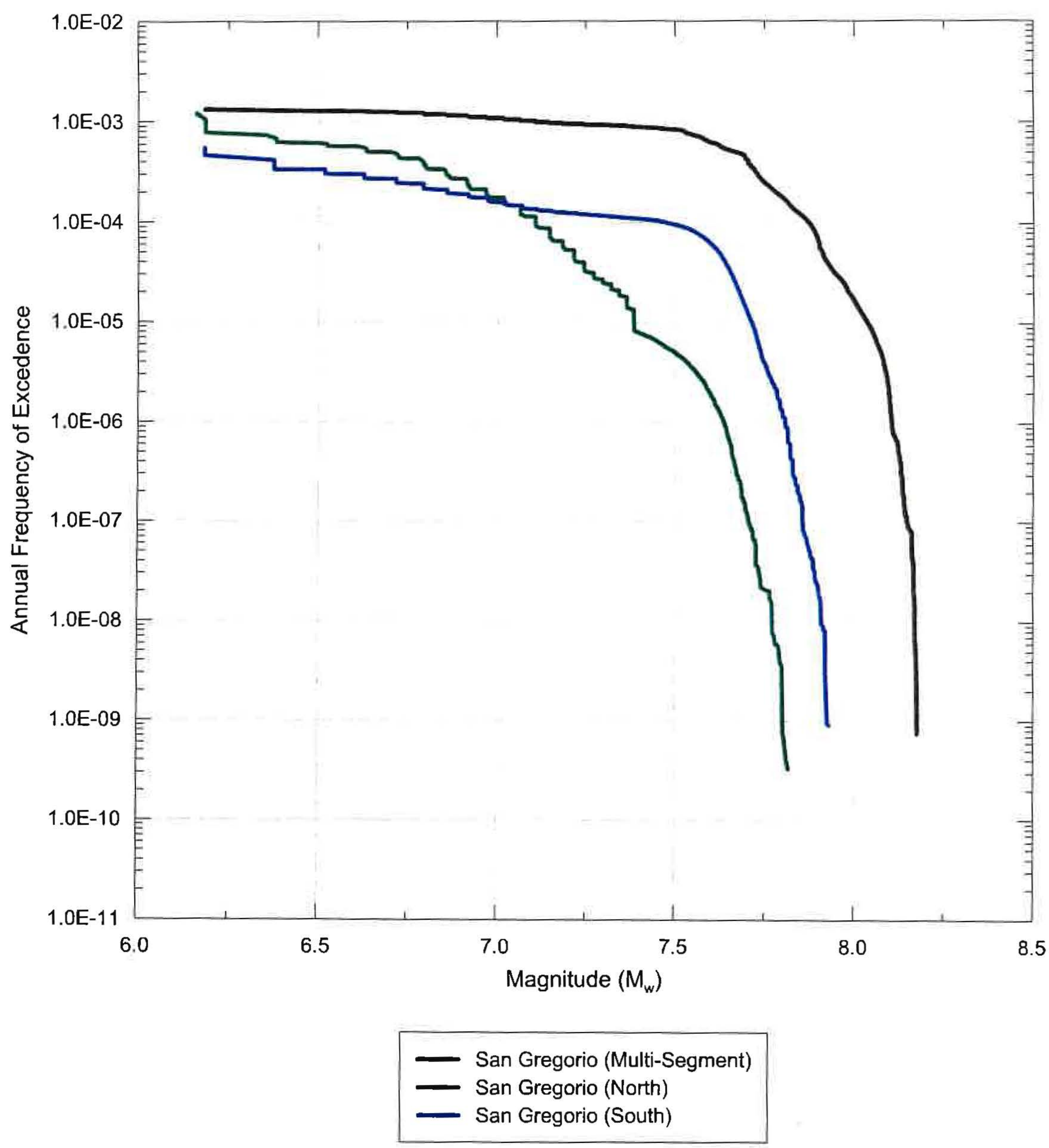


Project No: 18-001.00	Date: 11/29/2018	Created By: MLS	Checked By: DGM	Figure No: **D-5**

SLATE GEOTECHNICAL CONSULTANTS

RECURRENCE RATES FOR EARTHQUAKES ON THE SAN ANDREAS FAULT

Geotechnical Investigation for the Perimeter Pile Upgrade
City and County of San Francisco, California



Project No: 18-001.00	Date: 11/29/2018	Created By: MLS	Checked By: --	Figure No: **D-6**

SLATE GEOTECHNICAL CONSULTANTS

RECURRENCE RATES FOR EARTHQUAKES ON THE SAN GREGORIO

Geotechnical Investigation for the Perimeter Pile Upgrade
City and County of San Francisco, California

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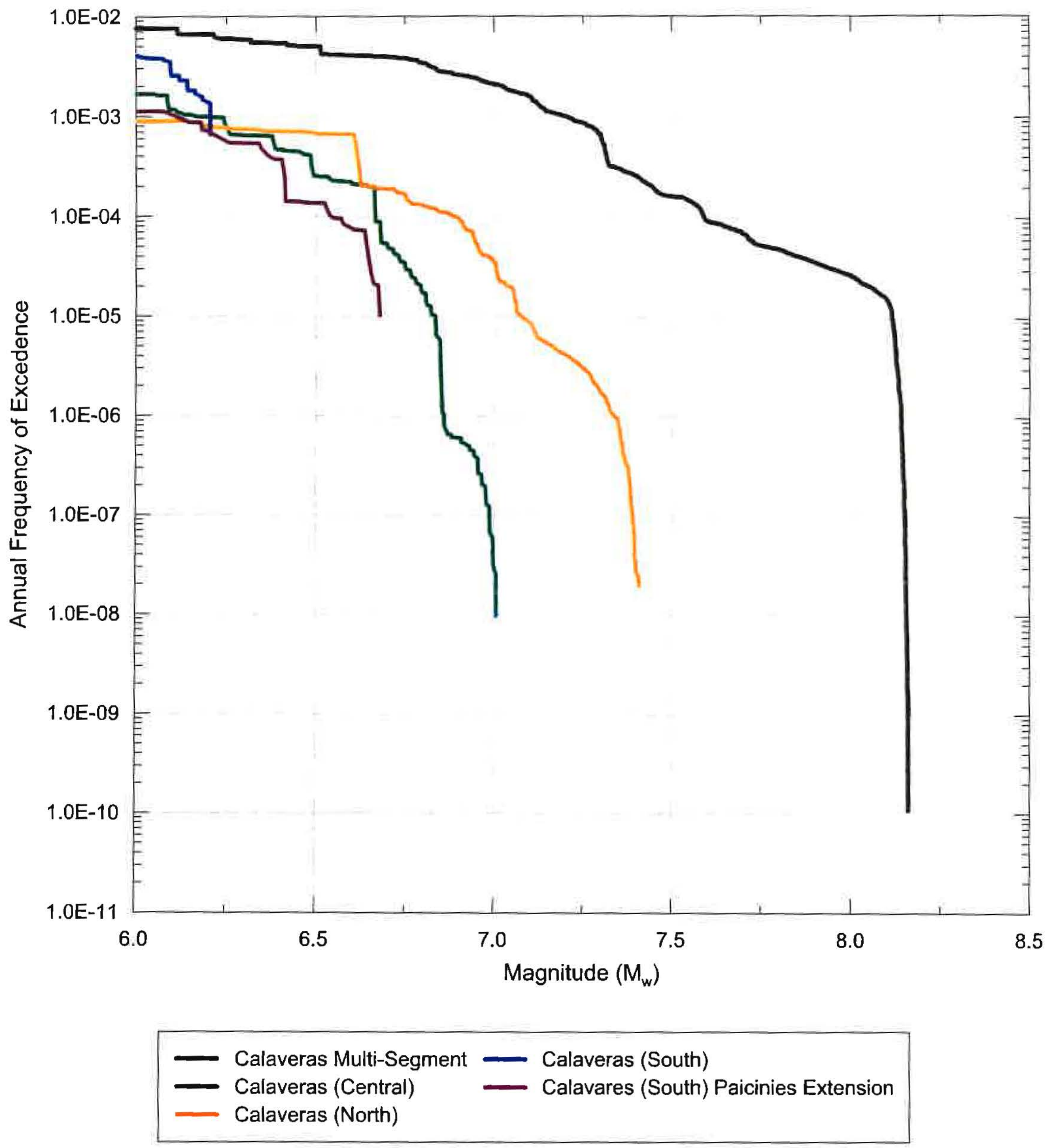

Project No: 18-001.00	Date: 11/29/2018	Created By: MLS	Checked By: --	Figure No: **D-7**

SLATE GEOTECHNICAL CONSULTANTS

RECURRENCE RATES FOR EARTHQUAKES ON THE CALAVERAS FAULT

Geotechnical Investigation for the Perimeter Pile Upgrade
City and County of San Francisco, California

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From:	Gregory Deierlein
To:	DBI-RonaldO.Hamburger; DBI-MarkoSchotanus; DBI-CraigS.Shields; "Shahriar Vahdani"; Tam, Richard (DBI); "Lachezar Handzhiyski"; DBI-StephenK.Harris; "John Egan"; "Debra Murphy"
Subject:	301 Mission - Ground Motions
Date:	Tuesday, February 12, 2019 1:24:23 PM

Dear Ron, Debra, John (and others),

This is to confirm that the EDRT has reviewed the ground motion report and agrees that the proposed MCE target spectrum and approach to selecting, scaling and processing of input ground motions conforms with the agreed upon basis of design. We do, however, have a few points that should be addressed before finalizing the report and ground motions:

1) Target Earthquake Ground Motion Duration - Please confirm the statistical basis of the target ground motion duration of D5-75=20 seconds (Section 13.5.2) and that it is consistent with the deterministic earthquake hazard (84th percentile of M8 on nearby segment of SF fault). We would also request a comparison of duration estimates from the two cited references (Bommer et al. 2009, Abrahamson and Silva 1996) and a more recent model by Afshari & Stewart (2016).
2) Duration of Proposed Ground Motions – Per Table D-8, there is only one ground motion with a significant duration above 20 sec. To the extent that EQ duration may influence degradation and deformation demand in the foundation components, we suggest that one or two of the proposed motions should be replaced with durations of longer duration. We would, for example, suggest replacing the 1158 record, which is one of the shorter motions. Replacing this by a ground motion from another earthquake would also reduce oversampling of motions from the Kocaelli earthquake.
3) Reference to ENGEO/LERA study - To the extent that the ground motion report relies on the LERA/ENGEO study referenced in Section 13.7, the EDRT needs to have access to review that report. Otherwise, if the proposed ground motions do not rely on the LERA/ENGEO study, please removed this discussion from the report. Also – we don't see the Figure D-10 (referenced in Section 13.7) in the report.
4) Controlling spectrum (Section 13.4.1.7) – It is stated that probabilistic ground motion is controlling at all periods. This is not correct.
5) Dynamic Soil Properties (Table 5) – Vucetic & Dobry degradation curves were used for sandy soil layers and Seed & Idriss degradation curves were used for clayey soils including Young Bay Mud. This may be a simple typo in the table as the reverse would be correct.
6) Number of simulations in Site Response Analysis – During the meeting of 2/6/2019 it was stated that 130 simulations were used. However, the report states 30 simulation were used. Please confirm which is correct.

We will add these questions to the next update to the comment log, but in the interest of expediting things, we are sending them now. Let us know if you have any questions or see the need to discuss.

Regards,

Greg Deierlein (EDRT chair)