TECHNICAL MEMORANDUM

To: Mr. Vision Winter, Esq.
O'Melveny & Myers LLP

Date: September 20, 2018

Project No.: 13553.001.000

Project Name: Millennium Tower
301 Mission Street
San Francisco, California

Subject: **GEOTECHNICAL MEMORANDUM – 301 MISSION RETROFIT DESIGN**

This memorandum provides a summary of the geotechnical work performed to date in support of the proposed structural retrofit of the Millennium Tower at 301 Mission Street, San Francisco, California. This memorandum provides a summary of the field exploration performed at Beale Street, the geology of the site, bedrock strength, seismic hazard analyses, existing and proposed pile analyses, and settlement analyses as they relate to the retrofit design. For background information on the tower's current condition and retrofit design approach, please refer to LERA's September 18, 2018, report.

1.0 BEALE STREET FIELD EXPLORATION

Our field exploration included drilling one boring (MPTB-1) at a location along Beale Street in the sidewalk adjacent to the mid-rise structure at the project site. The exploration was performed between January 18 and 26, 2018. ENGEO engineering geologists observed the drilling and logged the subsurface conditions encountered. The borings were drilled using rotary-wash methods in the soils and diamond-bit coring methods where bedrock was encountered. The boring was advanced to a depth of 300 feet below existing grade. The boring was permitted and backfilled in accordance with the requirements of the City and County of San Francisco Department of Health.

Disturbed and relatively undisturbed soil samples were retrieved at various intervals in the boring using standard penetration tests, a 2½-inch I.D. "California Modified" sampler, and a piston sampler advancing 3-inch Shelby tubes. The blow counts from the driven samplers were obtained by dropping a 140-pound hammer through a 30-inch free fall. The 2-inch O.D. split-spoon and 2.5-inch I.D. samplers were driven 18 inches and the number of blows was recorded for each 6 inches of penetration. Unless otherwise indicated, the blows per foot recorded on the boring log represent the accumulated number of blows to drive the last 1 foot of penetration. The blow counts have not been converted using any correction factors. When sampler driving was difficult, penetration was recorded only as inches penetrated for 50 hammer blows. The relatively undisturbed Shelby tube samples were pushed 32 inches, or less if stiff soil conditions were encountered.

The draft report log is shown in Appendix A. The logs depict subsurface conditions at the exploration locations for the date of exploration; however, subsurface conditions may vary with time.

2.0 GEOLOGY AND SUBSURFACE CONDITIONS

In the following sections, we summarize the site subsurface conditions. These descriptions are based on review of boring logs and CPT soundings from previous explorations at or near the project site, as well as information obtained during our subsurface investigation in January 2018.

2.1 EXISTING FILL AND SURFICIAL DEPOSITS (QF)

The project site is located within an area of reclaimed land and is entirely underlain by artificial fill. This fill consists of poorly graded, loose sands, gravels and debris, which includes concrete, wood, and other historic materials. As a result, its properties are inherently different than fill placed by engineered methods. Depths of fill range from approximately 15 to 25 feet at the site.

2.2 YOUNG BAY MUD (QYBM) AND MARINE SAND DEPOSITS (QMS)

The Holocene materials at the site include estuarine clays and sands deposited since the end of the last global glaciation. The clays are locally known as the Young Bay Mud (YBM), which consists of predominantly plastic clay with minor layers of lean to sandy clay, silt to clayey silt, and clayey sand, with some peat interbeds and lenses. The YBM typically is greenish gray to blue gray, very soft to medium stiff, and contains abundant shell fragments. The YBM generally is normally consolidated and moderately to highly compressible.

In the project vicinity, the YBM is interbedded with layers of sands, likely due to fluctuations in sea level during the last approximately 10,000 years. The YBM has been subdivided into Upper and Lower units. The sand between and below the YBM is composed of greenish gray to blue gray, fine- to medium-grained, poorly graded, medium dense to very dense, clean sand to clayey sand. The sand has lenses of stiff to very stiff lean clay and contains some shell fragments. Locally, these sand deposits may include sands known as the Colma Formation. The combined Holocene layers vary in depth from approximately 45 to 70 feet at the project site.

2.3 PLEISTOCENE SAND (QOS) AND OLD BAY CLAY (QOBC) DEPOSITS

Beneath the YBM and Marine Sands are early-Holocene to late-Pleistocene sands likely deposited during the last global glacial epoch, possibly as eolian deposits. These sands are composed of olive to yellowish brown, fine- to medium-grained, poorly graded, medium dense to very dense, clean sand to clayey sand. These sands are sometimes referred to as the Colma Formation.

At the base of the Pleistocene sands is a thick accumulation of estuarine to marine clay known locally as the Old Bay Clay. This unit was deposited during the last global interglacial period when a paleo-San Francisco Bay was present, likely with sea level much higher than current levels. Old Bay Clay is composed of greenish gray to blue gray, stiff to very stiff, silty to sandy clay and fat clay, and varies from 70 to 120 feet thick at the project site.

2.4 ALAMEDA FORMATION (QAF)

Below the thick layer of Old Bay Clay are interfingered layers of sand and clays extending to the bedrock surface, locally called the Alameda formation. These deposits may reflect cycles of glacial and interglacial deposition prior to the previous interglacial period. These deposits consist primarily of clay to sandy clay, sandy silt, clayey to silty sand, clean sand, and silty gravel and are from 45 to 60 feet thick at the project site.

2.5 FRANCISCAN COMPLEX (KJF)

At the project site, Franciscan Complex is encountered in boreholes beneath Quaternary deposits. The Franciscan Complex is a mixed complex of lithologically distinct rock types of exotic origin, including low-grade metamorphosed greenschist facies, chloritized greywacke sandstone, and minor serpentinite that are tectonically emplaced together via subduction-zone tectonics over a time of 200 to 50 million years ago. Bedrock recovered in cores is Hunter's Point Shear Zone mélange, consisting of pervasively sheared to pulverized rock blocks within a clayey gouge matrix. The blocks encased in matrix range from a few inches to approximately 5 feet in size. The blocks are predominately gray, calcite-veined medium- to very-fine-grained meta-siltstone to meta-shale. In the Beale Street boring (MPTB-1), approximately 5% of recovered blocks were ultramafic serpentine. The top of the Franciscan Complex bedrock was encountered at depths that range between 220 and 240 feet. A discussion of the rock strength is contained in the following sections.

3.0 BEDROCK STRENGTH

As discussed above, Franciscan Complex is a mixture of lithologically distinct rock types of exotic origin which contains shear zones, such as the Hunter's Point Shear Zone that underlies the site. The rock encountered in the Beale Street boring was Franciscan Complex mélange, consisting of pervasively sheared to pulverized rock blocks within a clayey gouge matrix. The blocks varied in size from several inches to several feet with varying amounts of fat clay matrix encasing the blocks. This type of material is referred to as block-in-matrix rock (bimrock), with a rock strength that is variable and discontinuous. The strength of bimrocks is generally dependent on the relative percentage of block to matrix, as well as the size and competence of the blocks. Point-load testing on blocks and matrix is currently being conducted to provide insights and a comparison to similar work performed around the site.

In addition to our subsurface investigation, rock coring and laboratory testing performed by Cotton, Shires and Associates (CSA) at the project site in December 2016 was reviewed. In addition, Arup's 2013 geotechnical data report (GDR) and geotechnical interpretive report (GIR) for the Salesforce Tower project located across Fremont Street from the project site was also reviewed. Because of the complex and chaotic nature of the Franciscan Complex, rock types and strength can vary greatly over a short distance, both within a corehole and between coreholes. To assess the relative rock strength of the bedrock at the site, the CSA and Arup investigations were compared to the current exploration (MPTB-1) through visual and descriptive techniques as well as selective laboratory testing of representative rock units present in the cores.

Based on visual comparison of the cores from the MPTB-1, the CSA exploration and the Arup exploration had similar materials and similar recovery. However, in the Arup boring logs, a high Rock Quality Index (RQD) appears to be applied to the rock cores. According to the Caltrans Soil and Rock Logging, Classification, and Presentation Manual (2010), only pieces of "sound intact rock equal to or greater than 4 inches long" should be counted for RQD, and "sound rock" generally cannot be hand broken. Arup's high RQDs were assigned for rock that appears to have fractures, veins, and clay infill and are described in logs as pervasively sheared and/or fractured. Thus, the RQDs calculated in MPTB-1 and CSA's corelog should be more heavily weighted when considering relative rock strength.

Materials encountered in the coreholes can generally be divided into three groups: fractured graywacke, sheared shale, and mélange matrix, as described by Arup (2013). In MPTB-1, one block of ultramafic serpentinite was encountered, which may be expected due to the chaotic and mixed nature of the Franciscan complex. Based on visual classification of MPTB-1 and CSA's borings, fractured graywacke varies from 0 to 14% of recovered core, sheared shale from 5 to 40% of recovered core, and the remaining core consists of mélange matrix or no recovery. Recovery of core varied from 0 to 100% due to the chaotic nature of the mélange. In many cases, the 100% recovery zones were due to high concentrations of clay matrix and not segments of continuous rock. In MPTB-1, all RQDs had a '0' value. In the CSA coreholes, the RQD was generally 0 with occasional RQDs up to 60. These relative percentages of block versus matrix can be correlated to rock strength, where the larger and less fractured blocks will behave more like a fractured block mass and where pervasive clayey matrix will behave like a sheared clay gouge.

In addition to visual classification, some laboratory testing can be used to describe the strength of the materials. However, due to the discontinuous nature of the mélange, it can be difficult to recover samples for lab testing using rock coring techniques. Because of the fractured nature of the blocks, any specimens competent enough for testing may only represent the strongest blocks present at depth. Additionally, the behavior of bimrock material depends on the relative percentages of blocks and matrix, but laboratory testing can only be performed on elements of blocks or matrix. Thus, strength of individual blocks may only be roughly correlative to strength of the rock mass at depth.

A variety of lab tests have been performed on the graywacke, sheared shale, and mélange matrix based on relative competency for testing. Arup and CSA were able to run a handful of Unconfined Compression (UC) tests on relatively unfractured graywacke core pieces, with strength values ranging from 56 to 1250 ksf. Arup ran Isotopically Consolidated Undrained Triaxial Compression (I-CUTX) and Isotropically Consolidated-Drained Triaxial Compression (I-CUTX) tests on sheared shale and mélange matrix, respectively. CSA performed Unconsolidated Undrained Triaxial Compression (UUTX) tests on sheared shale. Strength values for the sheared shale were 7.7 to 15.2 ksf for the I-CUTX tests and 2.5 to 5.6 ksf for the UUTX tests. The mélange matrix strength ranged from 7.7 to 15.2 ksf with the I-CDTX tests. Lastly, Arup performed point loads on the graywacke and sheared shale, with average strength values of 57.5 and 13.8 ksf, respectively.

Based on the testing done to date, our on-going laboratory analyses include I-CUTX on the recovered core, and point-load testing on bimrocks. Results of this testing will be presented in a future memorandum.

4.0 SEISMIC HAZARD ANALYSIS AND GROUND MOTIONS

The following sections discuss the seismic hazard analyses for the subject site. The analyses utilized a site-specific site-response analysis to define how the subsurface materials will amplify or attenuate ground motions as they propagate from the underlying bedrock. A brief discussion of this analysis, including its benefits, is provided below. This discussion is followed by a detailed discussion of the analysis performed.

4.1 OVERVIEW OF SITE-SPECIFIC SITE RESPONSE ANALYSES

Seismic hazard analyses (SHA) are usually performed with semi-empirical ground motion models (GMMs) following the ergodic assumption whereby average source, path, and site effects from global databases apply for a specific site of interest. Site-specific site response is likely to differ from the global (ergodic) average used in GMMs. Relative to ergodic, site-specific hazard analyses often reduce ground motions at long return periods. This potential reduction is achieved by replacing the ergodic site amplification term used in the GMM and reducing the standard deviation of the ground motions.

The amplification (*Y*) of the ground motion experienced at the surface (*Z*) of a site relative to the reference/rock ground motion (*X*) is typically expressed as follows:

$$Y = \frac{Z}{X} \; or \; \ln(Y) = \ln(Z) - \ln(X) \qquad \text{Eq. (4.1-1)}$$

Ergodic ground motion models typically define ln(*Z*) as the sum of an event (F_E), path (F_P), site (F_S), and uncertainty/variability term ($\varepsilon\sigma$) as shown in Eq 4.1-1. The uncertainty term is the product of an epsilon (ε) value and a standard deviation (σ). The ε value represents how many standard deviations the surface motion is above (+) or below (-) the mean surface motion.

The F_S term represents how the subsurface soils amplify or attenuate ground motions relative to a reference rock condition. The average/ergodic F_S is calculated using simplistic parameters and may be biased at the site of interest. Thus, it is beneficial to replace the ergodic site term (F_s) in Eq. 4.1-2 with a site-specific term (μ_{lnY}) developed from a more rigorous consideration of wave propagation at the site:

$$lnZ = F_E(M, Fault\ Type) + F_p(R, M, Region) + F_s\left(v_{S30} + Z_{\frac{1.0}{2.5}}\right) + \epsilon\sigma \qquad \text{Eq. (4.1-2)}$$

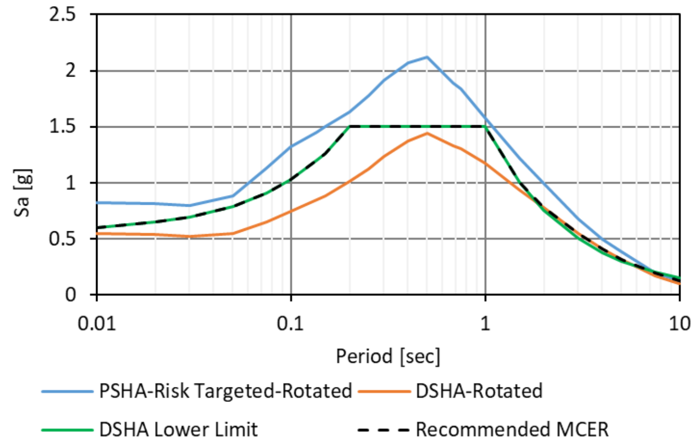
The amplification function can be developed based on any intensity measure (IM), but the IM is often taken as the peak ground acceleration (PGA). In ergodic analyses, the above equation is developed using large datasets from across the world. Conversely, site-specific analyses involve using local ground motion (GM) observations, if available and geologically compatible with the site of interest, and ground-response analyses (GRA) to develop a site-specific amplification function.

For the Millennium Tower, the available nearby GM records are not suitable for use in development of the site-specific amplification function because they are incompatible with the subsurface conditions at the Tower site. Therefore, the amplification function was developed based on the results of GRAs. To account for the uncertainties in the analyses and the lack of available GM records, the ergodic standard deviation (σ) was used in development of the final spectrum at the surface.

5.0 DETERMINATION OF CONTROLLING SEISMIC HAZARD ANALYSIS

The first step in performing the site-specific site response analysis is to determine whether deterministic or probabilistic seismic hazard analysis (PSHA and DSHA) controls the target spectrum according to ASCE 7-10 Chapter 21 specifications. For this purpose, seismic hazard analysis was performed at the surface using the program EZ-FRISK (Risk Engineering, 2015). As shown in Figure 5.0-1, the DSHA spectrum is below the PSHA spectrum and thus controls at this site. The deterministic event represented in Figure 5.0-1 represents a moment magnitude 8.1 event occurring on the Northern San Andreas Fault approximately 13 kilometers from the site. The seismic hazard analyses shown in Figure 5.0-1 were performed based on the shear wave velocity measurements adopted from Arup (2010) (refer to Section 6.1-1). The time-averaged shear wave velocity over the top 30 meters (V_{s30}) was estimated as 170 m/sec and the depths at which the shear wave velocity reaches 1,000 m/sec and 2,500 m/sec (z_1 and $z_{2.5}$, respectively) were estimated to be 70 meters and 230 meters, respectively.

FIGURE 5.0-1: **Seismic Hazard Analysis at the Surface**



6.0 REFERENCE GMM AND SITE RESPONSE SPECTRA

6.1 GROUND MOTION MODEL REFERENCE CONDITION

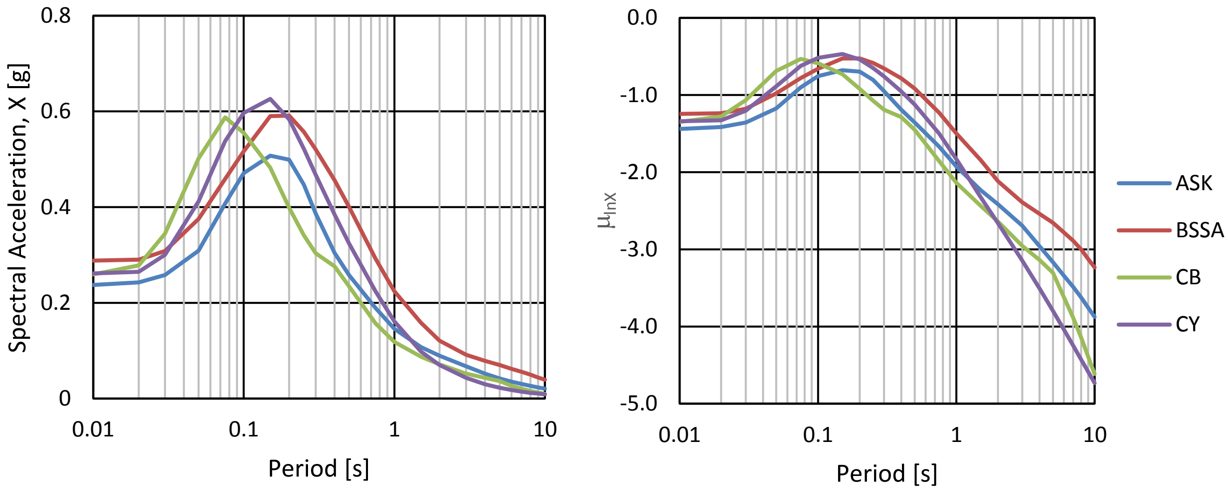
The site-specific approach used here is based on the guidelines provided by Tall Building Initiative (2017). For DSHA, this approach involves developing the mean (i.e. ε = 0) deterministic response spectrum at the reference condition for each of the NGA West2 GMMs (Abrahamson et al, 2014, or ASK; Boore et al., 2014, or BSSA; Campbell and Bozorgnia, 2014, or CB; and Chiou and Youngs, 2014, or CY) and calculating the natural log of this spectrum, which represents the sum of the event and path terms as presented in Eq. 6.1-1 and Eq. 6.1-2.

$$lnX = F_E(M, Fault\ Type) + F_p(R, M, Region) + \varepsilon.\sigma_{lnx} \qquad \text{Eq. (6.1-1)}$$

$$\mu_{lnX} = F_E(M, Fault\ Type) + F_p(R, M, Region) \qquad \text{Eq. (6.1-2)}$$

The reference condition varies between the ground motion models, with V_{S30} values ranging from 760 to 1180 m/sec. The basin depth terms (z_1 and $z_{2.5}$) are zero for the reference condition. Figure 6.1-1 shows the response spectra for the reference condition for each GMM for the controlling deterministic scenario. These response spectra are used in the DSHA discussed in Section 6.0.

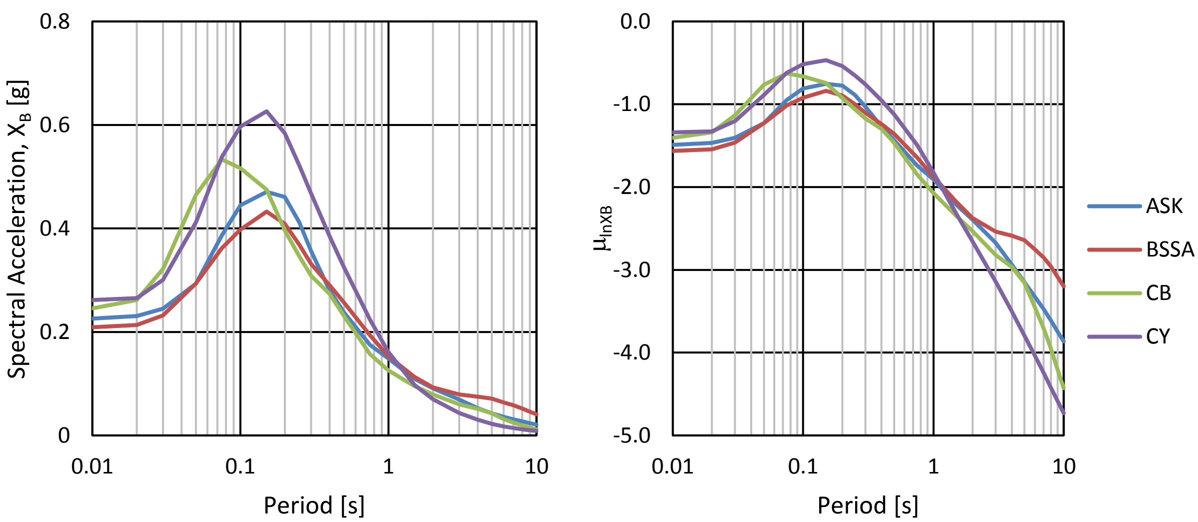
FIGURE 6.1-1: **Response Spectra for the Controlling Deterministic Scenario for the *GMM* Reference Condition for the ASK, BSSA, CB, and CY GMMs.**



6.2 SITE REFERENCE CONDITION

The site reference condition corresponds to bedrock at the Millennium Tower site and differs from the GMM reference condition. The site reference condition was taken as the first layer in the V_S profile that exceeds 1,300 m/sec. Accordingly, z_1 and $z_{2.5}$ were estimated as zero and 50 meters below the reference site condition, respectively (Note that these V_{S30}, Z_1, and $Z_{2.5}$ differ from those of the GMM reference condition discussed in Section 6.1). The response spectra associated with the site reference condition (X_B) for the four GMMs are shown in Figure 6.2-1. These response spectra are used for selecting and scaling ground motions for use in GRA as discussed in Section 4.0.

FIGURE 6.2-1: **Response Spectra for the Controlling Deterministic Scenario for the *Site* Reference Condition for the ASK, BSSA, CB, and CY GMMs.**



7.0 GROUND RESPONSE ANALYSIS

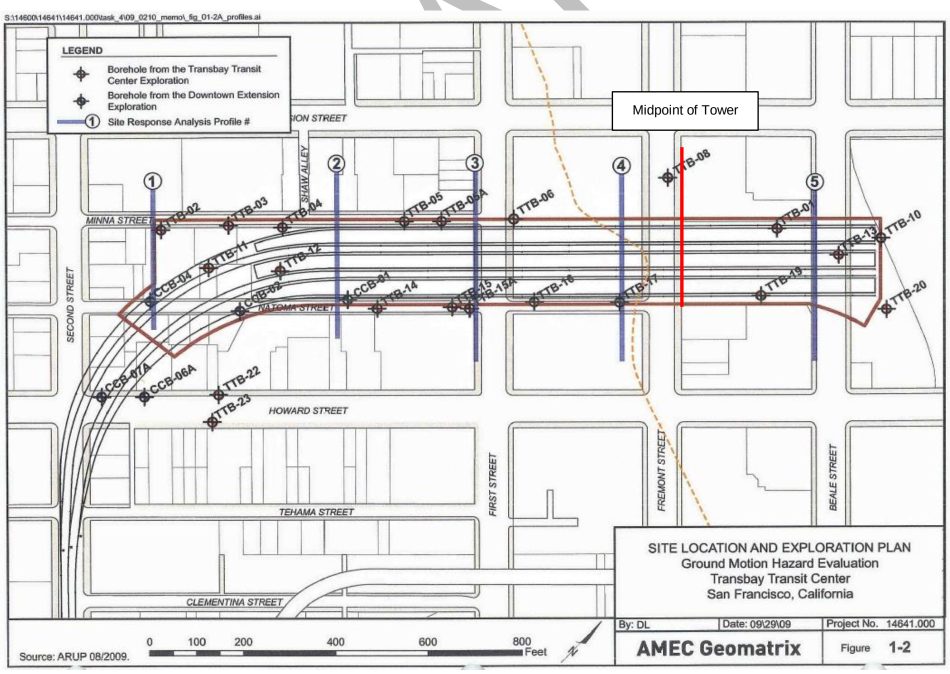
One-dimensional Ground Response Analysis (1D GRA) was performed for the purpose of estimating site effects and developing a site-specific mean amplification function for the site-specific analyses (Eq. 4.1-1). The GRA produces a series of discrete results, consisting of period-dependent amplification *Y* given a particular set of dynamic soil properties and a particular input motion with amplitude x_{IMref} (Stewart et al., 2014). These results are used in developing the site-specific mean amplification function. This section discusses GRA; specifically, it describes the selection of required soil dynamic properties (i.e. shear wave velocity, V_S profile, and Modulus Reduction and Damping, MRD, curves) and their uncertainties, input motion selection and scaling, and the method of analysis used to perform the GRA.

7.1 DYNAMIC SOIL PROPERTIES

7.1.1 Shear Wave Velocity

Shear wave velocity profiles were adopted from the PS suspension logging tests reported by Arup (2010) and Amec Geomatrix (2010) for the Transbay Terminal Center. The Vs profiles at Sections 4 and 5 were selected to represent the subsurface conditions along Fremont and Beale Streets. A third shear wave velocity profile was developed through the middle of the tower based on knowledge of subsurface conditions and interpolation between the other two shear wave velocity profiles. Vs profiles are shown in Figure 7.1.1-1.

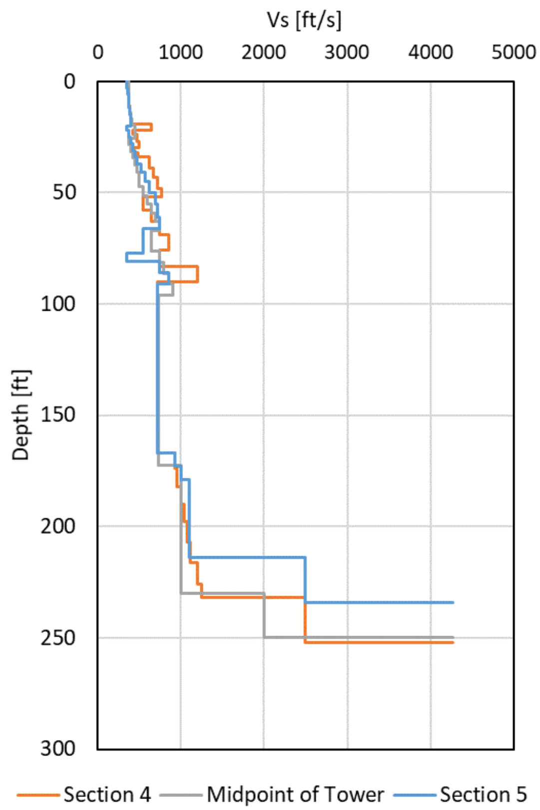
FIGURE 7.1.1-1: **Section 4 and Section 5 Locations**



Amec Geomatrix (2010)

Figure 7.1.1-2 compares these shear wave velocity profiles. The results presented in this report are corresponding to the GRA developed based on the shear wave velocity profile at Section 4. To account for the uncertainties in the shear wave velocity profile, the final report will include the GRA results using all three shear wave velocity profiles.

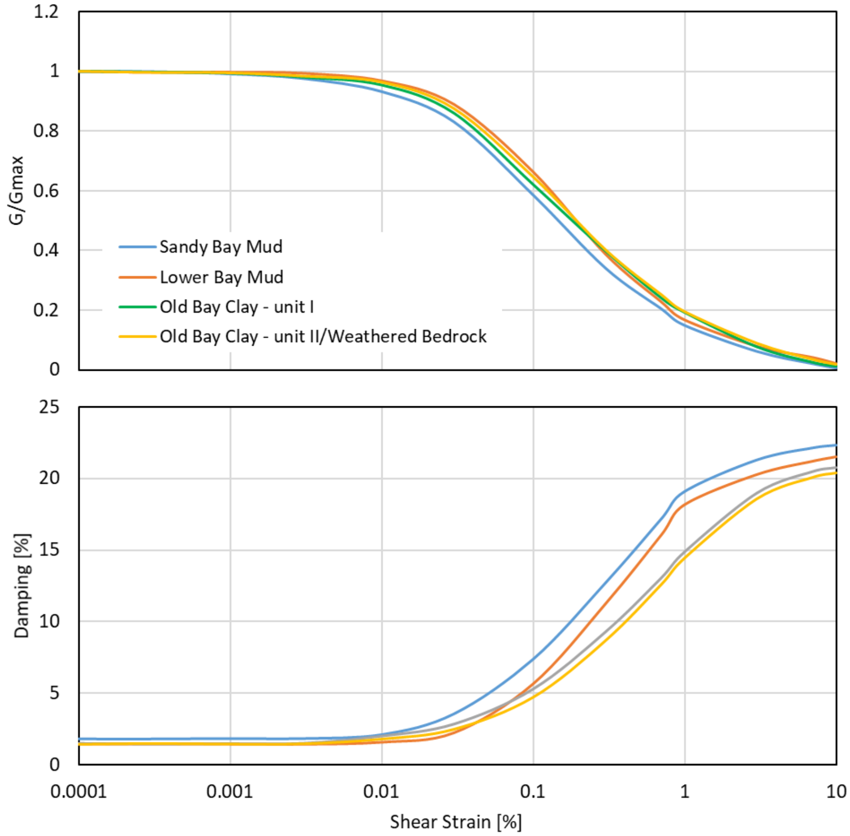
FIGURE 7.1.1-2: **Shear Wave Velocity Profiles**



7.1.2 Modulus Reduction and Damping

For Young Bay Mud (YBM) and Old Bay Clay (OBC), the modulus reduction (MR or G/G_{max}) and damping (D) curves were developed using previously measured and peer-reviewed data from nearby projects including the Bay Bridge and San Francisco International Airport (SFO). These curves are shown in Figure 7.1.2-1. For granular materials, MR and D curves were developed using laboratory-based relationships developed by Menq (2003). In the final report, the MR and D will be developed based on Darendeli (2001) correlations. The MR curves tend to become flatter (i.e., higher G/G_{max} at a given strain value) and the damping decreases for the deeper layers. This is due to the increase in mean effective stress with depth. At large strains (greater than approximately 0.5%), the MR and D curves from the empirical relationships are unbounded by laboratory measurements and can imply unrealistic shear strengths. Thus, when large strains are expected in the GRA, it is necessary to adjust the large-strain portions of the MR and D curves to account for the soil shear strength. Shear strength adjustments are described in Section 7.3.

FIGURE 7.1.2-1: Modulus Reduction (G/G_{max}) and Damping Curves for the Deepsoil Soil Profile



7.2 INPUT MOTION SELECTION

This section discusses the selection and scaling of input ground motions used in the GRA. Note that these ground motions were only selected/scaled for the purpose of performing the GRA. The principal factors in development of the input time histories for use in the GRA are:

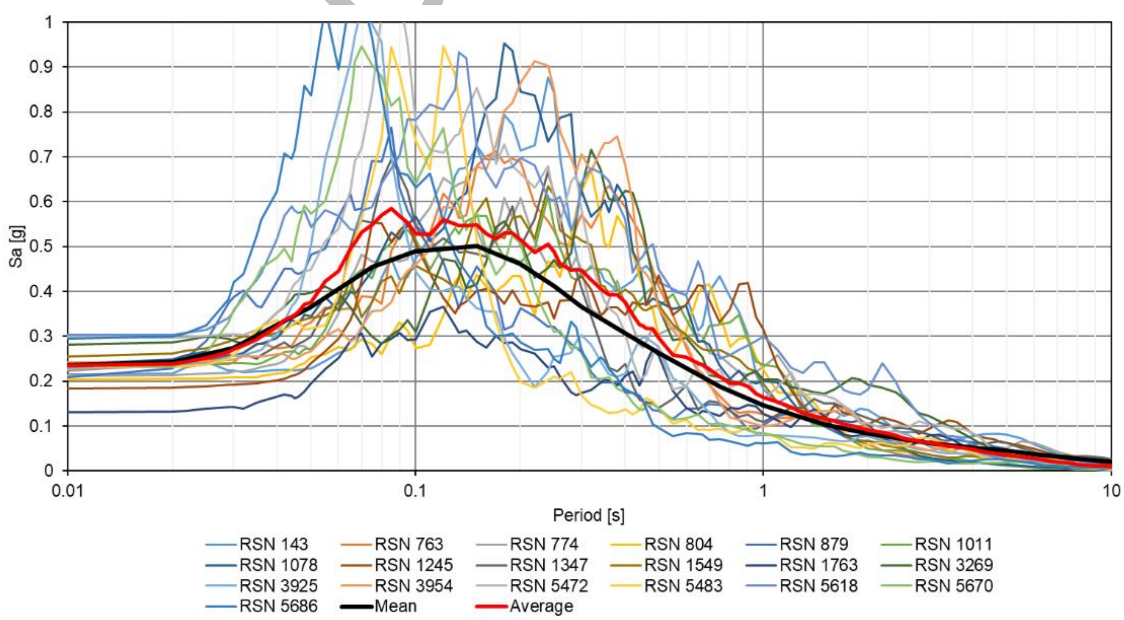
- Development of the target spectra for the site reference condition (refer to Section 6.2)
- Selection of compatible ground motion time histories
- Scaling of the ground motion time histories

Outcropping ground motion time histories were developed for the site reference condition for the controlling deterministic scenario (i.e., Figure 6.2-1). A mean response spectrum was computed from the four response spectra shown in Figure 6.2-1 and ground motions were scaled to this spectrum. Considerations were given to the seismic source controlling the target spectrum such as geologic conditions, distance to the controlling fault, ground motion duration, and intensity that could be expected at this site. In general, input ground motions were selected to be consistent with the site reference condition and to cover the range of input peak acceleration values of 0.15g to 0.3g. The selected scaled ground motions are presented in Table 7.2-1 and Figure 7.2-1.

TABLE 7.2-1: Selected Ground Motions for Spectral Scaling at the Reference Site Condition

NO.	EARTHQUAKE	RSN	PULSE PERIOD (sec)	MAG. (M_W)	R_{rup} (km)	MECHANISM	V_{S30} (m/s)	D_{5-95} (sec)	SCALING FACTOR
1	Tabas_ Iran	143	6.188	7.35	2.05	Reverse	766	16.5	0.26
2	Loma Prieta	763	-	6.93	9.96	Reverse Oblique	729	5.0	0.66
3	Loma Prieta	774	-	6.93	55.11	Reverse Oblique	735	19.5	4.00
4	Loma Prieta	804	-	6.93	63.15	Reverse Oblique	1020	12.1	2.50
5	Landers	879	5.124	7.28	2.19	strike slip	1369	13.8	0.28
6	Northridge-01	1011	-	6.69	20.29	Reverse	1222	8.7	1.67
7	Northridge-01	1078	-	6.69	16.74	Reverse	715	8.3	0.92
8	Chi-Chi_ Taiwan	1245	-	7.62	37.72	Reverse Oblique	804	36.2	4.00
9	Chi-Chi_ Taiwan	1347	-	7.62	61.06	Reverse Oblique	996	23.3	2.59
10	Chi-Chi_ Taiwan	1549	-	7.62	1.83	Reverse Oblique	511	30.8	0.34
11	Hector Mine	1763	-	7.13	89.98	strike slip	724	23.3	4.00
12	Chi-Chi_ Taiwan-06	3269	-	6.30	41.36	Reverse	544	21.7	1.42
13	Tottori_ Japan	3925	-	6.61	15.23	strike slip	940	19.6	1.51
14	Tottori_ Japan	3954	-	6.61	15.59	strike slip	967	12.8	1.19
15	Iwate_ Japan	5472	-	6.90	33.76	Reverse	643	23.4	1.53
16	Iwate_ Japan	5483	-	6.90	39.41	Reverse	829	25.9	2.82
17	Iwate_ Japan	5618	-	6.90	16.27	Reverse	825	22.6	1.10
18	Iwate_ Japan	5670	-	6.90	82.93	Reverse	1423	20.2	3.73
19	Iwate_ Japan	5686	-	6.90	57.19	Reverse	748	20.7	2.65

FIGURE 7.2-1: Scaled Ground Motion Time Histories at the Reference Site Condition



7.3 NONLINEAR AND EQUIVALENT LINEAR ANALYSES

The available boring logs (TTB-01, TTB-08, TTB-19 from Arup (2010)), were used to delineate the subsurface material types and their depth ranges. The subsurface profile was divided into sublayers, with thicknesses chosen such that the maximum frequency (V_s/4H) transmitted through the layers are not less than 30 Hz. The sublayers in the profile range from approximately 2 to 20 feet thick, with the layers becoming progressively thicker with depth.

Nonlinear (NL) and equivalent linear (EQL) GRAs were performed using the program DEEPSOIL (Hashash et al. 2016). The results were compared to ensure reasonable consistency between the NL and EQL analyses. Both equivalent-linear (EQL) and nonlinear (NL) analyses account for the degradation of soil stiffness and increase in damping with increasing strain levels. However, EQL analyses tend to produce flat response spectra at high frequencies (short periods) when the input motions induce large shear strains (Kim et al. 2016). These flat, short-period response spectra are inconsistent with actual observed ground motions. Thus, NL analyses are more appropriate for high-amplitude ground motions, which induce higher strains in the soil materials. Given the large amplitude of many of the ground motions used on this project, NL analyses were ultimately used to develop the site amplification function. The NL analyses were performed using the General Quadratic Model (GQ/H) in DEEPSOIL v6.1 (Groholski et al. 2016).

8.0 DEVELOPMENT OF SITE-SPECIFIC AMPLIFICATION FUNCTION

The site-specific amplification for each GM was extracted from the GRA results as shown in Figure 8.0-1. However, because the site reference/base condition of the subsurface profiles used in the GRA (V_{S30} = 1300 m/sec) does not match the GMM reference condition (V_{S30} = 760 to 1180 m/sec), the GRA-based amplification and input PGA values were adjusted as described in Stewart et al. (2017) and in Eq. 8.0-1 and Eq. 8.0-2.

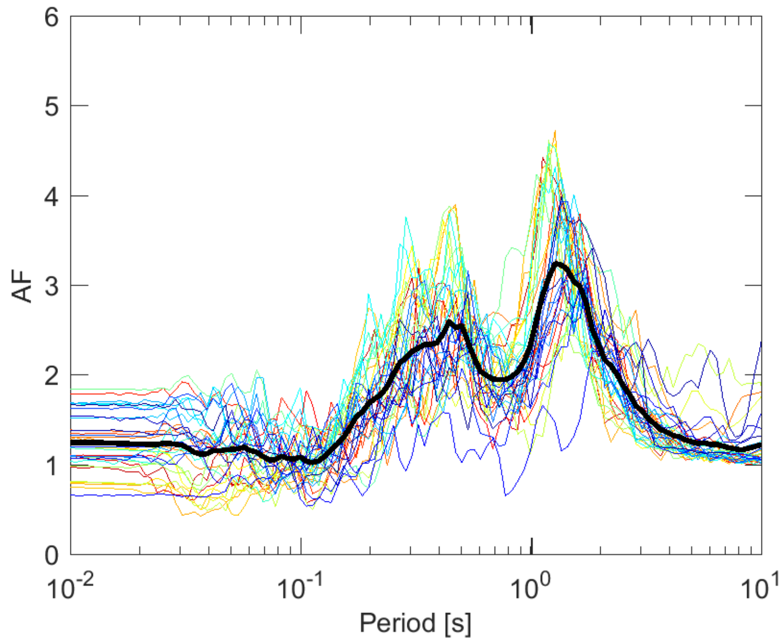
$$\ln(Y) = \ln(Y^B) + \mu_{lnY}(V_{S30}^B) \qquad \text{Eq. (8.0-1)}$$

where $\mu_{lnY}(V_{S30}{}^B)$ is the mean site amplification from an ergodic model for the base-of-profile site condition. Likewise, the input ground motion amplitude used in the site-specific amplification function is taken as:

$$\ln(x_{IMref}) = \ln(x_{IMref}^B) - \mu_{lnYref}(V_{S30}^B) \qquad \text{Eq. (8.0-2)}$$

where $x_{IMref}{}^B$ is the corresponding value of that intensity measure (IM) for the base-of-the-profile site condition and $\mu_{lnYref}(V_{S30}{}^B)$ is the ergodic amplification of that reference IM for the site condition represented by $V_{S30}{}^B$. The site amplification and x_{IMref} values shown in Figure 8.0-1 were adjusted in this manner.

FIGURE 8.0-1: Amplification Function from Individual GRAs and the Mean Amplification Function (in black)



9.0 IMPLEMENTATION AND RESULTS

9.1 SITE-SPECIFIC DETERMINISTIC SURFACE SPECTRA

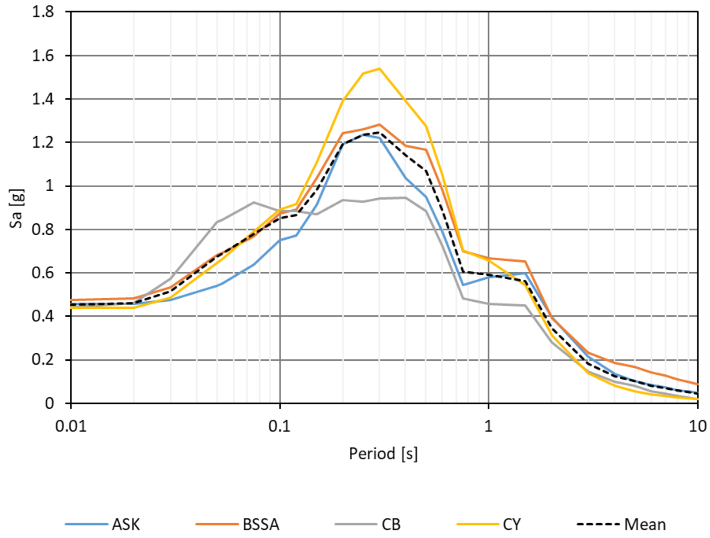
The 84th percentile (i.e. $\varepsilon = 1.0$) site-specific deterministic response spectrum (Z) was developed at the ground surface using the reference condition mean response spectrum (μ_{lnx}), the mean amplification function (μ_{lny}), and the fully ergodic standard deviation (σ_{lnZ}) as described in Eq. 9.1-1 and Eq. 9.1-2.

$$\ln(\mathrm{Z}) = \mu_{\ln\mathrm{X}} + \mu_{\ln\mathrm{Y}} + \varepsilon.\sigma_{\ln\mathrm{Z}} \qquad \text{Eq. (9.1-1)}$$

$$S_{a,surface} = \exp(\mathrm{Z}) \qquad \text{Eq. (9.1-2)}$$

It should be noted that the fully ergodic standard deviation was applied to account for the uncertainty in the GRA analyses since suitable GM recordings were not available to be included in the development of the site-specific site amplification function. Figure 9.1-1 presents the site-specific deterministic response spectra at the surface.

FIGURE 9.1-1: Site-Specific Deterministic Response Spectra at the Surface



9.2 FINAL RECOMMENDED TARGET SPECTRA

Per ASCE 7-10 Chapter 21, the maximum rotation factors based on Shahi and Baker (2013) were applied to obtain the maximum-rotated response spectrum (MCE_R). Figure 9.2-1 depicts the results of the site-specific analysis. The same figure presents the mapped MCE_R, as well as 80 percent of the mapped MCE_R. Per ASCE 7-10 Section 21.4, the final MCE_R is controlled by the higher of the site-specific MCE_R and 80 percent of the mapped MCE_R. The final Recommended MCE_R represents the envelope of these two spectra. The final Recommended MCE_R is tabulated in Table 9.2-1.

FIGURE 9.2-1: Recommended Site-Specific MCER at the Surface

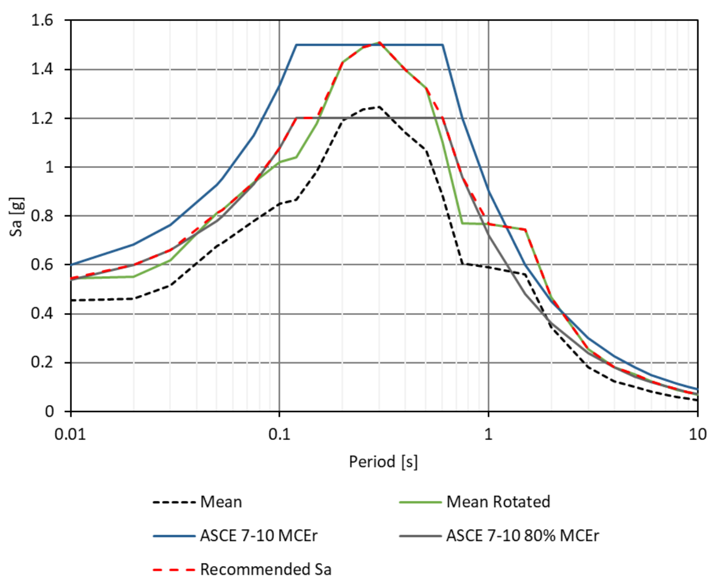


TABLE 9.2-1: Site-Specific Response Spectrum

PERIOD (seconds)	RISK TARGETED – MAXIMUM ROTATED MCE_R (g)
0.01	0.545
0.02	0.600
0.03	0.660
0.05	0.811
0.075	0.936
0.1	1.080
0.15	1.200
0.2	1.429
0.25	1.491
0.3	1.511
0.4	1.396
0.5	1.323
0.6	1.200
0.75	0.960
1	0.767
1.5	0.745
2	0.467
3	0.256
4	0.181
5	0.153
6	0.123
7	0.105
8	0.090
9	0.080
10	0.072

9.3 HORIZONTAL GROUND MOTION SELECTION AND SCALING

Eleven pairs of recorded ground motions were selected as seed motions. Considerations were given to the seismic source controlling the target spectrum such as geologic conditions, distance to the controlling fault, ground motion duration, and intensity that could be expected at this site. In addition, due to proximity of the site to the San Andreas Fault, five of the eleven seed motions were selected to have a velocity pulse with the pulse period being close to the fundamental period of the tower (T = 4.6 seconds). In identifying pulse-like motions, the PEER Ground Motion Database was used, which uses the procedure developed by Shahi and Baker (2010).

For each horizontal ground motion pair, a maximum-direction spectrum (RotD100) was constructed from the two horizontal ground-motion components. Each ground motion was then scaled, with an identical scale factor applied to both horizontal components, such that the average of the maximum-direction spectra from all ground motions do not fall below 90% of the target spectrum for any period within the period range of interest.

The period range of interest was selected based on the fundamental period of the tower. The upper bound of the period range was selected as about 2.0T (9.0 seconds), and the lower bound of the period range was selected such that it includes at least the number of elastic modes necessary to achieve 90% mass participation in each principal horizontal direction. The lower bound of the period range was provided by the structural engineer, LERA, as 0.3 seconds.

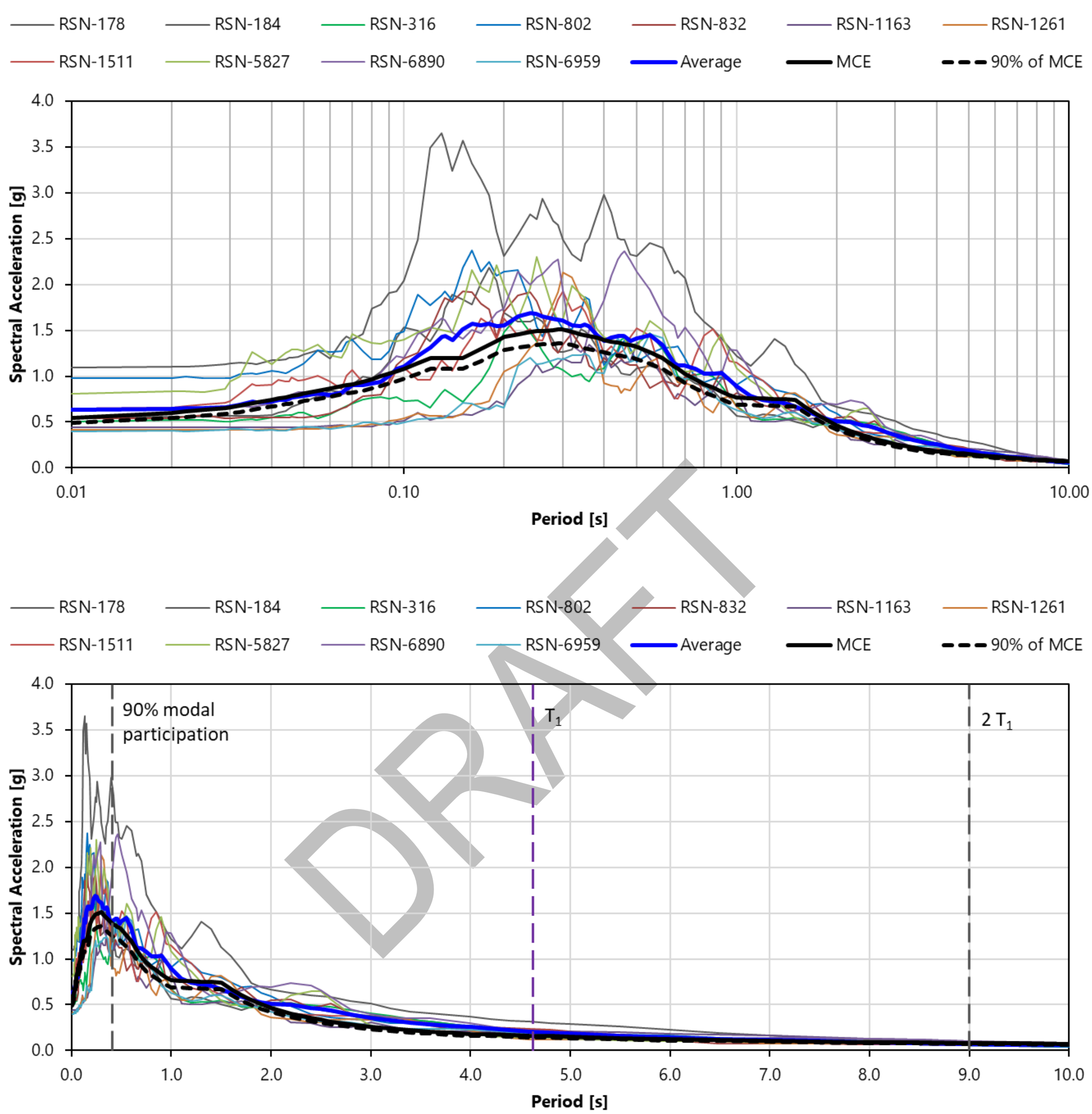
The angle of application for each pair of ground motions was selected such that the average (or mean) of the component response spectrum for the records applied in each direction is within ±10% of the mean of the component response spectra of all record applied for the period range of interest.

Selected and scaled ground motions are presented in Table 9.3-1 and in Figure 9.3-1.

TABLE 9.3-1: Selected Ground Motions for Spectral Scaling at the Surface - MCE_R

NO.	EARTHQUAKE	RSN	Pulse Period (sec)	MAG. (M_W)	R_{rup} (km)	Mechanism	V_{S30} (m/s)	D_{5-95} (sec)	Scaling Factor	Angle of Application (deg)
1	"Imperial Valley-06"	178	4.501	6.53	12.85	strike slip	163	14.1	1.88	22/112
2	"Imperial Valley-06"	184	6.265	6.53	5.09	strike slip	202	7.00	2.20	90/180
3	"Westmorland"	316	4.389	5.9	16.66	strike slip	348	18.7	2.04	173/263
4	"Loma Prieta"	802	4.571	6.93	8.5	Reverse Oblique	381	9.4	1.88	0/90
5	"Landers"	832	-	7.28	69.21	strike slip	383	28.5	3.66	62/152
6	"Kocaeli_ Turkey"	1163	-	7.51	60.05	strike slip	354	36.7	3.83	106/196
7	"Chi-Chi_ Taiwan"	1261	-	7.62	56.06	Reverse Oblique	373	33.4	4.00	41/131
8	"Chi-Chi_ Taiwan"	1511	4.732	7.62	2.74	Reverse Oblique	615	29.5	1.50	136/226
9	"El Mayor-Cucapah_ Mexico"	5827	-	7.2	15.91	strike slip	242	34.5	1.50	46/136
10	"Darfield_ New Zealand"	6890	-	7.0	17.64	strike slip	204	20.0	2.50	92/182
11	"Darfield_ New Zealand"	6959	12.019	7.0	19.48	strike slip	141	30.5	1.21	90/180

FIGURE 9.3-1: Scaled RotD100 (maximum direction) Ground Motions at the Surface - MCE_R



9.4 SERVICE LEVEL EARTHQUAKE SPECTRUM AND TIME HISTORIES

The Service-Level Earthquake (SLE) spectral response was evaluated by performing a probabilistic hazard analysis to develop a geometric mean response spectrum corresponding to a 50-percent probability of exceedance in 30 years (43-year return period).

Nine pairs of recorded ground motions were selected and scaled considering the criteria presented in Section 9.3. Table 9.4-1 depicts the recommended SLE response spectrum. Selected and scaled ground motions are presented in Table 9.4-2 and in Figure 9.4-1.

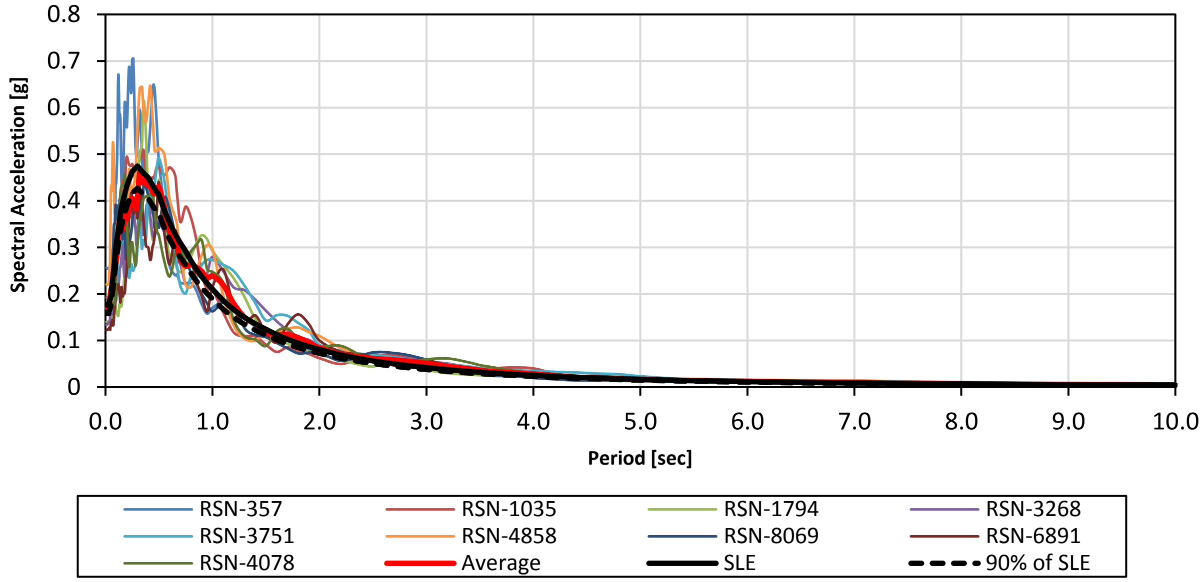
TABLE 9.4-1: SLE Response Spectrum

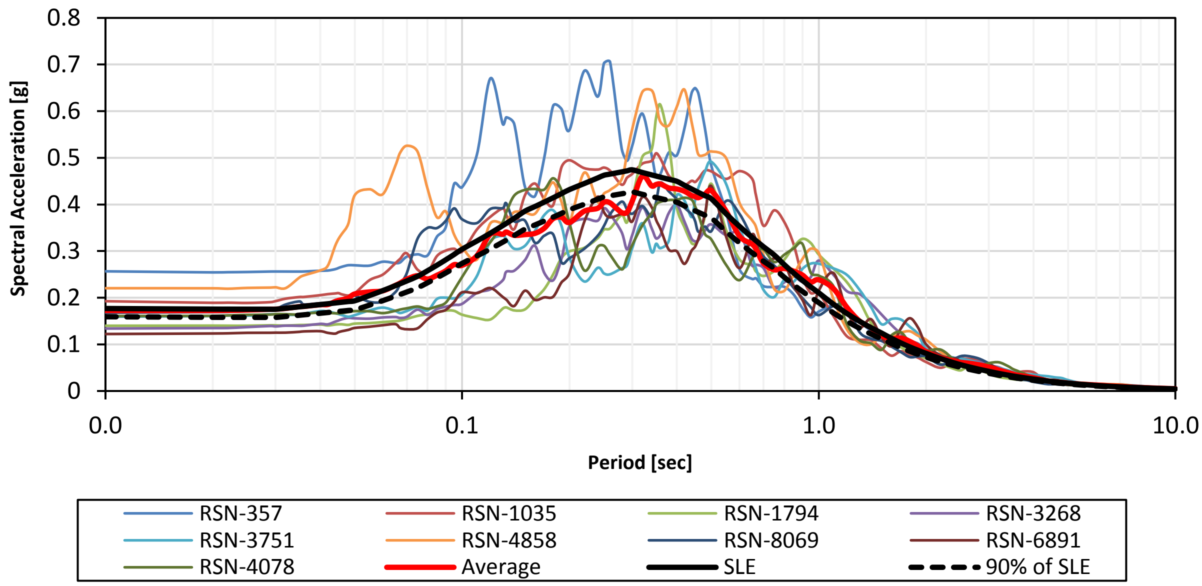
PERIOD (seconds)	SLE (g)
0.01	0.177
0.02	0.176
0.03	0.175
0.05	0.193
0.075	0.246
0.1	0.304
0.15	0.386
0.2	0.432
0.25	0.463
0.3	0.474
0.4	0.450
0.5	0.412
0.75	0.292
1	0.211
1.5	0.124
2	0.081
3	0.042
4	0.026
5	0.017
7.5	0.008
10	0.005

TABLE 9.4-2: Selected Ground Motions for Spectral Scaling at the Surface - SLE

RSN NO.	EARTHQUAKE	PULSE PERIOD (sec)	MAG. (M_W)	R_{rup} (km)	MECHANISM	V_{S30} (m/sec)	D_{5-95} (sec)	Scaling Factor
357	Coalinga-01		6.36	34	Reverse	565.08	12.2	1.67
1035	Northridge-01	-	6.69	39.29	Reverse	351.57	20.4	1.11
1794	Hector Mine	-	7.13	31.06	strike slip	379.32	14.6	0.73
3268	Chi-Chi_ Taiwan-06	-	6.3	33.61	Reverse	542.61	12.1	0.83
3751	Cape Mendocino	-	7.01	35.22	Reverse	459.04	14.7	0.7
4078	Parkfield-02_ CA	-	6.0	22.59	strike slip	333.61	27.7	0.8
4858	Chuetsu-oki_ Japan	-	6.8	30.65	Reverse	640.14	17	1.91
6891	Darfield_ New Zealand	-	7.0	43.6	strike slip	638.39	28.9	1.02
8069	Christchurch_ New Zealand	-	6.2	36.18	Reverse Oblique	332.73	12.9	1.94

FIGURE 9.4-1: Scaled RotD100 (maximum direction) Ground Motions at the Surface - SLE





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10.0 FOUNDATION ANALYSIS

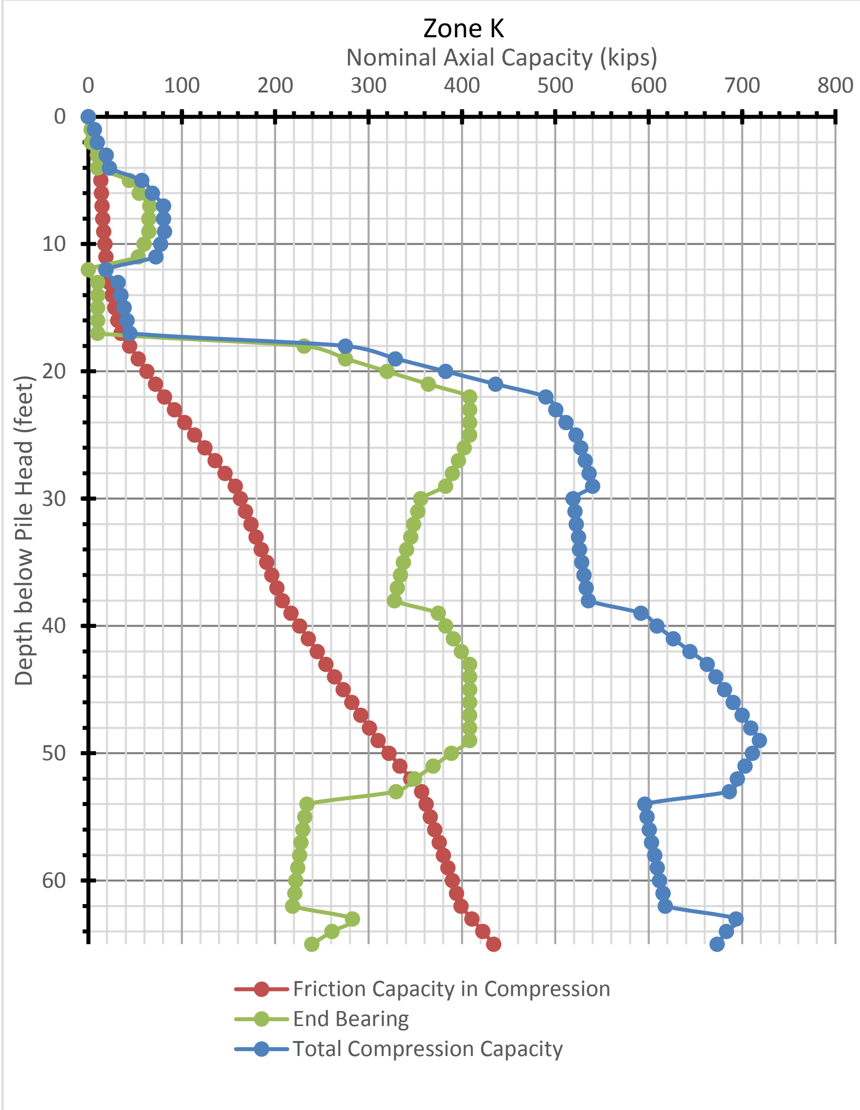
10.1 AXIAL PILE CAPACITIES

10.1.1 Existing Piles

A number of sources were evaluated and reviewed in order to assess the axial capacities of the existing piles. Available geologic and soil data, existing pile installation logs, existing pile indicator program data (including pile-driving analyzers (PDA), and case pile wave analysis program (CAPWAP)), and available data for similar piles driven in similar geological environments, were reviewed and used to estimate axial capacities. Available existing pile information includes 25 indicator piles installed at various locations across the site, which were advanced to the dense sands on top of the OBC and through the dense sands bearing on the Old Bay Clay. Of the indicator piles, only 5 piles had meaningful PDA and CAPWAP analyses that could be utilized for the capacity checks.

Empirical analyses were developed using alpha and beta methods to assess the theoretical skin friction for clayey and granular soils. Blow count data from existing geotechnical borings were used to calculate skin friction on the sandy materials, while undrained shear strength information from the existing laboratory data was used to develop skin friction for the clayey soils. In a similar way, onsite blow count data from geotechnical borings were used to estimate bearing capacity of the dense sands where the existing piles currently tip. A capacity chart is shown in Figure 10.1.1-1 as a sample of the results of the empirical calculations.

FIGURE 10.1.1-1: Zone K Empirical Geotechnical Nominal Resistance



The empirical analysis was then compared to the available PDA/CAPWAP data from the indicator piles. The PDA/CAPWAP for 5 of the indicator piles was then related to initial pile-driving blow counts per last foot. This was done to normalize PDA/CAPWAP capacities for all the piles, since initial blow counts were recorded for all existing piles. In addition, PDA/CAPWAP data from other sites with similar geology, pile and pile-installation processes were reviewed to bound the pile capacity for high installation blow counts. Figure 10.1.1-2 shows the indicator pile data table from the original tests. Figure 10.1.1-3 shows the CAPWAP-derived, end-bearing capacity of the existing 301 Mission piles plotted with CAPWAP data from similar piles driven in similar geological environments with similar hammers. The function in Figure 10.1.1-3 was then used to assign bearing capacities to existing piles as a function of blow count.

Figure 10.1.1-2 – PDA/CAPWAP Results from Original Indicator Pile Program

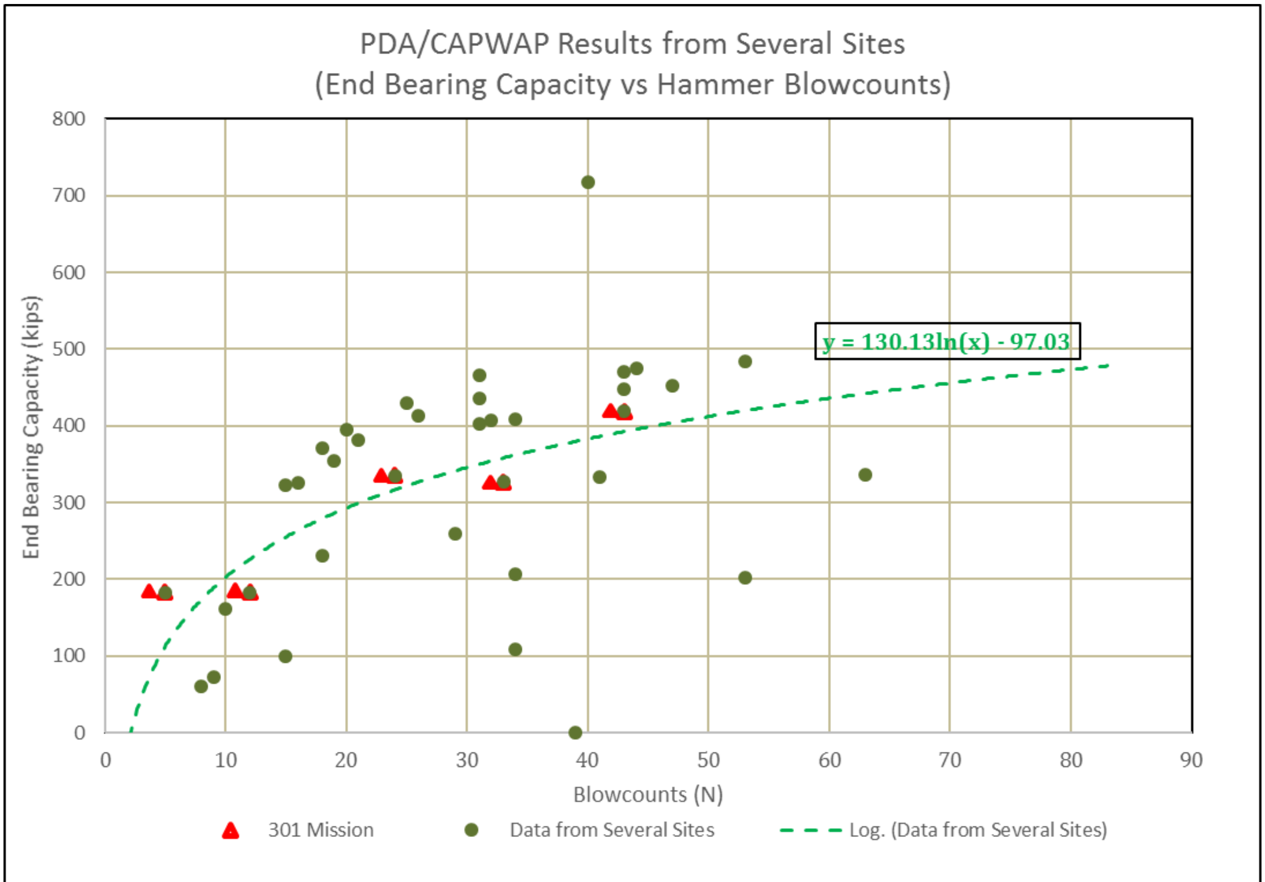
* * * DRAFT * * *
Table 1
301 Mission Street
San Francisco, California

Updated -- 11/10/05

		End of Installation					Beginning of Restrike						
Indicator Pile	Pile Location	2005 Date Installed	Penetration ft	Blowcount B/ft	Case Method Capacity kips	CAPWAP Method Capacity kips	Date Restruck	Penetration ft	Final Toe Elev., ft	Blowcount B/inch	Case Method Capacity kips	CAPWAP Method Capacity kips	Comments
I-1	1.5-A.2	28-Oct	65	40			31-Oct	65 84'-3"	-91.25	11 5/3"	810		Pile Head Spalled at BOR
I-2	6.5-A.2	28-Oct	62	33			31-Oct	62 75	-91.00	6,4,6,5,6,5 20/ft	805		
I-3	11.5-A.2	28-Oct	58	33			31-Oct	58 86.5	-93.50	4,5,4,4,3,4 12/6"	655	710	
I-4	2.6-B.8	28-Oct	66	34			31-Oct	66 85	?	11,7,7,6,6,6 N/A	845		
I-6	10-C	28-Oct	62	43			31-Oct	62 86	-93.00	20,12,6,6,5,6 24/ft	760+	812+	
I-8	D.2-9.5	3-Nov	78'-6" 100	12/6" 16	415	406			-107.00				
I-13	D.8-2.8	1-Nov	59	5			3-Nov	59 64'-10" 87.3	-93.33	4,2,2,2,2,2 10/10" 8/4"	395 330	405	
I-18	F-11.2	2-Nov	56	2			3-Nov	57 65 78'-10"	-85.83	15/ft 23/ft 95/10"	295 360		
I-20	F.8-3.2	1-Nov	67	12			3-Nov	67 71'-6" 75'-10"	-82.83	7,5,4,3,4,3 16/6" 92/10"	450+ 375	579	
I-21	F.5-9.5	2-Nov	61	1			3-Nov	62 66'-6" 91'-8"	-98.67	15/ft 8/6" 8/8"	175 180		Severely Eccentric Impact

Notes 14-inch-square PCPS concrete) driven with a Delmag D46-32 single-acting diesel hammer
Piles were generally monitored at Beginning of Restrike (BOR), and usually driven below grade with a follower after PDA gage removal.
Capacity values followed by a plus (+) sign signify lower-bound estimates due to refusal blowcounts or Case Method limitations in the field calculations.

FIGURE 10.1.1-3 – PDA/CAPWAP End Bearing Capacity versus Hammer Blowcounts



Based on the empirical analyses, geological conditions, and PDA/CAPWAP analysis, the site was divided into zones of differing ultimate geotechnical capacities for the existing piles. Figure 10.1.1.1-1 shows the capacity zones and the capacities assigned.

Existing pile capacities ranged from approximately 450 to 800 kilo-pounds (kips) based on a weighted average of the empirical analysis and the PDA/CAPWAP analysis. The weighting used to combine empirical and PDA/CAPWAP methods was set to 30/70, respectively. This ratio was selected based upon inherent conservatism and confidence levels within the methods and the information considered. Figure 10.1.1.1-2 shows the backbone curves for the existing piles per zone. Backbone curves were derived using the API 2009 Manual for axial capacity loaded in compression and in tension.

10.1.1.1 Geotechnical Bounding on Existing Pile Axial Capacities

In order to capture additional uncertainty, upper-bound and lower-bound backbone curves were developed by weighting the PDA/CAPWAP analysis and the empirical analysis differently for every zone. The net result of this was that a ±10-percent bounding per zone was appropriate to cover additional uncertainty.

FIGURE 10.1.1.1-1 – Pile Capacity Zones

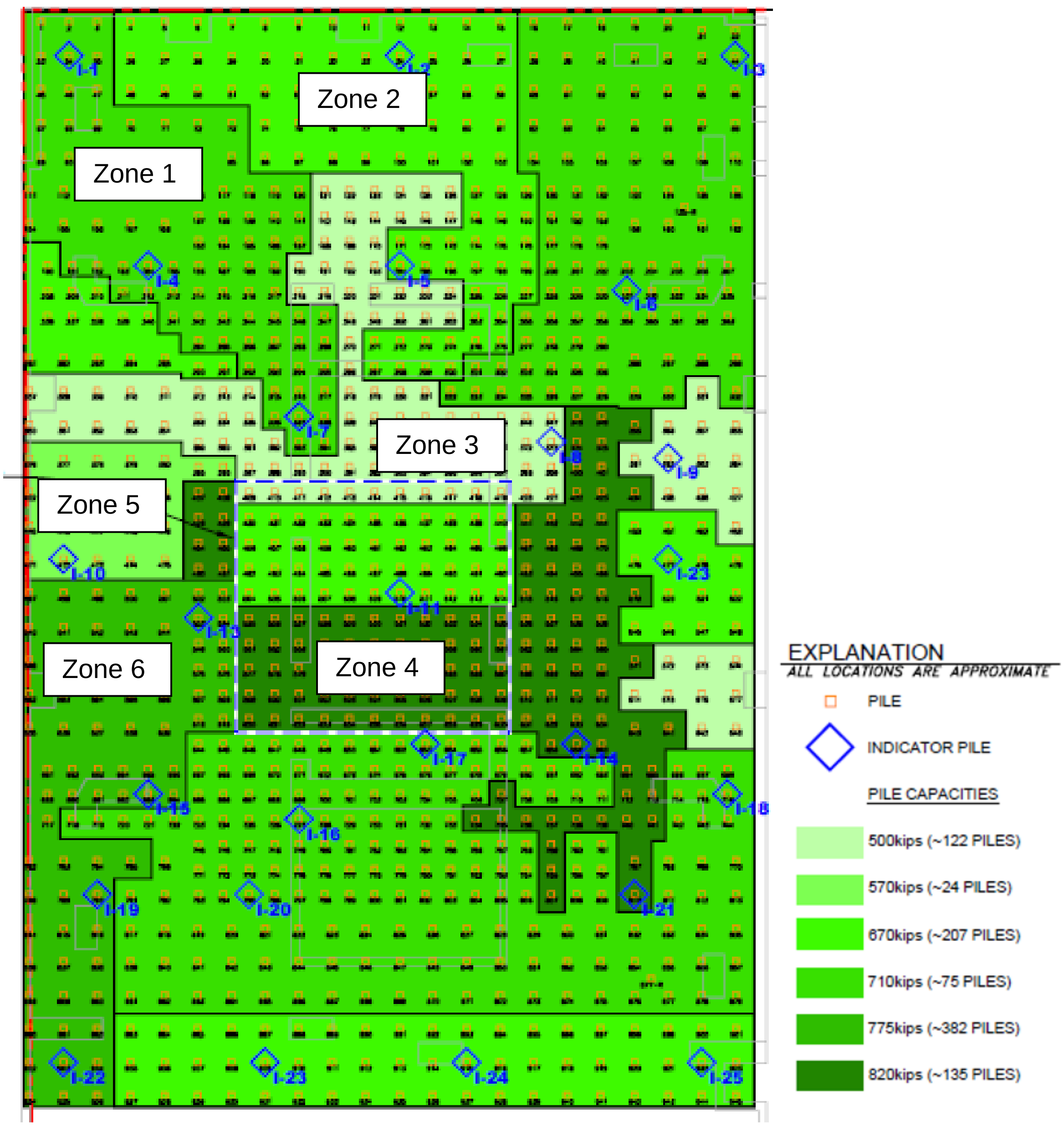
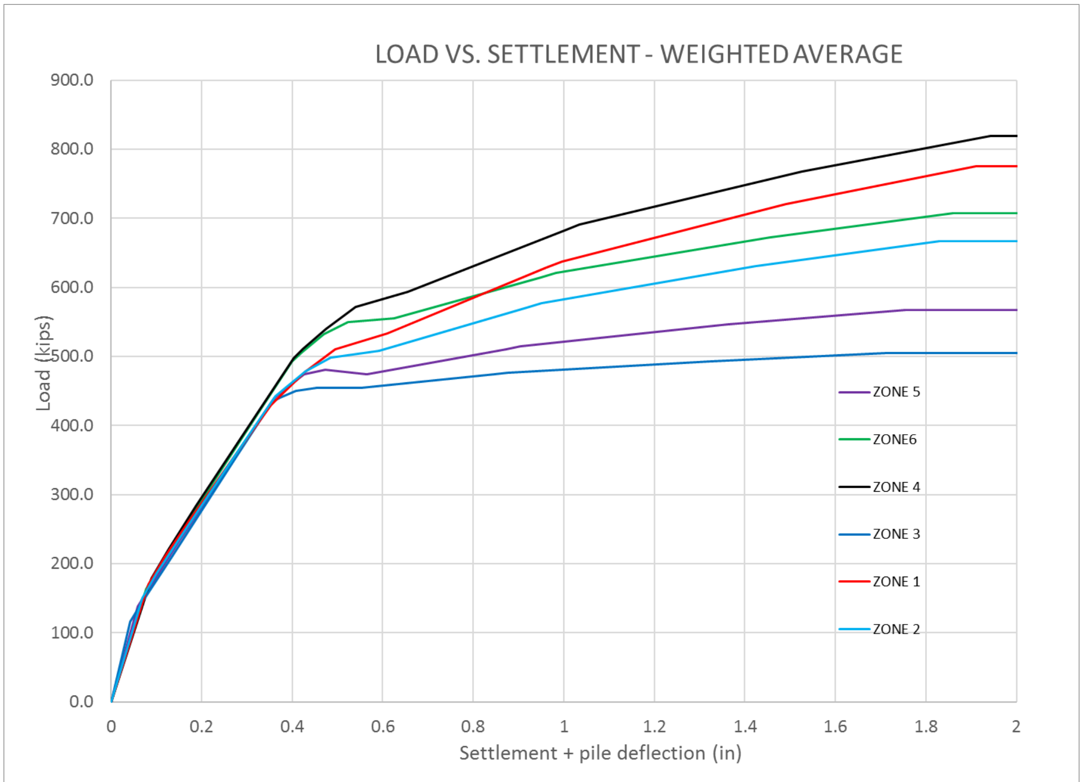


FIGURE 10.1.1.1-2 – Existing Pile Capacity



10.1.2 Proposed Piles

Based on the preliminary results of the pile load test in Beale Street, we believe that a full-displacement push pile will be the most efficient solution to fully engage the friction between the rock and pile. Since the Beale street sand-pile test demonstrated that the maximum load resistance that can be expected from the dense sands is less than 700 kips, we expect that displacement piles can be pushed readily through that layer with a frame that is able to deliver more than 2,500 kips of push force down to bedrock. This installation procedure will essentially provide a load test for every pile to be installed, with the certainty of achieving the ultimate geotechnical capacity needed within bedrock.

Proposed piles include rock piles and an alternative for consolidation sand piles, which will be embedded in the bedrock, and in sand layers above the Old Bay Clay, respectively. Each consolidation sand pile is proposed to be a 9.625-inch outside-diameter push pile embedded 20 to 30 feet in the dense sand above the Old Bay Clay. The consolidation sand push piles are a full-displacement pile and will not create voids during installation that can compromise existing piles. Each rock pile is proposed to be a 13.625-inch outside-diameter push-pile down to 30 feet below the mat, fitted with an inner 9.625-inch outside-diameter push pile that will extend approximately 60 feet into bedrock. Each pile will be installed until it reaches the ultimate geotechnical capacity of 2,000 kips.

Ultimate bond strengths between pile and soil were estimated using typical alpha and beta methods for clays and sands, respectively. Upper- and lower-bound sand pile capacity estimates were calculated based on the relative density of the soil. For the rock piles, ultimate bond strengths between the Old Bay Clay and pile were estimated using available laboratory strength data and alpha values of 0.5. Ultimate rock-to-pile strength was estimated using available test data from micropile tests within the Franciscan Bedrock, and test data from pile load tests around the site within the same bedrock material (see geology section of this memorandum). Table 10.1.2-1 shows the ultimate pile-to-soil/bedrock bond values used in our analyses.

TABLE 10.1.2-1: Ultimate Bond Strength

LAYER	BOND STRENGTH [ksf]
Dense Sands	2-4
Old Bay Clay	1-1.5
Alameda Formation	1.5-2
Bedrock	5.5-9.0

Figure 10.1.2.1-1 shows the estimated ultimate geotechnical capacities for the rock piles. In addition, Figure 10.1.2.1-1 shows the static and dynamic geotechnical capacities of the rock piles. The dynamic (undrained) behavior of the rock piles is assumed to engage the full bonded capacity of the Old Bay Clay and Franciscan Bedrock, while the static (drained) behavior assumes that the normally consolidated Old Bay Clay will not provide axial resistance. Bedrock-structure interaction was developed by normalizing the available micropile test data in similar Franciscan Bedrock (Figure 10.1.2.1-2). Derivation of the soil-structure interaction response to axial loading was performed using the API methodology (2000) formulations for displacement piles. Upper- and lower-bound capacities were provided to LERA and were developed using upper- and lower-bound bond strengths shown on Table 10.2.1.1-1. The backbone curves include the theoretical elastic shortening of the piles. We will provide the sand pile capacities in our final report.

10.1.2.1 Geotechnical Bounding on Proposed Pile Axial Capacities

To capture the variability and uncertainty in the subsurface geotechnical properties, we provided upper- and lower-bound axial capacities of the piles to LERA for their evaluation. The upper-bound and lower-bound axial capacities are based on the bond strength, Table 10.2.1.1-1.

FIGURE 10.1.2.1-1: Rock Pile Capacities

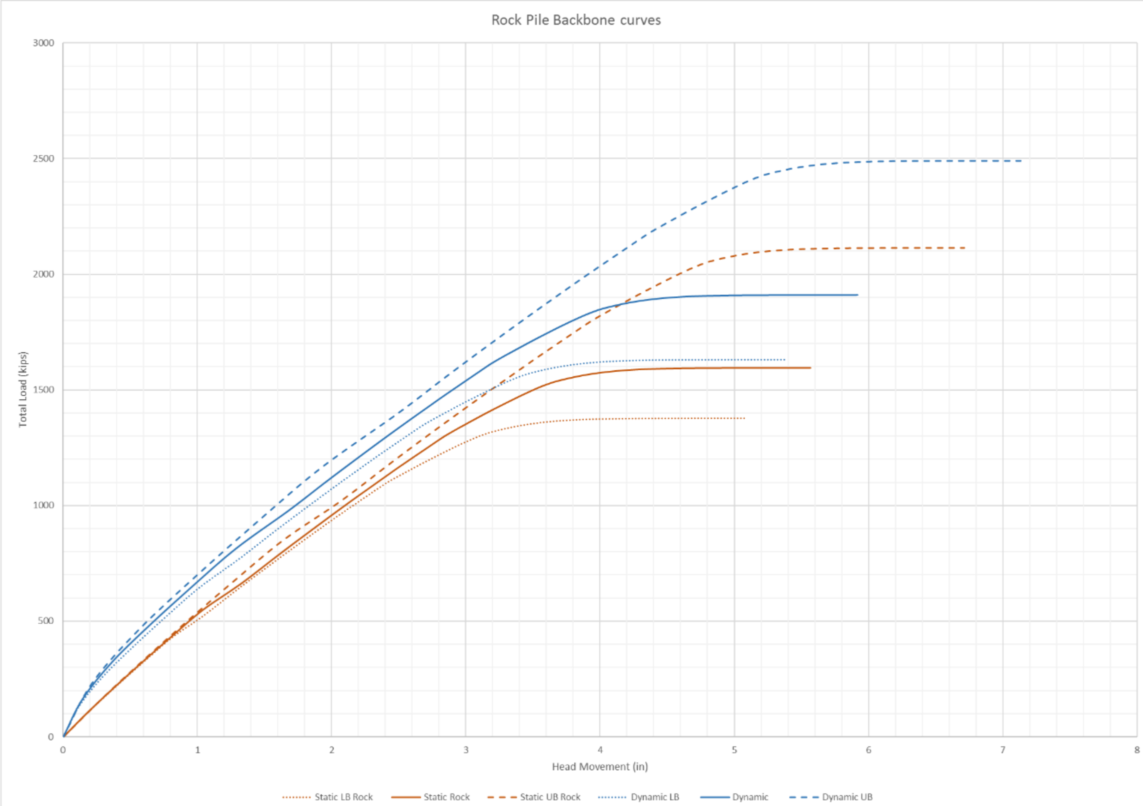
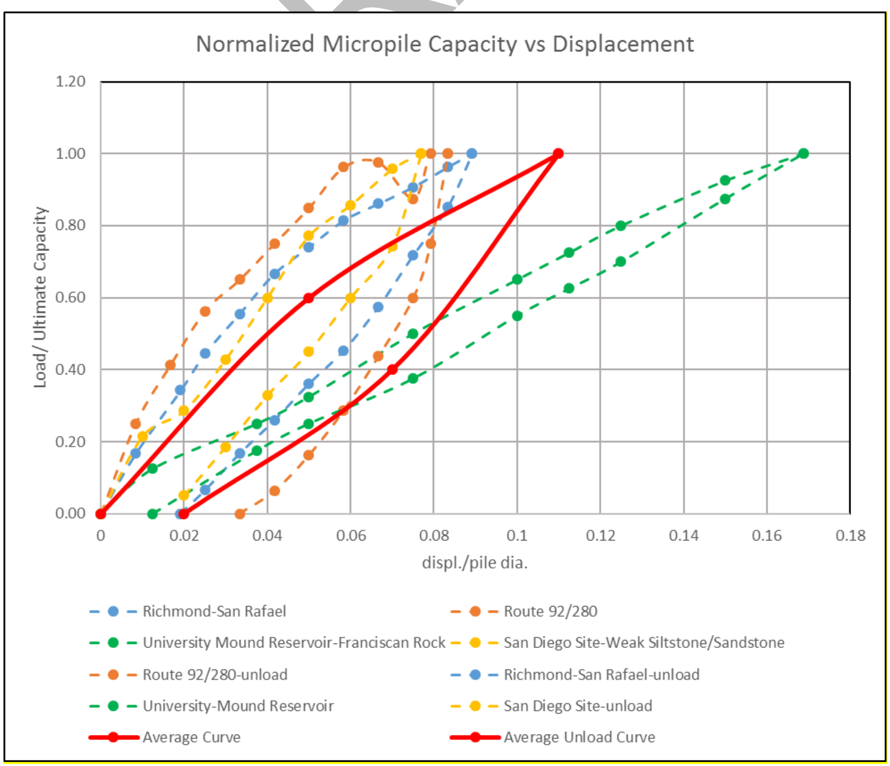


FIGURE 10.1.2.1-2 – Load Displacement Behavior for Push Piles in Bedrock



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10.2 LATERAL PILE CAPACITIES

Lateral resistance against pile movement is a function of soil stiffness. To model soil response to lateral pile movement, the program LPILE 2015 by Ensoft, Inc, was used. Table 10.2.1-1 shows the soil parameters used in the LPILE calculations. Soil properties selected in our model were based on available geotechnical data and our experience with similar soils in the area. To consider the group effects of the pile cap, the geometry and location of each pile was modeled and an average p-multiplier was selected. A p-multiplier of 0.65 was used in the LPILE analysis to account for group effects.

10.2.1 Geotechnical Bounding on Lateral Pile Evaluations

Upper- and lower-bound models were developed based on uncertainty of soil parameters as shown in Table 10.2.1-1. More than 10,000 LPILE runs were performed to evaluate bounding in addition to structural uncertainties.

TABLE 10.2.1-1: LPILE Analysis Parameters

DEPTH FROM TOP OF PILE (FT)	GENERALIZED SOIL TYPE	L-PILE SOIL TYPE	EFF. UNIT WEIGHT (PCF)	FRICTION ANGLE (DEG.)	UNDRAINED COHESION, C (PSF)	STRAIN FACTOR, E_{50}
18	Young Bay Mud	Soft Clay	47.6	-	500-750 (LB) 700-1000 (UB)	0.01 – 0.02
28	Marine Sand 1	Reese Sand	57.6	34	-	-
42	Young Bay Mud	Soft Clay	52.6	-	900-1100 (LB) 1200-1500 (UB)	0.01 – 0.02
52	Marine Sand 2	Reese Sand	67.6	35	-	-
67	Pleistocene Sand 1	Reese Sand	67.6	36	-	-
77	Old Bay Clay Crust	Stiff Clay w/o Free Water	57.6	-	2800-2400 (LB) 3800-3000 (UB)	0.005 – 0.007
177	Old Bay Clay	Stiff Clay w/o Free Water	50.6	-	2400-4000 (LB) 3000-5000 (UB)	0.005 – 0.007
202	Lower Old Bay Clay	Stiff Clay w/o Free Water	57.6	-	4000-4700 (LB) 5000-5700 (UB)	0.005 – 0.007
227	Alameda Formation	Reese Sand	67.6	35	-	-
248	Franciscan Bedrock	Reese Sand	67.6	43	-	-

Since liquefaction potential is calculated in the interbedded layers of sand within the Young Bay Mud, a sensitivity analysis was performed to understand the impact of liquefaction of deeper layers on lateral pile performance. A p-multiplier of 0.2 was used for layers that may experience liquefaction during the design earthquake (about 30 feet from the bottom of the mat) with an average thickness of 5 feet. Results indicate negligible impacts to the lateral performance of the existing piles.

The structural inputs for LPILE were provided by LERA as non-linear bending moments as a function of curvature for all piles. In addition, boundary conditions, ranging from axial loads to angle of head rotation, were provided by LERA to account for multiple scenarios. A sample of the LPILE results provided to LERA is shown in Figures 10.2.1-1 and 10.2.1-2.

FIGURE 10.2.1-1: Existing-Pile Lpile Results using Lower-Bound Soil Parameters

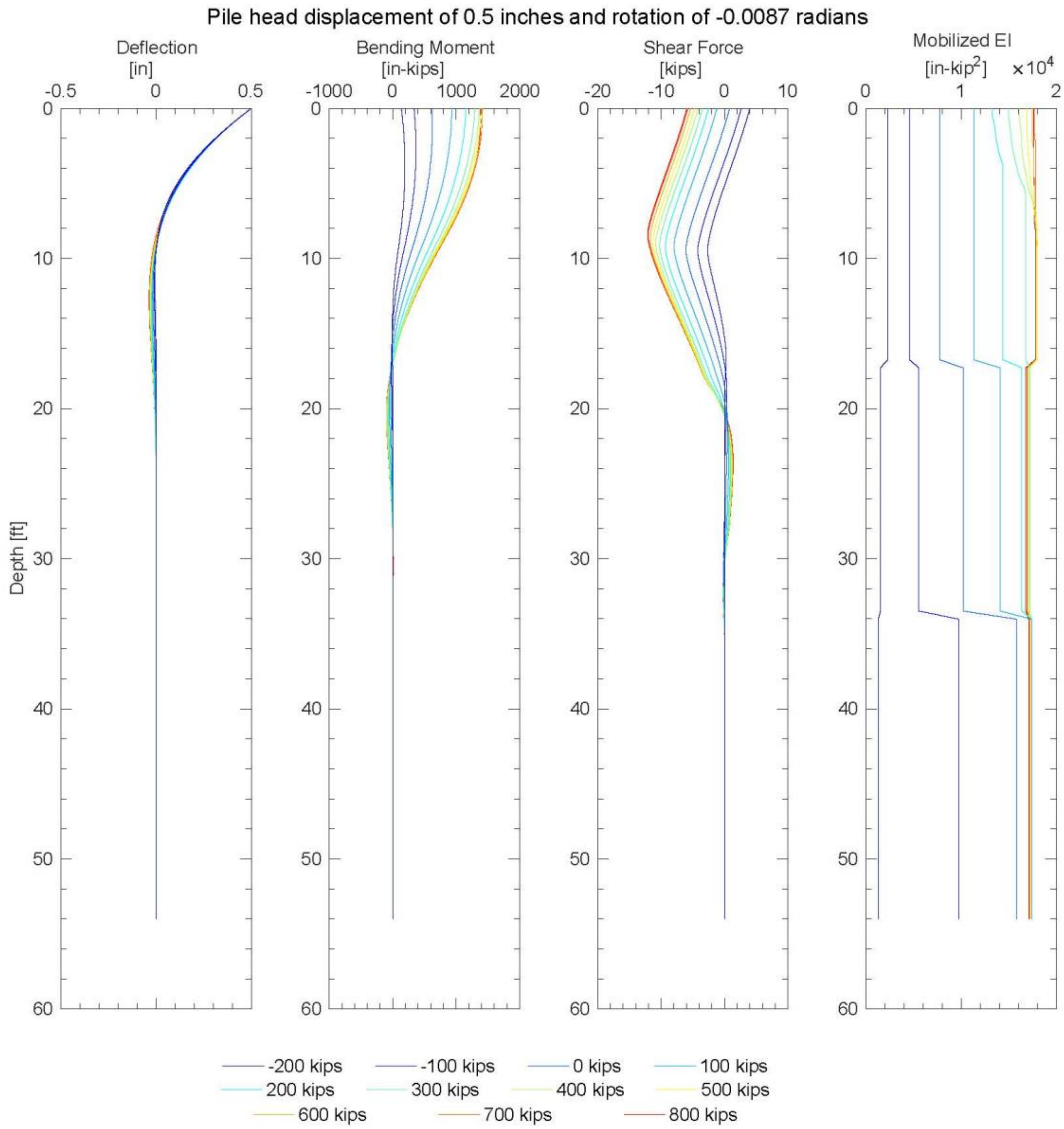
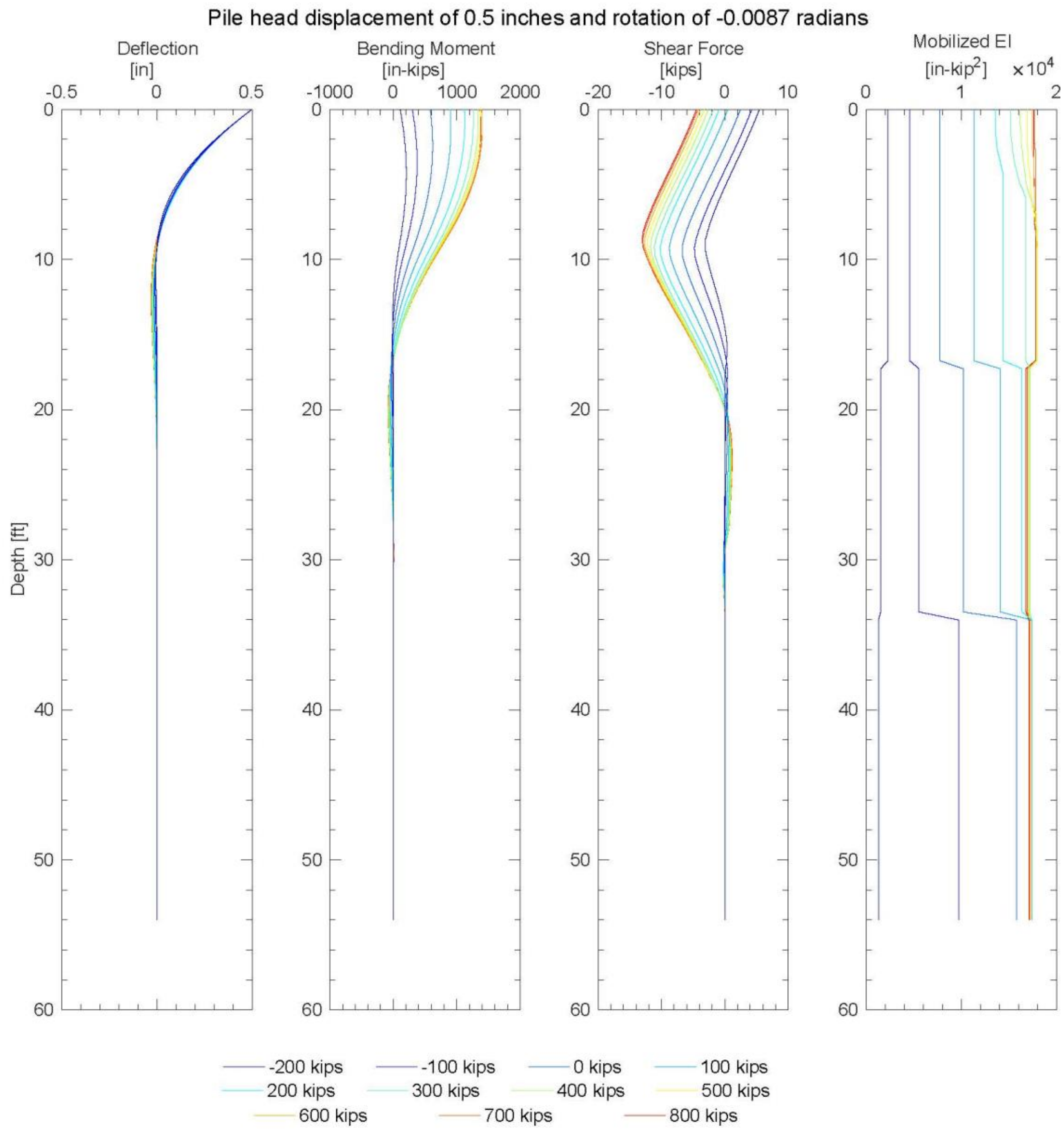


FIGURE 10.2.1-2 – Existing Pile Lpile Results using Upper-Bound Soil Parameters



10.3 BASEMENT WALL LATERAL RESISTANCE AND SEISMIC EARTH PRESSURES

Lateral resistance analyses of the basement walls were developed using the computer program LPILE 2015 and NCHRP 611 notes on p-y springs for wall elements (NCHRP 611, 2008). The p-y springs were calculated for the north and west basement walls and the elevator pit in all directions (the south wall resistance is currently under development). Springs were located at requested structural locations for modeling purposes at 0.5 feet, 9 feet, 16 feet, 21 feet and 32 feet below the top of the basement wall (Figure 10.3-1). Liquefied soils were included with p-y multipliers of 0.2 for portions of the walls with loose granular soils below the groundwater table. Buoyancy forces and additional surcharge for the cantilevered section were added.

The geotechnical uncertainties in these spring values are currently being evaluated, and will be refined during the design optimization phase.

FIGURE 10.3-1 – Location of Passive Springs at Basement Wall

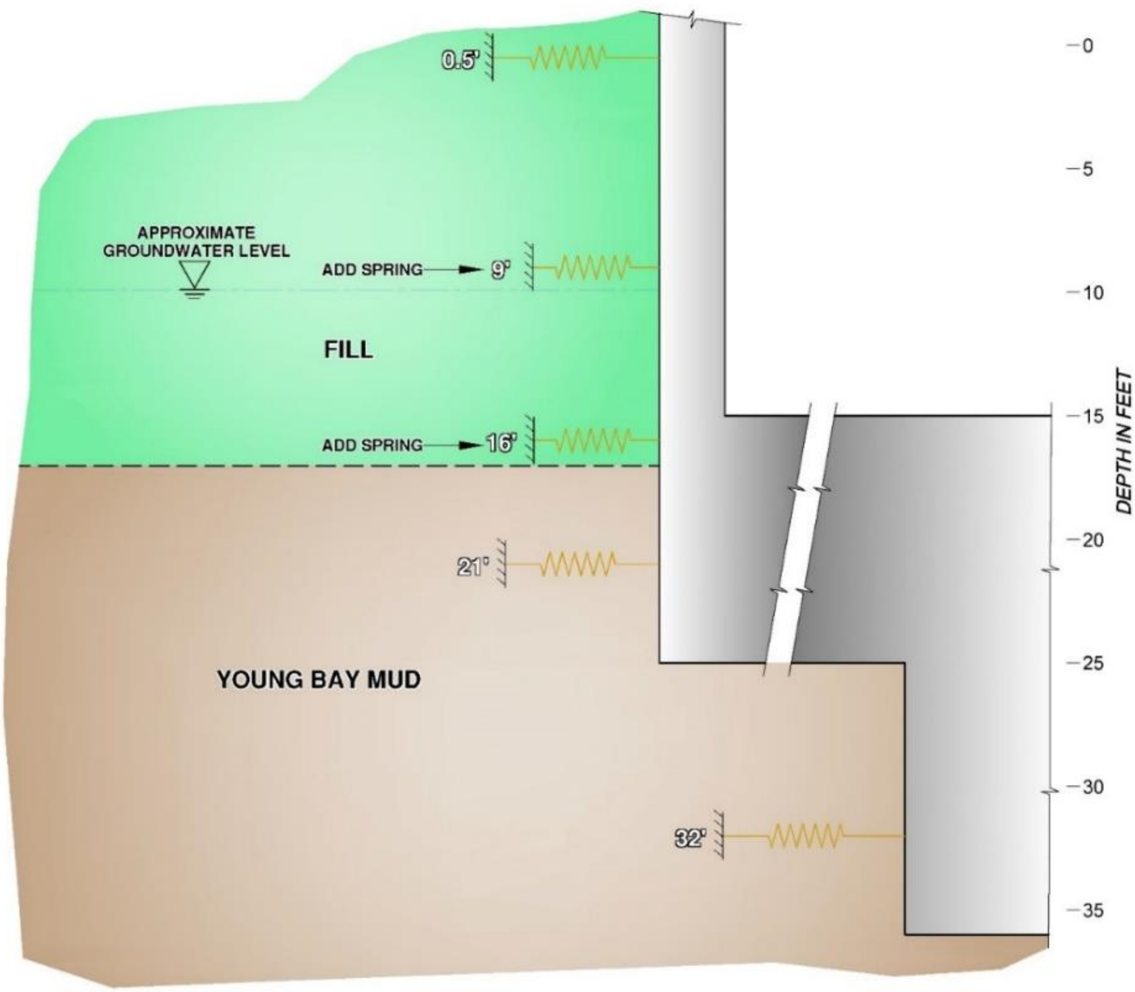
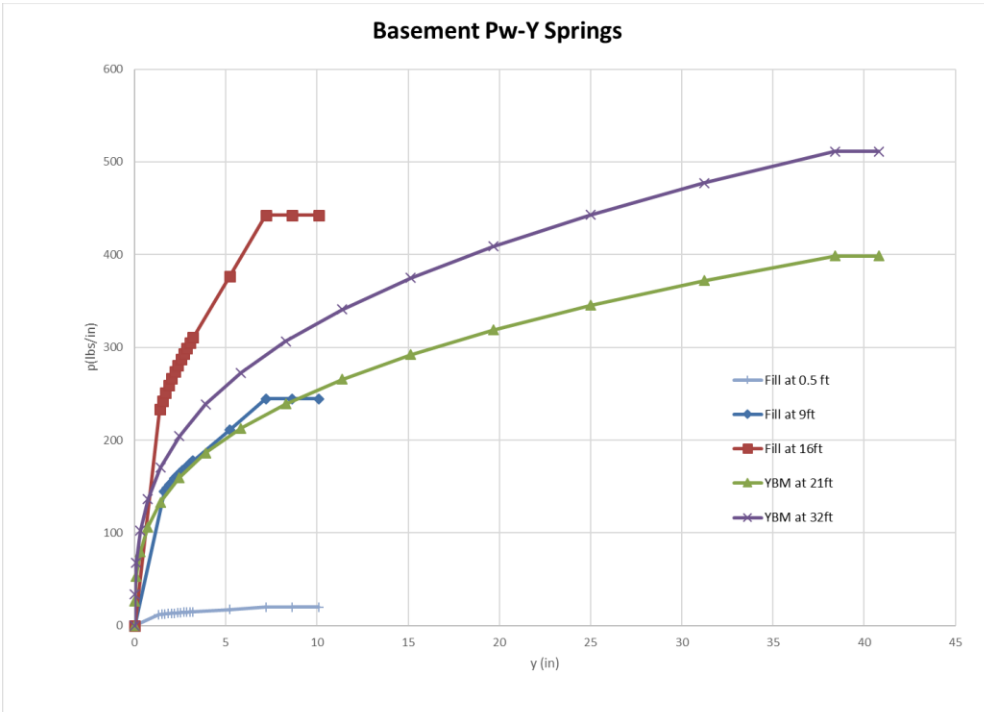


FIGURE 10.3-2 – Passive Springs at Basement Wall

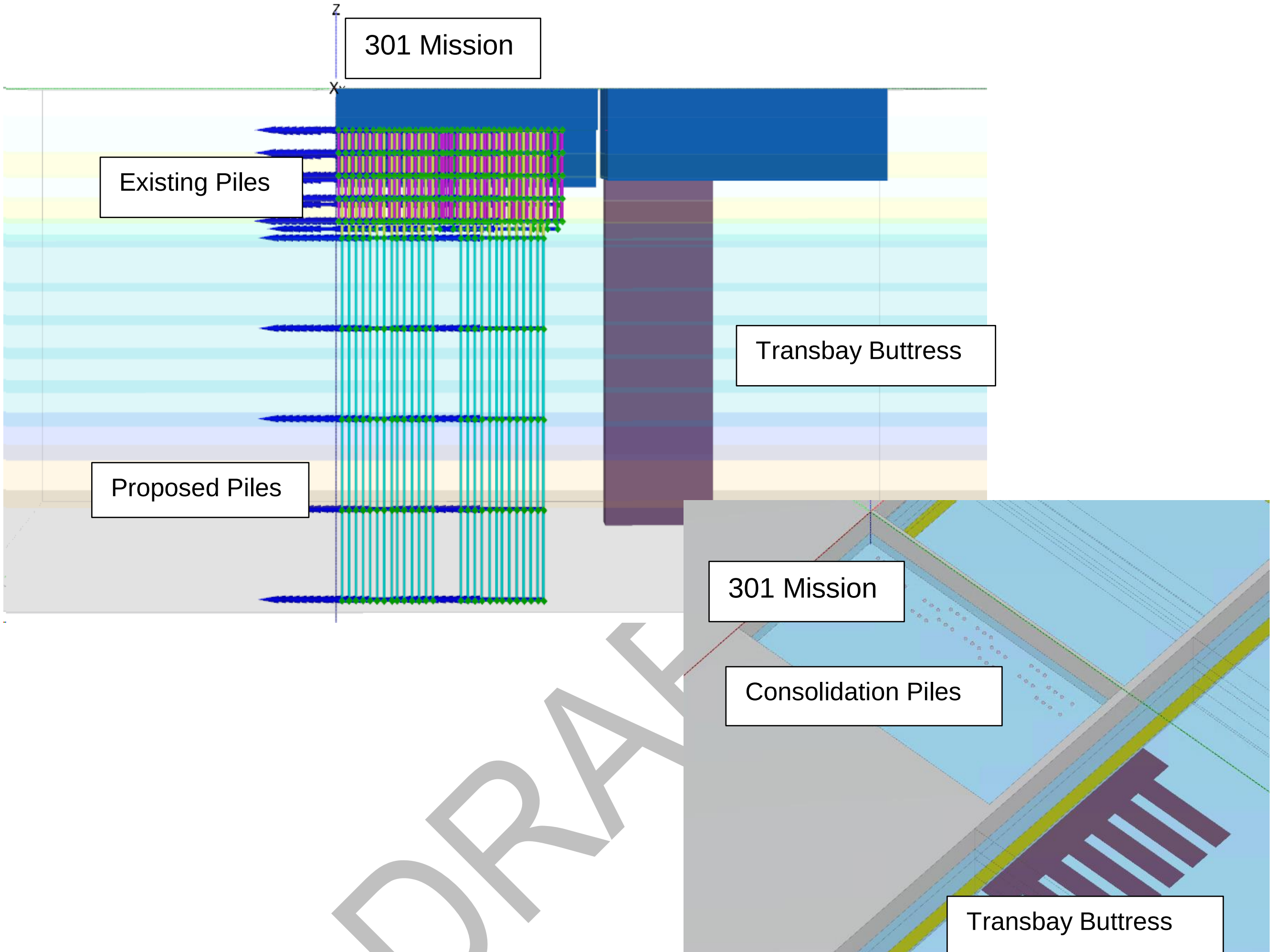


Based on the MCE seismic-hazard level, the seismic active increment is recommended to be 26 pounds per cubic foot (pcf) above the groundwater table and 13 pcf below the groundwater table, modeled as a triangular distribution (Lew, 2010)

11.0 SETTLEMENT MODELING

To understand the behavior of the proposed foundation elements, analyses were performed to evaluate the current states of stresses in the subsurface soils and settlements from imposed loads. The primary mode of settlement modeled consisted of consolidation of the Old Bay Clay layers (OBC). Modeling software used in this analyses included the 3-dimensional finite-element program, Plaxis 3D. As-built foundation conditions and proposed foundation systems developed by this design team were modeled. The model is shown in Figure 11.0-1.

Figure 11.0-1. Plaxis 3D model



11.1 SOIL PROPERTIES

Soil properties incorporated into the Plaxis 3D model were based on available laboratory tests, which were reviewed and interpreted by ENGEO. The review and re-interpretation of laboratory and boring test data included unit weight, shear strength, void ratio, recompression and primary consolidation ratios, and over-consolidation ratios. Typical corrections were performed on consolidation test results, and maximum past pressures, recompression, and virgin compression indices were obtained.

In addition, stiffness parameters for the subsurface soils were evaluated based on the available relative density information, undrained shear strength of the soil, overconsolidation ratio, and plasticity indices. Young's modulus values for clayey soils were derived from plate load testing charts from Duncan & Buchignani (1976). Sand-layer stiffness values were estimated using engineering judgment and typical ranges provided by the U.S. Department of the Navy and Bowles (1982).

Median soil parameters used in our analysis are shown in Table 11.2-1. Structural element parameters used in the model are consistent with parameters used by LERA.

11.2 CALIBRATION ANALYSIS

Classic Terzaghi one-dimensional (1D) consolidation analyses were performed and compared to baseline results from finite-element modeling. To confirm excess pore pressures due to loading within the 3D model, a comparison was performed with 1D stress distribution using the Boussinesq theory. For this comparison, a semi-infinite, elastic, isotropic, and homogenous foundation at the ground surface was assumed within Plaxis 3D. Using both 1D and 3D analyses, total stresses for an initial state (no structure) and with the imposed building load were calculated and compared. As shown in Figure 11.2-1, the one-dimensional analysis trend is very similar to the total stress at a point at the center of the mat foundation computed with Plaxis 3D before and after loading. This calibration was the first step in creating a reliable model for settlement and anticipated loading for the model.

TABLE 11.2-1 – Plaxis Parameters

	NAME	Thickness (ft)	Unit Weight Ysat (pcf)	initial eo	Ccep	Cr/Cseps	OCR (initial before building construction)	Youngs (ksf)	Cohesion (ksf)	Cv ft2/yr	Kxyz ft/day
Soil Layer 1	FILL	17	120	0.5	"-"	"-"	"-"	600	"-"	"-"	0.28
Soil Layer 2	Young Bay Mud	22	115	1.7	"-"	"-"	"-"	295	"-"	"-"	0.000025
Soil Layer 3	Upper Marine Sand	14	120	0.5	"-"	"-"	"-"	900	"-"	"-"	0.28
Soil Layer 4	Lower Young Bay Mud	14	115	1.7	"-"	"-"	"-"	350	"-"	"-"	0.000025
Soil Layer 5	Lower Marine Sand	13	120	0.5	"-"	"-"	"-"	1200	"-"	"-"	0.28
Soil Layer 6	Marine Sand/Clay	10	125	0.5	"-"	"-"	"-"	2000	"-"	"-"	0.28
Soil Layer 7	Old Bay Clay	4	112	1.22	0.315	0.05	2.46	977	2.0	39.0	0.000020
Soil Layer 8	Old Bay Clay	26	112	1.22	0.315	0.05	1.63	977	2.0	39.0	0.000020
Soil Layer 9	Old Bay Clay	20	112	1.22	0.315	0.05	1.63	1165	2.8	39.0	0.000020
Soil Layer 10	Old Bay Clay	20	112	1.22	0.315	0.05	1.63	1315	3.6	39.0	0.000020
Soil Layer 11	Old Bay Clay	20	112	1.22	0.315	0.05	1.63	1461	3.6	39.0	0.000020
Soil Layer 12	Old Bay Clay	20	112	1.22	0.315	0.05	1.63	1600	3.8	39.0	0.000020
Soil Layer 13	Old Bay Clay Unit II	20	120	1.22	0.315	0.05	1.63	1840	4	39.0	0.000020
Soil Layer 14	Gravels	28	125	0.5	"-"	"-"	"-"	4000	"-"	"-"	2.8
Soil Layer 15	Bedrock	32	130	0.5	"-"	"-"	"-"	10000	"-"	"-"	0.28

FIGURE 11.2-1: Total Stress Comparison (Boussinesq and Plaxis 3D)

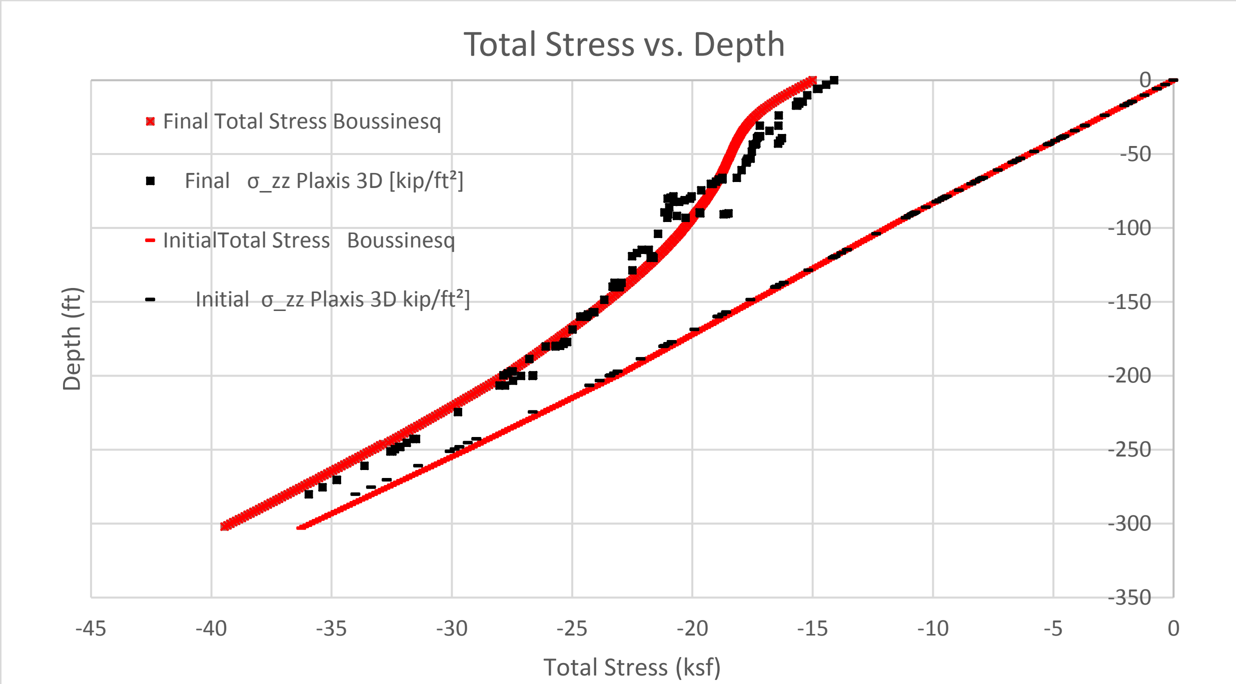
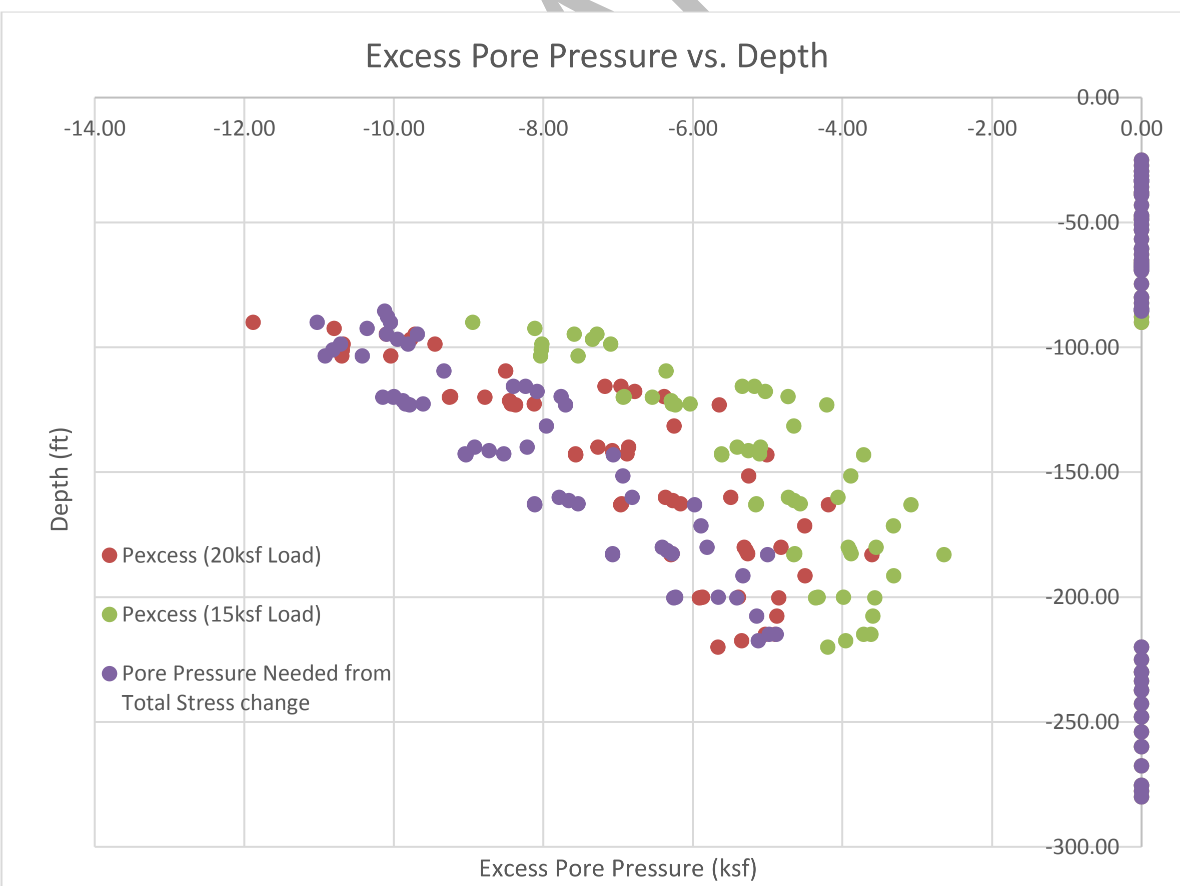


FIGURE 11.2-2: Excess Pore Pressure Calibration from Total Stress Change



Following calibration of the total stress distribution from imposed loading, calibration of the excess pore pressures generated from loading was performed. In theory, stresses from load placement will create excess pore pressure instantaneously equal to the amount of load distributed in the Old Bay Clay strata. Consolidation is thus the change in volume from expulsion of the excess pore pressure as a result of stress increase. To calibrate settlements calculated using Plaxis 3D, generated pore pressures with the 1D analysis and pressure generated by Plaxis 3D were compared. Excess pore pressures generated in Plaxis 3D were initially 15 to 30 percent, depending on depth, less than that anticipated from load placement. To match anticipated pore pressures, the load imposed was refined and regenerated as shown in Figure 11.2-2. Based on our discussions with the Plaxis development team, due to numerical issues, some percentage of the initial excess pore pressure is not present in the analysis. This calibration analysis provided a representative method to replicate in-situ pore pressures based on actual building loads and provided an appropriate model for settlement behavior of the OBC.

11.3 SOIL MODELS

The "Soft Soil Model" was used in our Plaxis 3D analysis. This model considers stress-dependent stiffness, primary loading and unloading-reloading, preconsolidation stress memory and failure behavior using the Mohr-Coulomb criterion. This model was used for all OBC layers in consolidation phases. Since the main purpose of this model was to predict the behavior of the OBC, the other soils were modeled with a combination of Mohr-Coulomb and linear-elastic models. See Table 11.0-1 for parameters used.

11.4 STAGING AND LOADING

Staging of construction, structural loading, and outside effects related to the building of 301 Mission were modeled in approximately 30 stages. Timing of the phases was derived based on available documentation. Assumed construction sequencing built into the model and adjacent excavations was also modeled to evaluate the overall OBC behavior.

Since this design effort did not include a detailed analysis of settlement cause, in order to calibrate the settlement model with the observed settlement, external effects following building construction were modeled by changing groundwater elevations. Secondary compression (or creep) will be evaluated in the optimization phase of this design.

Figures 11.4-1 through 11.4-3, show that the Plaxis 3D model closely matches the observed settlement of the building. This provides the basis for predictions of the performance of the proposed design.

FIGURE 11.4-1: Settlement Measured Compared to Plaxis 3D Model

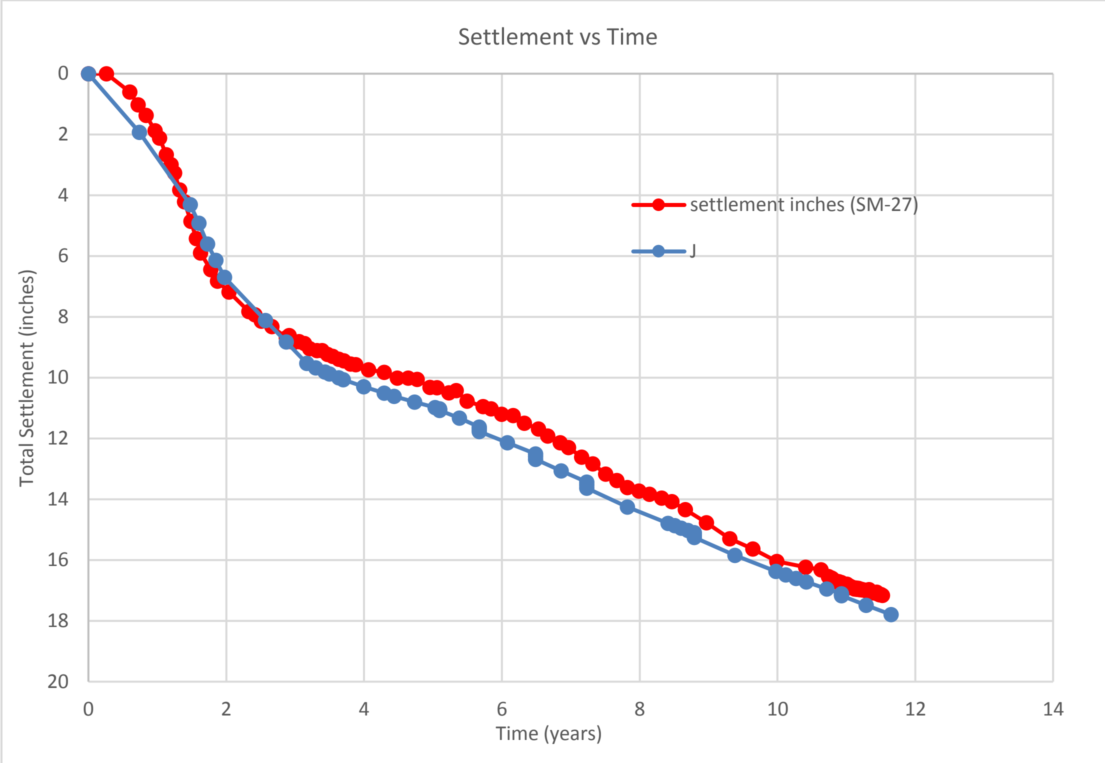


FIGURE 11.4-2: Settlement versus Time on Select Points on Mat

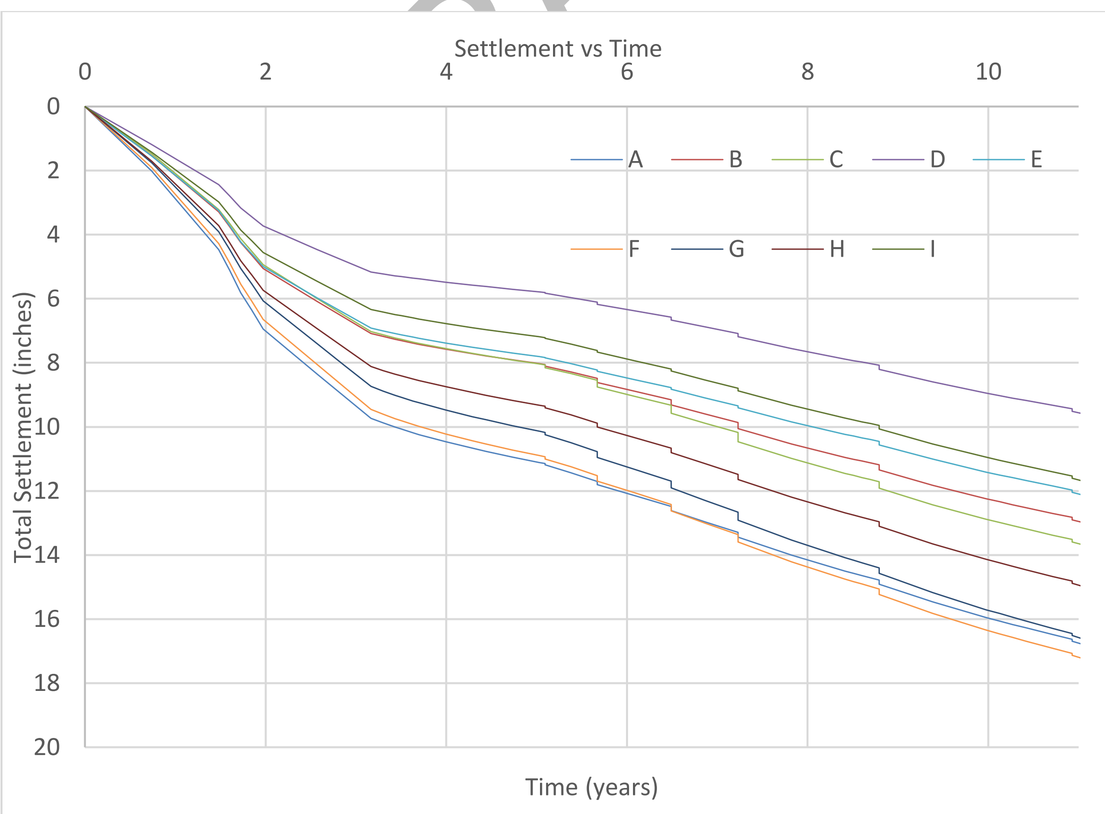
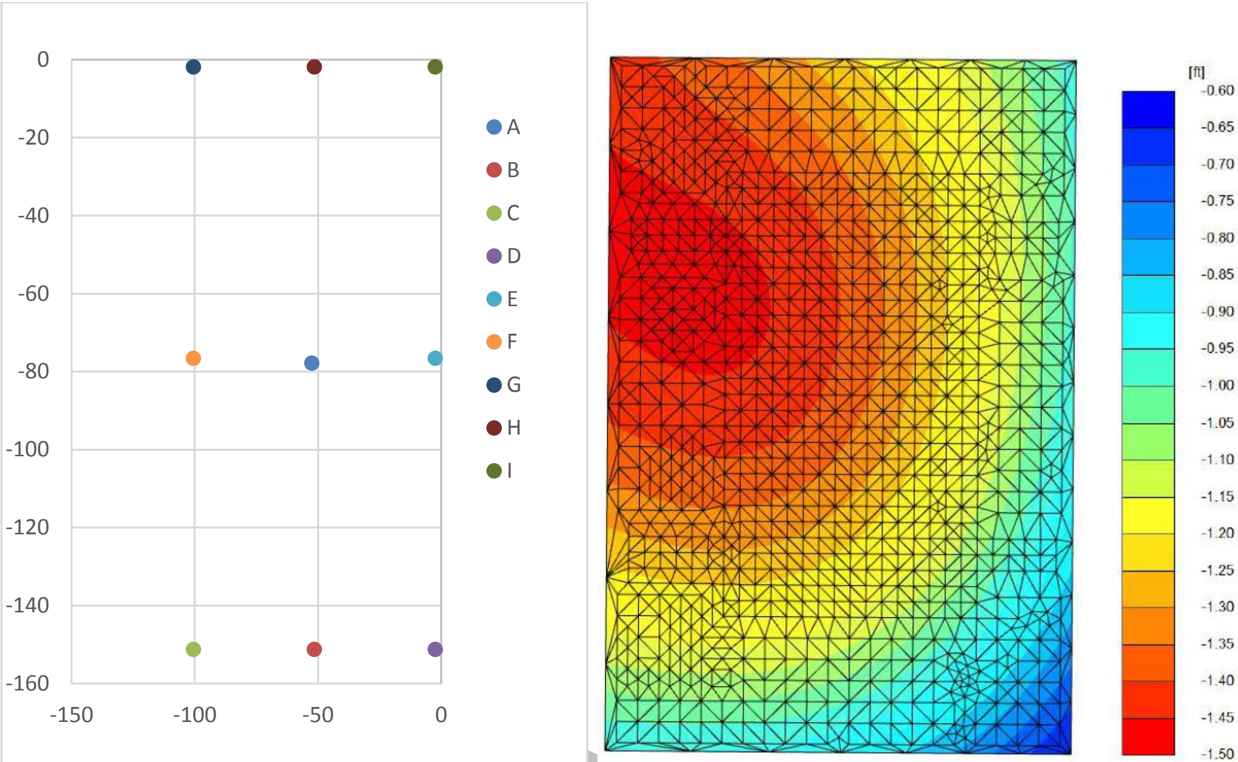


Figure 11.4-3 –Location of Points on the Mat and Heat Map of Vertical Settlements to Date



11.5 EVALUATION OF PROPOSED DESIGN

As described above, proposed foundation improvements at 301 Mission include construction of "rock" piles to bedrock and potential tilt correction using "consolidation" piles located on the east side of the structure. Rock piles are proposed to be embedded approximately 60 feet into bedrock and consolidation piles approximately 30 feet into Holocene and Pleistocene sands.

The first stage of the retrofit (excluding tilt correction) involves preloading rock piles on and transferring a portion of the building load to bedrock (approximately 5 ksf for the lower-bound scenario). In the Plaxis 3D analysis, the unloading from the existing piles to the new rock piles was modeled simply by unloading the mat. Load reductions were modeled based on LERA's estimate of load transfer from existing piles to new piles. For the final design, we will include the new push piles in the 3D model to calibrate the load transfer and preloads needed. Based on the symmetric unloading of the current retrofit approach, the analyses show that the Old Bay Clay will reliably have stress history equivalent to an over-consolidation ratio (OCR) greater than 1.0. The OCR changes based on our 3D evaluation are shown in Figure 11.5-1

There are multiple benefits of a symmetrical unloading of the Old Bay Clay under the tower. Based on settlement analyses and existing piezometric data, parts of the Old Bay Clay still have excess pore pressures. Only by unloading the Old Bay Clay symmetrically, can we reliably relieve the excess pore pressures and bring the Old Bay Clay to an OCR greater than 1.0 under the entire mat footprint. In addition, as shown in the literature (Ladd and others, 1994), by unloading, the secondary compression coefficient is reduced by as much as half and the coefficient becomes non-linear, thereby, reducing even further the secondary settlements for overconsolidated clays. The secondary compression under the lower-bound unloading of the building is calculated to be less than 2 inches 50 years after completion of the retrofit. Since the retrofit piles will serve to unload (equivalent to surcharge removal) the entire footprint of the mat, we expect that the Old Bay Clay will rebound (as shown in Exhibit 11-5-2 as a schematic), making the net total settlement after 50 years negligible.

The actual load transfer from existing piles to new rock piles has been modeled in Plaxis 3D. As of this writing, the analysis of all the rock piles is not complete. This analysis will be completed during final design. The load-transfer model in Plaxis is described below and the results of the proof of concept are shown.

FIGURE 11.5-1: Old Bay Clay Stress History Under the Center of the Tower

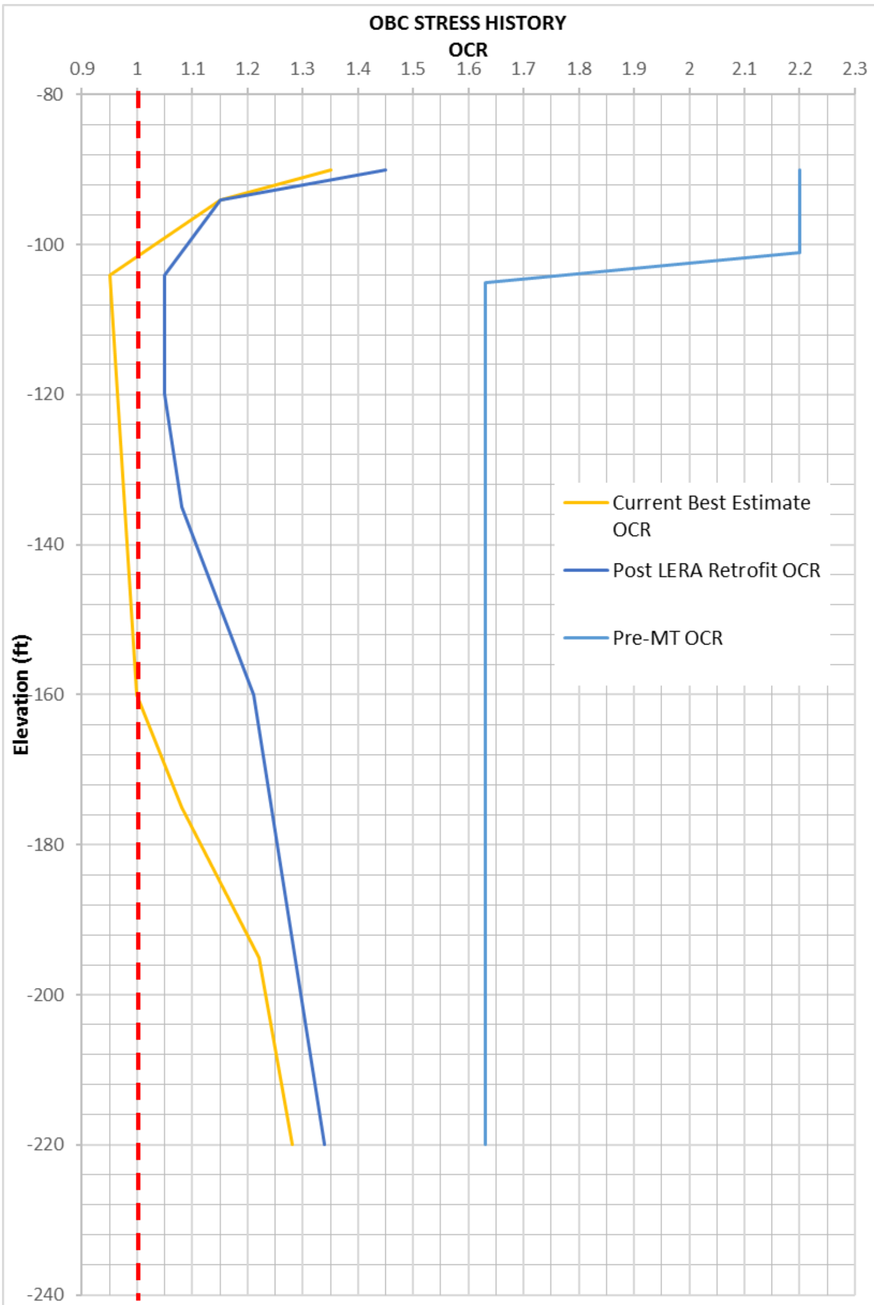
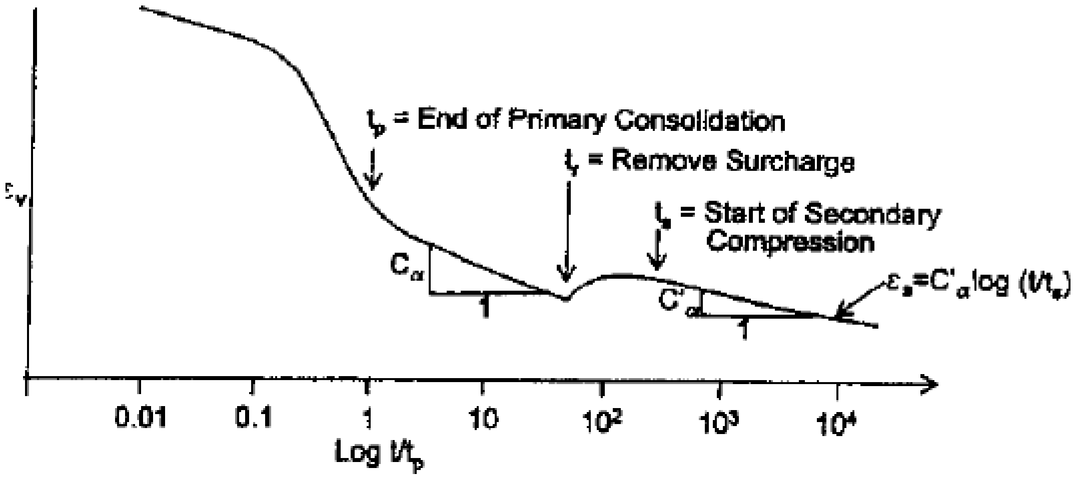


FIGURE 11.5-2: Effects of Unloading on Secondary Compression (Ladd, 1994)



11.5.1 Rock Piles Load Transfer

Additional modeling was performed to confirm the load transfer from the existing piles to the rock piles socketed in bedrock. Our proof-of-concept model consisted of a simplified model with four existing piles and one rock pile at the center. The rock pile was modeled with an anchor to the top of the OBC, connected to an embedded beam row extending from this node to 60 feet into rock (bonded zone). A uniform arbitrary load was applied at the mat level and additional phases of loading were added to the model before and after rock-pile placement. Prior to rock-pile construction, load is taken by the existing piles, which is observed to be equal for each existing pile. Pre-stressing of the proposed rock pile is achieved by adding a load to the anchor. As shown in Figures 11.5.1-1 and 11.5.1-2, load is removed from the existing piles and transferred to the rock pile by this preload modeling method.

FIGURE 11.5.1-1: Plaxis 3D Model Schematic

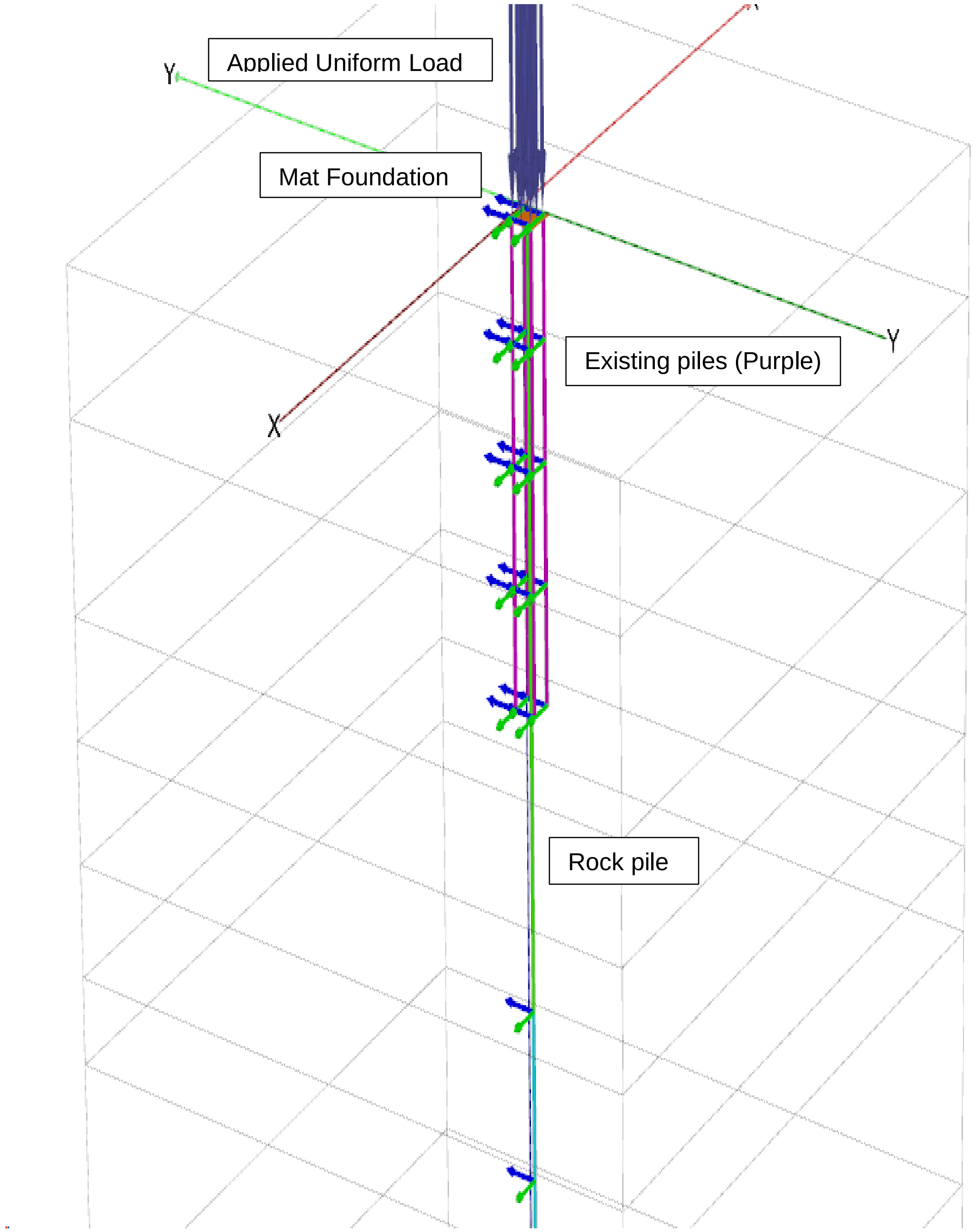
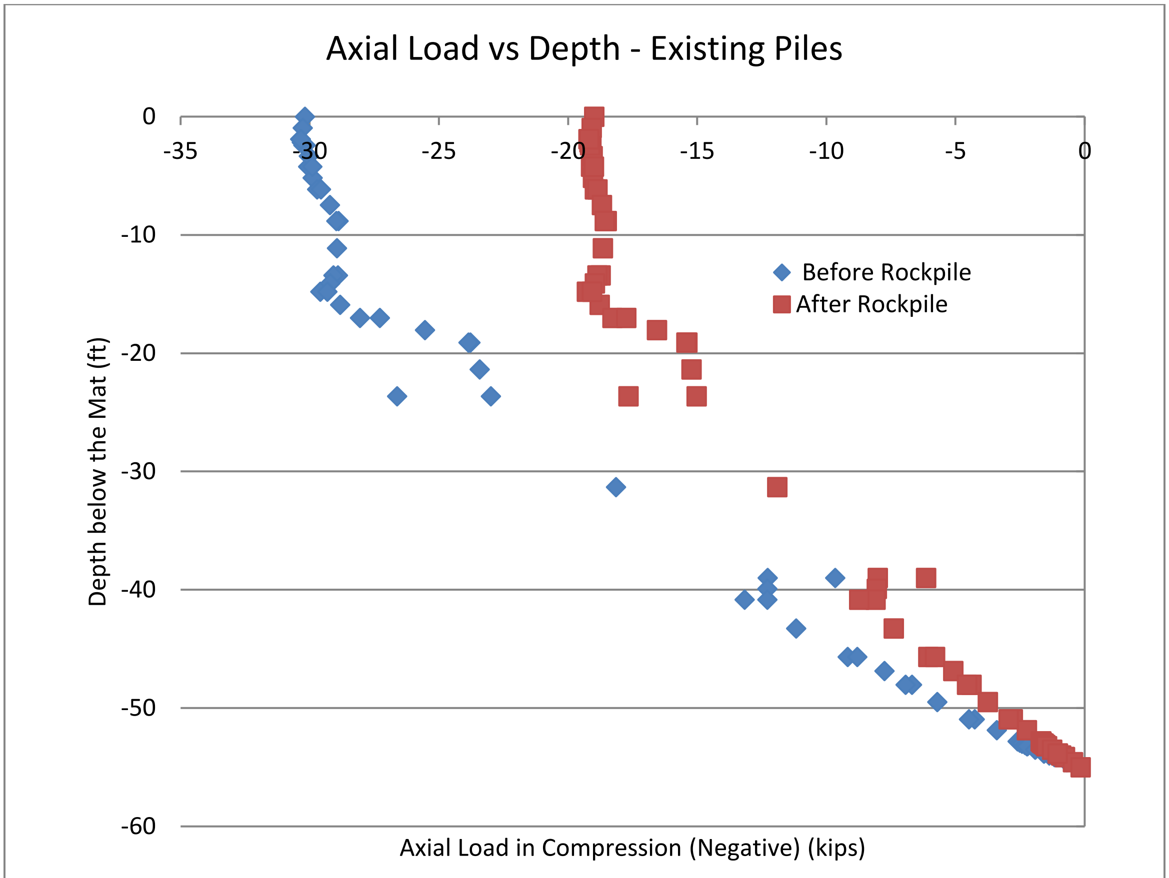


FIGURE 11.5.1-2: Axial Load vs. Depth Before and After Rock Pile



11.5.2 Load Transfer from Old Bay Clay to Bedrock

To model pore pressure changes and observe load transfer from the OBC to the underlying bedrock, a two-dimensional Plaxis (2D) model was used. This model assumed that the majority of the OBC is close to a normally consolidated state. Initially, the model loads the top of the new rock pile to 1,000 kips. Undrained results from this model indicate that load transfer to the Old Bay Clay occurs initially. However, during the consolidation phase, the majority of the load is directly transferred to the bedrock. The estimated time of load transfer to bedrock (excess pore pressure dissipation from the OBC) is 7 days. Figures 11.5.2-1 and 11.5.2-2 show the described process.

Figure 11.5.2-1: Undrained and Drained Load Shed from Old Bay Clay to Bedrock

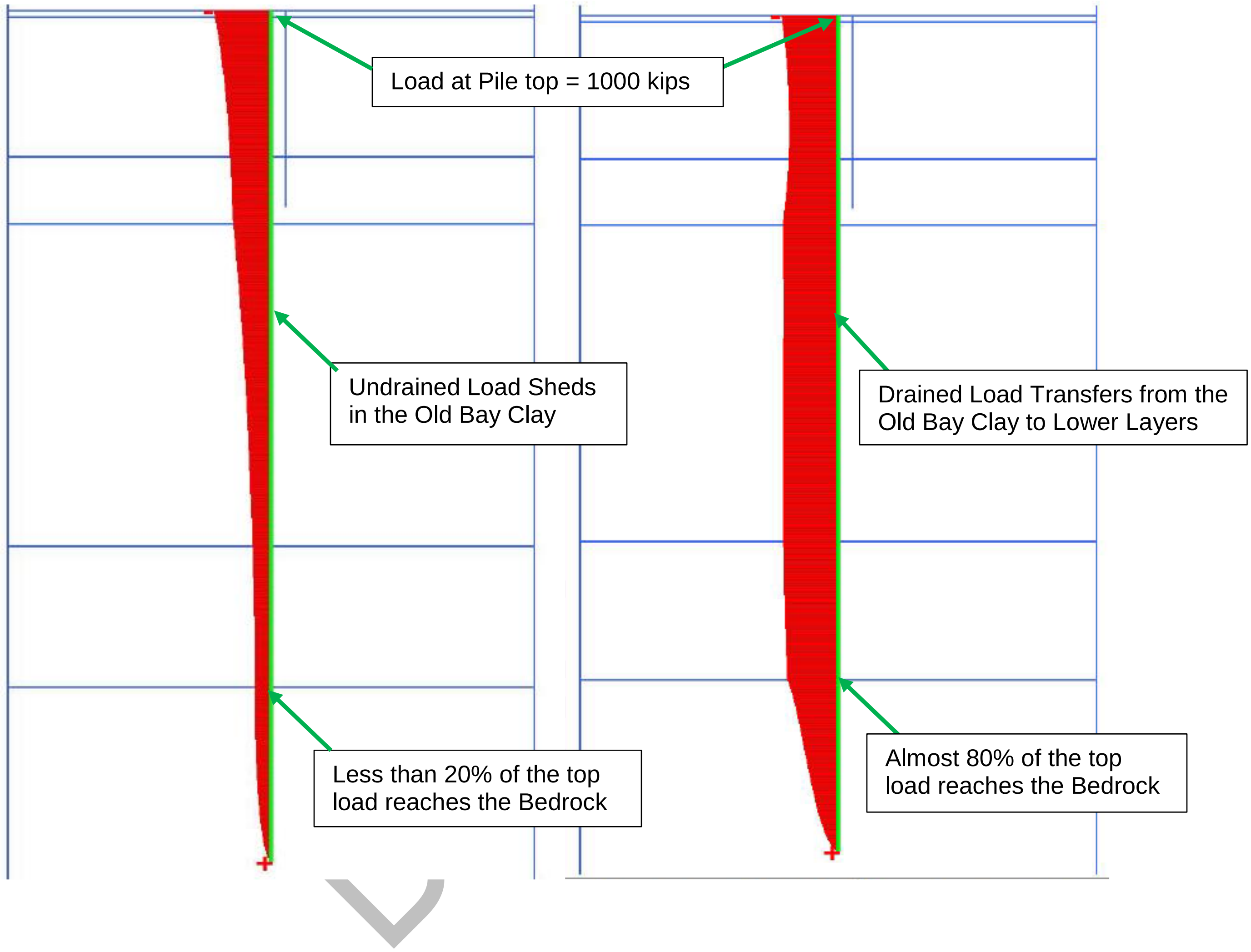
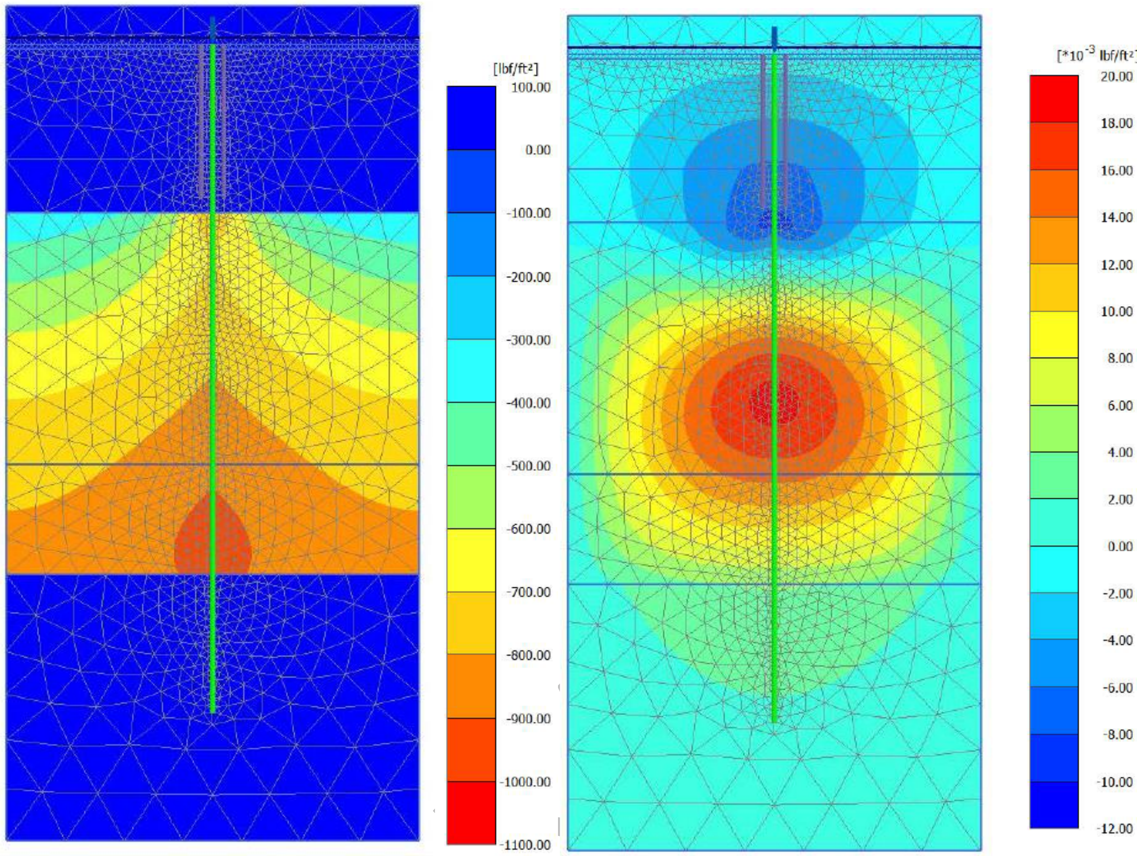


FIGURE 11.5.2-2: Excess Pore Pressure during Undrained Loading (Left) Drained Loading (Right)



11.5.3 Consolidation Piles

With the tilt-correction alternative east-side consolidation sand piles are proposed in order to reduce westward tilt of the structure. Modeling of the consolidation piles consisted of embedded beam rows (typically used for piles) located in sets of two at proposed positions. Loading of each pair of sand piles will be generated with a load frame applying tension to a rock pile between the two sand piles. Modeling consisted of two sand piles 1 foot below the mat, since the subject piles are planned to be detached such that the load transfer will bypass the mat. Consolidation sand piles were modeled to extend approximately 5 feet from the top of OBC. The majority of the resistance will come from the dense sands over the OBC. Point loads were placed at the top of the embedded beam row to simulate the applied load from the consolidation frame. Figure 11.5.3-1 shows the model of the consolidation piles.

As shown in Figure 11.5.3-2, a point set at the mat level on the east side shows increased settlement rates from additional loading. Similar to the calibration performed for the mat loading, calibration of the load increase was performed to confirm realistic pore pressure generation. Figure 11.5.3-3 shows the relative increase of excess pore pressures before and after consolidation pile installation.

FIGURE 11.5.3-1: 3D Model looking North from the Bottom (Left)
3D Model looking in Plan View (Right)

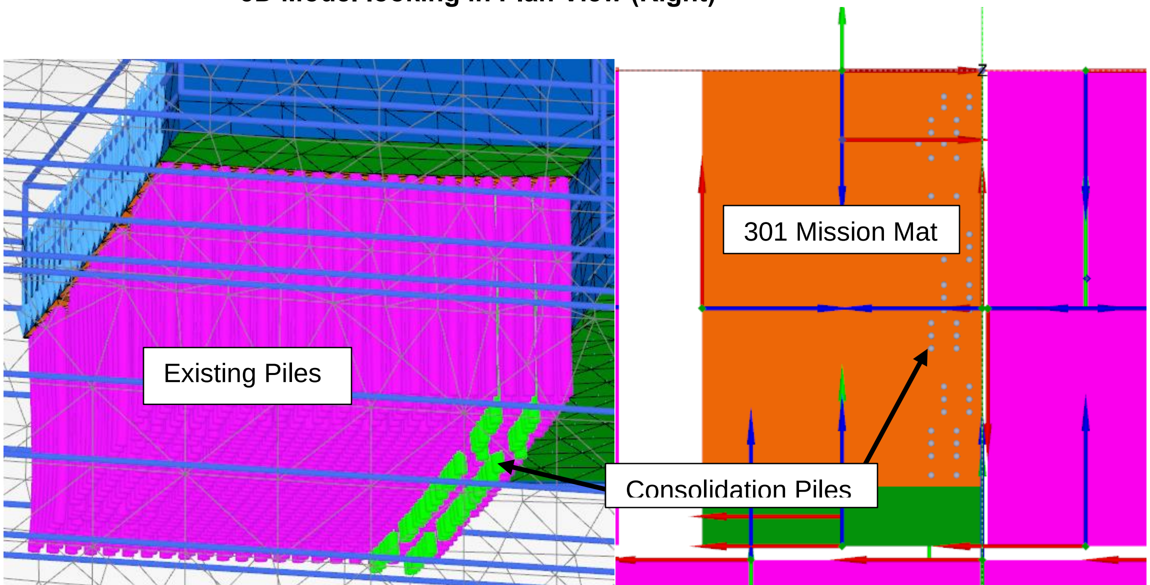


FIGURE 11.5.3-2: Predicted Settlement at Point E on the Mat
from the Start of Consolidation Frames

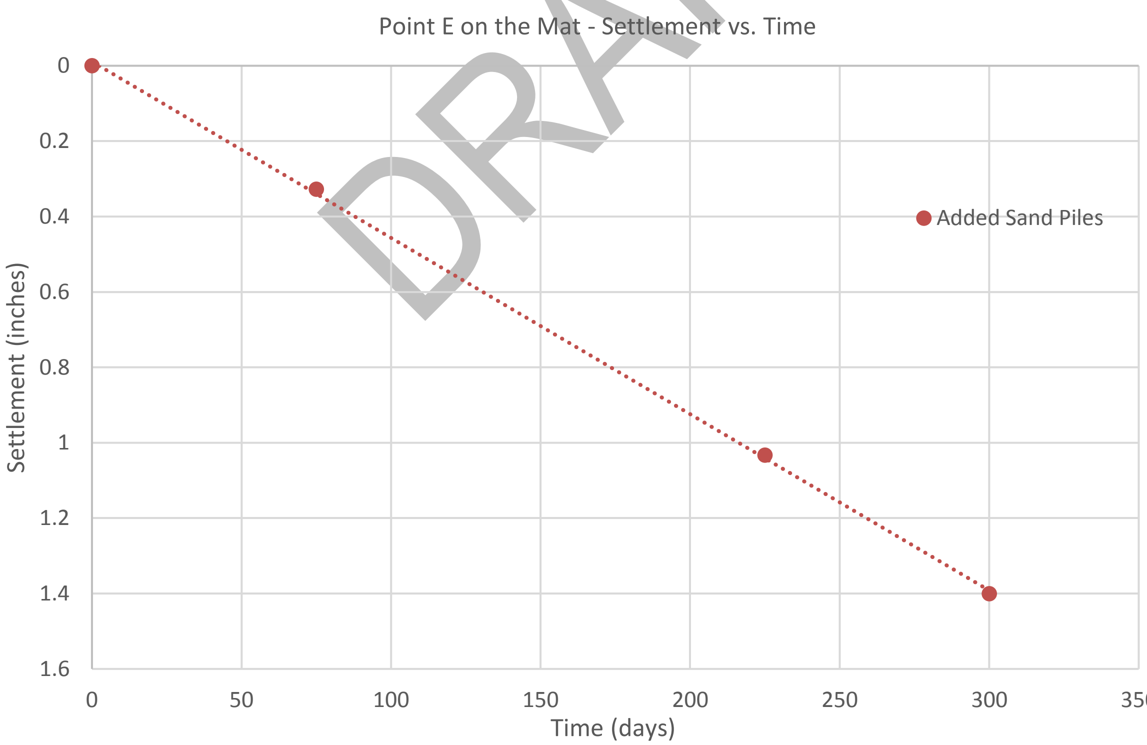
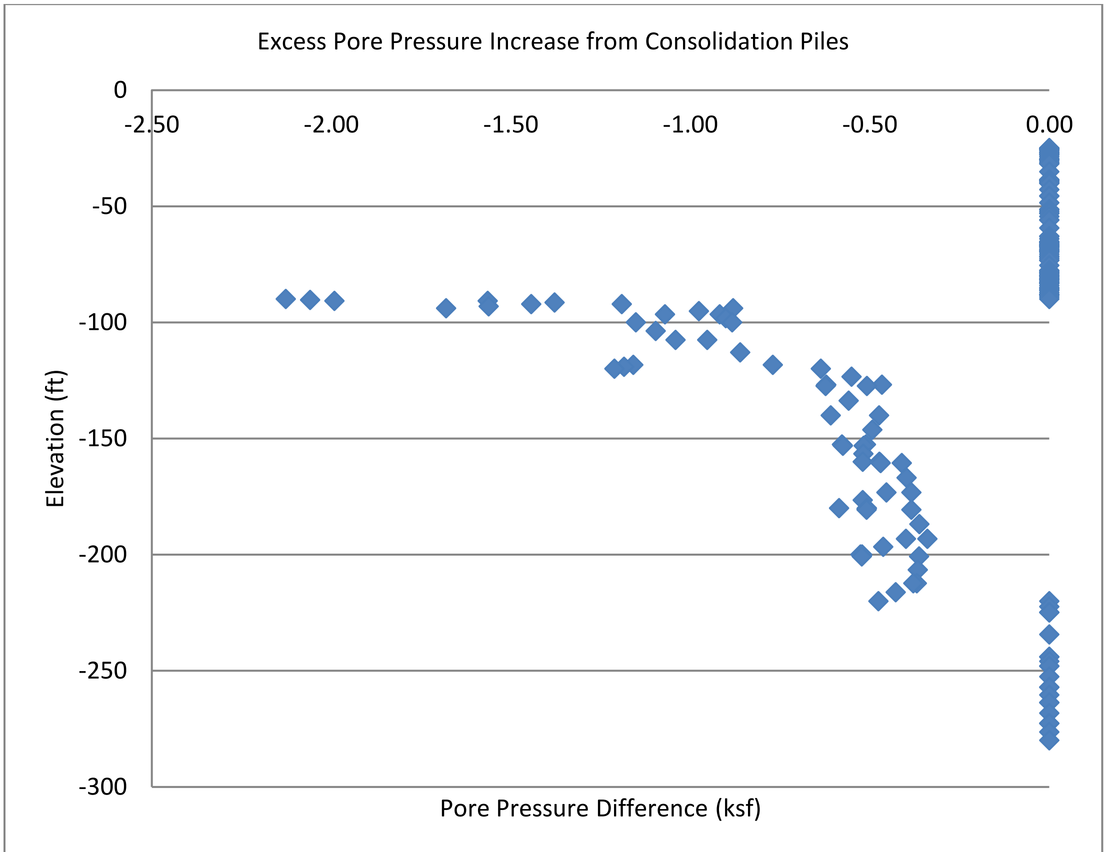


FIGURE 11.5.3-3: Increase in Excess Pore Pressures of the Old Bay Clay due to Consolidation Piles



12.0 CLOSING

This letter provides a summary of the geotechnical basis for the retrofit design. ENGEO is currently working to optimize and improve various aspects of this geotechnical analysis. The outstanding push-pile load test, modeling and additional bounded analyses will provide additional information for final design of the retrofit.

Prepared By: Pedro Espinosa, GE

Reviewed By: Uri Eliahu, GE

Attachment: Appendix A – Boring Logs

APPENDIX A

Boring Logs

KEY TO BORING LOGS

MAJOR TYPES			DESCRIPTION
COARSE-GRAINED SOILS MORE THAN HALF OF MAT'L LARGER THAN #200 SIEVE	GRAVELS MORE THAN HALF COARSE FRACTION IS LARGER THAN NO. 4 SIEVE SIZE	CLEAN GRAVELS WITH LESS THAN 5% FINES	GW - Well graded gravels or gravel-sand mixtures
			GP - Poorly graded gravels or gravel-sand mixtures
		GRAVELS WITH OVER 12 % FINES	GM - Silty gravels, gravel-sand and silt mixtures
			GC - Clayey gravels, gravel-sand and clay mixtures
	SANDS MORE THAN HALF COARSE FRACTION IS SMALLER THAN NO. 4 SIEVE SIZE	CLEAN SANDS WITH LESS THAN 5% FINES	SW - Well graded sands, or gravelly sand mixtures
			SP - Poorly graded sands or gravelly sand mixtures
		SANDS WITH OVER 12 % FINES	SM - Silty sand, sand-silt mixtures
			SC - Clayey sand, sand-clay mixtures
FINE-GRAINED SOILS MORE THAN HALF OF MAT'L SMALLER THAN #200 SIEVE	SILTS AND CLAYS LIQUID LIMIT 50 % OR LESS		ML - Inorganic silt with low to medium plasticity
			CL - Inorganic clay with low to medium plasticity
			OL - Low plasticity organic silts and clays
	SILTS AND CLAYS LIQUID LIMIT GREATER THAN 50 %		MH - Elastic silt with high plasticity
			CH - Fat clay with high plasticity
			OH - Highly plastic organic silts and clays
	HIGHLY ORGANIC SOILS		PT - Peat and other highly organic soils

For fine-grained soils with 15 to 29% retained on the #200 sieve, the words "with sand" or "with gravel" (whichever is predominant) are added to the group name.

For fine-grained soil with >30% retained on the #200 sieve, the words "sandy" or "gravelly" (whichever is predominant) are added to the group name.

GRAIN SIZES

U.S. STANDARD SERIES SIEVE SIZE: 200, 40, 10, 4 — CLEAR SQUARE SIEVE OPENINGS: 3/4", 3", 12"

SILTS AND CLAYS	SAND			GRAVEL		COBBLES	BOULDERS
	FINE	MEDIUM	COARSE	FINE	COARSE		

RELATIVE DENSITY

SANDS AND GRAVELS	BLOWS/FOOT (S.P.T.)
VERY LOOSE	0-4
LOOSE	4-10
MEDIUM DENSE	10-30
DENSE	30-50
VERY DENSE	OVER 50

CONSISTENCY

SILTS AND CLAYS	STRENGTH*
VERY SOFT	0-1/4
SOFT	1/4-1/2
MEDIUM STIFF	1/2-1
STIFF	1-2
VERY STIFF	2-4
HARD	OVER 4

SAMPLER SYMBOLS

- Modified California (3" O.D.) sampler
- California (2.5" O.D.) sampler
- S.P.T. - Split spoon sampler
- Shelby Tube
- Dames and Moore Piston
- Continuous Core
- Bag Samples
- Grab Samples
- NR No Recovery

MOISTURE CONDITION

DRY	Dusty, dry to touch
MOIST	Damp but no visible water
WET	Visible freewater

LINE TYPES

- Solid - Layer Break
- Dashed - Gradational or approximate layer break

GROUND-WATER SYMBOLS

- ▽ Groundwater level during drilling
- ▼ Stabilized groundwater level

(S.P.T.) Number of blows of 140 lb. hammer falling 30" to drive a 2-inch O.D. (1-3/8 inch I.D.) sampler

* Unconfined compressive strength in tons/sq. ft., asterisk on log means determined by pocket penetrometer

ENGEO
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DRAFT

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LOG OF BORING MPTB -1

LATITUDE: 37.7908 LONGITUDE: -122.3956

Geotechnical Exploration
Beale St.
San Francisco, CA
13553001000

DATE DRILLED: 1/18/2018
HOLE DEPTH: Approx. 300 ft.
HOLE DIAMETER: 4.0 in.
SURF ELEV (NAV88): Approx. 10 ft.

LOGGED / REVIEWED BY: M. Parks / JA
DRILLING CONTRACTOR: Pitcher Drilling
DRILLING METHOD: Mud Rotary
HAMMER TYPE: 140 lb. Auto Trip

Depth in Feet	Elevation in Feet	Sample Type	DESCRIPTION	Log Symbol	Water Level	Blow Count/Foot	Atterberg Limits: Liquid Limit	Atterberg Limits: Plastic Limit	Atterberg Limits: Plasticity Index	Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Shear Strength (psf) *field approximation	Unconfined Strength (tsf) *field approximation	Strength Test Type
			Concrete sidewalk approximately 5 inches thick												
			CLAYEY SAND WITH GRAVEL (SC), dark yellowish brown (10YR 3/6), medium dense, moist, well graded subangular gravel, some silt [FILL]												
			Scattered roots												
5	5														
			Grades to more clay												
10	0														
			Dried grout in cuttings												
			Wood debris												
			POORLY GRADED FINE SAND WITH GRAVEL (SP), dark grayish brown (2.5Y 4/2), loose, wet, fine sand, scattered gravel												
15	-5														
						4				4	14.6				
20	-10														

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			POORLY GRADED FINE SAND WITH GRAVEL (SP), dark grayish brown (2.5Y 4/2), loose, wet, fine sand, scattered gravel												
			CLAYEY SAND (SC), very dark greenish gray (10G 3/1), very loose, moist, fine sand, medium plastic clay, shell fragments												
25	-15														
						0	37	17	20	43	36.6	90	510*		PP+TV
			POORLY GRADED SAND (SP), very dark greenish gray (10G 3/1), very loose, wet, fine to medium sand, clay pockets, sand lense												
			FAT CLAY (CH), very dark greenish gray (5GY 3/1), soft, moist, shell fragments [YOUNG BAY MUD]												
30	-20														
35	-25		Scattered fiberous material, shells												
			Coarser sand in cutting												
40	-30														

LOG - GEOTECHNICAL_SU+QU W/ ELEV BEALE_ST.GPJ ENGEO INC.GDT 2/23/18

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			FAT CLAY (CH), very dark greenish gray (5GY 3/1), soft, moist, shell fragments [YOUNG BAY MUD]			0	64	29	35	91	58	66.6	565*		PP+TV
45	-35														
			CLAYEY SAND (SC), very dark greenish gray (10Y 3/1), medium dense, wet, fine to medium sand, some silt [HOLOCENE SAND]												
			Fiberous material in cuttings												
50	-40					23				31	24.9				
55	-45														
60	-50														

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			CLAYEY SAND (SC), very dark greenish gray (10Y 3/1), loose, wet, poorly graded fine sand, grades to more clay			3				31					
65	-55		Abundant fiberous material in cuttings												
			SILTY SAND (SM), very dark greenish gray (5G 3/1), medium dense, wet, fine to medium sand												
70	-60		Very dense			50/5.5"				17	17.4				
75	-65		Sand, fiberous material in cuttings												
80	-70														

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HAMMER TYPE: 140 lb. Auto Trip

Depth in Feet	Elevation in Feet	Sample Type	DESCRIPTION	Log Symbol	Water Level	Blow Count/Foot	Atterberg Limits: Liquid Limit	Plastic Limit	Plasticity Index	Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Shear Strength (psf) *field approximation	Unconfined Strength (tsf) *field approximation	Strength Test Type
			CLAYEY SAND (SC), olive brown (2.5Y 4/4), medium dense, moist, reddish brown oxidation, fine to medium sand [PLEISTOCENE SAND]			27				23					
85	-75														
			SANDY SILT (ML), very dark greenish gray (10GY 3/1), medium dense, moist, fine sand, thin silt laminations, cross bedding												
90	-80					23				51	20.6				
95	-85														
			SANDY FAT CLAY (CH), dark greenish gray (5G 4/1), very stiff, moist, fine sand, reddish brown oxidation, medium plasticity [OLD BAY CLAY]												
100	-90														

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LOG OF BORING MPTB -1

LATITUDE: 37.7908 LONGITUDE: -122.3956

Geotechnical Exploration
Beale St.
San Francisco, CA
13553001000

DATE DRILLED: 1/18/2018
HOLE DEPTH: Approx. 300 ft.
HOLE DIAMETER: 4.0 in.
SURF ELEV (NAV88): Approx. 10 ft.

LOGGED / REVIEWED BY: M. Parks / JA
DRILLING CONTRACTOR: Pitcher Drilling
DRILLING METHOD: Mud Rotary
HAMMER TYPE: 140 lb. Auto Trip

Depth in Feet	Elevation in Feet	Sample Type	DESCRIPTION	Log Symbol	Water Level	Blow Count/Foot	Atterberg Limits: Liquid Limit	Atterberg Limits: Plastic Limit	Atterberg Limits: Plasticity Index	Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Shear Strength (psf) *field approximation	Unconfined Strength (tsf) *field approximation	Strength Test Type
			SANDY FAT CLAY (CH), dark greenish gray (5G 4/1), very stiff, moist, fine sand, reddish brown oxidation, medium plasticity [OLD BAY CLAY]			29							2815*	2.0*	PP+TV
105	-95														
			FAT CLAY (CH), dark greenish gray (5G 4/1), stiff, moist, scattered organics, grades to less sand [OLD BAY CLAY]												
110	-100														
						14	57	24	33	97			2460*	1.75*	PP+TV
											40	81.5	3050		UU
115	-105														
			Abundant organic material in cuttings												
120	-110														

LOG - GEOTECHNICAL_SU+QU W/ ELEV BEALE_ST.GPJ ENGEO INC.GDT 2/23/18

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DRILLING CONTRACTOR: Pitcher Drilling
DRILLING METHOD: Mud Rotary
HAMMER TYPE: 140 lb. Auto Trip

Depth in Feet	Elevation in Feet	Sample Type	DESCRIPTION	Log Symbol	Water Level	Blow Count/Foot	Atterberg Limits: Liquid Limit	Atterberg Limits: Plastic Limit	Atterberg Limits: Plasticity Index	Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Shear Strength (psf) *field approximation	Unconfined Strength (tsf) *field approximation	Strength Test Type
			FAT CLAY (CH), dark greenish gray (10G 4/1), stiff, moist, scattered organics, reed/grass in sample [OLD BAY CLAY]			20							2200*	1.5*	PP+TV
125	-115														
130	-120										40.4	80.9	3230		UU
135	-125														
140	-130														

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Depth in Feet	Elevation in Feet	Sample Type	DESCRIPTION	Log Symbol	Water Level	Blow Count/Foot	Atterberg Limits: Liquid Limit	Atterberg Limits: Plastic Limit	Atterberg Limits: Plasticity Index	Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Shear Strength (psf) *field approximation	Unconfined Strength (tsf) *field approximation	Strength Test Type
			FAT CLAY (CH), greenish gray (10G 5/1), stiff, moist, scattered organics [OLD BAY CLAY]												
													2250*	2.0*	PP+TV
145	-135														
150	-140													1.75*	PP
155	-145														
160	-150														

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DRILLING CONTRACTOR: Pitcher Drilling
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Depth in Feet	Elevation in Feet	Sample Type	DESCRIPTION	Log Symbol	Water Level	Blow Count/Foot	Atterberg Limits: Liquid Limit	Atterberg Limits: Plastic Limit	Atterberg Limits: Plasticity Index	Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Shear Strength (psf) *field approximation	Unconfined Strength (tsf) *field approximation	Strength Test Type
			FAT CLAY (CH), dark greenish gray (5G 4/1), stiff, moist, scattered organics [OLD BAY CLAY]												
											43.8	77.6	3870		UU
165	-155														
			SILTY SAND (SM), very dark greenish gray (10GY 3/1), dense, moist, fine to medium sand, tightly packed grains with some silt, high organics content, woody debris including rounded wood fragments and fiber [ALAMEDA FORMATION]												
170	-160														
						34				14	23.6	99			
175	-165		Grades to no woody debris in cuttings												
			SILTY FAT CLAY (CH), dark greenish gray (5G 4/1), very stiff, moist, reddish brown oxidation, some silt												
180	-170														

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			SILTY FAT CLAY (CH), dark greenish gray (5G 4/1), very stiff, moist, reddish brown oxidation, some silt			29	56	20	36	96	27.5				
185	-175		Abundant organics in cuttings												
190	-180														
195	-185		CLAYEY SAND (SC), very dark greenish gray (5G 3/1), dense, moist, fine to medium sand, rip-ups of bluish gray clay nodules approximately 1/2-inch, some organics, high energy depositional environment												
200	-190														

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DATE DRILLED: 1/18/2018
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HOLE DIAMETER: 4.0 in.
SURF ELEV (NAV88): Approx. 10 ft.

LOGGED / REVIEWED BY: M. Parks / JA
DRILLING CONTRACTOR: Pitcher Drilling
DRILLING METHOD: Mud Rotary
HAMMER TYPE: 140 lb. Auto Trip

Depth in Feet	Elevation in Feet	Sample Type	DESCRIPTION	Log Symbol	Water Level	Blow Count/Foot	Atterberg Limits: Liquid Limit	Atterberg Limits: Plastic Limit	Atterberg Limits: Plasticity Index	Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Shear Strength (psf) *field approximation	Unconfined Strength (tsf) *field approximation	Strength Test Type
			CLAYEY SAND (SC), very dark greenish gray (5G 3/1), dense, moist, fine to medium sand, rip-ups of bluish gray clay nodules approximately 1/2-inch, some organics, high energy depositional environment			34				43	16.5				
205	-195														
210	-200		Grades to more sand												
215	-205														
			Angular pea-gravel size rocks in cuttings, Franciscan-complex derived												
220	-210														

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DATE DRILLED:	1/18/2018	LOGGED / REVIEWED BY:	M. Parks / JA
HOLE DEPTH:	Approx. 300 ft.	DRILLING CONTRACTOR:	Pitcher Drilling
HOLE DIAMETER:	4.0 in.	DRILLING METHOD:	Mud Rotary
SURF ELEV (NAV88):	Approx. 10 ft.	HAMMER TYPE:	140 lb. Auto Trip

Depth in Feet	Elevation in Feet	Sample Type	DESCRIPTION	Log Symbol	Water Level	Blow Count/Foot	Atterberg Limits: Liquid Limit	Plastic Limit	Plasticity Index	Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Shear Strength (psf) *field approximation	Unconfined Strength (tsf) *field approximation	Strength Test Type
			Base of Alameda Formation; clayey sand with subangular gravels			50/5"									
225	-215		SEE NEXT PAGE FOR CORE LOG												
230	-220														
235	-225														
240	-230														

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CORELOG MPTB -1

LATITUDE: 37.7908 LONGITUDE: -122.3956

DATE DRILLED:	1/18/2018	LOGGED / REVIEWED BY:	M. Parks / JA
HOLE DEPTH:	Approx. 300 ft.	CORING CONTRACTOR:	Pitcher Drilling
HOLE DIAMETER:	4.0 in.	CORING METHOD, DRILL BIT SIZE/TYPE:	Wireline, HQ
SURF ELEV (NAV88):	Approx. 10 ft.	NO. OF CORE BOXES:	5

Run Number	Drill Rate (min/ft)	Recovery (ft)/ Run Length (ft)	RQD	Depth in Feet	Elevation in Feet	Graphic Log	DESCRIPTION
							CLAYEY SAND (SC), very dark greenish gray (5G 3/1), dense, moist, fine to medium sand, rip-ups of bluish gray clay nodules approximately 1/2-inch, some organics, high energy depositional environmentContinued from previous Base of Alameda Formation; clayey sand with subangular gravels
							MELANGE, very dark greenish gray GLEY 3/1 10 GY, very weak (R1) to weak (R2), very closely fractured, highly weathered (WH), melange matrix of very closely sheared rock, relict calcite veins suggest graywacke protolith, sedimentary (graywacke and shale) melange [FRANCISCAN COMPLEX MELANGE - Hunter's Point Shear Zone terrane]
				225	-215		
1		1.5/1.5 (100%)	0				Begin HQ drilling
2		0/3 (0%)	0			NR	
				230	-220		
3		0.2/4 (5%)	0			NR	
4		0.7/1 (70%)	0				Crushed fracture spacing
				235	-225		
5		2.2/2 (110%)	0				236.5' to 237': graywacke blocks up to 2 inches, in clay matrix
6		2.1/3 (70%)	0				Clayey; 30-degree veinlets of calcite
						NR	
				240	-230		

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DATE DRILLED:	1/18/2018	LOGGED / REVIEWED BY:	M. Parks / JA
HOLE DEPTH:	Approx. 300 ft.	CORING CONTRACTOR:	Pitcher Drilling
HOLE DIAMETER:	4.0 in.	CORING METHOD, DRILL BIT SIZE/TYPE:	Wireline, HQ
SURF ELEV (NAV88):	Approx. 10 ft.	NO. OF CORE BOXES:	5

Run Number	Drill Rate (min/ft)	Recovery (ft)/ Run Length (ft)	RQD
7		3.2/3 (107%)	0
8	6.4	2.25/2.5 (90%)	0
9	13.71	1.3/3.5 (37%)	0
10		0/2 (0%)	0
11	4.57	3.5/3.5 (100%)	0
12	3.43	3.5/3.5 (100%)	0

Depth in Feet	Elevation in Feet
245	-235
250	-240
255	-245
260	-250

Graphic Log

DESCRIPTION

MELANGE, very dark greenish gray GLEY 3/1 10 GY, very weak (R1) to weak (R2), crushed fractured spacing, highly weathered (WH), clayey brecciated rock, graywacke gravels up to 1 inch, internally polished and sheared throughout [FRANCISCAN COMPLEX MELANGE - Hunter's Point Shear Zone terrane]

CaCO3 veining

1-inch greenstone fragments, gouge and sheared clay matrix

Crushed

SERPENTINITE, greenish gray GLEY 6/1 5G, strong (R5), pervasive quartz veining, veining is 0- to 10-degrees and undulatory, 8-inch block in melange matrix [FRANCISCAN COMPLEX MELANGE - Hunter's Point Shear Zone terrane]

Crushed

Clay gouge, 2 inches thick

SERPENTINITE, greenish gray GLEY 6/1 5G, weak (R2), internally sheared, brittle clay-lined microfractures, polished slicks 30- to 40-degrees throughout, block in melange matrix from 251.5' to 254' [FRANCISCAN COMPLEX MELANGE - Hunter's Point Shear Zone terrane]

MELANGE, very dark greenish gray GLEY 3/1 10 GY, very weak (R1) to weak (R2), clayey brecciated rock, graywacke gravels up to 1 inch [FRANCISCAN COMPLEX MELANGE - Hunter's Point Shear Zone terrane]

254': Shear, 27 degrees, polished

Clay gouge, melange matrix

Calcite veining, 2 inches thick

Clay gouge, stiff

257' to 258.5': Graywacke block, closely fractured, 60-degree joints

258.5' to 259.5': Sheared shale block, 65-degree sheared contact with graywacke block above, crushed, polished shale

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CORELOG MPTB -1

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DATE DRILLED:	1/18/2018	LOGGED / REVIEWED BY:	M. Parks / JA
HOLE DEPTH:	Approx. 300 ft.	CORING CONTRACTOR:	Pitcher Drilling
HOLE DIAMETER:	4.0 in.	CORING METHOD, DRILL BIT SIZE/TYPE:	Wireline, HQ
SURF ELEV (NAV88):	Approx. 10 ft.	NO. OF CORE BOXES:	5

Run Number	Drill Rate (min/ft)	Recovery (ft)/ Run Length (ft)	RQD	Depth in Feet	Elevation in Feet	DESCRIPTION
13	2	2.3/5 (46%)	0			MELANGE, very dark greenish gray GLEY 3/1 10 GY, very weak (R1) to weak (R2), clayey brecciated rock, graywacke gravels up to 1 inch [FRANCISCAN COMPLEX MELANGE - Hunter's Point Shear Zone terrane]
14	4.33	1.3/3 (43%)	0	265	-255	Chaoticly fractured graywacke and shale with clay gouge, rock pieces are weak (R2) to moderately strong (R4)
15	3.14	3.3/5 (94%)	0			Fine-grained sandstone and shale fragments, very clayey brecciated rock, clay gouge fills fractures throughout
16	4.28	3.5/3.5 (100%)	0	270	-260	271' to 272': Graywacke block 272' to 274.5': Sheared shale blocks
17	4.4	2.3/5 (92%)	0	275	-265	70-degree joints with pervassive clay-lined shears 274.5' to 275': Graywacke blocks, moderately fractured, clay matrix
18	2	2.25/2.5 (90%)	0			3" clay gouge Fine, angular graywacke gravels in sheared clay matrix
19	3	3/3 (100%)	0	280	-270	Crushed 60-degree CaCO3 vein 278.5' to 280.5': Graywacke blocks, very dark gray GLEY 1, moderately fractured, 3-inch fracture spacing, incipient fractures 55-degree fracture

Graphic Log

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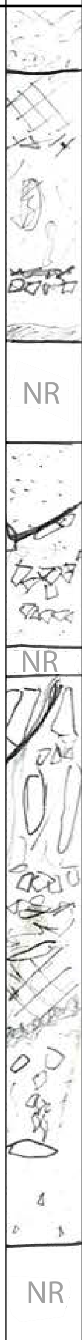
DATE DRILLED:	1/18/2018	LOGGED / REVIEWED BY:	M. Parks / JA
HOLE DEPTH:	Approx. 300 ft.	CORING CONTRACTOR:	Pitcher Drilling
HOLE DIAMETER:	4.0 in.	CORING METHOD, DRILL BIT SIZE/TYPE:	Wireline, HQ
SURF ELEV (NAV88):	Approx. 10 ft.	NO. OF CORE BOXES:	5



Run Number	Drill Rate (min/ft)	Recovery (ft)/ Run Length (ft)	RQD	Depth in Feet	Elevation in Feet	DESCRIPTION
						MELANGE, very dark greenish gray GLEY 3/1 10 GY, very weak (R1) to weak (R2), very highly fractured, clayey matrix [FRANCISCAN COMPLEX MELANGE - Hunter's Point Shear Zone terrane]
						281' to 282': Graywacke blocks, highly fractured, no clay infill
20	6.67	3/3 (100%)	0			282.5' to 284': Graywacke blocks, very highly fractured with clay infilling, clay pockets
						Crushed Very highly fractured with clay infilling
21	4.4	1/2.5 (40%)	0	285	-275	Very soft drilling, no recovery
						Clay matrix with fine angular gravel
						60-degree fracture with shearing, 1/8-inch CaCO3 veining, smooth
22	6.29	2.8/3.5 (80%)	0			Fractured gravel up to 1 inch
				290	-280	70- to 80-degree sheared clay seam
						Graywacke blocks in sheared clay matrix
23	4.57	3.3/3.5 (94%)	0			
						294' to 295': Graywacke blocks, highly fractured, little to no clay infilling, chaotic fracture pattern
				295	-285	60-degree clay seam, fine angular gravel
24	5.71	3.7/3.5 (106%)	0			Graywacke blocks up to 1-inch in clay matrix
						ncreased clay content, clay becomes less firm
25	10	1.6/3 (53%)	0			Very soft drilling
				300	-290	
						Boring terminated at 300 feet below ground surface on 01/25/2018. Groundwater not observed due to drilling method.