GEOTECHNICAL INVESTIGATION

for

1ST AND MISSION STREETS DEVELOPMENT
San Francisco, California

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0.0 EXECUTIVE SUMMARY

This report presents the results of our geotechnical investigation for the proposed 1st and Mission Streets Development in San Francisco. The 1st and Mission Streets development occupies portions of one block northwest of the intersection of 1st Street and Mission Street in San Francisco. The project site is currently occupied by several low- to mid-rise buildings and a paved parking lot. The site abuts several existing structures.

We understand the current improvements within the project site will be demolished and removed, and two new towers will be constructed. Tower 1 is planned on the west side of 1st Street, between Stevenson Street and Elim Alley; it will be a 61-story, steel, mixed-use, office and residential building. Tower 2 will be a 53-story concrete residential building on the north side of Mission Street between Ecker Street and an existing building (510 Mission Street). Tower 1 will have four basement levels and Tower 2 will have three basement levels; with the exception of the lowest level, the basements will be interconnected. The finished floor of the lowest basements beneath Tower 1 and 2 will be approximately 60 and 50 feet below the surrounding sidewalk grades, respectively.

The subsurface conditions below the planned basements consists of Marine Deposits (under the shallower Tower 2) and dense to very dense Colma formation sand (beneath the deeper Tower 1) which is underlain by more than 100 feet of stiff to hard, moderately compressible Old Bay Clay. The Old Bay Clay is underlain by interbedded layers of very stiff to hard, slightly compressible clay and dense to very dense sands and gravels, referred to as alluvium/colluvium in our report. Franciscan Complex bedrock underlies the alluvium/colluvium. Bedrock was encountered between about 260 and 273 feet beneath the existing ground surface.

The basement excavations will expose weak Marine Deposits and strong Colma formation soil. The Marine Deposits are weak, compressible, and in some areas may lose strength during an earthquake. The Colma formation is strong and relatively incompressible; however, if the buildings are supported in this layer, the load would spread into the underlying Old Bay Clay and alluvium/colluvium, causing excessive consolidation settlement. We therefore conclude the buildings should be supported on deep foundations that gain their capacity in friction in the soil and bedrock below the basements. We judge that large-diameter, drilled cast-in-place piers (also known as drilled shafts), or rectangular-section load bearing elements (LBEs, also known as barrettes) are feasible. Considering the large loads on each of these deep foundation elements and the need to limit the amount of settlements of the towers, these elements should extend into bedrock.

Construction of the planned basements and foundations will require excavations extending as deep as about 72 feet below the existing ground surface. The excavation should be shored to protect the surrounding improvements. A cutoff wall, consisting of deep soil-cement mixed (DSM) columns or panels or a concrete diaphragm wall, is deemed the most suitable method of excavation support for this project. The bottom of the walls should extend into the Old Bay Clay to create an effective groundwater cutoff.

This summary omits detailed recommendations; therefore, anyone relying on the report must read it in its entirety. The recommendations contained in this report are based on a relatively limited subsurface exploration program. Consequently, variations between expected and actual soil conditions may be found in localized areas during construction. We should be retained to observe shoring and foundation installation, during which time we may make changes to our recommendations as necessary.

1.0 INTRODUCTION

This report presents the results of our geotechnical investigation for the proposed 1st and Mission Streets development in San Francisco. Our services were performed in general accordance with our proposal dated 24 February 2014.

The 1st and Mission Streets Development occupies portions of one block bound by 1st Street on the east[1], Mission Street on the south, Ecker Street on the west, and Stevenson Street on the north. The approximate site location is shown on the Site Location Map, Figure 1. The site is currently occupied by an asphalt-paved lot in the southwestern portion of the site and multiple low to mid-rise buildings over the majority of the remaining project area. The project site abuts several existing structures. Prior to construction, we understand plans are to demolish and remove the existing site improvements. The project area also spans over two existing narrow streets, Jessie Street and Elim Alley, as shown on the Site Plan, see Figure 2.

As currently envisioned, the project includes construction of two new towers and associated improvements. Tower 1 is planned to be a 61-story, mixed-use, office and residential building with top floor height of about 860 feet (the top of the roof screen is expected to be approximately 910 feet above street grades). Tower 1 will be steel framed with exterior braces and concrete floor slabs cast on metal decks. Tower 1 will also have a four-level basement, with the lower level finished floor elevation about 60 feet below existing grade. Tower 2 will be a 53-story concrete, residential building with an overall height of about 605 feet. Tower 2 will have a three-level basement with a finished floor elevation about 50 feet below existing grade. As currently planned the upper three levels of the basements will be interconnected. In addition, the northern portion of Tower 1 will have a one-level basement west of the main tower footprint that will function as a ramp into the basement parking garage. The approximate building footprints are presented on Figure 2.

From conversations with the project structural engineer, Magnusson Klemencic Associates (MKA), we understand the building weight/seismic mass for Tower 1 will be about 190,000 kips. Column loads will range from about 600 kips to 27,000 kips. The building weight/seismic mass for Tower 2

1 Project north is toward Market Street, parallel to 1st Street.

will be about 165,000 kips. Because of some uncertainty in the current design of Tower 2, column loads are not known at this time.

Outside the tower excavations, site grading will be needed to create a level pad for on-grade improvements, including backfilling part of some of the existing basements that extend beyond the new basement footprints.

2.0 SCOPE OF SERVICES

Our scope of services was outlined in our proposal dated 24 February 2014. As part of our services, we reviewed existing subsurface data from explorations performed at the site and in the site vicinity and further explored the subsurface conditions by drilling nine new borings at the site, conducting in-situ downhole suspension logging in four borings, and installing piezometers in two of the borings. We conducted laboratory tests on samples recovered from the borings and performed engineering analyses to develop conclusions and recommendations regarding:

- soil, bedrock, and groundwater conditions at the site
- most appropriate foundation type(s) for the proposed buildings
- design criteria for the appropriate foundation types
- estimates of total and differential settlement of new foundations under design loads
- temporary shoring and dewatering
- underpinning and support of adjacent structures
- lateral earth pressures for design of permanent basement walls
- site seismicity and seismic hazards
- 2013 San Francisco Building Code (SFBC) seismic design criteria
- soil corrosivity
- site-specific earthquake spectra
- construction considerations.

3.0 FIELD EXPLORATION AND LABORATORY TESTING

The geotechnical exploration included drilling nine exploratory borings (labeled B-1 through B-9) to depths ranging from 268.5 to 323 feet beneath the existing ground surface (bgs) at the locations shown on Figure 2. Borings B-1 through B-8 were drilled about 50 feet into bedrock, while B-9 was drilled about four feet into bedrock. In-situ downhole suspension P- and S-Wave velocity logging was performed in borings B-3, B-4, B-6, and B-7. We installed groundwater monitoring wells with data loggers in B-7 and B-8. Laboratory tests were performed on selected samples of soil and bedrock to assist in classification and determination of engineering properties of the soil and bedrock. Details and results of the field exploration and laboratory testing programs are provided in Appendices A, B, and C.

4.0 SITE CONDITIONS

4.1 Surface Conditions

The site slopes down gradually from the west to the east, from about Elevation 7.5 feet[2] in the western portion of the site to about Elevation 4.5 feet at the eastern portion of the site. Currently, the site is occupied by an asphalt-paved lot in the southwestern portion of the site, two existing narrow streets (Jessie Street and Elim Alley) and multiple low to mid-rise buildings over the majority of the remaining project area. These buildings are shown on Figure 3 and include:

- 36 1st Street – 5 stories, brick, with basement (unknown depth)
- 42 1st Street – 7 stories, concrete, with basement (unknown depth)
- 62 1st Street – 5 stories, concrete, with basement (unknown depth)
- 78 1st Street – 6 stories, brick, with basement (unknown depth)

Several neighboring buildings are adjacent to or close to the project site as shown on Figure 2, including:

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[2] Elevations from survey titled "Preliminary Site Survey of a Portion of Assessor's Block No. 3708" by Martin M. Ron Associates dated 14 May 2014. All elevations presented herein reference San Francisco City Datum (SFCD).

- 1 Ecker Street – 4 stories, brick, with one basement level, set back about four to six feet from the proposed Tower 1 footprint and basement
- 25 Jessie Street – 20 stories, concrete, with one basement level over a portion of the building, set back about 1½ feet from the proposed Tower 1 footprint and 10 feet from the Tower 1 and 2 basements
- 84 1st Street – 3 stories, brick, with basement (unknown depth), set back about four feet from the Tower 1 and 2 basement footprints
- 510 Mission Street – 2 stories, brick, with basement (unknown depth), set back about 28 feet east of the Tower 2 footprint
- 525 Market Street – 40 stories, concrete, with basement (unknown depth), about 40 feet north of the proposed Tower 1 footprint
- 536 Mission Street – 5 stories, concrete, with basement (unknown depth), about 25 feet west of the Tower 2 footprint (across Ecker Street).

Test pits performed by Treadwell & Rollo, Inc. in 2005 indicated that 1 Ecker Street is supported on a grid of interconnected continuous shallow footings that extend about 3 to 4 feet beneath the existing basement slab.

Construction documents for 25 Jessie Street indicate that the building is constructed of concrete and is supported on driven, 12-inch-square, prestressed, precast, concrete piles. A portion of the interior of the building (which is not adjacent to the project site) has one basement level, while the remainder of the building is at grade. The piles supporting 25 Jessie Street that are adjacent to the proposed tower sites have approximate pile tip (the bottom of the piles) elevations at about -60 feet.

The foundation types or depths for the remaining buildings around the site are not currently known.

4.2 Subsurface Conditions

In general, the stratigraphy at the site consists of fill, Dune sand, marine deposits, Colma formation, Old Bay Clay, alluvium and colluvium, and bedrock. The boring logs provide further detail. Idealized subsurface profiles illustrating the general subsurface conditions at the site are presented on

Figures 4 and 5; the locations of these profiles are shown on Figure 2. The material types and a general description of their physical characteristics are summarized below:

Fill: The results of our investigation indicate the site is underlain by undocumented fill. Where explored, the fill is about 10 to 18½ feet thick and consists of sand with varying amounts of silt and clay. The fill is loose to medium dense and it does not appear to have been compacted during placement. Brick and rock fragments and construction debris were also encountered in the fill; it is likely these materials are debris from the 1906 earthquake. The fill does not appear to be expansive. Results of corrosivity analyses indicate the fill is mildly corrosive to corrosive. The bottom of fill extends to elevations ranging from about -3 to -11.5 feet.

Dune Sand: Beneath the fill is an 8- to 25-foot-thick layer of fine- to medium-grained, poorly graded sand (Dune sand). The sand is medium dense to very dense, and typically grades denser with depth. The sand is moist to wet, and contains varying amounts of silt. The bottom of this deposit extends to elevations ranging from about -18.5 to -31 feet.

Marine Deposits: Beneath the dune sand is a 9½- to 38-foot-thick marine deposit. Where encountered, this deposit consists of clay with varying amounts of silt and sand, clayey sands and clayey silty sands, and gravel. It is generally loose to dense (where sandy or gravelly) and soft to very stiff (where clayey). This layer is overconsolidated[3]; that is, it has experienced a load greater than the existing overburden. It can become remolded and lose strength if disturbed. The top of the marine deposit varies between Elevations -18.5 to -34 feet; the bottom between Elevation -26½ and -64 feet. The marine deposit is thinner under the northern portion of the site, along Stevenson Street, and becomes much thicker under the southern portion of the site, along Mission Street.

Colma Formation: Beneath the marine deposit is an 18- to 41½-foot thick layer of sandy soil with varying clay and silt content, known locally as the Colma formation. The Colma formation is generally dense to very dense; however, in borings B-3, B-6, and B-7, we encountered 4½- to 9-foot-thick layers of clayey sand that was medium dense (less dense than found at other sites in the vicinity). The Colma formation is generally strong and relatively incompressible. The top of the Colma formation ranges from about -26½ to -64 feet and extends down to about Elevation -66 to -82

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3 An underconsolidated clay has not yet achieved equilibrium under the existing load; a normally consolidated clay has completed consolidation under the existing load; and an overconsolidated clay has experienced a load greater than it is currently under.

feet. The Colma formation is thicker under the northern portion of the site, along Stevenson Street, and becomes thinner under the southern portion of the site, along Mission Street.

Old Bay Clay Beneath the Colma formation is an 83- to 106½-foot thick layer of marine clay known locally as Old Bay Clay. Old Bay Clay is stiff to hard and moderately overconsolidated, with overconsolidation ratios[4] ranging from about 4 near the top of the deposit to 1.2 near its base. The upper 10 to 15 feet make up the crust of the Old Bay Clay where the overconsolidation ratio is between about 2.5 and 4. Under the northern side of the site, dense to very dense sand layers of variable thickness are present within the Old Bay Clay (labeled as Old Bay Clay Sand on the idealized subsurface profiles). Where encountered, these layers ranged from approximately 10 to 25 feet thick which are thickest on the northern side of the site, along Stevenson Street. The top of the Old Bay Clay ranges from about -66 to -82 feet and extends down to about Elevation -151 to -182 feet.

Alluvium / Colluvium Beneath the Old Bay Clay and above the bedrock is a 77- to 106-foot thick layer referred to as alluvium/colluvium. The alluvium/colluvium consists of interbedded layers of very stiff to hard clay with variable sand and gravel content and sand with varying amounts of clay, silt, and gravel. Laboratory test results indicate the clay is overconsolidated and slightly compressible. Where explored, we observed an increase in gravel content towards the bottom of the layer. The top of the alluvium/colluvium ranges from about Elevation -151 to -182 feet and extends to about Elevation -253 to -266 feet, which is approximate top of bedrock.

Bedrock: Bedrock at the site consists of a Franciscan Complex Mélange which typically consists of a mixture of sheared and folded sedimentary, igneous, and metamorphic rocks resulting from large-scale tectonic processes. Bedrock beneath the 1st and Mission Streets development consists predominantly of shale Mélange mixed with graywacke sandstone. However, significant deposits of serpentinite and greenstone were also encountered across the site.

The shale and shale Mélange were generally crushed to closely fractured, soft to moderately hard, plastic to friable, with little to moderate weathering. In several borings, layers of the shale Mélange were completely sheared and broken down to a soil-like consistency.

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[4] Overconsolidation ratio refers to the ratio of the maximum past pressure a soil has experienced over the existing effective overburden pressure felt by the clay under today's conditions.

The graywacke sandstone encountered was crushed to moderately fractured, moderately hard, weak to strong, with little to deep weathering. Sandstone was mostly encountered as small fragments to less than 1 foot thick. However, in a few borings, layers of graywacke sandstone were 7½, 10, and 15 feet thick. See boring logs for more details.

The serpentinite encountered was crushed to closely fractured, low hardness to moderately hard, weak, with little to moderate weathering. Serpentinite was generally encountered as small and isolated fragments, but was up to 2½ feet thick in some borings.

In Boring B-1, an approximately 10-foot-thick layer of greenstone was encountered in the Mélange. The greenstone encountered was intensely fractured to moderately fractured, weak to moderately hard, weak to moderately strong, with little to no weathering. The approximate percentage of bedrock material encountered in each boring is shown Table 1.

Table 1

Percentage of Bedrock Material Encountered in Borings to Depths Explored

Boring	Shale and Shale Mélange	Graywacke Sandstone	Interbedded Shale and Sandstone	Greenstone	Serpentinite
B-1	47%	17%	8%	20%	8%
B-2	83%	17%	0%	0%	0%
B-3	32%	23%	45%	0%	0%
B-4	91%	9%	0%	0%	0%
B-5	62%	30%	5%	0%	3%
B-6	43%	23%	32%	0%	2%
B-7	68%	30%	2%	0%	0%
B-8	26%	55%	19%	0%	0%

Groundwater: Groundwater was encountered during the current investigation at depths of about 15 to 17 feet below ground surface, corresponding to approximate Elevation -8 to -12.5 feet. In 2000, during the previous investigation for 512 Mission, groundwater was measured about 14 bgs, corresponding to Elevation -7 feet.

In the piezometers installed in borings B-7 and B-8, we measured stabilized water levels ranging from about 17 to 20 feet bgs, corresponding to approximate Elevation –11 to -13 feet. The region is currently experiencing severe drought conditions. In addition, several nearby construction sites are actively dewatering. Piezometer readings taken in late December of 2014 (after over one foot of rainfall) rose about one foot over the course of a month. Groundwater data collected in the two piezometers through 29 December 2014 is presented in Figure 6.

The groundwater level will vary seasonally by several feet depending on rainfall infiltration, time of year, and severity of the drought in the region. The groundwater level will also vary from dewatering activities in the vicinity and utility leaks. In addition, the site is sufficiently close to the San Francisco Bay to be influenced by future sea level rise. Therefore, on the basis of the available groundwater information from past investigations in the vicinity of the site and to account for seasonal fluctuations, future sea level rise of several feet, and the effects of temporary dewatering in the vicinity, we judge the groundwater level within the project site may rise as high as Elevation -2 feet.

5.0 REGIONAL GEOLOGY

The site is in the northeast portion of the San Francisco peninsula, which lies within the Coast Ranges geomorphic province. The northwesterly trend of ridges and valleys characteristic of the Coast Ranges is obscured in San Francisco, except for features such as Russian Hill, Telegraph Hill, Hunters Point, and Potrero Hill. San Francisco Bay and the northern portion of the peninsula lie within a down-dropped crustal block bound by the East Bay Hills and the Santa Cruz Mountains. The San Francisco Bay depression resulted from interaction between the major faults of the San Andreas fault zone, particularly the Hayward and San Andreas faults east and west of the bay, respectively (Atwater, 1979).

San Francisco's topography is characterized by relatively rugged hills formed by Jurassic- to Cretaceous-aged[5] bedrock (Schlocker, 1974). The bedrock consists of rocks of the Franciscan Complex. The Franciscan Complex comprises a mix or mélange of sedimentary, igneous, and metamorphic rocks, which typically consists of mostly sandstone and shale. The rocks that make up the Franciscan Complex have undergone extensive faulting and shearing related directly to plate tectonics. This results in a chaotic mixture of materials, many of which have been broken down and weakened by shearing, which is referred to as mélange. The present topography resulted mainly from east-west compression of coastal California during the late Pliocene and Pleistocene[6] epochs (Norris and Webb, 1990).

The low-lying areas of the San Francisco peninsula are underlain by Quaternary[7] sediments deposited on eroded Franciscan Complex bedrock. Sediment deposition within the pre-historic[8] bay margin was influenced by oscillating late-Quaternary sea levels that resulted from the advance and retreat of glaciers worldwide.

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5 The *Jurassic* and *Cretaceous* periods spanned the time period from approximately 160 to 70 million years ago.

6 The *Pliocene* epoch spans from approximately 5 to 2 million years ago, while the *Pleistocene* epoch spans from approximately 2 million to 11,000 years ago.

7 The *Quaternary* period spans from approximately 2 million years ago to present, and includes the Pleistocene and Holocene epochs.

8 The present margin of San Francisco Bay is generally located seaward of its original location as a result of extensive land reclamation over the last century.

The resulting sequence of alternating estuarine[9] and terrestrial[10] sediments corresponds to high and low sea-level stands, respectively. In contrast, Quaternary sediments in the plains landward of the bay are predominantly terrestrial.

By late Pleistocene time, the high sea level associated with the Sangamon (about 125,000 years ago) interglacial resulted in deposition of the Yerba Buena Mud (Sloan, 1992). Also known locally as "Old Bay Clay", the Yerba Buena Mud was deposited in an estuarine environment similar in character and extent to the present bay. Sea level lowering associated with the onset of Wisconsin glaciation exposed the bay floor and resulted in terrestrial sedimentation, such as the Colma formation (and other sands), on the Yerba Buena Mud. Sea level rose again starting roughly 20,000 years ago, fed by the melting of Wisconsin-age glaciers.

The sea re-entered the Golden Gate about 10,000 years ago (Atwater, 1979). Inundation of the present bay resulted in deposition of estuarine sediments, such as Bay Mud and marsh deposits, which continue to accumulate within the active bay. During the same geologic time frame, much of San Francisco was covered with aeolian (wind-blown) sand deposits, referred to as Dune Sand.

Figure 7 presents the location of the project site on a regional geology map. The project site is located in the geologic unit labeled as artificial fill, which is consistent with our findings in our field investigation and our understanding of the site history.

9 *Estuarine* sediments typically consist of silt and clay, sometimes rich in organic matter and with occasional sand, deposited in inland marine areas affected by fresh water. Represents present environment of San Francisco Bay and includes the bay and adjacent tidal marshlands.

10 *Terrestrial* sediments generally consist of variable mixtures of clay, silt, sand and gravel deposited by rivers and streams ("alluvial deposits" or "alluvium"), and fine sand deposits deposited by wind ("aeolian deposits" such as dune sands).

6.0 REGIONAL SEISMICITY AND FAULTING

The major active faults in the area are the San Andreas, Hayward, San Gregorio, and Calaveras Faults. These and other faults of the region are shown on Figure 8. For each of these active faults, as well as other active faults within about 50 kilometers (km) of the site, the distance from the site and estimated mean characteristic Moment magnitude[11] [Working Group on California Earthquake Probabilities (WGCEP) (2008) and Cao et al. (2003)] are summarized in Table 2.

TABLE 2

Regional Faults and Seismicity

Fault Segment	Approx. Distance from Fault (km)	Direction from Site	Mean Characteristic Moment Magnitude
N. San Andreas - Peninsula	13.2	West	7.23
N. San Andreas (1906 event)	13.2	West	8.05
N. San Andreas - North Coast	16	West	7.51
Total Hayward	16	Northeast	7.00
Total Hayward-Rodgers Creek	16	Northeast	7.33
San Gregorio Connected	19	West	7.50
Rodgers Creek	33	North	7.07
Mount Diablo Thrust	33	East	6.70
Total Calaveras	34	East	7.03
Green Valley Connected	38	East	6.80
Monte Vista-Shannon	41	Southeast	6.50
Point Reyes	42	West	6.90
West Napa	44	Northeast	6.70

Figure 8 also shows the earthquake epicenters for events with magnitude greater than 5.0 from January 1800 through August 2014. Since 1800, four major earthquakes have been recorded on the San Andreas Fault. In 1836 an earthquake with an estimated maximum intensity of VII on the Modified Mercalli (MM) scale (Figure 9) occurred east of Monterey Bay on the San Andreas Fault (Toppozada and Borchardt 1998). The estimated Moment magnitude, M_w, for this earthquake is about 6.25. In 1838, an earthquake occurred with an estimated intensity of about VIII-IX (MM),

11 Moment magnitude is an energy-based scale and provides a physically meaningful measure of the size of a faulting event. Moment magnitude is directly related to average slip and fault rupture area.

corresponding to a M_w of about 7.5. The San Francisco Earthquake of 1906 caused the most significant damage in the history of the Bay Area in terms of loss of lives and property damage. This earthquake created a surface rupture along the San Andreas Fault from Shelter Cove to San Juan Bautista approximately 470 kilometers in length. It had a maximum intensity of XI (MM), a M_w of about 7.9, and was felt 560 kilometers away in Oregon, Nevada, and Los Angeles. The Loma Prieta Earthquake occurred on 17 October 1989 in the Santa Cruz Mountains with a M_w of 6.9, the epicenter of which is approximately 95 km from the site. The most recent earthquake to affect the Bay Area occurred on 24 August 2014 and was located on the West Napa fault, approximately 48 kilometers northeast of the site, with a M_W of 6.0.

In 1868 an earthquake with an estimated maximum intensity of X on the MM scale occurred on the southern segment (between San Leandro and Fremont) of the Hayward Fault. The estimated M_w for the earthquake is 7.0. In 1861, an earthquake of unknown magnitude (probably a M_w of about 6.5) was reported on the Calaveras Fault. The most recent significant earthquake on this fault was the 1984 Morgan Hill earthquake ($M_w = 6.2$).

The WGCEP at the U.S. Geologic Survey (USGS) predicted a 63 percent chance of a magnitude 6.7 or greater earthquake occurring in the San Francisco Bay Area in 30 years. More specific estimates of the probabilities for different faults in the Bay Area are presented in Table 3.

TABLE 3

WGCEP (2008) Estimates of 30-Year Probability of a Magnitude 6.7 or Greater Earthquake

Fault	Probability (percent)
Hayward - Rodgers Creek	31
N. San Andreas	21
Calaveras	7
San Gregorio	6
Concord - Green Valley	3
Mount Diablo Thrust	1

7.0 SEISMIC HAZARDS

During a major earthquake, strong to very strong ground shaking is expected to occur at the project site. These levels of ground shaking during an earthquake can result in ground failure such as that associated with soil liquefaction[12], lateral spreading[13], and seismic densification[14]. These conditions have been evaluated based on our field investigation and engineering analyses, and are discussed in this section.

7.1 Ground Shaking and Dynamic Soil-Structure Interaction

The seismicity of the site is predominantly governed by the activity of the San Andreas and Hayward Faults. However, ground shaking from future earthquakes on any of the nearby faults will be felt at the site. The intensity of earthquake ground motion at the site will depend upon the characteristics of the generating fault, distance to the earthquake fault, magnitude and duration of the earthquake, and specific subsurface conditions. On the basis of our knowledge of subsurface conditions, we conclude ground shaking at the site during a large earthquake on one of the faults discussed in Section 6 could be very strong.

To further estimate ground shaking at the site, we performed a Probabilistic Seismic Hazard Analysis (PSHA) and deterministic analysis to develop site-specific horizontal response spectra for three levels of shaking. Details regarding these analyses and development of the recommended spectra for the project are presented in Section 9.9. In addition, to account for the effect of the foundation, structures, and deep foundations on the dynamic performance of the buildings, we performed a non-linear dynamic Soil-Structure Interaction evaluation using the computer program FLAC (version 7.0), developed by Itasca Consulting Group Inc.

A discussion of the ground shaking and soil-structure interaction is presented in Appendix D.

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[12] Liquefaction is a transformation of soil from a solid to a liquefied state during which saturated soil temporarily loses strength resulting from the buildup of excess pore water pressure, especially during earthquake-induced cyclic loading. Soil susceptible to liquefaction includes loose to medium dense sand and gravel, low-plasticity silt, and some low-plasticity clay deposits.

[13] Lateral spreading is a phenomenon in which surficial soil displaces along a shear zone that has formed within an underlying liquefied layer. Upon reaching mobilization, the surficial blocks are transported downslope or in the direction of a free face by earthquake and gravitational forces.

[14] Seismic densification (also referred to as differential compaction) is a phenomenon in which non-saturated, cohesionless soil is densified by earthquake vibrations, causing ground-surface settlement.

7.2 Fault Rupture

Historically, ground surface fault ruptures closely follow the traces of geologically young faults. The site is not within an Earthquake Fault Zone, as defined by the Alquist-Priolo Earthquake Fault Zoning Act, and no active or potentially active faults exist on the site. In a seismically active area, the remote possibility exists for future faulting in areas where no faults previously existed; however, we conclude the risk of surface faulting and consequent secondary ground failure is low.

7.3 Liquefaction and Associated Hazards

When a saturated soil with little to no cohesion liquefies during a major earthquake, it experiences a temporary loss of strength as a result of a transient rise in pore water pressure generated by strong ground motion. Flow failure, lateral spreading, differential settlement, loss of bearing, ground fissures, and sand boils are evidence of excess pore pressure generation and liquefaction. The site is within a liquefaction hazard zone, as designated by the California Geological Survey (CGS) seismic hazard zone map for the area titled State of California Seismic Hazard Zones, City and County of San Francisco, Official Map, dated 17 November 2000, as shown on Figure 10. Ground failures including sand boils and settlement (Youd and Hoose, 1978) which are indicative of liquefaction were recorded at the southeast corner of 1st and Mission Streets and at the intersection of 1st and Market Streets during the 1868 and 1906 earthquakes. CGS has recommended the content for site investigation reports within seismic hazard zones be performed in accordance with Special Publication 117A titled Guidelines for Evaluating and Mitigating Seismic Hazard Zones in California, dated 11 September 2008. Our evaluation of site seismic hazards was performed in general accordance with these guidelines.

All nine of the borings drilled at the site encountered layers of loose to medium dense sand with varying clay and silt content below the water table that could be susceptible to liquefaction and strength loss during a major earthquake. These layers were encountered within the bottom portion of the fill, layers in the Dune sand and marine deposit, and isolated portions within the Colma formation. Where observed, potentially liquefiable layers varied from 2 to 14 feet thick and were encountered between depths of 9 and 70 feet bgs. The potentially liquefiable layers do not appear to be continuous between borings except for the lower portion of the fill. Depending on the basement depth proposed for the project, the majority of the potentially liquefiable material could be excavated to accommodate the planned basements.

The level of ground shaking used in our liquefaction evaluation was based on the Maximum Considered Earthquake (MCE) mapped values. A peak geometric mean ground acceleration (PGAM) of 0.51 times gravity was used in our analyses. This PGA was calculated using the procedures specified in the 2013 California Building Code (CBC), using site class D. We used the procedure outlined in the proceedings of the NCEER workshops (Youd 2001) and the Tokimatsu and Seed (1987) method for evaluating earthquake-induced liquefaction settlement. Using these methods, we estimate settlements at the ground surface ranging from ½ to 5 inches could occur. This settlement could be erratic and may vary significantly across the site; differential liquefaction-induced settlement could be on the order of 4 inches within a horizontal distance of 30 feet.

Upon initial screening, two potentially liquefiable soil layers were identified below the planned basement levels. These layers are described as follows

1. In boring B-2 between the depths of 62½ and 67 feet we encountered a loose clayey silty sand layer within the Marine Deposit. This layer has a Plasticity Index (PI) equal to 4. Using the methodology presented by Idriss and Boulanger (2008) we conclude it is likely that this layer will liquefy and lose strength during a large earthquake. We estimate this layer could experience up to one inch of liquefaction-induced settlement during and immediately following a major earthquake.

2. In boring B-7 between depths of 78 and 82 feet, a layer of medium dense clayey sand was encountered within the Colma formation. Using the shear wave velocity data from B-7 and the methodology presented by Andrus and Stokoe (2000), we determined the liquefaction potential of this layer is low. In addition, this layer has a Plasticity Index (PI) equal to 7. Using the methodology presented by Idriss and Boulanger (2008) we conclude it is unlikely that this layer will liquefy, although it may lose some strength (soften) during a large earthquake. Accordingly, it is unlikely that settlement will occur within this layer.

All other materials encountered beneath the deep tower basements were either sufficiently dense or had sufficient cohesion to resist liquefaction. We therefore, we conclude liquefaction-induced settlement below Tower 1 is likely negligible, but localized areas within the Marine Deposits beneath Tower 2 may experience liquefaction and lose strength. This phenomenon should be accounted for in the design of the foundations for Tower 2.

Below the planned single-level excavation for the driveway at the northwest portion of Tower 1, we estimate liquefaction-induced settlements on the order of 3 inches could occur after a major earthquake. This settlement was estimated based on the loose to medium dense marine deposit sands encountered in the nearby borings.

Because of the presence of relatively shallow potentially-liquefiable soil in the areas where no basements are planned, we conclude ground failure, such as lurch cracking and/or the development of sand boils, could occur during a major earthquake beneath the portions of the site where basements are not planned and immediately outside the site footprint. Where sand boils and associated ground failures occur, the ground surface settlement could be larger.

7.4 Lateral Spreading

Lateral spreading is a phenomenon in which a surficial soil displaces along a shear zone that has formed within a continuous underlying liquefied layer. The surficial blocks are transported downslope or in the direction of a free face, such as a channel, by earthquake and gravitational forces. Lateral spreading is generally the most pervasive and damaging type of liquefaction-induced ground failure generated by earthquakes. According to Youd, Hansen and Bartlett (2002), for significant lateral spreading displacements to occur, the soil must consist of saturated cohesionless sandy sediments with corrected SPT blow counts $(N_1)_{60}$ less than 15 blows per foot.

Potentially liquefiable layers that have $(N_1)_{60}$ values less than 15 were encountered near the bottom of the fill in borings B-4 through B-8; however, they are thin (2 to 5 feet thick) and are not continuous throughout the site (e.g. they were not encountered in the other borings). One 4-foot-thick layer with an $(N_1)_{60}$ values less than 15 was encountered in the Dune sand in boring B-6, and 5- to 10-foot-thick layers with an $(N_1)_{60}$ values less than 15 were encountered in the marine deposit in borings B-1 through B-3. While all of these layers are potentially liquefiable and may cause settlement at the ground surface, they are discontinuous across the site. The project site vicinity is also relatively flat and more than 2,200 feet from any slope or shoreline. In addition, basements and deep foundations are present in the vicinity of the project site. Considering these factors, we conclude the potential for lateral spreading at the site is low.

7.5 Seismic Densification

Seismic densification (also referred to as cyclic densification and differential compaction) can also occur during strong ground shaking in loose, clean granular deposits above the water table, resulting in ground surface settlement. Several of the borings encountered granular deposits above the groundwater table that are susceptible to seismic densification.

We evaluated the potential for seismic densification to occur at the site using methodology presented in Tokimatsu and Seed (1987). Based on this method we estimate settlements associated with seismic densification will range from about ½ to 2 inches across the site during a major earthquake. Combined with the liquefaction-induced settlement, we estimate 1½ to 6 inches of total seismic settlement could occur at the ground surface.

The seismic densification settlement could damage sidewalks, streets, and underground utilities; the design of utilities and exterior slabs at the building perimeter should take this settlement into account. However, because the proposed basements will extend below the design water table and will be founded beneath these layers, seismic densification should not occur beneath the proposed foundations.

8.0 DISCUSSION AND CONCLUSIONS

From a geotechnical engineering standpoint, we conclude the site can be developed as currently planned, provided the recommendations presented in this report are incorporated into the project plans and specifications and implemented during construction.

To construct the new buildings and planned basements, the project will include temporary shoring, dewatering, excavations to depths as much as about 72 feet below the existing ground surface, and installation of deep foundations. The primary geotechnical issues for the proposed project include:

- providing adequate foundation support for the new structures, including control of total and differential settlements
- providing temporary support of adjacent buildings, streets, and utilities, and other improvements during the excavation for the proposed basement levels
- dewatering the site during excavation and foundation installation.

Our conclusions and recommendations regarding these issues are discussed in the remainder of this report.

8.1 Foundations and Settlement

Tower 1 will be approximately 860 feet high with 61 floors of occupied space and will have a weight/seismic mass will be around 190,000 kips. Tower 1 will have a basement extending to a depth of 60 feet over the majority of its footprint, with a single-level basement on the northwest portion for the garage ramp. Column loads will range from about 600 kips to 27,000 kips. Tower 2 will be approximately 605 feet high with 53 floors of occupied space. The building weight/seismic mass will be around 165,000 kips. Tower 2 will have a basement extending to 50 feet below its entire footprint.

8.1.1 Shallow Foundations

If a shallow foundation system such as a concrete mat is used to support the buildings, the Tower 1 mat would bear on Colma formation sand, about one to eight feet above the Old Bay Clay, and

Tower 2 would bear on Marine Deposits. The Colma formation sand, where present, is relatively incompressible and is capable of supporting moderate to heavy foundation loads without excessive settlement. However, the loads would spread to the clay layers within the Old Bay Clay and alluvium/colluvium, causing considerable consolidation settlement. The Marine Deposits are compressible and not capable of supporting heavy foundation loads without excessive settlement. In addition, one sand layer encountered in the Marine Deposits may liquefy, lose strength, and settle during a major earthquake.

The results of consolidation tests indicate the Marine Deposits, Old Bay Clay, and alluvium/colluvium are slightly to moderately overconsolidated. That is, the clay within this unit has experienced a greater pressure than the existing overburden. These results are consistent with previous testing performed on Old Bay Clay at nearby sites.

If the anticipated building loads from the estimated bottom of the mats are spread through the Marine Deposits, Old Bay Clay, and alluvium/colluvium, using stress distribution theory, the new load within these layers would exceed the maximum past pressure they have experienced, and a new cycle of primary consolidation will begin. Our calculations indicate the Tower 1 mat would settle on the order of 6 inches on the northern side and 12 inches on the southern side of the building. The significant differential settlement anticipated is primarily a result of the variable thickness of Old Bay Clay and presence of sand within this unit across the building footprint. We anticipate Tower 2 would likely settle on the order of several feet. Considering the variable thickness and stress history of the Marine Deposits and Old Bay Clay and the overall magnitude of the computed settlements, we conclude the settlements could vary significantly across the buildings. We conclude if mat foundations were used to support the proposed towers, the estimated static total and differential settlements would be excessive and result in unacceptable performance of the buildings.

During a seismic event, the bearing pressures on the bottom of a mat foundation would likely exceed the allowable bearing capacity of the Marine Deposits and Old Bay Clay, and could result in punching failure of the underlying soil.

Considering the anticipated static settlements and limitations of the bearing capacity of the Marine Deposits and Old Bay Clay, we conclude mat foundations are not appropriate for the support of the proposed towers. Therefore, we conclude deep foundations should be used for building support.

8.1.2 Deep Foundations

To limit the total and differential settlement beneath the towers, we conclude that large-diameter, drilled, cast-in-place piers (also known as drilled shafts) are the most appropriate foundation type for the site and load conditions. As requested by the design team, rectangular-section load bearing elements (LBEs, also known as barrettes) are discussed as an alternative to the drilled piers. To gain sufficient capacity, the piers or LBEs should extend into bedrock.

Large-diameter drilled shafts have been used successfully in the Bay Area and are likely the most feasible foundation type for the 1st and Mission Street project. Drilled shafts are typically installed utilizing either the "dry" or "wet" methods depending on the subsurface and groundwater conditions encountered. Considering the subsurface conditions and the estimated depths of drilled shafts, we judge the "wet" method should be used. The wet method uses drilling fluid to stabilize the hole during the entire drilling operation. Upon attaining the required embedment into bedrock and implementing proper cleaning of the rock socket bottom, the steel reinforcing cage is lowered into the hole and concrete is placed utilizing tremie techniques to displace all of the drilling fluid.

The sands encountered within the fill, Dune Sand, Marine Deposits, Colma formation, and the Old Bay Clay sand have low fines contents (fraction of soil passing the #200 sieve) and are prone to caving during drilling of deep foundations, even with the use of slurry. Accordingly, temporary casing should be utilized in addition to slurry to minimize the potential for caving and to prevent the soil from mixing with concrete during construction of the foundations. The casing should extend to sufficient depth to prevent caving, and at a minimum key into the Old Bay Clay layer (see Figures 4 and 5).

Rectangular barrettes are less widely used in the Bay Area as foundation elements, although they have been recently installed on a couple of projects in San Francisco. Barrettes are typically excavated to depth using a wireline clamshell or hydrofraise in order to achieve the required depth into bedrock. The wireline clamshell and hydrofraise create a rectangular section. A hydrofraise uses two cutting wheels to excavate and transform soil/bedrock in which it is digging into slurry, which is then pumped to the ground surface. Once at the ground surface, the slurry is pumped through a de-sander unit which removes the larger solids from the slurry, and the slurry is pumped back into the excavation. Therefore, the drilling fluid/slurry acts to both to maintain the side walls of the excavation and transport the excavated soil to the ground surface. Upon attaining the required embedment into bedrock and implementing proper clean-out, a rectangular steel reinforcing cage is lowered into the hole, and concrete is placed utilizing tremie techniques to displace the drilling fluid.

Unlike drilled shafts, temporary casing is not used with barrettes because of the rectangular shape of the excavation; therefore, caving can only be controlled using slurry. Accordingly, if caving occurs there are limited means to prevent it, such as modifying the slurry mix. Consequently, barrettes are more likely to experience caving within the upper soil than cased drilled piers. In addition, the geometry of a barrette, with two long straight sidewalls, is less stable than the circular shape of a drilled pier, as the soil will have limited arching capability as compared to a circle. Therefore, the sidewalls of barrettes are more susceptible to stress relaxation and squeezing, potentially reducing the ultimate frictional capacity of the soil/barrette interface.

8.1.3 Drilling Slurry

With proper construction methods, bentonite or polymer slurry can be effective in maintaining a stable excavation and allowing thorough displacement of slurry by concrete. Bentonite slurry consists of powdered clay (predominantly the mineral montmorillonite) mixed with water. Bentonite slurry stabilizes an excavated shaft by 1) the formation of a filter cake which acts as a membrane on the sidewalls of the shaft, and 2) positive fluid pressure acting against the filter cake and side wall. Polymer slurry can consist of numerous natural or synthetic compounds combined with water. Polymer slurry stabilizes an excavated shaft by 1) penetration into porous soil formation and interaction with soil particles creating a bonding effect, and 2) positive fluid pressure acting against the side wall.

Studies suggest that when using bentonite slurry, the filter cake can become too thick when construction duration of a single deep foundation is long (as is expected for this project). The thickened filter cake can greatly reduce the interface friction between the concrete and soil or bedrock, decreasing the available perimeter shear and ultimately decreasing the frictional capacity of the drilled shaft or barrette. Multiple studies including Meyer (1996) and Brown (2002) have compared drilled shafts constructed using bentonite and polymer slurry. Their findings indicate that ultimate skin frictions are significantly lower in bentonite slurries than in polymer slurries. The results of these studies have been corroborated by the recent experience with barrettes being installed in San Francisco using bentonite slurry and drilled shafts being installed with polymer slurry. Based on conversations with the deep foundation construction community in San Francisco, we understand the barrettes recently installed using bentonite slurry obtained about half of the friction value capacity as drilled shafts installed using polymer slurry to support the foundation excavations. For this reason, we conclude the deep foundations for the project should be installed using a polymer-based slurry; the use of bentonite slurry should be excluded from the project.

Throughout the construction process, quality control measures should be maintained to keep the properties of the drilling slurry in the desired range for optimal performance. Slurry should be sampled immediately before its introduction into the drilled shaft, as well as at least every 2 hours after its introduction and before concrete is placed. Some or all of the following properties should be monitored during installation: density, viscosity, pH value, sand content, and fluid loss. Additionally, before selecting the polymer slurry for the site, it is important to consult with the supplier to match the appropriate product and additives with the anticipated ground conditions. During construction, care should be taken by the foundation contractor to maintain the polymer slurry. Polymer slurries should not be mixed by any equipment that will disrupt the chain structure of the polymer. Use of in-line mixers, diaphragm pumps, and splash plates are effective measures to avoid damage. Experience has shown that mixing with blades and cyclones should be avoided.

8.1.4 Estimated Settlement of Deep Foundations

Deep foundations will transfer the building loads into the soil and bedrock beneath the site; some strain will occur as the soil and bedrock strength is engaged. This, in combination with the elastic compression of the foundation element will result in settlement at the shaft or barrette head. For the expected design column loads and length of deep foundations, we estimate the deep foundations should settle about 1½ to 2 inches during static loading. More specific estimates of the anticipated settlement of the pier heads can be provided following the full-scale axial load testing which will be required as part of the project.

8.1.5 Ground Settlement

As discussed in Section 7, seismically-induced settlement (from liquefaction and seismic densification) could occur at the site and beneath the surrounding streets and improvements. This settlement could be abrupt and occur in isolated locations. During a major earthquake on a nearby fault, we estimate up to six inches of seismically-induced settlement could occur beneath surrounding sidewalks, streets, and portions of the site where basements are not installed.

Exterior slabs, driveways, utilities, and utility connections at the basement interface should be designed to accommodate potential differential settlement of up to six inches (where the improvements settle relative to the buildings) as well as the anticipated static settlement of 1½ to 2 inches of building settlement (where the building settles relative to exterior improvements). These settlements are expected to occur at different times during the life of the building and should be designed for separately. Entrances should be designed to accommodate the differential settlement, and flexible connections should be used where utilities enter the buildings.

8.2 Shoring Considerations

Construction of the planned basement levels and foundation installation for proposed towers will require excavations extending as deep as 72 and 58 feet below the existing exterior ground surface at Towers 1 and 2, respectively. As planned, these excavations are set back from 1½ to about 40 feet from the adjacent structures. The upper three levels of basements are planned to be connected. Additionally, a one-level basement is planned on the northwest portion of Tower 1 for the garage ramp. These excavations are close to or may directly abut several adjacent structures.

Considering the presence of buildings and other improvements surrounding the proposed basements, the excavation will need to be shored to protect the surrounding improvements. There are several key considerations in selecting suitable shoring and underpinning systems. Those we consider of primary concern are:

- protection of surrounding improvements, including roadways, utilities, and adjacent structures
- the ability of the shoring system to control the inflow of groundwater and therefore reduce the required dewatering
- the ability of the shoring system to reduce potential for ground movement
- the shape of the excavation
- cost.

Several methods of shoring are available; we have qualitatively evaluated the following systems:

- conventional soldier pile and lagging
- soil-cement walls.

On other projects in San Francisco with similar depths of excavation, soldier pile and lagging shoring systems have been used. A soldier pile and lagging system usually consists of steel beams and concrete placed in predrilled holes extending below the bottom of the excavation. Drilling of the holes for the soldier piles requires either specialty equipment (such as a soil-cement mixing drill rig), casing, or the use of drilling slurry to prevent caving. Wood lagging is then placed between the soldier beams as the excavation proceeds. Because of the cohesionless nature of the fill, Dune Sand, and the sand within the marine Deposits, caving may occur while lagging boards are installed, making the excavation process slow increasing the potential for settlement behind the shoring. Use of a soldier-pile-and-lagging shoring system requires that the area surrounding the project site be dewatered to a depth of at least three feet beneath the proposed excavation depth.

Dewatering to a depth of about 75 feet would result in lowering groundwater beyond the site limits, which could result in settlement of the ground surface and improvements in the vicinity. In addition, dewatering the site through the clayey portion of the marine deposit may be difficult. Experience with other deep excavations in San Francisco has shown that groundwater will be perched within the sand layers encountered immediately above the clayey marine deposit. During excavation and lagging installation through the perched water zone, sand will flow into the excavation.

For all of these reasons, we conclude that a soldier-pile-and-lagging shoring system is not appropriate for support of the 58- or 72-foot-deep excavation at the project site. However, for the excavations that will be less than 20 feet deep, including the single-level-basement for the garage ramp, this system may be appropriate because the excavations will not extend significantly below the existing groundwater table.

We conclude that a cutoff wall is the most suitable method of excavation support for the deeper excavations proposed at this site. There are several types of cutoff walls; we judge the two most appropriate for the project include deep concrete diaphragm walls or soil-cement mix (DSM) columns or panels.

8.2.1 Cutoff Wall

Mechanical deep soil-cement mixing strengthens soil in place by using mixing shafts consisting of auger cutting heads, discontinuous flight augers, or blades/paddles to create soil-cement columns or panels. The DSM columns or panels are installed in an overlapping pattern to create a continuous wall. Steel beams are placed in some of the DSM columns or panels to provide structural rigidity. DSM walls are considered temporary; permanent walls are typically built inside of the DSM walls. Because these walls are continuous, they should act to cut off groundwater infiltration.

Concrete diaphragm walls are reinforced concrete walls constructed by slurry trench method. The walls are constructed in sections, or panels, similar to the methodology discussed above for the barrettes. During excavation of a panel, slurry is pumped into the trench to prevent the soil from caving. After the excavation reaches the design depth and the reinforcement cage is placed, the slurry is displaced by concrete that is poured through a tremie pipe. One primary difference between concrete diaphragm walls and a DSM wall is that the diaphragm wall can be used as both temporary shoring and the permanent walls. However, when using a concrete diaphragm wall as the permanent basement wall, waterproofing can be challenging.

For either the DSM or concrete diaphragm walls the bottom of the walls should extend sufficiently into the Old Bay Clay deposits to create an effective groundwater cutoff. If properly installed, dewatering should not be required beyond the limits of the deep excavations.

The shoring will require either grouted tiebacks or internal bracing, depending on the construction sequence planned and whether encroachment permits can be obtained to drill beneath the city streets and/or adjacent structures.

Where existing buildings adjacent to excavations are within 4 feet of the excavation and their foundations do not extend to the bottom of the planned excavation, the structures should be underpinned or the shoring should be designed to support the surcharge from the neighboring buildings. These structures may include 1 Ecker Street and 84 1st Street, depending on the final set back distances from the planned excavations. 510 Mission Street, 535 Mission Street, and 525 Market Street are set back from the foundation too far to underpin and 25 Jesse Street is supported on pile foundations and does not require underpinning. However, the proposed excavations for the basements beneath the towers will likely extend deeper than all of the foundations of these buildings. The shoring should be designed to support the surcharge from the neighboring buildings that will not be underpinned. The permanent basement walls for the project will also need to be designed for the surcharge from these buildings.

The buildings at 1 Ecker Street and 84 1st Street are within four feet of the planned excavation. Treadwell & Rollo Inc. previously performed an investigation for a seismic retrofit at 1 Ecker Street. The test pits excavated during the investigation revealed that the building is supported on a grid of interconnected continuous shallow footings that extend about 3 to 4 feet beneath the existing basement slab. Foundation plans for 84 1st Street are not available; however, considering the building's age and construction type, it is also likely supported on shallow foundations.

Because the planned excavations will extend beneath the assumed depth of these building's foundations, the buildings should be underpinned or the shoring should be designed to support the surcharge from the buildings. If soldier-pile-and-lagging shoring is used, underpinning should consist of slant-drilled soldier piles (soldier piles where pile top is installed beneath the existing foundation system). If a cutoff wall is used, underpinning can still consist of slant-drilled soldier piles, but overlapping panels or columns of DSM or jet grouting will be needed to create an effective groundwater cutoff in the space between soldier piles

The design, construction, and performance of the shoring system should be the responsibility of the contractor and should be designed by a structural engineer knowledgeable in this type of construction. The design of the selected dewatering system should be provided to the shoring designer so that the temporary groundwater elevation can be incorporated in the shoring design. Geotechnical recommendations for the shoring design are provided in Section 9.2 of this report.

8.2.2 Top-Down Construction

As an alternative to temporary tiebacks or internal bracing, a top-down method of construction can be considered. With this method, the basement floors, which would be cast from the top down, would act as the bracing system of the temporary shoring. Usually, if the top-down construction methodology is selected, the temporary shoring system consists of a concrete diaphragm wall which will also serve as the permanent basement wall for the buildings. A temporary DSM wall could also be used; however, permanent basement walls would need to be cast against the DSM wall during the construction process. The typical construction procedure of the top-down method is as follows:

Prior to the installation of the production piers, a pier load test program should be performed. After the pier load tests are completed, the construction would proceed as follows:

1. install production piers and perform pier load tests
2. construct the cutoff walls
3. install the deep foundations
4. install the king posts (columns to support the basement slabs)
5. cast the ground floor slab, leaving a central opening for access and removal of the underlying soil

6. excavate to the next basement level and cast the slab
7. repeat step 5 floor by floor until reaching the lowest basement level and cast the mat and pier caps.

8.3 Excavation and Monitoring

As discussed in Section 8.2, we anticipate excavations from 15 to 72 feet will be needed to construct the planned basements for the project. The soil to be excavated consists of sandy fill, Dune sand, marine deposit sand and clay, and Colma formation sand, which can be excavated using conventional earth-moving equipment such as excavators. Remnants of previous concrete slabs and foundations may be encountered during excavation and the use of a jack hammer or hoe ram may be required to break them up and remove them. Excavations that will be deeper than five feet and will be entered by workers should be shored or sloped in accordance with the Occupational Safety and Health Administration (OSHA) standards (29 CFR Part 1926).

We anticipate the soil that will be encountered at the subgrade level of the proposed towers will consist of dense Colma formation sand and potentially medium dense clayey sand of the marine deposit (on the southern side of Tower 2). Because the soil will be saturated, it can be easily disturbed by construction activities. Therefore, the site should be adequately dewatered on the inside of the shoring system and the subgrade should be protected during construction activities.

During excavation, the shoring system is expected to yield and deform, which could cause surrounding improvements to settle slightly. The magnitude of shoring movements and resulting settlements of the ground surface behind shoring walls are difficult to estimate because they depend on many factors, including the method of installation and the contractor's skill in the shoring installation. Clough and O'Rourke (1990) summarized the measured settlements adjacent to excavations in sand and concluded that the settlements varied from 0.1 to 0.3 percent of the excavation depth. The data also show the settlement at some sites where the excavations were shored with a soldier-pile-and-lagging system were as high as 0.5 percent. Therefore, for an excavation depth of approximately 72 feet, we estimate settlement immediately behind the shoring wall could be on the order of 1 to 4 inches, depending on the shoring system. If a cutoff wall is designed for at-rest pressures, we estimate settlement behind the shoring should be approximately

1½ inches. These settlements assume the quality of construction will meet that considered standard in the construction industry. The settlement should decrease linearly with distance from the wall, and should be inconsequential at a distance twice the excavation depth.

A preconstruction survey and monitoring program are important for confirmation of appropriate installation and performance of the shoring and dewatering system. A monitoring program should be established to evaluate the effects of the construction on the adjacent buildings and improvements. The monitoring program should include survey points, inclinometers, and piezometers to monitor the movement of shoring, settlement of adjacent structures, and groundwater levels during excavation. The monitoring should provide timely data which can be used to modify the shoring system if needed.

8.4 Groundwater and Dewatering

We expect the groundwater level to vary depending upon the amount of rainfall and dewatering occurring at nearby sites. Based on the available subsurface information for the site and vicinity, we judge the groundwater surface is generally level. For design of the shoring, the groundwater level should be assumed to be at Elevation -2 feet for the site, equal to the design groundwater level for the building. However, just before construction commences we can obtain real-time groundwater levels from the piezometers, and the design can be modified, if appropriate, to reflect the actual groundwater levels. For design of permanent building walls and hydrostatic uplift on the bottom basement level, the groundwater level should be assumed to be at Elevation -2 feet. This design groundwater level is our estimate of the long-term high groundwater level. We estimate the proposed basement excavation will extend about 60 feet below the design groundwater level. The basement floor/mat and basement walls will need to be waterproofed and designed to resist hydrostatic uplift pressures.

To construct the basement, the groundwater will need to be lowered to a depth of at least three feet below the bottom of the planned excavation. Variables that will influence the performance of the dewatering system and the quantity of water produced include the shoring type installed, the number of wells, the depth and positioning of the wells, the interval over which each well is screened, and the rate at which each well is pumped. If continuous DSM or concrete diaphragm shoring walls are installed, dewatering should not be needed beyond the limits of the basement excavations. If small amounts of groundwater seep up through the base of the excavation, trench drains at the bottom of the excavation may be required to capture groundwater and direct it to a sump.

The site dewatering should be designed and implemented by an experienced dewatering contractor. We should review the dewatering system proposed by the contractor prior to installation. Prior to construction, the groundwater should be tested to evaluate if it can be discharged directly to the storm drain system or if it must be treated on-site prior to discharge. The contractor will need to obtain a dewatering and discharge permit from the City and County of San Francisco for discharging water into the local municipal storm drain system. Currently, there is a fee for disposing of construction generated water into the City's wastewater collection system. Selection of the dewatering system should be coordinated to minimize overall costs.

8.5 Soil Corrosivity

A corrosivity evaluation was performed by Cerco Analytical on two samples of near-surface fill collected from depths of one foot bgs in Boring B-8, and 6.5 feet bgs in Boring B-6 and on one sample of Colma formation sand collected near the bottom of the proposed excavation from 65 to 72 feet bgs in Boring B-6. The results of these tests are presented in Appendix B on Figures B-30a to B-30c. The results of the Cerco Analytical analyses indicate the fill at the site is moderately corrosive to corrosive and the Colma formation sand is mildly corrosive. Unprotected steel will corrode and concrete elements in contact with soil may deteriorate; protection of foundations, utilities, and other structural elements will be required. For more detail, see the recommendations of Cerco Analytical in Appendix B.

8.6 Construction Considerations

Because the bottom of the excavation will extend below groundwater, the soil at subgrade level will be near saturation even after dewatering. Additionally, the marine deposit which will likely be exposed at the excavation subgrade beneath Tower 2 is easily remolded and will lose its strength when disturbed; final site preparation and grading may be difficult. To protect the subgrade, we recommend heavy construction equipment (such as loaders or heavy excavators) not be allowed within three feet of subgrade and that final excavation be made with an excavator equipped with a smooth bucket. Following final excavation it will be useful to protect the soil subgrade by pouring a rat slab or creating a working surface comprised of a woven geotextile fabric, such as Mirafi HP 550, and overlain by at least 18 inches of uniformly graded crushed rock.

Brick and concrete pieces were encountered in the fill. In addition, building foundation elements from previous structures may be encountered. Installation of the shoring walls could be impeded by the presence of rubble and former foundations.

9.0 RECOMMENDATIONS

From a geotechnical standpoint, we conclude the site can be developed as planned, provided the recommendations presented in this section of the report are incorporated into the design and contract documents and are implemented during construction. Criteria for foundation design, together with recommendations for shoring, below grade walls, floor slabs, and seismic design are presented in this section of the report.

9.1 Foundations

We recommend that the buildings be supported on deep foundations consisting of large-diameter drilled shafts that gain support in friction in the soil and bedrock beneath the proposed basement levels. At the request of the design team, we are also providing recommendations for barrettes that gain support in friction in the soil and bedrock beneath the proposed basement levels. For both foundation types, we recommend polymer drilling slurry be used during the drilling operation of the deep foundation; the use of bentonite slurry should be excluded from the project. In addition to slurry, we recommend the use of casing if drilled shafts are used. At a minimum casing should extend into the Old Bay Clay. Casing cannot be used for barrette construction, and the control of caving in the upper portions of the barrettes could be difficult. For drilled shafts or barrettes the concrete should be placed utilizing tremie techniques to displace all of the drilling fluid.

Foundations should be designed to resist corrosion in accordance with recommendation in Appendix B. Detailed recommendations for large-diameter drilled shafts and barrettes are presented in the following sections.

9.1.1 Axial Capacity

The allowable skin friction values (compression and tension) presented in this section are based on our knowledge of the site and of the surrounding area as well as knowledge from the load tests performed on deep foundations being constructed in the site vicinity. The values are provided for estimating purposes only; actual skin friction values used in final design should be validated using full-scale load testing of piers or barrettes at the site, as recommended in Section 9.1.2. We used the Federal Highway Administration "Drilled Shafts: Construction Procedures and LRFD Design Methods" (FHWA, 2010) to estimate skin friction values. The FHWA method utilizes an alpha-method which considers undrained shear strength for cohesive soils and a beta-method which considers effective overburden and friction angle for cohesionless soil. In our analysis, we assumed the clay within the Old Bay Clay is cohesive, all sand layers were cohesionless, and the alluvium/colluvium was cohesionless. Geotechnical properties used in the calculation of the frictional capacity were developed from the results of the laboratory testing program. The shear strength used in our analyses for the clay within the Old Bay Clay unit was reduced by about 20 percent compared to the ultimate shear strength of these soils to account for the small strain anticipated within these layers (i.e. strain compatibility with the cohesionless layers).

Several methods presented in the FHWA paper were used to evaluate skin friction capacity in bedrock; however, because of the heterogeneity of the bedrock type, including its strength, we calculated a large range of likely capacities. In addition to our calculations, we reviewed the results of the recent Osterberg load testing of a large-diameter drilled shaft performed for the 181 Fremont project, which is approximately two blocks from the 1st and Mission project site (Arup, 2013). The test was performed on a 6-foot-diameter drilled shaft installed using polymer slurry. The results of the test show ultimate skin friction in bedrock of about 20,000 psf; however, the publication recommends using an ultimate skin friction of 10,000 psf in design because of variability in the bedrock characteristics. On the basis of our knowledge of the area and the bedrock present at both sites, we recommend an ultimate skin friction capacity in bedrock of 10,000 psf for large-diameter piers.

The geotechnical properties of the soil and rock and the recommended allowable skin friction capacities for the large-diameter drilled piers, are presented in Tables 4 and 5. Marine Deposits that will be exposed beneath Tower 2 are generally weak and, in isolated areas, potentially liquefiable. Accordingly, the Marine Deposits should be ignored in computation of axial capacity beneath Tower 2.

The allowable skin friction capacities in Tables 4 and 5 are for dead plus live loads and include a factor of safety (FS) of 2.0. These allowable capacities may be increased by one third for total loads, including wind and/or design seismic load conditions (i.e. a FS of 1.5 compared to the ultimate capacities). For the Risk-Targeted Maximum Considered Earthquake (MCE_R) of shaking at the site, we recommend using a FS of at least 1.15 as compared to the ultimate capacities. Tables 4 and 5 also present the soil layer types and depths beneath each tower.

TABLE 4

Estimated Allowable Skin Frictions for Large-Diameter Drilled Piers for Tower 1

Soil Layer	Depth to Top of Layer[1]	Depth to Bottom of Layer	Calculation Method[2]	Unit Weight	Effective Friction Angle	Average Undrained Shear Strength	Estimated Average Allowable Skin Friction[4]
	(feet)	(feet)		(pcf)	(°)	(psf)	(psf)
Colma	72	74	Beta	133	38	-	150
Old Bay Clay 1 (Crust)	74	88	Alpha	116	-	2600[3]	715
Old Bay Clay 2	88	102	Alpha	112	-	1600[3]	440
Old Bay Clay Sand	102	112	Beta	133	42		1030
Old Bay Clay 3	112	140	Alpha	112	-	1900[3]	520
Old Bay Clay 4	140	168	Alpha	112	-	2120[3]	580
Alluvium / Colluvium	168	266	Beta	125	21	-	1780
Bedrock	266	N. A.	FHWA / Previous Load Testing	140	-	-	5000

Notes:

1. Depth below existing ground surface.
2. Calculations performed using "Drilled Shafts: Construction Procedures and LRFD Design Methods" (FHWA, 2010).
3. Shear strengths used to develop allowable skin friction capacities in the Old Bay Clay were reduced to account for strain compatibility with other layer types.
4. Estimated average allowable skin friction values assume polymer slurry is used. Values include a factor of safety of 2.0.

TABLE 5

Estimated Allowable Skin Frictions for Large-Diameter Drilled Piers for Tower 2

Soil Layer	Depth to Top of Layer[1]	Depth to Bottom of Layer	Calculation Method[2]	Unit Weight	Effective Friction Angle	Average Undrained Shear Strength	Estimated Average Allowable Skin Friction[4]
	(feet)	(feet)		(pcf)	(°)	(psf)	(psf)
Marine Deposits[5]	58	67	NA	120	–	–	NA
Colma	67	85	Beta	133	38	-	290
Old Bay Clay 1 (Crust)	85	99	Alpha	116	-	2600[3]	715
Old Bay Clay 2	99	128	Alpha	112	-	1700[3]	465
Old Bay Clay 3	128	156	Alpha	112	-	1900[3]	520
Old Bay Clay 4	156	184	Alpha	112	-	2120[3]	580
Alluvium / Colluvium	184	268	Beta	125	21	-	1840
Bedrock	268	N. A.	FHWA / Previous Load Testing	140	-	-	5000

Notes:

1. Depth below existing ground surface.
2. Calculations performed using "Drilled Shafts: Construction Procedures and LRFD Design Methods" (FHWA, 2010).
3. Shear strengths used to develop allowable skin friction capacities in the Old Bay Clay were reduced to account for strain compatibility with other layer types.
4. Estimated average allowable skin friction values assume polymer slurry is used. Values include a factor of safety of 2.0.
5. Skin friction should be ignored in the Marine Deposits.

The estimated allowable skin friction versus depth (FS of 2.0) is presented on Figures 11 and 12 for 6-, 7-, and 8-foot-diameter drilled piers.

The axial load versus settlement response may be modeled as equivalent linear springs for static loading and design earthquake loading. Equivalent linear springs for 6-, 7-, and 8-foot-diameter drilled piers are presented below in Table 6. The values in Table 6 are applicable for use when the piers are loaded up to two-thirds of their ultimate capacity (i.e. the piers have a factor of safety equal to at least 1.5).

TABLE 6

Calculated Equivalent Linear Springs

Pier Diameter	**Equivalent Linear Spring (kips/in)**
(feet)	(feet)
6	10,500
7	13,500
8	16,500

As requested by the project structural engineer, we are also providing load-settlement backbone curves for MCE_R shaking at the site which will load the piers close to their ultimate capacity. We modeled the axial load versus settlement response as multi-linear and it is defined in terms of a percentage of the ultimate pier capacity. At the ultimate pier capacity, plunging failure is assumed to occur and additional displacement will not result in additional capacity (perfectly plastic). Due to the uncertainties of the dynamic response, upper and lower bounds for the load-settlement backbone curves were calculated and are presented in Table 7.

TABLE 7

Calculated Equivalent Linear Springs for Large-Diameter Drilled Piers at Towers 1 and 2

Pier Diameter	**Type**	**Range A, 0-50% Ultimate Capacity**	**Range B, 50-67% Ultimate Capacity**	**Range C, 67-87% Ultimate Capacity**	**Range D, 87-100% Ultimate Capacity**
(feet)		(kips/inch)	(kips/inch)	(kips/inch)	(kips/inch)
6	Lower Bound	11,000	9,500	5,500	2,500
	Upper Bound	20,500	15,500	9,500	6,000
7	Lower Bound	14,000	12,000	7,000	4,000
	Upper Bound	27,000	20,500	12,500	7,500
8	Lower Bound	17,000	15,000	8,500	4,000
	Upper Bound	33,000	24,000	15,000	9,000

The values provided in Tables 6 and 7 are valid for drilled piers with bedrock embedment lengths between 10 and 50 feet. Values for different embedment lengths can be provided upon request. Estimated spring values can be provided for barrettes once their geometry is known.

During installation of barrettes, the soil along the flat excavated sidewalls is more likely to experience squeezing and stress relaxation prior to the casting of concrete than for a drilled pier. Because of this condition, we recommend that skin friction values for barrettes be reduced from the values presented in Tables 4 and 5. For planning purposes the allowable skin friction capacities presented above in Tables 4 and 5 and in Figures 11 and 12 should be reduced by 25% for preliminary barrette design provided polymer slurry is used, caving does not occur, and similar cleanout to drilled shafts is achieved.

Considering the lengths of the deep foundations and the required time to construct the foundations, it will be difficult to properly clean the bottom of the excavations prior to concrete placement. This is likely due, in part, to the precipitation of solids out of the drilling slurry during the placement of the steel reinforcing cage. For these reasons, we recommend the contribution of end bearing to the axial capacity of the deep foundations be ignored. This recommendation is also, in part, derived from the recent testing of the large-diameter drilled piers at 181 Fremont Street, where virtually no resistance in end bearing was mobilized.

For the anticipated design column loads, we estimate the deep foundations will likely require between zero and 70 feet of embedment into bedrock. However, to limit anticipated settlements of the deep foundations and provide a more uniform foundation performance for the towers, all drilled shafts or barrettes should extend at least 1½ times their diameter or 8 feet into bedrock, whichever is greater.

A review of the preliminary foundation layout provided by MKA indicates that pier caps will incorporate 3 or fewer piers. With the large pier sections and few piers in each cap, we judge group interaction is not a factor. Accordingly, the conventional minimum pier spacing of three pier diameters to limit reductions in axial capacities is not warranted. We recommend that drilled shafts or barrettes be spaced at least 8 feet clear of one another to limit pier interaction effects or disturbance during drilling; the outer auger-tip diameter should be used when determining the pier spacing. Once the final pier layout is established, we should review the layout to check for appropriate spacing and evaluate if any group reduction factors are necessary.

The structural engineer and/or pier designer should confirm the structural capacity of the large-diameter drilled shafts or barrettes is not exceeded. As the geotechnical engineer of record, we should observe the installation and testing of the piers or barrettes to check that the subsurface conditions are as anticipated and the pier installation and testing are performed in accordance with the project documents.

9.1.2 Load Testing Program

We recommend that before production depths are selected, at least two load tests be performed on full-scale drilled shafts or barrettes to: 1) evaluate the contractor's installation procedures, 2) confirm the design frictional capacities of the drilled shafts within the various soil units, and 3) estimate production drilled shaft lengths.

Because of the large vertical load demands on the large-diameter deep foundations, full-scale load testing is typically performed using Osterberg-cells (O-Cells), which is one or a series of hydraulic jacks that are embedded within the drilled shaft or barrette. The O-Cell load test is performed by applying load with expandable jacks located between an upper and lower load plate within the test pier or barrette. After the pier or barrette has been cast and the concrete has had sufficient time to cure, the O-cell is pressurized to break the tack welds holding the cell together and to "crack" the element into an upper and lower section. Pressure can then be applied incrementally to apply a bi-directional load to the upper and lower sections of the pier or barrette. During loading, instrumentation within the foundation element allows for the continuous measurements of the stresses and strain within the foundation as well as strain along the foundation-soil interface. Because the top section provides reaction for the bottom section, and viçe versa, the load test will be limited to the maximum vertical capacity on either side of the O-cell. Once design loads are finalized, we can provide recommendations for the location of the O-cell or work with the specialty contractor hired to develop the testing methodology.

We recommend a minimum of two shafts or barrettes, one at each tower, be installed and tested at production locations. We expect test elements can be used for support of the building if they are installed in the proper locations and are not damaged during installation or testing. Test elements should be installed with the same equipment and using the same procedure that will be used for production foundations. Following testing, the void space required to perform the load tests should be filled with grout. We suggest that the load test program be performed prior to final design to optimize the deep foundation system for inclusion on the final version of the project documents. If an evaluation is to be performed for both drilled shafts and barrettes, we recommend that two of each foundation type be tested.

The test locations should be selected by the geotechnical engineer and approved by the structural engineer. If variable diameter drilled piers are planned for the project the load testing should be performed on a pier with the largest planned diameter that will be used at the project. The tests should be performed until the ultimate skin friction is mobilized in one portion of the pile (either in the upward or downward direction). This ultimate skin friction value should correspond to at least two times the allowable dead plus live load or 1½ times the total design load (including wind or seismic forces) plus the contribution of overlying soil layers that will be removed for the basement and pier cap excavations. Additional pier load tests will be required if, during production pier installation, the equipment or installation procedure deviates from the approved work plan and indicator pier load test program. The success of the load testing program is critical to validating the allowable frictional capacities of the production piers. In addition, O-cell testing and successful documentation of the results of the test can prove challenging. We recommend a specialty contract experienced in O-cell testing of similar deep foundations be retained to develop and help implement the appropriate testing procedure and design submittals.

A design submittal and installation work plan should be submitted to Langan Treadwell Rollo for review and approval at least four weeks prior to the test shaft and load test programs. The submittal and work plan should include calculations and shop drawings, and should describe the proposed pier/barrette installation equipment and methodology, including, but not limited to, equipment and drilling fluid to be used, casing depth, pier/barrette geometry and length, corrosion protection, slurry testing, concrete testing, and quality control measures, as well as the proposed load test set-up and procedure. The work plan should include a site plan showing the locations of tests relative to permanent foundation elements and a drawing showing the layout of the load test set up. The actual locations of the test elements should be selected by the geotechnical engineer in coordination with the structural engineer and the specialty pier load test contractor. Following the completion of load tests and upon receipt of the load test report, we will require at least one week to review and evaluate the load test results and propose recommendations for production shaft installation. The structural engineer should review mill certificates for the steel and any welding procedures.

9.1.3 Foundation Installation and Quality Control

It is the responsibility of the foundation contractor to design and implement an installation methodology and provide the appropriate equipment to construct the piers. In particular, the contractor should provide equipment which can drill to the required depths in bedrock of widely variable strengths; Appendix A and B provide the core logs and laboratory test results. Samples of the rock core obtained during our field exploration program can be made available to selected foundation contractors if desired. The contractor is responsible for the integrity of the finished piers/barrettes. Caving during the casting of the deep foundation elements should be prevented, and the concrete integrity should be maintained throughout the pour.

During casting of the foundation elements, concrete should be placed from the bottom up in a single operation using a tremie and/or a pumper pipe. The tremie pipe should be maintained at least 5 feet below the upper surface of the concrete during casting. The concrete should have a slump between 7 and 9 inches. As the concrete is placed, casing used to stabilize the hole can be withdrawn. The bottom of the casing should be maintained at least 3 feet below the surface of the concrete.

Because the integrity of the shaft relies on the installation procedures, we recommend quality control during and after pier production. In order to maintain hole stability, minimize relaxation of ground stresses, and leave the sides of the excavation stable, the drilling slurry should be monitored closely. The drilling slurry should be tested before introduction into the excavation and every two hours after during the beginning of production shafts. Once acceptable slurry mixes have been developed and consistent results are achieved, testing frequency can be reduced. We recommend density, Mash funnel viscosity, pH, and fluid loss tests are performed. Testing personnel should be familiar with published standardized procedures (API 2003).

We recommend crosshole sonic logging (CSL) be performed to check the structural integrity of foundation elements. Drilled shafts or barrettes are prepared for integrity testing by CSL during their construction by installation of at least four 2-inch-inside-diameter tubes. These tubes are usually attached to the reinforcement cage along the full length of the elements. After concrete has been poured, the tubes are filled with water. During testing, a transmitter emits an ultrasonic signal in one tube, and the signal is sensed some time later by the receiver in another tube. Poor concrete or soil inclusions between the tubes will delay or disrupt the signal. Crosshole sonic logging integrity testing is standardized by ASTM D6760- Standard Test Method for Integrity Testing of Concrete Deep Foundations by Ultrasonic Crosshole Testing.

9.1.4 Lateral Resistance

Large-diameter drilled shafts and barrettes should develop lateral resistance from the passive pressure acting on the upper portion of the shafts and their structural rigidity. The lateral capacity of the shafts/barrettes depends on:

- the element stiffness
- the strength of the surrounding soil
- vertical load on the element
- the allowable deflection at the top of the element
- the allowable moment capacity of the element.

We have evaluated the lateral capacity of a 6-, 7-, and 8-foot-diameter drilled shaft for use on the project. We performed our analyses for both fixed- and free-head conditions assuming deflections of between ½ and 2½ inches at the pier top. The results of our lateral pier analyses and the associated moment and shear profiles along the upper portion of the shafts are presented on Figures 13 through 18. Figures 11 and 12 are appropriate for drilled shafts beneath both Towers 1 and 2. These results take into account that some soil within the Marine Deposits beneath Tower 2 could liquefy during a major earthquake and that Old Bay Clay will likely be present a few feet beneath the top-of-pier elevation beneath Tower 1.

At this time, we do not know the geometry of the barrettes that may be used for the project. We can provide lateral capacity recommendations for barrettes if needed.

These lateral capacities are for a single shaft only. To account for group effects, the lateral load capacity of a single shaft should be multiplied by the appropriate reduction factors shown in Table 8; however, the moment profile for a single shaft with an unfactored load should be used to check the design of individual shafts in a group. If additional pier groups are used in design, we can provide additional reduction factors.

TABLE 8

Lateral Group Reduction Factors

Number of Piers within Pier Cap	Lateral Reduction Factor
1	1.00
2	0.84
3	0.78

If the basement walls are designed to resist lateral forces during an earthquake, they should be checked using passive pressures. The passive pressure mobilized is a function of the height and lateral movement of the wall. Table 9 presents the passive resistance and deformation relationship for use on this project based on the relationship developed in ASCE 41-13. On the basis of the subsurface information at the site, we recommend ultimate passive equivalent fluid weights of 390 pcf and 190 pcf for the fill above and below the water table, 225 pcf for the dune sand, 115 pcf for the marine deposit, and 350 pcf for the Colma formation. The compressibility of the waterproofing and/or drainage panel has not been accounted for in the deformation and soil response evaluation.

TABLE 9

Passive Resistance and Deformation Relationship

Deformation (δ/H)	P / P_{ult}
0.0	0.00
0.002	0.32
0.005	0.46
0.01	0.55
0.02	0.70
0.03	0.83
0.04	0.90
0.05	0.96
0.06	1.00

Notes: 1. δ/H denotes the ratio of lateral deformation (δ) over the height of the foundation element (H).

2. P/P_{ult} denotes the ratio of mobilized passive resistance over the ultimate passive resistance.

To calculate a specific horizontal soil modulus (spring), the structural designer should iterate between the demand loading and allowable deformation performance of the walls.

9.2 Shoring

The excavations for the proposed improvements should be shored. For excavations deeper than 20 feet we recommend a cutoff wall system be installed using either DSM or a concrete diaphragm methods to provide lateral support for adjacent properties and improvements. To support the excavation for the one-story deep basement at the northwestern corner of Tower 1 or for other localized shallow excavations, a soldier-pile-and-lagging temporary shoring system could be used, provided the overall depth of excavation is less than about 20 feet bgs.

The selection, design, construction, and performance of the shoring system should be the responsibility of the contractor. The shoring system should be designed by a licensed engineer experienced in the design of retaining systems and installed by an experienced shoring specialty contractor. The shoring engineer should be responsible for the design of temporary shoring in accordance with applicable regulatory requirements. Control of ground movement will depend as much on the timeliness of installation of lateral restraint as on the design. We should review the shoring plans and a representative from our office should observe the installation of the shoring.

9.2.1 Cutoff Walls

A DSM or concrete diaphragm wall with tiebacks or internal bracing should be designed using the lateral earth pressures presented on Figure 19. The pressures presented in Figure 19 are for excavations for three or four basement levels and assume a level ground condition behind the shoring. If additional depths of the excavation are wanted, we can provide additional recommendations, as needed. Where nearby buildings are above a 1:1 line drawn up from the bottom of the excavation and are not underpinned, an additional surcharge to account for the weight of the buildings should be added to the shoring pressures shown. In computing the passive pressure, we have assumed the groundwater level within the site will be lowered to a depth of at least three feet below the bottom of the excavation, while the groundwater level outside the shoring remains close to the design groundwater level. The passive resistance and the active pressure are shown to a depth of 10 feet below the bottom of the excavation. Penetration greater than 10 feet may be required to achieve lateral stability and resist downward loading of the tiebacks, if they are used. Additionally, to provide groundwater cutoff, the shoring system should extended sufficiently into the Old Bay Clay.

If traffic is allowed within 10 feet of the shoring, a uniform surcharge load of 100 psf over the top 10 feet of wall should be added to the design. An increase in lateral design pressure for the shoring could be required where heavy construction equipment or stockpiled materials are within a distance equal to the shoring depth. Construction equipment should not be allowed within five feet from the edge of the excavation unless the shoring is specifically designed for the appropriate surcharge. The increase in pressure should be computed after the surcharge loads are known. The anticipated deflections of the shoring system should be estimated to check if they are acceptable.

9.2.2 Soldier-Pile-and-Lagging Shoring

A soldier pile and lagging shoring system with tiebacks or internal bracing should be designed using the lateral earth pressures presented on Figure 20. The pressures presented on Figure 20 are for a level ground condition behind the shoring. Soldier-pile-and-lagging shoring may be used where the anticipated excavation depth is less than 20 feet deep. Where nearby buildings are within a distance equal to the height of the shoring system, and are not underpinned, an additional surcharge to account for the weight of the buildings should be added to the shoring pressures shown. In calculating the pressures on Figure 20, we assumed the interior and exterior of the excavation will be dewatered to a depth of at least three feet below the bottom of the excavation.

Penetration of the soldier piles for the tied-back shoring system should extend below the excavation bottom to a depth sufficient to achieve lateral stability and resist the downward loading of the tiebacks.

If traffic occurs within 10 feet of the shoring depth, a uniform surcharge load of 100 psf should be added to the design on the top 10 feet of the shoring. An increase in lateral design pressure for the shoring may be required where heavy construction equipment or stockpiled materials are within a distance equal to the shoring depth. Construction equipment should not be allowed within five feet from the edge of the excavation unless the shoring is specifically designed for the appropriate surcharge. The increase in pressure should be computed after the surcharge loads are known. The anticipated deflections of the shoring system should be estimated to check if they are acceptable.

9.2.3 Penetration Depth of Shoring

The shoring designer should evaluate the required penetration depth of the shoring walls for support of lateral and vertical loads and for groundwater cutoff. The cutoff walls should be designed with sufficient embedment into the Old Bay Clay to provide groundwater cutoff. In addition, the shoring walls should have sufficient vertical capacity to support the vertical load component of the tiebacks and any other vertical load acting on the walls (e.g. adjacent building loads or underpinning loads). Above the excavation level, the vertical capacity of the shoring will be provided by the friction along the back of the shoring walls; below the excavation level, by friction along both sides of the shoring wall. To compute the vertical capacity of the shoring above the excavation level we recommend neglecting the friction in the fill, Dune sand, and marine deposits, and computing the allowable friction in the Colma formation and underlying strata using a friction coefficient of 0.3 times the horizontal component of the tiebacks or braces. Below the excavation level, the vertical capacity should be determined using an allowable skin friction of 800 psf on both sides of shoring for cutoff walls extending into Colma formation and Old Bay Clay and 500 psf on perimeter of the soldier piles for soldier-pile-and-lagging.

9.2.4 Tiebacks

Temporary tiebacks may be used to restrain the shoring. The vertical load from the temporary tiebacks should be accounted for in the design of the vertical elements. Design criteria for tiebacks are presented on Figure 19.

Tiebacks should derive their load-carrying capacity from the soil behind an imaginary line sloping upward from a point H/5 feet away from the bottom of the excavation and sloping upwards at 60 degrees from the horizontal, where H is the wall height in feet. Tiebacks with bar and strand tendons should have a minimum unbonded length of 10 and 15 feet, respectively. The unbonded length should be created by placing an oversized rigid smooth plastic casing (i.e. PVC pipe) over the bars or strands; flexible plastic does not provide adequate bond-break for the unbonded zone. All tiebacks should have a minimum bonded length of 15 feet and be spaced at least six times the grouted diameter of the bonded zone or four feet, whichever is less. The bottom of the excavation should not extend more than two feet below a row of unsecured tiebacks.

Tieback allowable capacity will depend upon the drilling method, hole diameter, soil type, grout pressure, post grouting, and workmanship. The use of solid-flight augers to install tiebacks in sand and the fill can result in loss of soil and settlement of structures or the ground surface located above the tiebacks. Therefore, solid flight augers or Titan type anchors should not be used for tieback installation. We recommend a smooth cased tieback installation method (such as a Klemm type rig) be used. For estimating purposes, we recommend using an allowable skin friction value of 1,000 psf for post-grouted tiebacks, as shown on Figure 19.

The contractor should be responsible for determining the actual length of tiebacks required to resist the lateral earth and water pressures imposed on the temporary retaining systems. Determination of the tieback length should be based on the contractor's familiarity with his installation method. The computed bond length should be confirmed by a performance- and proof-testing program under our observation. Replacement tiebacks should be installed for tiebacks that fail the load tests.

9.2.5 Tieback Testing

Each tieback should be tested. The first two production tiebacks and two percent of the remaining tiebacks should be performance-tested to at least 1.25 times the design load. All other temporary tiebacks should be proof-tested to at least 1.25 times the design load. The performance tests will be used to determine the load carrying capacity of the tiebacks and the residual movement. The performance-tested tiebacks should be checked 24 hours after initial lock off to confirm stress relaxation has not occurred. The geotechnical engineer should evaluate the results of the performance tests and determine if creep testing is required and select the tiebacks that should be creep tested. If any tiebacks fail to meet the proof-testing requirements, additional tiebacks should be added to compensate for the deficiency, as determined by the shoring designer.

During testing the maximum test load should not exceed 80 percent of the yield strength of the tendons or bars. The movement of each tieback should be monitored with a free-standing, tripod-mounted dial gauge during performance and proof testing.

9.2.5.1 Performance Tests

The performance tests will be used to determine the load carrying capacity and the load-deformation behavior of the tiebacks. It is also used to separate and identify the causes of movement, and to check that the designed unbonded length has been established.

In the performance test, the load applied to the tieback and its movement is measured during several cycles of incremental loading and unloading. The maximum test load should be held for a minimum of 10 minutes, with readings taken at 1, 2, 3, 4, 5, 6 and 10 minutes. If the difference between the 1- and 10-minute reading is less than 0.04 inch during the loading, the test is discontinued. If the difference is more than 0.04 inch, the holding period is extended to 60 minutes, and the movements should be recorded at 15, 20, 25, 30, 45, and 60 minutes.

The geotechnical engineer should evaluate the results of the performance tests and determine if creep testing is required and select the tiebacks that should be creep tested. Creep tests should be performed in accordance with provision of "Recommendations for prestressed Rock and Soil Anchors" of Post-Tensioning Institute.

9.2.5.2 Proof Tests

A proof test is a simple test which is used to measure the total movement of the tiebacks during one cycle of incremental loading. The maximum test load should be held for a minimum of 10 minutes, with readings taken at 1, 2, 3, 4, 5, 6, and 10 minutes. If the difference between the 1- and 10-minute reading is less than 0.04 inch, the test is discontinued. If the difference is more than 0.04 inch, the load should be maintained and the observation is continued until the creep rate can be determined. The proof test results should be compared to the performance test results. Any significant variation from the performance test results will require performance testing on the tieback.

We should evaluate the results of performance and proof tests to check that the tiebacks can resist the design load. For any tiebacks that fail to meet the performance and proof testing requirements, additional tiebacks should be installed to compensate for the deficiency, as required by the shoring designer.

9.2.5.3 Acceptance Criteria

The geotechnical engineer should evaluate the tieback test results and determine whether the tiebacks are acceptable. A performance- or proof-tested tieback with a ten-minute hold is acceptable if the tieback carries the maximum test load with less than 0.04 inch movement between one and ten minutes, and total movement at the maximum test load exceeds 80 percent of the theoretical elastic elongation of the unbonded length.

A performance- or proof-tested tieback with a 60-minute hold is acceptable if the tieback carries the maximum test load with a creep rate that does not exceed 0.08 inch/log cycle of time, and total movement at the maximum test load exceeds 80 percent of the theoretical elastic elongation of the unbonded length.

If the total movement of the tiebacks at the maximum test load does not exceed 80 percent of the theoretical elastic elongation of the unbonded length, the tieback should be replaced by the contractor.

9.2.6 Underpinning

If the planned shoring walls are within four feet of adjacent structures supported on shallow foundations and the surcharge loads from the adjacent structures are not designed for an additional surcharge, these buildings should be underpinned. This will likely include 84 1st Street and 1 Ecker Street. Based on the current alignment of the proposed basement levels, the remaining surrounding buildings are set back far enough away from the excavation that they will not be underpinned; however, the shoring wall should be designed to support loads from the existing foundations by applying lateral surcharges to the wall. Once the actual foundations of these buildings have been determined, either from a review of plans at the Department of Building Inspection or by excavating test pits adjacent to these buildings, we can assist the shoring designer with determining the distribution of the buildings loads onto the shoring system.

Where a soldier-pile-and-lagging wall or a cutoff wall is used, underpinning elements can consist of steel piles installed in slant-drilled shafts. Underpinning piles should extend at least ten feet below the bottom of the planned excavation and should be designed for the pressures presented on Figures 19 and 20.

For vertical resistance, the friction above the excavation depth should be ignored for underpinning piles. Below the excavation level underpinning piles should be designed using an allowable friction of 500 psf in the Dune sand and marine deposits and or 800 psf in the Colma formation and Old Bay Clay.

We recommend underpinning piles be preloaded (jacked) prior to dry packing/grouting to reduce settlement as the foundation load is transferred to the piles. To reduce movement and provide adequate foundation support during installation of the underpinning piles, adjacent piles should not be drilled until they have been dry packed or grouted.

9.3 Dewatering

The groundwater should be drawn down to a depth of at least three feet below the bottom of the excavation. This level should be maintained until sufficient building weight and/or uplift capacity is available to resist the hydrostatic uplift pressure of the groundwater based on a design groundwater elevation of -2 feet. Elevator and sump pits can be locally dewatered. Adjacent site improvements should be monitored for vertical movement caused by the dewatering.

The number and depth of dewatering wells should be determined by a specialty dewatering contractor. The volume of water discharged should be monitored and a record of the amount should be submitted to the owner. The design of the selected dewatering system should be provided to the shoring designer so that the temporary groundwater elevation can be incorporated in the shoring design. The structural engineer should evaluate and provide recommendations when the dewatering system can be turned off.

9.4 Tiedown Anchors

If the weight of a building is not sufficient to overcome the hydrostatic uplift loads or the mat cannot resist the uplift pressure between columns, the net upward pressure can be resisted by tiedowns. Tiedowns consist of relatively small-diameter, drilled, grout-filled shafts with steel bars or tendons embedded in the grout. The tiedowns develop their uplift resistance from friction between the perimeter of the shaft and the surrounding soil.

Tiedowns should be spaced at least four shaft diameters apart, with a minimum spacing of four feet. Because specialty contractors who install these tiedowns use different installation procedures, the uplift capacity of the tiedowns will vary with the procedure. For planning purposes, however, we recommend using an allowable friction of 950 psf for permanent uplift loads. Higher values can be obtained depending upon the installation techniques employed by the contractor and the results of pullout tests; anchors should be designed with factors of safety of at least 2.0.

Tiedowns will extend significantly beneath the bottom of the excavation and planned dewatering wells. Therefore, it is very likely the tiedowns will be subjected to a significant upgradient of groundwater pressure. The specialty tiedown contractor should anticipate this condition and take the necessary precautions to reduce water flow up the tiedown interface, which can significantly disrupt the tiedown installation, grout cover, and ultimate capacity of the tiedown.

Special attention should be given to waterproofing the connections between the tiedowns and the mat slab. Because the tiedowns will be permanent, we recommend they be double corrosion protected. Corrosion protection requirements for the bonded and unbonded length, and stressing anchorage are outlined below:

- encapsulations used to provide an additional corrosion protection layer over the tendon bond length should consist of a grout-filled, corrugated plastic encapsulation, or grout-filled deformed steel tube; the prestressing steel can be grouted inside the encapsulation prior to inserting the tendon into the drill hole or after the tendon has been placed; centralizers or grouting techniques should provide a minimum of ½ inch of grout cover over the encapsulation
- a sheath filled with corrosion inhibiting compound or grout, or a heat shrinkable tube internally coated with a mastic compound should be used to provide corrosion protection of the unbonded length
- the trumpet should be sealed to the bearing plate and overlap the unbonded length corrosion protection by at least four inches; it should be completely filled with a corrosion inhibiting compound or grout.

The tiedowns will be installed below the water table; therefore, the contractor should use an auger-cast system or be prepared to case the holes to prevent caving. High strength bars or strands may be used as tensile reinforcement in the anchors. For stressing, the steel bars and strands should have at least 10 and 15 feet of free length, respectively. If strands are used, a significant lock-off load will be required (roughly 50 to 75 percent of the design load), to limit deformation of the tiedown under the hydrostatic loading. If a steel bar is used, a significantly lower lock-off load can be used.

The bond length of tiedowns should be at least 15 feet. The design capacity of the tiedowns should be confirmed by a performance- and proof-test program conducted under our observation. We recommend the first two production tiedowns and five percent of the remaining tiedowns be performance tested to two times the design load. The remainder should be proof tested to 1.5 times the design load. The tiedowns should be tested under our observation. The test procedure and acceptance criteria described in Section 8.1.5 for tieback testing should also be used for tiedowns. Replacement tiedowns should be provided, as directed by the structural engineer, for tiedowns that fail the test. All tiedowns should be locked off. The allowable amount of deformation after the tiedown is locked off should be determined by the structural engineer. The actual lock-off load will be determined after initial testing of the first few tiedowns.

9.5 Below-Grade Walls

We recommend that basement walls be designed using the lateral pressures presented in Table 10. Walls that are within 10 feet of the streets should be designed for an additional lateral pressure of 100 psf in the upper 10 feet. If surcharge loads occur within the zone of influence (defined by an imaginary plane projected up from the bottom of the wall at a 30-degree angle from horizontal), a surcharge pressure should be included in the wall design.

Because the site is in a seismically active area, the design should be checked for static and seismic conditions to evaluate the governing condition. Under earthquake loading conditions, there will be a seismic increment that should be added to the active earth pressures. We used the procedures outlined in Sitar, et. al., (2012) to compute the seismic earth pressure. The more critical condition of either at-rest pressure or active pressure plus seismic increment should be checked. At-rest and total pressures (active plus seismic pressure increment) for a Design Earthquake (DE) level and for the Risk-Targeted Maximum Considered Earthquake (MCE_R) of shaking at the site, for level backfill are presented in Table 10.

TABLE 10

Earth Pressures for Below-Grade Wall Design

Retained Condition	Static Conditions		Seismic Conditions	
			Total Pressure (Active Pressure plus Seismic Pressure Increment)	
	Active Pressure (pcf)	At-Rest Pressure (pcf)	DE (PGA = 0.40g) (pcf)	MCE_R (PGA = 0.60g) (pcf)
Soil above groundwater, drained condition	38	57	64	81
Soil below groundwater, or undrained wall above water	83	92	97	106

To protect against moisture migration, basement walls should be waterproofed and water stops should be placed across all construction joints. The waterproofing should be placed directly against the backside of the walls. The waterproofing should be designed by a consultant with local experience.

Walls should be properly backdrained if they are designed for the drained condition. One acceptable method for backdraining the walls is to place a prefabricated drainage panel against the back side of the wall. The drainage panel should extend at least three feet below the design groundwater elevation, to Elevation -5 feet. We should check the manufacturer's specifications for the proposed drainage panel material to verify it is appropriate for its intended use.

9.6 Site Preparation

Remnants of previous buildings and structures should be excavated and removed from the areas to receive new improvements. Existing utility lines may be abandoned in place or removed. All remaining utilities below the proposed building may be abandoned in place provided they will not impact future utilities or building foundations and are filled with lean concrete or cement grout to the property line. Excavations made during site preparation to remove utilities, old foundations, tanks, or other improvements should be filled with lean concrete or properly compacted engineered fill, as recommended in Section 9.7.

9.7 Earthwork

The majority of the site grading for the project will consist of a deep excavation for the proposed basements. Outside the excavation, additional grading activities at the site will likely include site grading to create a level pad for on-grade improvements around the new buildings, backfilling existing basements that that extend beyond the new basement footprints, utility trench backfill, and subgrade preparation for new sidewalks after construction of the building. Prior to placing any new fill, the exposed ground surface should be:

- scarified to a minimum depth of six inches
- moisture conditioned to near optimum
- compacted to at least 95 percent relative compaction[15].

All fill and backfill may consist of either on-site material or approved imported fill provided that it meets the criteria set forth below. Fill should be placed in horizontal layers not exceeding eight inches in loose thickness, moisture-conditioned to near optimum moisture content, and compacted to at least 95 percent relative compaction. Subgrade beneath city streets and sidewalks should be prepared in accordance with City and County of San Francisco Standard Specifications.

From a geotechnical standpoint, most on-site soil free of organic matter and rocks or lumps larger than three inches in greatest dimension should be suitable for use as fill or backfill provided it is properly moisture conditioned. Clay from the marine deposit is not suitable material for use as fill. Lean concrete or controlled density fill may be used to backfill areas not accessible to compaction equipment.

Fill material should also be free of organic debris, hazardous material, and rocks or lumps larger than three inches in greatest dimension. All material to be used as fill should be non-corrosive and should

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15 Relative compaction refers to the in-place dry density of soil expressed as a percentage of the maximum dry density of the same material, as determined by the ASTM D1557-09 laboratory compaction procedure.

have a low expansion potential, defined by a liquid limit (LL) less than 25 and a plasticity index (PI) lower than 8. Samples of all imported fill should be submitted to the geotechnical engineer for testing at least 72 hours before delivery to the site

9.8 Utilities

Utilities should be designed to accommodate the estimated 6 inches of differential seismically-induced settlement between the building and exterior improvements, in addition to the 1½ to 2 inches of building settlement that will likely occur under static conditions. Where utilities enter and exit the building these settlements should be accommodated over a short span, as the differential settlement will be abrupt. Flexible connections which allow for the anticipated differential movement should be used.

Utility trenches should be excavated a minimum of four inches below the bottom of pipes or conduits and have clearances of at least four inches on both sides. Where necessary, trench excavations should be shored and braced, in accordance with all safety regulations, to prevent cave-ins. If trenches extend below the groundwater level, it will be necessary to dewater them to keep the trench base from softening and to allow for placement of the pipe utilities and backfill.

To provide uniform support, pipes or conduits should be bedded on a minimum of four inches of sand or fine gravel. After pipes and conduits are tested, inspected (if required), and approved, they should be covered to a depth of six inches with sand or fine gravel, which should then be mechanically tamped. Backfill should be placed in accordance with the recommendations for engineered fill in Section 9.7.

Utility backfilling in the sidewalks and streets should comply with Section 703 of the City of San Francisco Standard Specifications where utilities are in the City and County of San Francisco right-of-way, except jetting of trench backfill should not be permitted. Special care should be taken in controlling utility backfilling in pavement areas. Poor compaction may cause excessive settlements, resulting in damage to exterior improvements.

9.9 Seismic Design

The following subsections present the recommended site-specific response spectra and time histories (Section 9.9.1) and the code based mapped values per 2013 SFBC (Section 9.9.2).

9.9.1 Site-Specific Response Spectra and Time Histories

We expect this site will experience very strong ground shaking during a major earthquake on any of the nearby faults. To estimate ground shaking at the site, we performed a Probabilistic Seismic Hazard Analysis (PSHA) and deterministic analysis to develop site-specific horizontal response spectra for three levels of shaking corresponding to the Risk-Targeted Maximum Considered Earthquake (MCE_R) and the Design Earthquake (DE) per the 2013 SFBC/ASCE 7-10 and the Serviceability Level Earthquake (SLE) per the San Francisco Administrative Bulletin (AB) 083 *Requirements and Guidelines for the Seismic Design of New Tall Buildings using Non-Prescriptive Seismic-Design Procedures.*

The MCER is defined in the 2013 CBC as the lesser of the probabilistic spectrum having 2 percent probability of exceedance in 50 years (2,475 year return period) or the 84th percentile deterministic event on the governing fault both in the maximum direction; the DE is defined as 2/3 of the MCER. The SLE spectrum is defined as a probabilistic spectrum with a 50 percent probability of exceedance in 30 years (43 year return period).

The probabilistic seismic hazard analysis (PSHA) was performed using the computer code EZFRISK 7.62 (Risk Engineering 2012). This approach is based on the probabilistic seismic hazard model developed by Cornell (1973) and McGuire (1976). Our analysis modeled the faults in the Bay Area as linear sources and earthquake activities were assigned to the faults based on historical and geologic data. Details of our analyses are presented in Appendix D.

To account for the effect of the new buildings and foundation on the dynamic response of the site, we performed dynamic soil structural interaction analyses. The objectives of our dynamic soil-structure interaction (SSI) analyses were to evaluate the potential effects the proposed basement and deep foundations upon the seismic response of the site, and develop recommended site-specific response spectra for use in the seismic structural evaluations and design for the three levels of shaking considered for the project. Our modeling considered the approximate configurations of the proposed structure, and a generalized soil profile below the site down to bedrock. The SSI analyses was performed using the computer program FLAC (version 7.0 by Itasca Consulting Group Inc.) which is a two dimensional finite difference computer program.

Based on the results of the seismic and SSI modeling we developed a recommended horizontal ground surface spectra for the three levels of shaking. This recommended spectra is shown on Figure 21. Digitized values of the recommended MCER, and DE for a damping ratio of 5 percent and the SLE spectrum for a damping ratio of 2.5 percent are presented in Table 11.

TABLE 11

Recommended Foundation Level Spectral Acceleration (g)

Period (Seconds)	MCE - 5% damping
0.00	0.480
0.12	1.200
0.26	1.200
0.30	1.320
0.41	1.320
0.44	1.200
0.60	1.200
0.75	0.960
1.00	0.720
1.10	0.655
1.20	0.600
1.30	0.554
1.40	0.514
1.50	0.480
1.60	0.450
1.70	0.424
1.80	0.400
1.90	0.403
2.00	0.407
2.10	0.420
2.20	0.439
2.30	0.428
2.40	0.417
2.50	0.405
2.60	0.395
2.70	0.383
2.80	0.370
2.90	0.357
3.00	0.346
3.20	0.323
3.40	0.298
3.60	0.276
3.80	0.247
4.00	0.237
4.25	0.225
4.50	0.211
4.75	0.197
5.00	0.185
5.50	0.165
6.00	0.154
7.00	0.135
8.00	0.113
9.00	0.088
10.00	0.072

Period (Seconds)	DE - 5% damping
0.00	0.320
0.12	0.800
0.20	0.800
0.29	1.000
0.41	1.000
0.46	0.800
0.60	0.800
0.75	0.640
1.00	0.480
1.10	0.436
1.20	0.419
1.30	0.407
1.40	0.373
1.50	0.338
1.60	0.335
1.70	0.314
1.80	0.302
1.90	0.297
2.00	0.294
2.10	0.284
2.20	0.295
2.30	0.287
2.40	0.274
2.50	0.265
2.60	0.258
2.70	0.252
2.80	0.244
2.90	0.235
3.00	0.228
3.20	0.216
3.40	0.199
3.60	0.185
3.80	0.168
4.00	0.160
4.25	0.153
4.50	0.144
4.75	0.135
5.00	0.127
5.50	0.113
6.00	0.105
7.00	0.091
8.00	0.077

Period (Seconds)	SLE – 2.5% damping
0.01	0.182
0.10	0.356
0.20	0.490
0.30	0.499
0.40	0.445
0.50	0.381
0.60	0.324
0.75	0.266
1.00	0.198
1.50	0.124
2.00	0.087
3.00	0.050
4.00	0.033
5.00	0.024
6.00	0.017
7.00	0.013
8.00	0.010

Because site-specific procedure was used to determine the recommended MCER and DE response spectra, the corresponding values of SMS, SM1, SDS and SD1 per Section 21.4 of ASCE 7-10 should be used as shown in Table 12.

TABLE 12

Design Spectral Acceleration Value

Parameter	Spectral Acceleration Value (g's)
S_{MS}	1.200
S_{M1}	0.814*
S_{DS}	0.900
S_{D1}	0.588*

*Governed by the spectral value at 2.0 seconds

9.9.2 Code Based Seismic Design Values

Using subsurface data from our geotechnical investigation and the measurements of shear wave velocity at the site, we estimated the average shear wave velocity of the upper 100 feet to be about 700 feet per second (210 meters per second). Based on the subsurface conditions, the site is classified as a stiff soil site, site class D. For seismic design in accordance with the provisions of the 2013 CBC/ASCE 7-10 we recommend the following:

- Site Class D
- Risk Targeted Maximum Considered Earthquake (MCE_R) S_S and S_1 of 1.500g and 0.600g, respectively.
- Site Modification Factors, F_a and F_v of 1.0 and 1.5.
- Risk Targeted MCE_R spectral response acceleration parameters at short periods, S_{MS}, and at one-second period, S_{M1}, of 1.500g and 0.900g, respectively.
- Design Earthquake (DE) spectral response acceleration parameters at short period, S_{DS}, and at one-second period, S_{D1}, of 1.000g and 0.600g, respectively.

9.10 Construction Monitoring

To monitor ground movements, groundwater levels, and shoring movements during the excavation activities, we recommend monitoring include:

- **Slope inclinometers:** We recommend installing slope inclinometers adjacent to the proposed shoring systems. Where possible, slope inclinometers should be installed behind each of the exterior walls. We recommend six inclinometers be installed behind the shoring system around the deep Tower 1 excavation and four inclinometers be installed behind the shoring around the deep Tower 2 basement excavation. If space does not permit some of the inclinometers to be installed behind the shoring, they may be embedded in the temporary shoring walls; however, it is important that some of the inclinometers be installed to a depth of at least two times the maximum anticipated excavation depth.

- **Piezometers:** We recommend at least six piezometers be installed at the site, with at least four located outside of the excavation and two inside the excavation. The upper portions of the piezometers should be properly sealed with cement-bentonite mix to limit the potential for surface water infiltration.

- **Survey:** Survey points should be installed on the adjacent buildings, streets, and improvements that are within two times the depth of the proposed excavation. These points should be used to monitor the vertical and horizontal movements of the shoring and these improvements. These points should be selected with our input, as geotechnical engineer of record, so the surveying can provide the most value to the project. In addition, 3D laser scanning can be used to provide a baseline survey and documentation of the site and surrounding improvements prior to construction. This data can then be compared to a post-construction 3D laser scan if required.

We should obtain inclinometer and piezometer readings regularly. Initially, depending upon the speed of excavation, the instrumentation should be read about every one to two weeks. Surveying points should also be read about every one to two weeks. The frequency of instrumentation and survey readings may, in the later stage of construction, be modified as appropriate. Results of survey monitoring should be submitted to us and the construction team after each weekly reading for review in a format that is easy to interpret.

In addition, the conditions of existing buildings within 140 feet of the site should be photographed and surveyed prior to the start of construction and monitored periodically during construction. A thorough crack survey of the adjacent buildings, especially those surrounding the proposed excavation should be performed prior to the start of construction and immediately after its completion.

10.0 SERVICES DURING DESIGN, CONSTRUCTION DOCUMENTS, AND CONSTRUCTION QUALITY ASSURANCE

During final design we should be retained to consult with the design team as geotechnical questions arise. Technical specifications and design drawings should incorporate Langan Treadwell Rollo's recommendations. When authorized, Langan Treadwell Rollo will assist the design team in preparing specification sections related to geotechnical issues such as foundation installation and testing, temporary shoring and excavation support, earthwork, and backfill. Langan Treadwell Rollo should also, when authorized, review the project plans, as well as Contractor submittals relating to materials and construction procedures for geotechnical work, to check that the designs incorporate the intent of our recommendations.

Langan Treadwell Rollo has investigated and interpreted the site subsurface conditions and developed the foundation design recommendations contained herein, and is therefore best suited to perform quality assurance observation and testing of geotechnical-related work during construction. The work requiring quality assurance confirmation and/or special inspections per the Building Code includes, but is not limited to, installation and testing of foundations, earthwork, backfill, and excavation support. In fulfillment of these duties, during construction we should observe the installation of the test piers/barrettes, pier/barrette load testing, production pier/barrette installation, installation of temporary shoring, including testing of tiebacks, and excavation. Prior to excavation activities we should observe the installation of piezometers and inclinometers and obtain baseline readings. During excavation, we should obtain readings on a regular basis. We will also review monitoring data pertaining to shoring system performance and settlement of adjacent structures provided by the surveyor. We should also observe any fill placement and perform field density tests to check that adequate fill compaction has been achieved.

Recognizing that construction observation is the final stage of geotechnical evaluation, quality assurance observation during construction by Langan Treadwell Rollo is necessary to confirm the design assumptions and design elements, to maintain our continuity of responsibility on this project, and allow us to make changes to our recommendations, as necessary. The foundation system and general geotechnical construction methods recommended herein are predicated upon Langan Treadwell Rollo reviewing the final design and providing construction observation services for the owner. Should Langan Treadwell Rollo not be retained for these services, we cannot assume the role of geotechnical engineer of record, and the entity providing the final design and construction observation services must serve as the engineer of record.

11.0 OWNER AND CONTRACTOR RESPONSIBILITIES

The Contractor is responsible for construction quality control, which includes satisfactorily constructing the foundation system and any associated temporary works to achieve the design intent while not adversely impacting or causing loss of support to neighboring properties, structures, utilities, roadways, etc. Construction activities that can alter the existing ground conditions such as excavation, fill placement, foundation construction, dewatering, etc. can also induce stresses, vibrations, and movements in nearby structures and utilities, and disturb occupants. Contractors are solely responsible to ensure that their activities will not adversely affect the structures and utilities. Contractors must also take all necessary measures to protect the existing structures, utilities, etc. during construction.

12.0 LIMITATIONS

The conclusions and recommendations provided in this report result from our interpretation of the geotechnical conditions existing at the site inferred from a limited number of borings as well as architectural and structural information provided by Heller Manus Architects and MKA. Actual subsurface conditions could vary. Recommendations provided are dependent upon one another and no recommendation should be followed independent of the others. Any proposed changes in structures, depths of excavation, or their locations should be brought to Langan Treadwell Rollo's attention as soon as possible so that we can determine whether such changes affect our recommendations. Information on subsurface strata and groundwater levels shown on the logs represent conditions encountered only at the locations indicated and at the time of investigation. If different conditions are encountered during construction, they should immediately be brought to Langan Treadwell Rollo's attention for evaluation, as they may affect our recommendations.

This report has been prepared to assist the Owner, architect, and structural engineer in the design process and is only applicable to the design of the specific project identified. The information in this report cannot be utilized or depended on by engineers or contractors who are involved in evaluations or designs of facilities on adjacent properties which are beyond the limits of that which is the specific subject of this report.

Environmental issues (such as permitting or potentially contaminated soil and groundwater) are outside the scope of this study and should be addressed in a separate evaluation.

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FIGURES

EXPLANATION

Reference:
Location Map of Ground Failures in San Francisco City and County, USGS and
World street basemap is provided through Langan's Esri ArcGIS software licensing and ArcGIS online.
Credits: Sources: Esri, DeLorme, NAVTEQ, USGS, Intermap, iPC, NRCAN.

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1ST AND MISSION STREETS DEVELOPMENT
San Francisco, California

SITE LOCATION MAP

LANGAN TREADWELL ROLLO

Date 07/01/15	Project No. 750621401	Figure 1

Path: \\langan.com\data\OA\data4\750621401\ArcGIS\ArcMap_Documents\Site Location Map.mxd User: cyoung

Reference: Base map from a drawing titled "Site Survey of a Portion of Assessor's Block No. 3708 for FM Owner LLC," by Martin M. Ron Associates, Sheets 1 and 2, undated.

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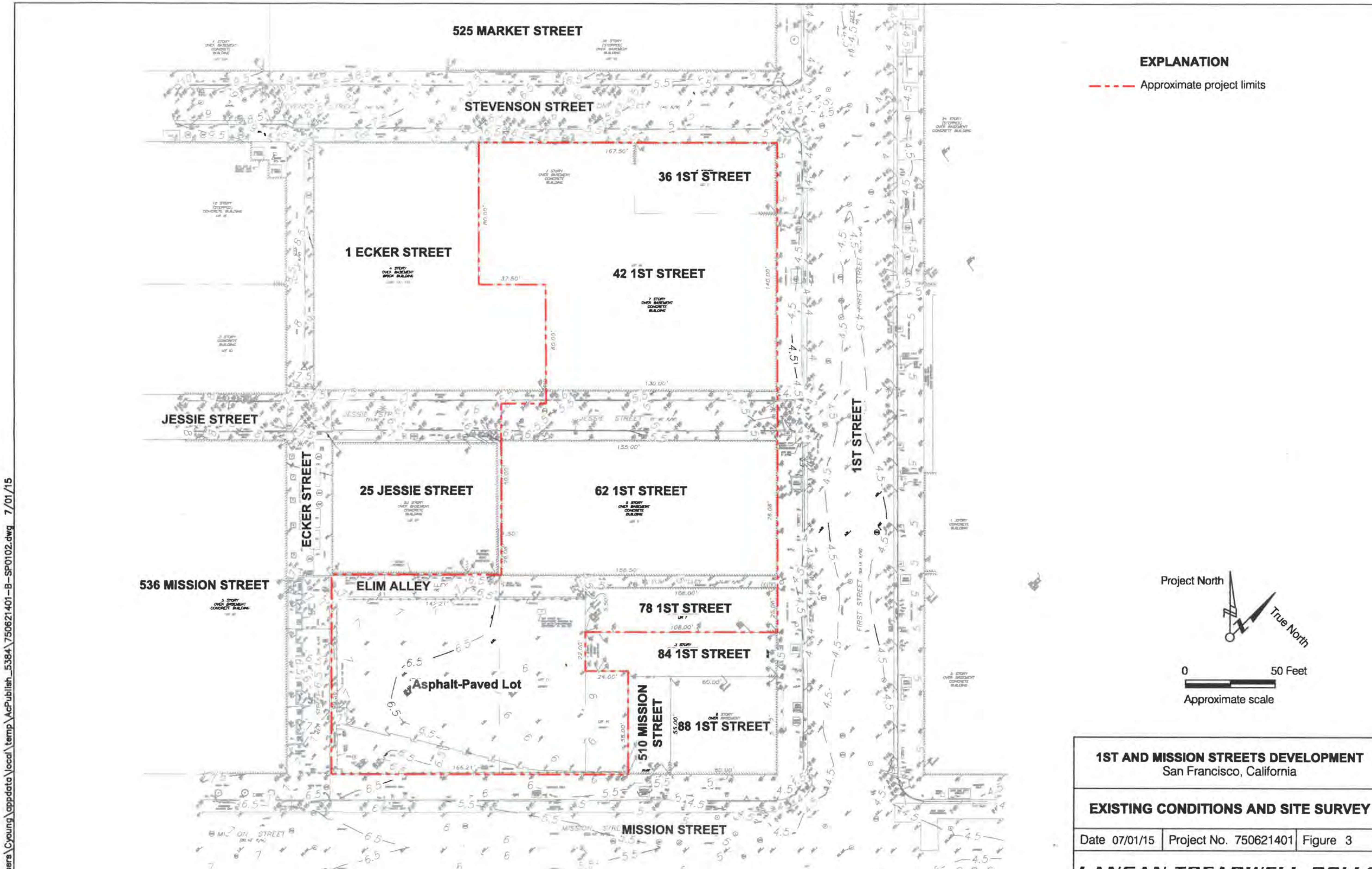
1ST AND MISSION STREETS DEVELOPMENT
San Francisco, California

SITE PLAN

Date 07/01/15	Project No. 750621401	Figure 2

LANGAN TREADWELL ROLLO



Reference: Base map from a drawing titled "Site Survey of a Portion of Assessor's Block No. 3708 for FM Owner LLC," by Martin M. Ron Associates, Sheets 1 and 2, undated.

1ST AND MISSION STREETS DEVELOPMENT
San Francisco, California

EXISTING CONDITIONS AND SITE SURVEY

Date 07/01/15	Project No. 750621401	Figure 3

LANGAN TREADWELL ROLLO

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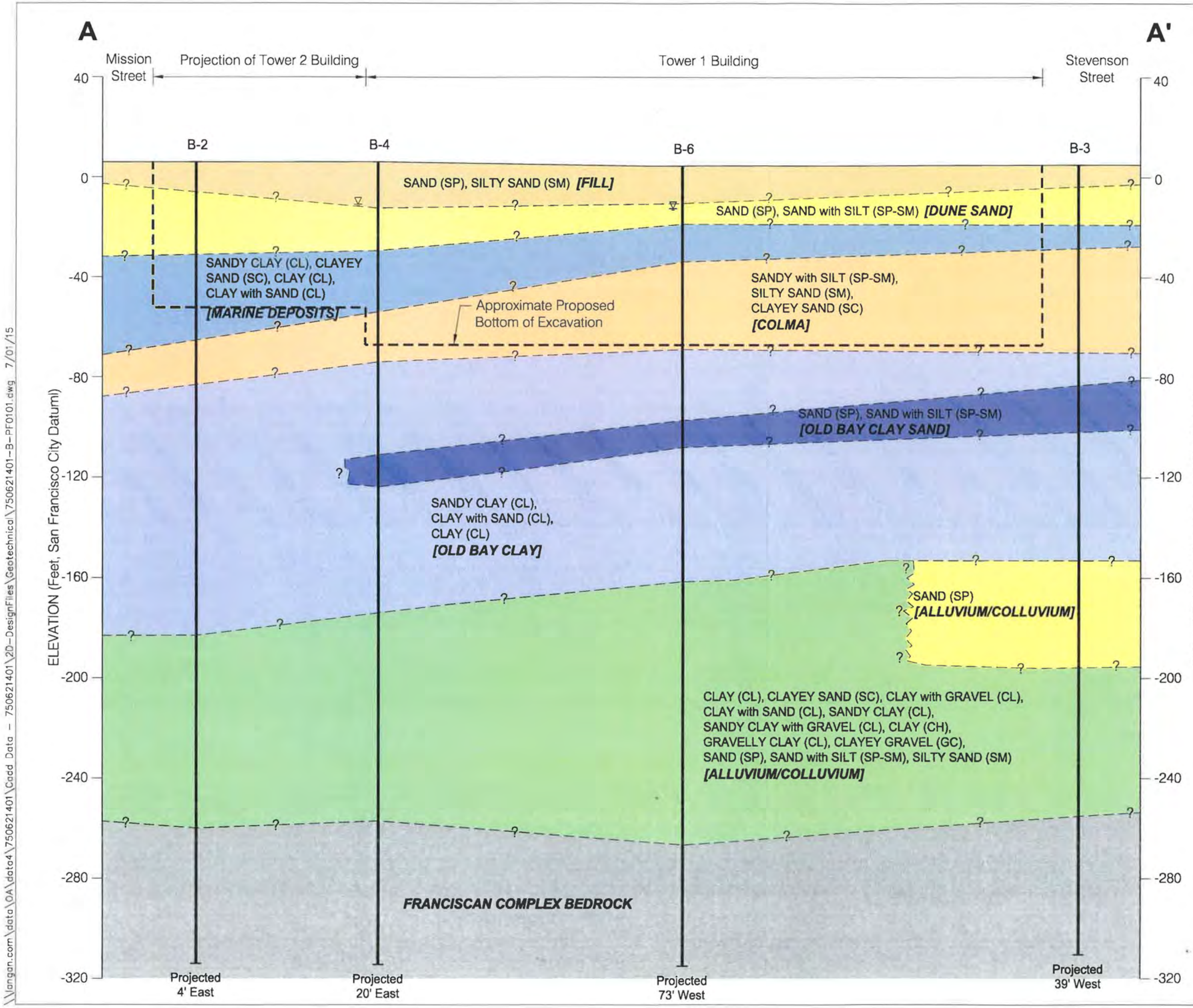
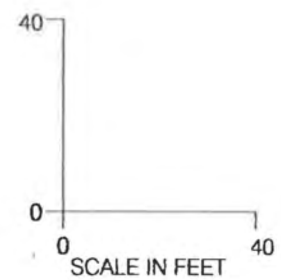


Notes:
1. The above profile represents a generalized soil and rock cross section interpreted from widely spaced borings. Soil and rock deposits may vary in type, strength, and other important properties between points of exploration.
2. This section does not extend through Tower 2, but Tower 2 has been shown for schematic purposes only.

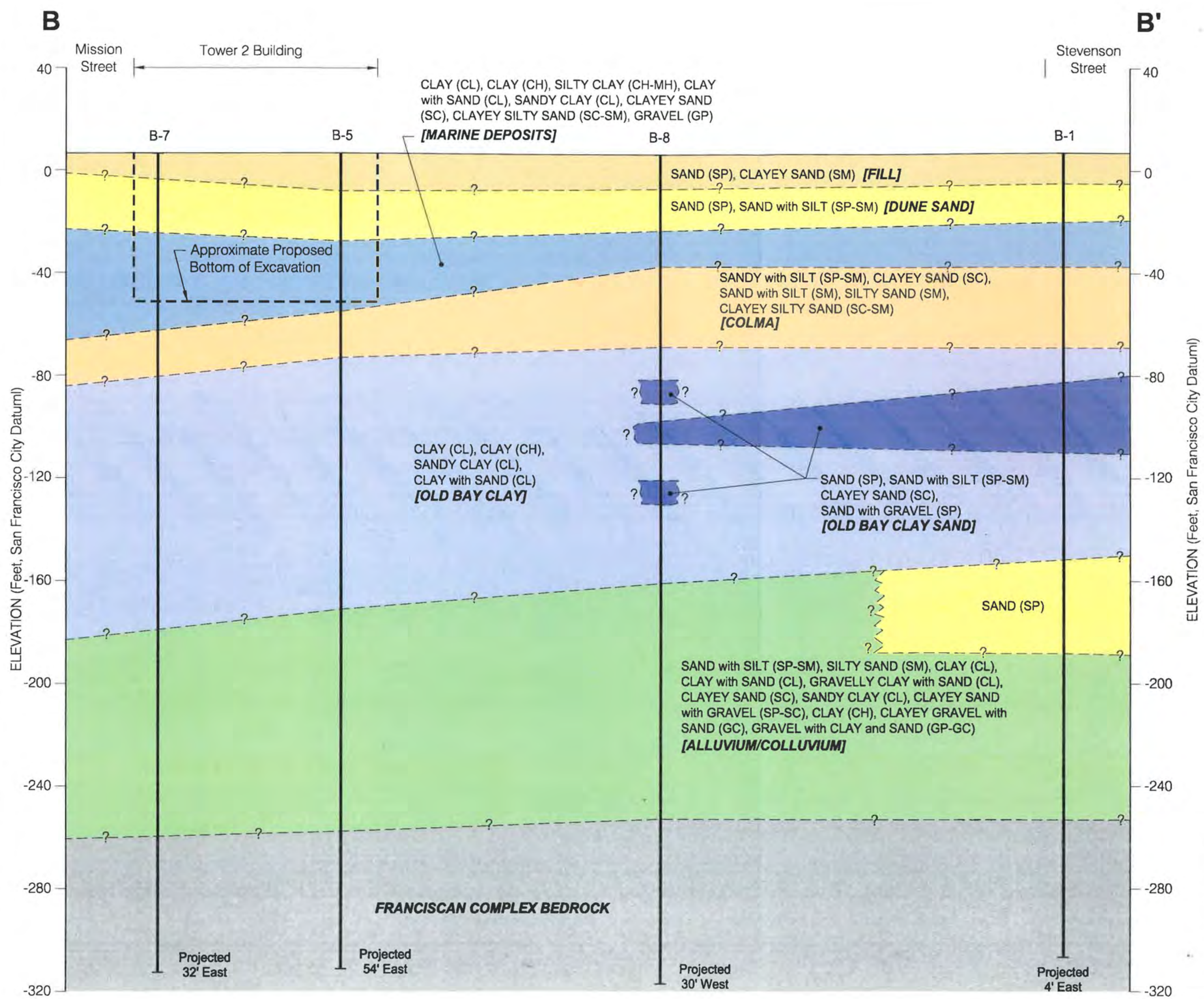


1ST AND MISSION STREETS DEVLOPMENT
San Francisco, California

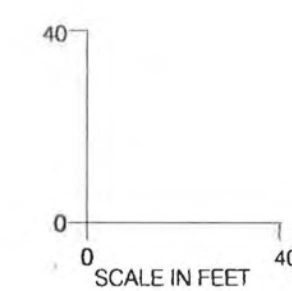
IDEALIZED SUBSURFACE PROFILE A-A'

Date 07/01/15	Project No. 750621401	Figure 4

LANGAN TREADWELL ROLLO



Notes:
1. The above profile represents a generalized soil and rock cross section interpreted from widely spaced borings. Soil and rock deposits may vary in type, strength, and other important properties between points of exploration.



1ST AND MISSION STREETS DEVLOPMENT
San Francisco, California

IDEALIZED SUBSURFACE PROFILE B-B'

Date 07/01/15	Project No. 750621401	Figure 5

LANGAN TREADWELL ROLLO

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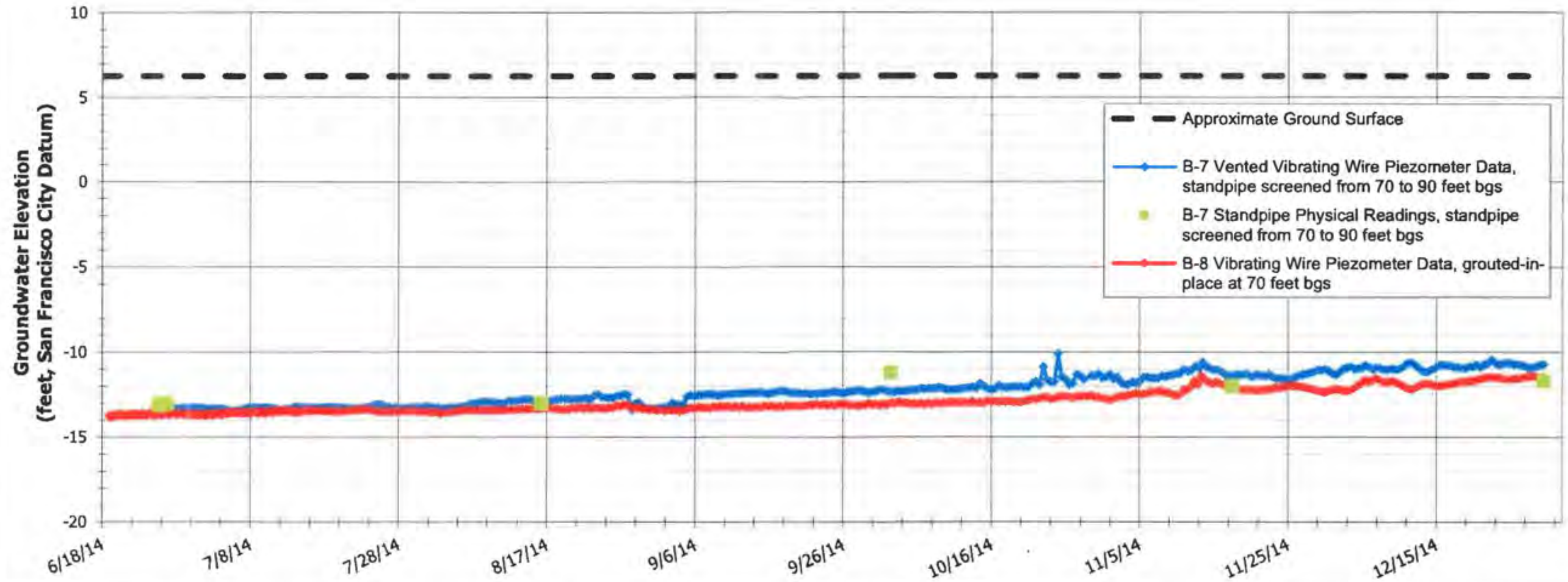
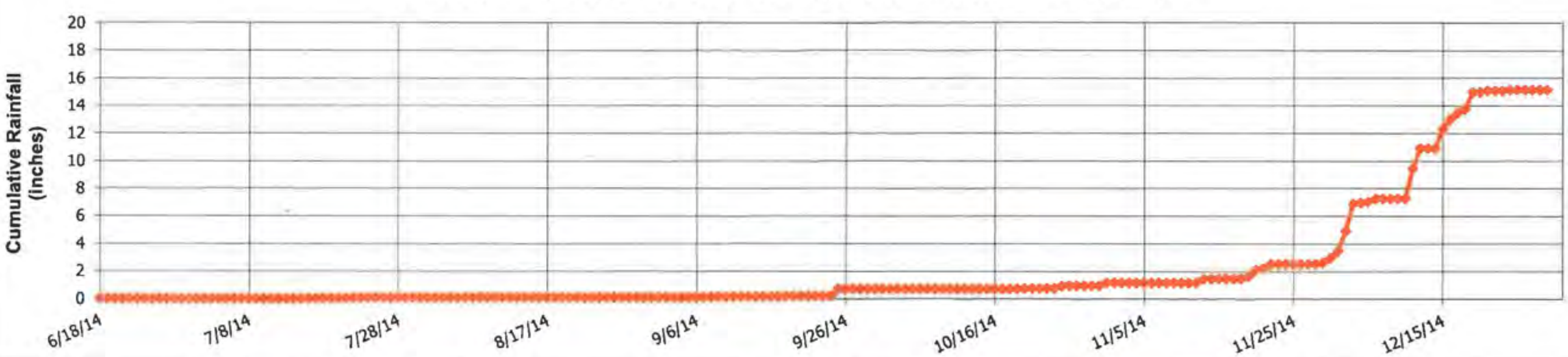


Cumulative Rainfall at San Francisco Recording Station since Piezometer Installation



1ST AND MISSION STREETS DEVELOPMENT	**GROUNDWATER ELEVATION & RAINFALL DATA**		
San Francisco, California			
LANGAN TREADWELL ROLLO	Date: 07/01/2015	Project No.750621401	Figure 6

Base map: Geologic Map of the San Francisco North Quadrangle, San Francisco and Marin Counties, California, mapped in 1948-61 by J. Schlocker.

EXPLANATION

Qaf	**Artificial fill**
Qu	**Surficial deposits, undivided**
Ql	**Landslide deposits**
Qc	**Colma Formation**
Qsr	**Slope debris and ravine fill**
KJs	**Sandstone and Shale, undifferentiated**
KJss	**Sandstone**
KJsh	**Shale and thin bedded Sandstone**

Geologic contact dashed where approximate and dotted where concealed

Fold axis

anticline

0 2000 Feet

Approximate scale

1ST AND MISSION STREETS DEVELOPMENT
San Francisco, California

LANGAN TREADWELL ROLLO

REGIONAL GEOLOGIC MAP

Date 07/01/15	Project No. 750621401	Figure 7

Notes:

1. Quaternary fault data displayed are based on a generalized version of USGS Quaternary Fault and fold database, 2010. For cartographic purposes only.
2. The Earthquake Epicenter (Magnitude) data is provided by the U.S Geological Survey (USGS) and is current through 08/26/2014.
3. Basemap hillshade and County boundaries provided by USGS and California Department of Transportation.
4. Map displayed in California State Coordinate System, California (Teale) Albers, North American Datum of 1983 (NAD83), Meters.

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Miles

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1ST AND MISSION STREETS DEVELOPMENT
San Francisco, California

MAP OF MAJOR FAULTS AND EARTHQUAKE EPICENTERS IN THE SAN FRANCISCO BAY AREA

LANGAN TREADWELL ROLLO

Date 07/01/15 | Project No. 731649501 | Figure 8

I **Not felt by people, except under especially favorable circumstances. However, dizziness or nausea may be experienced.**
Sometimes birds and animals are uneasy or disturbed. Trees, structures, liquids, bodies of water may sway gently, and doors may swing very slowly.

II **Felt indoors by a few people, especially on upper floors of multi-story buildings, and by sensitive or nervous persons.**
As in Grade I, birds and animals are disturbed, and trees, structures, liquids and bodies of water may sway. Hanging objects swing, especially if they are delicately suspended.

III **Felt indoors by several people, usually as a rapid vibration that may not be recognized as an earthquake at first. Vibration is similar to that of a light, or lightly loaded trucks, or heavy trucks some distance away. Duration may be estimated in some cases.**
Movements may be appreciable on upper levels of tall structures. Standing motor cars may rock slightly.

IV **Felt indoors by many, outdoors by a few. Awakens a few individuals, particularly light sleepers, but frightens no one except those apprehensive from previous experience. Vibration like that due to passing of heavy, or heavily loaded trucks. Sensation like a heavy body striking building, or the falling of heavy objects inside.**
Dishes, windows and doors rattle; glassware and crockery clink and clash. Walls and house frames creak, especially if intensity is in the upper range of this grade. Hanging objects often swing. Liquids in open vessels are disturbed slightly. Stationary automobiles rock noticeably.

V **Felt indoors by practically everyone, outdoors by most people. Direction can often be estimated by those outdoors. Awakens many, or most sleepers. Frightens a few people, with slight excitement; some persons run outdoors.**
Buildings tremble throughout. Dishes and glassware break to some extent. Windows crack in some cases, but not generally. Vases and small or unstable objects overturn in many instances, and a few fall. Hanging objects and doors swing generally or considerably. Pictures knock against walls, or swing out of place. Doors and shutters open or close abruptly. Pendulum clocks stop, or run fast or slow. Small objects move, and furnishings may shift to a slight extent. Small amounts of liquids spill from well-filled open containers. Trees and bushes shake slightly.

VI **Felt by everyone, indoors and outdoors. Awakens all sleepers. Frightens many people; general excitement, and some persons run outdoors.**
Persons move unsteadily. Trees and bushes shake slightly to moderately. Liquids are set in strong motion. Small bells in churches and schools ring. Poorly built buildings may be damaged. Plaster falls in small amounts. Other plaster cracks somewhat. Many dishes and glasses, and a few windows break. Knickknacks, books and pictures fall. Furniture overturns in many instances. Heavy furnishings move.

VII **Frightens everyone. General alarm, and everyone runs outdoors.**
People find it difficult to stand. Persons driving cars notice shaking. Trees and bushes shake moderately to strongly. Waves form on ponds, lakes and streams. Water is muddied. Gravel or sand stream banks cave in. Large church bells ring. Suspended objects quiver. Damage is negligible in buildings of good design and construction; slight to moderate in well-built ordinary buildings; considerable in poorly built or badly designed buildings, adobe houses, old walls (especially where laid up without mortar), spires, etc. Plaster and some stucco fall. Many windows and some furniture break. Loosened brickwork and tiles shake down. Weak chimneys break at the roofline. Cornices fall from towers and high buildings. Bricks and stones are dislodged. Heavy furniture overturns. Concrete irrigation ditches are considerably damaged.

VIII **General fright, and alarm approaches panic.**
Persons driving cars are disturbed. Trees shake strongly, and branches and trunks break off (especially palm trees). Sand and mud erupts in small amounts. Flow of springs and wells is temporarily and sometimes permanently changed. Dry wells renew flow. Temperatures of spring and well waters varies. Damage slight in brick structures built especially to withstand earthquakes; considerable in ordinary substantial buildings, with some partial collapse; heavy in some wooden houses, with some tumbling down. Panel walls break away in frame structures. Decayed pilings break off. Walls fall. Solid stone walls crack and break seriously. Wet grounds and steep slopes crack to some extent. Chimneys, columns, monuments and factory stacks and towers twist and fall. Very heavy furniture moves conspicuously or overturns.

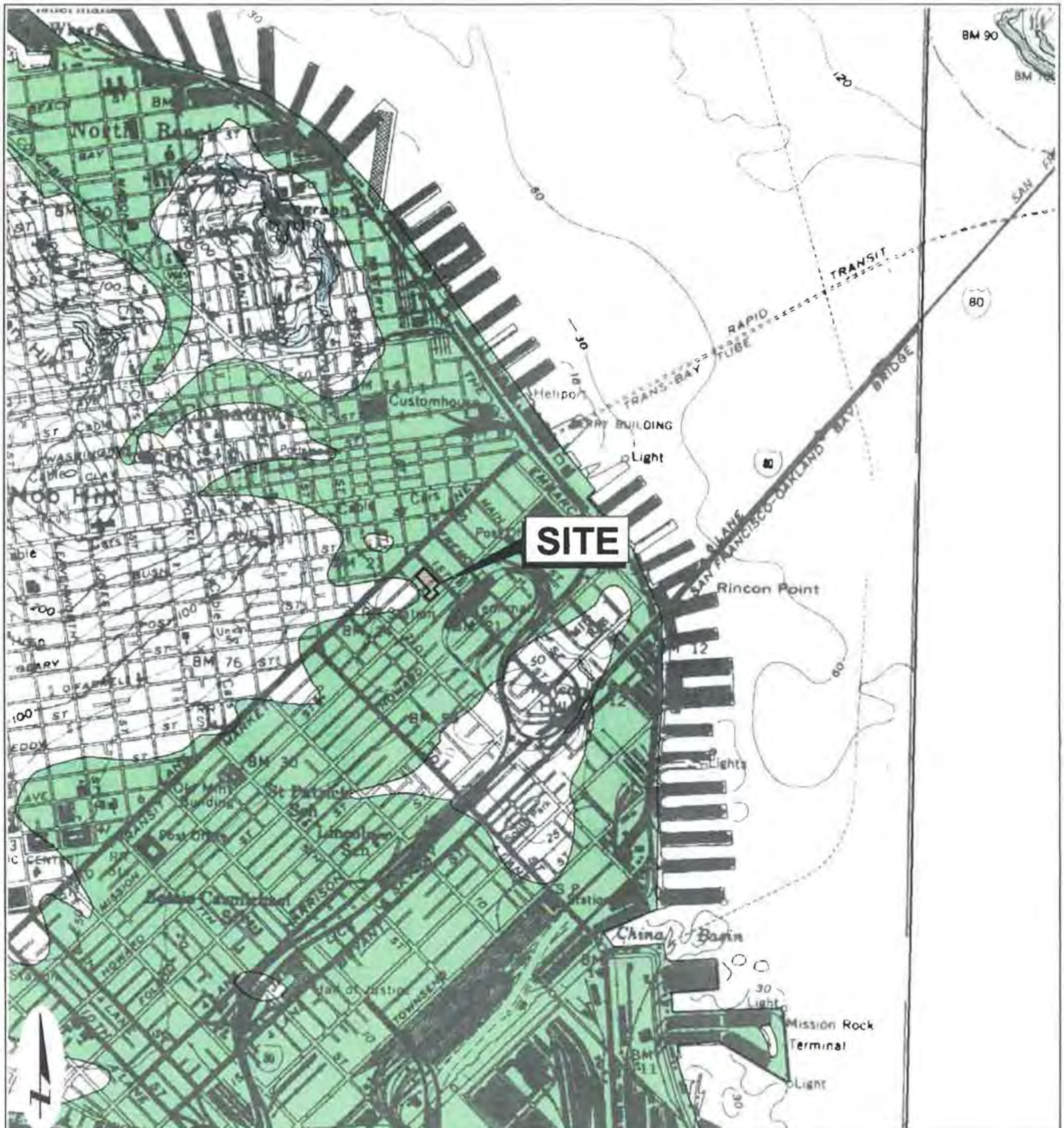
IX **Panic is general.**
Ground cracks conspicuously. Damage is considerable in masonry structures built especially to withstand earthquakes; great in other masonry buildings - some collapse in large part. Some wood frame houses built especially to withstand earthquakes are thrown out of plumb, others are shifted wholly off foundations. Reservoirs are seriously damaged and underground pipes sometimes break.

X **Panic is general.**
Ground, especially when loose and wet, cracks up to widths of several inches; fissures up to a yard in width run parallel to canal and stream banks. Landsliding is considerable from river banks and steep coasts. Sand and mud shifts horizontally on beaches and flat land. Water level changes in wells. Water is thrown on banks of canals, lakes, rivers, etc. Dams, dikes, embankments are seriously damaged. Well-built wooden structures and bridges are severely damaged, and some collapse. Dangerous cracks develop in excellent brick walls. Most masonry and frame structures, and their foundations are destroyed. Railroad rails bend slightly. Pipe lines buried in earth tear apart or are crushed endwise. Open cracks and broad wavy folds open in cement pavements and asphalt road surfaces.

XI **Panic is general.**
Disturbances in ground are many and widespread, varying with the ground material. Broad fissures, earth slumps, and land slips develop in soft, wet ground. Water charged with sand and mud is ejected in large amounts. Sea waves of significant magnitude may develop. Damage is severe to wood frame structures, especially near shock centers, great to dams, dikes and embankments, even at long distances. Few if any masonry structures remain standing. Supporting piers or pillars of large, well-built bridges are wrecked. Wooden bridges that "give" are less affected. Railroad rails bend greatly and some thrust endwise. Pipe lines buried in earth are put completely out of service.

XII **Panic is general.**
Damage is total, and practically all works of construction are damaged greatly or destroyed. Disturbances in the ground are great and varied, and numerous shearing cracks develop. Landslides, rock falls, and slumps in river banks are numerous and extensive. Large rock masses are wrenched loose and torn off. Fault slips develop in firm rock, and horizontal and vertical offset displacements are notable. Water channels, both surface and underground, are disturbed and modified greatly. Lakes are dammed, new waterfalls are produced, rivers are deflected, etc. Surface waves are seen on ground surfaces. Lines of sight and level are distorted. Objects are thrown upward into the air.

1ST AND MISSION STREETS DEVELOPENT San Francisco, California	**MODIFIED MERCALLI INTENSITY SCALE**
LANGAN TREADWELL ROLLO	Date 07/01/15 | Project No. 750621401 | Figure 9



EXPLANATION

Liquefaction; Areas where historic occurence of liquefaction, or local topographic, geological, geotechnical, and subsurface water conditions indicate a potential for permanent ground displacements.

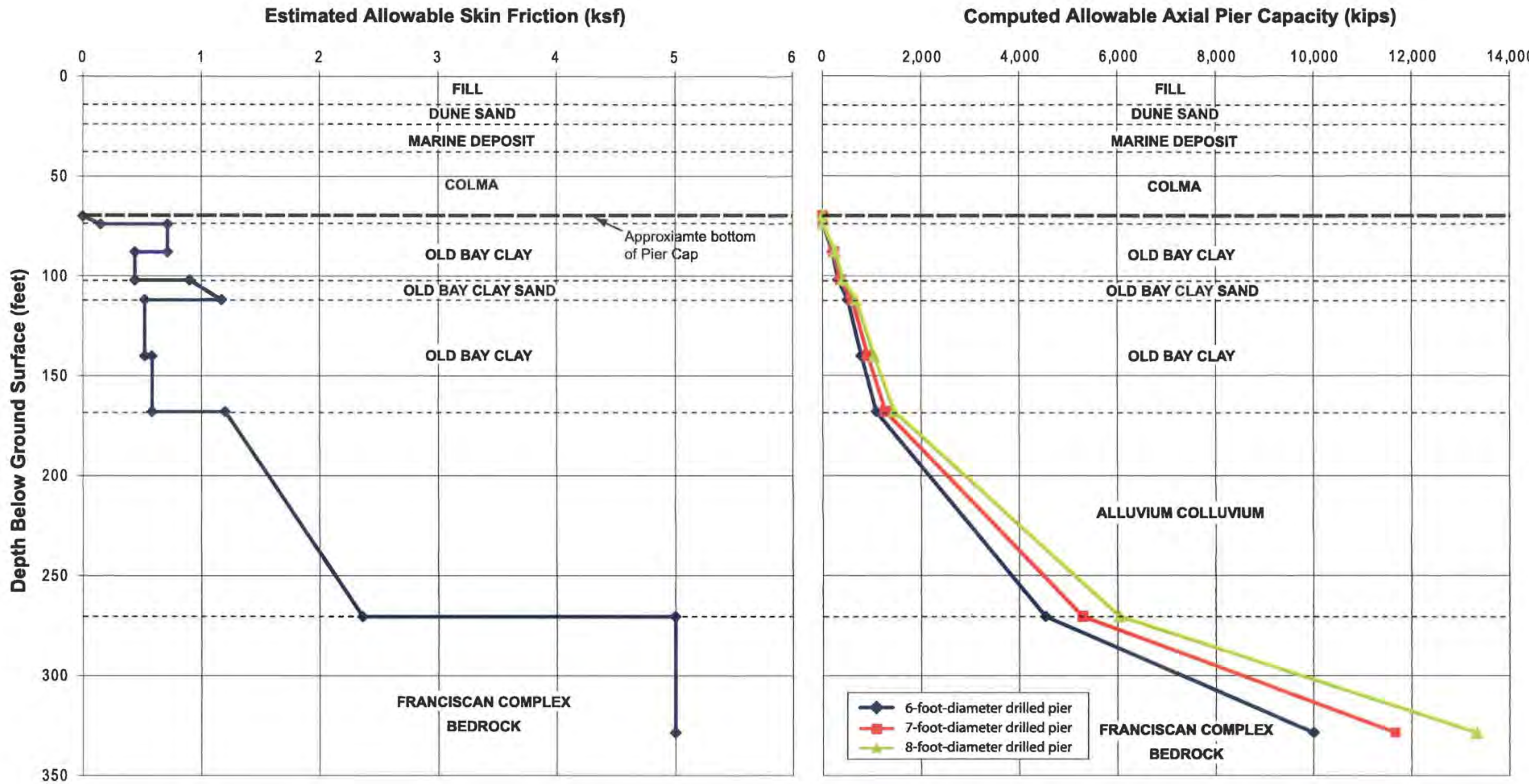
Earthquake-Induced Landslides; Areas where previous occurence of landslide movement, or local topographic, geological, geotechnical, and subsurface water conditions indicate a potential for permanent ground displacements.

0 2,000 4,000 Feet

Approximate scale

Reference:
State of California "Seismic Hazard Zones"
City and County of San Francisco
Released on November 17, 2001

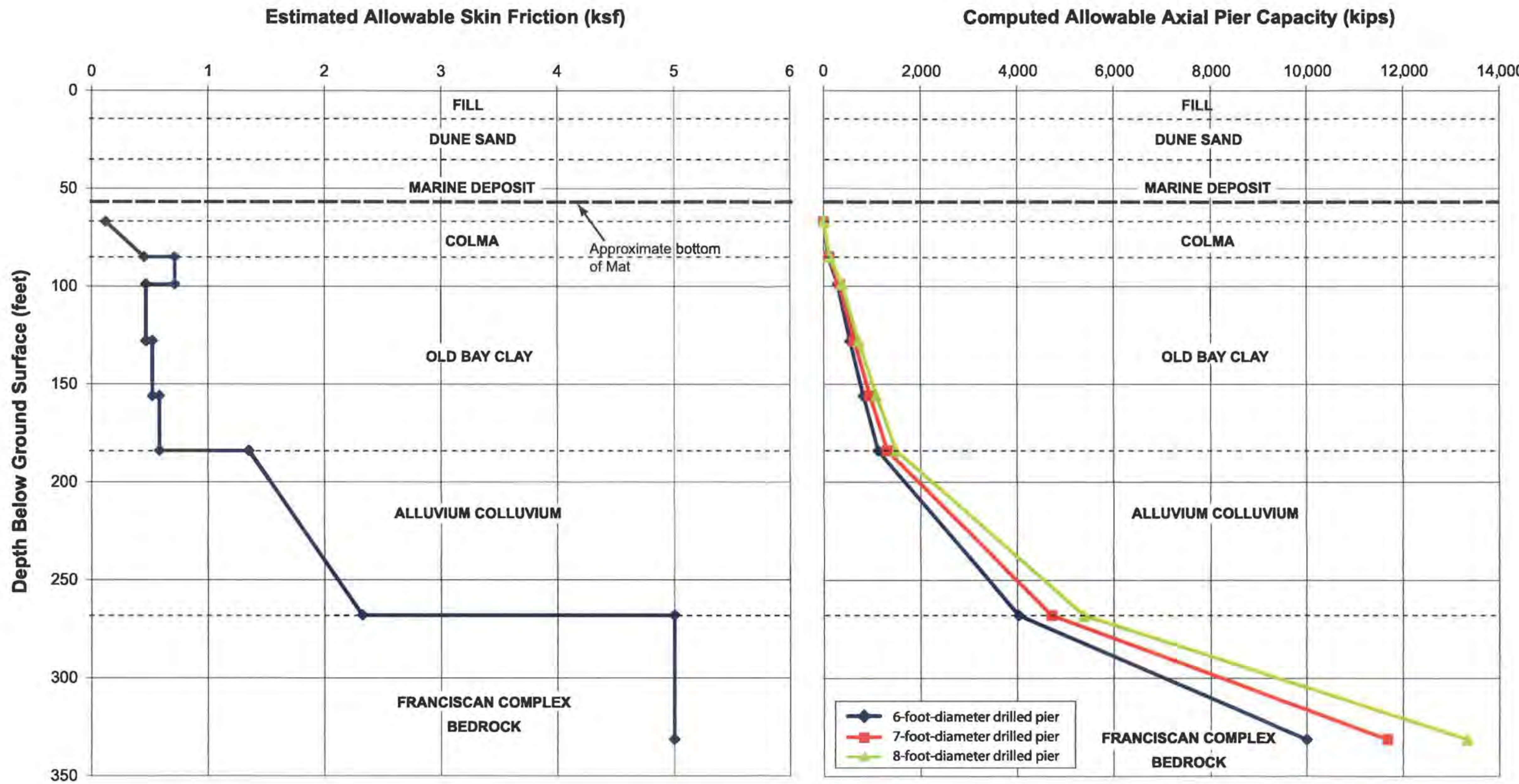
1ST AND MISSION STREETS DEVELOPMENT San Francisco, California	**REGIONAL SEISMIC HAZARD ZONES MAP**
LANGAN TREADWELL ROLLO	Date 07/01/15 | Project No. 750621401 | Figure 10



Notes:

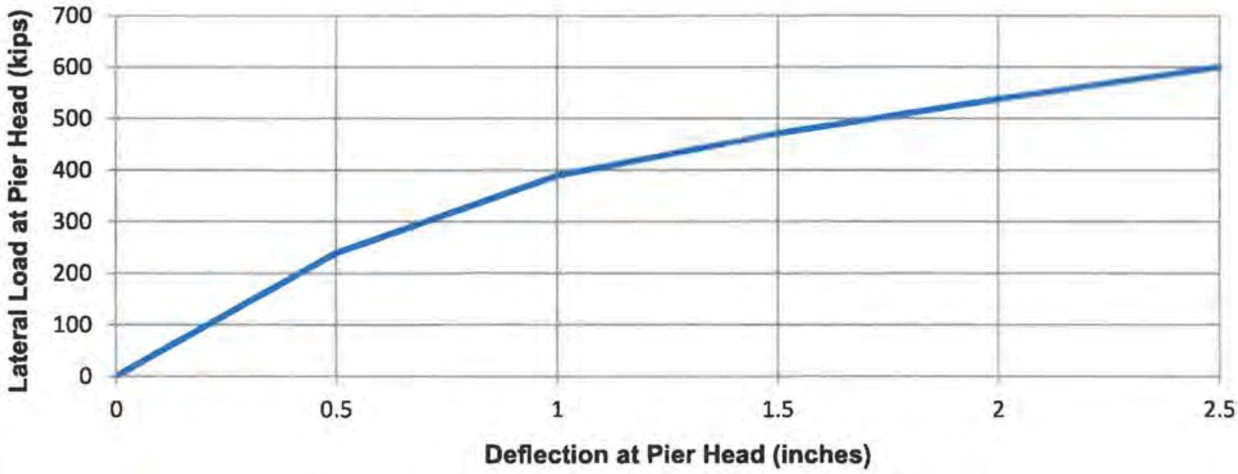
1. Allowable skin frictions assume Factor of Safety = 2.0.
2. Allowable capacity not shown for barrettes because of unknown geometry.

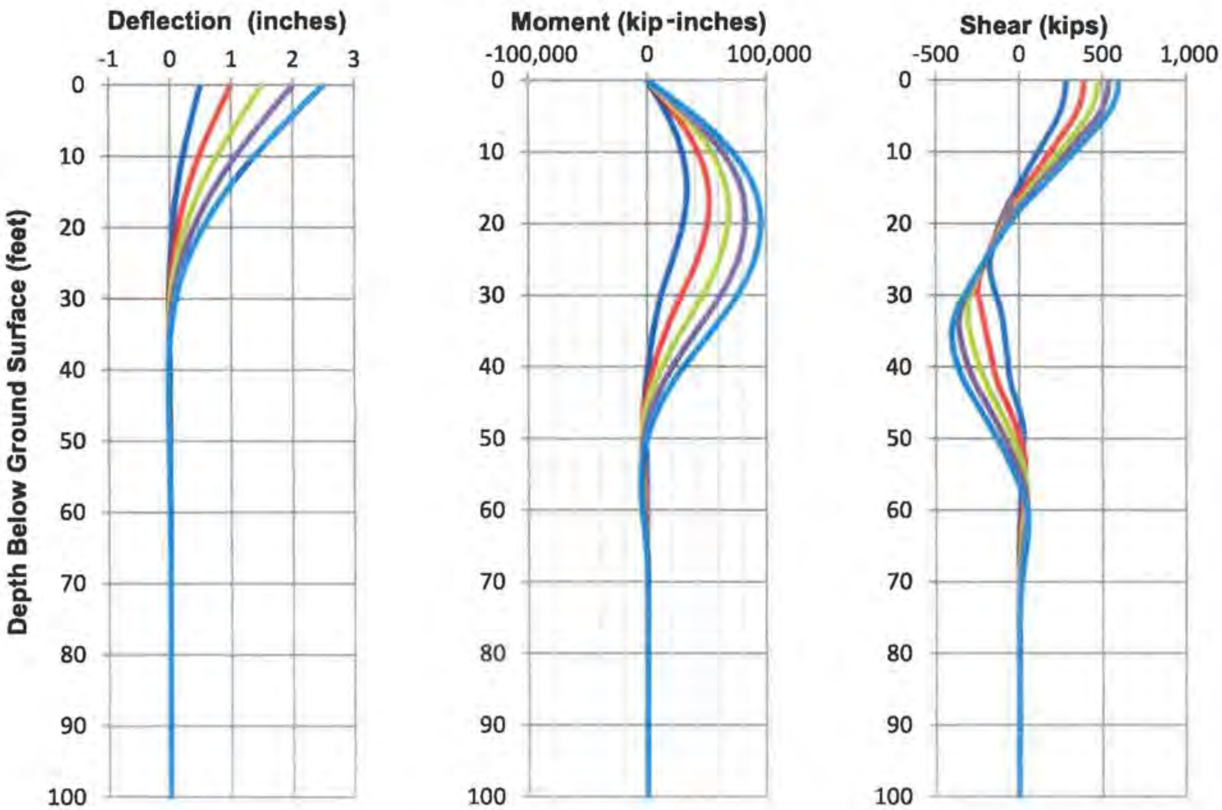
1ST AND MISSION STREETS DEVELOPMENT San Francisco, California		
ESTIMATED AXIAL PIER CAPACITY **TOWER 1**		
Date 07/01/15	Project No. 750241401	Figure 11
LANGAN TREADWELL ROLLO		



Notes:
1. Allowable skin frictions assume Factor of Safety = 2.0.
2. Allowable capacity not shown for barrettes because of unknown geometry.

1ST AND MISSION STREETS DEVELOPMENT San Francisco, California		
ESTIMATED AXIAL PIER CAPACITY TOWER 2		
Date 07/01/15	Project No. 750241401	Figure 12
LANGAN TREADWELL ROLLO		





Notes:

1. Used drilled pier cracked moment of inertia (50% of theoretical value) for concrete section.
2. Used modulus of elasticity of 5.1 x 10^6 psi for concrete.
3. Steel reinforcement not considered in analysis.

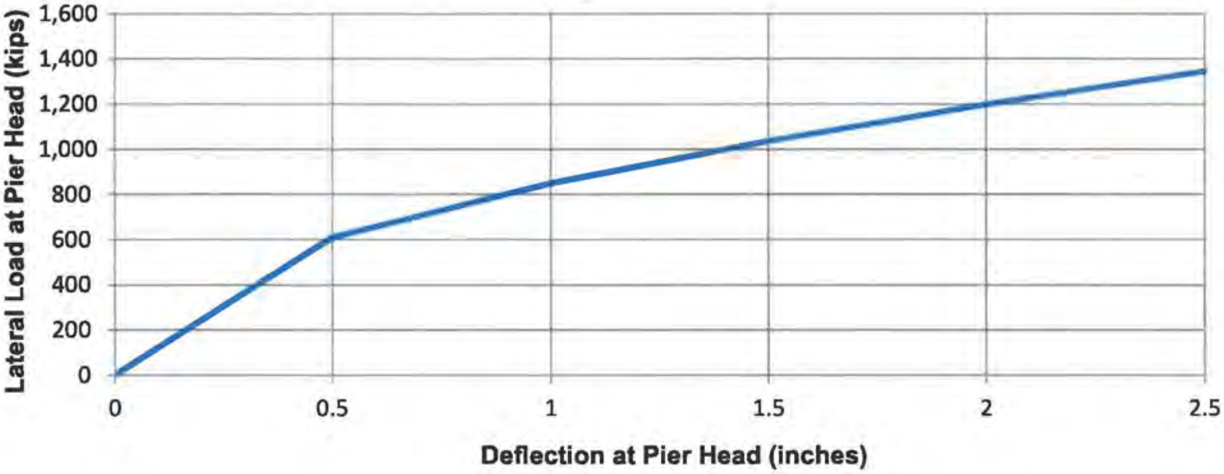
	Pier Head Deflection (inches)	Lateral Load (kips)	Maximum Bending Moment (kip-inch)	Depth to Maximum Bending Moment (feet)
(dark blue)	0.5	280	33,600	16
(red)	1.0	390	52,800	18
(yellow-green)	1.5	470	69,000	18
(purple)	2.0	540	82,800	20
(light blue)	2.5	600	95,600	20

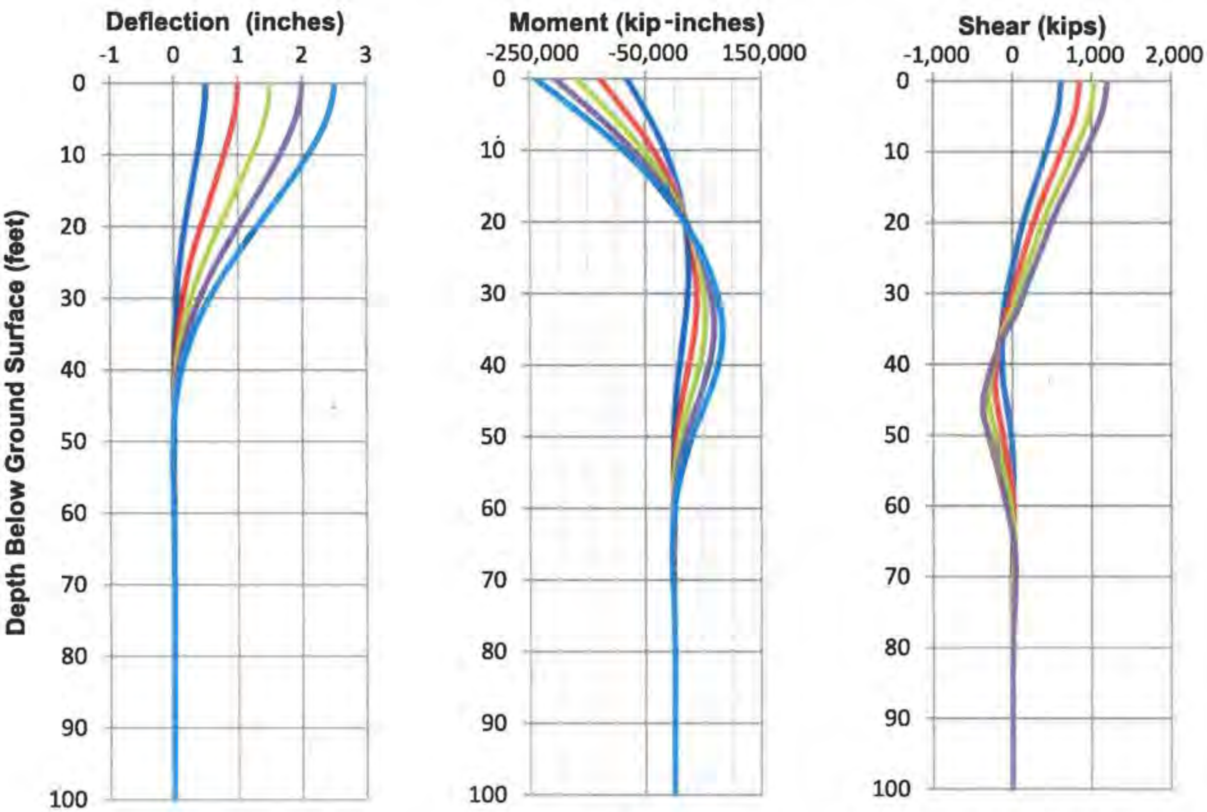
1ST AND MISSION STREET DEVELOPMENT
San Francisco, California

LANGAN TREADWELL ROLLO

LATERAL PROFILES FOR 6-FOOT-DIAMETER DRILLED PIER
FREE HEAD CONDITION
(BELOW A THREE- OR FOUR-LEVEL BASEMENT)

Date 07/01/15	Project No. 750621401	Figure 13





Pier Head Deflection (inches)	Lateral Load (kips)	Maximum Bending Moment (kip-inch)	Depth to Maximum Bending Moment (feet)
0.5	610	81,800	0
1.0	850	128,300	0
1.5	1,000	168,700	0
2.0	1,200	205,900	0
2.5	1,300	240,600	0

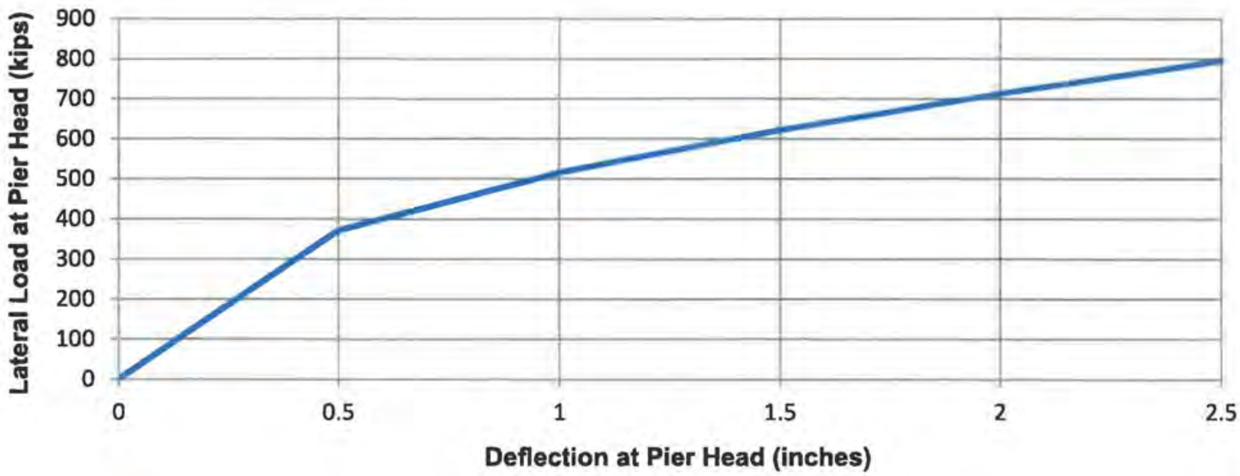
Notes:
1. Used drilled pier cracked moment of inertia (50% of theoretical value) for concrete section.
2. Used modulus of elasticity of 5.1×10^6 psi for concrete.
3. Steel reinforcement not considered in analysis.

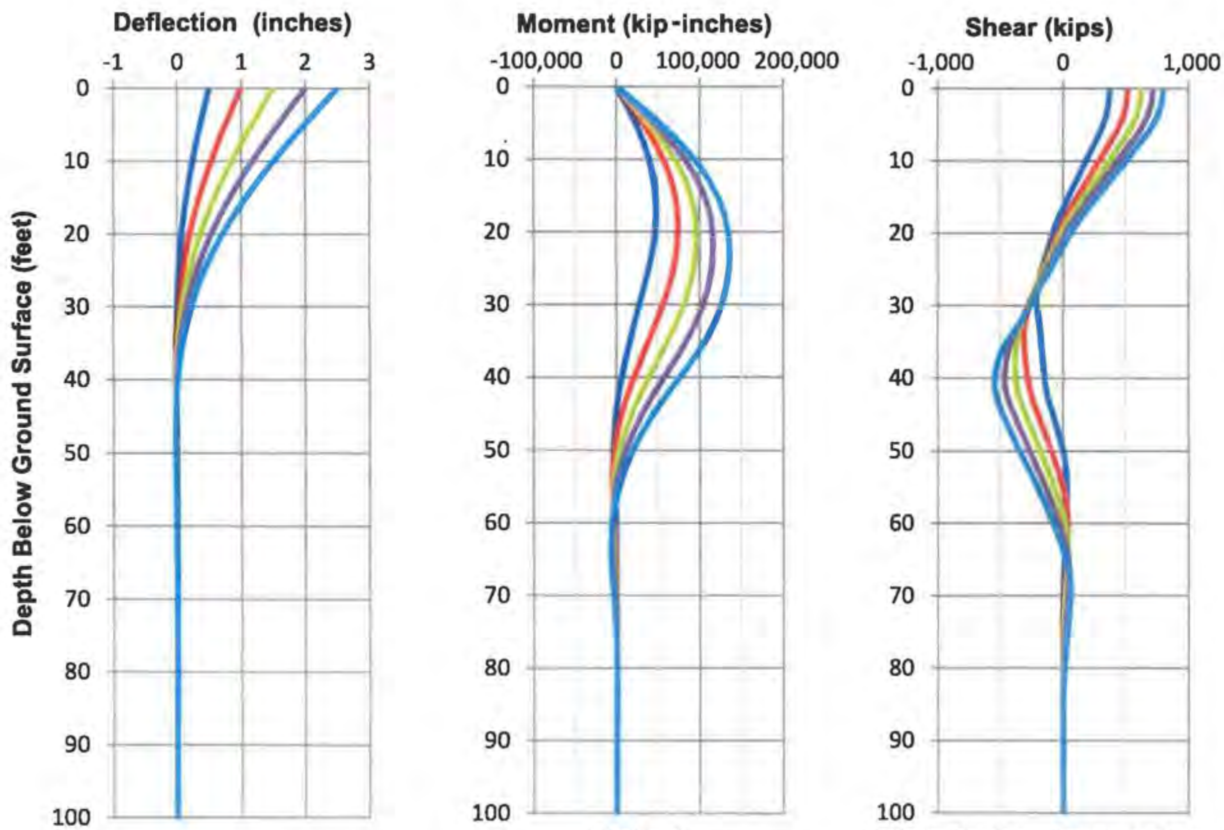
1ST AND MISSION STREET DEVELOPMENT
San Francisco, California

LANGAN TREADWELL ROLLO

LATERAL PROFILES FOR 6-FOOT-DIAMETER DRILLED PIER FIXED HEAD CONDITION (BELOW A THREE- OR FOUR-LEVEL BASEMENT)

Date 07/01/15	Project No. 750621401	Figure 14





Pier Head Deflection (inches)	Lateral Load (kips)	Maximum Bending Moment (kip-inch)	Depth to Maximum Bending Moment (feet)
0.5	370	47,800	18
1.0	520	74,500	20
1.5	620	96,700	22
2.0	710	116,400	22
2.5	800	136,000	24

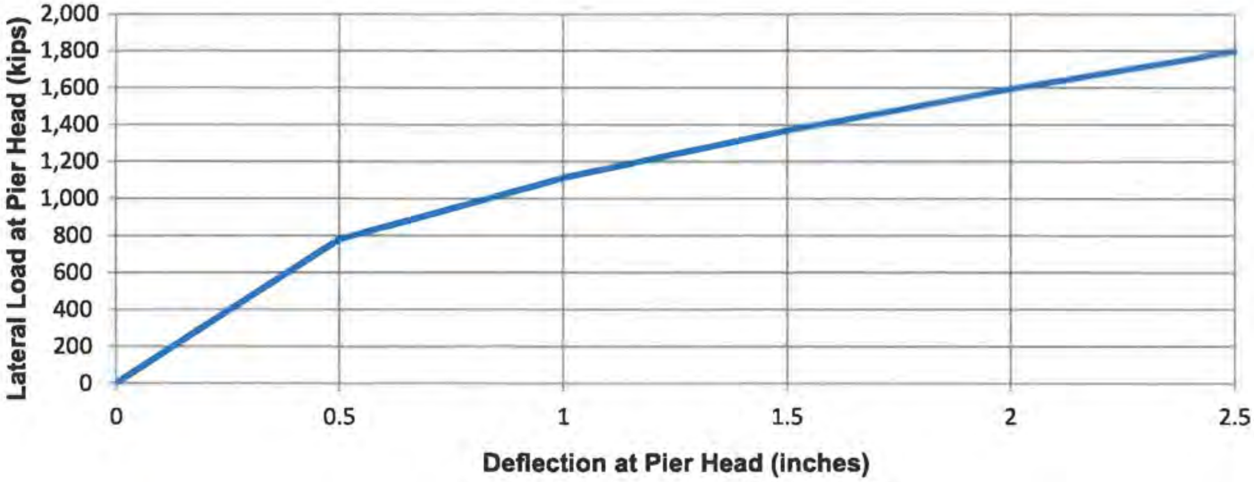
Notes:
1. Used drilled pier cracked moment of inertia (50% of theoretical value) for concrete section.
2. Used modulus of elasticity of 5.1 x 10^6 psi for concrete.
3. Steel reinforcement not considered in analysis.

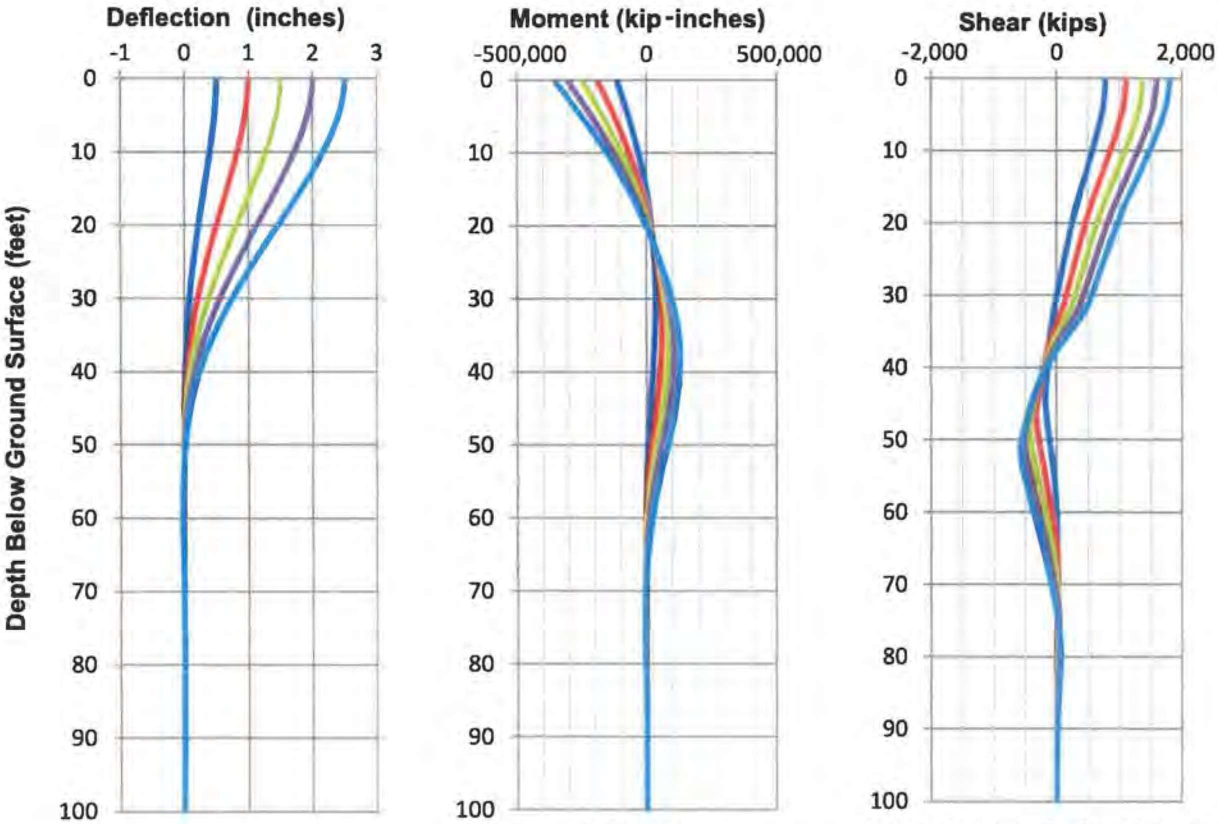
1ST AND MISSION STREET DEVELOPMENT
San Francisco, California

LANGAN TREADWELL ROLLO

LATERAL PROFILES FOR 7-FOOT-DIAMETER DRILLED PIER FREE HEAD CONDITION (BELOW A THREE- OR FOUR-LEVEL BASEMENT)

Date 07/01/15 | Project No. 750621401 | Figure 15





Pier Head Deflection (inches)	Lateral Load (kips)	Maximum Bending Moment (kip-inch)	Depth to Maximum Bending Moment (feet)
0.5	780	116,700	0
1.0	1,110	188,600	0
1.5	1,370	249,900	0
2.0	1,590	306,300	0
2.5	1,800	359,400	0

Notes:

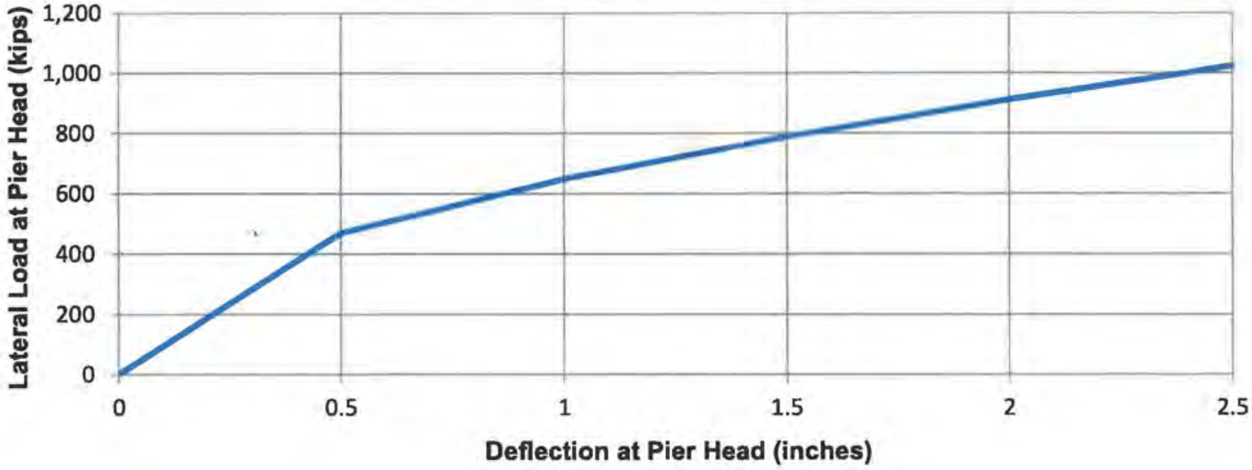
1. Used drilled pier cracked moment of inertia (50% of theoretical value) for concrete section.
2. Used modulus of elasticity of 5.1×10^6 psi for concrete.
3. Steel reinforcement not considered in analysis.

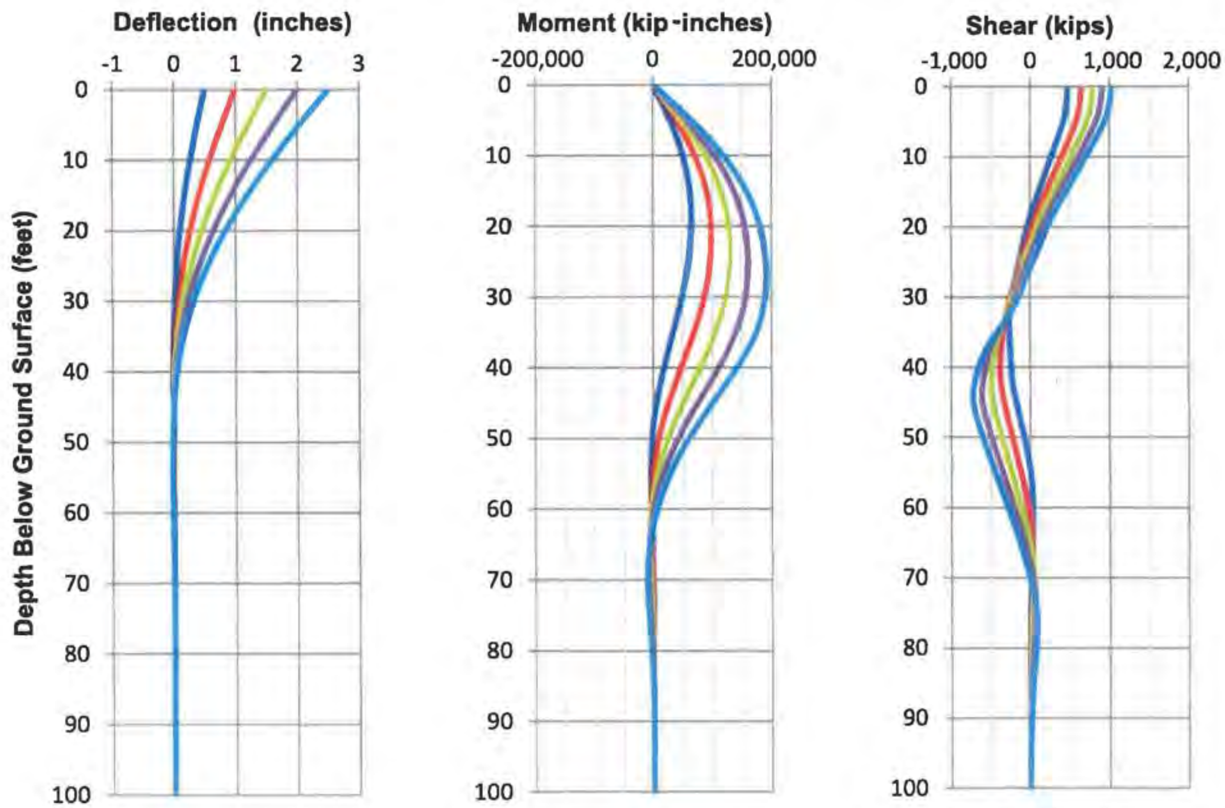
1ST AND MISSION STREET DEVELOPMENT
San Francisco, California

LANGAN TREADWELL ROLLO

LATERAL PROFILES FOR 7-FOOT-DIAMETER DRILLED PIER FIXED HEAD CONDITION (BELOW A THREE- OR FOUR-LEVEL BASEMENT)

Date 07/01/15	Project No. 750621401	Figure 16





	Pier Head Deflection (inches)	Lateral Load (kips)	Maximum Bending Moment (kip-inch)	Depth to Maximum Bending Moment (feet)
	0.5	470	64,800	20
	1.0	650	99,500	22
	1.5	790	130,700	24
	2.0	910	161,300	26
	2.5	1,020	191,100	27

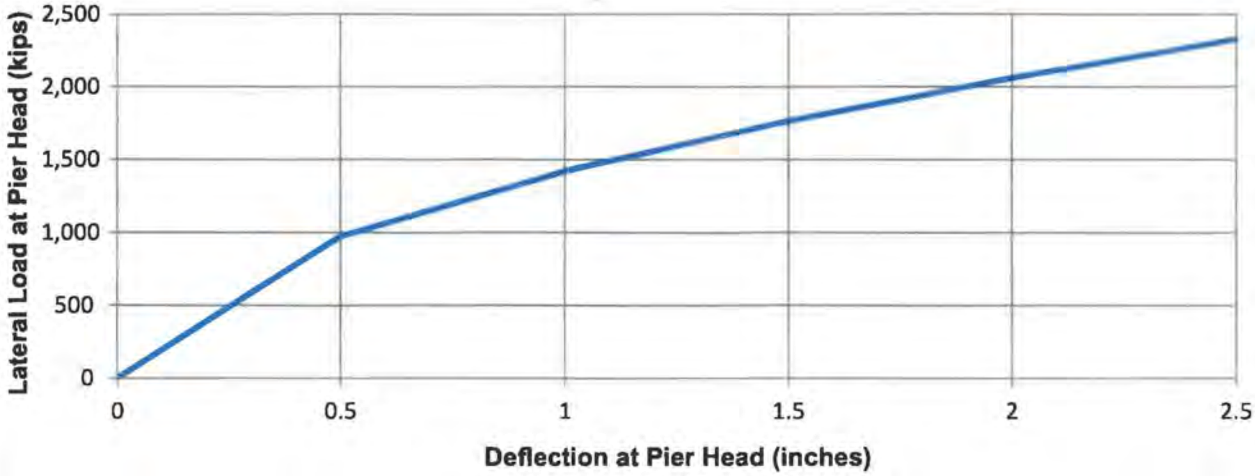
Notes:
1. Used drilled pier cracked moment of inertia (50% of theoretical value) for concrete section.
2. Used modulus of elasticity of 5.1×10^6 psi for concrete.
3. Steel reinforcement not considered in analysis.

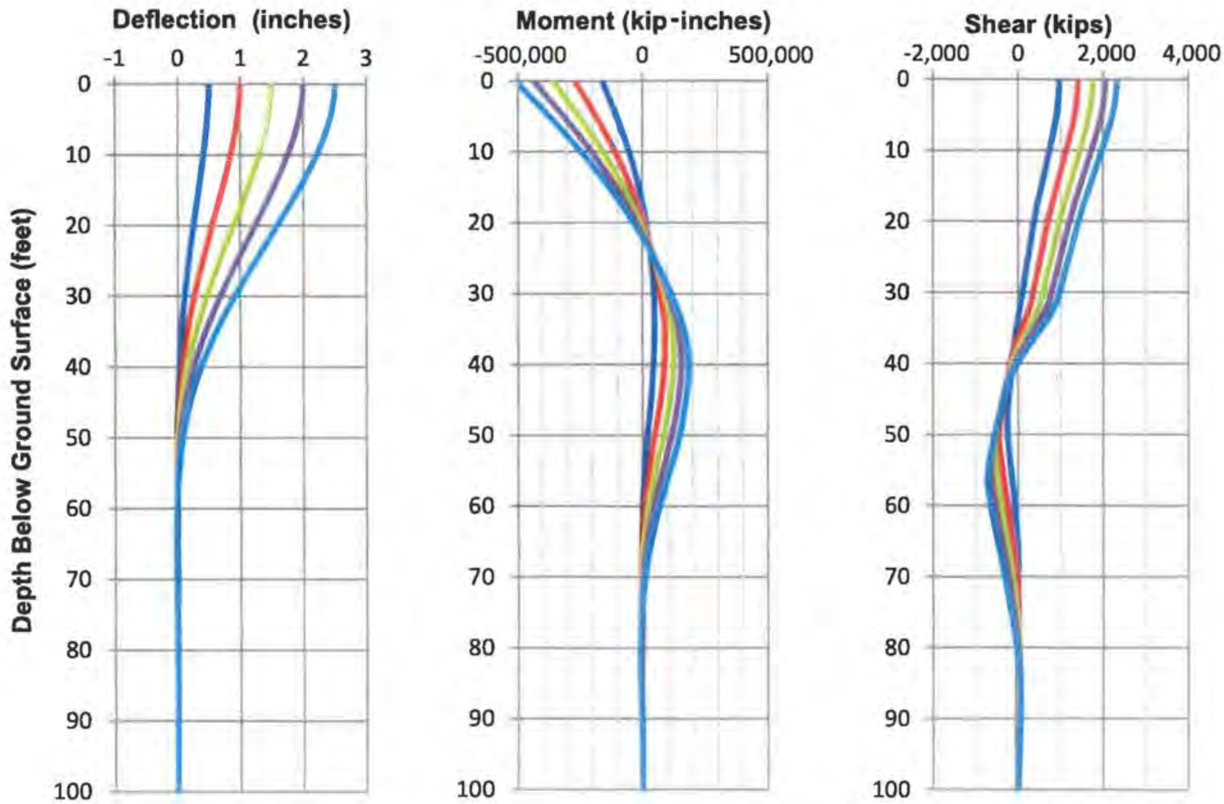
1ST AND MISSION STREET DEVELOPMENT
San Francisco, California

LANGAN TREADWELL ROLLO

LATERAL PROFILES FOR 8-FOOT-DIAMETER DRILLED PIER
FREE HEAD CONDITION
(BELOW A THREE- OR FOUR-LEVEL BASEMENT)

Date 07/01/15	Project No. 750621401	Figure 17





	Pier Head Deflection (inches)	Lateral Load (kips)	Maximum Bending Moment (kip-inch)	Depth to Maximum Bending Moment (feet)
	0.5	970	160,900	0
	1.0	1,420	266,800	0
	1.5	1,770	356,200	0
	2.0	2,060	436,900	0
	2.5	2,330	512,200	0

Notes:

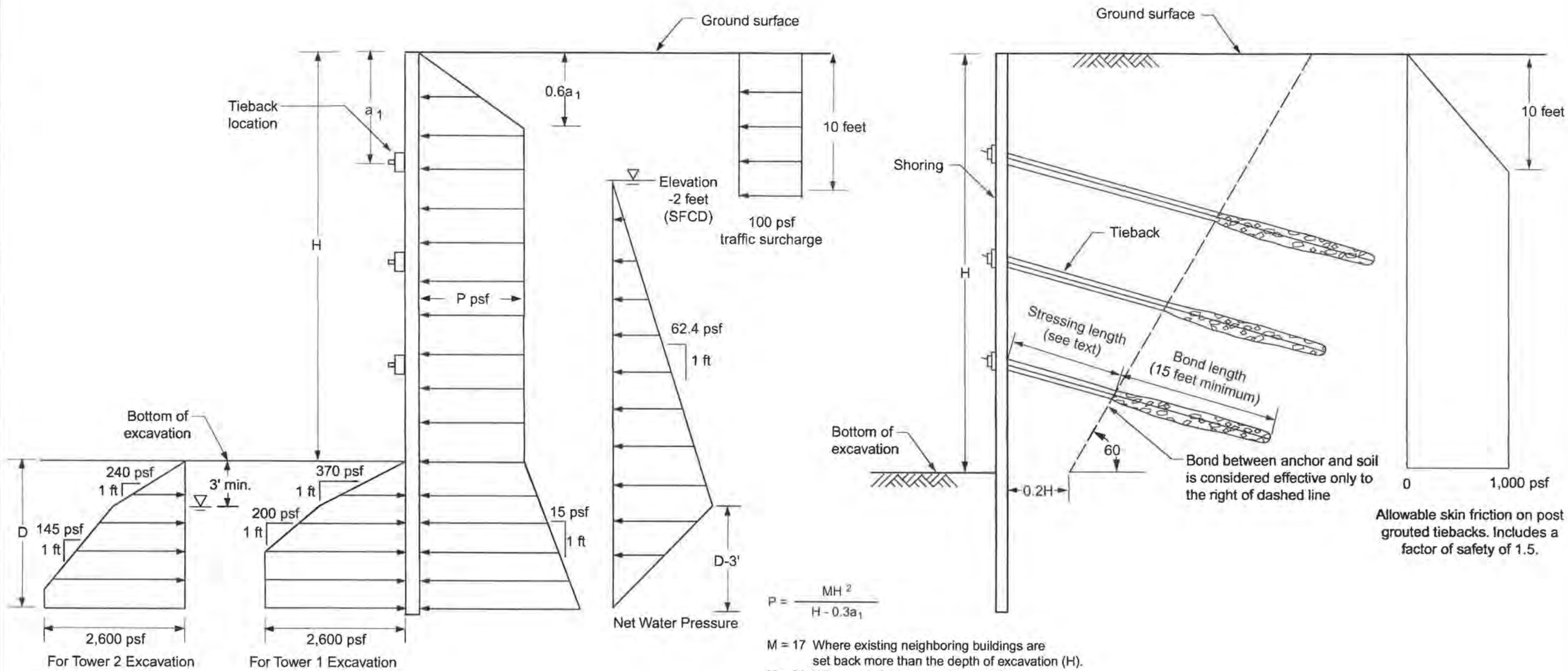
1. Used drilled pier cracked moment of inertia (50% of theoretical value) for concrete section.
2. Used modulus of elasticity of 5.1×10^6 psi for concrete.
3. Steel reinforcement not considered in analysis.

1ST AND MISSION STREET DEVELOPMENT
San Francisco, California

LANGAN TREADWELL ROLLO

LATERAL PROFILES FOR 8-FOOT-DIAMETER DRILLED PIER FIXED HEAD CONDITION (BELOW A THREE- OR FOUR-LEVEL BASEMENT)

Date 07/01/15	Project No. 750621401	Figure 18



$$P = \frac{MH^2}{H - 0.3a_1}$$

M = 17 Where existing neighboring buildings are set back more than the depth of excavation (H).

M = 21 Where existing neighboring buildings are within a distance equal to the excavation depth (H) and/or underpinned. Additional surcharge for adjacent building foundations should be added.

Not to scale

Notes:

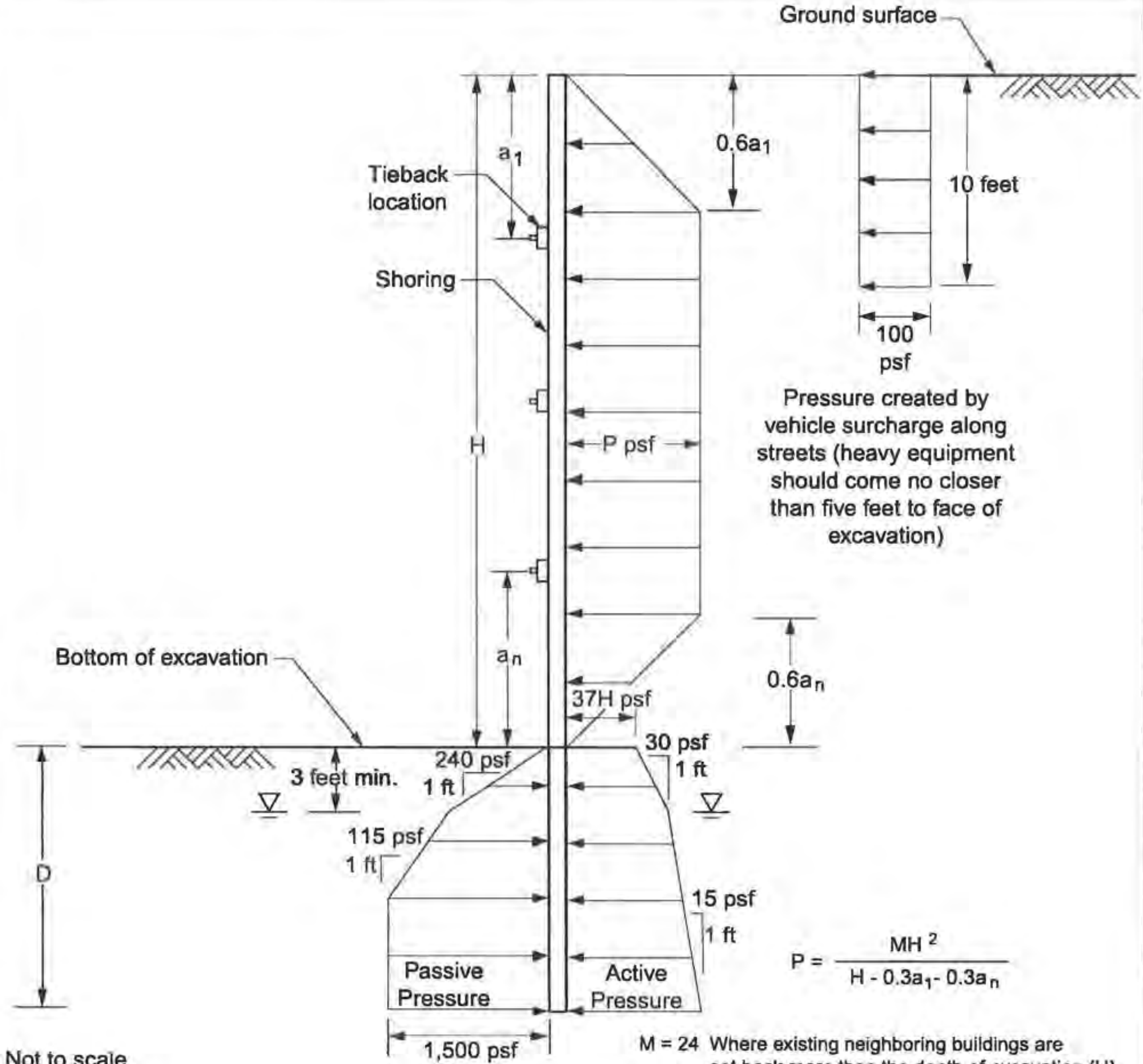
1. Passive values include a factor of safety of 1.5.
2. Pressures assume the groundwater within the site will be lowered at least 3 feet below the bottom of the excavation.
3. Shoring should be braced or tied back.
4. Surcharge pressure due to construction equipment, if any, should be added to the above shoring pressure.
5. Penetration depth (D) should be a minimum of 10 feet; actual penetration depth should extend sufficiently into the Old Bay Clay to create an effective groundwater cutoff. Penetration depth should be determined by the shoring designer
6. Pressure provided applicable for basement excavation depths (H) ranging from 55 to 75 feet below ground surface.

1ST AND MISSION STREETS DEVELOPMENT
San Francisco, California

DESIGN PARAMETERS FOR TIED-BACK OR BRACED SOIL CEMENT OR CONCRETE DIAPHRAGM WALLS TEMPORARY SHORING (FOR A THREE- OR FOUR-LEVEL BASEMENT EXCAVATION)

Date 07/01/15	Project No. 750621401	Figure 19

LANGAN TREADWELL ROLLO



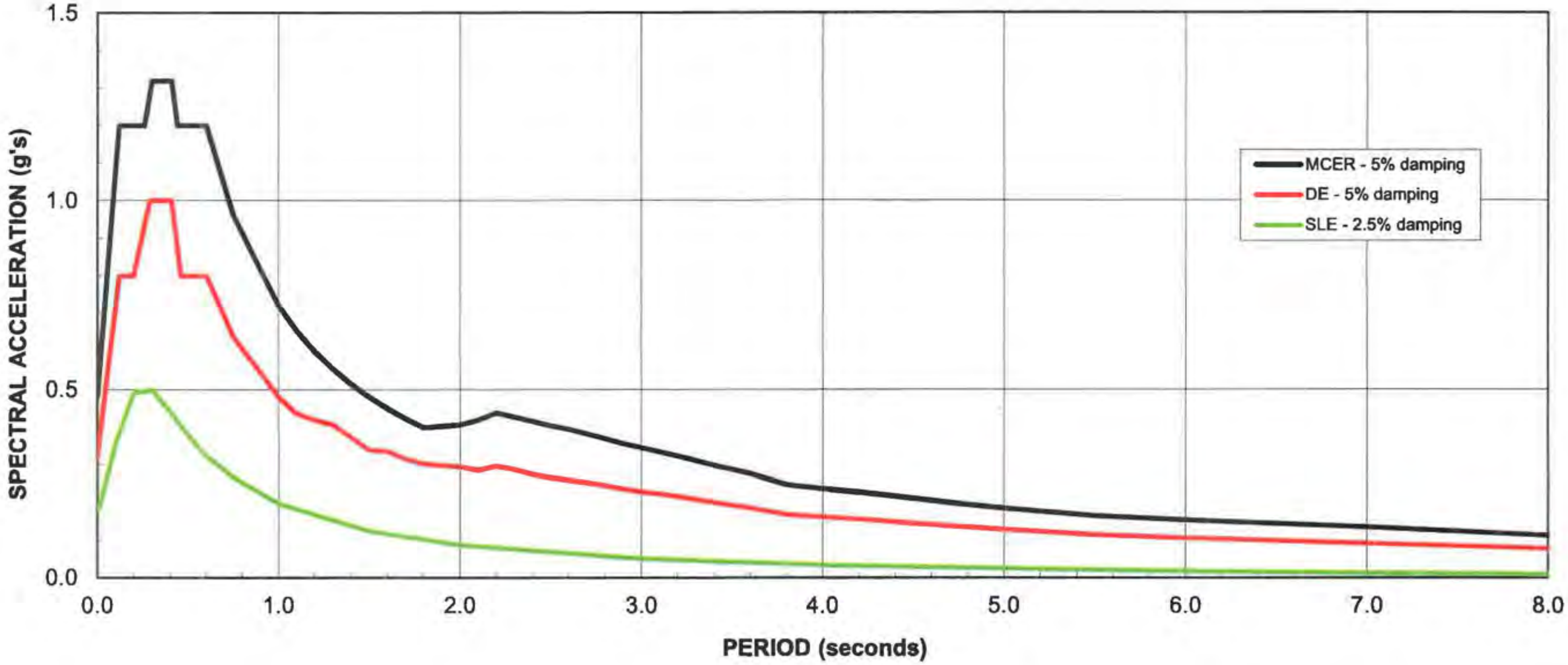
$$P = \frac{MH^2}{H - 0.3a_1 - 0.3a_n}$$

M = 24 Where existing neighboring buildings are set back more than the depth of excavation (H).

M = 29 Where existing neighboring buildings are within a distance equal to the excavation depth (H) and/or underpinned. Additional surcharge for adjacent building foundations should be added.

Notes:
1. Passive pressure includes a factor of safety of about 1.5.
2. Passive pressure should be assumed to act over the pile spacing or three pile diameters, whichever is smaller.
3. Surcharge pressure due to construction equipment, if any, should be added to the above shoring pressure.
4. Active pressure below the excavation should be assumed to act over one pile diameter.
5. Penetration depth (D) should be a minimum of 10 feet; actual penetration depth required should be determined by the shoring designer.
6. Pressures assume site and surrounding ground are successfully dewatered at least 3 feet below the bottom of the excavation.
7. Shoring should be braced or tied back. For tieback design see Figure 19.

1ST AND MISSION STREETS DEVELOPMENT San Francisco, California	**DESIGN PARAMETERS FOR TIED-BACK OR BRACED SOLDIER-PILE-AND-LAGGING TEMPORARY SHORING SYSTEM** **(for excavations less than 20 feet deep)**
LANGAN TREADWELL ROLLO	Date 07/01/15 | Project No. 750621401 | Figure 20



1ST AND MISSION STREETS DEVELOPMENT San Francisco, California		
RECOMMENDED SPECTRA		
Date 07/01/15	Project No. 750621401	Figure 21
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