PRELIMINARY GEOTECHNICAL INVESTIGATION
469 Stevenson Street
San Francisco, California

Prepared For:

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1.0 INTRODUCTION

This report presents the results of Langan Engineering and Environmental Services, Inc. (LANGAN) preliminary geotechnical investigation for the proposed development at 469 Stevenson Street in San Francisco, California (Site Location Map, Figure 1). The site is a 28,790 -square-foot asphalt-paved surface parking lot in the South of Market (SoMa) District of the City of San Francisco, between Stevenson Street and Jessie Street, east of 6th Street. The –Clearway Energy Station T high-pressure steam cogeneration plant bounds the site to the northeast; three, three-story residential hotel building (35-37, 39-41, and 43-45 6th Street) and a seven-story residential hotel building (47-55 6th Street) bound the site to the southwest. Information regarding basements and foundations for the adjacent structures is not available at this time.

We understand the proposed structure would include a 27-story tower (274 feet tall) with a 1- to 6-level podium. The structure would include a three-level basement that would extend beneath the entire site. Based on schematic drawings by Solomon Cordwell Buenz, dated 25 May 2021 , the tower would occupy the majority of the site and would abut Jessie Street. Assuming a 4-foot-thick mat for the podium and a 10-foot thick mat for the tower, the excavation for a three-level basement would extend approximately 46 to 52 feet below existing site grades. Per Magnusson Klemencic Associates (MKA), the project structural engineer, preliminary average dead plus live foundation pressures are 7,040 pounds per square foot (psf) for the 27-story tower, 2,860 psf for the six-level podium, and 1,760 psf for the one-level podium portions.

This report is based on the results of our Phase 1 investigation (Section 3) performed at the project site. The results of the Phase 1 investigation indicate that on a preliminary basis the proposed structure is feasible from a geotechnical standpoint. In addition, the results of our preliminary engineering analyses indicate a mat foundation is feasible for the support of the proposed structure.

This report presents preliminary conclusions regarding the geotechnical aspects of the project based on the results of a limited geotechnical investigation, and is not intended to meet requirements of AB-082[1] and AB-111[2]. AB-082 presents guidelines and procedures for Structural, Geotechnical, and Seismic Hazard

[1] Guidelines and Procedures for Structural, Geotechnical, and Seismic Hazard Engineering Design Review, November 21, 2018 (Updated 01/01/2020 for code references).

[2] Guidelines for Preparation of Geotechnical and Earthquake Ground Motion Reports for Foundation Design and Construction of Tall Buildings, 15 June 2020.

Engineering Design Review of buildings and other structures. Such review may be required by the San Francisco Building Code, by another Administrative Bulletin, or at the request of the Director of the Department of Building Inspection (SFDBI).

AB-111 presents requirements and guidelines for developing geotechnical site investigations and preparing geotechnical reports for the foundation design and construction of tall buildings. Because the project classifies as a Tall Building (height of levels above the average level of the ground surface adjacent to the structure greater than 240 feet), a design level geotechnical investigation report would need to comply with AB-111. Per AB-111:

1. The review of geotechnical design shall meet the requirements of AB-082. The geotechnical member(s) of the Engineering Design Review Team (EDRT) shall participate in the Early Site Permit phase of the project to review the Geotechnical Engineer of Record (GEOR)'s plan for geotechnical site investigations and the GEOR's geotechnical basis-of-design document. During the subsequent design review, the EDRT will use the AB-111 guidelines to review the geotechnical report prepared for foundation design and construction. At the conclusion of the review, the geotechnical members of the EDRT shall provide a written statement if, in their professional opinion, the geotechnical site-investigation plan and geotechnical reports meet the requirements of the SFBC and this bulletin.

2. Project submittal documents shall be in accordance with the SFBC and Department of Building Inspection (DBI) interpretations, Administrative Bulletins, and policies. In addition, documents relevant to the Geotechnical Design Review shall be submitted by the Engineer of Record to the Director and to the geotechnical members of the EDRT.

Commented [MF1]: Langan cannot interpret AB-111.

The design level geotechnical investigation report should be prepared per AB-082 and AB-111 guidelines, for review by the EDRT assigned to the project by DBI during the review of the site permit. Qualifications and selection of reviewers is detailed in AB-082. Per AB-082, Section 4, Qualifications and Selection of Reviewers, *Geotechnical Engineering Reviewers shall have experience in geotechnical engineering pertinent to the review scope and type of site and foundation. In addition to having the experience described above* (experience detailed in AB-082*), the lead Geotechnical Engineering Reviewer shall be registered as a Geotechnical Engineer (G.E.) or a Civil Engineer (C.E.) in California.* Per AB-082, *Reviewers of seismic hazard and ground motions shall have experience in these fields pertinent to the review scope and the hazard and ground motion approaches being used. In addition to having the experience described above (*experience detailed in AB-082*), the Reviewer of seismic hazard and ground motions shall be registered as a Professional Engineer in California or shall provide his or her services under the responsible charge of a registered Professional Engineer on the Review team.*

2.0 SCOPE OF SERVICES

Our preliminary geotechnical investigation report was prepared in general accordance with the scope of services outlined in our proposal dated 1 March 2022. As part of our services, we reviewed the results of our Phase 1 field investigation and laboratory testing program. The Phase 1 investigation included drilling two borings to bedrock at the site, to depths of 250 and 265 feet below site grades, and performing laboratory testing on representative soil samples. We used this information to perform engineering analyses and develop preliminary conclusions regarding:

- soil, bedrock and groundwater conditions at the site
- site seismicity and seismic hazards, including potential for fault rupture, ground shaking, and seismically induced settlements, as appropriate
- feasible foundation type(s) for the proposed structure
- estimates of foundation settlements, including total and differential settlements;
- feasible shoring and underpinning systems for adjacent structures;
- 2019 San Francisco Building Code (SFBC) seismic design parameters
- site specific response spectra
- construction considerations, including underpinning of adjacent structures, as needed.
- description of the regulatory review and compliance process regarding the geotechnical aspects of the project.

For compliance with AB-111, the design geotechnical investigation report should include the results of additional geotechnical investigation (drilling a third boring to bedrock, anticipated at a depth of approximately 260 feet, and a fourth boring with a 50-foot rock core, performing a seismic survey to obtain additional shear wave velocity measurements in the boring within the bedrock, performing laboratory testing of additional soil and rock samples), earthquake time series, additional engineering analyses, and recommendations for the foundation and other geotechnical aspects of the project. The design level geotechnical investigation report would be reviewed by geotechnical reviewer(s) who are part of the EDRT assigned to the project by DBI.

3.0 FIELD INVESTIGATION AND LABORATORY TESTING

Our Phase 1 geotechnical investigation included drilling two borings within the site; obtaining shear wave velocity data in one of the borings, and performing laboratory tests on representative soil samples as discussed in this section.

3.1 Exploratory Borings

We drilled two borings, designated LB-1 and LB-2, at the locations shown on the Site Plan, Figure 2. Prior to drilling the borings, we obtained drilling permits from the San Francisco Department of Public Health (SFDPH), notified Underground Service Alert (USA) at least 72 hours prior to drilling start time, and retained the services of a private utility locator to check the boring locations for potential underground utilities.

Borings LB-1 and LB-2, were drilled from 16 to 23 December 2020, under the direction of our field engineer. Pitcher Drilling, of East Palo Alto, drilled the borings using a truck-mounted rig equipped with rotary wash. The borings extended to the top of bedrock, at 256 and 250.5 feet bgs, respectively.

The logs of the borings are presented on Figures A-1 through A-2 in Appendix A. The soil and rock encountered in the borings were classified in accordance with the Soil Classification Chart presented on Figure A-3 and the physical properties criteria for rock descriptions on Figure A-4, respectively.

Soil and rock samples were obtained using two types of driven split-barrel samplers and one push sampler:

- Sprague and Henwood (S&H) split-barrel with a 3.0-inch outside diameter and 2.5-inch inside diameter, lined with steel tubes and an inside diameter of 2.43 inches.
- Standard Penetration Test (SPT) split barrel sampler with a 2.0-inch outside diameter and 1.5-inch inside diameter, without liners.
- Shelby Tube (ST) sampler with a 3-inch outside diameter and a 2.93-inch inside diameter.

The sampler types were chosen on the basis of soil type and desired sample quality for laboratory testing. In general, the SPT sampler was used to evaluate the relative density of sandy soil and the S&H sampler was used to obtain samples in medium stiff to very stiff cohesive soil. The ST sampler was used to obtain relatively undisturbed samples of soft to very stiff cohesive soils.

The SPT and S&H samplers were driven with a 140-pound, above-ground, automatic safety hammer falling 30 inches. The samplers were driven up to 18 inches and the hammer blows required to drive the samplers every six inches of penetration were recorded and are presented on the boring logs. A "blow count" is defined as the number of hammer blows per six inches of penetration. The blow counts required to drive

the S&H and SPT samplers were converted to approximate SPT N-values to account for sampler type and hammer energy using factors of 0.7 and 1.2, respectively. The blow counts used for the conversions were: 1) the last two blow counts if the sampler was driven more than 12 inches, 2) the last one blow count if the sampler was driven more than six inches but less than 12 inches, or 3) the only blow count if the sampler was driven six inches or less. The final converted blow counts for each sample are shown on the boring logs.

The ST samplers were pushed hydraulically into the soil; the piston pressures measured in pounds per square inch (psi) required to advance the samplers are shown on the logs.

Upon completion of drilling, each boring was tremie grouted with cement grout in accordance with SFDPH requirements. Boreholes were patched with concrete or asphalt at the ground surface. The soil and rock cuttings and drilling fluids from the borings were collected in 55-gallon drums, which were stored temporarily at the site, tested, and eventually transported off-site for proper disposal.

3.2 Laboratory Testing

In the office, we reviewed the soil and rock samples obtained from our borings to confirm field classifications and select samples for laboratory testing. Soil samples were tested to measure moisture content, dry density, strength, plasticity, and compressibility. The laboratory test results are presented in Appendix B and are summarized on the boring logs.

3.3 Downhole Suspension Logging

Upon the completion of drilling and prior to grouting in Boring LB-1, NorCal Geophysical performed in-situ downhole suspension logging to measure shear and compression wave velocities of the subsurface materials within the boring. The details of the suspension logging methodology, procedures, and the results are presented in Appendix C.

4.0 SITE AND SUBSURFACE CONDITIONS

LANGAN's understanding of the site and subsurface conditions described in this section of the report are based on the results of our 2020 Phase 1 geotechnical investigation for the proposed development and a review of published literature.

4.1 Site Conditions

The site is a 28,790 square-foot asphalt-paved surface parking lot. Site grades are relatively level and range between Elevation 28.5 and 31 feet.[3]

The Clearway Energy Station T high-pressure steam cogeneration plant bounds the site to the northeast; three, three-story residential hotel building (35-37, 39-41, and 43-45 6th Street) and a seven-story residential hotel building (47-55 6th Street) bound the site to the southwest. Information regarding basements and foundations for the adjacent structures is not available at this time.

4.2 Subsurface Conditions

The site is outside of the historical shoreline, locally referred to as the Sullivan Marsh (see Figure 3) and within the regional seismic hazards zones map (Figure 6)

The available subsurface information indicates that in general, the site is underlain fill, Dune sand, Marsh deposit, Colma Formation sand, Old Bay Clay, alluvium, and Franciscan Complex bedrock.

4.2.1 Soil and Rock Conditions

The material types and general descriptions of their physical characteristics are summarized below:

Fill: The site is blanketed by 8 to 8½ feet of very loose to medium dense sand with varying silt and clay contents, with brick, concrete, and other debris fragments.

Dune Sand: The fill is underlain by a 19- to 19½-foot-thick layer of fine-grained, poorly graded sand (Dune sand). The sand is loose to dense, and typically grades denser with depth. The sand is moist to wet and extends to depths of 27 to 28 feet below site grades, an approximate Elevation of 2 feet.

Marsh Deposit: A 6½- to 10-foot-thick Marsh deposit underlays the Dune sand. This deposit consists of medium dense clayey sand and medium stiff sandy clay. The bottom of the Marsh deposit extends to depths of 37 to 38 feet below site grades, approximate elevations of -7 to -9 feet.

Colma Formation: Beneath the marine deposit (below depths of 37 to 38 feet bgs) is a 60- to 77½-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, is generally strong and relatively incompressible. The

Commented [MF2]: Added text clarifies the top of Colma (foundation bearing material) is within 37 to 38 feet bgs.

[3] Elevations from *Topographic and Boundary Survey of 469 Stevenson Street*, by Luk and Associates dated 24 August 2018, and are based on the Historic City of San Francisco datum.

Colma Formation extends to depths of 98 and 114.5 feet bgs, about Elevation -69 and -84.5 feet. A 2-foot-thick medium still clay layer was encountered at 89 feet bgs within the Colma Formation at Boring LB-2.

Old Bay Clay: The Colma Formation is underlain by a 24- to 37-foot thick layer of marine clay known locally as Old Bay Clay. Old Bay Clay is medium stiff to very stiff with overconsolidation ratios[4] about 1.8 to 2.0. The Old Bay Clay extends to depths of 135 to 138.5 feet bgs, about Elevation -106 to -108.5 feet.

Alluvium/Residual Soil: The Old Bay Clay is underlain by dense to very dense sand and very stiff to hard clay (alluvium and residual soil) to bedrock. Consolidation test results indicate the alluvial clay is overconsolidated and slightly compressible. The alluvium/residual soil extends to depths of about 243 to 249 feet, about Elevation -220 to -213 feet, which is approximate top of bedrock.

Bedrock: Bedrock at the site consists of a Franciscan Complex Mélange, typically a mixture of sheared and folded sedimentary, igneous, and metamorphic rocks resulting from large-scale tectonic processes. Bedrock consists predominantly of siltstone and sandstone, and is intensely fractured to fractured, low to moderately hard, weak to friable, and little weathered.

4.2.2 Groundwater

During our 2020 investigation groundwater levels were measured in the borings at approximately 19.5 and 32 feet from existing site grades during and after drilling. However, these measurements do not represent stabilized groundwater levels. The groundwater level would vary seasonally depending on rainfall infiltration and time of year. In addition, the groundwater level would vary from dewatering activities in the vicinity and utility leaks. The site is also sufficiently close to the San Francisco Bay to be influenced by future sea level rise (see Section 4.3). On the basis of the available groundwater information (including the historic groundwater levels, between 10 and 30 feet bgs, assuming an average of 20 feet bgs) and past investigations in the vicinity of the site, and to account for seasonal fluctuations and a reasonable consideration for near-future sea level rise, we judge the groundwater level within the project site could rise to within 16 feet from existing street grades, which corresponds to Elevation 13 feet.

4.3 Sea Level Rise

Per Sea Level Rise Vulnerability (SLRV) and Consequences Assessment (2020)[5] for the City of San Francisco, by the end of the century (year 2100), about 5.5 feet of sea level rise (SLR) could occur, which represents the upper-bound projection. For long-range planning, Capital Planning Committee

[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.

[5] https://sfplanning.org/sea-level-rise-action-plan#info

Guidance defines a SLRV Zone based on the National Research Council's (NRC) upper range (unlikely, but possible), end-of-century SLR estimate, in the event that future greenhouse gas emissions and land ice melting accelerates beyond current predictions. The Zone, therefore, includes shoreline areas that could be exposed to 66 inches of permanent SLR inundation with temporary flooding from a 100-year extreme tide if no adaptation measures or actions are taken. The 100-year extreme tide is consistent with Preliminary Flood Insurance Rate Maps (FIRMs) released by the Federal Emergency Management Agency (FEMA) in November 2015 and with FEMA's West Coast SLR Pilot Study (2015). For ongoing planning and development purposes related to environmental review and project approvals, the City uses the NRC's most likely SLR projection of 36 inches. The project site is not within the San Francisco SLRV zone.

5.0 REGIONAL SEISMICITY AND FAULTING

The project site is in a seismically active region. Numerous earthquakes have been recorded in the region in the past, and moderate to large earthquakes should be anticipated during the service life of the proposed development. The San Andreas, San Gregorio, and Hayward faults are the major faults closest to the site. These and other faults of the region are shown on Figure 4. For each of these faults, as well as other active faults within about 50 kilometers (km) of the site, the distance from the site and estimated mean Moment magnitude[6] [2014 Working Group on California Earthquake Probabilities (WGCEP) (2015) and Uniform California Earthquake Rupture Forecast Version 3 (UCERF3) as detailed in the United States Geological Survey Open File Report 2013-1165] are summarized in Table 1. The mean moment magnitude presented in Table 1 was computed assuming full rupture of the segment using Hanks and Bakun (2008) relationship.

TABLE 1
Regional Faults and Seismicity

Fault Segment	Approx. Distance from Fault (km)	Direction from Site	Mean Characteristic Moment Magnitude
San Andreas 1906 event	13.3	Southwest	8.1
Total Hayward-Rodgers Creek Healdsburg	17	East	7.6
Total San Gregorio	18	West	7.6
Pilarcitos	20	Southwest	6.7
Contra Costa (Lafayette)	29	East	6.1
Contra Costa Shear Zone (connector)	30	East	6.6
Franklin	31	Northeast	6.7

[6] 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.

Fault Segment	Approx. Distance from Fault (km)	Direction from Site	Mean Characteristic Moment Magnitude
Contra Costa (Larkey)	32	East	6.0
Contra Costa (Dillon Point)	33	Northeast	6.1
Total Calaveras	33	East	7.5
Monte Vista - Shannon	34	South	7.0
Mount Diablo Thrust	34	East	6.6
Mission (connected)	35	East	6.1
Concord	39	East	6.4
Green Valley	41	Northeast	6.8
Contra Costa (Vallejo)	41	Northeast	5.6
Contra Costa (Lake Chabot)	42	Northeast	5.6
Clayton	45	East	6.4
West Napa	46	Northeast	6.8
Greenville	48	East	7.1

Note:
1. The table above is a summary and does not include all the fault segmentation, alternate traces and low activity faults included in the UCERF3 model.

Figure 4 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 5) 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), corresponding to an 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), an 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 an M_w of 6.9, the epicenter of which is approximately 95 km from the site.

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 an 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 most recent earthquake to affect the Bay Area occurred on 24 August 2014 and was located on the West Napa fault, approximately 49 km northeast of the site, with an M_W of 6.0.

The 2016 U.S. Geologic Survey (USGS) predicted a 72 percent chance of a magnitude 6.7 or greater earthquake occurring in the San Francisco Bay Area in 30 years (Aagaard et al. 2016). More specific estimates of the probabilities for different faults in the Bay Area are presented in Table 2.

TABLE 2
Estimates of 30-Year Probability (2014 to 2043) of a Magnitude 6.7 or Greater Earthquake

Fault	Probability (percent)
Hayward-Rodgers Creek	33
Calaveras	26
N. San Andreas	22
San Gregorio	16
Mount Diablo Thrust	16
Greenville	6

6.0 SEISMIC HAZARDS

During a major earthquake on a segment of one of the nearby faults, strong to very strong shaking is expected to occur at the project site. Strong shaking during an earthquake can result in ground failure such as that associated with soil liquefaction[7], lateral spreading[8], and seismic densification[9]. Each of these conditions has been evaluated based on our geotechnical investigation, literature review and analyses, and is discussed in this section.

6.1 Ground Shaking

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 could be felt at the site.

[7] 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.

[8] 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.

[9] 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.

The intensity of earthquake ground motion at the site would depend upon the characteristics of the generating fault, distance to the earthquake fault, magnitude and duration of the earthquake, and specific subsurface conditions.

To quantify 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 on the development of the recommended spectra for the project are presented in Section 8 and Appendix D.

6.2 Liquefaction and Associated Hazards

When a saturated soil with little to no cohesion liquefies during a major earthquake, it experiences a temporary loss of shear strength as a result of a transient rise in excess 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 Divisions of Mines and Geology (CDMG) 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 2001 (Figure 6). California Geological Survey (CGS; former CDMG) has recommended the content for site investigation reports within seismic hazard zones be performed in accordance with Special Publication 117A, *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. No observations of liquefaction and lateral spreading were documented near the project site during either the 1906 San Francisco or 1989 Loma Prieta earthquakes (Youd and Hoose, 1978) and (Holzer, 1998).

We generally used the procedures from the Boulanger and Idriss (2014) method for the evaluation of liquefaction triggering for the soil at the site. The level of ground shaking used in our liquefaction evaluation was based on the Maximum Considered Earthquake Geometric Mean (MCE_G). A site-specific MCE_G peak ground acceleration (PGA_M) value of 0.614 times gravity was used in our analyses. We used a design ground water depth of 16 feet (Elevation 13 feet) and a magnitude of 8.05 earthquake, which is the maximum Moment Magnitude for the San Andreas Fault, located about 13.3 kilometers from the site as shown in Table 1.

The results of our analyses indicate that the loose to medium dense Dune sand and medium dense clayey sand within the Marsh deposit, encountered below the design groundwater level, are susceptible to liquefaction during a major seismic event on a nearby fault. Using the procedures described by Tokimatsu and Seed (1984) and Cetin (2009), which includes a factor that scales the contribution of individual

liquefiable layers to total surface settlement depending on the depth of the layer, we estimate that liquefaction-induced settlement in the Dune sand and Marsh deposit sand could be on the order of 2 inches during an MCE_G event. The potentially-liquefiable soil would be removed in its entirety beneath the proposed structure during basement excavation. Therefore, liquefaction-induced settlement would not affect the performance of the proposed structure.

6.3 Lateral Spreading

Lateral spreading is a phenomenon in which a surficial soil displaces along a shear zone that has formed within an 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 liquefied soil should consist of saturated cohesionless sediments with penetration resistance, $(N_1)_{60}$, less than 15.

Our evaluation indicates the soil susceptible to liquefaction in the borings generally had a corrected blow counts $(N_1)_{60\text{-}cs}$ value greater than 15, and therefore the potential for lateral spreading at the site is low.

6.4 Seismic Densification

Seismic densification can occur during strong ground shaking in loose, clean granular deposits above the water level, resulting in ground surface settlement. The degree of susceptibility to seismic densification is directly related to the relative density of the existing granular soil.

In general, the loose to medium dense, granular fill and Dune sand encountered above the groundwater table at the site is susceptible to seismic densification. Using the Pradel (1998) method for evaluating seismically-induced settlement in dry sand, we expect localized seismic densification on the order of ½ inch to 8 inches can occur in these layers near the project site. This settlement is in addition to liquefaction induced settlement discussed in Section 6.2. As with the liquefiable soils, the fill and Dune sand susceptible to seismic densification would be removed in their entirety by the proposed basement excavation, and therefore seismic densification is not expected to affect the performance of the proposed structure.

6.5 Fault Rupture

Historically, ground surface fault rupture closely follows the trace 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 known active or potentially active faults exist on the site. We therefore conclude the risk of fault offset rupture at the site from a known active fault is low. 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.

6.6 Tsunami

Based on recent published maps (California Emergency Management Agency (CEMA), 2009), the site is not within the limits of the tsunami inundation area. Therefore, the potential for tsunami inundation is low

7.0 DISCUSSION AND CONCLUSIONS

From a geotechnical standpoint, and based on the results of the Phase 1 investigation we conclude the proposed structure is feasible as planned. However, the project feasibility should be further confirmed based on the results of the Phase 2 field investigation and laboratory testing program.

To construct the proposed structure and basement, temporary shoring, dewatering, excavations on the order of 46 to 52 feet bgs, and installation of an appropriate foundation would be required. The primary geotechnical considerations for the proposed project include:

- selection of appropriate foundation(s) for the proposed structure
- selection of appropriate shoring system(s) to support the excavation, surrounding buildings, streets, and utilities during construction of the basement and foundation
- presence of groundwater within 16 feet from the existing site grades
- presence of adjacent buildings
- earthquake-induced ground deformations outside of the proposed basement footprint

A summary of the geotechnical issues is presented below; these and other geotechnical issues are discussed in the remainder of this section.

7.1 Mat Foundation and Settlement

The excavation for the proposed structure and mat foundation would extend below the fill, Dune sand, and Marsh deposit; Colma Formation would be encountered at the foundation subgrade. The proposed structure can be supported on a mat bearing on dense to very dense Colma Formation provided the settlement induced by the anticipated building loads is acceptable.

To evaluate ground settlement from the anticipated building loads we developed a settlement model using Settle3[10]. Our settlement analysis is based on the following assumptions:

- average foundation pressures by MKA for dead plus live loads of 7,040 psf for the 30-story tower 2,860 for the 6-level podium, and 1,760 for the 1-level podium portions a 4-foot-thick mat for the podium and a 10-foot thick mat for the tower,
- bottom of excavation including the mat at 46 feet bgs for the podium and 52 feet bgs for the tower
- groundwater at 16 feet bgs (Elevation 13 feet)
- unit weight of 150 pounds per cubic foot for the mat

We used the following soil properties for our settlement analyses:

- Confined modulus, E, of 6,000 kips per square foot for Colma and alluvium dense to very dense sand.
- Overconsolidation ratio (OCR) of 1.8, and recompression ratio of 0.05 in the stress range of interest, for Old Bay Clay.
- OCR of 2, and a recompression ratio of 0.03 in the stress range of interest, for alluvium clay.

For our settlement analyses, we modeled site dewatering and excavation, and building construction, using the following assumed stages and durations:

- Dewatering and excavation to the bottom of the mat occurs over a three month period (time t = 0 when dewatering begins; t= 3 months for excavation completion).
- Open excavation for mat construction occurs over a one month period; (t= 4 months).
- Building is constructed, dewatering is turned off and water returns to original elevation one year after the mat is constructed (t= 1 year and 4 months).

The results of our settlement analyses indicate ground settlements between 1 to 2 inches, 50 years after the end of construction, at the podium and 2 to 3¾ inches at the tower portions of the structure. Anticipated settlement contours 50 years after construction are included in Figure 6. The settlement contours do not include settlement under the weight of the mat (contours present settlement after mat placement).

[10] Settle3, version 4.023

Final settlement analyses for a mat foundation should include the results of the Phase 2 design level geotechnical investigation.

The mat foundation should be waterproofed and designed to resist hydrostatic pressures. If the weight of the building and mat are not sufficient to resist uplift loads, additional uplift resistance may be provided using tiedown anchors gaining capacity in Colma Formation. During construction the structural engineer needs to determine when the dewatering can be turned off.

7.2 Deep Foundations

The results of the settlement analyses indicate a mat foundation is feasible for the support of the proposed structure. The feasibility of the mat needs to be confirmed with additional settlement analyses that would incorporate the results of the Phase 2 field investigation and laboratory testing program. If the supplemental settlement evaluation indicates a mat is not feasible, then deep foundations that extend through the Old Bay Clay into the underlying alluvium and residual soil and/or Franciscan Formation bedrock would be required. Large-diameter, drilled cast-in-place piers (also known as drilled shafts) are feasible.

Drilled shafts should be installed using polymer drilling slurry; the use of bentonite slurry should be excluded. In addition to slurry, casing should be installed extending to the bottom of the shaft or top of bedrock. For drilled shafts the concrete should be placed using tremie techniques to displace all of the drilling fluid.

Drilled shafts should transfer structural loads to the relatively incompressible sand and clay deposits and/or bedrock below the Old Bay clay; however, some settlement of the foundations would still occur. Considering the anticipated foundation lengths and loads, the foundation elements could compress about 1 to 2 inches. Differential settlement of about one inch is anticipated between adjacent foundation elements.

7.3 Ground Settlement Outside the Proposed Structure

Exterior slabs, driveways, utilities, and utility connections at the building interface should be designed to accommodate potential differential settlement of up to 10 inches where the improvements settle relative to the building as a result of liquefaction and seismic densification. They should also accommodate the anticipated static building settlement of up to 2.5 inches where the building settles relative to exterior improvements. These settlements are expected to occur at different times during the life of the building. The total anticipated differential settlement at the building interface is on the order of 10 inches.

7.4 Shoring

We anticipate the soil at the site can be excavated with conventional earthmoving equipment such as loaders and backhoes. However, remnants of any buried foundations and/or building slabs and other debris may be encountered, which could require the use of jack hammers or hoe-rams to break apart and remove. Additionally, concrete debris should be expected within the fill.

Construction of the basement and mat would require an excavation on the order of about 46 to 52 feet bgs. The excavation would need to be shored to protect the surrounding improvements. There are several key considerations in selecting a suitable shoring system. Those we consider of primary concern are:

- protection of surrounding improvements, including roadways, utilities, and nearby structures
- penetration of shoring system into the dense Colma Formation below the bottom of the excavation
- control of groundwater inflow to limit groundwater drawdown levels
- presence of and potential difficulty of dewatering the marsh deposit along the sides of the excavation
- proper construction of the shoring system to reduce the potential for ground movement
- construction costs.

As noted in Section 4.1, the site is adjacent to buildings on the northeast and southwest. The planned excavation would need to be retained with a stiff shoring system designed to limit the shoring deflections adjacent to the existing structures. The shoring would need to be designed for the surcharge pressures from the buildings or the buildings should be underpinned prior to site excavation.

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

- conventional soldier pile and lagging
- DSM impervious walls
- deep concrete diaphragm walls.

Soldier pile and lagging is typically the most economical shoring system, consisting of steel beams and concrete placed in predrilled holes extending below the bottom of the excavation. Wood lagging is placed between the piles as the excavation proceeds, and tiebacks and/or internal bracing can be installed if additional support is needed. Experience with other deep excavations in San Francisco has shown that groundwater would likely perch on top of the clay within the marsh deposits, which could cause the sand

above the clay to flow into the excavation and cause settlement beyond the limits of the excavation footprint. Dewatering in and through the marsh deposit could be difficult. Because it would be necessary to dewater within the proposed excavation, the selected shoring system should be relatively impervious in order to limit seepage and soil loss through the shoring and reduce the drawdown of groundwater outside the site to limit settlement. Therefore, we conclude that a soldier pile and lagging system is not a viable shoring system for the project, and an impervious system should be used.

Impervious temporary shoring walls can be constructed using deep soil mixed (DSM) elements. The walls are constructed by treating soil in place with cement grout using mixing shafts consisting of auger cutting heads (referred to as the cutter soil mix method, or CSM), discontinuous flight augers, blades/paddles, or a specialty mixing tool to create DSM columns or panels. The DSM columns or panels are installed in an overlapping pattern to create a continuous impervious wall. Steel beams are placed in some of the DSM columns or panels to provide rigidity. DSM walls are considered temporary; permanent walls are built within the shoring. Because these walls are continuous, they would temporarily reduce groundwater infiltration, resulting in the need for less dewatering. In addition, DSM walls are generally more rigid than soldier piles and lagging and can result in less shoring deformations. To properly reduce groundwater inflow in the excavation, the impervious wall would need to extend at least 30 feet below the bottom of the excavation; the actual embedment below the bottom of the excavation wouldl need to be determined by the shoring/dewatering design engineer.

Concrete diaphragm walls are reinforced concrete walls constructed by slurry trench method. The walls are constructed in sections or panels; careful, alternating panel installation and sequencing is necessary to provide sufficient support to adjacent structures until a previously installed panel has attained sufficient strength. During excavation of a panel, slurry is pumped into and maintained within 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 placed through a tremie pipe. One primary difference between concrete diaphragm walls and a DSM wall is that the diaphragm wall is comprised of structural strength concrete and 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.

Due to the planned excavation depths, the shoring walls would require grouted tiebacks and/or internal bracing for additional lateral support. Tiebacks would require encroachment agreements from adjacent property owners as well as permits from the City of San Francisco. If adjacent buildings have basements, the basements would inhibit installation of tiebacks and depth of the basements should be checked for shoring design stresses. **Consequently, internal bracing (diagonal, cross-lot, and/or rakers) could be required to retain the shoring walls**. If tiebacks are used, they should be drilled using a smooth-cased method to reduce the potential for loss of ground beneath adjacent buildings and street improvements.

Commented [MF3]: Highlighted section outlines methods that can be implemented if encroachment permits cannot be obtained from adjacent owners..

Installation of tie-backs below the groundwater level could be problematic from both soil caving and water control perspectives.

To support the adjacent buildings during excavation underpinning consisting of slant-drilled piles gaining support in the Colma Formation (below bottom of the excavation), as discussed in Section 7.6, can be used.

Where underpinning is not feasible, the shoring should be designed for the surcharge from adjacent foundations.

The design, construction, and performance of the shoring and underpinning systems should be the responsibility of the contractor and should be designed by an engineer knowledgeable in this type of construction. We should review the geotechnical aspects of the shoring system proposed by the contractor prior to installation.

7.5 Excavation Settlement and Monitoring

Shoring systems are expected to deflect during installation and excavation. This lateral displacement could manifest itself as settlement and/or lateral movement of adjacent improvements. The magnitude of these movements is difficult to estimate because it depends on many factors, including the type of shoring system used and the contractor's skill in installing it. Clough and O'Rourke (1990) analyzed measured lateral displacements and associated ground settlements behind actual excavations in sand and concluded that both the lateral movements and settlements varied from 0.1 to 0.3 percent of the excavation depth. Therefore, for the anticipated excavation depths of about 46 to 52 feet, these empirical relationships would suggest impervious DSM shoring would likely displace laterally about ½ inch to 1.5 inches. These estimates assume the quality of construction would meet or exceed that considered standard in the construction industry. Control of ground movement would depend on the timeliness of installation of lateral restraint as well as on the design and construction techniques. Potential shoring deformations should be calculated by the shoring designer.

The associated settlements predicted from the empirical data suggest ground surface settlements behind the shoring would have a similar magnitude as the lateral movement. The settlement typically manifests as a trough, with the greatest settlement occurring at a horizontal distance behind the shoring at between about ½ and **1** times the height of the excavation. Beyond this length, the estimated settlement should decrease with distance from the wall, and should be very small at a distance twice the excavation depth. A monitoring program should be established prior to installing the shoring system to monitor and evaluate the effects of the construction on the adjacent improvements. The monitoring program would be

included in the shoring drawings, and reviewed by the GEOR. The GEOR would confirm implementation of the monitoring program.

A pre-construction conditions documentation and monitoring program of existing improvements should be implemented for identifying conditions of areas before construction commences, and to confirm impacts (if any) due to the installation and performance of the shoring (Section 7.9). A monitoring program should be implemented to establish a baseline of conditions before starting construction and identify the effects of the construction on the adjacent buildings and improvements. The monitoring program should include survey points, vibration and sound-level monitors, tilt-meters, and crack meters installed in and on adjacent structures, and inclinometers to monitor the movement of shoring walls, and piezometers to monitor groundwater levels.

7.6 Underpinning

Where the proposed excavation extends deeper than the foundations of adjacent buildings and if the shoring is not designed for the surcharge from the adjacent foundations, underpinning should be provided to support the adjacent building loads. Surcharge from adjacent foundations would need to be considered in the design of the shoring and permanent basement walls of the proposed structure, or, the adjacent buildings would need to be underpinned

Underpinning could consist of steel piles installed in slant-drilled shafts (slant piles). The excavation face between the underpinning piles should be retained using soil mixed piers, provided the existing footing can span between piles. The underpinning piles should be designed to resist vertical building loads, vertical tieback loads (if tiebacks are used), and lateral earth pressures. The piles should be pre-loaded by jacking against the foundation, and the top of the pile dry-packed to fit tightly with the base of the underpinned foundation. Underpinning piles should act in end bearing in the Colma Formation below the depth of the proposed excavation, while slant piles gain their capacity in friction along the sides of the shaft. Alternatively, the shoring system can be designed for the foundation surcharge imposed by the adjacent structures.

7.7 Groundwater and Dewatering

Groundwater in the borings drilled on in the site was encountered within 19.5 feet bgs; the high groundwater level can be 16 feet bgs (approximately Elevation 13 feet). Elevation 13 feet should be assumed as the design groundwater level for preliminary evaluations.

The mat foundation would extend below the design groundwater level. The mat and below-grade walls would need to be waterproofed and designed to resist uplift and hydrostatic pressures based on the high anticipated groundwater level.

For an impervious wall shoring system (such as a DSM wall, secant pile wall, or concrete diaphragm wall), we anticipate dewatering only within the site would be required to facilitate excavation for the basement. The dewatering system would need to account for excavation of soil beneath the mat. The use of an impervious shoring system would limit the potential for lowering of the groundwater level outside of the excavation. The contractor should be prepared to control groundwater after final subgrade has been reached.

Commented [MF4]: A DSM wall is a feasible shoring system, see Section 7.4.

Variables that would influence the performance of the dewatering system and the quantity of water produced include the shoring design (e.g., the depth of the impervious wall), 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. The site dewatering should be designed by an experienced dewatering designer and implemented by an experienced dewatering contractor to reduce potential for settlement outside the excavation, relative to the baseline groundwater elevation established prior to excavation. The dewatering designer should establish soil hydraulic conductivity values, as needed, and perform site specific pump tests or other appropriate laboratory or field tests needed to confirm hydraulic conductivity values for soil.

A monitoring program should be implemented to establish the baseline pre-construction groundwater levels at the site for a period of at least twelve months to capture seasonal fluctuations in groundwater. Groundwater monitoring should continue for the duration of the operation of the dewatering system, at a minimum. The monitoring program should be included in the shoring drawings, and reviewed by the GEOR. The GEOR would confirm implementation of the monitoring program.

The contractor would need to obtain a dewatering and discharge permit from the City and County of San Francisco Public Utility Commission (SFPUC) for discharging water into the local combined sewer system. Currently, there is a fee for disposing of construction generated water into the City's wastewater collection system. Selection of the shoring and dewatering systems should be coordinated to reduce overall costs.

7.8 Construction Considerations

Because the excavation would extend below groundwater, the soil at subgrade level would be near saturation even after dewatering. To protect the subgrade, heavy construction equipment (such as loaders or heavy excavators) should not be allowed within three feet of subgrade and the final excavation

can be made with an excavator equipped with a smooth bucket. Following final excavation, the mat subgrade can be protected by pouring a slab consisting of 3 to 4 inches of lean concrete.

Concrete fragments were encountered in the fill in one of the borings. In addition, building foundation elements from previous structures could be encountered. Temporary shoring installation could be impeded by the presence of rubble in the fill. Coring or other means would need to be used to install shoring through buried foundation elements, or, the buried elements would need to be removed prior to shoring installation.

Because the project site is in the Maher area, handling and disposal of the fill material would need to be performed in accordance with a site mitigation plan (SMP) that includes health and safety criteria.

7.9 Construction Monitoring

A pre-construction survey and monitoring program should be undertaken prior to installation of shoring, excavation, and foundation installation to monitor the effects of these operations. The requirement for a pre-construction survey should be included in the shoring drawings. The survey should include documenting the condition of the surrounding structures, including a crack survey, prior to and following construction. The monitoring should provide timely data, which can be used to modify the shoring system if needed. Survey points should be installed on the shoring and on the adjacent streets, buildings, and other improvements that are within 150 feet 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 the help of the geotechnical engineer, so they can provide the most value to the project.

Commented [MF5]: As discussed previously, a pre-construction survey and monitoring program should be included in the shoring drawings, and reviewed by the GEOR. The GEOR will confirm implementation of the monitoring program.

To monitor ground movements, shoring movements, and dewatering outside the site, we recommend installing the instrumentation listed below:

Slope indicators: We recommend installing a slope indicator on each side of the shoring. Inclinometers should extend to a depth of at least 50 feet below the maximum excavation depth.

Piezometers: We recommend installing a piezometer on Stevenson and Jessie Streets behind the shoring walls.

Survey points: Survey points should be installed on the shoring, underpinning, adjacent streets, and neighboring buildings within 50 feet of the excavation perimeter prior to the start of excavation. These survey points should be used to monitor the movement of the shoring and surrounding facilities during excavation.

The survey points and slope inclinometers should be measured every week until construction of the below-grade garage is complete. In addition, a thorough crack survey of buildings within 50 feet of the excavation should be performed prior to starting construction to provide a baseline in case claims of building damage caused by the proposed construction are made. The contractor should provide safe access to all inclinometer locations. Where limited space is available, platforms may need to be constructed.

8.0 SEISMIC DESIGN

We expect this site would experience 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 two levels of shaking corresponding to the Risk-Targeted Maximum Considered Earthquake (MCE_R) per the 2019 SFBC/ASCE 7-16.

The MCE_R is defined in the ASCE 7-16 as the lesser of the probabilistic spectrum having two 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 SLE spectrum is defined as a probabilistic spectrum with a 50 percent probability of exceedance in 30 years (43 year return period).

We performed probabilistic seismic hazard analysis (PSHA) and deterministic analysis to develop recommended horizontal spectra at the ground surface for the buildings for the Risk-Targeted Maximum Considered Earthquake (MCE_R) and Design Earthquake (DE) consistent with ASCE 7-16 and 2019 SFBC. Details of our analysis are presented in Appendix D.

The recommended horizontal basement level spectra are shown on Figure 8. Digitized values of the recommended MCE_R spectrum for a damping ratio of 5 percent are presented in Table 3.

TABLE 3
Recommended MCE_R and SLE Spectra Spectral Acceleration (g's)

Period (seconds)	MCE_R (5% Damping)
0.01	0.731
0.10	1.170
0.20	1.599
0.30	1.791
0.40	1.807
0.50	1.721

0.75	1.377
1.00	1.200
1.50	0.800
2.00	0.600
3.00	0.400
4.00	0.300
5.00	0.240
7.50	0.160
10.00	0.120

Because site-specific procedure was used to determine the recommended response spectra, the corresponding values of S_{MS} and S_{M1} per Section 21.4 of ASCE 7-16 should be used, as shown in Table 4.

TABLE 4
Design Spectral Acceleration Value

Parameter	Spectral Acceleration Value (g's)
S_{MS}	1.626*
S_{M1}	1.200

* Governed by the spectral value at 0.4 seconds

9.0 LIMITATIONS

The discussion and conclusions provided in this report result from LANGAN's interpretation of the geotechnical conditions existing at the site inferred from a limited number of borings. Actual subsurface conditions could vary. Recommendations for site grading, foundation and basement wall design, temporary shoring, seismic design, and other geotechnical aspects of this project should be developed after a design level (including a Phase 2) field investigation and laboratory testing program, and supplemental engineering analyses are performed. The results of the preliminary (Phase 1) field investigation and laboratory testing program and settlement analyses indicate a mat is feasible for the support of the proposed structure. Mat feasibility should be confirmed with the Phase 2 investigation program. The mat would be supported on dense to very dense sand of the Colma Formation. If drilled shafts to bedrock are required, design recommendations should be developed based on the results of the Phase 1 and Phase 2 exploration programs.

This report is preliminary and presents preliminary conclusions regarding the geotechnical aspect of the project based on the results of a limited geotechnical investigation, and is not intended to meet

requirements of AB-082 and AB-111. The design level geotechnical investigation report should be prepared per AB-082 and AB-111 guidelines, for review by the EDRT assigned to the project by DBI.

Any proposed changes in structures, depths of excavation, or their locations should be brought to LANGAN's attention as soon as possible so that LANGAN can determine whether such changes affect the recommendations for the design level geotechnical investigation. 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.

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's recommendations. When authorized, LANGAN would assist the design team in preparing specification sections related to geotechnical issues such as earthwork, foundation design, backfill, and excavation support. Langan 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.

Commented [MF6]: Please see first paragraph in Section 9.0 that addresses that a preliminary report cannot be used for foundation design and that a design level report is needed to confirm feasibility of foundations discussed in the preliminary report.

LANGAN should 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, earthwork, backfill, tiedowns, and foundations, and excavation support. In fulfillment of these duties, during construction we should observe the installation of the temporary shoring, including testing of tiebacks. 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 would review monitoring data pertaining to shoring system performance and settlement of adjacent structures provided by the surveyor. Our engineer should observe installation and testing of any tiebacks and tiedowns, mat foundation subgrade preparation and installation of drilled piers, if used. 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 design, quality assurance observation during construction by LANGAN 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 that would be included in Lagan's design level geotechnical investigation would be predicated upon LANGAN reviewing the final design and providing construction observation services for the owner.

REFERENCES

2014 Working Group on California Earthquake Probabilities, 2015, "UCERF3: A new earthquake forecast for California's complex fault system," U.S. Geological Survey 2015–3009, 6 p., http://dx.doi.org/10.3133/fs20153009.

Boulanger, R.W. and Idriss, I.M. (2008). "Soil Liquefaction during Earthquakes," Earthquake Engineering Research Institute.

Bray, J. D. and Sancio, R. B. (2006). "Assessment of the Liquefaction Susceptibility of Fine-Grained Soils," Journal of Geotechnical and Geoenvironmental Engineering, Vol. 132, No. 9, ASCE.

California Building Standards Commission, 2019 California Building Code.

California Code of Regulations (CCR) Title 27, Environmental Protection.

California Division of Mines and Geology (1996). "Probabilistic Seismic Hazard Assessment for the State of California." DMG Open-File Report 96-08.

California Division of Mines and Geology, (1997). "Fault Rupture Hazard Zones in California," CDMG Special Publication 42.

California Department of Conservation Division of Mines and Geology (1997). Guidelines for Evaluating and Mitigating Seismic Hazards in California. Special Publication 117.

Cao, T., Bryant W.A., Rowshandel, B., Branum D. and Wills, C.J. (2003). "The revised 2002 California Probabilistic Seismic Hazard Maps June 2003," California Geological Survey.

Fugro West Inc., Geotechnical Study, 680 Folsom Street Seismic Retrofit, December 2007.

Harding Lawson Associates, "Soil Investigation, 22-Story Office Building and Parking Garage, 95 Hawthorne Street, San Francisco, California" project number 13165,001.04, dated 19 May 1983. The investigation was performed at 75 Hawthorne Street, not 95 Hawthorne Street.

Helley, E.J., and Herd, D.G., 1977, Map showing faults with Quaternary displacement, northwestern San Francisco Bay region, California: U.S. Geological Survey Miscellaneous Field Studies map MF-881, scale 1:125,000.

Idriss, I.M. and R.W. Boulanger (2008). "Soil Liquefaction During Earthquakes," EERI, MNO-12.

Ishihara, K. and Yoshimine, M. (1992). "Evaluation of Settlements in Sand Deposits Following Liquefaction during Earthquakes," Soils and Foundations, Vol. 32, No. 1, pp. 173-188.

Jennings, C.W. (1994). "Fault Activity Map of California and Adjacent Areas," California Division of Mines and Geology Geologic Data Map No. 6, scale 1: 750,000.

REFERENCES
(Continued)

Map titled "State of California Seismic Hazard Zones, City and County of San Francisco Official Map" prepared by the California Department of Conservation, Division of Mines and Geology, dated 17 November 2001.

Pradel, Daniel (1998). "Procedure to Evaluate Earthquake-Induced Settlements in Dry Sand," Journal of Geotechnical and Geoenvironmental Engineering, April, and errata October 1998, pp1048

Seed, H.B. and Idriss, I.M. (1971). "Simplified Procedure for Evaluating Soil Liquefaction during Earthquakes," Journal of Geotechnical Engineering Division, ASCE, 97 (9), 1249-1273.

Sitar, N., Mikola, R. G. and Candia, G. (2012). "Seismically Induced Lateral Earth Pressures on Retaining Structures and Basement Walls." Geotechnical Engineering State of the Art and Practice, Keynote Lectures from GeoCongress 2012, Geotechnical Special Publication No. 226, ASCE.

Tokimatsu, K. and Seed H. B. (1987). "Evaluation of Settlements in Sand due to Earthquake Shaking." Journal of Geotechnical Engineering, Vol. 113, No. 8, pp. 861-878.

Toppozada, T. R. and Borchardt G. (1998). "Re-Evaluation of the 1836 "Hayward Fault" and the 1838 San Andreas Fault earthquakes." Bulletin of Seismological Society of America, 88(1), 140-159.

Townley, S. D. and Allen, M. W. (1939). "Descriptive Catalog of Earthquakes of the Pacific Coast of the United States 1769 to 1928." Bulletin of the Seismological Society of America, 29(1).

U.S. Geological Survey, 1996, Database of Potential Sources for Earthquakes Larger than Magnitude 6 in Northern California.: Open File Report 96-706.

Wesnousky, S. G. (1986). "Earthquakes, Quaternary Faults, and Seismic Hazards in California." Journal of Geophysical Research, 91(1312).

Working Group on California Earthquake Probabilities (WGCEP) (2003). "Summary of Earthquake Probabilities in the San Francisco Bay Region: 2002 to 2031." Open File Report 03-214.

Working Group on California Earthquake Probabilities (WGCEP) (2008). "The Uniform California Earthquake Rupture Forecast, Version 2." Open File Report 2007-1437.

Youd, T.L., and Idriss, I.M. (2001). "Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER/NSF Workshops on Evaluation of Liquefaction Resistance of Soils," Journal of Geotechnical and Geoenvironmental Engineering, Vol. 127, No. 4.

Youd, T.L., and Garris, C.T. (1995). "Liquefaction-induced ground-surface disruption." Journal of Geotechnical Engineering, American Society of Civil Engineers, Vol. 121, 805-809.

FIGURES

APPENDIX A

BORING LOGS

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APPENDIX B

LABORATORY TEST RESULTS

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APPENDIX C

DOWNHOLE SUSPENSION LOGGING

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APPENDIX D

SITE SPECIFIC RESPONSE SPECTRA

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