PRELIMINARY GEOTECHNICAL INVESTIGATION
469 Stevenson Street
San Francisco, California

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1.0 INTRODUCTION

This report presents the results of Langan Engineering and Environmental Services, Inc.'s (LANGAN) preliminary geotechnical investigation for the proposed development at 469 Stevenson Street in San Francisco, California (Site Location Map, Figure 1). At the request of the San Francisco Planning Department, this report includes references to AB-082[1] and AB-111[2], as appropriate.

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 buildings (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 (approximately 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. 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

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[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.

preliminary engineering analyses indicate a mat foundation is feasible for the support of the proposed structure. The feasibility of a mat foundation should be confirmed with a Phase 2 field investigation and laboratory testing program, and additional engineering analyses using the results of the Phase 2 investigation. If the supplemental field investigation and engineering analyses indicate a mat is not feasible, then deep foundations that extend to bedrock would be required to support the proposed structure. Accounting for 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. The actual mat thicknesses would be determined during foundation design by the project structural engineer.

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 and AB-111.

AB-082 presents guidelines and procedures for Structural, Geotechnical, and Seismic Hazard 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). Per AB-082:

> "If the director determines that review is required, the director shall request one or more Structural, Geotechnical, or Seismic Hazard reviewers having specialized knowledge and experience to provide their professional opinion on identified aspects of a project. The purpose of the review is to provide an independent, objective, technical review of those aspects of the project design that are identified in the scope of the review.
>
> The director shall require review for projects where review is required by the San Francisco Building Code. The director may require review for other projects at the director's discretion. Table 1, below, lists project characteristics commonly considered by the Director in determining whether Review is required."

TABLE 1

Project Characteristics Considered by the Director in Determining Whether Review is Required

	Review Discipline		
	Structural	**Geotechnical**	**Site-Specific Hazard**[e]
Projects that require review			
Projects where review is required by the SFBC [a,b,c]	X	X	X
Projects that typically require review			
Projects incorporating exception(s) to prescriptive requirements of the SFBC [c]	X	X	X
Projects incorporating materials, systems, or technologies that are not directly addressed by the SFBC [c]	X	X	X
Buildings with structural height (h_n as defined in ASCE 7) 240 feet or taller, including projects designed to the prescriptive provisions of the SFBC [d]	X	X	X
Projects that may require review, depending on size, occupant load, importance, and similar considerations [h]			
Addition or alteration of existing structures, where seismic retrofit is required by the SFEBC [f]	X	X	
Projects on Site Class F sites requiring site responses analysis		X	X
Projects on sites with mapped or potential geologic or seismic ground deformation hazards		X	X
Projects on sites with compressible soils below the foundation, having potential for long-term consolidation settlement under gravity loads [g]		X	
Projects using ground improvement or special foundation systems		X	X
Projects with dewatering that lowers groundwater by more than 10 feet, located adjacent to major structures or utilities		X	
Projects with below-grade excavation deeper than 15 feet, located adjacent to major structures or utilities		X	

Notes

[a] Ground Motion Review is required whenever response-history analysis is used.

[b] Where Review is required by the SFBC, such review process shall also conform to the specific requirements of the SFBC. The 2019 SFBC references ASCE 7-16, which requires design review in Sections 16.5 (Seismic Response History Procedures), 17.7 (Seismically Isolated Structures), and 18.5 (Structures with Damping Systems)

[c] The Director shall determine which Review disciplines are required based on which disciplines relate to the code requirements, code exceptions, or technologies proposed for the project.

[d] All projects of new buildings 240 feet or taller located in the City's softest soils and/or liquefaction zones, as defined by the California Seismic Hazard Zone Map, released by the California Department of Conservation, Division of Mines and Geology, dated November 17, 2000, shall include two Geotechnical Reviewers on the Engineering Design Review Team unless the project will include piles/drilled piers anchored to bedrock. Only one Geotechnical Reviewer is required for a project that will anchor piles/piers to bedrock.

[e] Review of site-specific hazard is not required if the general (rather than site-specific) earthquake response spectrum is used.

[f] See commentary regarding Review of existing structures.

[g] Soils with potential for long-term consolidation settlement typically include normally to lightly overconsolidated clayey soils, such as Bay Mud and Old Bay Clay, though other soils may also exhibit such behavior.

[h] It is intended that most projects in this category would not require Review, except for major structures based on the list of considerations above this table.

> "Along with the characteristics in Table 1, the Director's determination of whether a project requires Review, and what Review disciplines are required, may depend on factors such as:
>
> - Size, importance, occupant load, post-earthquake functionality requirements, or risk category of the structure
> - Characteristics of the site, foundation system, and adjacent structures
> - Irregular or unusual structural configurations
> - Pertinent qualifications within SFDBI to conduct an in-house review"

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 (including Administrative Bulletins), and Department of Building Inspection (DBI) implementing procedures 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.

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; and
- 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; and
- 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 buildings (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 by 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.

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[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.

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 marsh 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 Colma Formation extends to depths of 98 and 114.5 feet bgs, about Elevation -69 and -84.5 feet. A 2-foot-thick medium stiff 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

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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.

close to the San Francisco Bay to be influenced by future sea level rise; however, it is not within the San Francisco Sea Level Rise Vulnerability zone (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

According to the 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 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 environmental review and project approvals, the City uses the NRC's most likely SLR projection of 36 inches of sea level rise by 2100. 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

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[5] City and County of San Francisco. February 2020. Sea level rise Vulnerability and Consequences Assessment. Online https://sfplanning.org/sea-level-rise-action-plan#info. Accessed July 1, 2022.

[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.

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
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 the next 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. 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

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[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.

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; and
- 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;

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[10] Settle3, version 4.023

- 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); and
- 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; and
- Overconsolidation ratio (OCR) of 1.8, and recompression ratio of 0.05 in the stress range of interest, for Old Bay Clay; and
- 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); and
- 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). Per AB-111, "*the total short-term and long-term computed settlement of the foundation under gravity and seismic loads should not exceed 4 inches.*" 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; and
- 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; and
- 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 would 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. 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. Recommendations for the monitoring program should be included in the design level geotechnical investigation report. The monitoring –program should be included in the shoring documents. During construction, the GEOR would confirm the monitoring program is implemented in accordance with the GEOR’s recommendations. Pre-construction documentation and a monitoring program are standard construction practices.

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 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 (see Section 7.4 for recommended shoring wall systems), we anticipate dewatering would be required only within the site 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.

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. During construction, the geotechnical engineer would confirm the monitoring program is implemented per the geotechnical engineer's recommendations. . 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.

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

> <u>Slope indicators</u>: 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.
>
> <u>Piezometers</u>: We recommend installing a piezometer on Stevenson and Jessie Streets behind the shoring walls.
>
> <u>Survey points</u>: 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. Per Sections 7.5 and 7.7, during construction, LANGAN as the project geotechnical engineer would review the data from the monitoring program.

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.

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 LANGAN's design level geotechnical investigation would be predicated upon LANGAN reviewing the final design and providing construction observation services for the owner.

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FIGURES

Note:
Topographic base map is provided through Langan's Esri
Arc GIS software licensing and Arc GIS online,
National Geographic Society, i-cubed.

0 1,000 Feet

Approximate scale

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Project

469
STEVENSON STREET

SAN FRANCISCO
SAN FRANCISCO COUNTY CALIFORNIA

Figure Title

SITE
LOCATION MAP

Project No.
731690402

Date
04/01/2021

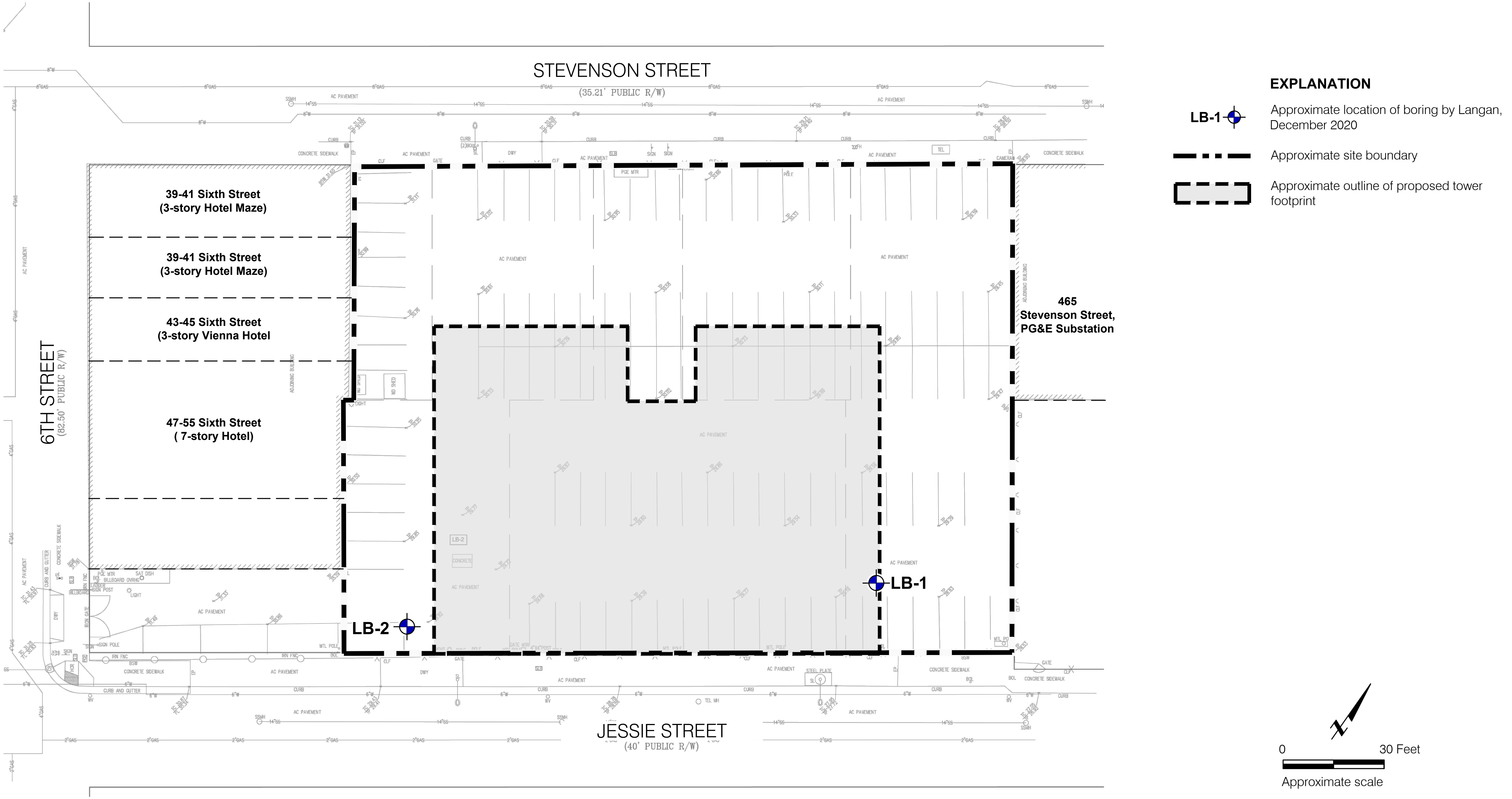
Drawn By
AG

Checked By
PB

Figure
1

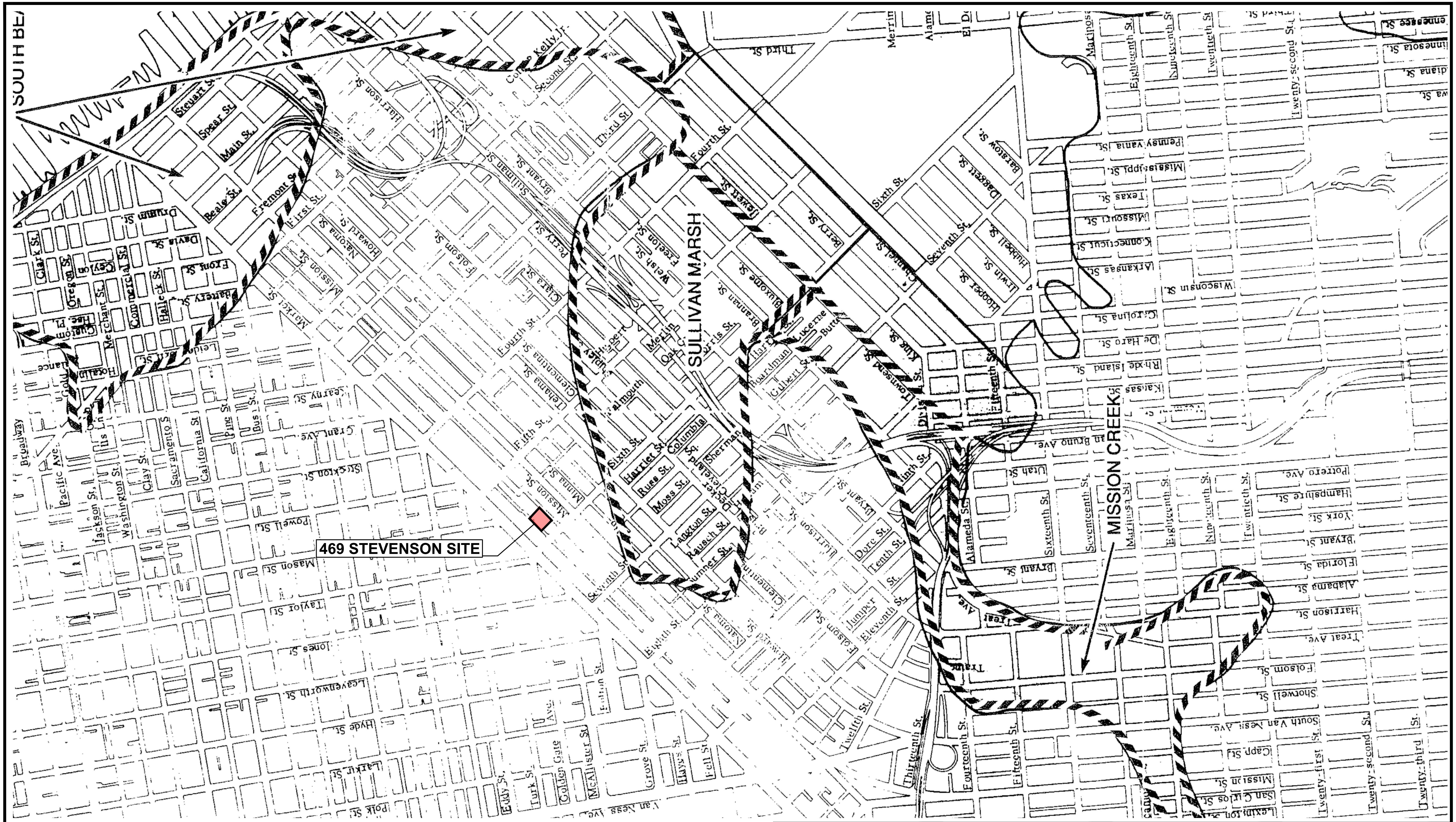
© 2019 Langan

Filename: \\langan.com\data\SF\data4\731690402\Project Data\CAD\02\2D-DesignFiles\Geotechnical\731690402-B-GI0101_SiteLocMap-FaultMap-MM Scale.dwg Date: 3/17/2022 Time: 10:03 User: agekas Style Table: Langan.stb Layout: Figure 1 Site Loc



Reference: 2016 Microsoft Corporation, Bing. Topographic base map by Luk and Associates titled :Topographic & Boundary Survery" , dated August, 2018.
Elevations based on old San Francisco City Datum.

LANGAN Langan Engineering and Environmental Services, Inc. 135 Main Street, Suite 1500 San Francisco, CA 94105 T: 415.955.5200 F: 415.955.5201 www.langan.com	Project **469 STEVENSON STREET** SAN FRANCISCO SAN FRANCISCO COUNTY CALIFORNIA	Figure Title **SITE PLAN**	Project No. 731690402 Date 04/01/2021 Drawn By AG Checked By PB	Figure **2**



EXPLANATION

🟥 Site location

Reference: Base map by Harding Lawson Associates, Dames & Moore, Kennedy/Jenks/Chilton, and EQE Engineering titled "Map of Study Areas Liquefaction Study" dated 5/91.

0 1,000 Feet

Approximate scale

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Project

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SAN FRANCISCO

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Figure Title

SITE LOCATION AND SULLIVAN MARSH

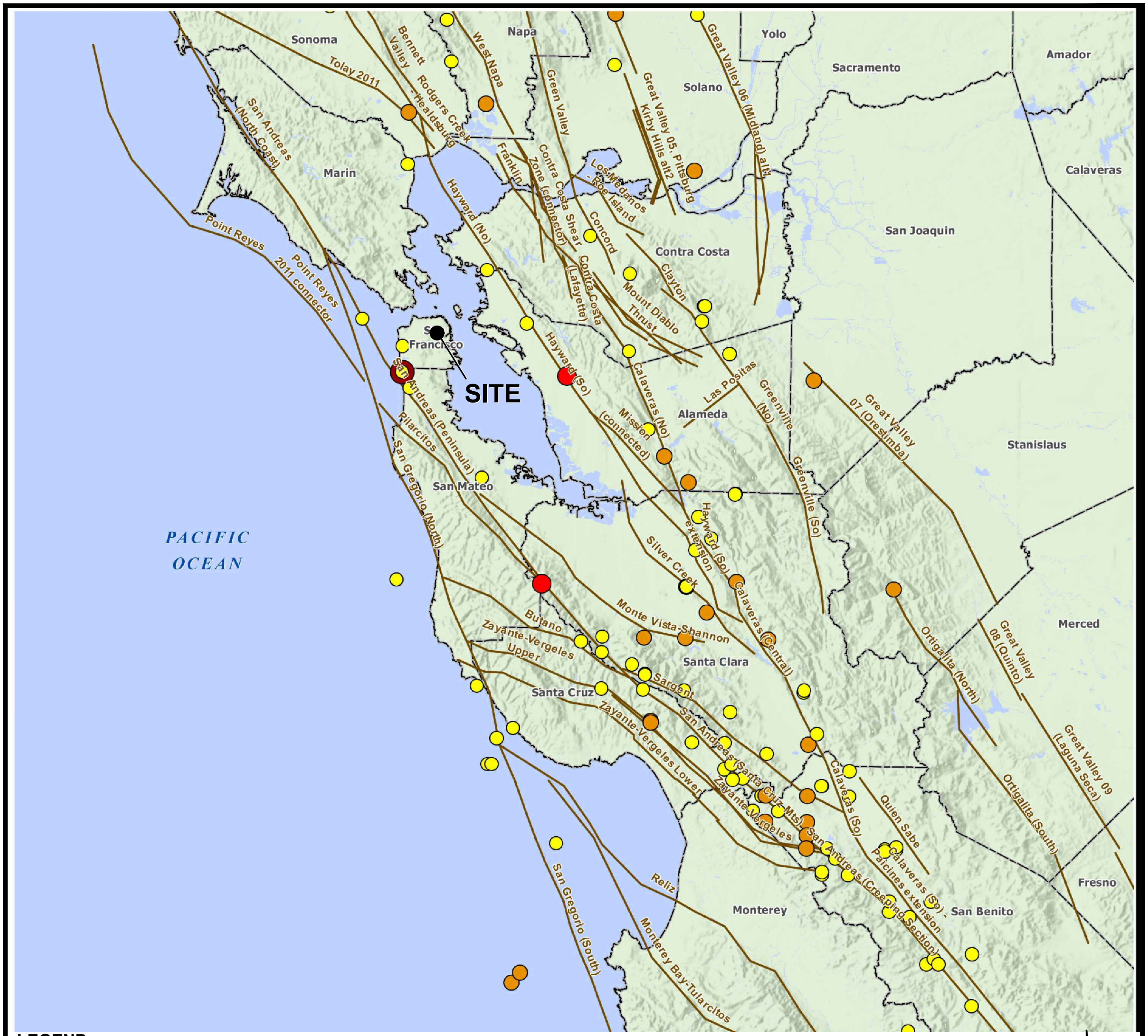
Project No. 731690402

Date 04/01/2021

Drawn By AG

Checked By PB

Figure **3**



LEGEND

⬚ County Boundary

— Fault

Earthquake Epicenter Magnitude

- ● Magnitude 5 to 5.9
- ● Magnitude 6 to 6.9
- ● Magnitude 7 to 7.4
- ● Magnitude 7.5 to 8

0 20 Miles

Approximate scale

Notes:

1. The fault line work data shows fault-based seismic sources used in the time-independent component of the Uniform California Earthquake Rupture Forecast, Version 3 (UCERF3), which provides authoritative estimates of the magnitude, location, and time-averaged frequency of potentially damaging earthquakes in California. Fault model slip rates are given in millimeters per year. The data depicts the surface traces of modeled faults, which are simplified from the CGS - USGS Quaternary Fault and Fold database. For cartographic purposes only.

2. The Earthquake Epicenter (Magnitude) data is provided by the U.S Geological Survey (USGS) and is current through 2015.

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.

LANGAN Langan Engineering and Environmental Services, Inc. 135 Main Street, Suite 1500 San Francisco, CA 94105 T: 415.955.5200 F: 415.955.5201 www.langan.com	Project 469 STEVENSON STREET SAN FRANCISCO SAN FRANCISCO COUNTY CALIFORNIA	Figure Title MAP OF MAJOR FAULTS AND EARTHQUAKE EPICENTERS IN THE SAN FRANCISCO BAY AREA	Project No. 731690402 Date 03/17/2022 Drawn By AG Checked By PB	Figure 4

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.

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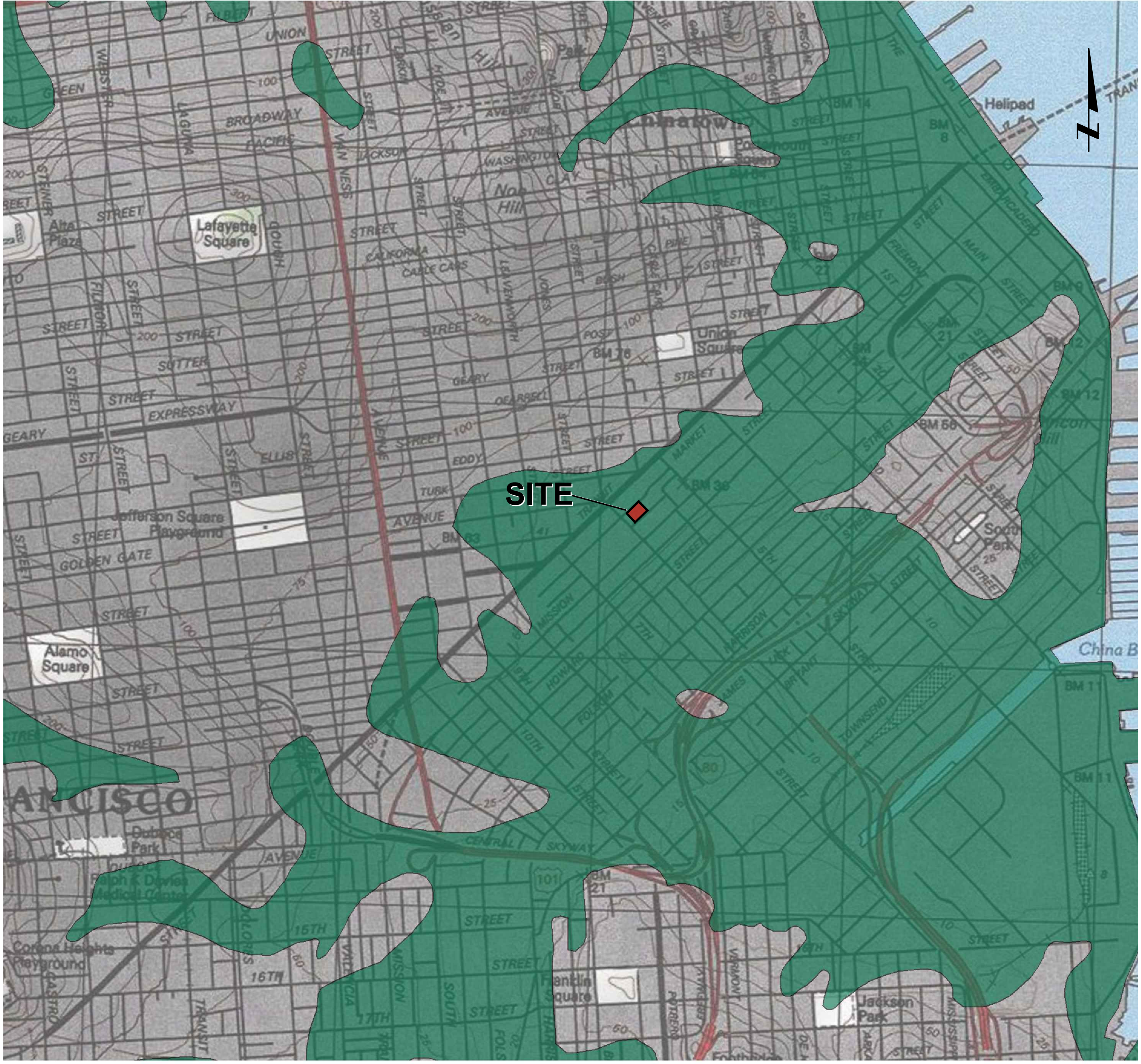
Project
469
STEVENSON STREET
SAN FRANCISCO
SAN FRANCISCO COUNTY CALIFORNIA

Figure Title
MODIFIED MERCALLI
INTENSITY SCALE

Project No.
731690402
Date
03/17/2022
Drawn By
AG
Checked By
PB

Figure
5



EXPLANATION

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

Notes:
1. Topographic basemap is provided through Langan's Esri Arc GIS software licensing and Arc GIS online, National Geographic Society, i-cubed and the USGS.
2. Data provided by the CGS through the GIS Seismic Hazard Zone Map presenting areas where liquefaction and land slides may occur during a strong earthquake.

0 2000 Feet

Approximate scale

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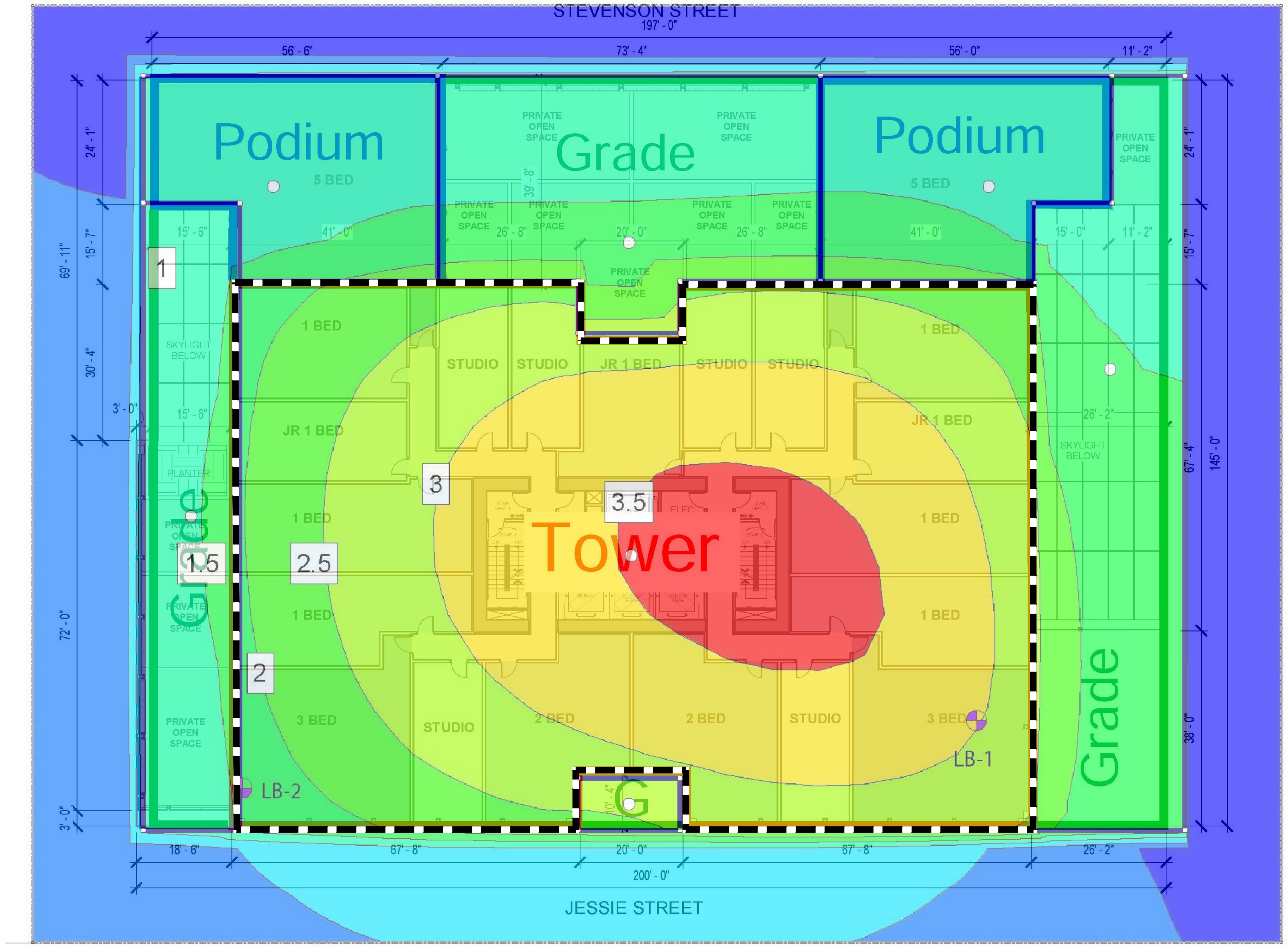
Project	Figure Title	Project No.	Figure
469 STEVENSON STREET SAN FRANCISCO SAN FRANCISCO COUNTY CALIFORNIA	REGIONAL SEISMIC HAZARD ZONES MAP	731690402	6
		Date 03/17/2022	
		Drawn By AG	
		Checked By PB	



Total Settlement (in)

- 0.0
- 0.5
- 1.0
- 1.5
- 2.0
- 2.5
- 3.0
- 3.5
- 4.0

EXPLANATION

▬ ▬ ▬ Approximate outline of proposed tower footprint

Note:
Average podium pressure applied at bottom of mat at Elevation -17 feet Old SF City datum, and average tower foundation pressured applied at the bottom of the mat at Elevation -23 feet Old SF City datum

Reference:
1. Preliminary settlement contours generated using Settle3D program, version 4.023, on July 2021.
2. Average foundation pressures for dead plus live loads provided by MKA via email dated 2 July 2021
    - Average Tower Foundation Pressure: 7,040 psf
    - Average Podium Foundation Pressure: 2,860 psf
    - Average One-Level Podium (Labeled as Grade in Figure) Foundation Pressure: 1,760 psf.

0 30 Feet
Approximate scale

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Project
469 STEVENSON STREET
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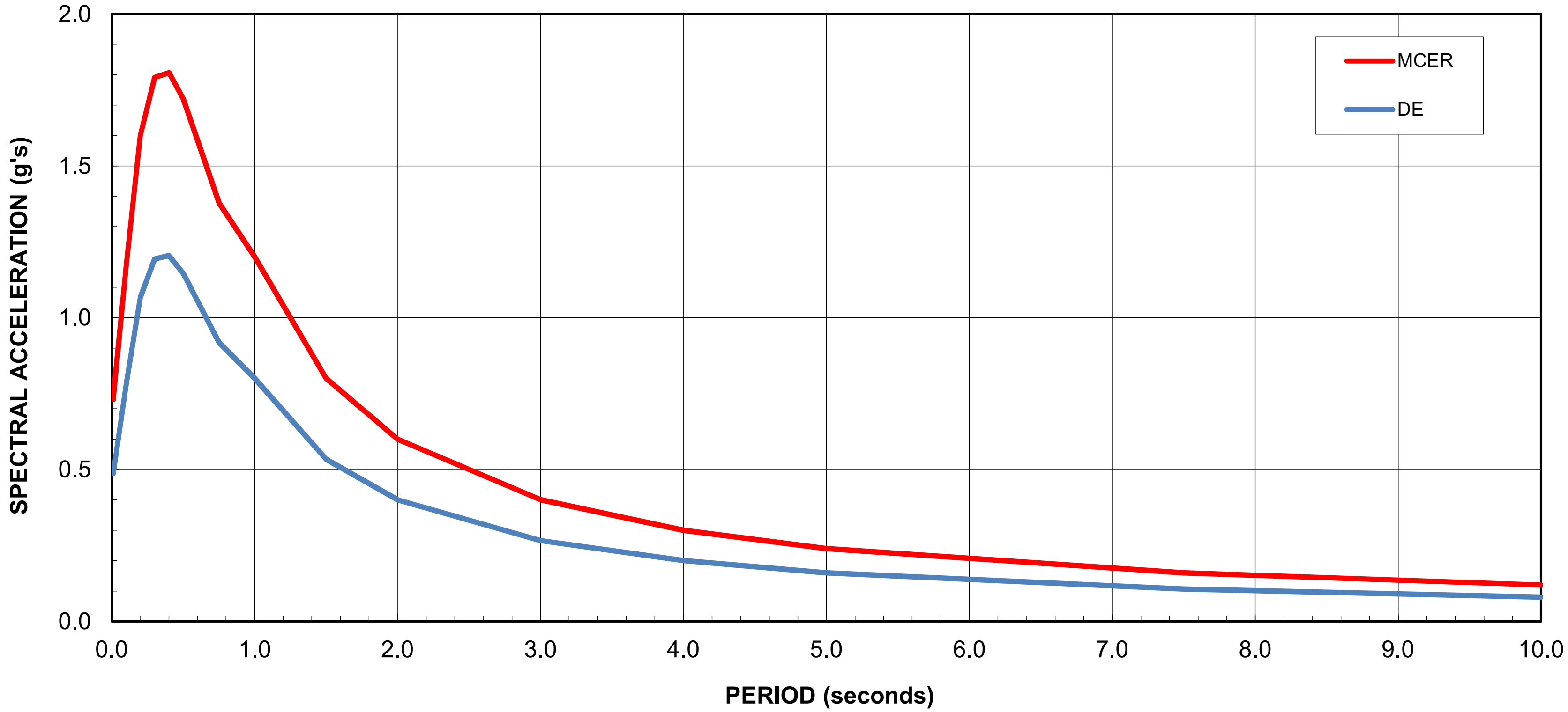
SAN FRANCISCO COUNTY CALIFORNIA

Figure Title
PRELIMINARY ESTIMATED 50 YEAR SETTLEMENT CONTOURS FOR 3 BASEMENT LEVELS

Project No.	731690402
Date	03/17/2022
Drawn By	AG
Checked By	PB
Figure	7

Filename: \\langan.com\data\SF\data4\731690402\Project Data\CAD\02\2D-DesignFiles\Geotechnical\731690402-B-SP0102.dwg Date: 4/14/2022 Time: 13:42 User: agekas Style Table: Langan.stb Layout: Figure 7



Notes

1. Damping Ratio = 5%
2. Estimated V_{S30} = 330 m/s

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Project

469 Stevenson Street

SAN FRANCISCO
CITY & COUNTY CALIFORNIA

Figure Title

RECOMMENDED HORIZONTAL SPECTRA

Project No.
731690402
Date
8/20/2021
Scale
AS SHOWN
Prepared By:
PDB

Figure No.
8

APPENDIX A

BORING LOGS

PROJECT: **469 STEVENSON STREET**
San Francisco, California

Log of Boring LB-1

PAGE 1 OF 9

Boring location: See Site Plan, Figure 2
Date started: 12/16/20
Date finished: 12/21/20
Drilling method: Rotary Wash, Failing 1500
Hammer weight/drop: 140 lbs./30 inches
Hammer type: Automatic
Samplers: Sprague & Henwood (S&H), Standard Penetration Test (SPT), Shelby Tube (ST)

Logged by: R. Nelson
Drilled By: Pitcher Services LLC

DEPTH (feet)	SAMPLES Sampler Type	Sample	Blows/ 6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION Ground Surface Elevation: 29 feet[2]	LABORATORY TEST DATA Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
						5 inches asphalt concrete (AC)						
1						4 inches aggregate base (AB)						
2						SAND with SILT (SP-SM) brown, medium dense, moist, fine to medium rounded to subrounded gravel, fine sand, concrete, brick [FILL]						
3	BAG	☐										
4					SP-SM							
5												
6	S&H	◩	6 8 11	13								
7												
8						SAND (SP) yellow-brown, dense, moist, fine sand [DUNE SAND]						
9												
10												
11	SPT	◩	9 13 14	32								
12												
13												
14												
15						medium dense						
16	SPT	◩	5 11 13	29								
17					SP							
18	SPT	◩	6 11 17	34		dense						
19						▽ (12/17/20, 7:00 AM)						
20						yellow-brown, dense, wet						
21	SPT	◩	8 12 16	34								
22												
23	SPT	◩	8 15 19	47								
24												
25												
26												
27						CLAYEY SAND (SC) yellow-brown, medium dense, wet, fine sand [MARSH DEPOSIT]						
28												
29	ST	◨		80 to 250 psi	SC							
30												

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Project No.: 731690402
Figure: A-1a

PROJECT: **469 STEVENSON STREET**
San Francisco, California

Log of Boring LB-1

DEPTH (feet)	SAMPLES: Sampler Type	SAMPLES: Sample	SAMPLES: Blows/ 6"	SAMPLES: SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	LABORATORY TEST DATA: Type of Strength Test	LABORATORY TEST DATA: Confining Pressure Lbs/Sq Ft	LABORATORY TEST DATA: Shear Strength Lbs/Sq Ft	LABORATORY TEST DATA: Fines %	LABORATORY TEST DATA: Natural Moisture Content, %	LABORATORY TEST DATA: Dry Density Lbs/Cu Ft
31						CLAYEY SAND (SC) (continued)						
32						▽ mottled orange-brown and gray-brown, medium dense, wet, fine sand (12/21/20, 6:30 AM) LL= 22, PI = 8, see Appendix B increase in clay content				30.4	18.4	115
33	S&H	■	4 8 12	14								
34	SPT	◪	4 6 12	22	SC							
35												
36												
37						mottled orange-brown and gray-brown, dense, wet				13.9	21.5	109
38	S&H	■	15 24 29	37		SAND with Silt (SP-SM) yellow-brown and orange-brown, very dense, wet, fine sand [COLMA FORMATION]						
39												
40	SPT	◪	12 23 35	70								
41												
42												
43					SP-SM							
44						dense				10.1	20.3	
45	SPT	◪	18 20 21	49								
46												
47												
48						CLAYEY SAND (SC) yellow-brown, dense, wet [COLMA FORMATION]						
49												
50	SPT	◪	7 15 18	40	SC					17.0	20.1	
51												
52												
53						SAND WITH SILT (SP-SM) olive, dense, wet, fine sand [COLMA FORMATION]						
54												
55	SPT	◪	12 14 24	46						8.8	19.6	
56												
57					SP-SM							
58												
59						gray-brown and red, very dense						
60	SPT	◪		79								

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Project No.: 731690402
Figure: A-1b

PROJECT: **469 STEVENSON STREET**
San Francisco, California

Log of Boring LB-1

PAGE 3 OF 9

DEPTH (feet)	SAMPLES Sampler Type	 Sample	 Blows/ 6"	 SPT N-Value¹	LITHOLOGY	MATERIAL DESCRIPTION	LABORATORY TEST DATA Type of Strength Test	 Confining Pressure Lbs/Sq Ft	 Shear Strength Lbs/Sq Ft	 Fines %	 Natural Moisture Content, %	 Dry Density Lbs/Cu Ft
	SPT		28 31 35	79		SAND WITH SILT (SP-SM) (continued)						
61												
62												
63												
64										9.4	20.6	
65	SPT		25 34 46	96								
66												
67												
68												
69						yellow-brown						
70	SPT		19 30 32	74								
71												
72												
73												
74						yellow and orange-brown						
75	SPT		20 24 34	70	SP-SM							
76												
77												
78												
79												
80												
81												
82												
83												
84	SPT		31 50/ 6"	97/ 6"								
85												
86												
87												
88												
89						trace clay						
90	SPT			96								

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Project No.: 731690402

Figure: A-1c

PROJECT: **469 STEVENSON STREET**
San Francisco, California

Log of Boring LB-1

DEPTH (feet)	SAMPLES: Sampler Type	SAMPLES: Sample	SAMPLES: Blows/ 6"	SAMPLES: SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	LABORATORY TEST DATA: Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
90	SPT		31 38 42	96		SAND WITH SILT (SP-SM) (continued)						
91												
92												
93												
94	ST			400 psi	SP-SM	dark gray						
95	S&H		32 50/ 5"	57/ 11"								
96												
97												
98						CLAY (CH) gray, medium stiff, wet, with fine sand [OLD BAY CLAY]						
99												
100				60 - 100 - 200 - 250 psi								
101	ST					Triaxial Test, see Appendix B	TxUU	10,000	2,330		43.9	78
102							PP		1,700			
103												
104												
105												
106	S&H		5 9 13	15		medium stiff to stiff						
107												
108												
109					CH							
110											51.9	69
111	ST			50 to 250 psi		Consolidation Test, see Appendix B Triaxial Test, see Appendix B	TxUU	11,000	3,160		52.6	71
112							PP		2,500			
113												
114												
115												
116	S&H		0 3 7	7		medium stiff	PP		1,700			
117												
118												
119												
120												

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Project No.: 731690402

Figure: A-1d

PROJECT: **469 STEVENSON STREET**
San Francisco, California

Log of Boring LB-1

DEPTH (feet)	SAMPLES: Sampler Type	SAMPLES: Sample	SAMPLES: Blows/ 6"	SAMPLES: SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	LABORATORY TEST DATA: Type of Strength Test	LABORATORY TEST DATA: Confining Pressure Lbs/Sq Ft	LABORATORY TEST DATA: Shear Strength Lbs/Sq Ft	LABORATORY TEST DATA: Fines %	LABORATORY TEST DATA: Natural Moisture Content, %	LABORATORY TEST DATA: Dry Density Lbs/Cu Ft
121	ST			60 to 200 psi		CLAY (CH) (continued) Consolidation Test, see Appendix B Triaxial Test, see Appendix B	TxUU PP	12,000	2,340 2,500		56.2 53.3	66 70
122												
123												
124												
125						medium stiff, wet						
126	S&H		0 2 10	8			PP		2,000			
127					CH							
128												
129												
130												
131	ST			40 to 180 psi		Triaxial Test, see Appendix B	TxUU	13,000	3,100		46.0	74
132												
133												
134												
135						SAND with CLAY (SP-SC) gray, very dense, wet, fine sand [ALLUVIUM]						
136	S&H		20 30 48	55								
137												
138												
139					SP-SC							
140						yellow-brown						
141	SPT		13 24 48	91								
142												
143						SANDY CLAY (CL) light brown, hard, wet, fine sand [ALLUVIUM]						
144												
145					CL							
146	S&H		15 25 35	39								
147												
148						CLAYEY SAND (SC) yellow-brown, very dense, wet, fine sand [ALLUVIUM]						
149					SC							
150												

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Project No.: 731690402

Figure: A-1e

PROJECT: **469 STEVENSON STREET**
San Francisco, California

Log of Boring LB-1

PAGE 6 OF 9

DEPTH (feet)	SAMPLES: Sampler Type	SAMPLES: Blows/ 6"	SAMPLES: SPT N-Value^1	LITHOLOGY	MATERIAL DESCRIPTION	LABORATORY TEST DATA: Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
	SPT	15 27 40	80		CLAYEY SAND (SC) (continued)						
151											
152											
153											
154											
155											
156											
157											
158											
159				SC							
160											
161	SPT	7 17 30	56								
162											
163											
164											
165											
166											
167											
168					SAND with SILT (SP-SM) yellow-brown, very dense, wet, fine sand, some silt [ALLUVIUM]						
169											
170	S&H	27 50/ 5"	54								
171	SPT	23 5/ 5"	88/ 11"								
172											
173											
174				SP- SM							
175											
176											
177											
178											
179											
180											

PROJECT: **469 STEVENSON STREET**
San Francisco, California

Log of Boring LB-1

PAGE 7 OF 9

DEPTH (feet)	SAMPLES: Sampler Type	SAMPLES: Sample	SAMPLES: Blows/ 6"	SAMPLES: SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	LABORATORY TEST DATA: Type of Strength Test	LABORATORY TEST DATA: Confining Pressure Lbs/Sq Ft	LABORATORY TEST DATA: Shear Strength Lbs/Sq Ft	LABORATORY TEST DATA: Fines %	LABORATORY TEST DATA: Natural Moisture Content, %	LABORATORY TEST DATA: Dry Density Lbs/Cu Ft
	SPT	◪	30/ 5.5	96/ 11"		SAND with SILT (SP-SM) (continued) SAND (SP)						
181												
182												
183												
184					SP-SM							
185												
186												
187												
188												
189						CLAY (CH) gray, hard, wet [ALLUVIUM]						
190												
191	SPT	◪	11 15 18	40								
192												
193												
194												
195					CH	Consolidation Test, see Appendix B Triaxial Test, see Appendix B						
196	ST	◨		60 to 250 psi			TxUU	19,500	2,200		43.4 42.5	77 80
197												
198												
199												
200												
201												
202						SANDY CLAY with GRAVEL (CL) mottled yellow-brown and olive, hard, wet, fine sand, fine to coarse gravel composed of chert [ALLUVIUM]						
203	S&H	■	20 50/ 6"	49/ 12"								
204												
205												
206					CL							
207												
208												
209												
210												

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Project No.: 731690402
Figure: A-1g

TEST GEOTECH LOG 731690402 STEVENSON.GPJ TEMPLATE_CA-MODIFIED - COPY.GDT 4/12/21

PROJECT: **469 STEVENSON STREET**
San Francisco, California

Log of Boring LB-1

DEPTH (feet)	SAMPLES: Sampler Type	SAMPLES: Sample	SAMPLES: Blows/ 6"	SAMPLES: SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	LABORATORY TEST DATA: Type of Strength Test	LABORATORY TEST DATA: Confining Pressure Lbs/Sq Ft	LABORATORY TEST DATA: Shear Strength Lbs/Sq Ft	LABORATORY TEST DATA: Fines %	LABORATORY TEST DATA: Natural Moisture Content, %	LABORATORY TEST DATA: Dry Density Lbs/Cu Ft
						SANDY CLAY with GRAVEL (CL) (continued)						
211												
212												
213												
214												
215												
216												
217												
218												
219												
220		▨		100 - 220 - 300 - 400 psi								
221	ST	▨										
222												
223												
224												
225					CL							
226												
227												
228												
229												
230		▨	20			gray, some fine angular grave, composed of siltstone,						
231	S&H	▨	31 38	48		trace fine sand	PP		>4,500			
232												
233												
234												
235												
236												
237												
238												
239												
240												

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Project No.: 731690402

Figure: A-1h

PROJECT: **469 STEVENSON STREET**
San Francisco, California

Log of Boring LB-1

	SAMPLES						LABORATORY TEST DATA					
DEPTH (feet)	Sampler Type	Sample	Blows/ 6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
241	S&H	■	20 39 50/ 6"	48		SANDY CLAY with GRAVEL (CL) (continued) mottled light brown and orange-brown, trace fine angular gravel [ALLUVIUM]						
242												
243												
244					CL							
245												
246												
247												
248												
249						SANDSTONE dark gray, close to intensely fractured, low hardness, friable, little weathered, clay infilled fractures						
250												
251												
252	SPT	▭	50/ 1"									
253												
254												
255												
256	SPT	▭	50/ 0.5"									
257												
258												
259												
260												
261												
262												
263												
264												
265												
266												
267												
268												
269												
270												

Boring terminated at a depth of 256.1 feet below ground surface (bgs).
Boring backfilled with cement grout.
Groundwater observed at a depth of 19.5 feet and of 32 feet.
PP = pocket penetrometer.

[1] S&H and SPT blow counts for the last two increments were converted to SPT N-Values using factors of 0.7 and 1.2, respectively to account for sampler type and hammer energy.

[2] Elevations based on old San Francisco City Datum.

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Project No.: 731690402

Figure: A-1i

PROJECT: **469 STEVENSON STREET**
San Francisco, California

Log of Boring LB-2

PAGE 1 OF 9

Boring location:	See Site Plan, Figure 2		
Date started:	12/21/20	Date finished:	12/23/20
Drilling method:	Rotary Wash, Failing 1500		
Hammer weight/drop:	140 lbs./30 inches	Hammer type:	Automatic
Samplers:	Sprague & Henwood (S&H), Standard Penetration Test (SPT), Shelby Tube (ST)		

Logged by: R. Nelson
Drilled By: Pitcher Services LLC

DEPTH (feet)	SAMPLES				LITHOLOGY	MATERIAL DESCRIPTION	LABORATORY TEST DATA					
	Sampler Type	Sample	Blows/ 6"	SPT N-Value^1		Ground Surface Elevation: 30 feet^2	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
						4.5 inches asphalt concrete (AC)						
1						3 inches aggregate base (AB)						
2					SP	SAND (SP) dark brown, moist, rubble, brick fragments [FILL]						
3												
4												
5					SM	SILTY SAND (SM) dark brown, very loose, moist, coarse angular gravel, rubble, brick fragments [FILL]						
6	S&H	▨	1 1 2	2								
7												
8												
9					SP	SAND (SP) yellow-brown, loose, moist, fine sand [DUNE SAND]						
10												
11	SPT	▨	2 3 4	8								
12												
13	SPT	▨	1 2 3	6								
14												
15												
16	SPT	▨	1 2 5	8								
17												
18	SPT	▨	4 7 10	20	SP	medium dense						
19												
20												
21	SPT	▨	3 5 8	16		medium dense						
22												
23	SPT	▨	2 6 12	22								
24						∇ (12/23/20, 6:20 PM)						
25						dark brown, wet						
26	SPT	▨	3 5 8	15								
27												
28												
29					CL	SANDY CLAY (CL) dark brown, medium stiff, wet, fine sand [MARSH DEPOSIT]						
30												

TEST GEOTECH LOG 731690402 STEVENSON.GPJ TEMPLATE_CA-MODIFIED - COPY.GDT 4/12/21

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Project No.: 731690402
Figure: A-2a

	SAMPLES						LABORATORY TEST DATA					
DEPTH (feet)	Sampler Type	Sample	Blows/ 6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
		◧		50 to 185 psi		SANDY CLAY (CL) (continued)						
31	ST	◧										
32		◧			CL							
33												
34												
35		■				CLAYEY SAND (SC) gray and yellow-brown, medium dense, wet, fine sand [MARSH DEPOSIT] LL = 25, PI = 9, see Appendix B						
36	S&H	■	5 8 11	13	SC					23.1	18.1	115
37						SILTY SAND (SM) yellow-brown, dense, wet, fine sand [COLMA FORMATION] LL = 22, PI = 8, see Appendix B						
38	S&H	■	14 19 29	34						20.6	19.7	110
39		◪	9									
40	SPT	◪	13 20	40	SM							
41												
42												
43						SAND with SILT (SP-SM) yellow-brown to brown, very dense, wet, fine-grained [COLMA FORMATION]						
44		◪	16								11.1	20.3
45	SPT	◪	25 24	65	SP- SM							
46												
47												
48						SILTY SAND (SM) gray-brown, very dense, wet, fine sand [COLMA FORMATION]						
49		◪	14								13.4	19.9
50	SPT	◪	22 33	55								
51												
52												
53												
54		◪	35		SM	olive-brown						
55	SPT	◪	35 38	88								
56												
57												
58												
59												
60												

DEPTH (feet)	SAMPLES: Sampler Type	SAMPLES: Sample	SAMPLES: Blows/ 6"	SAMPLES: SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	LABORATORY TEST DATA: Type of Strength Test	LABORATORY TEST DATA: Confining Pressure Lbs/Sq Ft	LABORATORY TEST DATA: Shear Strength Lbs/Sq Ft	LABORATORY TEST DATA: Fines %	LABORATORY TEST DATA: Natural Moisture Content, %	LABORATORY TEST DATA: Dry Density Lbs/Cu Ft
					SP-SM	SAND with SILT (SP-SM) gray-brown, very dense, we, find sand				9.7	17.6	
61	SPT	◩	22 37 33	84								
62												
63												
64												
65												
66												
67												
68												
69												
70												
71	SPT	◩	18 30 40	84								
72												
73												
74												
75												
76												
77												
78												
79												
80												
81	SPT	◩	16 31 37	82								
82												
83												
84												
85												
86												
87												
88												
89					CL	CLAY (CL) gray, medium stiff, wet [COLMA FORMATION]						
90												

PROJECT: **469 STEVENSON STREET**
San Francisco, California

Log of Boring LB-2

DEPTH (feet)	SAMPLES: Sampler Type	SAMPLES: Sample	SAMPLES: Blows/ 6"	SAMPLES: SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	LABORATORY TEST DATA: Type of Strength Test	LABORATORY TEST DATA: Confining Pressure Lbs/Sq Ft	LABORATORY TEST DATA: Shear Strength Lbs/Sq Ft	LABORATORY TEST DATA: Fines %	LABORATORY TEST DATA: Natural Moisture Content, %	LABORATORY TEST DATA: Dry Density Lbs/Cu Ft
	S&H	■	19 36 43	55	CL	CLAY (CL) (continued)	PP		1,500			
91						SAND (SP) gray, very dense, wet, fine-grained [COLMA FORMATION]						
92	SPT	◩	21 32 43	90								
93					SP							
94												
95						CLAYEY SAND (SC) gray, very dense, wet, fine sand [COLMA FORMATION]						
96	S&H	■	18 25 50	53								
97												
98												
99												
100												
101												
102	ST	◨		80 to 400 psi								
103												
104												
105					SC							
106												
107	ST	◨		40 to 200 psi								
108												
109												
110												
111						dense						
112	S&H	■	26 28 28	39								
113												
114												
115						CLAY (CH) gray, medium stiff, wet, [OLD BAY CLAY]						
116												
117	S&H	■	0 4 10	10	CH		PP		1,750			
118												
119												
120												

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Figure: A-2d

PROJECT: **469 STEVENSON STREET**
San Francisco, California

Log of Boring LB-2

DEPTH (feet)	SAMPLES: Sampler Type	SAMPLES: Sample	SAMPLES: Blows/ 6"	SAMPLES: SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	LABORATORY TEST DATA: Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
121	ST			40 to 250 psi		CLAY (CH) (continued) Consolidation Test, see Appendix B Triaxial Test, see Appendix B	TxUU PP	12,000	2,290 2,300		55.4 54.2	67 69
122												
123												
124												
125												
126	S&H		1 2 6	6			PP.		1,500			
127												
128												
129					CH							
130				50 - 80 - 150 - 300 psi		very stiff Consolidation Test, see Appendix B Triaxial Test, see Appendix B						
131	ST						TxUU PP	13,000	1,370 2,750		52.2 51.2	69 72
132												
133												
134												
135												
136	S&H		0 4 13	12			PP		2,300			
137												
138												
139						SAND (SP) gray, dense, wet [ALLUVIUM]						
140												
141	SPT		11 15 20	42	SP							
142												
143												
144												
145						SAND with CLAY (SP-SC) orange-brown and red-brown, very dense, wet, fine sand, some clay [ALLUVIUM]						
146	S&H		13 25 36	43								
147					SP-SC							
148												
149												
150												

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Project No.: 731690402

Figure: A-2e

PROJECT: **469 STEVENSON STREET**
San Francisco, California

Log of Boring LB-2

	SAMPLES						LABORATORY TEST DATA					
DEPTH (feet)	Sampler Type	Sample	Blows/ 6"	SPT N-Value^1	LITHOLOGY	MATERIAL DESCRIPTION	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
151	SPT	◪	8 15 31	55		SAND with CLAY (SP-SC) (continued) very dense						
152												
153												
154												
155												
156												
157												
158												
159												
160												
161	SPT	◪	13 20 28	58								
162												
163												
164												
165					SP-SC							
166												
167												
168												
169												
170												
171	SPT	◪	20 25 50/ 5"	90/ 11"								
172												
173												
174												
175												
176												
177												
178												
179												
180												

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Project No.: 731690402

Figure: A-2f

PROJECT: **469 STEVENSON STREET**
San Francisco, California

Log of Boring LB-2

DEPTH (feet)	SAMPLES: Sampler Type	SAMPLES: Sample	SAMPLES: Blows/ 6"	SAMPLES: SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	LABORATORY TEST DATA: Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
	S&H		50/ 6"	35/ 6"		SAND with CLAY (SP-SC) (continued)						
181	SPT		23 50/ 5"	88								
182												
183												
184												
185												
186												
187												
188												
189					SP-SC							
190			19									
191	SPT		31 28	71								
192												
193												
194												
195												
196												
197												
198						CLAY with GRAVEL (CL) green and orange-brown, very stiff to hard, some fine angular gravel (chert) [ALLUVIUM]						
199												
200												
201	ST			0 to 400 psi								
202												
203												
204					CL							
205												
206												
207												
208												
209												
210												

PROJECT: **469 STEVENSON STREET**
San Francisco, California

Log of Boring LB-2

DEPTH (feet)	SAMPLES: Sampler Type	SAMPLES: Sample	SAMPLES: Blows/ 6"	SAMPLES: SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	LABORATORY TEST DATA: Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
						CLAY (CL) (continued)						
211	S&H	▨	16 26 30	39								
212												
213												
214												
215												
216												
217												
218												
219												
220												
221	S&H	▨	16 29 38	47								
222					CL							
223												
224												
225												
226												
227												
228												
229												
230												
231												
232												
233												
234						CLAYEY GRAVEL (GC) brown, very dense, wet, with fine subangular and coarse sand [RESIDUAL SOIL]						
235	S&H	▨	25 50/ 6"	53/ 12"								
236												
237					GC							
238												
239												
240												

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Figure: A-2h

PROJECT: **469 STEVENSON STREET**
San Francisco, California

Log of Boring LB-2

DEPTH (feet)	SAMPLES: Sampler Type	SAMPLES: Sample	SAMPLES: Blows/ 6"	SAMPLES: SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	LABORATORY TEST DATA: Type of Strength Test	LABORATORY TEST DATA: Confining Pressure Lbs/Sq Ft	LABORATORY TEST DATA: Shear Strength Lbs/Sq Ft	LABORATORY TEST DATA: Fines %	LABORATORY TEST DATA: Natural Moisture Content, %	LABORATORY TEST DATA: Dry Density Lbs/Cu Ft
						CLAYEY GRAVEL (GC) (continued)						
241					GC							
242												
243						SILTSTONE brown, closely fractured with clay infill, moderately hard, weak, little weathered [FRANCISCAN COMPLEX]						
244												
245	SPT	▨	50/ 4"	60/ 4"								
246												
247												
248												
249												
250	SPT	▨	50/ 3"	50/ 4"								
251												
252												
253												
254												
255												
256												
257												
258												
259												
260												
261												
262												
263												
264												
265												
266												
267												
268												
269												
270												

Boring terminated at a depth of 250.5 feet below ground surface (bgs).
Boring backfilled with cement grout.
Groundwater encountered at a depth of 24 feet.
PP = pocket penetrometer.

---

[1] S&H and SPT blow counts for the last two increments were converted to SPT N-Values using factors of 0.7 and 1.2, respectively to account for sampler type and hammer energy.

[2] Elevations based on old San Francisco City Datum.

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Project No.: 731690402
Figure: A-2i

UNIFIED SOIL CLASSIFICATION SYSTEM

Major Divisions		Symbols	Typical Names
Coarse-Grained Soils (more than half of soil > no. 200 sieve size	**Gravels** (More than half of coarse fraction > no. 4 sieve size)	**GW**	Well-graded gravels or gravel-sand mixtures, little or no fines
		GP	Poorly-graded gravels or gravel-sand mixtures, little or no fines
		GM	Silty gravels, gravel-sand-silt mixtures
		GC	Clayey gravels, gravel-sand-clay mixtures
	Sands (More than half of coarse fraction < no. 4 sieve size)	**SW**	Well-graded sands or gravelly sands, little or no fines
		SP	Poorly-graded sands or gravelly sands, little or no fines
		SM	Silty sands, sand-silt mixtures
		SC	Clayey sands, sand-clay mixtures
Fine -Grained Soils (more than half of soil < no. 200 sieve size)	**Silts and Clays** LL = < 50	**ML**	Inorganic silts and clayey silts of low plasticity, sandy silts, gravelly silts
		CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, lean clays
		OL	Organic silts and organic silt-clays of low plasticity
	Silts and Clays LL = > 50	**MH**	Inorganic silts of high plasticity
		CH	Inorganic clays of high plasticity, fat clays
		OH	Organic silts and clays of high plasticity
Highly Organic Soils		**PT**	Peat and other highly organic soils

GRAIN SIZE CHART

Classification	Range of Grain Sizes	
	U.S. Standard Sieve Size	Grain Size in Millimeters
Boulders	Above 12"	Above 305
Cobbles	12" to 3"	305 to 76.2
Gravel	3" to No. 4	76.2 to 4.76
coarse	3" to 3/4"	76.2 to 19.1
fine	3/4" to No. 4	19.1 to 4.76
Sand	No. 4 to No. 200	4.76 to 0.075
coarse	No. 4 to No. 10	4.76 to 2.00
medium	No. 10 to No. 40	2.00 to 0.420
fine	No. 40 to No. 200	0.420 to 0.075
Silt and Clay	Below No. 200	Below 0.075

▽	Unstabilized groundwater level
▼	Stabilized groundwater level

SAMPLE DESIGNATIONS/SYMBOLS

⬒	Sample taken with Sprague & Henwood split-barrel sampler with a 3.0-inch outside diameter and a 2.43-inch inside diameter. Darkened area indicates soil recovered
◪	Classification sample taken with Standard Penetration Test sampler
◨	Undisturbed sample taken with thin-walled tube
☒	Disturbed sample
⊡	Sampling attempted with no recovery
◫	Core sample
◙	Analytical laboratory sample
▥	Sample taken with Direct Push or Drive sampler
▥	Sonic

SAMPLER TYPE

C Core barrel

CA California split-barrel sampler with 2.5-inch outside diameter and a 1.93-inch inside diameter

D&M Dames & Moore piston sampler using 2.5-inch outside diameter, thin-walled tube

O Osterberg piston sampler using 3.0-inch outside diameter, thin-walled Shelby tube

PT Pitcher tube sampler using 3.0-inch outside diameter, thin-walled Shelby tube

S&H Sprague & Henwood split-barrel sampler with a 3.0-inch outside diameter and a 2.43-inch inside diameter

SPT Standard Penetration Test (SPT) split-barrel sampler with a 2.0-inch outside diameter and a 1.5-inch inside diameter

ST Shelby Tube (3.0-inch outside diameter, thin-walled tube) advanced with hydraulic pressure

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Project
469
STEVENSON STREET
SAN FRANCISCO
SAN FRANCISCO COUNTY CALIFORNIA

Figure Title
SOIL CLASSIFICATION CHART

Project No. 731690402
Date 01/15/2021
Drawn By AG
Checked By PB

Figure
A-3

I FRACTURING

Intensity	Size of Pieces in Feet
Very little fractured	Greater than 4.0
Occasionally fractured	1.0 to 4.0
Moderately fractured	0.5 to 1.0
Closely fractured	0.1 to 0.5
Intensely fractured	0.05 to 0.1
Crushed	Less than 0.05

II HARDNESS

1. **Soft** - reserved for plastic material alone.
2. **Low hardness** - can be gouged deeply or carved easily with a knife blade.
3. **Moderately hard** - can be readily scratched by a knife blade; scratch leaves a heavy trace of dust and is readily visible after the powder has been blown away.
4. **Hard** - can be scratched with difficulty; scratch produced a little powder and is often faintly visible.
5. **Very hard** - cannot be scratched with knife blade; leaves a metallic streak.

III STRENGTH

1. **Plastic** or very low strength.
2. **Friable** - crumbles easily by rubbing with fingers.
3. **Weak** - an unfractured specimen of such material will crumble under light hammer blows.
4. **Moderately strong** - specimen will withstand a few heavy hammer blows before breaking..
5. **Strong** - specimen will withstand a few heavy ringing hammer blows and will yield with difficulty only dust and small flying fragments.
6. **Very strong** - specimen will resist heavy ringing hammer blows and will yield with difficulty only dust and small flying fragments.

IV WEATHERING - The physical and chemical disintegration and decomposition of rocks and minerals by natural processes such as oxidation, reduction, hydration, solution, carbonation, and freezing and thawing.

D. Deep - moderate to complete mineral decomposition; extensive disintegration; deep and thorough discoloration; many fractures, all extensively coated or filled with oxides, carbonates and/or clay or silt.

M. Moderate - slight change or partial decomposition of minerals; little disintegration; cementation little to unaffected. Moderate to occasionally intense discoloration. Moderately coated fractures.

L. Little - no megascopic decomposition of minerals; little of no effect on normal cementation. Slight and intermittent, or localized discoloration. Few stains on fracture surfaces.

F. Fresh - unaffected by weathering agents. No disintegration of discoloration. Fractures usually less numerous than joints.

ADDITIONAL COMMENTS:

V CONSOLIDATION OF SEDIMENTARY ROCKS: usually determined from unweathered samples. Largely dependent on cementation.

U = unconsolidated
P = poorly consolidated
M = moderately consolidated
W = well consolidated

VI BEDDING OF SEDIMENTARY ROCKS

Splitting Property	Thickness	Stratification
Massive	Greater than 4.0 ft.	very thick-bedded
Blocky	2.0 to 4.0 ft.	thick bedded
Slabby	0.2 to 2.0 ft.	thin bedded
Flaggy	0.05 to 0.2 ft.	very thin-bedded
Shaly or platy	0.01 to 0.05 ft.	laminated
Papery	less than 0.01	thinly laminated

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Figure Title
PHYSICAL PROPERTIES CRITERIA
FOR ROCK DESCRIPTIONS

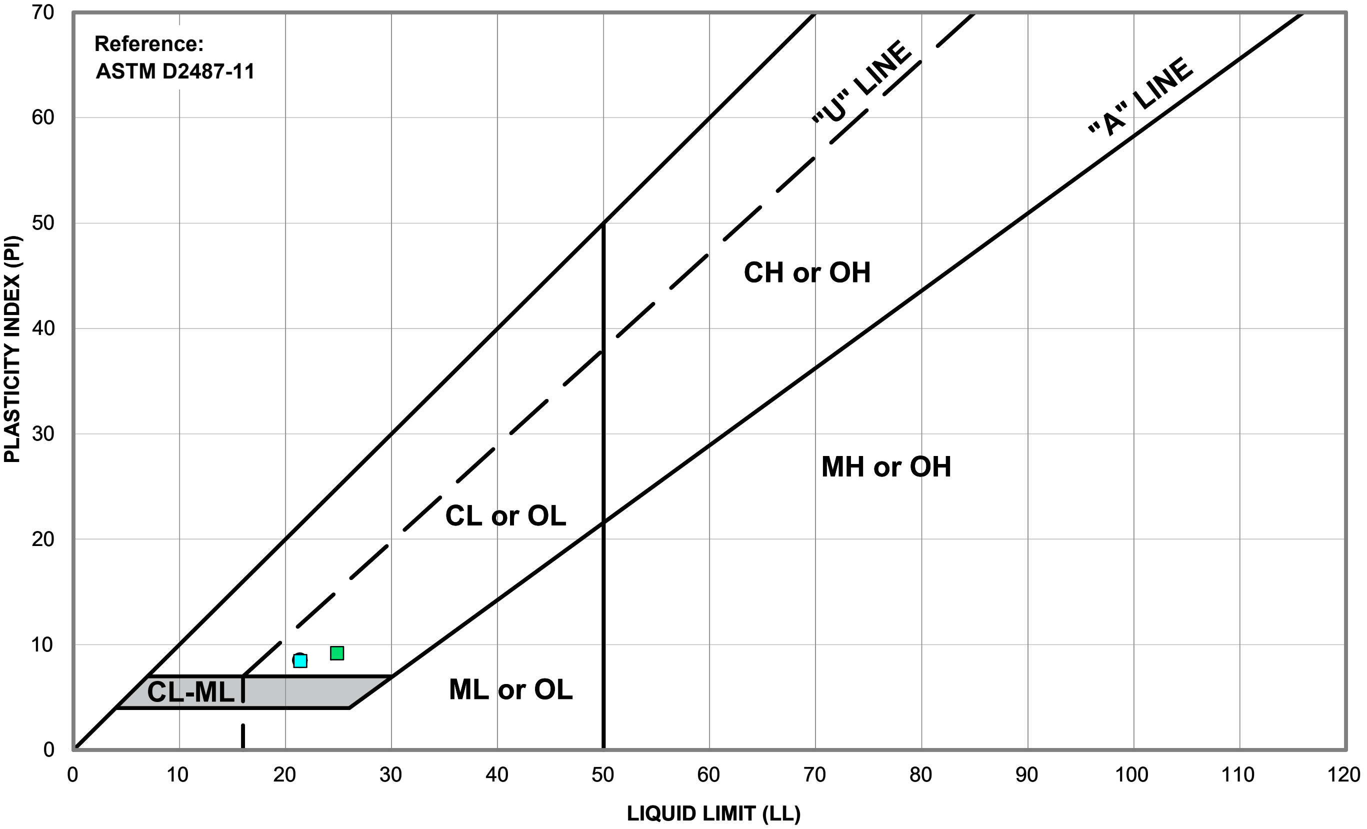
Project No. 731690402
Date 01/15/2021
Drawn By AG
Checked By PB

Figure
A-4

APPENDIX B

LABORATORY TEST RESULTS

PLASTICITY CHART



Symbol	Source	Description and Classification	Natural M.C. (%)	Liquid Limit (%)	Plasticity Index (%)	% Passing #200 Sieve
●	LB-1 at 32 feet	CLAYEY SAND (SC), mottled orange-brown and gray-brown	18.4	22	8	30.4
🟩	LB-2 at 36 feet	CLAYEY SAND (SC), gray	18.1	25	9	23.1
🟦	LB-2 at 37.5 feet	SILTY SAND (SM), yellow-brown	19.7	22	8	20.6

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Figure Title
PLASTICITY
CHART

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Date 02/16/2021
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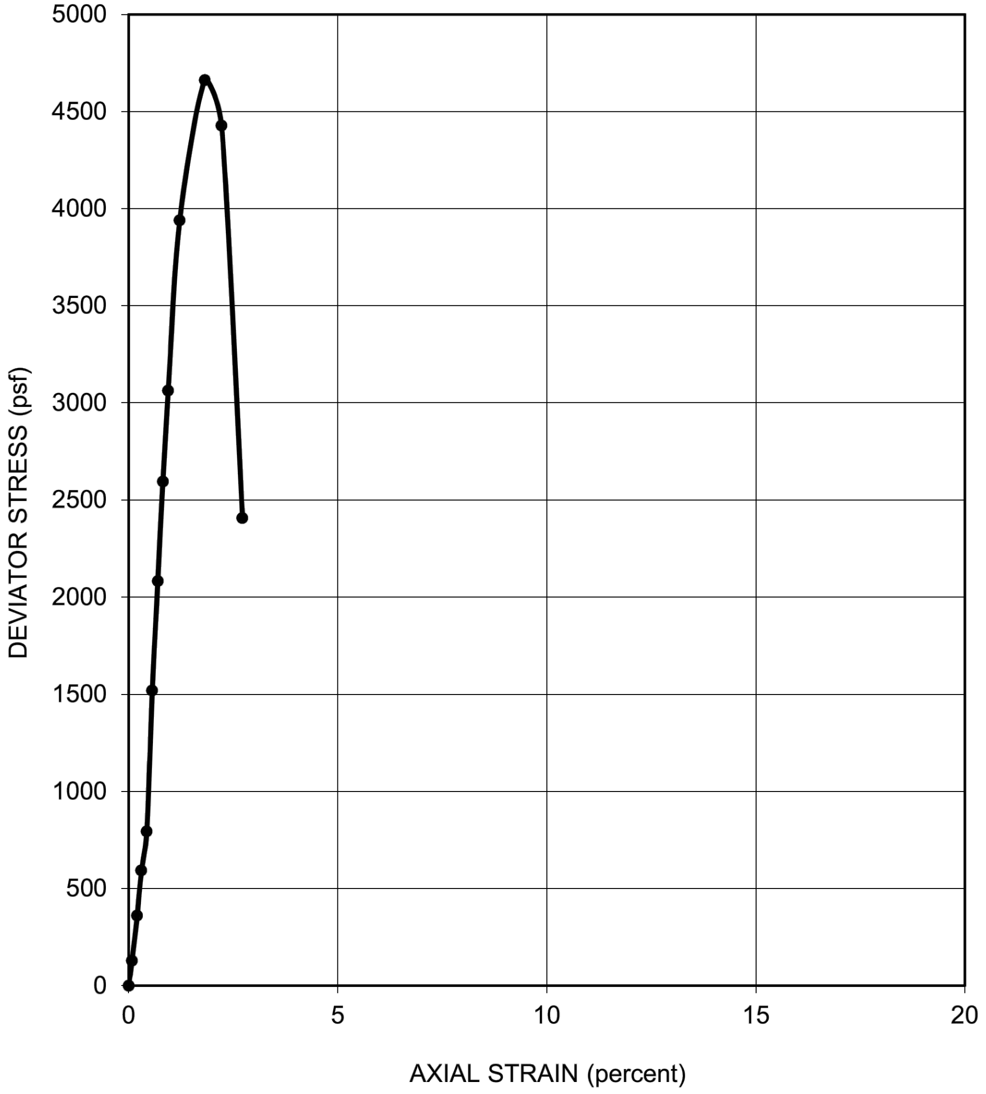
Checked By PB

Figure
B-1



SAMPLER TYPE: Shelby Tube		SHEAR STRENGTH:	2,330 psf
DIAMETER (in.): 2.86	HEIGHT (in.): 6.1	STRAIN AT FAILURE:	1.8 %
MOISTURE CONTENT: 43.9 %		CONFINING PRESSURE:	10,000 psf
DRY DENSITY: 78 pcf		STRAIN RATE:	0.50 % / min
DESCRIPTION: CLAY (CH), gray		SOURCE:	LB-1 at 100 feet

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Figure Title
UNCONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TEST

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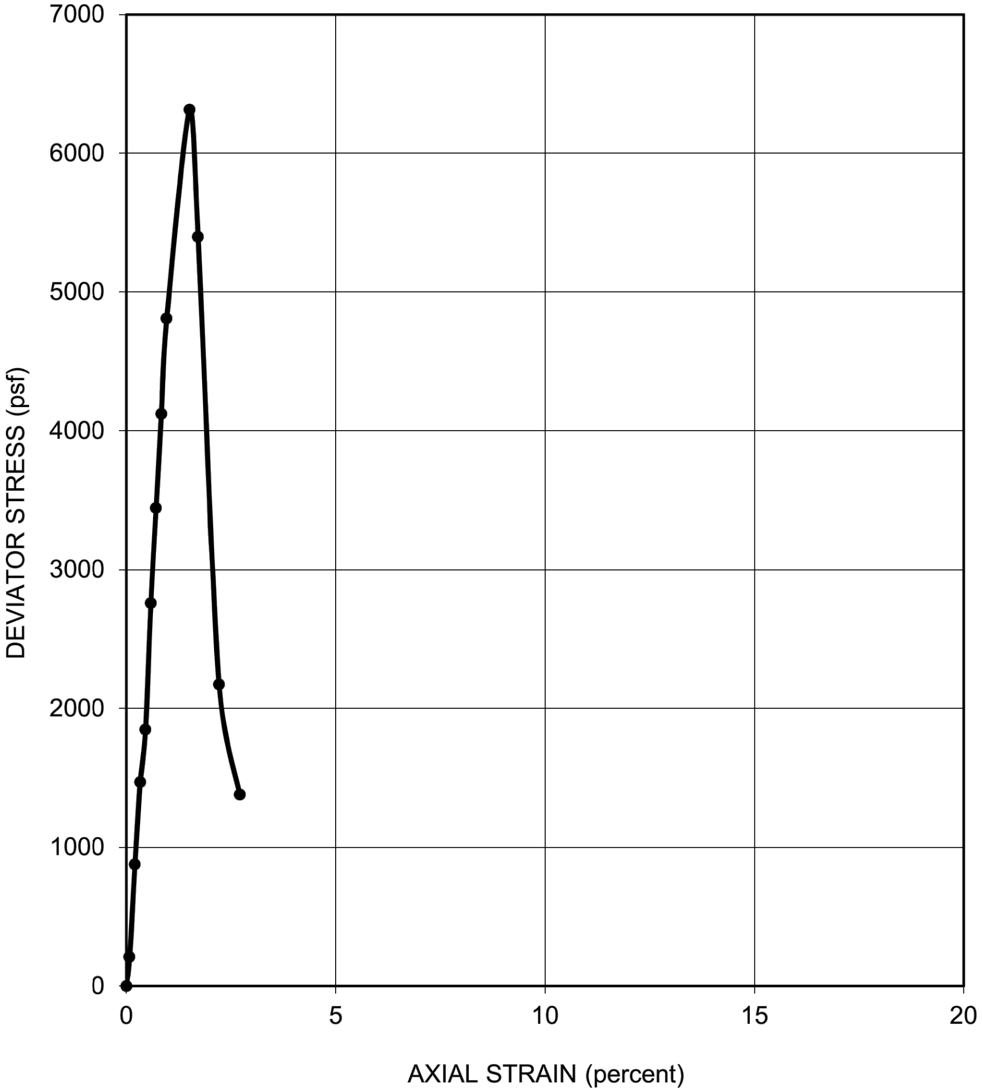
Drawn By AG
Checked By PB

Figure
B-2



SAMPLER TYPE: Shelby Tube			SHEAR STRENGTH:	3,160 psf
DIAMETER (in.): 2.86	HEIGHT (in.): 6.1		STRAIN AT FAILURE:	1.5 %
MOISTURE CONTENT:	52.6	%	CONFINING PRESSURE:	11,000 psf
DRY DENSITY:	71	pcf	STRAIN RATE:	0.50 % / min
DESCRIPTION: CLAY (CH), gray			SOURCE:	LB-1 at 110 feet

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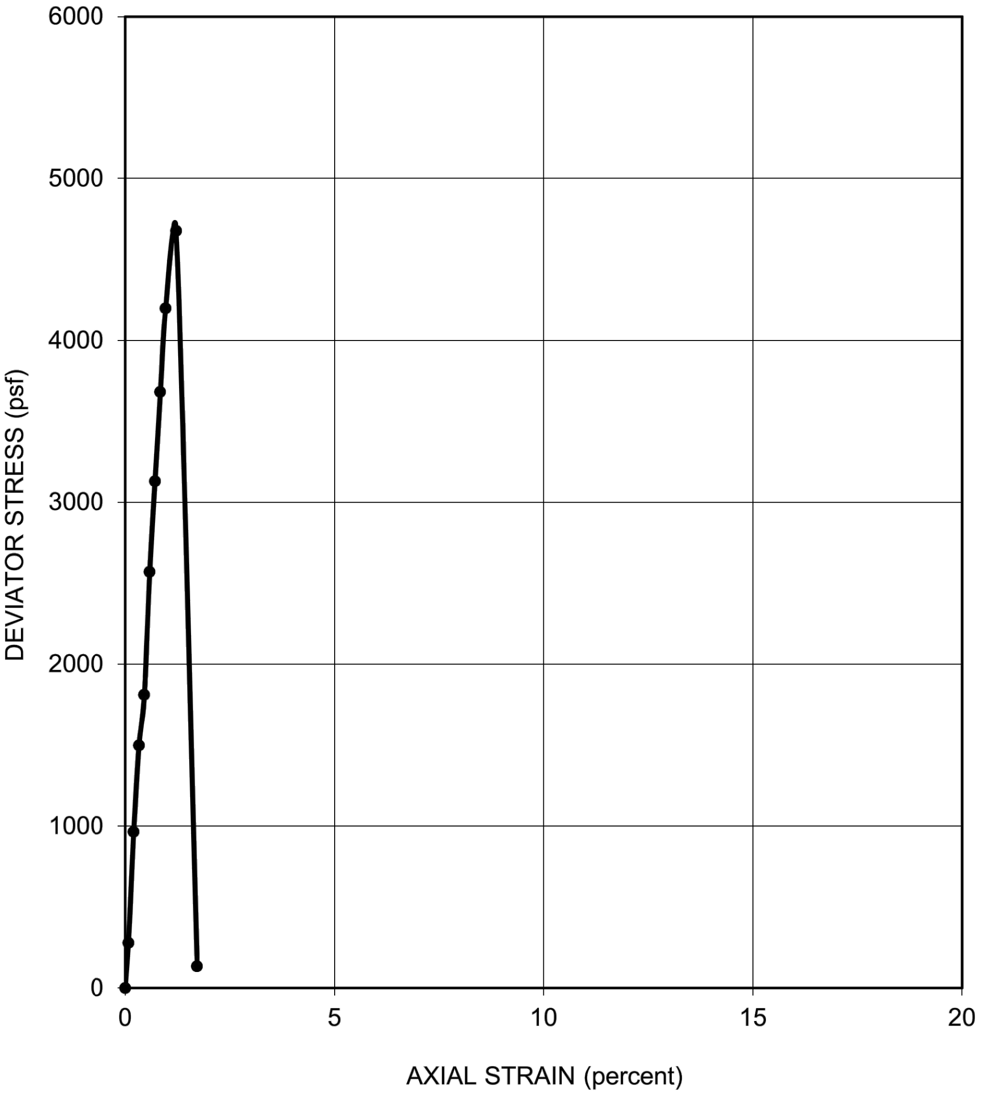
Figure B-3



SAMPLER TYPE: Shelby Tube			SHEAR STRENGTH:	2,340	psf
DIAMETER (in.): 2.86	HEIGHT (in.): 6.1		STRAIN AT FAILURE:	1.2	%
MOISTURE CONTENT:	53.3	%	CONFINING PRESSURE:	12,000	psf
DRY DENSITY:	70	pcf	STRAIN RATE:	0.50	% / min
DESCRIPTION: CLAY (CH), gray			SOURCE:	LB-1 at 120 feet	

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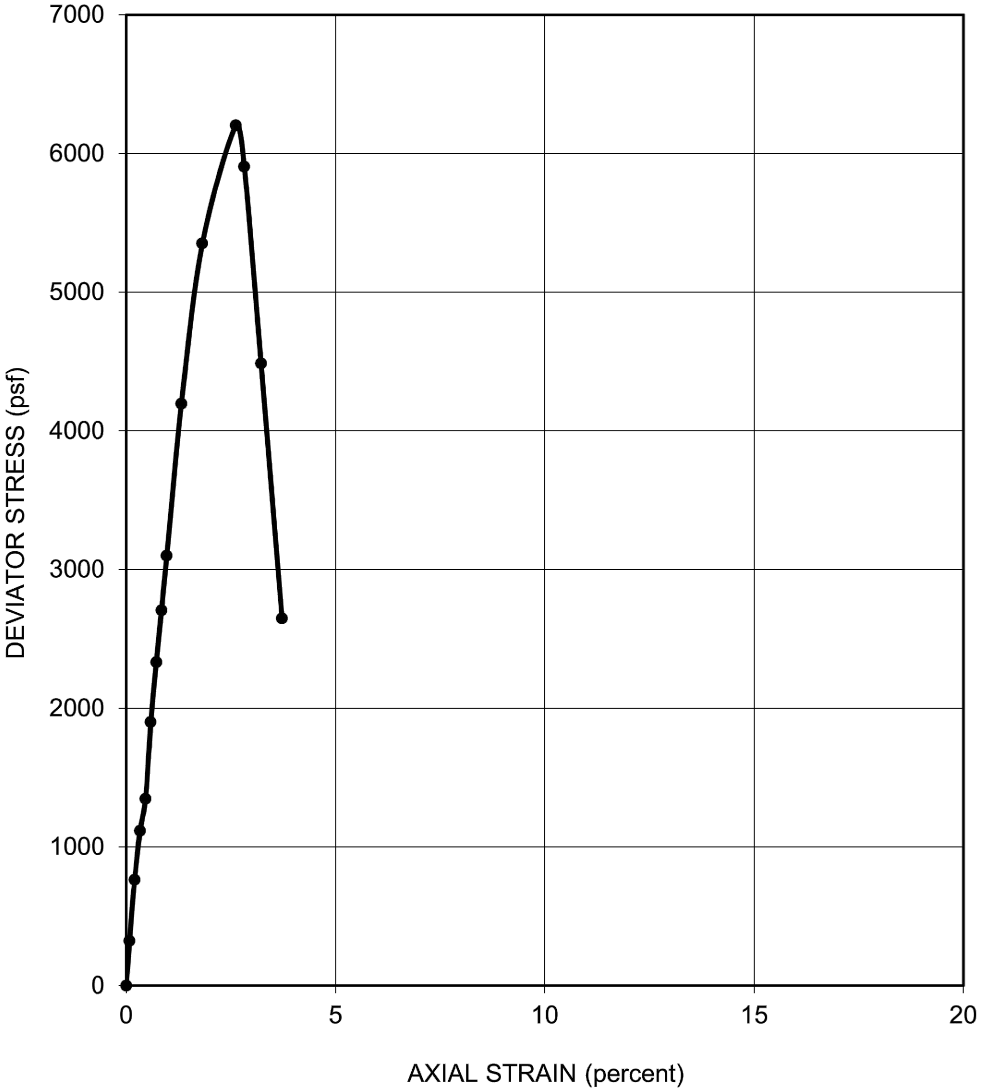

SAMPLER TYPE: Shelby Tube			SHEAR STRENGTH:	3,1000 psf
DIAMETER (in.): 2.86	HEIGHT (in.): 6.1		STRAIN AT FAILURE:	2.6 %
MOISTURE CONTENT:	46.0	%	CONFINING PRESSURE:	13,000 psf
DRY DENSITY:	74	pcf	STRAIN RATE:	0.50 % / min
DESCRIPTION: CLAY (CH), gray			SOURCE:	LB-1 at 130 feet

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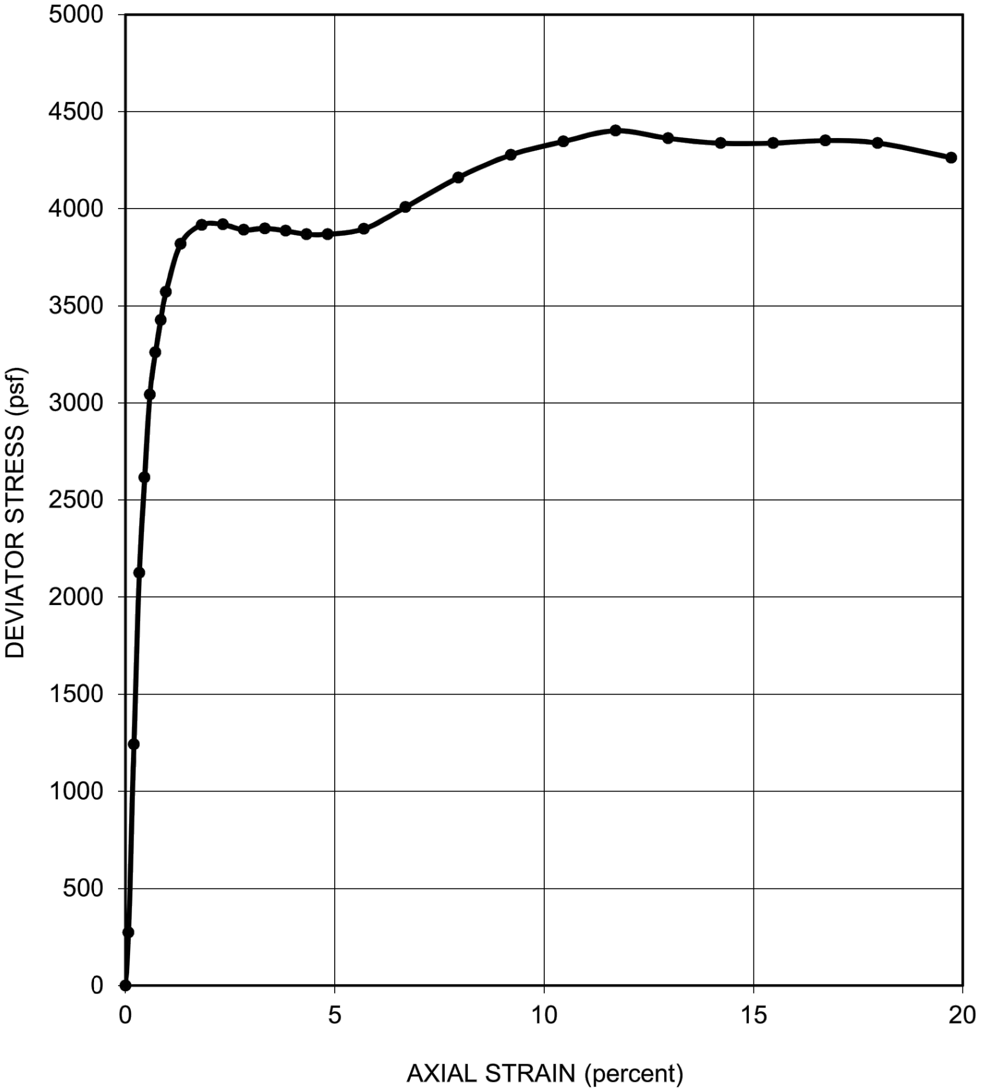
Figure
B-5

Filename: \\langan.com\data\SFO\data4\731690402\Project Data\CAD\02\2D-DesignFiles\Geotechnical\731690402-B-GI0101_Lab-Classification_recover.dwg Date: 2/16/2021 Time: 15:47 User: agekas Style Table: Langan.stb Layout: TXUU REPORT_4



SAMPLER TYPE: Shelby Tube				SHEAR STRENGTH:	2,200	psf
DIAMETER (in.): 2.86	HEIGHT (in.): 6.1			STRAIN AT FAILURE:	11.7	%
MOISTURE CONTENT:		42.5	%	CONFINING PRESSURE:	19,500	psf
DRY DENSITY:		80	pcf	STRAIN RATE:	0.50	% / min
DESCRIPTION: CLAY (CH), gray					SOURCE:	LB-1 at 195 feet

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Figure Title
UNCONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TEST

Project No. 731690402
Date 02/16/2021
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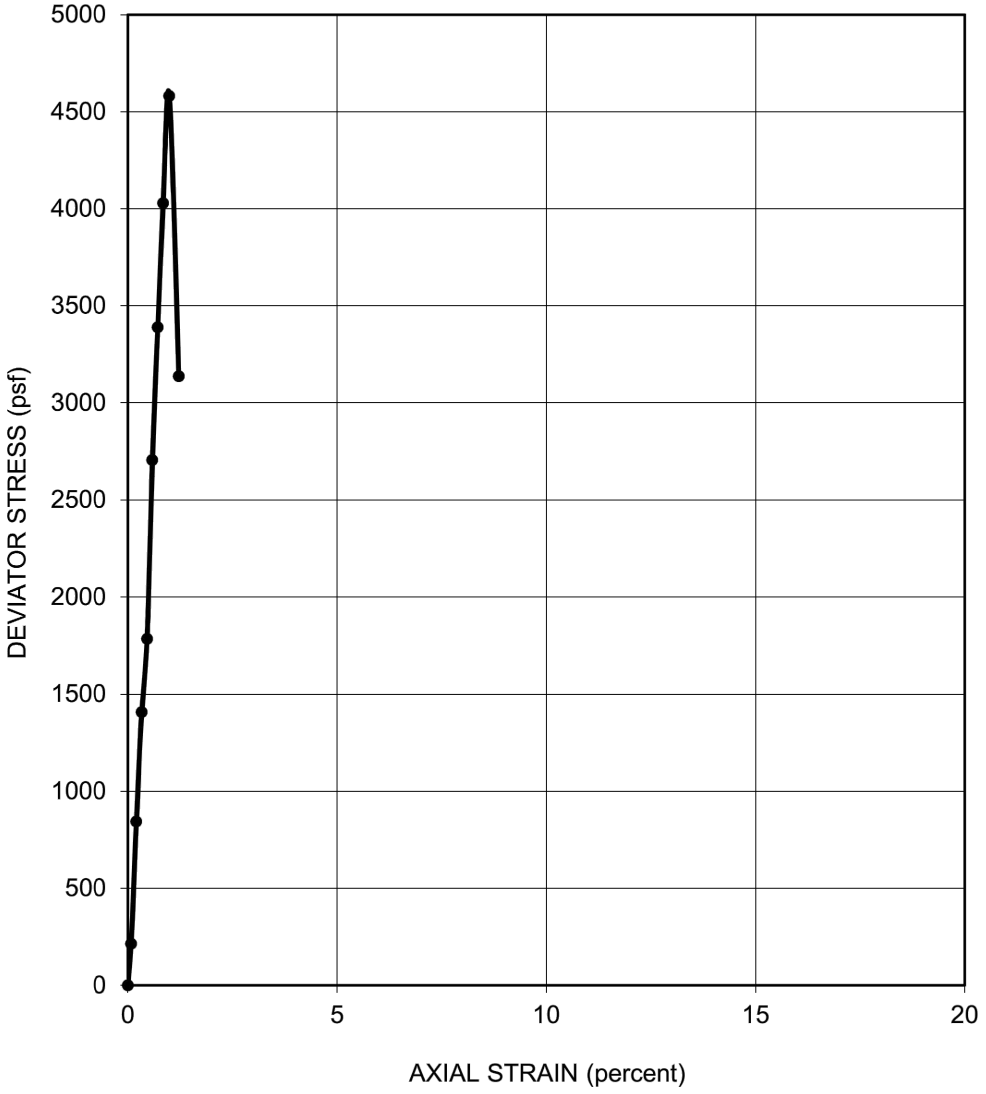
Figure
B-6



SAMPLER TYPE: Shelby Tube				SHEAR STRENGTH:	2,290	psf
DIAMETER (in.): 2.86	HEIGHT (in.): 6.1			STRAIN AT FAILURE:	1.0	%
MOISTURE CONTENT:		54.2	%	CONFINING PRESSURE:	12,000	psf
DRY DENSITY:		69	pcf	STRAIN RATE:	0.50	% / min
DESCRIPTION: CLAY (CH), gray				SOURCE: LB-2 at 120 feet		

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Figure
B-7

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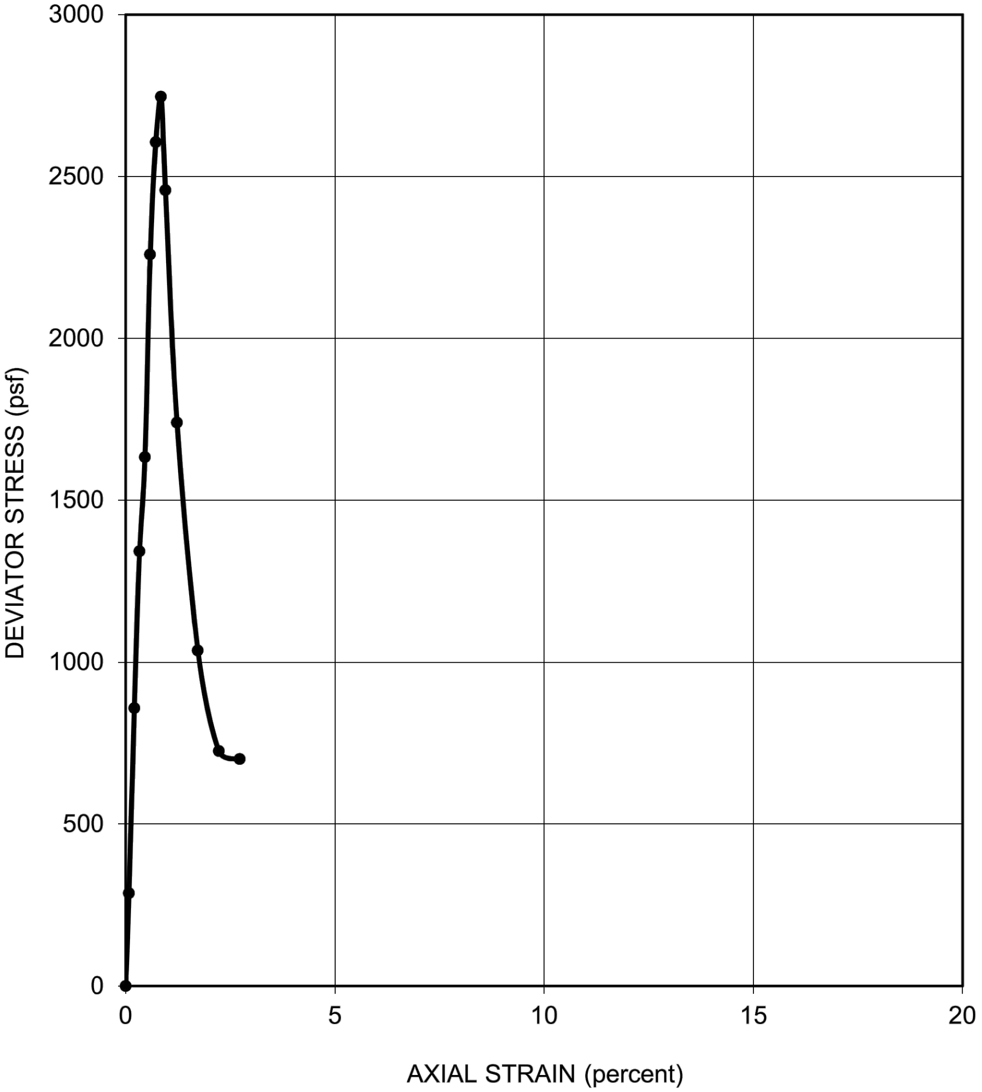

SAMPLER TYPE: Shelby Tube			SHEAR STRENGTH:	1,370	psf
DIAMETER (in.): 2.86	HEIGHT (in.): 6.1		STRAIN AT FAILURE:	0.8	%
MOISTURE CONTENT:	51.2	%	CONFINING PRESSURE:	13,000	psf
DRY DENSITY:	72	pcf	STRAIN RATE:	0.50	% / min
DESCRIPTION: CLAY (CH), gray			SOURCE:	LB-2 at 130 feet	

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Figure Title
UNCONSOLIDATED-UNDRAINED
TRIAXIAL COMPRESSION TEST

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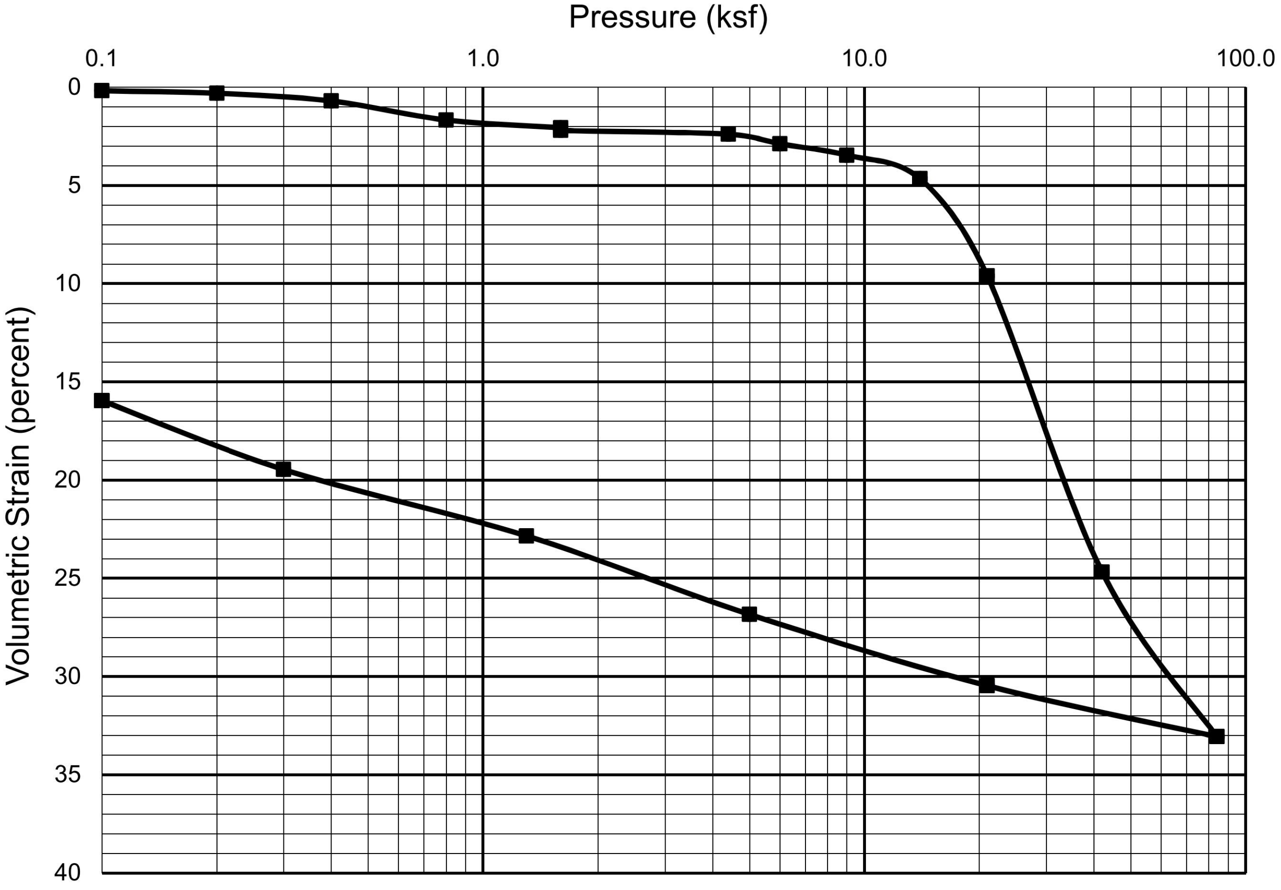
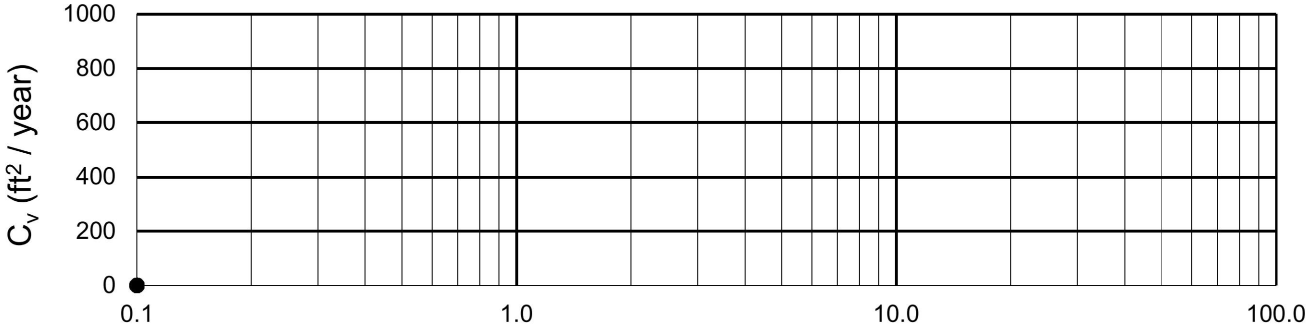
Figure B-8





Sampler Type					Condition Before Test			After Test	
Shelby Tube	Diameter (in)	2.42	Height (in)	1.00	Water Content	w_o	51.9%	w_f	38.9%
Overburden Pressure	P_o	8,250 psf			Void Ratio	e_o	1.43	e_f	1.05
Preconsol. Pressure	P_c	17,000 psf			Saturation	S_o	98%	S_f	100%
Compression Ration	$C\varepsilon_c$	0.42			Dry Density	γ_d	69 pcf	γ_d	82 pcf
Liquid Limit:	--	Plastic Limit:	--	Plasticity Index:	--	G_s	2.70 (assumed)		
Classification:	CLAY (CH), gray				Source:	LB-1 at 110 feet			

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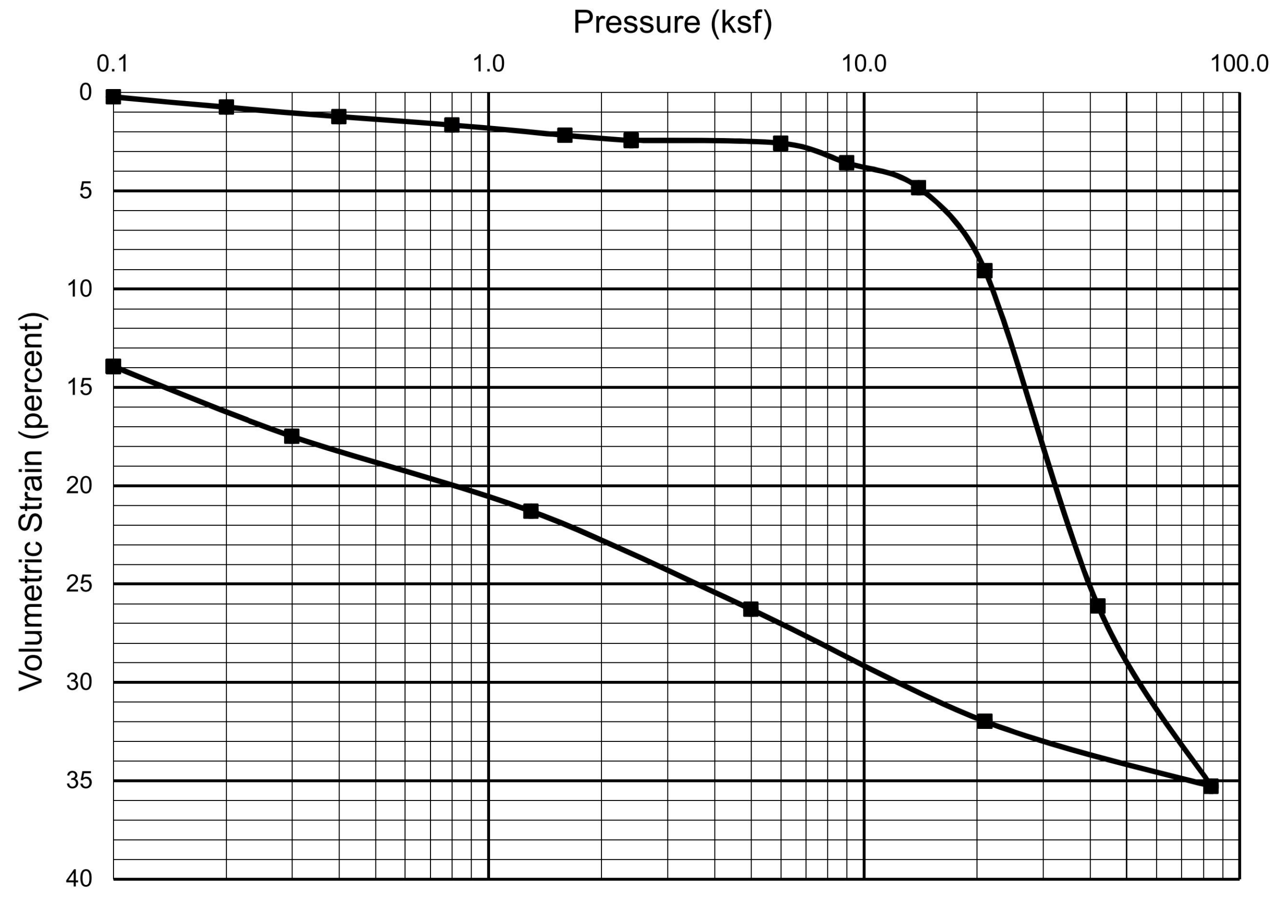
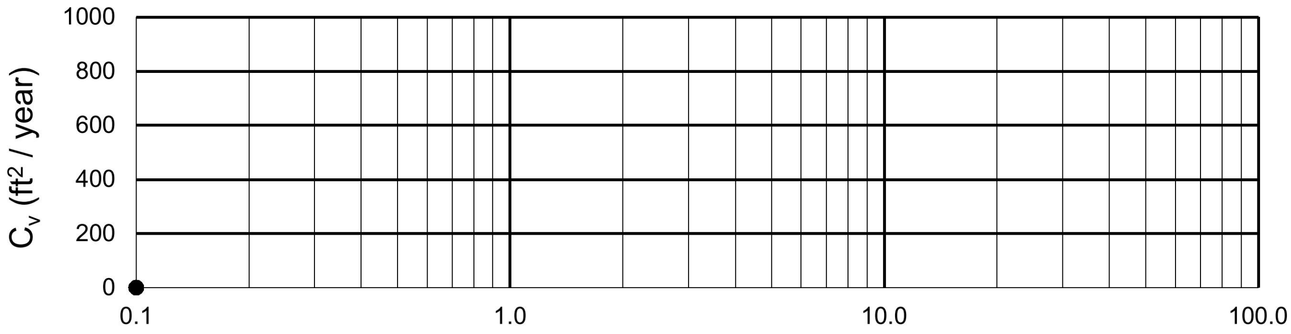
Figure
B-9





Sampler Type					Condition Before Test			After Test	
Shelby Tube	Diameter (in)	2.42	Height (in)	1.00	Water Content	w_o	56.2%	w_f	44.0%
Overburden Pressure	P_o	8,650 psf			Void Ratio	e_o	1.54	e_f	1.18
Preconsol. Pressure	P_c	18,000 psf			Saturation	S_o	99%	S_f	100%
Compression Ration	$C\varepsilon_c$	0.51			Dry Density	γ_d	66 pcf	γ_d	77 pcf
Liquid Limit:	--	Plastic Limit:	--	Plasticity Index:	--	G_s	2.70 (assumed)		
Classification:	CLAY (CH), gray				Source:	LB-1 at 120 feet			

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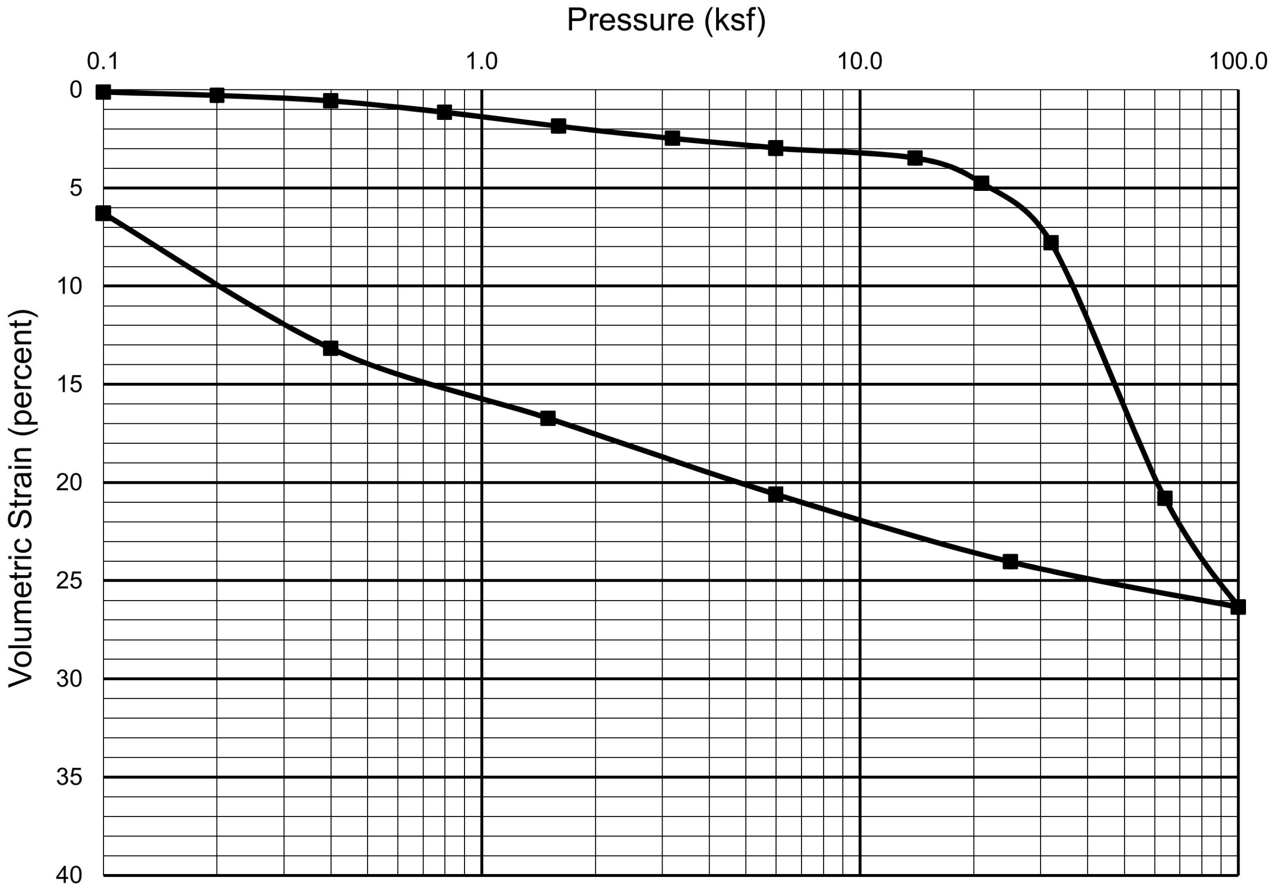
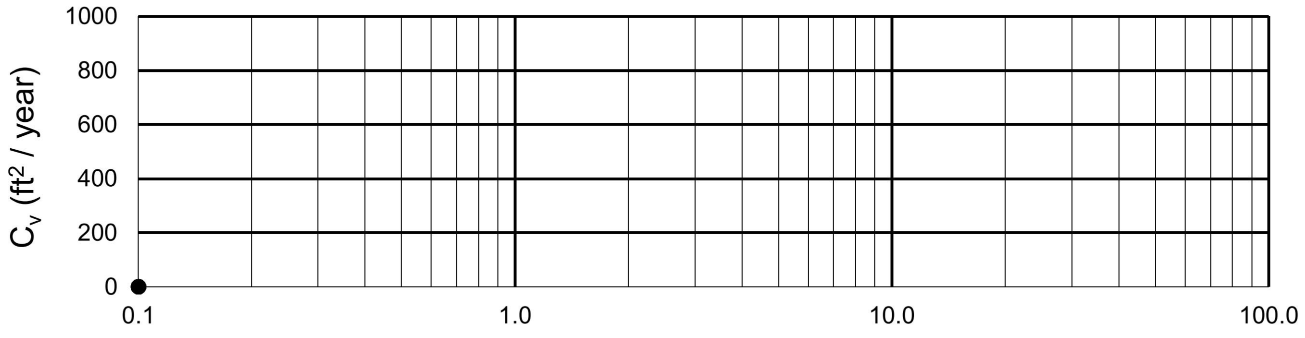
Figure
B-10





Sampler Type					**Condition Before Test**			**After Test**	
Shelby Tube	Diameter (in)	2.42	Height (in)	1.00	Water Content	w_o	43.4%	w_f	39.0%
Overburden Pressure	P_o	13,500 psf			Void Ratio	e_o	1.19	e_f	1.05
Preconsol. Pressure	P_c	30,000 psf			Saturation	S_o	99%	s_f	100%
Compression Ration	$C\varepsilon_c$	0.41			Dry Density	γ_d	77 pcf	γ_d	82 pcf
Liquid Limit:	--	Plastic Limit:	--	Plasticity Index:	--	G_s	2.70 (assumed)		
Classification:	CLAY (CH), gray				Source:	LB-1 at 195 feet			

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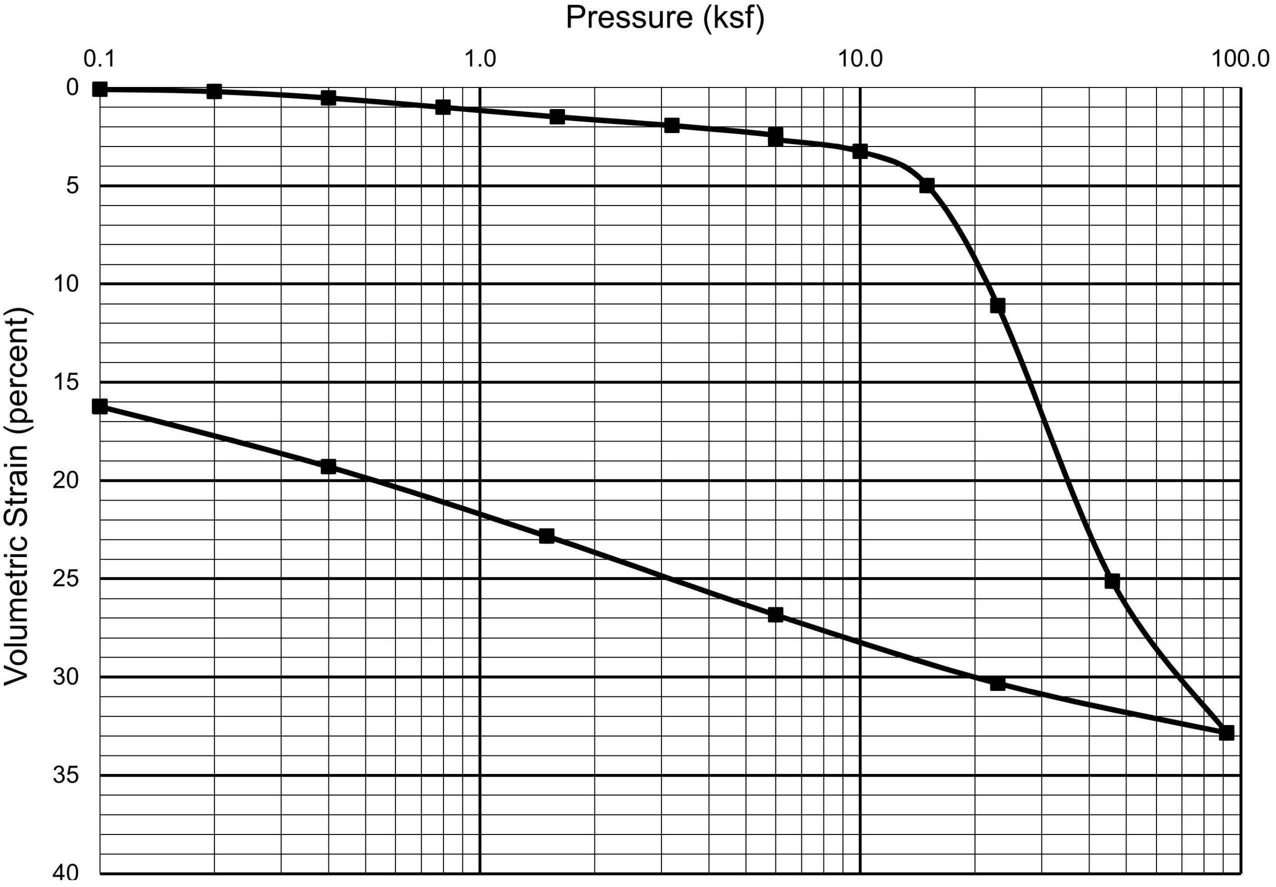
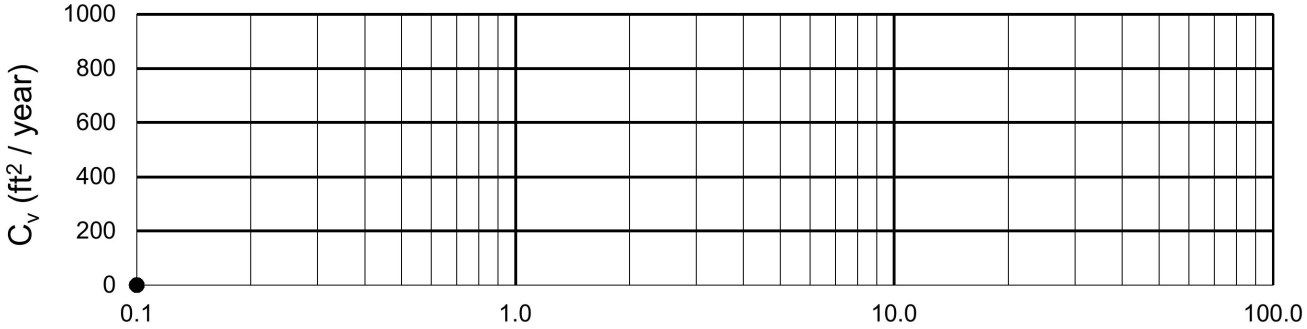
Figure
B-11





Sampler Type					Condition Before Test			After Test	
Shelby Tube	Diameter (in)	2.42	Height (in)	1.00	Water Content	w_o	55.4%	w_f	41.0%
Overburden Pressure	P_o	9,200 psf			Void Ratio	e_o	1.51	e_f	1.11
Preconsol. Pressure	P_c	17,000 psf			Saturation	S_o	99%	s_f	100%
Compression Ration	$C\varepsilon_c$	0.40			Dry Density	γ_d	67 pcf	γ_d	80 pcf
Liquid Limit:	--	Plastic Limit:	--	Plasticity Index:	--	G_s	2.70 (assumed)		
Classification:	CLAY (CH), gray				Source:	LB-2 at 120 feet			

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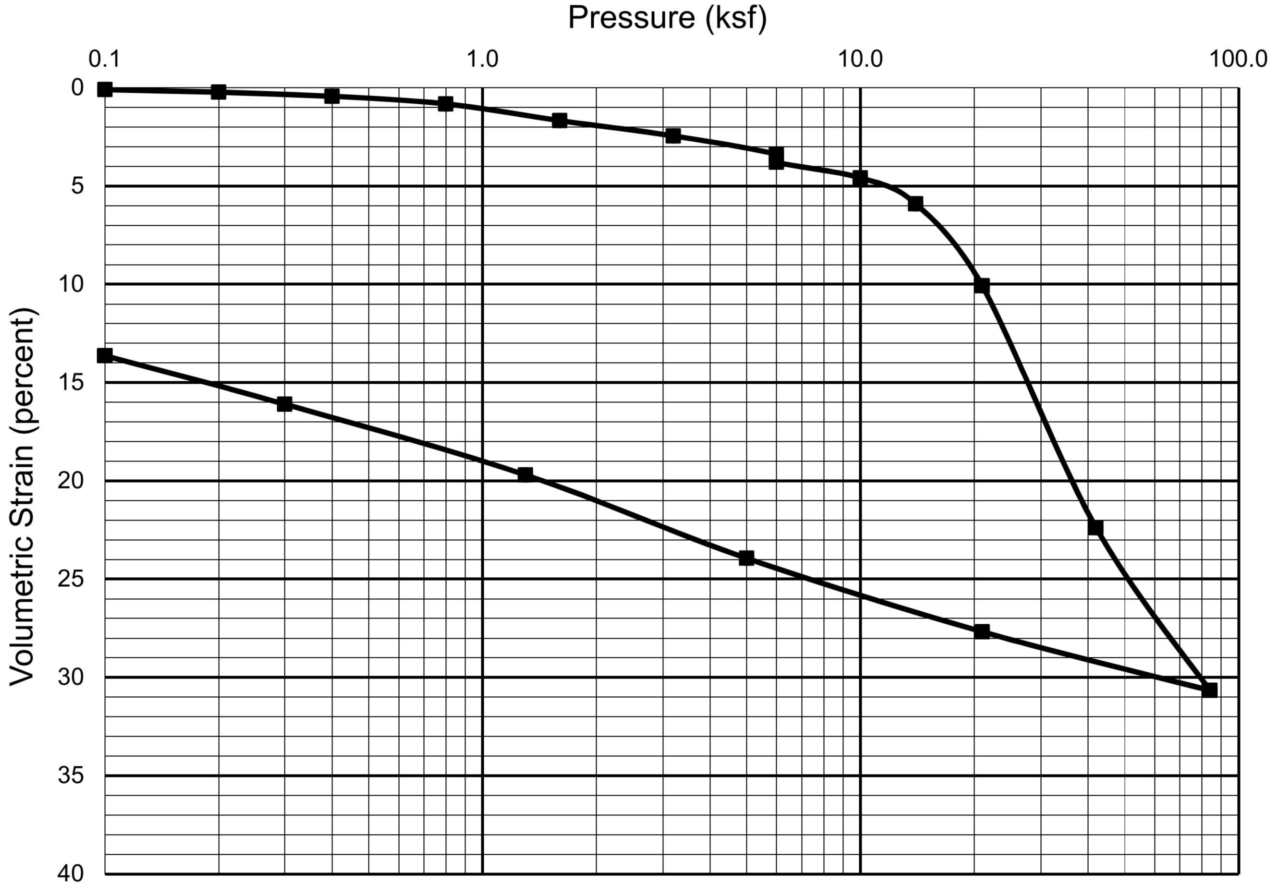
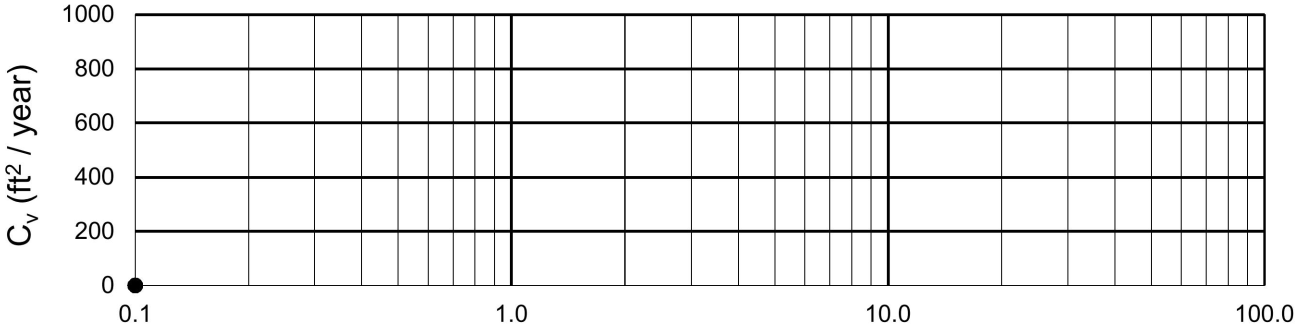
Figure
B-12

Filename: \\langan.com\data\SF\data4\731690402\Project Data\CAD\02\2D-DesignFiles\Geotechnical\731690402-B-GI0101_Lab-Classification_recover.dwg Date: 3/10/2021 Time: 11:08 User: agekas Style Table: Langan.stb Layout: CONSOL_B-12





Sampler Type					Condition Before Test			After Test	
Shelby Tube	Diameter (in)	2.42	Height (in)	1.00	Water Content	w_o	52.2%	w_f	41.1%
Overburden Pressure	P_o	9,650 psf			Void Ratio	e_o	1.44	e_f	1.11
Preconsol. Pressure	P_c	18,000 psf			Saturation	S_o	98%	S_f	100%
Compression Ration	$C\varepsilon_c$	0.36			Dry Density	γ_d	69 pcf	γ_d	80 pcf
Liquid Limit:	--	Plastic Limit:	--	Plasticity Index:	--	G_s	2.70 (assumed)		
Classification:	CLAY (CH), gray				Source:	LB-2 at 130 feet			

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Figure
B-13

APPENDIX C

DOWNHOLE SUSPENSION LOGGING

NORCAL
GEOPHYSICAL CONSULTANTS INC.
A Terracon COMPANY

January 11, 2021

LANGAN
135 Main Street, Suite 1500
San Francisco, CA 94105

Subject: Borehole Geophysical Logging Investigation
469 Stevenson Street
San Francisco, California

NORCAL Job No: NS205151

Attention: Mr. Peter Brady, PE

This report presents the findings of a Borehole Geophysical Logging (BGL) investigation performed by NORCAL Geophysical Consultants, Inc. in support of a geotechnical study being conducted by LANGAN for a site located in the parking lot south of 469 Stevenson Street in San Francisco, California. The BGL survey was authorized under LANGAN Work Authorization letter signed by Maria Flessas, Principal, dated December 16, 2020. The field work was performed on December 18, 2020 by NORCAL Professional Geophysicist Charles Carter (PGp 1051). Logistical support and safety information were provided onsite by LANGAN Engineer, Mr. Roman Nelson.

Figure 1. Borehole Location Map

The BGL investigation was conducted in one geotechnical borehole labeled as LB-1. This borehole was located in the northeast corner of the parking lot located between Stevenson Street and Jessie, as shown in Figure 1. The purpose of the BGL survey was to measure shear (S-) wave and compressional (P-) wave velocities in the upper 250-feet of sediments and soils to determine the site classification as this relates to earthquake ground motion studies for proposed new construction or building upgrades.

1.0 SCOPE OF SERVICES

Our scope of services for this project consisted of conducting a BGL investigation in borehole LB-1. Our scope also included processing and interpreting the BGL data and presenting our findings in a written report.

2.0 BOREHOLE CONDITIONS

Borehole LB-1 was advanced with a 6-inch diameter rotary wash drilling method to a depth of 11-ft below ground surface (bgs). A steel conductor casing was then installed in the upper 11-ft to prevent sloughing from loose fill or soil. Below 11-ft bgs, the drilling was advanced with a 4.9-inch rotary bit to 256-ft bgs. Borehole stability was very good. However, the borehole lost about one foot of depth due to sediment sloughing.

3.0 BOREHOLE GEOPHYSICAL LOGGING (BGL) SURVEY

Detailed descriptions of the PS-wave Suspension methodology, our data acquisition and data analysis procedures and how the results are presented, are provided in Appendix A. The appendix includes a table listing the interval P- and S-wave velocities measured from 13- to 241-ft bgs.

4.0 RESULTS

The results of the suspension borehole logging survey are illustrated by the velocity versus depth graph shown on the left side of Plate 1. The graph depicts the variation in S-wave velocity (Vs) and P-wave velocity (Vp) versus depth. Vs are indicated by the red triangles and Vp are indicated by the blue circles. These are interval velocities (see Appendix A) and, as such, are the values that should be used in any evaluation of foundation characteristics. To the right of the PS-wave velocity plots, we present a shaded caliper log (labeled as Borehole Diameter).

5.0 DISCUSSION

As shown on Plate 1, the seismic velocities in Borehole LB-1 range from 650 to 2,100 feet per second (ft/s) for Vs and 3,900 to 6,800 ft/s for Vp. The static water level was reported to be at 32 feet below ground surface. The P-wave data were generally of poor quality above 32 feet due to the very weak signal recorded at the far receiver. The P-wave data quality improved below 32 feet which may be due to a transition from partially saturated alluvium above the local water table to fully water saturated alluvium. There were several borehole washouts detected in the caliper log between the bottom of casing and 30-ft bgs which also contributed to poor P-wave signal. The Vp values of less than 5000 fps presented here are only approximations.

In general, the Vs velocities are somewhat variable due to changes in stratigraphy. Higher Vs velocities generally correspond to more consolidated coarse-grained sediments like sands or gravels while lower velocity zones may correspond to finer material such as clays or muds.

6.0 STANDARD CARE

The scope of NORCAL's services for this project consisted of using geophysical methods to characterize the subsurface. The accuracy of our findings is subject to specific site conditions and limitations inherent to the techniques used. We performed our services in a manner consistent with the standard of care ordinarily exercised by members of the profession currently employing similar methods. No warranty, with respect to the performance of services or products delivered under this agreement, expressed or implied, is made by NORCAL.

We appreciate the opportunity to provide our services to Langan for this project. If you have any questions or require additional geophysical services, please do not hesitate to contact us.

Sincerely,
NORCAL Geophysical Consultants, Inc.

Charles Carter

Charles Carter
Professional Geophysicist PGp 1051

PROFESSIONAL GEOPHYSICIST
Charles G Carter
1051
1-11-2021
STATE OF CALIFORNIA

Donald J. Kirker

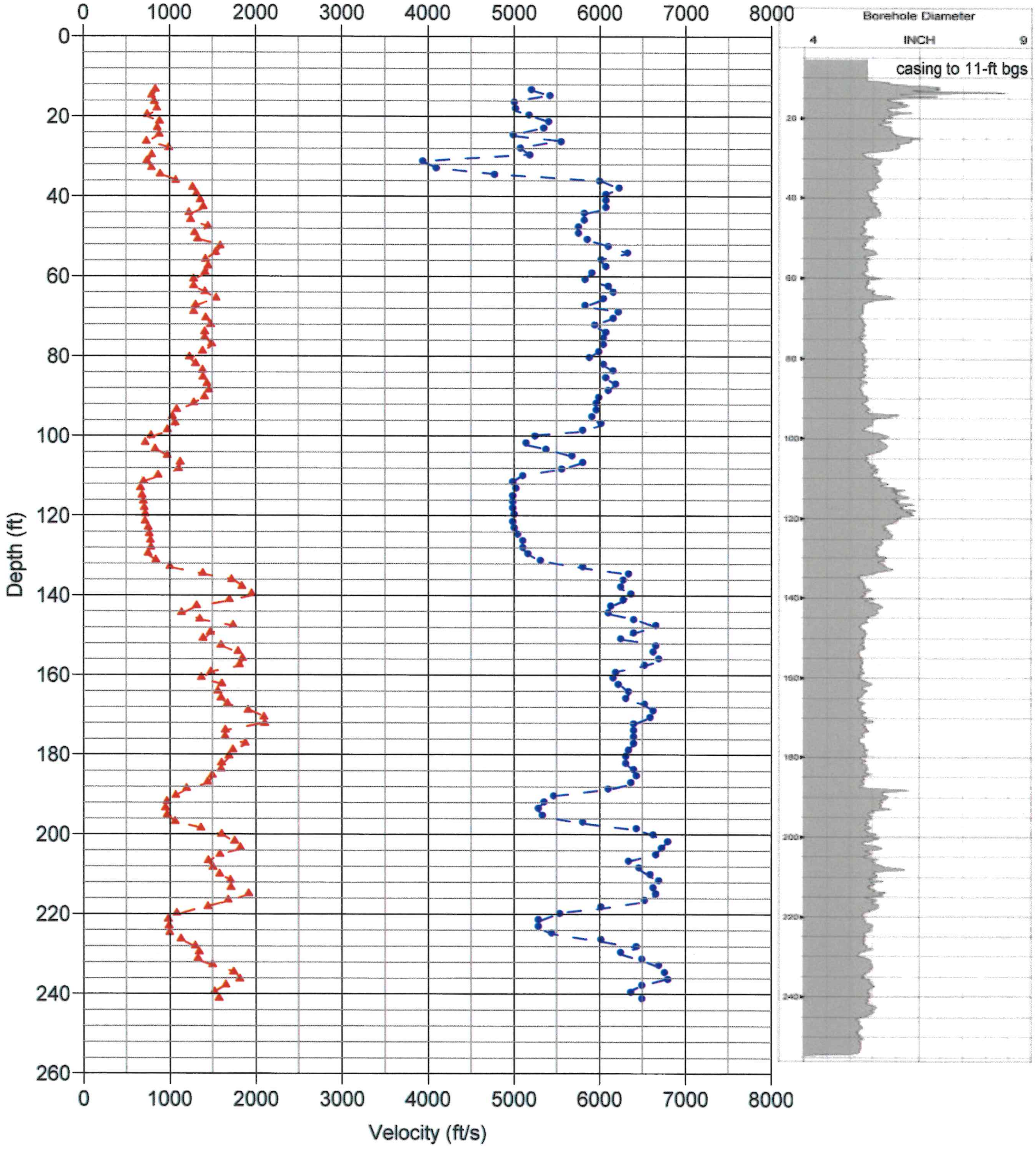
Donald J Kirker, PGp 997
Office Manager, Reviewer

PROFESSIONAL GEOPHYSICIST
DONALD KIRKER
No. 997
1-11-2021
STATE OF CALIFORNIA

WJH/DK

Enclosures: Plate 1: Suspension P- and S-wave Velocity Profile, LB-1
Appendix A: PS-wave Suspension Velocity Survey

LB-1



LEGEND

▲ Vs R1-R2
● Vp R1-R2

NORCAL

SUSPENSION P- &
S-WAVE VELOCITY PROFILE
BOREHOLE: LB-1

LOCATION: 469 Stevenson Street, San Francisco, CA

CLIENT: LANGAN

JOB #: NS205151

NORCAL GEOPHYSICAL CONSULTANTS INC.

Date: Dec., 2020

DRAWN BY: C. CARTER

APPROVED BY: CGC

PLATE
1

Appendix A:

Suspension P- and S-wave Velocity Survey

APPENDIX A

PS-WAVE SUSPENSION VELOCITY SURVEY

The Suspension logger is a highly specialized downhole tool that measures P- and S-wave velocities at discrete depths. The following presents a narrative on its operation and the data reduction procedures we used in computation of suspension P- and S-wave velocities. Also presented is a tabulated velocity data table interpreted for the suspension logging survey in Borehole LB-1.

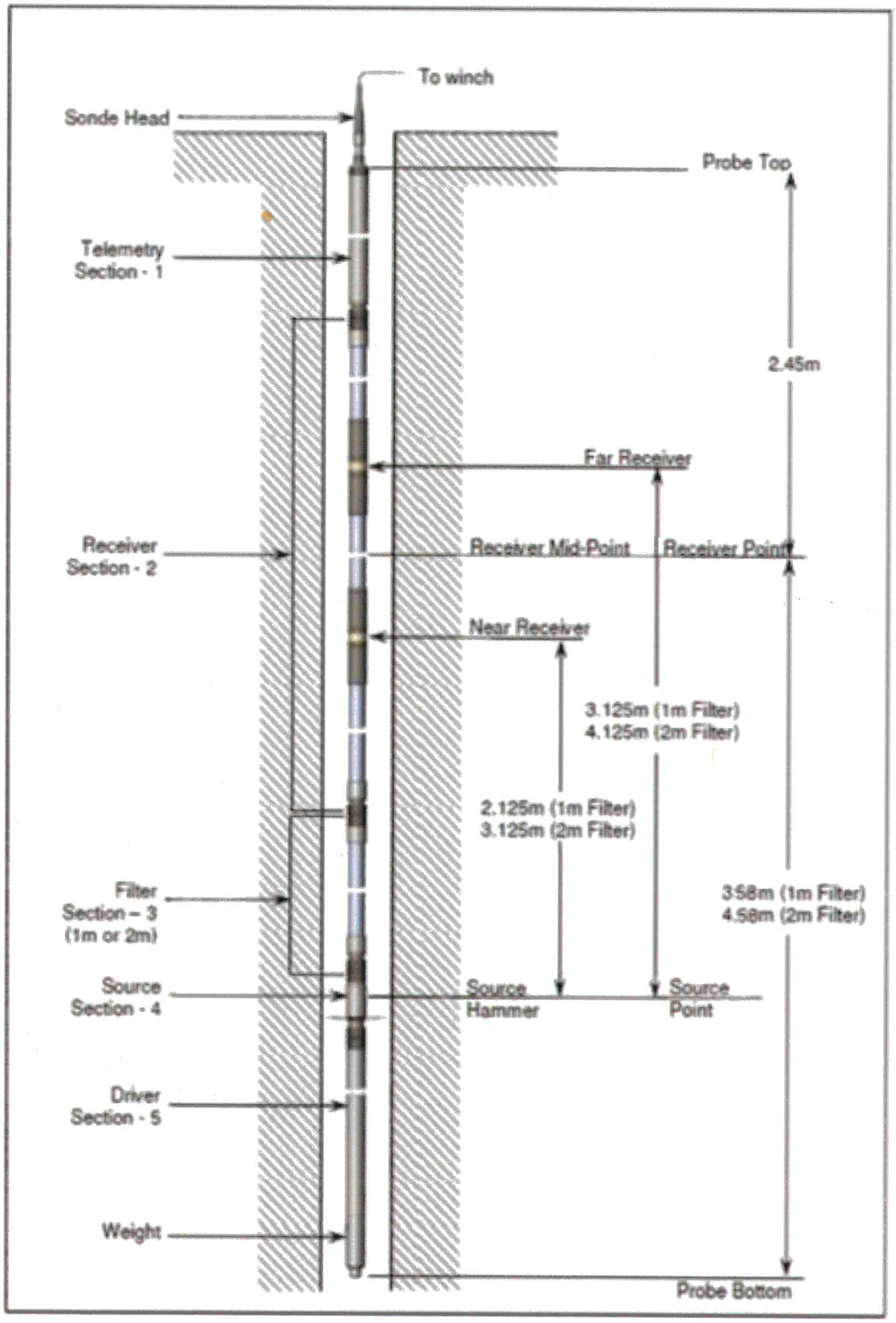
1.0 METHODOLOGY

We measured downhole compressional (P-) and shear (S-) wave velocities using a Robertson Geologging, Ltd. digital suspension logging system. A schematic diagram depicting the probe configuration is shown in Figure A-1. The suspension logging tool is equipped with a dipole seismic energy source hammer located near the base of the probe and a pair of detectors (receivers) designated as near receiver (R1) and far receiver (R2), located within the middle to the upper sections of the probe. The distance from the energy source to the closest receiver was 6.97 feet (2.125 meters) when assembled with a detachable 1-meter isolation tube. The in-line distance between the receiver pair was 3.28-feet (1.0 meter). Each receiver contains one horizontal and one vertical oriented element. The horizontal receiver elements preferentially record shear wave motion. The vertical receiver elements preferentially record first arriving P-wave energy.

Suspension seismic data are collected at discrete depths in the fluid-filled portion of the borehole. At each measurement depth, the energy source is activated via commands from the surface control console. This activation causes a metal solenoid to strike the metal strike cylinder. This energy transmits through the fluid to the borehole wall which initiates a seismic wave ("flexure") in the adjacent formation. As this wave propagates radially into the formation a physical interaction between the flexure wave and the borehole wall creates shear waves together with a refracted compressional P-wave that travels up the borehole to the two detectors. The source hammer and horizontal elements in the receivers are oriented parallel to one another. By striking the metal strike cylinder on opposite sides of the probe (referred to as left strike and right strike) the amplitudes of both the positive and negative polarity S-waves are maximized.

When assembled with a 1-meter isolation tube, the suspension logging tool is approximately 19.5 feet long (Figure A-1). By definition, the depth reference point of the tool is half-way between the two receivers. Since this point is approximately 13-ft from the probe tip, the maximum depth of a suspension logging survey, given a non-sloughing borehole, will always be reported as 13 feet less than the total depth of the borehole. When in operation, the probe is centralized in the borehole with flexible rubber rings positioned just below the source and just above the receiver section. This is necessary to maintain a gap between the probe housing and borehole wall.

Figure A-1: Suspension logger schematic diagram



2.0 DATA ACQUISITION

We measured seismic suspension velocities at stationary depth positions in the fluid-filled borehole distributed vertically at 0.5-meter intervals. The borehole was drilled to a maximum depth of 256-ft below ground (pavement) surface. Steel conductor casing was installed to 11-ft bgs. The water level in the borehole was less than 10-ft bgs during data acquisition. Since the PS logging needs a fluid-filled, open borehole to operate, the upper bound of the PS survey was approximately13-ft bgs.

At each measurement station, the energy source fired 3 times in succession. This cycling of the seismic energy resulting in a "left" and "right" strike for the shear waves (recorded with the horizontal elements) and a third strike for the P-waves (recorded with the vertical elements). Depending on frequency response, we recorded S-waves by applying various low pass filters (2.4 KHz) to the signals detected by the horizontal receiver elements. These filter settings were much higher than the observed center fundamental frequencies of the S-wave signal. This filtering was used to suppress interference from the onset of earlier arriving P-waves. We recorded P-wave waveforms using a 20 KHz low pass filter.

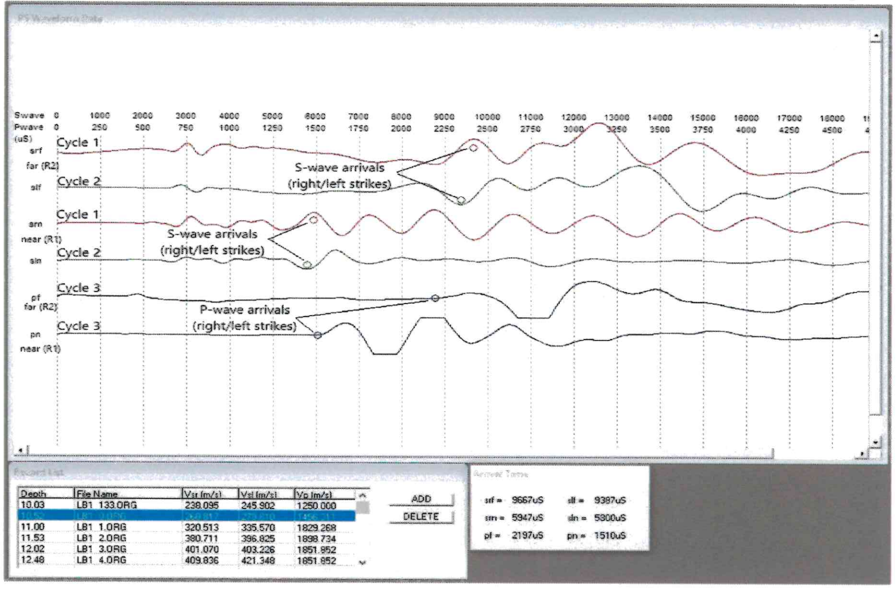
3.0 DATA ANALYSIS

3.1 SEISMIC RECORDS

Suspension P- and S-wave velocities were calculated with the computer software programs ***PSLogger Application*** Version 1.121 and ***PSLOG Analysis*** Version 1.0.001. Both programs are published by ***Robertson Geologging, Ltd.*** (2009). Sample suspension seismic records are presented in Figures A-2, -3, and A-4. These records display six seismic wave-traces. The upper four traces were detected by the horizontal receiver elements and are used to identify S-wave arrivals. These traces are labeled according to the wave type (S-wave=shear), the direction of impact (l=left and r=right) and the relative distance from the source (n=near and f=far). For example, the top wave trace is labeled "srf" because it was recorded to identify S-waves using a right strike and was detected by the far receiver. The seismic wave traces produced by a right-hand strike of the source (Cycle 1) are color coded red. Those resulting from a left-hand strike (Cycle 2) are color coded green.

The lower two traces were detected by the vertical receiver elements and are used to identify P-wave arrivals. These traces are labeled according to the wave type (P-wave = compressional P or primary) and the relative distance from the source (n=near and f=far). For example, the bottom wave trace is labeled "pn" because it was recorded to identify P-waves by the near receiver. These seismic wave traces are produced during Cycle 3 and are colored blue. Since they were detected by the vertical elements in the receivers, the direction of impact is inconsequential and is not addressed by separate waveforms.

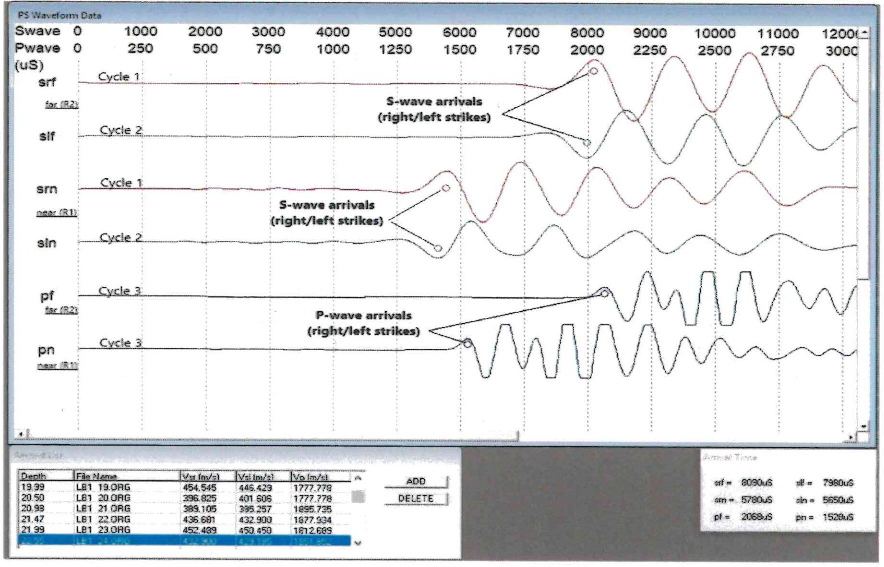
Figure A-2: Sample seismic record from LB-1 at depth ~10.5-m bgs.



Depth	File Name	Vsr (m/s)	Vsl (m/s)	Vp (m/s)
10.03	LB1 133.0RG	238.095	245.902	1250.000
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
11.00	LB1 1.0RG	320.513	335.570	1829.268
11.53	LB1 2.0RG	380.711	396.825	1898.734
12.02	LB1 3.0RG	401.070	403.226	1851.852
12.48	LB1 4.0RG	409.836	421.348	1851.852

srf = 9667uS sIf = 9387uS
srn = 5947uS sln = 5800uS
pf = 2197uS pn = 1510uS

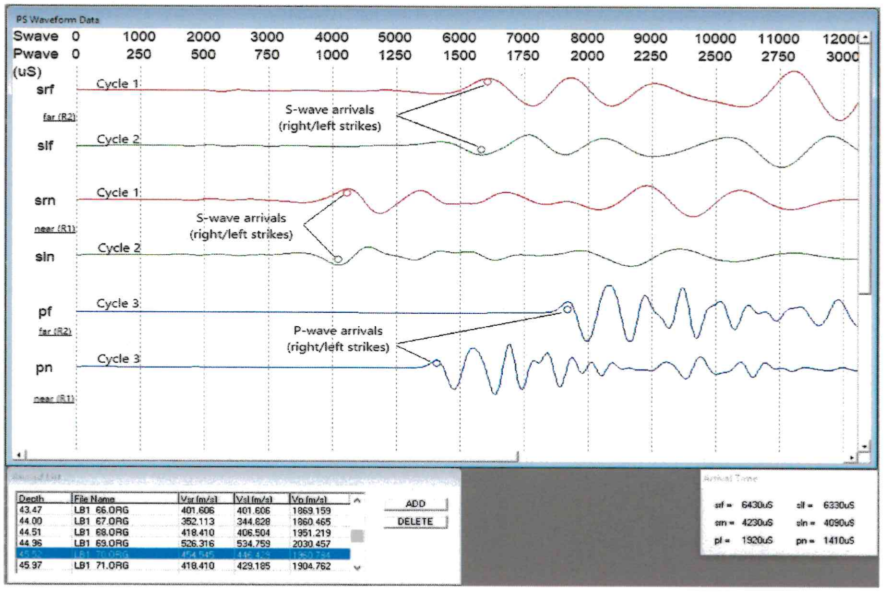
Figure A-3 Sample seismic record from LB-1 at depth ~22.6-m bgs



Depth	File Name	Vsr (m/s)	Vsl (m/s)	Vp (m/s)
19.99	LB1 19.0RG	454.545	446.429	1777.778
20.50	LB1 20.0RG	396.825	401.606	1777.778
20.98	LB1 21.0RG	389.105	395.257	1895.735
21.47	LB1 22.0RG	436.681	432.900	1877.934
21.99	LB1 23.0RG	452.489	450.450	1812.689
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]

srf = 8090uS slf = 7980uS
srn = 5780uS sln = 5650uS
pf = 2068uS pn = 1528uS

Figure A-4 Sample Waveform Record from LB-1 at a depth of ~45.5-m bgs



Depth	File Name	Vsr (m/s)	Vsl (m/s)	Vp (m/s)
43.47	LB1 66.ORG	401.606	401.606	1869.159
44.00	LB1 67.ORG	352.113	344.828	1860.465
44.51	LB1 68.ORG	418.410	406.504	1951.219
44.96	LB1 69.ORG	526.316	534.759	2030.457
45.52	LB1 70.ORG	454.545	446.429	1960.784
45.97	LB1 71.ORG	418.410	429.185	1904.762

srf = 6430uS slf = 6330uS
srn = 4230uS sln = 4090uS
pf = 1920uS pn = 1410uS

3.2 S-WAVE ARRIVALS

On the seismic records shown in Figures A-2, A-3 and A-4, the red traces (Cycle 1) were created by right strikes or impacts and the green traces (Cycle 2) were created by left strikes. Pairing the traces produced by opposite directions of impact reveals a phase reversal that is associated with the onset (arrival) of S-wave energy. However, because there can be slight discrepancies in timing between Cycle 1 and Cycle 2, the reversal point may not occur at the same exact time on both traces. Therefore, the onset of S-wave energy is further defined as the point where there is also a significant increase or decrease in amplitude within the phase reversal time window. These arrival times are depicted by open dots on the upper four wave traces.

3.3 P-WAVE ARRIVALS

P-wave arrivals are identified as the point where the wave traces produced by Cycle 3 (blue) change from straight lines to sinusoidal wave forms. These points are referred to as "first breaks". First breaks are either up or down and exceed a minimum threshold. Often it is less ambiguous to pick the first peak of the wavelet from each receiver (R1 and R2). The point of the break or peak are represented by blue circles on the lower two wave traces.

3.4 SEISMIC VELOCITY CALCULATIONS

Velocities are calculated by dividing the distance between the two receivers R1 and R2 by the difference in arrival times (AT in seconds) to those receivers. These are considered interval velocities. The general form of the equation is as follows:

$V_{(R2-R1)} = (X_{R2}-X_{R1})/(AT_{R2}-AT_{R1})$

Where $(X_{R2}-X_{R1})$ = 1 meter (3.2809 feet); AT_{R2} = arrival time at **far** detector, $AT_{R1 =}$ = arrival time at **near** detector.

The ***PSLogger Application*** program calculates two interval Vs velocities based on arrival time differences based on the right strike polarity of Cycle 1 (***Vsr***) records; the left strike polarity of Cycle 2 (***Vsl***) records and one ***Vp*** interval velocity based on the arrival time differences selected on Cycle 3 records.

3.5 INTERVAL SEISMIC VELOCITIES

Final output of velocities and arrival times were made with the reduction program ***PSLOG Analysis***. The output of this program consists of four interval velocities; three S-wave (Vs) and one P-wave (Vp). The interval Vs were computed using the Cycle 1 (VsRight) and Cycle 2 (VsLeft) travel times and their average (VsAvg). The interval Vp was computed using the Cycle 3 travel times.

4.0 DATA PRESENTATION

All P- and S-wave velocities interpreted for Borehole LB-1 are presented in Table 1 below and are differentiated into both Metric and Imperial Units. The columns below indicating metric units are shown on the left. The columns indicating imperial units are shown on the right and present the Vs and Vp data as depths in feet and velocities in feet per second. These average or interval Vs and Vp velocities have columns shaded blue and are displayed graphically on the depth versus velocity profile on Plate 1 within the main body of the report. The depths of the measurements are based on a probe reference point that is half-way between the near and far receivers.

Table 1, LB-01 PS-wave Velocity Table

METRIC UNITS DEPTHS & INTERVAL VELOCITIES			IMPERIAL UNITS DEPTHS AND INTERVAL VELOCITIES		
Depth	VsAvg	Vp	Depth	VsAvg	Vp
Meters	M/sec.	M/sec.	Ft./sec.	Ft./sec.	Ft./sec.
4.0	256.5	1587.3	13.3	841.2	5206.4
4.5	242.0	1652.9	14.8	793.7	5421.5
5.0	252.6	1526.7	16.4	828.5	5007.6
5.5	261.4	1530.6	18.0	857.2	5020.4
6.0	226.9	1578.9	19.6	744.2	5178.9
6.5	270.6	1648.4	21.3	887.5	5406.6
7.0	263.2	1630.4	22.9	863.3	5347.8
7.5	269.8	1522.8	24.5	885.1	4994.9
8.0	223.9	1692.5	26.2	734.3	5551.5
8.5	303.3	1548.4	27.9	994.9	5078.7
9.0	243.1	1581.0	29.6	797.4	5185.8
9.5	225.5	1202.4	31.1	739.7	3943.9
10.0	242.0	1250.0	32.9	793.8	4100.0
10.5	273.8	1456.3	34.5	898.1	4776.7
11.0	328.0	1829.3	36.1	1076.0	6000.0
11.5	388.8	1898.7	37.8	1275.2	6227.8
12.0	402.1	1851.9	39.4	1319.0	6074.1
12.5	415.6	1851.9	40.9	1363.1	6074.1
13.0	426.2	1851.9	42.6	1397.8	6074.1
13.5	375.0	1775.1	44.1	1230.0	5822.5
14.0	381.7	1775.1	45.9	1251.9	5822.5
14.5	442.5	1754.4	47.5	1451.3	5754.4
15.0	395.3	1754.4	49.1	1296.6	5754.4
15.5	406.6	1785.7	50.7	1333.6	5857.1
16.0	485.4	1860.5	52.4	1592.3	6102.3
16.5	469.5	1929.3	54.1	1539.9	6328.0
17.0	433.8	1834.9	55.8	1423.0	6018.3
17.5	443.5	1851.9	57.4	1454.7	6074.1
18.0	432.1	1801.8	59.0	1417.4	5909.9
18.5	392.2	1777.8	60.7	1286.5	5831.1
19.0	392.2	1860.5	62.4	1286.3	6102.3
19.5	432.0	1877.9	63.9	1416.9	6159.6
20.0	470.6	1843.3	65.5	1543.5	6046.1

METRIC UNITS DEPTHS & INTERVAL VELOCITIES			IMPERIAL UNITS DEPTHS AND INTERVAL VELOCITIES		
Depth	VsAvg	Vp	Depth	VsAvg	Vp
Meters	M/sec.	M/sec.	Ft./sec.	Ft./sec.	Ft./sec.
20.5	399.2	1777.8	67.2	1309.4	5831.1
21.0	392.2	1895.7	68.8	1286.4	6218.0
21.5	434.8	1877.9	70.4	1426.1	6159.6
22.0	451.5	1812.7	72.1	1480.8	5945.6
22.6	431.0	1851.9	74.0	1413.8	6074.1
23.0	431.2	1843.3	75.4	1414.5	6046.1
23.5	455.6	1843.3	77.0	1494.5	6046.1
24.0	422.8	1826.5	78.8	1386.9	5990.9
24.5	376.7	1793.7	80.3	1235.4	5883.4
25.0	398.4	1843.3	82.0	1306.8	6046.1
25.5	422.9	1877.9	83.5	1386.9	6159.6
26.0	424.7	1851.9	85.3	1392.9	6074.1
26.5	438.6	1886.8	86.9	1438.6	6188.7
27.0	445.4	1860.5	88.5	1461.0	6102.3
27.5	430.2	1826.5	90.2	1411.1	5990.9
28.0	392.9	1818.2	91.7	1288.8	5963.6
28.5	332.2	1818.2	93.4	1089.7	5963.6
29.0	316.5	1801.8	95.1	1038.0	5909.9
29.5	325.7	1834.9	96.7	1068.4	6018.3
30.0	299.0	1769.9	98.5	980.6	5805.3
30.5	240.1	1600.0	99.9	787.6	5248.0
31.0	219.6	1568.6	101.7	720.4	5145.1
31.5	255.4	1639.3	103.3	837.9	5377.0
32.0	297.7	1731.6	104.9	976.4	5679.7
32.5	344.2	1769.9	106.6	1129.1	5805.3
33.0	337.9	1694.9	108.2	1108.3	5559.3
33.5	266.0	1556.4	109.9	872.3	5105.1
34.0	213.2	1520.9	111.5	699.4	4988.6
34.5	202.8	1532.6	113.1	665.3	5026.8
35.0	208.1	1520.9	114.9	682.6	4988.6
35.5	213.0	1520.9	116.4	698.6	4988.6
36.0	215.8	1520.9	118.0	707.7	4988.6
36.5	219.3	1526.7	119.6	719.3	5007.6
37.0	219.1	1520.9	121.4	718.6	4988.6
37.5	229.6	1526.7	123.0	753.2	5007.6

METRIC UNITS DEPTHS & INTERVAL VELOCITIES			IMPERIAL UNITS DEPTHS AND INTERVAL VELOCITIES		
Depth	VsAvg	Vp	Depth	VsAvg	Vp
Meters	M/sec.	M/sec.	Ft./sec.	Ft./sec.	Ft./sec.
38.0	233.9	1538.5	124.6	767.3	5046.2
38.5	237.5	1556.4	126.2	779.1	5105.1
39.0	239.2	1556.4	127.9	784.7	5105.1
39.5	229.4	1574.8	129.5	752.3	5165.4
40.0	257.4	1619.4	131.2	844.3	5311.7
40.5	306.3	1769.9	132.8	1004.8	5805.3
41.0	422.8	1932.4	134.5	1386.9	6338.2
41.5	525.1	1913.9	136.0	1722.4	6277.5
42.0	561.8	1904.8	137.7	1842.8	6247.6
42.5	595.3	1941.7	139.5	1952.4	6368.9
43.0	517.2	1913.9	141.0	1696.5	6277.5
43.5	401.6	1869.2	142.6	1317.3	6130.8
44.0	348.5	1860.5	144.3	1143.0	6102.3
44.5	412.5	1951.2	146.0	1352.9	6400.0
45.0	530.5	2030.5	147.5	1740.2	6659.9
45.5	450.5	1951.2	149.3	1477.6	6400.0
46.0	423.8	1904.8	150.8	1390.1	6247.6
46.5	487.3	2030.5	152.5	1598.2	6659.9
47.0	547.5	2020.2	154.1	1795.8	6626.3
47.5	563.4	2040.8	155.8	1847.9	6693.9
48.0	552.8	1990.1	157.4	1813.0	6527.4
48.5	451.5	1886.8	159.2	1480.8	6188.7
49.0	417.6	1877.9	160.7	1369.7	6159.6
49.5	490.3	1895.7	162.2	1608.2	6218.0
50.0	476.2	1932.4	164.0	1561.9	6338.2
50.5	487.8	1923.1	165.8	1600.0	6307.7
51.0	510.3	1990.1	167.2	1673.9	6527.4
51.5	583.2	2020.2	168.9	1912.9	6626.3
52.0	639.0	2010.1	170.5	2095.9	6593.0
52.5	641.3	1951.2	172.2	2103.3	6400.0
53.0	502.2	1951.2	173.8	1647.1	6400.0
53.5	501.3	1951.2	175.4	1644.1	6400.0
54.0	573.1	1951.2	177.1	1879.8	6400.0
54.5	529.3	1932.4	178.8	1736.2	6338.2
55.0	515.5	1923.1	180.3	1690.7	6307.7

METRIC UNITS DEPTHS & INTERVAL VELOCITIES			IMPERIAL UNITS DEPTHS AND INTERVAL VELOCITIES		
Depth	VsAvg	Vp	Depth	VsAvg	Vp
Meters	M/sec.	M/sec.	Ft./sec.	Ft./sec.	Ft./sec.
55.5	489.1	1923.1	182.1	1604.2	6307.7
56.0	487.8	1951.2	183.7	1600.0	6400.0
56.5	456.6	1960.8	185.2	1497.7	6431.4
57.0	439.6	1941.7	186.9	1441.8	6368.9
57.5	365.7	1860.5	188.6	1199.4	6102.3
58.0	327.3	1666.7	190.3	1073.7	5466.7
58.5	296.3	1632.7	191.8	971.9	5355.1
59.0	290.7	1612.9	193.5	953.5	5290.3
59.5	297.2	1626.0	195.2	974.8	5333.3
60.0	325.2	1769.9	196.9	1066.8	5805.3
60.5	415.8	1960.8	198.5	1363.8	6431.4
61.0	489.2	2020.2	200.0	1604.7	6626.3
61.5	535.0	2072.5	201.7	1754.8	6797.9
62.0	555.6	2051.3	203.3	1822.2	6728.2
62.5	483.1	2030.5	205.0	1584.6	6659.9
63.0	442.5	1932.4	206.6	1451.4	6338.2
63.5	458.8	1970.4	208.3	1504.7	6463.1
64.0	482.0	2010.1	210.0	1580.8	6593.0
64.5	520.8	2040.8	211.5	1708.3	6693.9
65.0	522.2	2020.2	213.3	1712.8	6626.3
65.5	584.8	2030.5	214.9	1918.2	6659.9
66.0	511.5	1990.1	216.4	1677.8	6527.4
66.5	440.6	1834.9	218.1	1445.0	6018.3
67.0	331.1	1687.8	219.8	1086.1	5535.9
67.5	300.8	1612.9	221.4	986.6	5290.3
68.0	303.0	1612.9	223.0	994.0	5290.3
68.5	305.4	1659.8	224.7	1001.5	5444.0
69.0	346.1	1834.9	226.3	1135.1	6018.3
69.5	396.1	1960.8	228.0	1299.1	6431.4
70.0	409.8	1904.8	229.5	1344.3	6247.6
70.5	405.7	1980.2	231.2	1330.8	6495.0
71.0	456.7	2040.8	232.7	1497.8	6693.9
71.5	532.1	2061.9	234.5	1745.1	6762.9
72.0	554.0	2072.5	236.2	1817.2	6797.9
72.5	503.8	1980.2	237.8	1652.4	6495.0

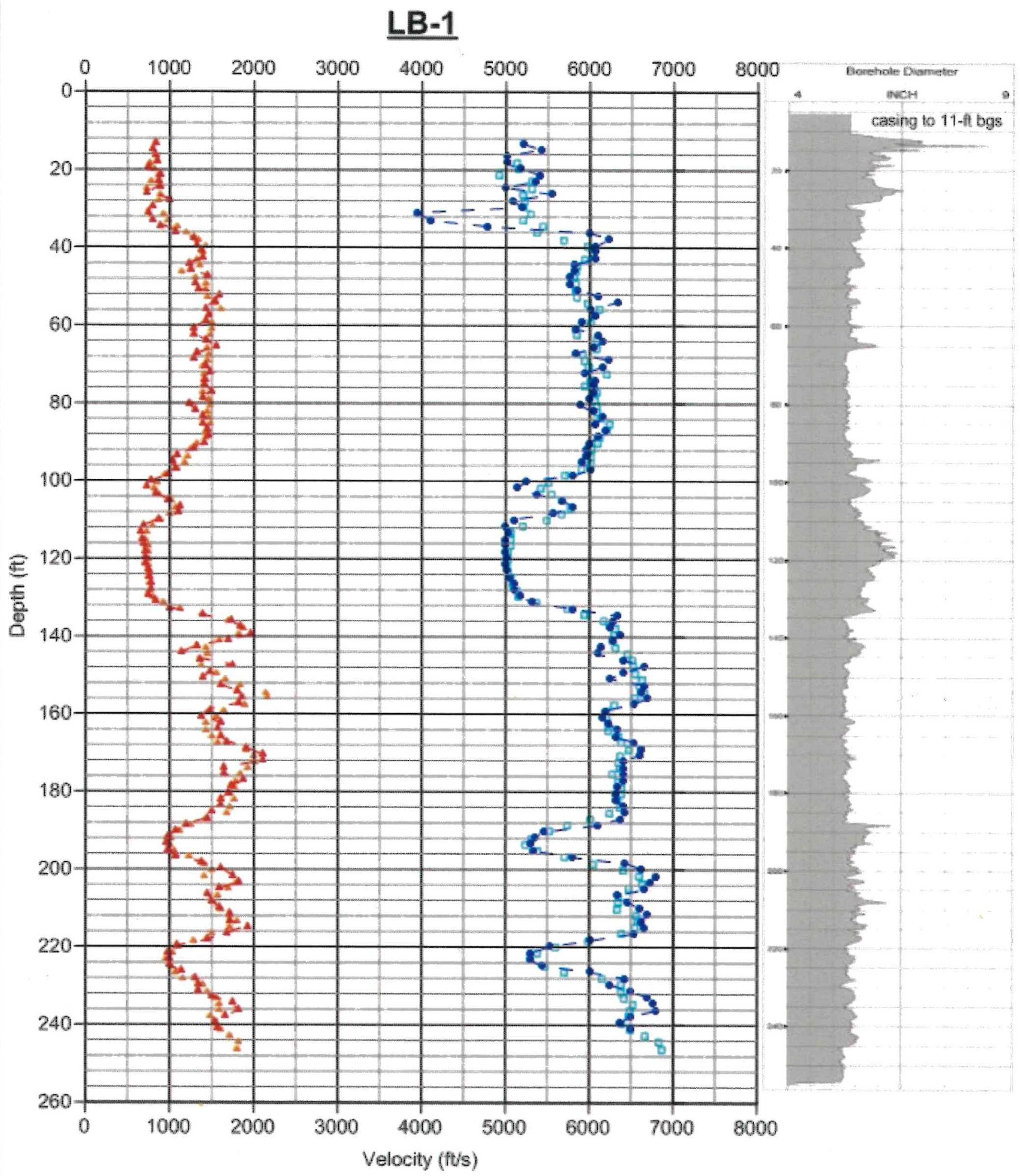
METRIC UNITS DEPTHS & INTERVAL VELOCITIES			IMPERIAL UNITS DEPTHS AND INTERVAL VELOCITIES		
Depth	VsAvg	Vp	Depth	VsAvg	Vp
Meters	M/sec.	M/sec.	Ft./sec.	Ft./sec.	Ft./sec.
73.0	464.4	1941.7	239.4	1523.4	6368.9
73.5	479.7	1980.2	241.1	1573.5	6495.0

5.0 QUALITY ASSURANCE

For quality assurance purposes, a source to receiver analysis is conducted. Depth verses interval velocities are displayed with velocities obtained from a source to near receiver (S-R1) analysis in Figure A-5 below. There is a 2.125 m separation between the source and near receiver. Therefore, the S-R1 velocities are an average velocity over 2.1 m rather than the average velocity over 1 m between the near and far receivers. The center point between the source and near receiver is 1.56 m lower on the probe than the mid-point between the two receivers. The S-R1 velocity for the record acquired at 10m, for example, is reported at a depth of 11.56 m. The source to receiver velocity is calculated with the following equation

$$V_{(S\text{-}R1)} = (X_S - X_{R1})/(AT_{R1} - SD)$$

where the S-R1 separation is 2.125m, AT_{R1} = arrival time at **near** detector and SD is the source delay which was 0.29 ms for this survey. The source delay varies from log to log but is influenced by the probe's internal electronics and amount of wear on the source springs.



SUSPENSION P- &
S-WAVE VELOCITY PROFILE with S-R1
BOREHOLE: LB-1

NORCAL

LOCATION: 469 Stevenson Street, San Francisco, CA

CLIENT: LANGAN

JOB #: NS205151

NORCAL GEOPHYSICAL CONSULTANTS INC.

Date: Dec., 2020

DRAWN BY: C. CARTER

APPROVED BY: CGC

FIGURE
A-5

APPENDIX D

SITE SPECIFIC RESPONSE SPECTRA

APPENDIX D

SITE-SPECIFIC RESPONSE SPECTRA

This appendix presents the details of our estimation of the level of ground shaking at the site during future earthquakes. We developed site-specific response spectra for two levels of ground shaking. These correspond to the Risk-Targeted Maximum Considered Earthquake (MCE_R) and Design Earthquake (DE) as defined in the 2019 California Building Code (CBC). Consistent with the provisions of ASCE 7-16 and Tall Building Initiative (TBI 2017, Version 2.03), we performed a Probabilistic Seismic Hazard Analysis (PSHA), deterministic analysis and ground response analysis to develop site-specific horizontal response spectra for two levels of shaking, corresponding to:

- Risk-Targeted Maximum Considered Earthquake (MCE_R), which corresponds to the lesser of two percent probability of exceedance in 50 years (2,475-year return period) or 84th percentile of the controlling deterministic event both considering the maximum direction as described in ASCE 7-16.
- Design Earthquake (DE), which corresponds to 2/3 of the MCE_R

D1.0 PROBABILISTIC SEISMIC HAZARD ANALYSIS

Because the location, recurrence interval, and magnitude of future earthquakes are uncertain, we performed a PSHA, which systematically accounts for these uncertainties. The results of a PSHA define a uniform hazard for a site in terms of a probability that a particular level of shaking will be exceeded during the given life of the structure.

To perform a PSHA, information regarding the seismicity, location, and geometry of each source, along with empirical relationships that describe the rate of attenuation of strong ground motion with increasing distance from the source, are needed. The assumptions necessary to perform the PSHA are that:

- the geology and seismic tectonic history of the region are sufficiently known, such that the rate of occurrence of earthquakes can be modeled by historic or geologic data
- the level of ground motion at a particular site can be expressed by an attenuation relationship that is primarily dependent upon earthquake magnitude and distance from the source of the earthquake
- the earthquake occurrence can be modeled as a Poisson process with a constant mean occurrence rate.

As part of the development of the site-specific spectra, we performed a PSHA to develop a site-specific response spectrum for 2 percent probability of exceedance in 50 years and for 50 percent probability of exceedance in 30 years. The PSHA spectra were developed using the

OpenSHA platform. The approach used in the PSHA is based on the probabilistic seismic hazard model developed by Cornell (1968) 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. The levels of shaking were estimated using ground motion prediction equations (attenuation relationships) that are primarily dependent upon the magnitude of the earthquake and the distance from the site to the fault, as well as the average shear wave velocity of the upper 30 meters, V_{S30}.

D1.1 Probabilistic Model

In probabilistic models, the occurrence of earthquake epicenters on a given fault is assumed to be uniformly distributed along the fault. This model considers ground motions arising from the portion of the fault rupture closest to the site rather than from the epicenter. Fault rupture lengths were modeled using fault rupture length-magnitude relationships given by Wells and Coppersmith (1994).

The probability of exceedance, $P_e(Z)$, at a given ground-motion, Z, at the site within a specified time period, T, is given as:

$$P_e(Z) = 1 - e^{-V(z)T}$$

where V(z) is the mean annual rate of exceedance of ground motion level Z. V(z) can be calculated using the total-probability theorem.

$$V(z) = \sum_i v_i \iint P[Z > z \mid m, r] f_{M_i}(m) f_{R_i|M_i}(r; m) dr\, dm$$

where:

v_i = the annual rate of earthquakes with magnitudes greater than a threshold M_{oi} in source i

$P\ [Z > z \mid m,r]$ = probability that an earthquake of magnitude m at distance r produces ground motion amplitude Z higher than z

f_{Mi} (m) and $f_{Ri|Mi}$ (r;m) = probability density functions for magnitude and distance

Z represents peak ground acceleration, or spectral acceleration values for a given frequency of vibration. The peak accelerations are assumed to be log-normally distributed about the mean with a standard error that is dependent upon the magnitude and attenuation relationship used.

D1.2 Source Modeling and Characterization

The segmentation of faults, maximum magnitudes, and recurrence rates were modeled using the data presented in the Uniform California Earthquake Rupture Forecast Version 3 (UCERF3) as detailed in the United States Geological Survey Open File Report 2013-1165. These and other faults of the region are shown on Figure 3 in the main text. Table D-1 presents the distance and direction from the site to the fault, mean moment magnitude, mean slip rate, and fault length for individual fault segments in UCERF3 source model. The mean moment magnitude presented on Table D-1 was computed assuming full rupture of the segment using Hank and Bakun (2008) relationship.

TABLE D-1
Source Zone Parameters

Fault Segment	Approx. Distance from fault (km)	Direction from Site	Mean Moment Magnitude[1]	Mean Slip Rate (mm/yr)	Fault Length (km)
San Andreas 1906 event	13.3	Southwest	8.1	17.2	464
Total Hayward-Rodgers Creek Healdsburg	17	East	7.6	7.3	213
Total San Gregorio	18	West	7.6	3.6	219
Pilarcitos	20	Southwest	6.7	0.7	51
Contra Costa (Lafayette)	29	East	6.1	0.8	8
Contra Costa Shear Zone (connector)	30	East	6.6	0.9	30
Franklin	31	Northeast	6.7	1.1	38
Contra Costa (Larkey)	32	East	6.0	0.8	8
Contra Costa (Dillon Point)	33	Northeast	6.1	0.7	11
Total Calaveras	33	East	7.5	8.0	186
Monte Vista - Shannon	34	South	7.0	0.8	60
Mount Diablo Thrust	34	East	6.6	1.6	25
Mission (connected)	35	East	6.1	0.8	28
Concord	39	East	6.4	3.4	18
Green Valley	41	Northeast	6.8	3.8	43
Contra Costa (Vallejo)	41	Northeast	5.6	0.6	4
Contra Costa (Lake Chabot)	42	Northeast	5.6	0.7	4
Clayton	45	East	6.4	0.7	16
West Napa	46	Northeast	6.8	1.3	44
Greenville	48	East	7.1	2.3	80
Bennett Valley	51	North	6.5	1.0	33
Butano	52	South	6.7	0.7	46
Great Valley 05 Pittsburg - Kirby Hills	53	Northeast	6.3	1.0	21
Great Valley 04b Gordon Valley	70	Northeast	6.6	0.9	28
Hunting Creek - Berryessa	73	North	6.7	4.3	44
Great Valley 07 (Orestimba)	77	East	6.8	0.5	66
Sargent	81	Southeast	6.8	1.7	57
Maacama	84	North	7.4	7.9	175
Great Valley 04a Trout Creek	92	Northeast	6.4	1.2	19
Monterey Bay-Tularcitos	98	Southeast	7.2	0.6	86

---

[1] Mean Moment Magnitude based on entire fault length rupturing using Hank and Bakun (2008)

D1.3 Attenuation Relationships

We understand the proposed building will have a basement and mat foundation that extends 46 to 52 feet below the ground surface. Based on the subsurface conditions, the site is classified as a stiff soil profile, Site Class D. Using the subsurface information including shear wave velocity measurements, we estimated the shear wave velocity of the upper 100 feet (30 meters), V_{S30}, is approximately 1,080 feet per second (330 meters per second). Furthermore, NGAW-2 database indicates that depths Z_1 and $Z_{2.5}$ are about 125 meters and 0.850 kilometer, respectively. These values were used in the development of site-specific spectra.

The Pacific Earthquake Engineering Research Center (PEER) embarked on the NGA-West 2 project to update the previously developed ground motion prediction equations (attenuation relationships), which were mostly published in 2014. We used the relationships by Abrahamson et al. (2014), Boore et al. (2014), Campbell and Bozorgnia (2014) and Chiou and Youngs (2014). These attenuation relationships include the average shear wave velocity in the upper 100 feet. These relationships were developed using the same earthquake database and hence equally credible, therefore, the average of the relationships (using equal weights for each attenuation relationship) is appropriate and was used to develop the recommended spectra.

The NGA relationships database includes the most up-to-date recorded and processed data. They were developed for the "average" (Rot_{D50}) horizontal components of spectral acceleration.

D2.0 PSHA RESULTS

Figure D-1 presents the Rot_{D50} results of the PSHA for the 2 percent probability of exceedance in 50 years hazard level (2,475-year return period) using the four relationships discussed above as well as the average of these relationships. These results were developed from OpenSHA Hazard Spectrum Application 1.5.2 (UCERF3 model).

ASCE 7-16 specifies the development of MCE_R site-specific response spectra in the maximum direction. Shahi and Baker (2014) provide scaling factors that modify the Rot_{D50} spectra to provide spectral values for the maximum response (maximum direction). We used the scaling factors presented in Table 1 of Shahi and Baker (2014) for ratios of $Sa_{RotD100}$/ Sa_{RotD50} to modify the average of the PSHA results for two percent probability of exceedance in 50 years. The maximum direction spectrum is also shown on Figure D-1.

Figure D-2 presents the deaggregation plots of the PSHA results for the 2 percent probability of exceedance in 50 years hazard level. From the examination of these results, it can be seen that the San Andreas fault dominates the hazard at the project site at different periods of interest.

D3.0 DETERMINISTIC ANALYSIS

We also performed a deterministic analysis to develop the MCE_R spectrum at the site. In a deterministic analysis, a given magnitude earthquake occurring at a certain distance from the source is considered as input into an appropriate ground motion attenuation relationship. On the basis of the deaggregation results, we developed deterministic spectra for a Moment Magnitude of 8.06 on the San Andreas fault at a distance of 13.3 kilometers from the site.

The same attenuation relationships and weighting factors as discussed in Section D1.3 were used in our deterministic analysis. Figure D-8 presents the 84th percentile deterministic results for the San Andreas scenario. The average of the four attenuation relationships for theRot_{D50} are also presented on that figure. Similar to the PSHA results, we developed the 84th percentile deterministic spectrum in the maximum direction using the Shahi and Baker (2014) ratios.

D4.0 RECOMMENDED SPECTRA

The MCE_R as defined in ASCE 7-16 is the lesser of the maximum direction PSHA spectrum having a two percent probability of exceedance in 50 years (2,475-year return period) or the maximum direction 84th percentile deterministic spectrum of the governing earthquake scenario and the DE spectrum is defined as 2/3 times the MCE_R spectrum. In addition, the MCE_R spectrum is defined as a risk targeted response spectrum, which corresponds to a targeted collapse probability of one percent in 50 years. The USGS Risk-Targeted Ground Motion calculator was used to determine the risk coefficients for each period of interest for the probabilistic spectrum. We used these risk coefficients to develop the risk-targeted PSHA spectrum.

Furthermore, we followed the procedures outlined in Chapter 21 of ASCE 7-16 and Supplement No. 1 to develop the site-specific spectra for MCE_R and DE. Chapter 21 of ASCE 7-16 requires the following checks:

- the largest spectral response acceleration of the resulting 84th percentile deterministic ground motion response spectra shall not be less than 1.5 × Fa where F_a is equal to 1.0.
- the DE spectrum shall not fall below 80 percent of Sa determined in accordance with Section 11.4.6, where F_a (1.0) and F_v (2.5) are determined using Table 11.4-1 and 11.4-2, respectively for site class D.
- The site-specific MCE_R spectral response acceleration at any period shall not be taken as less than 150 percent of the site-specific design response spectrum determined in accordance with Section 21.3.

Table D-2 presents digitized values of the site-specific spectra for the PSHA 2,475-year return period (max. dir.) and the 84th percentile deterministic (max. dir.). The largest spectral response acceleration of the 84th percentile deterministic response spectrum is 1.807g and is greater than 1.5×F_a (where F_a = 1.0 for Site Class D); therefore, no further scaling of the 84th percentile deterministic spectra was needed.

Figure D-4 and Table D-2 present a comparison of the site-specific spectra for the risk-targeted 2,475-year return period PSHA and the 84th percentile deterministic spectra in the maximum direction. In this case, the 84th percentile deterministic spectrum is less than or equal to the risk-targeted PSHA spectrum for a 2 percent probability of exceedance in 50 years (2,475 year return period) for periods less than or equal to 10.0 seconds. The lower of these two spectra should be used as the basis for the development of the MCE_R spectrum. The DE spectrum is defined as 2/3 times the MCE_R; however the DE spectrum should not be less than 80 percent of the DE code spectrum as determined using F_a equal to 1.0 and F_v equal to 2.5 (per Section 21.3 of ASCE 7-16). As shown on Figure D-4 and Table D-2 the DE spectrum is greater than or equal to 80 percent of the of the DE code spectrum for all periods less than 1.0 second, and is less for periods of 1.0 second and greater. 80 percent of the DE code spectrum is used for periods at and greater than 1.0 second.

TABLE D-2

Comparison of Site-specific and Code Spectra for Development of MCE$_R$ Spectrum per ASCE 7-16
Sa (g) for 5 percent damping

Period (sec.)	**Risk Targeted PSHA – 2,475-Year Return Period Max. Dir.**	**Deter-ministic 84th Percentile Max. Dir.**	**Lesser of PSHA and Deter-ministic (Initial MCE$_R$)**	**2/3 of Initial MCE$_R$ (Initial DE)**	**ASCE 7-16 - 80% DE per Section 21.3 Site Class D; F$_v$ = 2.50**	**Recommended Spectra**	
						DE	**MCE$_R$**
0.01	0.926	0.731	0.731	0.487	0.320	0.487	0.731
0.10	1.669	1.170	1.170	0.780	0.560	0.780	1.170
0.20	2.182	1.599	1.599	1.066	0.800	1.066	1.599
0.30	2.295	1.791	1.791	1.194	0.800	1.194	1.791
0.40	2.195	1.807	1.807	1.205	0.800	1.205	1.807
0.50	2.022	1.721	1.721	1.148	0.800	1.148	1.721
0.75	1.565	1.377	1.377	0.918	0.800	0.918	1.377
1.00	1.196	1.090	1.090	0.727	0.800	0.800	1.200
1.50	0.779	0.749	0.749	0.500	0.533	0.533	0.800
2.00	0.569	0.546	0.546	0.364	0.400	0.400	0.600
3.00	0.371	0.365	0.365	0.243	0.267	0.267	0.400
4.00	0.272	0.270	0.270	0.180	0.200	0.200	0.300
5.00	0.213	0.208	0.208	0.138	0.160	0.160	0.240
7.50	0.122	0.118	0.118	0.079	0.107	0.107	0.160
10.00	0.077	0.073	0.073	0.048	0.080	0.080	0.120

The recommended MCE$_R$ and DE spectra are presented in Table D-3 and on Figure D-5.

TABLE D-3

Recommended MCE_R and DE Spectra
Sa (g) for 5 percent damping

Period (seconds)	**MCE_R**	**DE**
0.01	0.731	0.487
0.10	1.170	0.780
0.20	1.599	1.066
0.30	1.791	1.194
0.40	1.807	1.205
0.50	1.721	1.148
0.75	1.377	0.918
1.00	1.200	0.800
1.50	0.800	0.533
2.00	0.600	0.400
3.00	0.400	0.267
4.00	0.300	0.200
5.00	0.240	0.160
7.50	0.160	0.107
10.00	0.120	0.080

Because site-specific procedure was used to determine the recommended response spectra, the corresponding values of S_{MS}, S_{M1}, S_{DS} and S_{D1} per Section 21.4 of ASCE 7-16 should be used as shown in Table D-4.

TABLE D-4

Design Spectral Acceleration Value

Parameter	**Spectral Acceleration Value (g's)**
S_{MS}	1.626
S_{M1}	1.200
S_{DS}	1.084[2]
S_{D1}	0.800[3]

---

[2] S_{DS} is based on the site-specific response spectra and is based on 90 percent of the maximum spectral acceleration within the period range of 0.2 to 5 seconds; it is governed by 90 percent of the spectral acceleration at a period of 0.4 seconds.

[3] S_{D1} is based on the site-specific response spectra and is the maximum of the product of period, T, and spectral acceleration, Sa, for periods from 1.0 to 5.0 seconds; it is governed by the product of the period and spectral acceleration at a period of 1.5 seconds.

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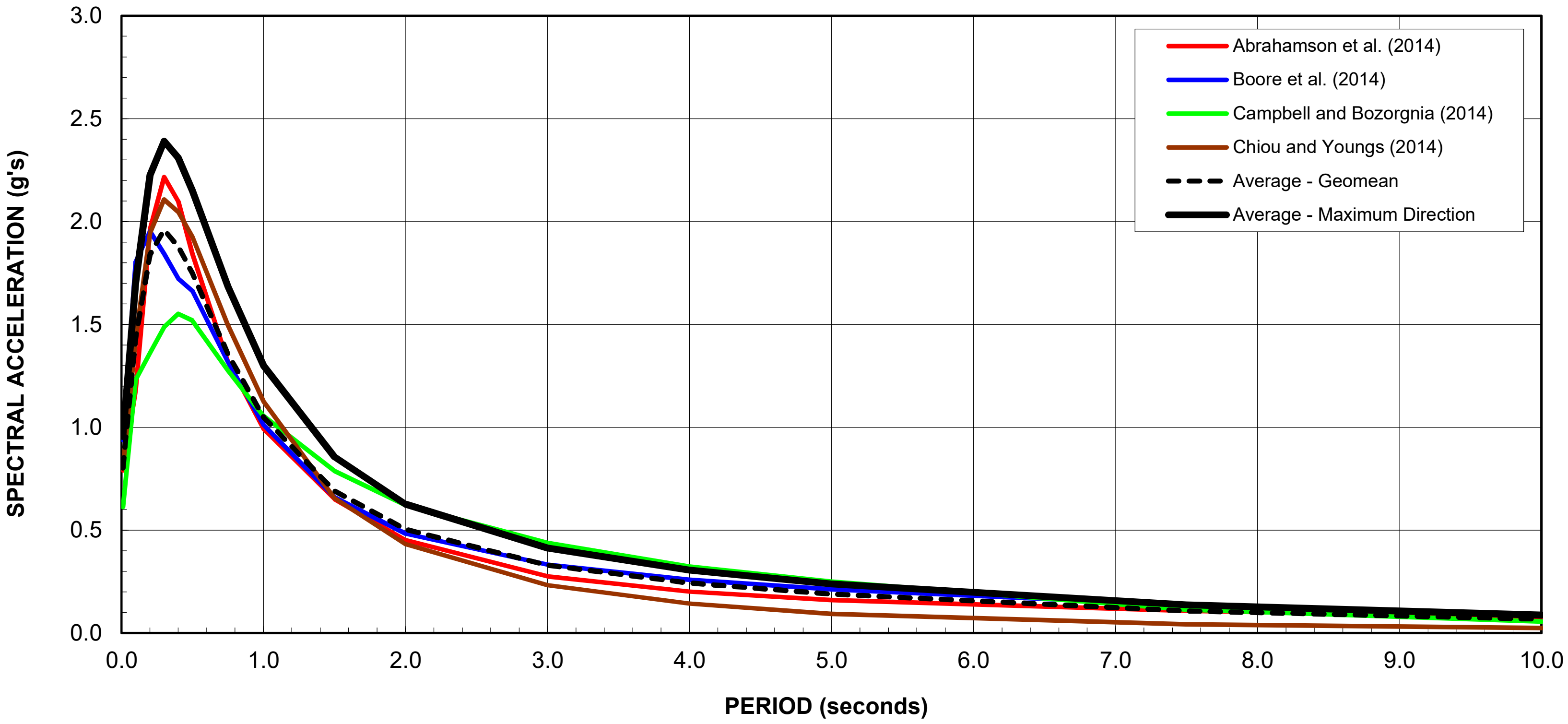

Notes

1. Damping Ratio = 5%
2. Estimated V_{S30} = 330 m/s
3. Maximum direction factors from Shahi and Baker (2014)
4. Z_1 = 125 m, $Z_{2.5}$ = 0.850 km

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Project

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SAN FRANCISCO
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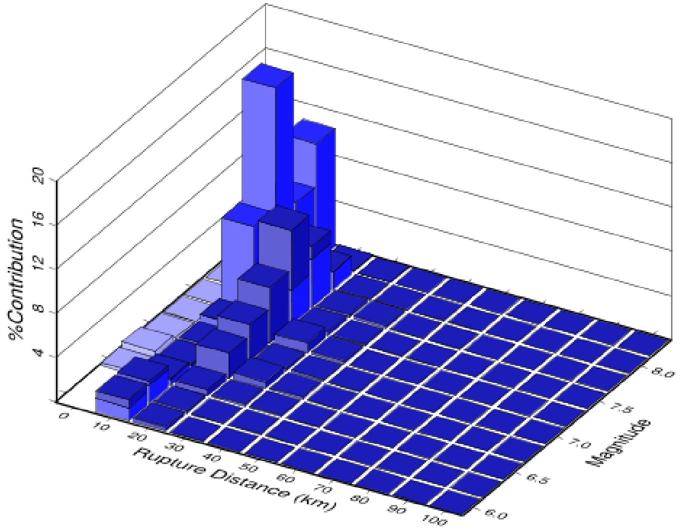
Figure Title

RESULTS OF PSHA, 2 PERCENT PROBABILITY OF EXCEEDANCE IN 50 YEARS

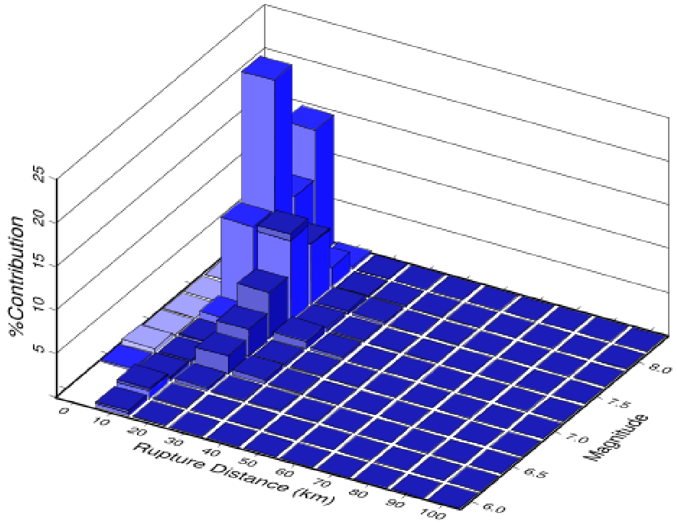
Project No.
731690402
Date
8/20/2021
Scale
AS SHOWN
Prepared By:
PDB

Figure No.

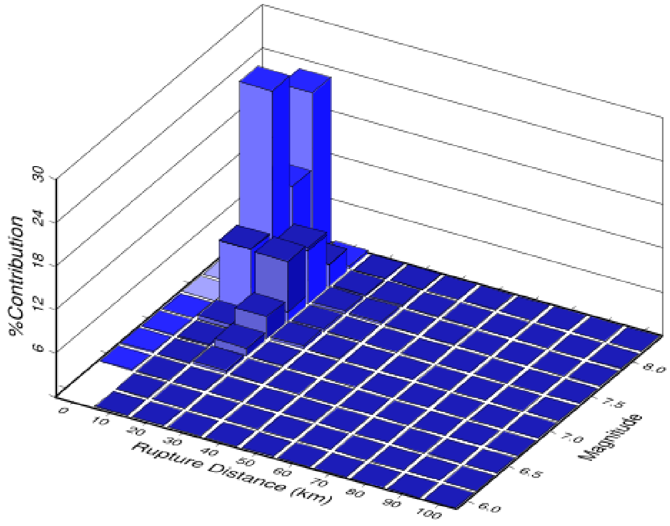
D-1



(a) PGA



(b) S_a, T = 1.0 second



(c) S_a, T = 4.0 seconds

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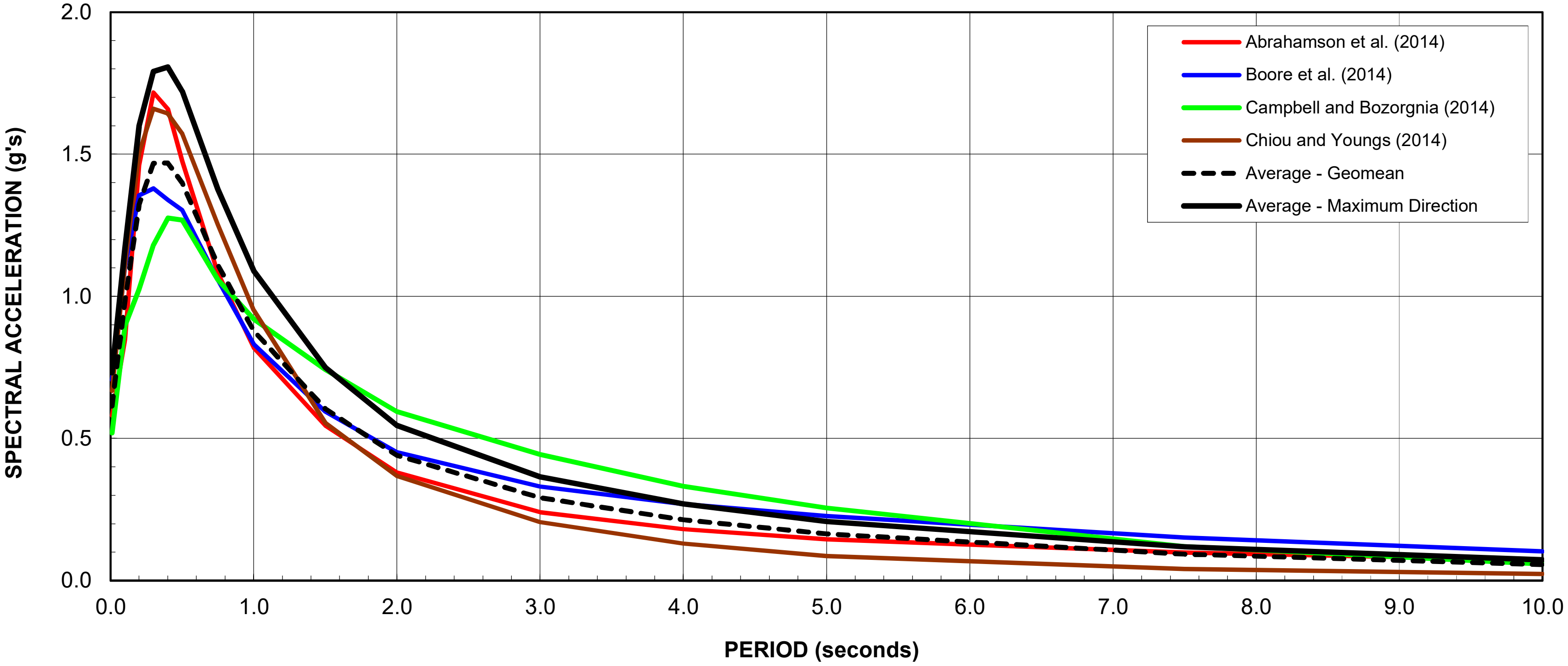
Figure Title

2% PROBABILITY OF EXCEEDANCE IN 50 YEARS - MAGNITUDE DISTANCE DEAGGREGATION PLOTS

Project No.
731690402
Date
8/20/2021
Scale
AS SHOWN
Prepared By:
PDB

Figure No.

D-2



Notes

1. Damping Ratio = 5%
2. Estimated V_{S30} = 330 m/s
3. Maximum direction factors from Shahi and Baker (2014)
4. Deterministic results are for a Moment Magnitude 8.06 occuring on the Northern San Andreas about 13.3 km from the site

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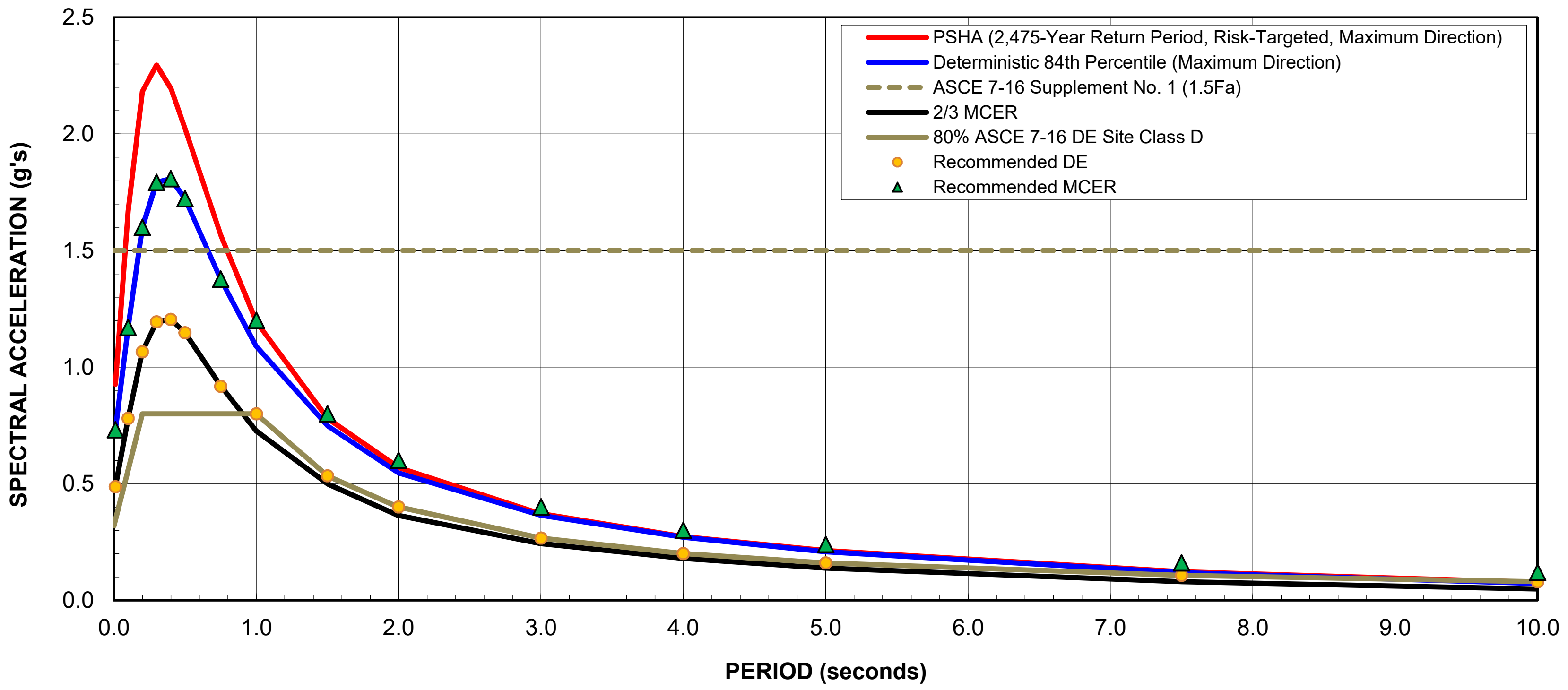
Figure Title

RESULTS OF DETERMINISTIC ANALYSIS - 84th PERCENTILE SPECTRA - MAXIMUM DIRECTION - NORTHERN SAN ANDREAS

Project No. 731690402
Date 8/20/2021
Scale AS SHOWN
Prepared By: PDB

Figure No.

D-3



Notes

1. Damping Ratio = 5%
2. Estimated V_{S30} = 330 m/s
3. Maximum direction factors from Shahi and Baker (2014)
4. Deterministic results are for a Moment Magnitude 8.06 occuring on the Northern San Andreas about 13.3 km from the site

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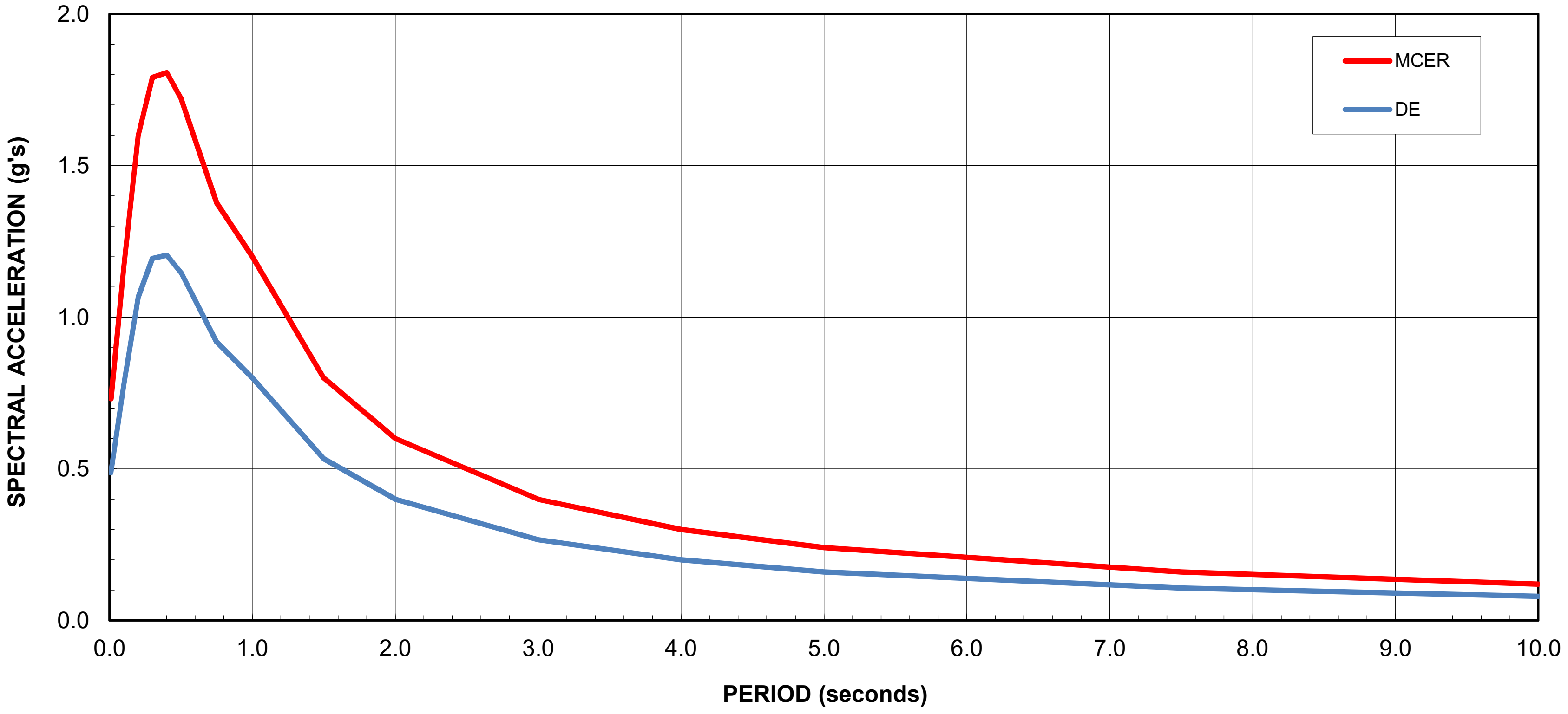
Project	**469 Stevenson Street** SAN FRANCISCO CITY & COUNTY CALIFORNIA
Figure Title	**COMPARISON OF PROBABILISTIC, DETERMINISTIC AND DETERMINISTIC LOWER LIMIT SPECTRA**
Project No.	731668503
Date	8/20/2021
Scale	AS SHOWN
Prepared By:	PDB
Figure No.	**D-4**



Notes

1. Damping Ratio = 5%
2. Estimated V_{S30} = 330 m/s

LANGAN 135 Main Street, Suite 1500 San Francisco, California 94105 T: 415.955.5200 F: 415.955.5201 www.langan.com LANGAN ENGINEERING AND ENVIRONMENTAL SERVICES, INC.	Project **469 Stevenson Street** SAN FRANCISCO CITY & COUNTY CALIFORNIA	Figure Title **RECOMMENDED HORIZONTAL SPECTRA**	Project No. 731690402 Date 8/20/2021 Scale AS SHOWN Prepared By: PDB	Figure No. **D-5**

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QUALITY CONTROL REVIEWER

Richard D. Rodgers

Richard D. Rodgers, GE
Senior Consultant

APPENDIX L2

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Technical Excellence
Practical Experience
Client Responsiveness

27 September 2022

Jenny Delumo, AICP (she/hers)
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Subject: Geotechnical Feasibility
469 Stevenson Street
San Francisco, California
Langan Project No.: 731690403

Dear Ms. Delumo:

This letter presents our conclusions regarding the geotechnical feasibility of the proposed development at 469 Stevenson Street in San Francisco. Our conclusions are based on (1) results of our preliminary geotechnical investigation report dated 30 June 2022, and (2) performance of existing developments near the project site with similar foundations (mats), foundation loads and subsurface conditions.

DESCRIPTION AND GEOTECHNICAL FEASIBILITY OF PROPOSED DEVELOPMENT

A summary of the proposed development, site subsurface conditions, and our conclusions regarding the geotechnical feasibility of the project and probable foundation is presented below.

Project Description

The proposed development addressed in our June 2022 report, includes a 27-story tower (approximately 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. Average dead plus live foundation pressures would be about 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. Assuming a 4-foot-thick mat for the podium and a 10-foot-thick mat for the tower, the excavation for the basement and mat would extend 46 to 52 feet below existing site grades.

The 2022 preliminary report is based on the results of our Phase 1 field investigation performed at the project site in 2020. The June 2022 report presents preliminary conclusions regarding the geotechnical aspects of the project including project feasibility from a geotechnical standpoint and feasibility of a mat for the support of the proposed structure; the report was not intended to meet requirements of Administrative Bulletin (AB) -111[1]. Because the project would be classified

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

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.

The Phase 1 field investigation performed in 2020 included drilling two borings to bedrock (depths of 250 and 265 feet below site grades), performing laboratory testing on representative soil samples, and performing suspension logging to measure soil shear wave velocities in one of the borings. We used this information to perform preliminary engineering analyses and to develop preliminary conclusions regarding the geotechnical aspects of the proposed development.

Subsurface Conditions and Mat Foundation

The site is outside of the historical Bay shoreline, locally referred to as the Sullivan Marsh and within the regional seismic hazards zones map. In general, the site is underlain by fill, Dune sand, Marsh deposit, Colma Formation, marine clay known locally as Old Bay Clay, alluvium/residual soil, and Franciscan Complex bedrock. The groundwater level within the project site could rise to within 16 feet from existing street grades, which corresponds to Elevation 1.7 feet[2].

The excavation for the proposed structure and mat would extend below the fill, Dune sand, Marsh deposit, and upper portion of the Colma Formation. The dense to very dense Colma Formation anticipated below depths of 37 to 38 feet bgs at the project site, is about 60 to 77½ feet thick, and is generally strong and relatively incompressible. The top of Colma Formation is near Elevation 4.4 to 2.4 feet; the bottom of Colma Formation is near Elevation -57.7 to -73.2 feet. The Colma Formation is underlain by a 24- to 37-foot thick layer of Old Bay Clay.

The proposed structure could be supported on a mat bearing on dense to very dense Colma Formation provided the settlement in the Old Bay Clay induced by the anticipated building loads is acceptable.

Geotechnical Feasibility

The results of our June 2022 preliminary investigation indicate construction of the proposed structure would be feasible from a geotechnical standpoint. Furthermore, the results of our preliminary settlement analyses indicate settlement under the static building loads would be on the order of 1 to 2 inches at the podium, and 2 to 3¾ inches at the tower portions of the structure, with differential settlement of up to 2.5 inches between the center and corners of the building footprint, and less than one inch of differential settlement between a lateral distance of 30 feet. These settlements, calculated to within the next 50 years, meet the settlement criteria included in AB-111. Per AB-111, "the total short-term and long-term computed settlement of the foundation under gravity and seismic loads should not exceed 4 inches."

The feasibility of a mat foundation should be confirmed with a Phase 2 field investigation and laboratory testing program, and additional engineering analyses using the results of the Phase 1

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[2] Elevations reference CCSF-VD13 NAVD88 Vertical Datum (CCSF-VD13). The historic SFCD is about 11.35 feet below the CCSF 2013 datum.

and Phase 2 investigations. If the supplemental field investigation and engineering analyses indicate a mat is not feasible, then deep foundations that extend to bedrock would be required to support the proposed structure.

Per Section 13 of AB-111, recommendations for geotechnical instrumentation and construction monitoring would be included in the design level geotechnical investigation report. In addition, settlement monitoring of the completed structure would be performed for a period of 10 years after construction per AB-111 Section 15, and submitted annually to DBI to confirm the estimated time rate of settlement of the building is not exceeded.

DEVELOPMENTS COMPLETED NEAR THE PROJECT SITE WITH SIMILAR FOUNDATION LOADS AND SUBSURFACE CONDITIONS

To support our conclusion regarding the project feasibility and feasibility of a mat foundation, we present two recent developments completed in the vicinity of the site, with similar foundation loads and subsurface conditions, supported on mats. In addition, we are presenting a summary of existing developments near the project site, with similar foundation loads and subsurface conditions that are also supported on mat foundations. The locations of the projects are shown on the attached site location map, Figure 1.

5M Development – M2 Tower – 434 Mina Street, San Francisco

The 220-foot tall (20-story) residential tower at 434 Mina Street is a concrete building with two basement levels (finished floor at Elevation 7.5 feet) that was completed in early 2022. It is supported on a mat that varies from 4- to 8-foot-thick that bears on improved soil that extends to dense to very dense Colma Formation. The Colma Formation extends to Elevation -66.5 to -75 feet, and is underlain by 56 to 61 feet of stiff to very stiff Old Bay Clay. The Old Bay Clay is underlain by alluvial/colluvium deposits (hard clay and very dense sand and gravel) to the top of bedrock, anticipated at 260 to 280 feet below street grades. Foundation pressures range from 8,000 psf at the tower and 4,100 to 5,900 psf for the podium. Long term settlement under the static building loads was calculated on the order of 2.5 to 3 inches, with differential settlement of up to 2 inches between the center and corners of the building footprint, and less than one inch of differential settlement between columns, which meets the AB-111 settlement criteria.

The building settlement is being monitored, and measured settlement is forwarded to the DBI; settlement last measured in September 2021 (95 percent of building load), was about 3/8 to ½ inch; settlement will be monitored for at least 10 years per City requirements.

5M Development – H1 Tower – 415 Natoma Street

The 395-foot (25-story) tall office tower at 415 Natoma Street is a steel frame building with two basement levels extending about 25 feet below street grades (finished floor near Elevation -2 feet), completed in February 2021. It is supported on a 6- to 9-foot-thick mat (bottom of mat extends to depths of 32 to 35 feet below street grades, corresponding to about Elevation -9 feet), bearing on soil improvement that extends to dense to very dense Colma Formation, or directly on competent Colma Formation. The Colma Formation extends to Elevation -62 to -69 feet, and is underlain by 75 to 88 feet of stiff to very stiff Old Bay Clay. The Old Bay Clay is underlain by alluvial deposits (hard clay and very dense sand) and colluvium (very dense sand and gravel) to the top of bedrock, at about 260 feet below street grades. Foundation pressures range from 5,000 psf at the tower to 3,000 psf for the remainder of the building footprint. Long term settlement under the static building loads was calculated on the order of 2.5 to 3 inches, with differential settlement of about 1.5 to 2 inches between the center and corners of the building footprint, and less than one inch of differential settlement between columns, which meets the AB-111 settlement criteria.

The geotechnical investigation and seismic studies for the 415 Natoma Street project were prepared per Administrative Bulletins AB-082[3] and AB-111 and were approved by the geotechnical member(s) of the Engineering Design Review Team (EDRT) assigned to the project by SFDBI during the review of the site permit.

The 350-foot-tall building is also being monitored for settlement and the data is forwarded to DBI. Settlement last measured in September 2021 was about ½ inch; settlement will be monitored for at least 10 years per City requirements.

Summary of Other Similar Existing Improvements

Existing improvement with similar or higher foundation pressures, supported on a mat bearing either directly on Colma Formation, or, on soil improvement extending into the Colma Formation are listed below:

- 39-story Marriott Hotel at 55 Fourth Street, completed in 1989
- 48-story Four Seasons Hotel at 757 Market Street, completed in 2001
- 39-story Intercontinental Hotel at 888 Howard Street, completed in 2008
- 40-story Paramount Rental Apartments at 680 Mission Street, completed in 2002

Settlement data is not available for the four structures listed above, because they were constructed prior to the 2019 AB-111.

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

CONCLUSIONS

On the basis of the results of our June 2022 preliminary geotechnical investigation report and performance of existing developments in the site vicinity with similar foundation loads and subsurface conditions supported on mats, we conclude the project is feasible from a geotechnical standpoint. In addition, our initial conclusion is that a mat would be feasible for the support of the proposed structure. However, the feasibility of a mat will need to be confirmed by a design level geotechnical investigation report approved by the EDRT assigned to the project by SFDBI during the review of the site permit. If you have any questions, please call.

Sincerely,
Langan Engineering & Environmental Services, Inc.

Peter Brady, GE
Senior Project Engineer

REGISTERED PROFESSIONAL ENGINEER
PETER D. BRADY
NO. GE3190
Exp. 9/30/24
GEOTECHNICAL
STATE OF CALIFORNIA

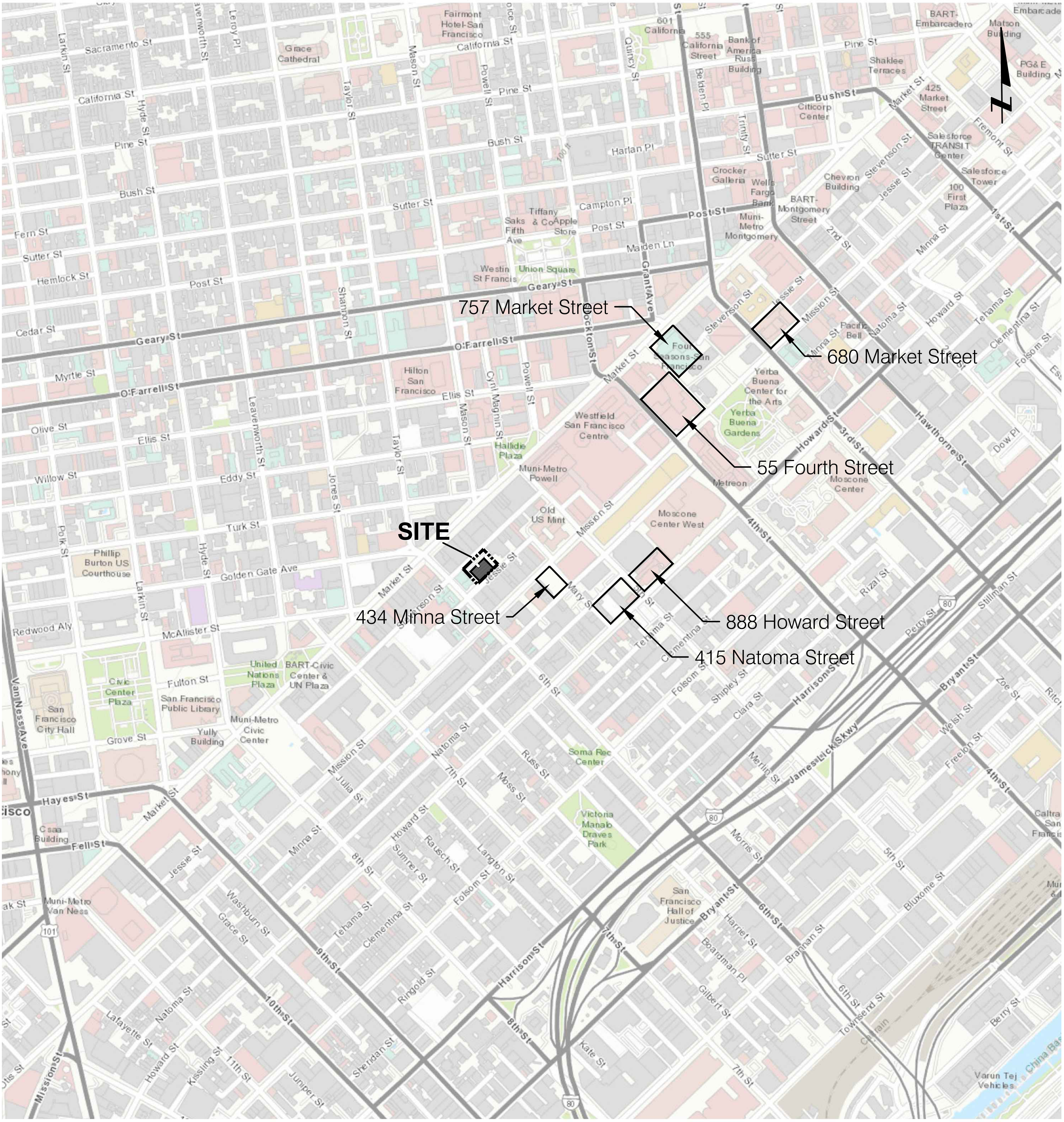
Maria G. Flessas

Maria Flessas, GE
Principal

REGISTERED PROFESSIONAL ENGINEER
MARIA G. FLESSAS
No. 2502
Exp. 9/30/2022
GEOTECHNICAL
STATE OF CALIFORNIA

Attachment: Figure 1, Site Location Map

FIGURE



Note:
Topographic base map is provided through Langan's Esri
Arc GIS software licensing and Arc GIS online,
National Geographic Society, i-cubed.

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Project	Figure Title	Project No.	Figure
469 STEVENSON STREET SAN FRANCISCO SAN FRANCISCO COUNTY CALIFORNIA	SITE LOCATION MAP	731690403	1
		Date 09/09/2022	
		Drawn By AG	
		Checked By PB	

Filename: \\langan.com\data\SF\data4\731690402\Project Data\CAD\02\2D-DesignFiles\Geotechnical\731690402-B-GI0101_SiteLocMap-FaultMap-MM Scale.dwg Date: 9/9/2022 Time: 15:23 User: jfrank Style Table: Langan.stb Layout: Figure 1 Site Loc

APPENDIX L3

AGDI

1205 Contra Costa Dr
El Cerrito, CA 94530
(510) 816-1323

October 27, 2022

City & County of San Francisco, Department of Building Inspection (SFDBI)
49 South Van Ness Avenue
San Francisco, California 94013

Attention: Mr. Willy Yau – Acting Manager, Plan Review Services Division

Subject: **Peer Review Letter – Geotechnical Peer Review Services Related to Foundation Design of the Proposed Highrise Building Located at 469 Stevenson Street in San Francisco, California.**

Dear Mr. Yau:

This letter presents the results of peer review services provided by Applied GeoDynamics, Inc. (AGDI) related to foundation design of the proposed high-rise building to be located at 469 Stevenson Street in San Francisco, California. Langan is the project geotechnical engineer of record.

PROJECT DESCRIPTION & UNDERSTANDING

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 buildings (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 are not available at this time.

We understand the proposed structure will include a 27-story tower (approximately 274 feet tall) with a 1- to 6-level podium. The structure will include a three-level basement that would extend beneath the entire site. The tower will occupy the majority of the site and will abut Jessie Street. 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.

The site is covered by 8 to 8.5 feet of sandy fill; 19 to 19.5 feet of loose to medium dense native sand; 6.5 to 10 feet of marsh deposit; and dense to very dense sand to a depth of 98 to 114.5 feet below ground surface (bgs). The loose to medium dense sandy fill, native sand, and marsh deposit, extend about 40 bgs. A 24 to 37 feet thick stiff sandy clay layer, locally referred to as Old Bay clay, is present below the dense to very dense sand. The Old Bay clay layer is underlain by alluvium consisting of dense to very dense sand and hard clay layers extending to bedrock. Bedrock of Franciscan Formation is about 250 feet below ground surface.

A proposed foundation system consists of a mat foundation placed directly at the surface of dense Colma sand layer about 40 feet bgs. The mat foundation will be 4 feet thick under the podium and 10 feet thick under the tower.

PEER REVIEW OBJECTIVE

The objective of the project was to render an opinion regarding the short-term and long-term (consolidation) settlement of the building and whether there would be a need to support the mat foundation on piles extending down to bedrock as potentially necessitated by the presence of Old Bay clay layer between the dense Colma sand layer and the alluvium. It is noted that excessive settlement of the Millennium Tower (i.e., 301 Mission Street building) in San Francisco caused by consolidation of the Old Bay clay layer has prompted the engineering community and building officials to pay extra attention to long-term settlement of shallow foundations where Old Bay clay layer is present.

More specifically, the objective of the peer review was to answer the following questions: Based on AGDI's professional opinion and on available information:

1. Does AGDI agree with the conclusions reached in the preliminary geotechnical report about the feasible foundation type for the proposed project at 469 Stevenson Street?
2. Could the geotechnical issues that occurred at 301 Mission Street building (Millennium Tower) occur at this site?

SCOPE OF WORK

AGDI's scope of services included the following tasks:

1. Review of Langan's preliminary geotechnical report titled "Preliminary Geotechnical Study, 469 Stevenson Street, San Francisco, California", dated June 30, 2022;
2. Review of Langan's report titled "Scope of Geotechnical Services, Seismic Studies, Design Level Geotechnical Investigation, 469 Stevenson Street, San Francisco, California", dated August 31, 2022, with respect to number and depth of borings / cone penetration tests, drilling and sampling method, number and type of laboratory tests, and approach to settlement calculation / foundation design;
3. Review of foundation loads during static loading conditions;
4. Performing simplified (hand calculation) of short-term and long-term foundation settlement;
5. Discussion with the design team and review of their short-term and long-term foundation settlement calculations; and
6. Preparing this peer review letter.

DISCUSSION

We understand that (a) top of 10-foot mat foundation under the tower will be 35 feet below the existing ground surface, (b) 25 feet of post-construction hydrostatic uplift pressure will act against the bottom of the mat foundation, and (c) static gravity load applied to the foundation is about 7,000 psf.

Based on the results of consolidation tests performed as part of preliminary geotechnical investigation, it appears that the past pressure, P_P, within Old Bay clay ranges from 15 ksf to 27.4 ksf with an average value of 18.8 ksf with an OCR (over-consolidation ratio) ranging from 1.63 to 2.11 with an average value of 1.92.

The post construction pressure applied to the center of Old Bay clay layer is estimated to be on average about 65 percent of the past pressure, and as such, the settlement within Old Bay clay will be controlled by recompression index which ranges from 0.04 to 0.08 with an average value of 0.06. The average recompression index is six times lower than the average compression index ranging from 0.33 to 0.44 with an average value of 0.38 based on laboratory tests performed. We note that the moisture content and average recompression index of Old Bay clay at this site is somewhat higher than those of typical Old Bay clay layers in the area.

Due to the presence of dense and thick Colma sand layer, the ground heave associated with rebound caused by removal of 40 feet of soil expected to be fairly small.

Based on the above parameters and results of simplified (hand) calculation, the post-construction, long-term (consolidation) foundation settlement is expected to fall well within values generally accepted from foundation and structural design point of view.

SUMMARY OF FINDINGS

On the basis of our review of material / information provided including:

- Preliminary geotechnical report,
- Proposed scope of geotechnical services during final geotechnical studies,
- Proposed foundation system which consists of 4 to 10 feet thick mat placed directly at the surface of dense Colma sand layer about 40 feet below the existing ground surface,
- Foundation gravity loads of about 7,000 pounds per square feet under the tower,
- Subsurface conditions including thickness and compressibility (past pressure, over-consolidation ratio, compression index, and recompression index) of Old Bay clay layer below dense Colma sand layer, and
- Discussion with Maria Flessas of Langan on October 6, 2022,

we judge that the proposed foundation system will have an acceptable settlement as predicted by the geotechnical engineer of record based on our independent, simplified (hand) calculation of short-term and long-term (consolidation) settlement provided that the thickness and compressibility of Old Bay clay layer to be identified through additional borings, sampling, and testing during the final phase of geotechnical investigation will prove to be consistent with those of subsurface conditions at the existing boring locations.

Furthermore, we judge that (a) considering the magnitude of gravity loads applied to the mat foundation at this site, (b) unloading which will occur due to 40 feet of excavation / soil removal, (c) hydrostatic uplift pressure after termination of dewatering, and (d) limited thickness of Old Bay clay layer at this site, large / unacceptable settlements observed at the Millennium Tower will not occur at this site. While deep foundation extending down to bedrock would have been an appropriate foundation system for the Millennium Tower, it won't necessarily be an appropriate foundations system for this project. This is because depending on type / number of piles and axial load on each pile, the elastic shortening of deep foundation may fall within the same range as that of the proposed shallow foundation placed directly at the surface of dense Colma sand layer. This is to say that the use of deep foundations extending to bedrock would not necessarily result in less settlement than a mat foundation supported directly at the surface of dense Colma sand layer.

In summary, it is our opinion that the proposed foundation system and plans for final geotechnical investigation are in compliance with the guidelines provided in the SFDBI AB-111.

It is noted that AGDI's conclusions were solely based on project-specific information and didn't use any information / data from other projects in San Francisco including those related to the Millennium Tower design, litigation, or foundation retrofit.

SUGGESTED LABORATORY TESTING DURING FINAL PHASE OF GEOTECHNICAL INVESTIGATION

It is suggested that two complete sets of laboratory tests be performed at each of the two proposed boring locations during the final phase of geotechnical investigation. Each set would include (a) two consolidation tests with unloading performed at the level of past pressure, (b) consolidated triaxial tests (TXCU) on samples taken from 1/3 and 2/3 of Old Bay clay layer, and (c) moisture-density tests.

Pocket Penetorameter and Field Torvane tests may be used for field classification and rough estimate of consistency of cohesive soil, but values obtained from these tests should not be used in engineering analysis.

Furthermore, considering geometry and subsurface conditions present at this site, we concur that settlement calculations based on SETTLE 3D program would be appropriate.

The structural design of mat foundation should use a long-term modulus of subgrade reaction considering long-term (consolidation) settlement of the foundation under sustained gravity loads.

LIMITATION OF PEER REVIEW

AGDI's scope of peer work was limited to evaluation of short-term and long-term (consolidation) foundation settlement and didn't include geotechnical review of seismic design parameters, shoring, dewatering, underpinning of adjacent foundations (if required), or impact of construction on adjacent buildings, streets, and other improvements.

CLOSURE

We appreciate the opportunity to provide geotechnical peer review services to SFDBI on this project. If you have any question regarding this report, please call us at (510) 816-1323.

Sincerely,

APPLIED GEODYNAMICS, INC.

Shahria-Vahdan

Shahriar Vahdani, Ph.D., P.E.
Senior Consultant

SV:ap

Copies Submitted:	Addressee
	Ms. Maria Flessas (Langan)
	Ms. Jenny Delumo (San Francisco Planning Department)
	Ms. Anna Radonich (Stantec)
	Ms. Kaela Johnson (Stantec)