GEOTECHNICAL ENGINEERING
AND GEOLOGIC HAZARDS REPORT
FRONT STREET HOTEL
1180 FRONT STREET
MORRO BAY, CALIFORNIA
APN 066-034-015

December 9, 2025

Prepared for

Mr. Don Daniels

Prepared by

Earth Systems Pacific
4378 Old Santa Fe Road
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December 9, 2025

FILE NO.: 307453-001

Mr. Don Daniels
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PROJECT: FRONT STREET HOTEL
1180 FRONT STREET
MORRO BAY, CALIFORNIA
APN 066-034-015

SUBJECT: Geotechnical Engineering and Geologic Hazards Report

CONTRACT REF: Proposal to Provide a Geotechnical Engineering Investigation and a Geologic Hazards Assessment, Front Street Hotel, 1180 Front Street, Morro Bay, California, APN 066-034-015, by Earth Systems Pacific, Doc. No. SLO-2504-067.PRP, dated April 30, 2025

Dear Mr. Daniels:

In accordance with your authorization of the referenced proposal, this geotechnical engineering and geologic hazards report has been prepared for use in the development of plans and specifications for the Front Street Hotel project in Morro Bay, California. Preliminary geotechnical recommendations for site preparation, grading, utility trenches, mat slab foundations, exterior pedestrian flatwork, retaining walls, drainage and maintenance, and observation and testing are presented herein. An assessment of the geologic hazards on the site and their potential effects on the proposed project was also completed. Digital copies of this report have been furnished to you and to Mr. Joey Cox of RRM Design Group. Additional digital copies can be forwarded to others as requested.

We appreciate the opportunity to have provided services for this project and look forward to working with you again in the future. If there are any questions concerning this report, please do not hesitate to contact the undersigned.

Sincerely,
Earth Systems Pacific

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APPENDICES

1.0 INTRODUCTION AND SITE SETTING

Based on information provided by Mr. Joey Cox of RRM design Group (RRM 2025), we understand that the planned Front Street Hotel in Morro Bay, California, will be a ten-unit, two-story hotel, with landscape, exterior pedestrian flatwork and other typical on-site and off-site improvements. The general location of the project is shown on Figure 1 – Site Vicinity Map in Appendix A. The structure will have a footprint of approximately 2,600 square feet; we have assumed that it will be of cast-in-place concrete, masonry, structural steel and/or stud construction. Conventional shallow continuous and spread (pad) foundations are desired, with slabs-on-grade for the lower level. Maximum isolated loads of 50 kips and wall (line) loads on the order of 3 kips per foot have been assumed. Common deck spaces will be provided at the rear of the upper units, and access to the upper units will be by stairways and an elevator at the rear of the building. Retained cuts on the order of 12 feet tall will be made at the rear of the building and fills of 1 to 2 feet will be needed at the front. The site will be served by the existing municipal utility lines in the area. Drainage will be by sheet flow to the existing improvements to the west. We understand the project will not utilize Low Impact Development/Best Management Practices (LID/BMPs) for stormwater control, and that infiltration testing for such improvements will not be needed.

The approximate center of the site is at latitude 35.3700°N, and longitude 120.8546°W, and at an elevation of approximately 14 feet MSL (Google Earth 2025). The west half (approximately) of the site slopes gently toward the west, with only 2 to 3 feet of total elevation change. The east half slopes upward at an estimated gradient of 3:1 (horizontal to vertical), with a total slope height of approximately 25 feet. Developed residential lots are to the east at the top of the slope, and an asphalt concrete paved public parking area, Front Street and The Embarcadero are to the west. A single-story hotel is on the commercial property to the south, and to the north is the right-of-way for Surf Street, and a public restroom facility. At the time of our field investigation, the site was covered with low vegetation and shrubs; the preliminary plans (RRM 2025) indicate that the two mature trees at the southwest corner of the site will remain. The locations and dispositions of utility lines on the site are unknown.

2.0 SCOPE OF SERVICES

The scope of work for this report included the following: a field reconnaissance by a registered geotechnical engineer and a certified engineering geologist; subsurface exploration; geotechnical laboratory testing of samples obtained during the field investigation; geotechnical and geologic analyses of the data; and preparation of this report. The analysis and subsequent recommendations were based on preliminary information (RRM 2025) provided.

This report and geotechnical recommendations are intended to comply with the considerations of Sections 1803.1 through 1803.7, J104.3 and J104.4, as applicable, of the CBC (CBSC 2022); and common geotechnical engineering and engineering geology practice regarding Coastal Zone developments in the City of Morro Bay under similar conditions at this time. The geotechnical test procedures were accomplished in general conformance with the standards noted, as modified by common geotechnical engineering practice in this area under similar conditions at this time.

Geotechnical recommendations for site preparation, grading, utility trenches, mat slab foundations, exterior pedestrian flatwork, retaining walls, drainage and maintenance, and observation and testing are provided in this report. This report also describes the general geologic characteristics, identifies existing and potential geologic hazards, and discusses the impacts the geologic conditions may have upon the project. It is our intent that this geotechnical/geologic report be used exclusively by the client to form the geotechnical/geologic basis of the design, and to guide the preparation of the project's plans and specifications. Application beyond this intent is strictly at the user's risk.

This report does not address dewatering and other issues in the domain of contractors such as, but not limited to, site safety, loss of volume due to stripping of the site, shrinkage of soils during compaction, excavatability, shoring, temporary slope angles, construction means and methods, etc. Analyses of the soil for mold potential, asbestos in man-made products, lead, radioisotopes, hydrocarbons, or chemical properties (other than geotechnical corrosivity) are beyond the scope of this report. Ancillary features such as temporary access roads, and non-structural fills are not within our scope and are also not addressed.

As there may be unresolved geotechnical and/or geologic issues with respect to this project, the geotechnical engineer and the engineering geologist should be retained to provide consultation as the design progresses, to review project plans as they near completion to assist in verifying that pertinent geotechnical and geologic issues have been addressed and to aid in conformance with the intent of this report. In the event that there are any changes in the nature, design, or location of improvements, or if any assumptions used in the preparation of this report prove to be incorrect, the conclusions and recommendations contained in this report shall not be considered valid unless the changes are reviewed and the conclusions of this report are verified or are modified in writing. The criteria presented in this report are considered preliminary until such time as any peer review or review by any jurisdiction has been completed, conditions are observed by the geotechnical engineer and/or engineering geologist in the field during construction, and the recommendations have been verified as appropriate or modified in writing.

3.0 FIELD INVESTIGATION

Three exploratory borings were drilled at the site on November 6, 2025, to assess the subsurface conditions. One boring was drilled with 4-inch outside diameter solid stem hand augers, to a maximum depth of approximately 10 feet below the existing ground surface (bgs). The other two borings were drilled to maximum depths of approximately 24.5 and 51.5 feet bgs, using a Gtech GT-8 truck-mounted drill rig, equipped with 6-inch outside diameter hollow stem auger and an automatic trip hammer for sampling. The approximate locations of the borings are shown on Figure 2 – Exploration Location Map in Appendix A. As the rig borings were drilled, soil samples were obtained using a ring-lined barrel sampler (ASTM D 3550-17 with shoe similar to D 2937-24). Bulk soil samples were also obtained from the auger cuttings in all borings. The borings were backfilled per San Luis Obispo County Department of Environmental Health requirements.

Subsurface conditions encountered in the borings were categorized and logged in general accordance with the Unified Soil Classification System and ASTM D 2488-17. Copies of the boring logs and a Boring Log Legend are included in Appendix A. In reviewing the boring logs and legend, the reader should recognize that the legend is intended as a guideline only, and there are a number of conditions that may influence the soil characteristics as observed during drilling. These include, but are not limited to, the presence of, cementation, variations in soil moisture, presence of groundwater, and other factors. Consequently, the logger must exercise judgment in interpreting the subsurface characteristics, possibly resulting in soil descriptions that vary somewhat from the legend.

4.0 LABORATORY ANALYSIS

Selected ring samples obtained from the borings were tested for unit weight and moisture (ASTM D 2937-17, modified for ring liners and ASTM D 2216-19). Ring samples were also tested for cohesion and angle of shearing resistance (ASTM D 3080-11), and percentage passing a No. 200 sieve (ASTM D 1140-17). A bulk sample was tested for expansion index (ASTM D 4829-21). The geotechnical laboratory test results are presented in Appendix B.

5.0 GENERAL SUBSURFACE AND GEOLOGIC PROFILE

The subsurface conditions encountered in Boring 1 (which was performed within the sloped eastern portion of the site) consisted of loose to medium dense poorly graded sand (dune sand) to the maximum depth explored of approximately 10 feet bgs. Fill consisting of poorly graded sand with variable amounts of clay was encountered in the upper 15 feet (approximately) of Borings 2 and 3. Underlying the fill in these two borings were paralic estuarine deposits consisting of poorly graded sand with variable amounts of clay and gravel. The fill was logged as being loose to medium dense, and the paralic estuarine deposits were medium dense to very dense. The soils were also described during drilling as being slightly moist to wet. Groundwater was encountered in Boring 2 at 15 feet bgs, and it was logged at 10 feet bgs upon completion of drilling. In Boring 3, groundwater was encountered at 10 feet bgs, during and after drilling. Groundwater was not encountered in Boring 1.

Please refer to the boring logs in Appendix A for a more complete description of the subsurface conditions encountered.

6.0 GEOLOGY

Geologic Setting

Regionally, the site is between the Santa Lucia Range to the north and Irish Hills to the south, within the Coast Ranges geomorphic province of California (CGS 2002). The northern and southern coast ranges are separated by a depression containing the San Francisco Bay. The Santa Lucia Range is bounded by the Pacific Ocean to the west and the Salinas River to the east and extends south to the Cuyama River (USGS 2025a). Tectonically, the region is dominated by northwest-trending faults, which include the Los Osos, San Luis Range, Oceanic-West Huasna, Rinconada, and Hosgri-San Simeon faults (Lettis and Hall 1994). The Los Osos fault is the closest known *Holocene Active* fault to the site.

Locally, the site is located at the northern end of Morro Bay, an estuary and salt marsh, within the larger Estero Bay, which extends from Point Estero in the north to Point Buchon in the south. The Morro Bay estuary and harbor have been created by a sand spit extending from the south northward towards Morro Rock. The estuary is fed by two primary fluvial systems, Los Osos and Chorro Creeks, which deliver sediments that have formed the salt marsh. The site is at the transition from the historical coastal bluff, which is now just a hill at the eastern side of the property; it is comprised of older eolian deposits (wind-blown sand/sand dunes, map symbol Qoe of Wiegers), similar to the sand spit that forms the western limit of Morro Bay. The western, generally level, part of the site is located upon “reclaimed” land that was created during the late 1940s as the United States Army Corps of Engineers (USACE) improved the harbor by constructing the breakwater that extends south from Morro Rock and filling (creating) and/or stabilizing a strip of land along the northeastern part of the bay that became the location of the future power plant, the Embarcadero, and Tidelands Park. At the site the dredged fill appears to be about 15 feet thick. Although this strip of land is mapped as young alluvial flood plain deposits (map symbol Qya of Wiegers), it is clear from historical aerial photographs that the strip of land described above did not exist in 1937 (USDA 1937) and from archived writing that Morro Rock was an isolated land mass protruding from the Pacific Ocean prior to construction of the breakwater extending northwest from Morro Rock to the mainland by USACE in 1933 (Gates and Bailey 1982), and therefore should have been mapped as artificial fill (map symbol af of Wiegers). With this exception, the regional geology depicted on the Regional Geologic Map and Legend, presented as Figures 3a and 3b, respectively, in Appendix C is generally consistent with what was observed in our borings.

Faulting

Faults are classified by the State of California based on the likelihood of generating ground motions and surface rupture. The classification system applies to known faults that have been compiled by numerous researchers through various methods of investigation. The State evaluates faults with demonstrated ground rupture during the last 11,700 years and considers them for inclusion in Earthquake Fault Zones requiring investigation (Alquist-Priolo or A-P Zones) which encompass traces of *Holocene-active* faults as defined by the State's Alquist-Priolo Earthquake Fault Zoning Act (California 1972). The State's guidance is intended to prohibit developments and structures for human occupancy across the trace of active faults (CGS 2018). Other active faults capable of generating strong ground motion are present in the region but are not included in A-P Zones because they do not meet the criteria of "sufficiently active and well-defined."

The site is within a seismically active region, and the project will experience seismic shaking during its design life. There are several faults considered capable of earthquakes located near the site, but there are no known Holocene-Active faults on the site. The closest Holocene-Active fault included in an A-P Zone is the Los Osos fault located approximately 4.5 miles (7.2 km) south of the site (SLOCO 2025, USGS 2013). There are several late Quaternary faults mapped within 10 miles of the site (SLOCO 2025).

Although there are no known *Holocene-Active* faults on the site, the site is within a seismically active region, and the project will experience seismic shaking. Known faults and fault systems within the region that potentially could generate earthquakes affecting the site include the Los Osos, Oceanic-West Huasna, Hosgri-San Simeon, San Luis Range, Rinconada, and San Andreas faults (USGS 2013). These are several of the known faults within a 65-mile radius of the site; other unknown faults may exist in the region and movement on any of these faults could affect operations at the site. Table D-1, Fault Parameters, presented in Appendix D, lists the 40 closest faults to the site.

Los Osos Fault

The Los Osos fault is the closest *Holocene-active* fault to the site, mapped as a series of discontinuous segments within Los Osos Valley south and southeast of the site. The Los Osos fault consists of four distinct segments; from northwest to southeast these are the Estero Bay, Irish Hills, Lopez Reservoir, and Newsome Ridge segments. The Irish Hills segment is approximately two miles long and starts near Los Osos and extends to just past San Luis Obispo Creek. A portion of the Irish Hills segment southwest of the site is zoned as a State Earthquake Fault Zone under the Alquist-Priolo Act (Treiman 1989, CDMG 1990). The Los Osos fault is modeled as a reverse fault, dipping to the southwest (USGS 2013), although it is a very complex set of fault segments that exhibits both normal and reverse faulting. At its western end, the Los Osos fault may extend offshore and intersect with the Hosgri fault (SCEDC 2025). The Los Osos fault is generally mapped as an inferred trace approximately 4.5 miles (7.2 km) south of the site. The Los Osos fault is considered capable of a magnitude 7.15 earthquake (BSSC 2014).

Oceanic-West Huasna Fault

The Oceanic-West Huasna fault zone separates the Santa Lucia and San Rafael mountains from a series of distinct tectonic domains stretching from Cambria to the western Transverse Ranges (Lettis et al. 2004). This fault system trends northwest-southeast for approximately 75 miles and is modeled about 6.1 miles (9.8 km) northeast of the site. The Oceanic-West Huasna fault is considered capable of a magnitude 7.2 earthquake (BSSC 2014).

Hosgri-San Simeon Fault System

The Hosgri-San Simeon fault system lies offshore, approximately 8.2 miles (13.2 km) west of the site. A northwest-trending strike-slip fault, the San Simeon fault extends from offshore of Ragged Point southward across San Simeon to just offshore of San Simeon Point, where it joins the northern end of the Hosgri fault; the Hosgri fault extends to an ocean shelf 2 miles west of Point Buchon, and then trends toward the Point Sal area. The fault system is considered *Holocene-active* and is included in an A-P Zone between San Simeon Point and Arroyo de le Cruz (CDMG 1986b, 1986c). The Hosgri fault is considered capable of a magnitude 7.54 earthquake (BSSC 2014).

San Luis Range Fault System

The San Luis Range fault system consists of a series of west-northwest trending faults that include the Shoreline, Santa Maria River, Wilmar Avenue, Oceano, San Luis Bay, and San Miguelito faults. The San Luis Range fault system is modeled about 9.2 miles (14.8 km) southwest of the site. The San Luis Range fault system is considered capable of a magnitude 7.49 earthquake (BSSC 2014).

Rinconada Fault Zone

The Rinconada fault zone lacks obvious Holocene offset and therefore does not meet the State's criteria for inclusion in an A-P zone; nonetheless, it is considered an active fault and is included as a seismic source in regional models. The Rinconada fault is a northwest trending high angle fault that forms the boundary between two dissimilar geologic terranes. Northeast of the Rinconada fault is the Salinian Block, composed of crystalline plutonic and metamorphic rock overlain by a thick sequence of marine sedimentary rocks. Southwest of the Rinconada fault is the Coastal Block, composed of Franciscan mélange overlain by marine sedimentary rocks (Dibblee 1976). The Rinconada fault is located about 14.8 miles (23.8 km) northeast of the site and is considered capable of a magnitude 7.5 earthquake (Cao et al. 2003).

San Andreas Fault

The San Andreas fault is considered the most active fault in the region. Several large magnitude earthquakes have occurred on the San Andreas fault during historical time, including the 1857 great Fort Tejon earthquake, estimated magnitude 7.9; the 1906 San Francisco earthquake, estimated magnitude 7.8; and the 1989 Loma Prieta earthquake, which had an estimated magnitude of 6.9. The 1989 Loma Prieta earthquake did not rupture the ground surface but was nevertheless responsible for many deaths and many millions of dollars in damages. The San Andreas fault is modeled as segments with some segments capable of earthquakes up to magnitude 7.5 (Cao et al. 2003). Simultaneous rupture of more than one segment could cause an earthquake of magnitude 8 or more (BSSC 2014). The Cholame-Carrizo segment is located approximately 41 miles (66 km) east of the site, is approximately 201 km long and is in an A-P Zone (CDMG 1986a). This fault ruptured during the magnitude 7.9 earthquake in 1857 (USGS 2019).

Groundwater

The site is located approximately 260 feet from the Pacific Ocean on permeable sediments. Groundwater should be anticipated to be a reflection of sea level and may be influenced by tidal flux. Groundwater was observed in Borings 2 and 3 approximately 10 feet bgs, which is slightly higher than local sea level and may reflect latent tidal influence or groundwater backed up behind the sheet piles that stabilize the Embarcadero.

7.0 SEISMICITY

Earthquake History

The historic seismicity in the site's region was researched using a catalog of historical California earthquakes (ANSS 2025). We compiled the epicentral distance for earthquakes within the following search parameters: magnitudes greater than 5.0, within a 65-mile radius from the site, from 1800 to December 2024. Epicentral distances should be considered estimates, particularly for earthquake data prior to 1932, when modern instruments were first used to record earthquake data. The site coordinates used in this search were latitude 35.3700°N and longitude 120.8546°W. Figure 4 – Historical Seismicity Map presented in Appendix C graphically depicts historical earthquake epicenters, their corresponding magnitudes, and the faults within the general region of the project.

Results of the search indicated that within the search parameters, over 39 earthquakes with magnitude greater than or equal to 5.0 have occurred within 65 miles of the site. The largest magnitude earthquake that occurred during the 224-year time period was the magnitude 7.9 Tejon Ranch earthquake on January 9, 1857. The closest earthquake to the site occurred approximately 10 miles from the site on September 5, 1922 and had an estimated magnitude of 5.5. The historical earthquakes are presented in Table D-2, Historical Earthquakes in Vicinity of Project Site, M≥5.0, in Appendix D.

Historical earthquakes that resulted in damage within the region include the Lompoc Earthquake of 1927 and the San Simeon Earthquake of 2003. The Lompoc event is believed to have occurred on the offshore Hosgri fault and had a magnitude of greater than 7.0 (Helmberger et. al. 1992). The event triggered a tsunami that was measured by tidal gauges at San Francisco and San Diego and liquefaction phenomenon, including sand boils, at several locations within and around Lompoc. Reportedly, structures were damaged in Lompoc and Guadalupe (SCEDC 2025).

The San Simeon earthquake that occurred on December 22, 2003, is an example of a significant regional earthquake during recent times, the results of which are well documented. Significant damage was reported from Paso Robles to Oceano. The highest recorded PGA from the San Simeon earthquake was at the hospital in Templeton and liquefaction effects were concentrated at Oceano, approximately 50 miles south of the earthquake epicenter suggesting a PGA of 0.4g or greater (EERI 2005).

Ground Motion Analyses

In accordance with the CBC (CBSC 2022) and ASCE 7-16 (2017, 2018, 2021), an assessment was made to determine the need for employing “Site-Specific Ground Motion Procedures” to calculate the seismic design parameters for the project. Based on our site evaluation and Standard Penetration Tests (SPT), the subsurface characteristics are those of Site Class D (Stiff Soil) as defined by ASCE 7-16 Table 20.3-1, Site Classification. Our analysis indicated that the site is underlain by soils that are vulnerable to potential failure or collapse under seismic loading; Section 20.3 of ASCE 7-16 stipulates that such sites should be classified as Site Class F, and a site response analysis is required unless the fundamental period of the structure is 0.5 seconds or less. Our understanding is that the fundamental period of the planned structure is less than 0.5 seconds, and therefore a site response analysis is not required, and the Site Class may be determined per Section 20.3 (Site Class D).

The mapped S_1 ground motion value obtained from the Structural Engineers Association of California website (SEAOC 2025), using ASCE 7-16 (2017) and Site Class D (Stiff Soil), was 0.357g, which is greater than 0.2; therefore, per Section 11.4.8 of ASCE 7-16, the project requires site-specific ground motion analyses unless certain procedures are used during the structural design that comply with the exceptions allowed. To provide flexibility for the structural design professionals, we have provided “General Procedure” seismic design parameters, which are only valid if structural calculations are performed in accordance with the exception permitted by Supplement 3 of ASCE 7-16 (ASCE 2021), and “Site-Specific” seismic design parameters developed through a site-specific ground motion hazard analysis in accordance with Chapter 21 of ASCE 7-16.

The site may be subject to strong ground shaking due to potential fault movements along regional faults including the Los Osos fault. Engineered design and earthquake-resistant construction increase safety and allow development of seismic areas. The minimum seismic design should comply with the CBC (CBSC 2022) and ASCE 7-16 (2017, 2018, 2021) using the seismic design coefficients given in the table below.

Seismic Design Category

Section 1613.2.5 of the CBC (CBSC 2022) states that structures classified as Risk Category I, II, or III that are located where the mapped spectral response acceleration parameter at 1-second period, S_1, is greater than or equal to 0.75 shall be assigned to Seismic Design Category E…others shall be assigned to Seismic Design Category D. The S_1 for the site is 0.357, which is less than 0.75; therefore, the site should be assigned to Seismic Design Category D. We understand that the site falls under Risk Category II, per Table 1604.5 of the CBC (CBSC 2022).

General Procedure Seismic Design Parameters

The General Procedure seismic parameters presented in Table 1 below were generally obtained from the Structural Engineers Association of California (SEAOC 2025), consider seismic Site Class D, and an earthquake probability of 2-percent in 50 years, equivalent to a 2,475-year return period. The general procedure seismic design parameters presented below are only valid if structural calculations are performed in accordance with the exception permitted by Supplement 3 of ASCE 7-16, which stipulates that the parameter S_{M1} be increased by 50 percent for all applications in the ASCE 7-16 Standard (ASCE 2021). The resulting value of the parameter S_{D1} shall be determined by Eq. (11.4-4) and shall be used for all applications of S_{D1} in the ASCE 7-16 Standard (ASCE 2021). The values presented in the following table HAVE NOT been increased by 50 percent.

Table 1: General Procedure Seismic Parameters (2022 CBC)

Site Class (ASCE 7-16)	D
Occupancy (Risk) Category	II
Seismic Design Category	D
Maximum Considered Earthquake (MCE) Ground Motion	
Spectral Response Acceleration, Short Period – S_s	0.959 g
Spectral Response Acceleration at 1 sec. – S_1	0.357 g
General Procedure Site Coefficient – F_a	1.12
General Procedure Site Coefficient – F_v	1.94
Site-Modified Spectral Response Acceleration, Short Period – S_{MS}	1.071 g
Site-Modified Spectral Response Acceleration at 1 sec. – S_{M1}	0.694 g
Design Earthquake Ground Motion	
Short Period Spectral Response – S_{DS}	0.714 g
One Second Spectral Response – S_{D1}	0.462 g
Site Modified Peak Ground Acceleration - PGA_M	0.50 g
Design Response Spectrum Transition Periods (Seconds)	
T_0 = 20% of S_{D1}/S_{DS}	0.13
$T_S = S_{D1}/S_{DS}$	0.65
T_L = Mapped Long Period Transition	8

Site Specific Seismic Design Parameters

As an alternative, we have also developed site-specific seismic design parameters for the site. The site-specific seismic design parameters are summarized in Table 2 below, and the results of our analysis are presented in Appendix D.

A risk-targeted maximum considered earthquake (MCE_R) modeling procedure was performed in accordance with ASCE 7-16 (2017, 2018, 2021), including a Probabilistic Seismic Hazard Analysis (PSHA) using ground motion data from the United States Geologic Survey Earthquake Hazard Toolbox (USGS 2025b), based on the National Seismic Hazard Model (NSHM) Conterminous U.S. 2023, and a Deterministic Seismic Hazard Analysis (DSHA) using the Third Uniform California Earthquake Rupture Forecast (UCERF3) fault model (USGS 2013) and NGA-West2 ground motion prediction equations (PEER 2015). These analyses are based on knowledge of the regional tectonic setting, geology, and seismicity. These analyses were performed as described in ASCE 7-16 (2018) Section 21.2.1.1 (Method 1) to estimate the peak ground motion corresponding to the uniform hazards earthquake and MCE_R which has a 2 percent probability of being exceeded in 50 years. Our site-specific ground motion hazard analysis compared two likely earthquake scenarios on faults modeled near the site. The Los Osos and Oceanic-West Huasna faults have similar magnitude potential and distances from the site. The Oceanic-West Huasna fault has a slightly larger magnitude potential, but it is also slightly farther from the site than the Los Osos fault. Both faults dip away from the site at 45 degrees or greater so hanging wall effects were not considered for either scenario. Our analysis indicated that these two earthquake scenarios produced very similar ground responses, but the Los Osos fault scenario produced slightly higher accelerations at periods of 0.10 seconds or less. Fault parameters used in our analyses are summarized in Table D-1, Fault Parameters, presented in Appendix D, modified in accordance with the USGS Earthquake Scenario web page (BSSC 2014). The resulting site-specific seismic coefficients considering Site Class D are summarized in the table below and the analysis is presented as Figure D-3 and D-4 in Appendix D.

Table 2: Site-Specific Seismic Parameters (2022 CBC/ASCE 7-16)

Seismic Design Category	D
Occupancy (Risk) Category	II
Site Class	D
Mapped and Code Based Ground Motion	
Short Period Spectral Response, S_S	0.959 g
1 second mapped Spectral Response, S_1	0.357 g
Design Earthquake Ground Motion	
Short Period Spectral Response, S_{DS}	0.901 g
1 second Spectral Response, S_{D1}	0.800 g
Peak Ground Acceleration (PGA_M)	0.557 g
MCE Spectral Response Acceleration	
Short Period Spectral Response, S_{MS}	1.351 g
1 Second Period Spectral Response, S_{M1}	1.200 g
Site Amplification Factors	
Short Period Site Coefficient, F_a	1.12
1 Second Period Site Coefficient, F_v	2.50
Vertical Site Coefficient, C_v	1.28
Risk Coefficient (Short Period), C_{RS}	0.896
Risk Coefficient (1 Second Period), C_{R1}	0.905
Design Response Spectrum Transition Periods (Seconds)	
T_0 = 20% of S_{D1}/S_{DS}	0.178
$T_S = S_{D1}/S_{DS}$	0.888
T_L = Mapped Long Period Transition	8

8.0 GEOLOGIC HAZARDS

Surface Ground Rupture

Surface ground rupture generally occurs at sites that are traversed by, or lie very near to, an active fault. The site is not located within any State Earthquake Fault Zones (Hart and Bryant 2007) and there are no mapped faults crossing or trending towards the site. The closest mapped *Holocene-active* fault to the site included in an A-P Zone is the Los Osos fault system, located approximately 1.8 miles (2.9 km) south. Although the potential for ground rupture at the site is very low, the possibility cannot be ruled out entirely for any site in a seismically active area, especially where lateral spreading or ground rupture resulting from liquefaction are potential secondary effects of an earthquake.

Seismic Settlement and Lateral Spreading

Liquefaction Settlement

Liquefaction refers to a phenomenon that tends to occur in saturated soils (soils below the groundwater table) of low density that have grain sizes within a certain range, usually fine- to medium-grained poorly graded sands, silty sands, and silts. A sufficiently strong earthquake is also required to cause liquefaction. During liquefaction, the energy from the earthquake causes the water pressure within the pores of the soil to increase. The increase in water pressure decreases the friction between the soil grains, allowing the soil grains to move relative to one another. During this state, the soil will behave as a viscous liquid, temporarily losing its ability to support foundations and other improvements. The high-pressure water will flow through the soil along the path of least resistance. As the pressure is released, the soils typically settle in a process called “liquefaction settlement.” Liquefaction settlement can cause damage to structures and other surface and subsurface improvements.

The planned building area is mapped by San Luis Obispo County as having moderate to high potential for liquefaction (SLOCO 2025). Due to the loose to medium dense subsurface conditions encountered within the soil profile and the presence of shallow groundwater, the potential for liquefaction was assessed. Groundwater was encountered as shallow as 10 feet bgs in both borings drilled for this project; however, we used a groundwater depth of 5 feet bgs for our liquefaction analysis. Our analysis of the subsurface soil conditions indicate there is a high potential for liquefaction and seismically induced settlement to occur. The potentials for liquefaction and subsequent dynamic settlement were evaluated using the data from Boring 2. Based on our analysis, the potential for settlement due to liquefaction was estimated to be on the order of 5.2 inches using Idriss and Boulanger’s 2004 procedure (Idriss and Boulanger 2004). Copies of our liquefaction calculations are included in Appendix E.

Although our analyses indicated potential liquefaction settlement to be on the order of 5.2 inches, it is our opinion that this figure is overly conservative. Based on the depth of some of the deeper potentially liquefiable layers (such as the layer at 46 feet bgs), the potential for void redistribution within the areas above the liquefiable layers, and the potential for arching effects, all of which have the potential to reduce the magnitude of liquefaction settlement at the surface, we estimate maximum settlement due to liquefaction will be on the order of 3 inches.

Additionally, the work of Youd and Garris (1995) indicates the non-liquefiable cap above the soils susceptible to liquefaction *does not* have sufficient thickness to prevent ground rupture resulting from liquefaction; therefore, actual settlement *may be more* than the estimated settlements if mitigation is not implemented.

<u>Lateral Spreading</u>

Lateral spreading is horizontal movement of slightly sloped soil deposits or relatively level soil toward a nearby free face, such as the harbor to the west. The movement within a soil layer is typically associated with liquefaction of that layer or an underlying layer. Based on the work of Zhang et al. (2004), lateral displacement toward the harbor of approximately 32 inches was calculated to be possible within the building footprint during the design seismic event.

<u>Settlement of Unsaturated Soils</u>

Seismically induced settlement of unsaturated soils is also caused by a significant seismic event and typically occurs in lower density sandy and silty soils that are not saturated by groundwater. During a major earthquake, the void spaces between the unsaturated soil particles that are filled with air tend to compress which translates to a decrease in volume resulting in settlement. Considering the loose soils above the design groundwater level of 5 feet bgs, there will likely be up to 1 inch of potential settlement within this zone.

Slope Stability

The site is in an area designated Class 0 for deep seated landslide susceptibility, which is considered low potential (Wills et al. 2011). The County has mapped the site as low landslide risk (SLOCO 2025). Global slope stability is not considered a hazard to the project or hotel occupants although the slope on the east side of the hotel may be prone to localized erosion due to the unconsolidated nature of eolian sediments and the previously stated potential for lateral spreading during an earthquake.

Flooding

According to the Federal Emergency Management Agency (FEMA) Flood Insurance Rate Map No. 06079C1026H (FEMA 2017), the site is located within Flood Zone X, which indicates areas of 0.2 percent annual chance (500-year) flood or 1-percent annual chance (100-year) flood with a depth of less than 1-foot. The flood hazard zone for this site is presented on Figure 5 – FEMA Flood Zone Map in Appendix C.

The site is not within a mapped downstream dam inundation zone based on the County's hazard map (SLOCO 2025).

Tsunami and Seiches

According to the State of California Tsunami Inundation Zone map for the County of San Luis Obispo (CGS 2021), the project site is within a Tsunami Inundation Zone. The site is also within a tsunami design zone. The web-based Tsunami Hazard Tool (ASCE 2025) indicates that the anticipated run-up elevation of a tsunami in the Embarcadero area is about 65 feet (NAVD88), the offshore amplitude is reported as 15.0 feet with a period of 45 minutes (ASCE 2025). As the elevation of the site is at an elevation of approximately 14 feet the potential for a tsunami to inundate the site is high, although tsunami that impact the central California coastline are uncommon. Several tsunami have impacted Morro Bay in historical times; the most notable tsunami to impact Morro Bay followed the March 11, 2011 magnitude 9.0 earthquake off the coast of Japan that reportedly produced an 8-foot tidal surge in Morro Bay. The Tsunami hazard zone for this site is presented on Figure 6 – Tsunami Inundation Zone Map in Appendix C.

A seiche is a water wave that can be generated in a reservoir, lake, or pond as the result of barometric pressure anomalies or long-period seismic waves generated by strong local earthquakes. Because the site is not within any downstream dam inundation zone and there are no reservoirs, lakes, or ponds in the vicinity of the site there is no potential for a seiche to affect the project.

Naturally Occurring Asbestos

Asbestos is a term used for several types of naturally occurring fibrous minerals found in many parts of California. The most common type of asbestos is chrysotile, but other types are also found in California. When rock containing asbestos is broken or crushed, asbestos fibers may be released and become airborne. Exposure to asbestos fibers may result in health issues such as lung cancer, mesothelioma (a rare cancer of the thin membranes lining the lungs, chest and abdominal cavity), and asbestosis (a non-cancerous lung disease which causes scarring of the lungs) (CARB 2002).

Asbestos minerals are generally limited to only a few types of rocks known to be present in the coast ranges of California. These are ultra-mafic igneous rocks and their metamorphic equivalents which include serpentinite and some types of schist. There are no naturally occurring asbestos-bearing rock formations (serpentinite or ultramafic rock) known in the vicinity of the site. The area is underlain by alluvial and eolian sediments, which are not considered to be asbestos-bearing geologic units. Therefore, there are no indications that friable asbestos is present in the area and the potential for naturally occurring asbestos is considered very low.

Radon

Radon is a naturally occurring, colorless, odorless gas present in certain soils and rocks, which is derived from the decay of uranium atoms. The occurrence of radon correlates with the presence of specific minerals, and its concentrations in soil or rock will vary depending on the mineralogy of the surrounding bedrock, temperature, barometric pressure, moisture and other factors. Prolonged exposure to elevated levels of radon is associated with an increased risk of lung cancer. Radon exposure can occur after it becomes trapped indoors after it enters buildings through cracks and other holes in foundations. The U.S. EPA recommends remedial action if indoor radon concentrations exceed 4 picocuries per liter (pCi/L) (Churchill 2008).

No rock types associated with elevated radon were observed during our investigation. According to the Radon Potential Zone Map for Western San Luis Obispo County (Churchill 2008), the site is mapped as having low potential for indoor radon. An excerpt of the State's Indoor Radon Potential Map depicting the site is presented as Figure 7 in Appendix C.

9.0 CONCLUSIONS

In our opinion, the site is suitable, from a geotechnical engineering and engineering geology standpoint, for the construction of the proposed improvements as described in the "Introduction and Site Setting" Section of this report, provided the recommendations contained herein are implemented in the design and construction. The primary geotechnical engineering and engineering geology concerns are the potential for strong ground shaking; the existing fill and the potentials for total and differential static settlement, seismic settlement and lateral spreading; shallow groundwater; and the soil's erosion potential. The site soils were found to be nonexpansive, therefore no special measures with respect to expansive soils are considered necessary.

<u>Strong Ground Shaking</u>

The site is in a region of high seismic activity, with the potential for large seismic events that could generate strong ground shaking. Seismic analyses were undertaken to provide General Procedure and Site-Specific seismic acceleration design parameters. Our methods and the results of the seismic analyses are presented in the "Seismicity" Section of this report. Seismic acceleration parameters should be utilized in the design of the structures so that potential damage is reduced during a seismic event.

Existing Fill and Potentials for Total and Differential Static Settlement

Fill soils were found in Borings 2 and 3 to a maximum depth of approximately 15 feet bgs. The fill soils were poorly graded sands with clay and were likely placed during construction of the Embarcadero area. However, we are unaware of any documentation regarding the placement of the fill, therefore for the purposes of this report, the fill is considered to be undocumented. As undocumented fill typically has variable moisture, density and consolidation characteristics, it is not considered suitable for support of foundations in its current condition.

To reduce the potential for destructive total and differential settlement, we recommend removing and moisture conditioning the existing fill to the degree practicable and then replacing the soils in a controlled earthwork program.

Seismic Settlement and Lateral Spreading

As discussed in the "Geologic Hazards" Section of this report, potential total seismic settlements up to 4 inches are anticipated, with lateral spreading resulting from liquefaction estimated to be on the order of 32 inches. Ground surface rupture is also possible.

Recommendations for reducing the potential for damage resulting from liquefaction settlement, lateral spread and ground surface rupture are included in the "Grading" and "Mat Slab Foundations" Sections of this report. In general, the recommendations consist of removing the upper feet of existing fill soil and using an imported gravel reinforced with geogrid and a select fill layer to underlie a mat slab foundation system that will support the structure. The potential for liquefaction, lateral spread and dynamic settlement will still be present, but the recommended earthwork and foundation system is intended to provide an increase in rigidity within and below the foundation to reduce the potential effects of liquefaction, lateral spread, and surface rupture. Utility lines that will span between the reinforced gravel/select fill area and any unreinforced zones should be provided with flexible connections, to reduce the potential for damage in the event liquefaction and resulting dynamic settlement, lateral spread and surface rupture were to occur.

Shallow Groundwater

Following completion of our exploratory borings, groundwater stabilized in Borings 2 and 3 at 10 feet bgs (groundwater was not encountered in Boring 1, which was performed upslope from the western portion of the site). This depth to groundwater is expected to remain relatively constant, and could rise as high as 5 feet bgs, depending on the weather conditions during and immediately preceding construction. As noted in the previous section, a program to remove the upper soils and replace them with a reinforced geogrid/select fill layer is recommended to reduce the potential for damage to the structure due to potential liquefaction and subsequent dynamic settlement, lateral spread and ground rupture. The recommended overexcavation depth will likely be at or very near the depth of groundwater found in the exploratory borings, especially within the elevator pit area. Construction details should note this expected groundwater condition, and the need for the contractor to utilize adequate equipment, experienced personnel, and specialized techniques, including temporary dewatering as needed, to successfully complete the recommended reinforced gravel/select fill layer to support the planned building's foundations.

Erosion Potential

The site soils are considered to be highly erodible. It is essential that all surface drainage be controlled and directed to appropriate discharge points, and that surface soils, particularly those disturbed during construction, are stabilized by vegetation or other means during and following construction. The architect/engineer should ensure appropriate nonerosive overland escape if storm water drainage systems fail or are overwhelmed during significant storm event(s), so that soils are not eroded.

10.0 GEOTECHNICAL RECOMMENDATIONS

The following recommendations are for improvements constructed as described in the "Introduction and Site Setting" Section of this report. If locations, elevations, structural loads, etc., change, the recommendations contained herein may require modification. In developing the following recommendations, it was assumed that irrigated landscaping or flatwork will be installed within a zone of at least five feet around the perimeter of the structure; the intent is to keep the soils in a relatively uniform moisture condition year-round.

Unless otherwise noted, the following definitions are used in the recommendations presented below. Where terms are not defined, definitions commonly used in the construction industry are intended.

- **Building Area –** The building areas are defined as the areas within and extending a minimum of 5 feet beyond the perimeter of the building's foundations. The building area includes any retaining walls, covered walkways or other improvements that are connected to the building and that are intended to act in a manner similar to it.

- **Flatwork Areas –** The footprints of all areas to receive exterior pedestrian flatwork constructed of Portland cement concrete (PCC).
- **Sitework Retaining Wall Areas** – The areas within and extending a minimum of 3 feet beyond the foundation limits of all sitework retaining walls.
- **Grading Area –** The entire area to be graded, including building, flatwork and sitework retaining wall areas.
- **Existing Grade –** Elevations of the site that existed as of the date of this report.
- **Finish Pad Grade –** The elevation in the building area where earthwork operations are typically considered to be complete. It does not include any sand or gravel that might be placed below slabs-on-grade in association with vapor protection for the slabs.
- **Subgrade –** The elevation of the surface upon which a sand cushion/nonexpansive imported material or AB will be placed to support PCC flatwork.
- **Finish Grade –** The elevations beyond the building and flatwork areas where earthwork operations are typically considered to be complete. It does not include any topsoil or other select material used to facilitate plant growth.
- **Aggregate Base (AB) Grade –** The finished surface of any AB used to support PCC flatwork. AB should conform to Section 26 of the Standard Specifications (Caltrans 2025)
- **Scarified –** Plowed or ripped in two orthogonal directions to a depth of not less than 8 inches.
- **Moisture Conditioned –** Adjusting the soil moisture to optimum moisture content or just above, prior to application of compactive effort, unless stated otherwise.
- **Compacted/Recompacted –** Soils placed in level lifts not exceeding 8 inches in loose thickness and compacted to a minimum of 95 percent of maximum dry density (unless stated otherwise), based on maximum dry density by ASTM D 1557-12/21 and field density by ASTM D 6938-23, or other methods acceptable to the geotechnical engineer and jurisdiction.

Site Preparation

1. The ground surface in the grading areas should be prepared for construction by removing existing vegetation, large roots, debris, organic topsoil, and other deleterious materials. Existing utility lines that will not remain in service should be either removed or abandoned. The appropriate method of abandonment will depend upon the type and depth of the utility. Recommendations for abandonment can be made as necessary.

2. Voids created by the removal of materials or utilities described above should be called to the attention of the geotechnical engineer. No fill should be placed unless the underlying soil has been observed by the geotechnical engineer.

Grading

1. Following site preparation, the existing fill soils within the building and sitework retaining wall areas should be removed to a level plane at 5 feet below lowest existing grade in the building area or 2 feet below the bottom of the mat slab elevation (including the elevator pit), whichever is deeper. The exposed surface should then be scarified, moisture conditioned and recompacted.

2. Following compaction, a bottom layer of Tensar NX750 (or approved equivalent) geogrid should be placed in the bottom of the building and sitework retaining wall areas. The grid should be unrolled along the long axis of the excavation. The geogrid should be held in place and overlapped per the manufacturer's installation instructions. Depending on groundwater conditions at the time of construction, the contractor may need to temporarily dewater the excavation or use other methods to properly place the geogrid.

3. After the bottom layer of geogrid has been placed and to reduce the potential for soil fines to enter the gravel zone, a layer of geotextile filter fabric (durable, nonwoven, minimum 6 oz. per square yard) should be placed over the bottom geogrid layer, with overlaps per the manufacturer, and extended up the excavation sidewalls on all sides with at least 4 feet of overlap. A minimum of 12 inches of clean crushed gravel (1 to 1.5-inch maximum size) should then be placed over the filter fabric in maximum 6-inch lifts. Each lift should be consolidated in two orthogonal directions using heavy vibratory equipment. The filter fabric extending up the excavation sidewalls should then be lapped over the top of the crushed rock, with additional filter fabric placed over the crushed rock for the remainder of the excavation and overlapped per the manufacturer. An additional layer

of Tensar NX750 (or approved equivalent) geogrid should then be placed over the top of the filter fabric. The remaining area between the top layer of geogrid and finish pad grade elevation should consist of previously removed site soils, Class 2 aggregate base per Section 26 of the Standard Specifications (Caltrans 2025) or locally termed nonexpansive “Decomposed Granite” (DG). Fill within this area should be placed in maximum 6-inch lifts, and should be compacted to a minimum of 95 percent of maximum dry density.

4. Following site preparation, the soils within flatwork areas should be removed to subgrade elevation. The resulting surfaces should be scarified, moisture conditioned and recompacted.

5. Following site preparation and excavations to finish grade, or prior to placement of fill, the soils within the balance of the grading areas should be scarified, moisture conditioned and recompacted.

6. Previously removed soils, as well as approved imported nonexpansive soil, may be placed in thin, moisture conditioned and compacted lifts to subgrade in flatwork areas, and to finish grade in all other areas. All imported fill should be nonexpansive.

7. Nonexpansive materials are defined as on-site or imported soils that fall in the GW, GM, GC, SP, SW, SC and SM categories per ASTM D 2487-17, and that have an expansion index of 10 or less (ASTM D 4829-21).

8. To create conditions that are as uniform as possible, all earthwork operations should be completed throughout the building area in a uniform manner at the same time, from the bottom of the overexcavation through fill placement to finish grade. Earthwork operations in the building area that complete only a portion of the area at one time (i.e., “flip-flopping”, “checkerboarding,” etc.) should not be allowed.

9. All fill soil (site derived or imported) should be placed in uniform lifts across the entire building or improvement area being constructed; these soils should not be mixed or placed in non-uniform layer thicknesses.

10. AB used to support PCC flatwork should be compacted to a minimum of 95 percent of maximum dry density.

11. Imported soils used in the building area should have strength qualities equal to or better than the site soils. Proposed imported materials should be reviewed by the geotechnical engineer before being brought to the site, and on an intermittent basis during placement.

12. All materials used as fill should be cleaned of any debris and rocks larger than 3 inches in diameter. When fill material includes rocks, the rocks should be placed in a sufficient soil matrix to ensure that voids caused by nesting of the rocks will not occur and that the fill can be properly compacted.

13. Imported soils to be used in landscape areas should be reviewed and approved by the landscape architect or others.

14. If the soils are overly moist so that they become unstable, or if the minimum recommended compaction cannot be readily achieved, drying the soil so that it is nearer optimum moisture content may be necessary. Placement of gravel layers, geotextiles, or geogrids may also be necessary to help stabilize unstable soils. Additional over-excavation may also be recommended to correct unstable conditions or if soft or loose conditions are encountered during grading. No fill should be placed in any grading area if the underlying soil is unstable. Recommendations for stabilization should be provided at the time of construction.

15. The recommended soil moisture contents should be maintained throughout construction, and during the lives of the structures and sitework improvements. Failure to maintain the soil moisture content can result in loosening of the soil and disturbance, which are an indication of degradation of the soil compaction. If soils near improvements such as foundations, flatwork, etc. are disturbed, damage to those improvements may result. Soils that have been disturbed should be removed, moisture conditioned, and recompacted.

16. The architect/engineer should designate any special measures for grading operations, as needed, to mitigate the low potential for radon.

17. Where feasible, temporary excavations for the retaining walls at the rear of the planned structure may be sloped per OSHA requirements from existing grades to the bottoms of the excavations required to construct the walls, as determined by the contractor's "competent person." Where temporary cut slopes are not feasible, due to the soil types,

the excavation depths or other factors, shoring should be implemented to support the planned cuts and other loading conditions. Geotechnical recommendations can be provided for shoring, if determined to be needed, at the time of construction.

Utility Trenches

1. Utility trenches adjacent to foundations should not be excavated within the zone of foundation influence, as shown in Typical Detail A in Appendix F.

2. Utilities that must pass beneath foundations should be placed with properly compacted utility trench backfill and the foundation should be designed to span the trench.

3. A select, noncorrosive, easily compacted sand should be used as bedding and shading immediately around utilities. The site soil or approved import soil may be used for trench backfill above the select material. In building and flatwork areas, the upper portion of the trench backfill should match the thickness of imported nonexpansive material, AB or other select material used to support these improvements.

4. Trench backfill should be compacted to a minimum of 90 percent of maximum dry density. AB used for trench backfill to support PCC flatwork should be compacted to a minimum of 95 percent of maximum dry density. Prior to applying compactive effort, trench backfill should be moisture conditioned and placed in level lifts not exceeding 6 inches in loose thickness.

5. Compaction of trench backfill by jetting or flooding is not recommended at this site, as the site soils are highly erodible. However, to aid in *encasing* utility conduits, particularly corrugated drainpipes, and multiple, closely-spaced conduits in a single trench with the bedding and shading material, jetting or flooding may be useful. Flooding or jetting should only be attempted with extreme caution, and any flooding or jetting operation should be subject to review by the geotechnical engineer.

6. Long-term settlement of properly compacted on-site soil and imported sand should be assumed to be about 0.25 to 0.5 percent of the depth of the backfill. Improvements that are constructed over or near trenches should be designed to accommodate the potential for settlement.

7. Utility lines may be placed within the geogrid-reinforced gravel supporting the mat foundation, provided the geogrid and geotextile filter material is properly lapped around the utility penetrations per the manufacturer's recommendations. Proper coordination of utility penetrations is essential to maintain the integrity of the reinforced gravel layer that will support the building. Utility line manufacturers should be contacted to verify acceptability of gravel backfill for their lines, instead of typical clean sand bedding and shading.

8. Flexible utility connections to accommodate differential movement on the order of 4 inches should be provided where the utility lines cross into the reinforced gravel supporting layer and/or the mat slab foundation area.

9. The recommendations of this section are minimums only and may be superseded by the architect/engineer based upon soil corrosivity or the requirements of pipe manufacturers, utility companies or the governing jurisdiction.

10. The architect/engineer should incorporate appropriate measures in the design of the utility systems to mitigate the low potential for radon.

Mat Slab Foundations

1. The building and sitework retaining walls should be supported by mat slab foundations ("mat foundations" or "mat slabs") bearing on the gravel/geogrid/compacted soil layer, as recommended in the "Grading" Section of this report.

2. Mat foundations can have a uniform thickness, or they can consist of variable thickness slabs and grade beams (i.e., a "waffle slab"). Mat slab edges should have a minimum depth of 12 inches below lowest adjacent grade. The mat foundations should be reinforced in accordance with the requirements of the architect/engineer and should contain minimum rebar meeting the criteria of ACI 318, Section 24.4. (ACI 2019/2022).

3. Mat foundations should be designed for maximum dead plus live areal bearing pressures of 800 psf, with isolated areas not exceeding 1,200 psf. A preliminary static modulus of subgrade reaction of 10 pci may be used; equal modulus of subgrade reaction contours can be provided as requested by the architect/engineer once a SAFE-type output has been performed.

4. For resistance to lateral loads, mat slabs may be designed using a passive equivalent fluid pressure of 200 pcf and a friction factor of 0.35. Passive and friction components may be combined in the analysis without reduction to either value. Lateral capacity is based on the assumption that any backfill adjacent to foundations has been properly compacted to a minimum of 95 percent of maximum dry density.

5. Allowable bearing capacities may be increased by one-third when transient loads such as wind or seismicity are included. Foundations may be designed as necessary using the seismic parameters provided in the "Seismicity" Section of this report.

6. Static settlement of the mat slabs is expected to be a maximum of 1 inch total and ½-inch differential over a horizontal distance of 30 feet. Potential seismic settlement due to liquefaction is estimated to be on the order of 3 inches, with differential seismic settlement on the order of half this amount over a horizontal distance of 30 feet. Total and differential combined (static plus seismic) settlement could therefore be on the order of 4 inches and 2 inches over a horizontal distance of 30 feet, respectively. The mat foundations should be designed to accommodate these potentials for total and differential settlements.

7. Mat slab excavations should be observed by the geotechnical engineer during excavation and prior to placement of reinforcing steel and concrete. Mat slab excavations should be moistened, and no desiccation cracks should be present, prior to placing concrete.

8. The architect/engineer should incorporate appropriate measures in the design of the foundation systems to mitigate the low potential for radon.

Moisture Vapor Transmission

1. Due to the current use of impermeable floor coverings, water-soluble flooring adhesives, and the speed at which buildings are now constructed, moisture vapor transmission through slabs is a much more common problem than in past years. Where moisture vapor transmitted from the underlying soil would be undesirable, slabs should be protected from subsurface moisture vapor. A number of options for vapor protection are discussed below; however, the means of vapor protection, including the type and thickness of the vapor retarder, if specified, are left to the discretion of the architect/engineer.

2. Where specified, vapor retarders should conform to ASTM Standard E 1745-17/23. This standard specifies properties for three performance classes; Class A, B and C. The appropriate class should be selected based on the potential for damage to the vapor retarder during placement of slab reinforcement and concrete. Unless it is determined that a permeance of 0.10 perms will not allow vapor to accumulate beneath moisture-sensitive flooring, adhesives, stored products and/or equipment, then a vapor retarder permeance of 0.010 perms is recommended, per ACI 302.1-15 (ACI 2015). Permeance of vapor retarders should remain below 0.010 perms after the conditioning tests of ASTM E 1745-17.

   Note: ASTM E 1745-17 has the same permeance threshold for Class A, B and C (0.1 perms). The class that is chosen will make a difference in how resistant the vapor retarder is to punctures and tears, but it will not ensure any better permeance values to protect floor coverings.

3. Several studies, including those of American Concrete Institute Committee 302 (ACI 2015), have concluded that excess water above the vapor retarder increases the potential for moisture damage to floor coverings and could increase the potential for mold growth or other microbial contamination. The studies also concluded that it is preferable to eliminate the typical sand layer beneath the slab and place the slab concrete in direct contact with a Class A vapor retarder, particularly during wet weather construction. However, placing the concrete directly on the vapor retarder requires special attention to using the proper vapor retarder, a very low water-cement ratio in the concrete mix, and special finishing and curing techniques.

4. Another option that may be a reasonable compromise between effectiveness and cost considerations is the use of a subslab vapor retarder protected by a sand layer. If a Class A vapor retarder is specified, the retarder can be placed directly on the subgrade. The retarder should be covered with a minimum 2 inches of clean sand. If a less durable vapor retarder is specified (Class B or C), a minimum of 4 inches of clean sand should be provided, and the retarder should be placed in the center of the clean sand layer. Clean sand is defined as a well or poorly graded sand (ASTM D 2487-17) of which less than 3 percent passes the No. 200 sieve. The site soils do not meet the criteria to be considered “clean sand”.

5. Regardless of the underslab vapor retarder selected, proper installation of the retarder per ASTM E 1643-18a is critical for optimum performance. Where utilized, the vapor retarder should be placed a minimum of 1 inch above the flow line of the drainage path surrounding the structures, or 1 inch above the area drain grates if area drains are used to collect runoff around the structures. As required by ASTM E 1643-18a, all seams and utility penetrations should be properly sealed. At terminating edges of the vapor retarder, the vapor retarder should be effectively sealed with accessories specifically designed to seal the material to new or existing concrete; details for edge sealing of the vapor retarder should be provided by the architect/engineer.

6. If the sand is used between the vapor retarder and the slab, it should be moistened only as necessary to promote concrete curing; saturation of the sand should be avoided, as the excess moisture would be on top of the vapor retarder, potentially resulting in vapor transmission through the slab for months or years.

7. Positive drainage away from the structure should be maintained; see the "Drainage and Maintenance" Section of this report for additional discussion of this issue. If water is allowed to pond near the structure, it may seep into the ground and migrate laterally through cracks or utility penetrations in the foundation, ultimately gaining access above the vapor retarder.

Exterior Pedestrian Flatwork

Interior Slabs-on-Grade

1. Interior slabs-on-grade should have a minimum thickness of 4 inches and should be reinforced and doweled to foundations per the specifications of the architect/engineer. As the site soils are nonexpansive, at a minimum, interior slabs should be reinforced with No. 3 rebar at 24 inches on center each way, placed as directed by the architect/engineer. All structural slabs should contain minimum rebar meeting the criteria of ACI 318, Section 7.6.1.1 (ACI 2019/22). At a minimum, foundation dowels should be lap spliced to the slab rebar. The size and spacing of the dowels should match the size and spacing of the slab rebar.

Exterior Pedestrian Flatwork

1. Exterior pedestrian flatwork should have a minimum PCC thickness of 4 inches. Minimum reinforcement for PCC exterior pedestrian flatwork should consist of No. 3 rebar placed at 24 inches on-center each way.

2. In conventional construction, it is common to use 4 to 6 inches of imported sand beneath flatwork. As the site soils are nonexpansive, this common practice would be suitable at this site. If additional cost savings are desired with minimal potential for loss of support, exterior pedestrian flatwork can be constructed directly over compacted site soils.

3. Flatwork should be constructed with frequent joints to allow articulation as the flatwork moves in response to seasonal soil temperature and moisture variations. The soil below flatwork should be moisture conditioned prior to casting the flatwork.

4. Flatwork at doorways, and at other areas where maintaining the elevation of the flatwork is desired, should be doweled to the perimeter foundations, at a minimum, by No. 3 dowels lapped to the flatwork rebar at 24 inches on-center. In other areas, the flatwork may be doweled to the foundation or the flatwork may be allowed to "float free," at the discretion of the architect/engineer. Flatwork that is intended to float free should be separated from foundations by a felt joint or other means.

5. To reduce shrinkage cracks in all interior slabs-on-grade and exterior pedestrian flatwork, the concrete aggregates should be of appropriate size and proportion, the water/cement ratio should be low, the concrete should be properly placed and finished, contraction joints should be installed, and the concrete should be properly cured. This is particularly applicable to slabs that will be cast directly upon a vapor retarder and those that will be protected from transmission of vapor by use of admixtures or surface sealers. Concrete materials, placement, and curing specifications should be at the direction of the architect/engineer; AC 302.1R-15 (ACI 2015) is suggested as a resource for the architect/engineer in preparing such specification.

Retaining Walls

1. Retaining walls should be supported on mat slabs over the gravel/geogrid/compacted soil layer per the "Grading" Section of this report. Foundations for all retaining walls should have minimum overall depths (not including any keyway) of 18 inches below lowest grade within 7 feet laterally of any adjacent slope.

2. Retaining wall mat foundations should be reinforced per the recommendations in the "Foundations" Section.

3. Retaining wall design may be based on the following parameters:

Table 3: Retaining Wall Design Parameters

Parameter	Backfill Type	Value
Active Equivalent Fluid Pressure	On Site Soils	40 pcf
Active Equivalent Fluid Pressure	Imported Well Graded Sand/Gravel	35 pcf
At-rest Equivalent Fluid Pressure	On Site Soils	55 pcf
At-rest Equivalent Fluid Pressure	Imported Well Graded Sand/Gravel	50 pcf
Passive Equivalent Fluid Pressure	Gravel/Geogrid/Compacted Soil	200 pcf
Maximum Toe Pressure	Gravel/Geogrid/Compacted Soil	1,200 psf
Coefficient of Sliding Friction	Gravel/Geogrid/Compacted Soil	0.35

4. No surcharges are taken into consideration in the values presented in the previous paragraph. The maximum toe pressure is an *allowable* value; no factors of safety, load factors or other factors have been applied to the remaining values. With the exception of the maximum toe pressure, these values will require application of appropriate factors of safety, load factors, and/or other factors as deemed appropriate by the architect/engineer.

5. The upper foot of backfill behind all retaining walls should consist of native soil, except in areas where exterior pedestrian flatwork will abut the top of the wall. In such cases, the gravel should extend to the aggregate base or other material below the improved surface, as appropriate. If gravel backfill is utilized, the gravel should be encased in a permeable synthetic filter fabric conforming to standard specification section 96-1.02B – Class C (Caltrans 2025).

6. The active and at-rest pressures presented in Table 2 are applicable to a horizontal retained surface behind the wall. Walls having a retained surface that slopes upward from the wall should be designed for an additional equivalent fluid pressure of 1 pcf for the active case and 1.5 pcf for the at-rest case, for every 2 degrees of slope inclination.

7. Under the 2022 CBC, the Risk-Targeted Maximum Considered Earthquake (MCE_R) must be used for determining seismic pressures on walls. Further, Section 1807.2.2 of the 2022 CBC requires that dynamic seismic lateral earth pressures be provided by the geotechnical engineer for walls retaining more than 6 feet of backfill. The MCE_R-based Geometric

Mean Peak Ground Acceleration (PGA_M) of 0.557 g was obtained from Table 2 in the Site Specific Seismic Design Parameters Section of this report. Then, using the methods presented by Lew et al. (2010) and this PGA_M, the appropriate incremental increase in lateral soil pressure above the static active equivalent fluid pressure for flexible (cantilevered) walls was determined to be 15 pcf for site soils, or imported sand or gravel backfill. Flexible (cantilevered) walls retaining over 6 feet of backfill should be designed using these incremental seismic pressures. Walls retaining 6 feet or less of backfill need not be designed for seismic pressures.

8. Research by Al Atik and Sitar (2010) confirmed that for flexible (cantilevered) walls, particularly those over 12 feet tall, an increase in soil pressure does occur under significant seismic accelerations. Further, they found that the increase is due to the out-of-phase interaction between the soil and the flexible wall. When considering rigid walls (i.e. those designed using at-rest criteria); however, they found that the incremental increase due to seismicity was typically less than 50 percent of the static wall pressure. Consequently, no incremental increase in lateral soil pressure is recommended for the design of walls where the static design utilizes the at-rest equivalent fluid pressure and they are designed with factors of safety and earth load factors of at least 1.5.

9. In typical structural design methods for retaining walls such as those found in Section 1605 of the 2022 CBC, lateral soil pressure is multiplied by a load factor of 1.6. According to Lew et al. (2010), a load factor of 1.6 is too conservative for seismic loads; this paper suggests that the seismic increase in lateral pressure be separated from the static active pressure and that a load factor of 1.0 be used for the seismic increase. Further, Al Atik and Sitar (2010) found that pressure increases due to seismic earth loads were minimal for walls retaining less than 12 feet of backfill. While Al Atik and Sitar's research is generally accepted among geotechnical and structural engineers in California, it is not entirely acknowledged by the CBC, as the CBC sets the height below which seismic loads may be ignored at 6 feet. Given this disparity, it is suggested that caution be used not to over-engineer walls retaining between 6 and 12 feet of backfill.

10. Long-term settlement of properly compacted site soil or imported sand/gravel retaining wall backfill should be assumed to be about 0.25 to 0.5 percent of the depth of the backfill. Improvements that are constructed near the tops of retaining walls should be designed to accommodate long-term settlement.

11. All retaining walls should be drained with perforated pipe encased in a free-draining gravel blanket. The pipe should be placed with perforations facing downward and should discharge in a nonerosive manner away from foundations and other improvements. The gravel blanket should have a width of approximately 1 foot and should extend upward to approximately 1 foot from the top of the wall backfill. The upper foot should be backfilled with native soil, except in areas where exterior pedestrian flatwork will abut the top of the wall. In such cases, the gravel should extend to the imported nonexpansive material, sand, aggregate base, or other material below the improved surface, as appropriate. To reduce infiltration of the soil into the gravel, a permeable synthetic filter fabric conforming to Standard Specifications Section 96-1.02B – Class C (Caltrans 2025), should be placed between the two materials. Manufactured synthetic drains, such as Miradrain or Enkadrain are acceptable alternatives to the use of gravel, provided that they are installed in accordance with the recommendations of the manufacturer.

12. Where weep hole drainage can be properly discharged, the perforated pipe may be omitted in lieu of weep holes on maximum 4-foot centers. A filter fabric as described above should be placed between the weep holes and the drain gravel.

13. Walls facing areas where moisture transmission through the wall would be undesirable should be thoroughly waterproofed in accordance with the specifications of the architect/engineer.

14. The architect/engineer should bear in mind that retaining walls by their nature are flexible structures, and that surface treatments on walls often crack. Where walls are to be plastered or otherwise have a finish applied, the flexibility should be considered in determining the suitability of the surfacing material, spacing of horizontal and vertical control joints, etc. The flexibility should also be considered where a retaining wall will abut or be connected to a rigid structure, and where the geometry of the wall is such that its flexibility will vary along its length.

15. The architect/engineer should incorporate appropriate measures in the design of retaining walls to mitigate the low potential for radon activity.

Drainage and Maintenance

1. Per Section 1804.4 of the CBC (CBSC 2022), unpaved ground surfaces should be *finish graded* to direct surface runoff away from foundations and other improvements at a minimum 5 percent grade for a minimum distance of 10 feet. The site should be similarly sloped to drain away from foundations, and other improvements during construction. Where this is not practicable due to other improvements, etc., swales with improved surfaces, area drains, or other drainage facilities, should be used to collect and discharge runoff.

2. All eaves of the building should be fitted with roof gutters. Runoff from flatwork, roof gutters, downspouts, planter drains, area drains, etc. should discharge in a non-erosive manner away from foundations and other improvements in accordance with the requirements of the governing agencies. Erosion protection should be placed at all discharge points unless the discharge is to a pavement surface.

3. To reduce the potential for planter drainage gaining access to subslab areas, any raised planter boxes adjacent to foundations should be installed with drains and sealed sides and bottoms. Drains should also be provided for areas adjacent to the structures and in landscape areas that would not otherwise freely drain.

4. Stabilization of the site soils and any similar imported soils by vegetation or other means, *during* and *following* construction, is essential to reduce the potential for erosion damage. Care should be taken to establish and maintain vegetation. The landscaping should be planned and installed to maintain the surface drainage recommended above. Surface drainage should also be maintained during construction.

5. Maintenance of drainage and other improvements is critical to the long-term stability of the site and the integrity of the structures. Site improvements should be maintained on a regular basis.

6. Finished flatwork surfaces should be sloped to freely drain toward appropriate drainage facilities. Water should not be allowed to stand or pond on or adjacent to exterior pedestrian flatwork, or other improvements as it could infiltrate into the AB and/or subgrade, causing premature deterioration of flatwork or other improvements.

7. All exterior drains, retaining wall drains, and drain outlets should be maintained to be free-flowing. Care should be taken to establish and maintain vegetation. Vegetation and erosion matting (if utilized) should be maintained or augmented as needed. Irrigation systems should be maintained so that soils around structures are maintained at a relatively uniform year-round moisture content and are neither over-watered nor allowed to dry and desiccate.

8. To reduce the potential for disruption of drainage patterns and undermining of structures, fill areas, etc., all rodent activity should be aggressively controlled.

Observation and Testing

1. It must be recognized that the recommendations contained in this report are based on a limited number of borings and rely on continuity of the subsurface conditions encountered.

2. It is assumed that the geotechnical engineer will be retained to provide consultation during the design phase, to interpret this report during construction, and to provide construction monitoring in the form of testing and observation.

3. At a minimum, the geotechnical engineer and/or engineering geologist should be retained to provide:

   - Review of project plans and specifications
   - Professional observation during grading, trench and retaining wall backfill and foundation construction
   - Oversight of special inspection and compaction testing during grading, trench and retaining wall backfill and foundation construction

4. Special inspection of grading and backfill should be provided as per Section 1705.6 and Table 1705.6 of the CBC (CBSC 2022). The special inspector should be under the direction of the geotechnical engineer and/or the engineering geologist. At a minimum, the following items should be inspected and/or tested by the special inspector:

   - Stripping and clearing of all existing improvements, vegetation, and deleterious materials
   - Overexcavation to the recommended depths

- Scarification, moisture conditioning and recompaction of excavated areas
- Fill quality; geogrid, fill and crushed rock placement; moisture conditioning and compaction
- Utility trench backfill, moisture conditioning and compaction
- Foundation excavations
- Retaining wall drains and backfill

5. A program of quality assurance should be developed prior to beginning construction. At a minimum, the program should include all geotechnical items shown on the testing and inspection schedule of the approved plans. It should also include any additional inspection items required by the engineer and/or the governing jurisdiction. These items should be discussed at a preconstruction site meeting among a representative of the owner, the geotechnical engineer, special inspector, the project inspector, the engineer, and contractors. The geotechnical engineer should be notified at least 48 hours prior to beginning grading operations.

6. Locations and frequency of compaction tests should be per the recommendation of the geotechnical engineer at the time of construction. The recommended test location and frequency may be subject to modification by the geotechnical engineer, based upon soil and moisture conditions encountered, size and type of equipment used by the contractor, the general trend of the results of compaction tests, or other factors.

11.0 CLOSURE

Our intent was to perform the investigation in a manner consistent with the level of care and skill ordinarily exercised by members of the profession currently practicing in the locality of this project under similar conditions. No representation, warranty, or guarantee is either expressed or implied. This report is intended for the exclusive use by the client as discussed in the "Scope of Services" Section. Application beyond the stated intent is strictly at the user's risk.

This report is valid for conditions as they exist at this time for the type of project described herein. The conclusions and recommendations contained in this report could be rendered invalid, either in whole or in part, due to changes in building codes, regulations, standards of geotechnical or construction practice, changes in physical conditions, or the broadening of knowledge.

If changes with respect to the project become necessary, if items not addressed in this report are incorporated into plans, or if any of the assumptions used in the preparation of this report are not correct, this firm shall be notified for modifications to this report. Any items not specifically addressed in this report should comply with the CBC of other applicable standards, and the requirements of the governing jurisdiction.

The recommendations presented in this geotechnical report are based upon the geotechnical conditions encountered at the site and may be augmented by additional requirements of the client, or by additional recommendations provided by the geotechnical engineer based on peer or jurisdiction reviews, or conditions exposed at the time of construction. If Earth Systems Pacific is *not* retained to provide construction observation and testing services, it shall not be responsible for the interpretation of the information by others or any consequences arising therefrom.

This document, the data, conclusions, and recommendations contained herein are the property of Earth Systems Pacific. This report shall be used in its entirety, with no individual sections reproduced or used out of context. Copies may be made only by Earth Systems Pacific, the client, and the client's authorized agents for use exclusively on the subject project. Any other use is subject to federal copyright laws and the written approval of Earth Systems Pacific.

Thank you for this opportunity to have been of service. If you have any questions, please feel free to contact this office at your convenience.

End of Text.

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APPENDIX A

Figure 1 – Site Vicinity Map
Figure 2 – Exploration Location Map
Boring Log Legend
Boring Logs

N

BASE MAP PROVIDED BY: GOOGLE EARTH (2025)

NOT TO SCALE

Earth Systems Pacific
4378 Old Santa Fe Road, San Luis Obispo, CA 93401
www.earthsystems.com
(805) 544-3276 • Fax (805) 544-1786

SITE VICINITY MAP

Front Street Hotel
1180 Front Street
Morro Bay, California

Date
December 2025

Project No.
307453-001

Figure 1

307453-001 FrontStreetHotel 111325 — Maps.dwg

Exhibit C

LEGEND

3 Boring Location (Approx.)

BASE MAP PROVIDED BY: RRM DESIGN GROUP (202)

N

NOT TO SCALE

Earth Systems Pacific
4378 Old Santa Fe Road, San Luis Obispo, CA 93401
www.earthsystems.com
(805) 544-3276 • Fax (805) 544-1786

EXPLORATION LOCATION MAP

Front Street Hotel
1180 Front Street
Morro Bay, California

Date
December 2025

Project No.
307453-001

Figure 2

307453-001 FrontStreetHotel 111325 - Maps.dwg

Exhibit C

Earth Systems Pacific

BORING LOG LEGEND

SAMPLE / SUBSURFACE WATER SYMBOLS	GRAPH. SYMBOL
CALIFORNIA MODIFIED	
STANDARD PENETRATION TEST (SPT)	
SHELBY TUBE	
BULK	
SUBSURFACE WATER DURING DRILLING	
SUBSURFACE WATER AFTER DRILLING	

UNIFIED SOIL CLASSIFICATION SYSTEM (ASTM D 2487)

MAJOR DIVISIONS		GROUP SYMBOL	TYPICAL DESCRIPTIONS	GRAPH. SYMBOL
COARSE GRAINED SOILS	MORE THAN HALF OF MATERIAL IS LARGER THAN #200 SIEVE SIZE	GW	WELL GRADED GRAVELS, 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, NON-PLASTIC FINES	
		GC	CLAYEY GRAVELS, GRAVEL-SAND-CLAY MIXTURES, PLASTIC FINES	
		SW	WELL GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES	
		SP	POORLY GRADED SANDS OR GRAVELLY SANDS, LITTLE OR NO FINES	
		SM	SILTY SANDS, SAND-SILT MIXTURES, NON-PLASTIC FINES	
		SC	CLAYEY SANDS, SAND-CLAY MIXTURES, PLASTIC FINES	
FINE GRAINED SOILS	HALF OR MORE OF MATERIAL IS SMALLER THAN #200 SIEVE SIZE	ML	INORGANIC SILTS AND VERY FINE SANDS, SILTY OR CLAYEY FINE SANDS OR CLAYEY SILTS WITH SLIGHT PLASTICITY	
		CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS	
		OL	ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY	
		MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SANDY OR SILTY SOILS, ELASTIC SILTS	
		CH	INORGANIC CLAYS OF HIGH PLASTICITY, FAT CLAYS	
		OH	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS	
		PT	PEAT AND OTHER HIGHLY ORGANIC SOILS	

OBSERVED MOISTURE CONDITION

DRY	SLIGHTLY MOIST	MOIST	VERY MOIST	WET (SATURATED)

CONSISTENCY

COARSE GRAINED SOILS

BLOWS/FOOT		DESCRIPTIVE TERM
SPT	CA SAMPLER	
0-10	0-16	LOOSE
11-30	17-50	MEDIUM DENSE
31-50	51-83	DENSE
OVER 50	OVER 83	VERY DENSE

FINE GRAINED SOILS

BLOWS/FOOT		DESCRIPTIVE TERM
SPT	CA SAMPLER	
0-2	0-3	VERY SOFT
3-4	4-7	SOFT
5-8	8-13	MEDIUM STIFF
9-15	14-25	STIFF
16-30	26-50	VERY STIFF
OVER 30	OVER 50	HARD

GRAIN SIZES

U.S. STANDARD SERIES SIEVE				CLEAR SQUARE SIEVE OPENING		
# 200	# 40	# 10	# 4	3/4"	3"	12"

SILT & CLAY	SAND			GRAVEL		COBBLES	BOULDERS
	FINE	MEDIUM	COARSE	FINE	COARSE		

TYPICAL BEDROCK HARDNESS

MAJOR DIVISIONS	TYPICAL DESCRIPTIONS
EXTREMELY HARD	CORE, FRAGMENT, OR EXPOSURE CANNOT BE SCRATCHED WITH KNIFE OR SHARP PICK; CAN ONLY BE CHIPPED WITH REPEATED HEAVY HAMMER BLOWS
VERY HARD	CANNOT BE SCRATCHED WITH KNIFE OR SHARP PICK; CORE OR FRAGMENT BREAKS WITH REPEATED HEAVY HAMMER BLOWS
HARD	CAN BE SCRATCHED WITH KNIFE OR SHARP PICK WITH DIFFICULTY (HEAVY PRESSURE); HEAVY HAMMER BLOW REQUIRED TO BREAK SPECIMEN
MODERATELY HARD	CAN BE GROOVED 1/16 INCH DEEP BY KNIFE OR SHARP PICK WITH MODERATE OR HEAVY PRESSURE; CORE OR FRAGMENT BREAKS WITH LIGHT HAMMER BLOW OR HEAVY MANUAL PRESSURE
SOFT	CAN BE GROOVED OR GOUGED EASILY BY KNIFE OR SHARP PICK WITH LIGHT PRESSURE, CAN BE SCRATCHED WITH FINGERNAIL; BREAKS WITH LIGHT TO MODERATE MANUAL PRESSURE
VERY SOFT	CAN BE READILY INDENTED, GROOVED OR GOUGED WITH FINGERNAIL, OR CARVED WITH KNIFE; BREAKS WITH LIGHT MANUAL PRESSURE

TYPICAL BEDROCK WEATHERING

MAJOR DIVISIONS	TYPICAL DESCRIPTIONS
UNWEATHERED	NO DISCOLORATION, NOT OXIDIZED
SLIGHTLY WEATHERED	DISCOLORATION OR OXIDATION IS LIMITED TO SURFACE OF, OR SHORT DISTANCE FROM, FRACTURES: SOME FELDSPAR CRYSTALS ARE DULL
MODERATELY WEATHERED	DISCOLORATION OR OXIDATION EXTENDS FROM FRACTURES, USUALLY THROUGHOUT; Fe-Mg MINERALS ARE "RUSTY", FELDSPAR CRYSTALS ARE "CLOUDY"
HIGHLY WEATHERED	DISCOLORATION OR OXIDATION THROUGHOUT; FELDSPAR AND Fe-Mg MINERALS ARE ALTERED TO CLAY TO SOME EXTENT, OR CHEMICAL ALTERATION PRODUCES IN SITU DISAGGREGATION
DECOMPOSED	DISCOLORATION OR OXIDATION THROUGHOUT, BUT RESISTANT MINERALS SUCH AS QUARTZ MAY BE UNALTERED; FELDSPAR AND Fe-Mg MINERALS ARE COMPLETELY ALTERED TO CLAY

drafting/masters/Boring Log Legend 062719.dwg

Earth Systems Pacific

Boring No. 1

PAGE 1 OF 1

LOGGED BY: T. Robison
AUGER TYPE: 3" Hand Auger

JOB NO.: 307453-001
DATE: 11/06/2025

Front Street Hotel
1180 Front Street
Morro Bay, California

DEPTH (feet)	USCS CLASS	SYMBOL	SOIL DESCRIPTION	INTERVAL (feet)	SAMPLE TYPE	DRY DENSITY (pcf)	MOISTURE (%)	BLOWS PER 6 IN.
0	SP-SC		POORLY GRADED SAND WITH CLAY: brown, loose, slightly moist (Dune Sand)	1.5 - 5.0	○			
5.5	SP		POORLY GRADED SAND: light brown, medium dense, slightly moist	7.0 - 10.0	○			
10			End of Boring @ 10.0' No subsurface water encountered					

LEGEND: ■ Ring Sample ○ Grab Sample ▭ Shelby Tube Sample ● SPT

NOTE: This log of subsurface conditions is a simplification of actual conditions encountered. It applies at the location and time of drilling. Subsurface conditions may differ at other locations and times.

Earth Systems Pacific

Boring No. 2
PAGE 1 OF 2

LOGGED BY: T. Robison
DRILL RIG: GTech GT 8 with Automatic Hammer
AUGER TYPE: 6" Hollow Stem

JOB NO.: 307453-001
DATE: 11/06/2025

Front Street Hotel
1180 Front Street
Morro Bay, California

DEPTH (feet)	USCS CLASS	SYMBOL	SOIL DESCRIPTION	INTERVAL (feet)	SAMPLE TYPE	DRY DENSITY (pcf)	MOISTURE (%)	BLOWS PER 6 IN.
0	SP-SC		POORLY GRADED SAND WITH CLAY: brown, loose, slightly moist (Fill)					
1				1.0 - 5.0	Grab Sample			
2				2.0 - 3.5	Ring Sample	95.1	2.9	3 3 6
3.5			dark brown					
5				5.0 - 6.5	Ring Sample	113.7	1.3	3 11 15
5.5			light gray brown, medium dense					
10	SP		POORLY GRADED SAND: light gray brown, loose, very moist	10.0 - 11.5	Ring Sample	115.1	18.5	5 4 8
15	SP-SC		POORLY GRADED SAND WITH CLAY: light gray brown, medium dense, wet, trace gravel (Paralic Estuarine Deposits)	15.0 - 16.5	Ring Sample	117.3	17.2	15 19 22
20	SP-SC		POORLY GRADED SAND WITH CLAY AND GRAVEL: dark brown, medium dense, wet	20.0 - 21.5	Ring Sample	93.7	19.2	16 23 22
25	SP-SC		POORLY GRADED SAND WITH CLAY: dark brown, medium dense, wet	25.0 - 26.5	Ring Sample	101.2	23.0	4 9 15

LEGEND: Ring Sample, Grab Sample, Shelby Tube Sample, SPT

NOTE: This log of subsurface conditions is a simplification of actual conditions encountered. It applies at the location and time of drilling. Subsurface conditions may differ at other locations and times.

Earth Systems Pacific

Boring No. 2

LOGGED BY: T. Robison
DRILL RIG: GTech GT 8 with Automatic Hammer
AUGER TYPE: 6" Hollow Stem

PAGE 2 OF 2
JOB NO.: 307453-001
DATE: 11/06/2025

Front Street Hotel
1180 Front Street
Morro Bay, California

DEPTH (feet)	USCS CLASS	SYMBOL	SOIL DESCRIPTION	INTERVAL (feet)	SAMPLE TYPE	DRY DENSITY (pcf)	MOISTURE (%)	BLOWS PER 6 IN.
27	SP-SC		POORLY GRADED SAND WITH CLAY: same as above					
30				30.0 - 31.5	■	102.9	25.1	6 9 12
35	SP-SC		POORLY GRADED SAND WITH CLAY AND GRAVEL: gray to dark brown, medium dense, wet	35.0 - 36.5	■	101.2	24.7	6 10 19
40			flow sands encountered, sampler plugged	40.0 - 41.5	■		18.4	
46			very dense, flow sands end					
50				50.0 - 51.5	■	97.3	27.5	12 32 50/5"
51.5			End of Boring @ 51.5' Subsurface water encountered @ 15.0' during and @ 10.0' after drilling					

LEGEND: ■ Ring Sample ○ Grab Sample □ Shelby Tube Sample ● SPT

NOTE: This log of subsurface conditions is a simplification of actual conditions encountered. It applies at the location and time of drilling. Subsurface conditions may differ at other locations and times.

Earth Systems Pacific

Boring No. 3

LOGGED BY: T. Robison
DRILL RIG: GTech GT 8 with Automatic Hammer
AUGER TYPE: 6" Hollow Stem

JOB NO.: 307453-001
DATE: 11/06/2025

Front Street Hotel
1180 Front Street
Morro Bay, California

DEPTH (feet)	USCS CLASS	SOIL DESCRIPTION	INTERVAL (feet)	SAMPLE TYPE	DRY DENSITY (pcf)	MOISTURE (%)	BLOWS PER 6 IN.
0	SP-SC	POORLY GRADED SAND WITH CLAY: reddish brown, loose, slightly moist (Fill)					
1			0.0 - 5.0	Grab Sample			
2			2.0 - 3.5	Ring Sample	93.3	4.2	3 5 7
4		brown					
5			5.0 - 6.5	Ring Sample	96.5	4.6	4 4 6
5.5		light gray brown					
10		medium dense, wet (water level)	10.0 - 11.5	Ring Sample	NO RECOVERY		4 8 10
12.5		light brown					
15	SP-SC	POORLY GRADED SAND WITH CLAY: light brown, medium dense, wet (Paralic Estuarine Deposits)	15.0 - 16.5	Ring Sample	112.6	19.5	13 13 19
20			20.0 - 21.5	Ring Sample	NO RECOVERY		7 8 9
23			23.0 - 24.5	Ring Sample	108.0	21.5	5 11 20
24.5		End of Boring @ 24.5' Subsurface water encountered @ 10.0' during and after drilling					

LEGEND: Ring Sample · Grab Sample · Shelby Tube Sample · SPT

NOTE: This log of subsurface conditions is a simplification of actual conditions encountered. It applies at the location and time of drilling. Subsurface conditions may differ at other locations and times.

APPENDIX B

Geotechnical Laboratory Test Results

Front Street Hotel 307453-001

BULK DENSITY TEST RESULTS

ASTM D 2937-17 (modified for ring liners)

November 20, 2025

BORING NO.	DEPTH feet	MOISTURE CONTENT, %	WET DENSITY, pcf	DRY DENSITY, pcf
2	3.0 - 3.5	2.9	97.9	95.1
2	6.0 - 6.5	1.3	115.2	113.7
2	11.0 - 11.5	18.5	136.4	115.1
2	16.0 - 16.5	17.2	137.5	117.3
2	21.0 - 21.5	19.2	111.7	93.7
2	26.0 - 26.5	23.0	124.5	101.2
2	31.0 - 31.5	25.1	128.7	102.9
2	36.0 - 36.5	24.7	126.2	101.2
2	41.0 - 41.5	18.4	128.2	108.3
2	51.0 - 51.5	27.5	124.1	97.3
3	3.0 - 3.5	4.2	97.3	93.3
3	6.0 - 6.5	4.6	101.0	96.5
3	16.0 - 16.5	19.5	134.5	112.6
3	24.0 - 24.5	21.5	131.3	108.0

EXPANSION INDEX TEST RESULTS

ASTM D 4829-21

BORING NO.	DEPTH feet	EXPANSION INDEX
2	1.0 - 5.0	16

Front Street Hotel 307453-001

PARTICLE SIZE ANALYSIS

ASTM D 422-63/07; D 1140-17

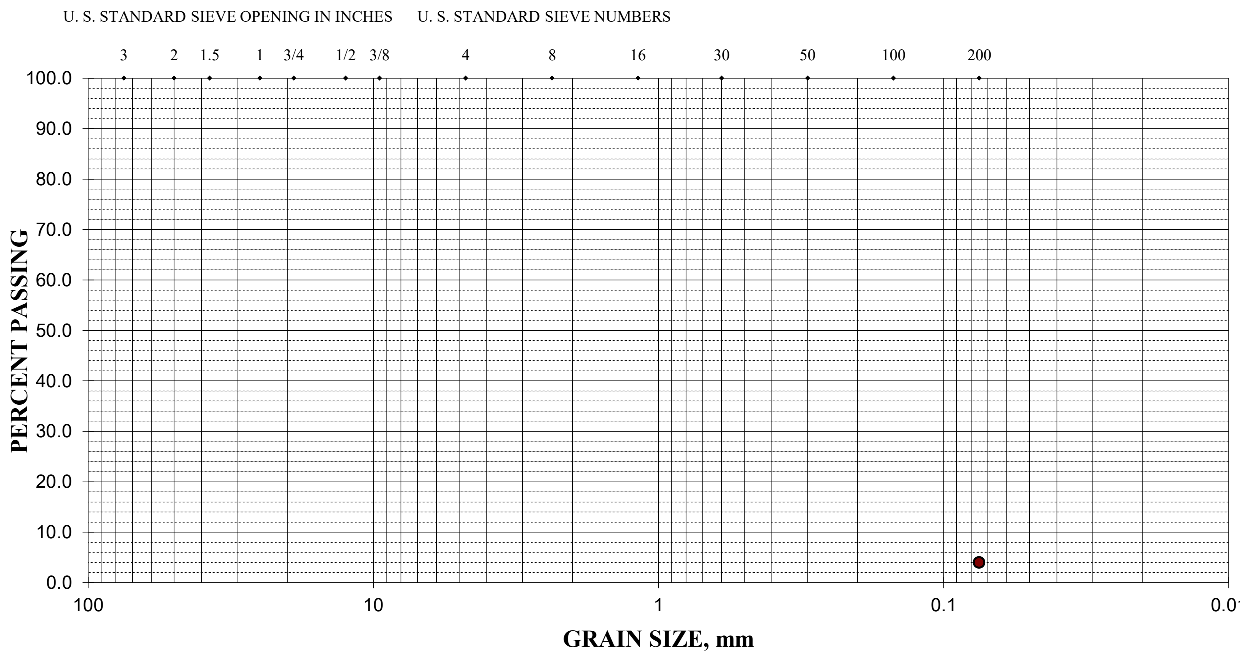
Boring #2 @ 11.0 - 11.5'

November 20, 2025

Poorly Graded Sand (SP)

Cu = 1.4; Cc = 0.9

Sieve size	% Retained	% Passing
#200 (75-µm)	96	4.0



Front Street Hotel 307453-001

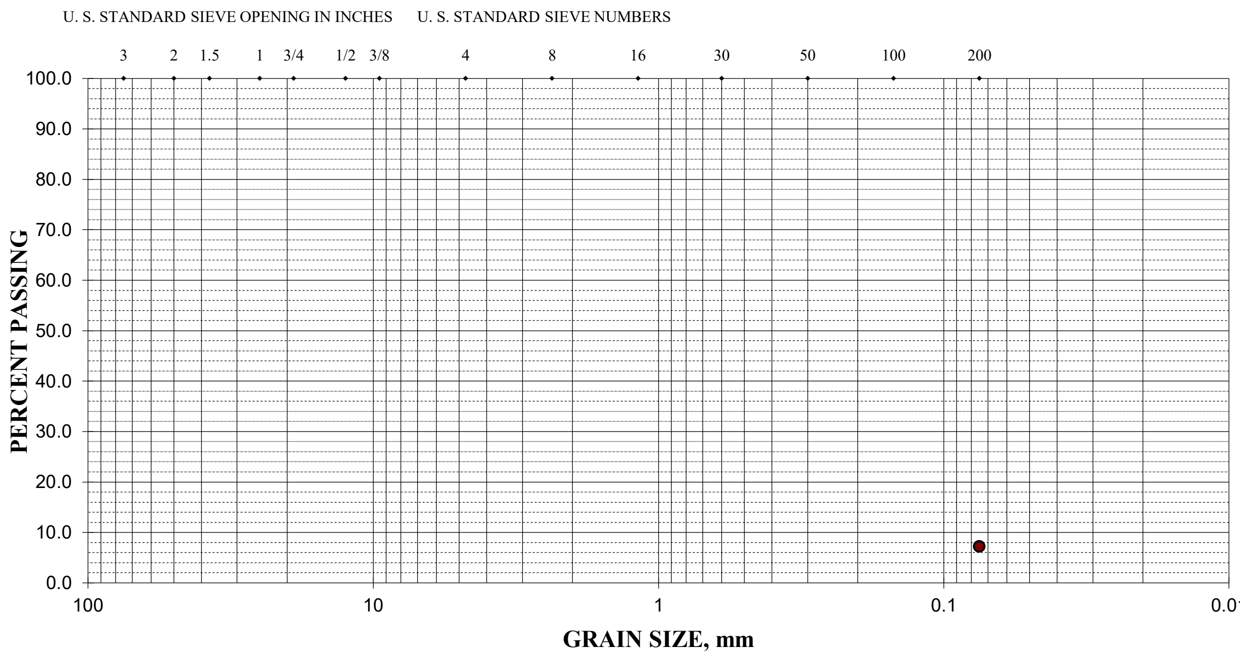
PARTICLE SIZE ANALYSIS

ASTM D 422-63/07; D 1140-17

Boring #3 @ 6.0 - 6.5'
Poorly Graded Sand with Clay (SP-SC)
Cu = 1.5; Cc = 0.9

November 20, 2025

Sieve size	% Retained	% Passing
#200 (75-µm)	93	7.2



Front Street Hotel 307453-001

DIRECT SHEAR

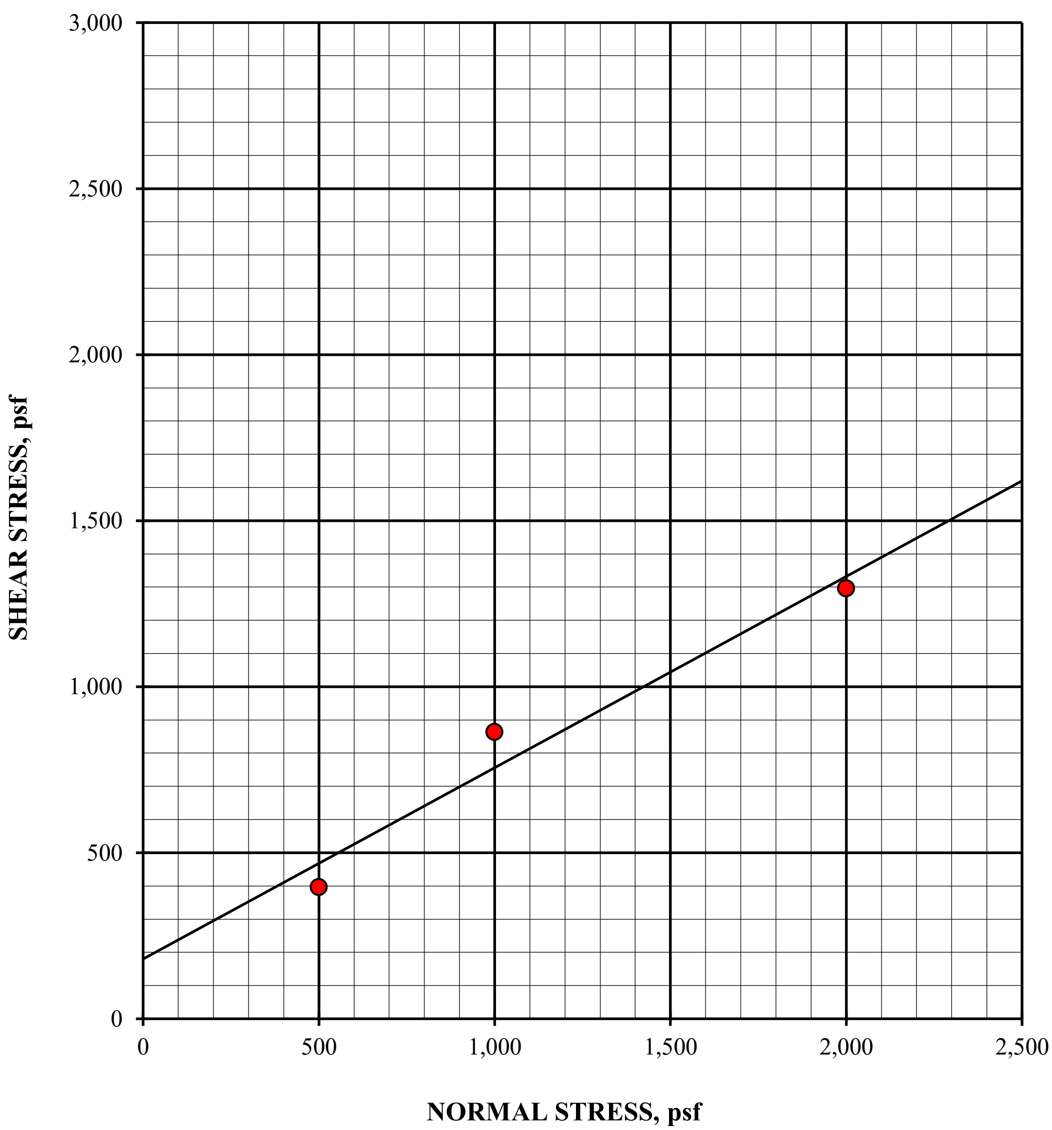
ASTM D 3080-23 (modified for consolidated, undrained conditions)

November 20, 2025

Boring #3 @ 6.0 - 6.5'
Poorly Graded Sand with Clay (SP-SC)
Ring sample, saturated

INITIAL DRY DENSITY: 95.6 pcf
INITIAL MOISTURE CONTENT: 4.6 %
PEAK SHEAR ANGLE (Ø): 30°
COHESION (C): 180 psf

SHEAR vs. NORMAL STRESS



Front Street Hotel 307453-001

DIRECT SHEAR continued

ASTM D 3080-23 (modified for consolidated, undrained conditions)

Boring #3 @ 6.0 - 6.5'

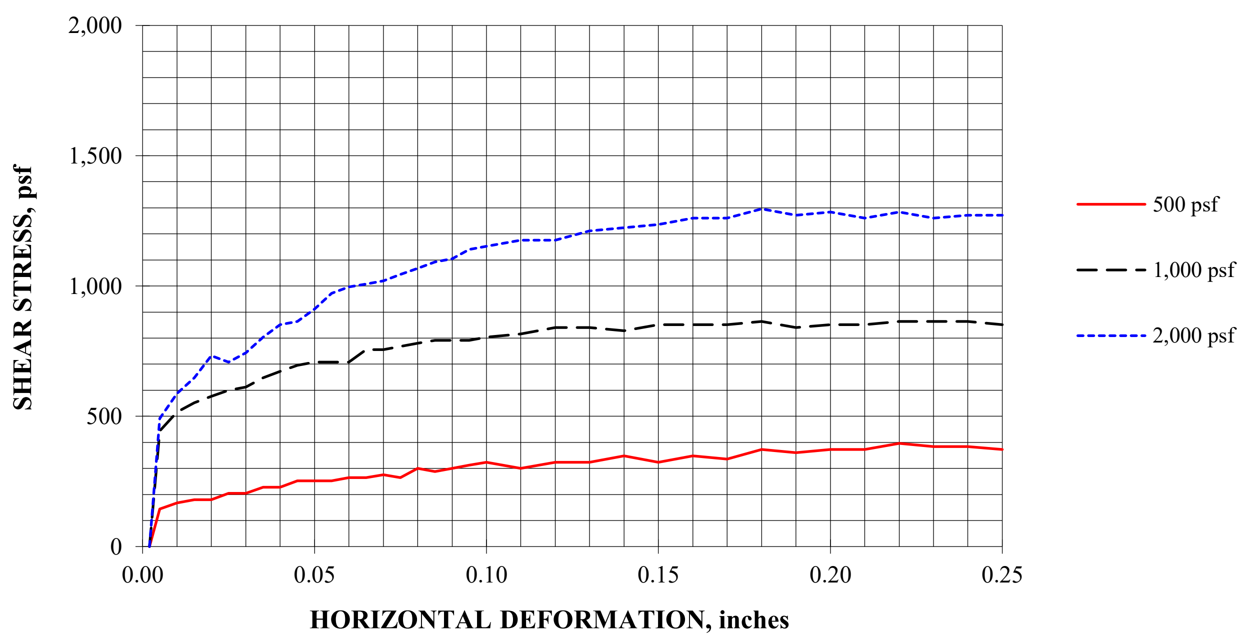
November 20, 2025

Poorly Graded Sand with Clay (SP-SC)

Ring sample, saturated

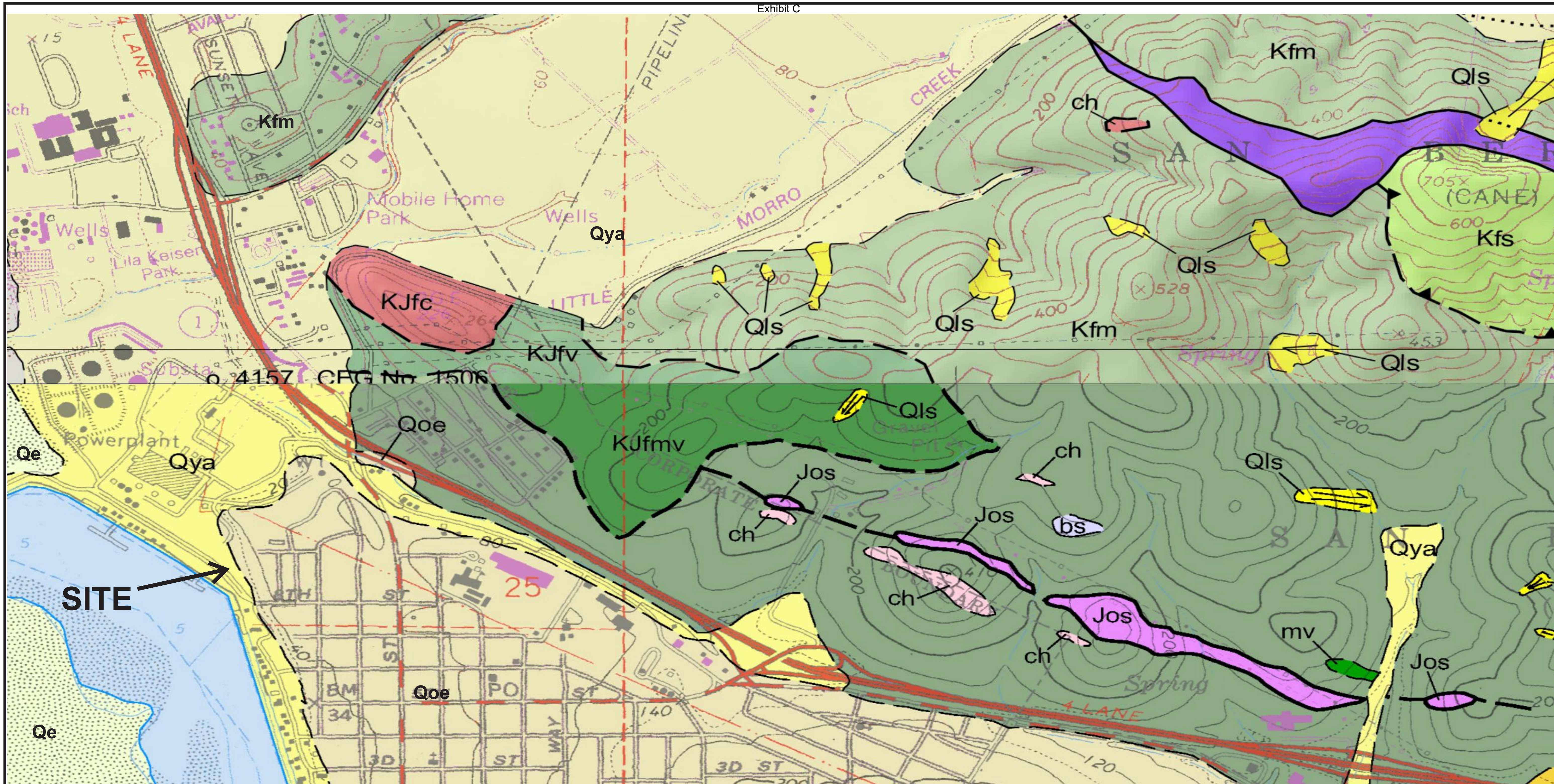
SPECIFIC GRAVITY: 2.65 (assumed)

SAMPLE NO.:	1	2	3	AVERAGE
INITIAL				
WATER CONTENT, %	4.6	4.6	4.6	4.6
DRY DENSITY, pcf	96.6	94.6	95.7	95.6
SATURATION, %	17.1	16.3	16.8	16.7
VOID RATIO	0.712	0.748	0.727	0.729
DIAMETER, inches	2.410	2.410	2.410	
HEIGHT, inches	1.00	1.00	1.00	
AT TEST				
WATER CONTENT, %	14.5	14.1	14.1	
DRY DENSITY, pcf	99.5	99.4	102.4	
SATURATION, %	57.9	56.4	60.8	
VOID RATIO	0.662	0.664	0.615	
HEIGHT, inches	0.97	0.95	0.94	



APPENDIX C

Exhibit C



Source: North 1/2 - Geologic Map of the Morro Bay North 7.5' Quadrangle, San Luis Obispo County, California: A digital Database, v. 1.0 by Marc O. Wiegers, 2016,
South 1/2 - Geologic Map of the Morro Bay South7.5' Quadrangle, San Luis Obispo County, California: A digital Database, v. 1.0 by Marc O. Wiegers, 2009, original scale 1:24,000, shown at 1:12,000

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REGIONAL GEOLOGIC MAP

Front Street Hotel
1180 Front Street
Morro Bay, California

Figure 3a

Date
December 2025

Project No.
307453-001

Qb **Beach and Active Dune Deposits (late Holocene)**—Unconsolidated, mostly fine- and medium-grained sand accumulated along the coastline; includes scattered cobbles.

Qd **Dune sands (late Holocene)**—Unconsolidated, well-sorted white to brown windblown sand. Forms active dunes behind modern beaches.

Qa **Alluvial flood plain and channel deposits (late Holocene)**—Active stream channel and recently active flood-plain deposits. Consist of unconsolidated, silty sand and sandy gravel with cobbles, scattered boulders with occasional lenses of silty clay.

Qls **Landslide deposits (Holocene to late Pleistocene)**—Includes comparatively shallow earth flow and debris slide deposits consisting of fragmented bedrock and soil mixtures, and deep rock slides of relatively intact bedrock displaced along rotational or translational slip surfaces. Most prevalent in ophiolitic serpentinite along the Oceanic Fault and in Franciscan melange.

Qya **Young alluvial flood-plain deposits, undivided (Holocene to late Pleistocene)**—Unconsolidated sand, silt and clay-bearing alluvium deposited on flood-plains and along valley floors. Surfaces on young deposits are undissected and lack soil development. Surfaces on older deposits are slightly dissected

Qpe 1-2 **Paralic estuarine deposits (late Holocene)** – Unconsolidated estuarine deposits composed of fine-grained sand and clay. Divided into:

- **Qpe1** **Salt marsh deposits -** Unconsolidated sand and clay underlying salt marshes at mouth of Chorro Creek and Los Osos Creek.
- **Qpe2** **Tidal flat deposits** Unconsolidated sand and clay underlying tidal flats of Morro Bay

Qe **Eolian Deposits (Holocene)** – Unconsolidated, well-sorted white to brown windblown sand. Forms active sand dunes along west side of Morro Bay.

SYMBOL EXPLANATION

Contact between map units - Solid where accurately located, dashed where approximately located, dotted where concealed.

Fault - Solid where accurately located, dashed where approximately located, dotted where concealed.

Synclinal axis - Solid where accurately located.

Anticlinal axis - Solid where accurately located.

t v

Linear features indicitave of faulting along the Los Osos Fault mapped by Treiman (1989) and shown on the Alquist-Priolo Earthquake Fault Zone Map, San Luis Obispo Quadrangle.
t = tonal contrast; v = vegetative lineament.

t v ld

Aerial photo lineaments along the Los Osos Fault (Lettis and Hall, 1994).
t = tonal contrast; v = vegetative lineament; ld = linear drainage.

Strike and dip of bedding.

Contour Interval 40 feet
Supplementary Contour Interval 20 feet
National Geodetic Vertical Datum of 1929

Tpm **Miguelito Member**—Brown to buff interbedded siltstone and claystone, moderately resistant, well –bedded, beds generally 2 to 4 inches thick. Locally includes beds and lenses of siliceous and dolomitic siltstone, opaline shale, porcelaneous shale, thin-bedded chert, diatomaceous shale, diatomite, friable and locally bituminous sandstone and locally conglomeratic or tuffaceous near base. (Hall and others, 1979).

Tpe **Edna Member**—Poorly to moderately well indurated, brown to gray, fine- to medium-grained arkosic sandstone. Locally interbedded with yellow claystone. Contains 35% to 80% quartz, 5% to 15% feldspar, up to 40% silt-sized particles (Hall, 1979).

Kfm embedded in a penetratively sheared matrix of argillite and crushed metasandstone. Penetrative deformation of the matrix postdates metamorphism of enclosed rock masses. Individual rock masses range from less than a meter to kilometers in scale and include altered mafic volcanic rocks (greenstone), chert, serpentinite, high-grade blueschist, graywacke, and conglomerate. Greenstone, chert, and serpentinite blocks are probably derived from the Coast Range Ophiolite and were emplaced and interleaved in the matrix during subduction. Small pods mapped locally are designated with abbreviated labels as follows:

mv – metavolcanic rock

sp – serpentinite

ch – chert

bs – blueschist

gw – graywacke

Larger slabs and blocks enclosed in mélange consist of the following:

Kfs **Sandstone of Cambria (Late Cretaceous)**—Light gray, fine- to coarse-grained, medium-bedded arkose and arkosic wacke interbedded with brown to black micaceous siltstone. Unit is more coherent and less sheared and fractured than other Franciscan units. Contains Late Cretaceous foraminifera and pollen (Graymer and others, 2014)

KJfg **Graywacke and Metagraywacke (Cretaceous and Jurassic?)**—Brown to greenish gray, fine- to medium-grained, massive- to thin-bedded graywacke sandstone interbedded with shale and siltstone. Composed of 60% to 70% quartz; 20% to 30% feldspar, 5% biotite and 10% shale fragments embedded in a muddy matrix (Hall and Prior, 1975). Rocks are generally moderately to intensely sheared, often obscuring original stratification. This unit lacks exotic blocks characteristic of mélange. Locally includes conglomerate beds with clasts of chert, sandstone and metavolcanic rock.

KJfc **Chert (Cretaceous and Jurassic)**—Red and green radiolarian chert associated with greenstone. Commonly veined and recrystallized, locally bleached to yellow or white. Deposited in deep oceanic setting on greenstone prior to influx of sandstone and shale. Locally interbedded with thin layers of argillite.

KJfv **Metavolcanic rocks (greenstone) (Cretaceous? and Jurassic)**—Primarily metamorphosed basalt and diabase. Includes massive to pillowed basalt flows, breccia and tuff. Commonly deeply weathered and extensively sheared, with intermingled pods of chert. Considered to be tectonic blocks incorporated into

Jsp **Serpentinized Ultramafic Rocks (Jurassic)**—Pervasively sheared serpentinite occurring as lenticular fault-bounded bodies in Franciscan mélange. Considered to be dismembered bodies of the Coast Range Ophiolite tectonically interleaved with mélange during subduction and entrained along faults. Locally altered to:

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REGIONAL GEOLOGIC MAP LEGEND

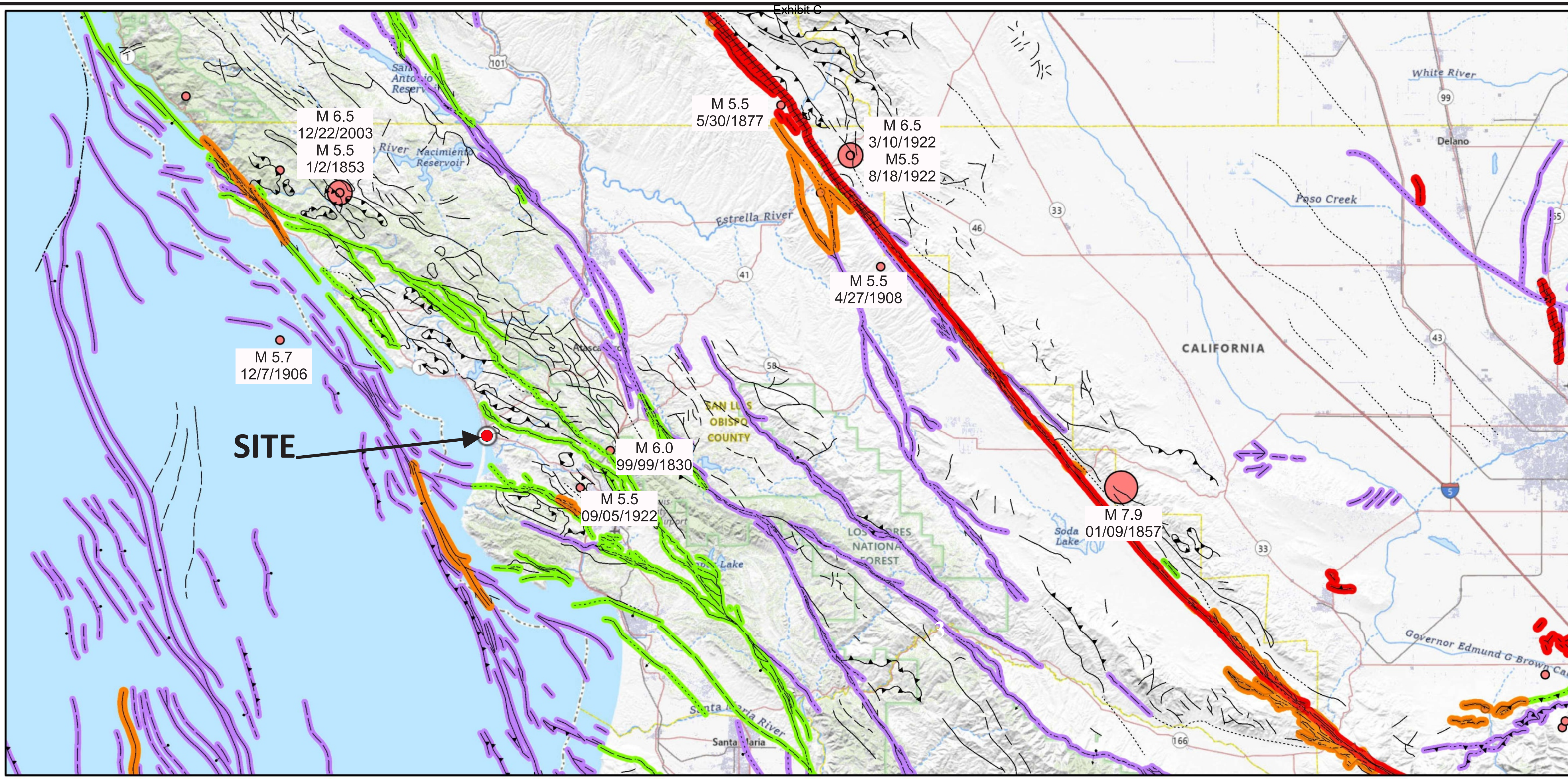
Front Street Hotel
1180 Front Street
Morro Bay, California

Figure 3b

Date
December 2025

Project No.
307453-001

Exhibit C



11/20/2025

Structural Discontinuity (Local)
structural discontinuity
Pre-Quaternary Faults
<all other values>
fault, certain
fault, approx. located
fault, concealed
thrust fault, certain
thrust fault, approx. located
thrust fault, approx. located, queried
fault, certain (ball and bar)
fault, approx. located (ball and bar)
fault, concealed (ball and bar)
Quaternary Faults
fault, certain
fault, approx. located
fault, approx. located, queried
fault, inferred, queried
fault, concealed
fault, concealed, queried
thrust fault, certain
thrust fault, approx. located
thrust fault, concealed
dextral fault, certain
dextral fault, approx. located
dextral fault, concealed
sinistral fault, certain
sinistral fault, approx. located
sinistral fault, concealed
thrust fault, certain (2)
thrust fault, approx. located (2)
thrust fault, concealed (2)
fault, certain (ball and bar)
fault, approx. located (ball and bar)
fault, concealed (ball and bar)
dextral fault, certain (ball and bar)
dextral fault, concealed, queried (ball and bar)
fault, concealed, queried (ball and bar, 2)
fault,certain (dip)
fault, approx. located (dip)
fault, concealed (dip)
reverse fault, certain
reverse fault, approx. located
reverse fault, concealed
Fault Creep
Fault Classification
Historic
Holocene
Late Quaternary
Quaternary
CGS Alquist Priolo Fault Zones
Epicenters
< 6.0
6.4 - 7.0
7.0 +
Quaternary Faults
fault, certain
fault, approx. located
fault, approx. located, queried
fault, inferred, queried
fault, concealed
fault, concealed, queried
thrust fault, certain
thrust fault, approx. located
thrust fault, concealed
dextral fault, certain
dextral fault, approx. located
dextral fault, concealed
sinistral fault, certain
sinistral fault, approx. located
sinistral fault, concealed
thrust fault, certain (2)
thrust fault, approx. located (2)
thrust fault, concealed (2)
fault, certain (ball and bar)
fault, approx. located (ball and bar)
fault, concealed (ball and bar)
dextral fault, certain (ball and bar)
fault, concealed, queried (ball and bar)
fault, concealed, queried (ball and bar, 2)
fault,certain (dip)
fault, approx. located (dip)
fault, concealed (dip)
reverse fault, certain
reverse fault, approx. located
reverse fault, concealed
Fault Classification
Historic
Holocene
Late Quaternary
Quaternary
USGSTopo
Red: Band_1
Green: Band_2
Blue: Band_3

N

USGS The National Map: National Boundaries Dataset, 3DEP Elevation Program, Geographic Names Information System, National Hydrography Dataset, National Land Cover Database, National Structures Dataset, and National Transportation Dataset; USGS Global Ecosystems; U.S. Census Bureau TIGER/Line data;

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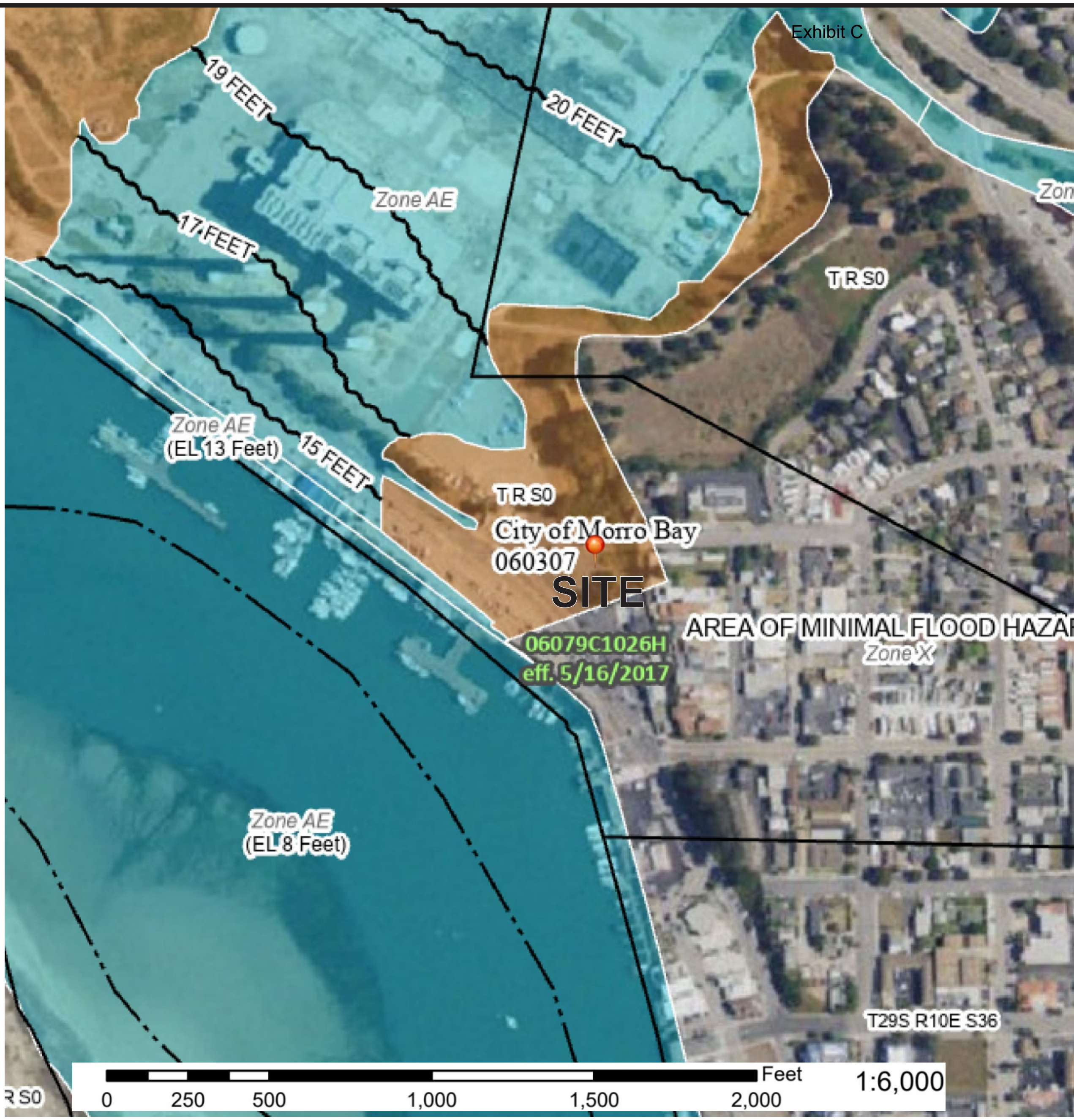
HISTORICAL SEISMICITY MAP
Front Street Hotel
1180 Front Street
Morro Bay, California

Figure 4

Date
December 2025

Project No.
307453-001



Legend

SEE FIS REPORT FOR DETAILED LEGEND AND INDEX MAP FOR FIRM PANEL LAYOUT

SPECIAL FLOOD HAZARD AREAS
- Without Base Flood Elevation (BFE) *Zone A, V, A99*
- With BFE or Depth *Zone AE, AO, AH, VE, AR*
- Regulatory Floodway

OTHER AREAS OF FLOOD HAZARD
- 0.2% Annual Chance Flood Hazard, Areas of 1% annual chance flood with average depth less than one foot or with drainage areas of less than one square mile *Zone X*
- Future Conditions 1% Annual Chance Flood Hazard *Zone X*
- Area with Reduced Flood Risk due to Levee. See Notes. *Zone X*
- Area with Flood Risk due to Levee *Zone D*

OTHER AREAS
- NO SCREEN Area of Minimal Flood Hazard *Zone X*
- Effective LOMRs
- Area of Undetermined Flood Hazard *Zone D*

GENERAL STRUCTURES
- Channel, Culvert, or Storm Sewer
- Levee, Dike, or Floodwall

OTHER FEATURES
- B 20.2 / 17.5 Cross Sections with 1% Annual Chance Water Surface Elevation
- 8 Coastal Transect
- 513 Base Flood Elevation Line (BFE)
- Limit of Study
- Jurisdiction Boundary
- Coastal Transect Baseline
- Profile Baseline
- Hydrographic Feature

MAP PANELS
- Digital Data Available
- No Digital Data Available
- Unmapped

N

The pin displayed on the map is an approximate point selected by the user and does not represent an authoritative property location.

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FEMA FLOOD ZONE MAP

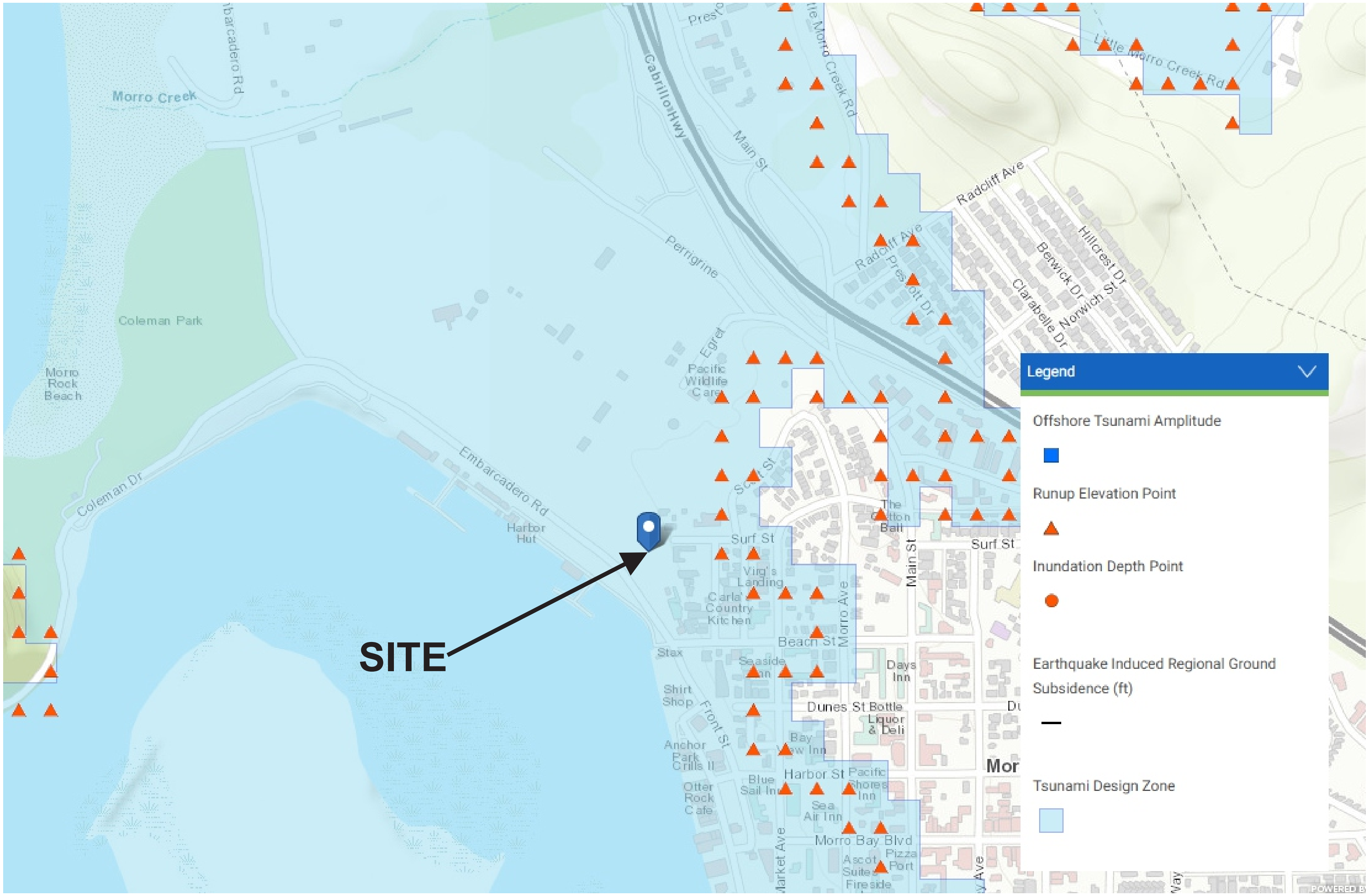
Front Street Hotel
1180 Front Street
Morro Bay, California

FIGURE 5

Date
December 2025

Project No.
307453-001

Exhibit C



Source: USGS National Map (https://apps.nationalmap.gov/viewer/ Not to Scale

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TSUNAMI INNUNDATION ZONE MAP
Front Street Hotel
1180 Front Street
Morro Bay, California

FIGURE 6

Date
December 2025

Project No.
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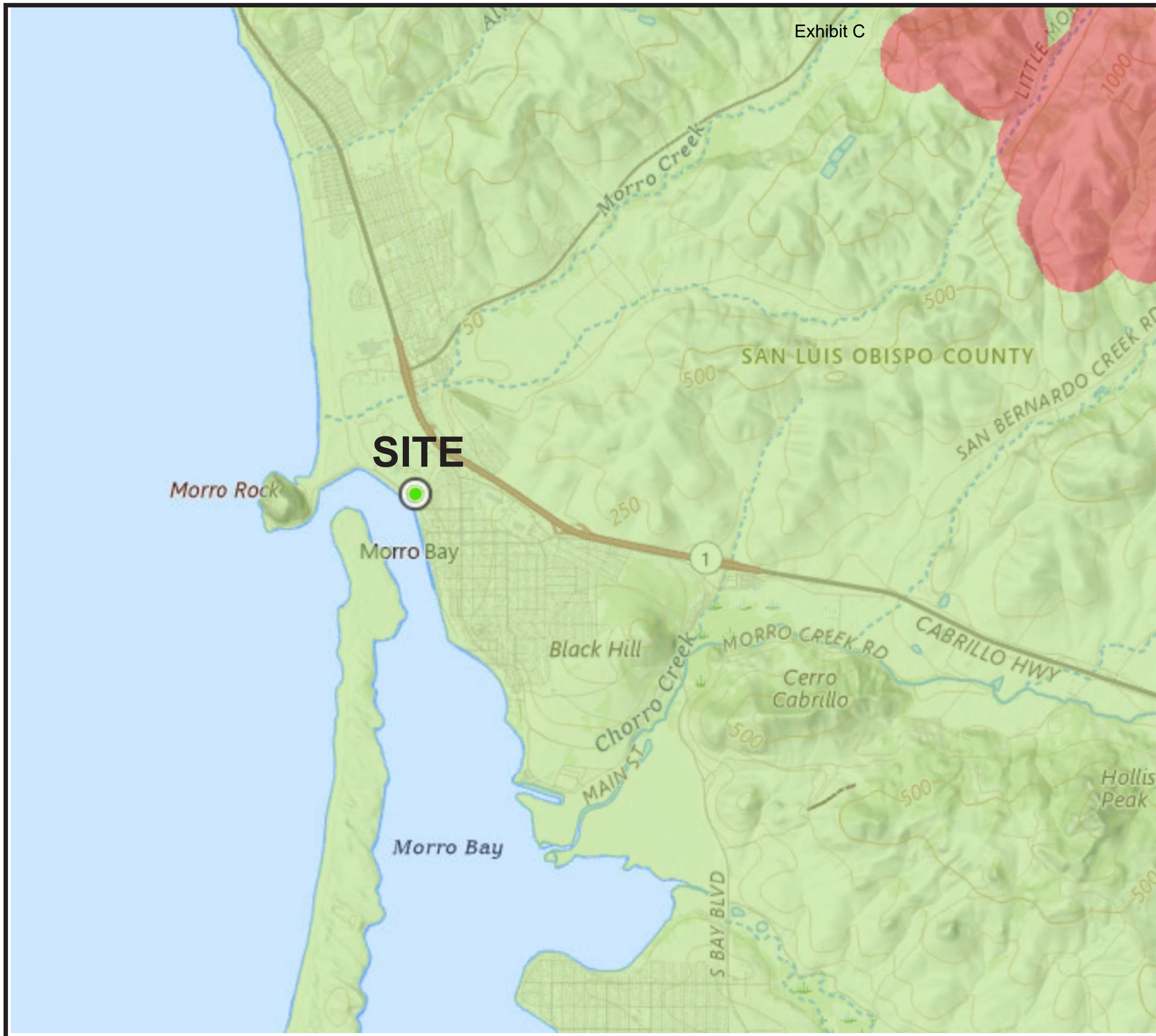

Legend

CGS Mineral Hazards: Indoor Radon Potential Zones

- Very High
- High
- Moderate
- Low
- Unknown

N

Site Coordinates: 35.3700, -120.8546, base map from California Geological Survey, after Churchill, 2008, not to scale.

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RADON POTENTIAL MAP
Front Street Hotel
1180 Front Street
Morro Bay, California

FIGURE 7

Date
December 2025

Project No.
307453-001

APPENDIX D

Site Specific Ground Motion Analysis – Tables D-1 through D-4

Table D-1
Fault Parameters

Fault Section Name	Distance (miles)	(km)	Upper Seis. Depth (km)	Lower Seis. Depth (km)	Avg Dip Angle (deg.)	Avg Dip Direction (deg.)	Avg Rake (deg.)	Trace Length (km)	Fault Type	**Mean Mag**	Mean Return Interval (years)	Slip Rate (mm/yr)
Los Osos 2011 CFM FM3.1, 3.2	4.5	7.2	0.0	12.0	45	208	90	58	B	**6.9**		0.5
Oceanic-West Huasna FM3.1, 3.2	6.1	9.8	0.0	7.0	58	49	na	122	B'	**7.1**		
Hosgri FM3.1, 3.2	8.2	13.2	0.0	6.8	80	59	180	171	B	**7.3**		2.5
Shoreline FM3.1, 3.2	9.2	14.8	0.0	12.0	90	na	na	23	B'	**6.5**		
San Luis Bay 2011 CFM FM3.2	12.4	19.9	0.0	10.0	90	na	na	16	B'	**6.3**		
San Luis Range - Pecho FM3.1, 3.2	12.5	20.1	0.0	12.0	90	na	na	26	B'	**6.6**		
San Luis Range 2011 CFM, FM3.1	13.0	21.0	0.0	12.0	52	na	na	79	B'	**7.2**		
San Luis Range (So Margin) FM3.2	13.5	21.8	0.0	12.0	45	37	90	115	B	**7.1**		0.2
Rinconada 2011 CFM FM3.1, 3.2	14.8	23.8	0.0	8.5	82	233	180	123	B	**7.5**		1
San Luis Range - Oceano 2011 CFM, FM3.1	20.4	32.9	0.0	12.0	45	na	na	21	B'	**6.6**		
East Huasna 2011 CFM FM3.1, 3.2	21.0	33.8	0.0	15.0	90	na	na	74	B'	**7.2**		
La Panza FM3.1, 3.2	22.0	35.4	0.0	13.9	51	45	na	72	B'	**7.3**		
Casmalia 2011 CFM	27.2	43.8	0.0	12.0	75	na	na	48	B'	**6.9**		
South Cuyama FM3.1, 3.2	29.8	47.9	0.0	13.9	33	210	na	83	B'	**7.5**		
Lions Head 2011 CFM FM3.1, 3.2	34.0	54.8	0.0	12.0	75	29	90	65	B	**6.7**		0.02
Hosgri (Extension) FM3.1, 3.2	36.1	58.1	0.0	7.5	80	79	na	29	B'	**6.4**		
San Juan FM3.1, 3.2	36.3	58.5	0.0	13.0	90	243	180	82	B	**7.1**		1
San Andreas (Cholame) rev FM3.1, 3.2	40.8	65.7	0.0	12.0	90	51	180	63	A	**6.8**	89	3.5
San Andreas (Parkfield) FM3.1, 3.2	41.7	67.1	0.0	10.2	90	50	180	36	A	**6.4**	13	20
San Andreas (Creeping Section) FM3.1, 3.2	46.7	75.1	0.0	12.0	90	227	180	121	A	**6.8**	89	9
Santa Ynez River FM3.1, 3.2	50.5	81.2	0.0	12.0	70	na	na	73	B'	**7.1**		
Los Alamos 2011 CFM FM3.1, 3.2	51.4	82.8	0.0	12.0	30	na	na	27	B'	**6.9**		
Morales (West) FM3.1, 3.2	54.6	87.8	0.0	8.6	32	49	na	28	B'	**6.8**		
San Andreas (Carrizo) rev FM3.1, 3.2	55.8	89.8	0.0	15.1	90	224	180	59	A	**6.8**	89	3.5
Lost Hills FM3.1, 3.2	60.3	97.0	4.2	12.0	29	233	na	33	B'	**6.8**		
Reliz 2011 CFM FM3.1, 3.2	62.0	99.8	0.0	10.9	58	240	na	127	B'	**7.4**		
Santa Ynez (West) FM3.1, 3.2	62.8	101.1	0.0	9.2	70	182	0	80	B	**6.9**		2
Great Valley 14 (Kettleman Hills) FM3.1, 3.2	66.1	106.4	8.1	22.5	22	215	90	24	B	**7.1**		1.5
Los Alamos extension FM3.1, 3.2	66.5	107.0	0.0	12.0	30	na	na	22	B'	**6.8**		
Great Valley 13 (Coalinga) FM3.1, 3.2	68.7	110.6	9.1	15.2	15	226	90	32	B	**7.0**		1.5
Morales (East) FM3.1, 3.2	71.8	115.6	0.0	8.6	32	14	na	18	B'	**6.6**		
San Gregorio (South) 2011 CFM FM3.1, 3.2	72.1	116.1	0.0	11.6	75	66	180	90	B'	**7.2**		
Great Valley 12 FM3.1, 3.2	72.7	117.0	7.0	9.6	15	243	90	17	B	**6.3**		1.5
Ozena FM3.1, 3.2	72.9	117.4	0.0	13.9	33	na	na	41	B'	**7.2**		
Calaveras (So) - Paicines extension FM3.1, 3.2	73.2	117.8	0.0	13.0	77	na	na	60	B'	**7.0**		
Red Mountain FM3.1, 3.2	74.8	120.4	0.0	14.1	56	2	90	101	B	**7.4**		2
Monterey Bay-Tularcitos FM3.1, 3.2	77.2	124.3	0.0	14.0	90	49	150	86	B	**7.3**		0.5
Great Valley 11 FM3.1, 3.2	80.9	130.3	7.0	9.6	15	221	90	24	B	**6.5**		1.5
Mission Ridge-Arroyo Parida-Santa Ana FM3.1, 3.	84.6	136.1	0.0	7.6	70	176	90	69	B	**6.8**		0.4
Big Pine (West) FM3.1, 3.2	84.7	136.3	0.0	11.0	50	2	na	18	B'	**6.5**		

Reference: USGS OFR 2013-1165 (CGS SP 228) Based on Site Coordinates of 35.37 Latitude, -120.8546 Longitude

Mean Magnitude for Type A Faults based on 0.1 weight for unsegmented section, 0.9 weight for segmented model (weighted by probability of each scenario with section listed as given on Table 3 of Appendix G in OFR 2008-1437). Mean magnitude is average of Ellworths-B and Hanks & Bakun moment area relationship.

Site Coordinates: 35.3700 N 120.8546 W

Table D-2
Historical Earthquakes in Vicinity of Project Site, M >= 5.0

Day	*Year*	*Epicenter* *Latitude* (Degrees)	*Longitude*	*Distance from Site (mi)*	*Magnitude* M_W
9/5	1922	35.30	120.70	10.0	**5.5**
99/99	1830	35.35	120.65	11.6	**6.0**
12/7	1906	35.50	121.20	21.4	**5.7**
2/1	1853	35.70	121.10	26.6	**5.5**
12/22	2003	35.70	121.10	26.6	**6.6**
11/22	1952	35.76	121.27	35.6	**6.2**
5/30	1877	35.70	120.30	38.6	**5.5**
4/27	1908	35.60	120.20	40.1	**5.5**
9/28	2004	35.82	120.37	41.3	**6.0**
12/28	1939	35.97	120.92	41.6	**5.2**
2/14	1987	35.96	120.70	41.7	**5.2**
3/10	1922	35.75	120.25	42.9	**6.3**
8/18	1922	35.75	120.25	42.9	**5.7**
6/8	1934	35.79	120.29	43.0	**6.0**
11/2	1955	36.00	120.92	43.7	**5.2**
2/26	1932	36.00	121.00	44.3	**5.0**
6/28	1966	35.81	120.27	44.7	**6.0**
9/29	2004	35.95	120.50	44.7	**5.1**
9/17	1991	35.81	121.45	45.2	**5.0**
11/16	1956	35.95	120.47	45.5	**5.0**
9/13	1975	36.00	120.55	46.7	**5.1**
7/28	1902	34.80	120.40	47.0	**5.8**
3/3	1901	36.00	120.50	47.8	**6.4**
2/2	1881	36.05	120.55	50.0	**6.0**
5/6	1881	36.05	120.55	50.0	**5.5**
1/9	1857	36.10	120.65	51.7	**6.1**
11/4	*1927	34.60	120.90	53.2	**7.1**
1/14	1855	36.15	120.70	54.5	**5.5**
7/31	1902	34.70	120.30	55.9	**5.8**
1/9	1857	36.20	120.80	57.4	**7.9**
4/12	1885	36.20	120.80	57.4	**6.5**
2/5	1947	36.23	120.65	60.5	**5.0**
9/2	1853	36.25	120.80	60.8	**6.3**
11/5	1969	34.65	121.50	61.7	**5.5**
1/9	1857	36.29	120.85	63.5	**5.6**
7/25	1983	36.22	120.41	63.8	**5.2**
6/11	1983	36.25	120.46	64.7	**5.4**
1/12	*1915	34.60	120.20	64.8	**5.7**
10/21	2012	36.31	120.86	65.0	**5.3**

From full earthquake catalog in USGS OFR 2008-1437h as updated with current events through 2024 (ANSS 2025). For events with an asterisk, alternate solutions are given in the OFR. Ordered By Closest Event. Maximum 40 Closest Events

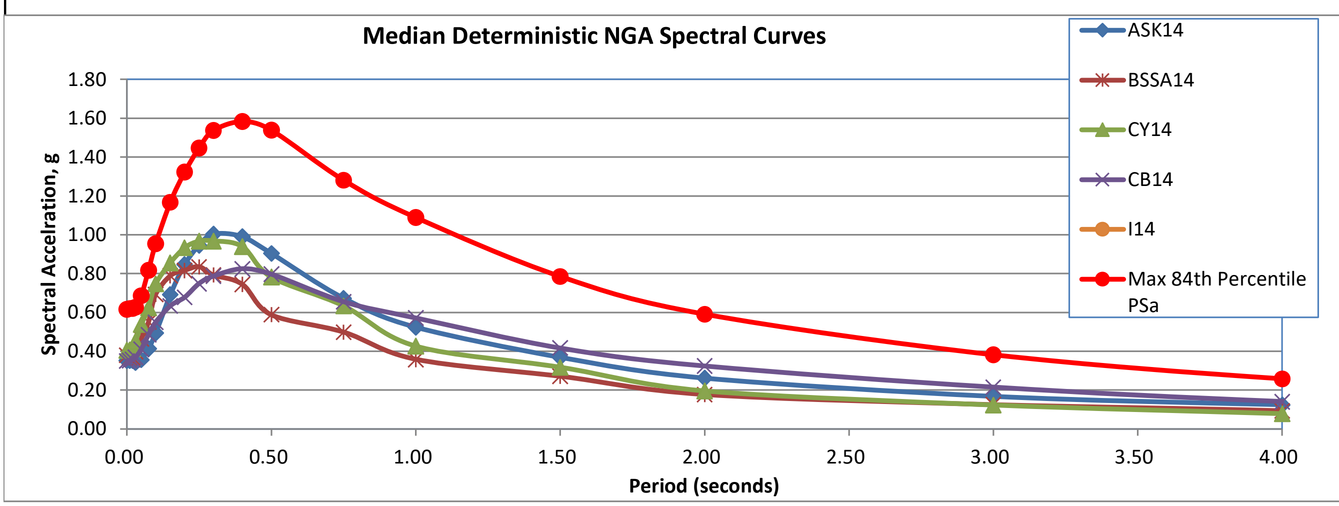
35.3700 -120.8546 Lat/Long

Table D-3 - Deterministic Spectral Response Values
Deterministic NGA Response Spectra for Largest Median Earthquake Ground Motion

Average of NGA: Abrahamson - Silva - Kamai (2014), Boore - Stewart - Seyhan - Atkinson (2013), Campbell-Bozorgnia (2013), Chiou - Youngs (2014), and Idriss (2013)

Mean Spectra Response from Attentuation Relationships

	ASK14		BSSA14	CB14	CY14	I14	Average			
Input Variables	**Period (sec)**	Median PSa (g)	Median PSa (g)	Median PSa (g)	Median PSa (g)	Median PSa (g)	**Period (sec)**	Mean PSa (g)	Max 84th Percentile PSa (g)	**Max Rotated Determ. PSa**
	Weight:	**0.25**	**0.25**	**0.25**	**0.25**	**0.00**				
M	**0.00**	0.35	0.37	0.35	0.40	-	**0.00**	0.370	0.616	**0.678**
7.15	**0.01**	0.35	0.38	0.35	0.40	-	**0.01**	0.372	0.620	**0.682**
R_{RUP}	**0.02**	0.35	0.37	0.36	0.41	-	**0.02**	0.371	0.621	**0.683**
7.20	**0.03**	0.34	0.37	0.37	0.41	-	**0.03**	0.373	0.626	**0.689**
R_{JB}	**0.05**	0.36	0.40	0.41	0.45	-	**0.05**	0.404	0.685	**0.754**
7.20	**0.075**	0.41	0.47	0.49	0.54	-	**0.075**	0.475	0.817	**0.899**
V_{S30}	**0.10**	0.49	0.55	0.55	0.62	-	**0.10**	0.555	0.954	**1.049**
260	**0.15**	0.69	0.69	0.63	0.75	-	**0.15**	0.691	1.167	**1.284**
F_{RV}	**0.20**	0.85	0.79	0.68	0.86	-	**0.20**	0.792	1.322	**1.454**
0	**0.25**	0.95	0.82	0.75	0.93	-	**0.25**	0.861	1.446	**1.659**
F_{NM}	**0.30**	1.00	0.83	0.79	0.97	-	**0.30**	0.898	1.537	**1.778**
0	**0.40**	0.99	0.79	0.82	0.97	-	**0.40**	0.893	1.583	**1.864**
W	**0.50**	0.90	0.74	0.80	0.94	-	**0.50**	0.846	1.538	**1.843**
17.00	**0.75**	0.67	0.59	0.65	0.78	-	**0.75**	0.674	1.281	**1.600**
Z_{TOR}	**1.00**	0.52	0.50	0.57	0.63	-	**1.00**	0.557	1.088	**1.415**
0.00	**1.50**	0.37	0.36	0.42	0.43	-	**1.50**	0.392	0.785	**1.040**
Z_{BOT}	**2.00**	0.26	0.27	0.32	0.32	-	**2.00**	0.294	0.590	**0.797**
12.00	**3.00**	0.17	0.18	0.21	0.20	-	**3.00**	0.189	0.382	**0.535**
dip	**4.00**	0.12	0.13	0.14	0.12	-	**4.00**	0.128	0.258	**0.374**
45	**5.00**	0.09	0.09	0.10	0.08	-	**5.00**	0.091	0.182	**0.273**
	7.50	0.05	0.05	0.04	0.03	-	**7.50**	0.043	0.085	**0.127**
	10.00	0.03	0.03	0.02	0.02	-	**10.00**	0.024	0.046	**0.070**



35.37 -120.8546 Lat/Long

Table D-4 - Site Specific Spectral Response Values
Probabilistic and Deterministic Response Spectra for MCE compared to Code Spectra
for 5% Viscous Damping Ratio

Natural Period T (seconds)	GeoMean Probab. 2% in 50 year MCE Spectrum	Rotated Probab. 2% in 50 year MCEr Spectrum	Rotated 84th Percentile Determ. MCE Spectrum	Determ. Lower Limit MCE Spectrum	Determ. MCE Spectrum	Site Specific MCE_R, Ground Response (SaM)	Site Specific MCE Spectrum Comparator	2022 CBC MCE Spectrum	Site Specific Design Spectrum (Sa)	2022 CBC Design Spectrum
	(1) 2475-year (ASCE 21.2.1)	(2) 2475-year (ASCE 21.2.1.1)	(3) (ASCE 21.2.2)	(4) (3) * 1.00=Scaling (ASCE 21.2.2)	(5) Max (3),(4) (ASCE 21.2.2)	(6) Min (2),(5) (ASCE 21.2.3)	(6b) Max (6),1.5*(8) (ASCE 21.2.3)	(7)	(8) (ASCE 21.3)	(9) 2/3*(7)
0.00	0.557	0.549	0.678	0.678	0.678	0.549	0.549	0.428	0.366	0.286
0.05	0.759	0.748	0.754	0.754	0.754	0.748	0.748	0.621	0.499	0.414
0.10	0.961	0.947	1.049	1.049	1.049	0.947	0.947	0.814	0.631	0.542
0.15	1.159	1.142	1.284	1.284	1.284	1.142	1.142	1.006	0.762	0.671
0.20	1.300	1.281	1.454	1.454	1.454	1.281	1.281	1.071	0.854	0.714
0.30	1.447	1.502	1.778	1.778	1.778	1.502	1.502	1.071	1.001	0.714
0.40	1.399	1.479	1.864	1.864	1.864	1.479	1.479	1.071	0.986	0.714
0.50	1.361	1.467	1.843	1.843	1.843	1.467	1.467	1.071	0.978	0.714
0.75	1.083	1.221	1.600	1.600	1.600	1.221	1.221	1.071	0.814	0.714
1.00	0.886	1.043	1.415	1.415	1.415	1.043	1.043	0.893	0.695	0.595
1.50	0.650	0.780	1.040	1.040	1.040	0.780	0.780	0.595	0.520	0.397
2.00	0.483	0.590	0.797	0.797	0.797	0.590	0.590	0.446	0.393	0.298
3.00	0.316	0.400	0.535	0.535	0.535	0.400	0.400	0.298	0.267	0.198
4.00	0.228	0.299	0.374	0.374	0.374	0.299	0.299	0.223	0.199	0.149
5.00	0.174	0.236	0.273	0.273	0.273	0.236	0.236	0.179	0.157	0.119
8.00	0.100	0.135	0.151	0.151	0.151	0.135	0.135	0.112	0.090	0.074
10.00	0.069	0.094	0.070	0.070	0.070	0.070	0.071	0.071	0.048	0.048
			0.036							

C_{RS}:	0.896	
C_{R1}:	0.905	
Site Specific To:	0.178	$= 0.2 * S_{D1}/S_{DS}$
Site Specific Ts:	0.888	$= S_{D1}/S_{DS}$

The value of Fa used in Column (3) is defined within ASCE 21.2.2 Supplement 1. This Fa value only applies within Column (3).

Site Coefficients	
F_{PGA}	1.17
F_a	1.12
F_v	2.50

Probabilistic spectrum from 2018 USGS Ground Motion Mapping Program adjusted for site conditions and scaled to represent maximum response in a horizontal plane, in accordance with ASCE 7-16 Section 21.2

Mapped MCE Acceleration Values		
PGA	0.426	g
S_S	0.959	g
S_1	0.357	g

Risk Coefficients have been applied to Column (2); If Method 1 was utilized the Risk Coefficients, CRS and CR1 are presented above, if Method 2 was utilized the Risk Coefficients were obtained from the USGS Risk Targeted Ground Motion Calculator (https://earthquake.usgs.gov/designmaps/rtgm).

Seismic Site Class	**D**
Risk Category	**II**

Reference: ASCE 7-16, Chapters 21.2, 21.3, 21.4, 21.5, 11.4, and 11.8

Calculation Utilized ASCE7-16, Section 21.2.1.1 - Method 1

Short-Period Seismic Design Category:	**1-Second Period Seismic Design Category:**
D	D

Site-Specific Design Acceleration Values		
PGA_M	**0.557**	**g**
S_{DS}	**0.901**	**g**
S_{D1}	**0.800**	**g**

Vertical Coefficient (C_V)
1.28

Deterministic Fault Parameters			
Los Osos 2011 CFM FM3.1, 3.		R_{JB} (km)	7.2
Magnitude	7.15	R_{RUP} (km)	7.2
Distance (km)	7.2	Z_{TOR} (km)	0.0
Width (km)	17	Z_{BOT} (km)	12.0
Dip (Deg.)	45	V_{S30} (m/s)	260

Site-Specific MCE_R, 5% damped, Spectral Response Acceleration Parameter		
S_{MS}	**1.351**	**g**
S_{M1}	**1.200**	**g**

1 g = 980.6 cm/sec^2 =32.2 ft/sec^2

PSV (ft/sec) = $32.2(S_a)T/(2p)$

Key: Probab. = Probabilistic, Determ. = Deterministic, MCE = Maximum Considered Earthquake

APPENDIX E

Liquefaction Settlement Calculations – Boring 2

Exhibit C

Project: Front Street Hotel
Job No: 307453-001
Date: 11/26/2025
Boring: B2

Methods: **Liquefaction Analysis using Idriss & Boulanger Method (2004)**
Semi-empirical Procedures for Evaluating Liquefaction Potential During Earthquakes, 11th SDEE and 3rd ICEGE, Univ. of California, Berkeley, 2004.
Settlement Analysis from Tokimatsu and Seed (1987),JGEE ,Vol 113, No.8, ASCE
Modified by Pradel, JGEE, Vol 124, No. 4, ASCE

EARTHQUAKE INFORMATION:

Magnitude: 6.87 7.5
PGA, g: 0.50 0.42
MSF: 1.18
GWT: 10.0 feet
Calc GWT 5.0 feet
Remediate to: 0.0 feet

SPT N VALUE CORRECTIONS:

Energy Correction to N60 (C_E): 1.48
Drive Rod Corr. (C_R): 1 Default
Rod Length above ground (feet): 3.00
Borehole Dia. Corr. (C_B): 1.05
Sampler Liner Correction for SPT?: 0 No
Cal Mod/ SPT Ratio: 0.63

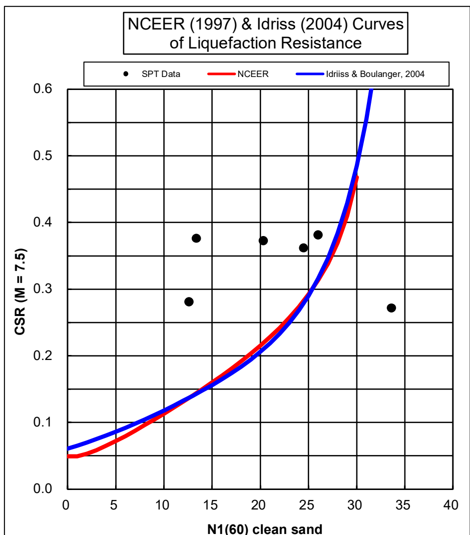
Total (ft) Liquefied Thickness	Total (in.) Induced Subsidence
26	5.2

Required SF: 1.20
Minimum SF: 0.38

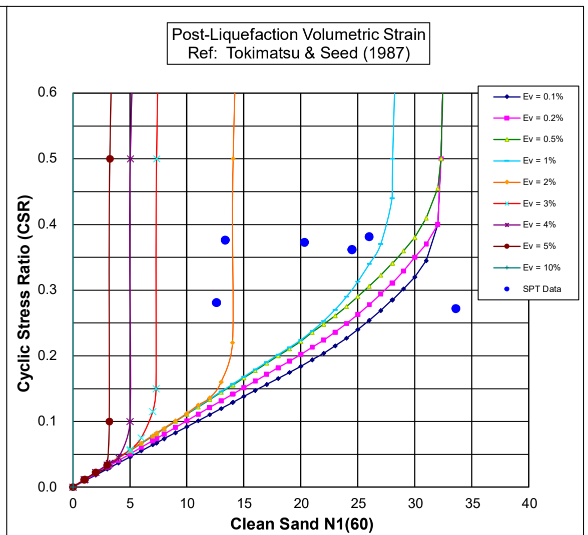
Nc = 9.9

Base Depth (feet)	Cal Mod N	SPT N	Liquef. Suscept. (0 or 1)	Total Unit Wt. (pcf)	Fines Content (%)	Depth of SPT (feet)	Rod Length (feet)	Tot.Stress at SPT σv (tsf)	Eff.Stress at SPT σ'v (tsf)	rd	C_N	C_R	C_S	$N_{1(60)}$	Rel. Dens. Dr (%)	FC Adj. $\Delta N_{1(60)}$	Equiv. Sand $N_{1(60)CS}$	Kσ	$CRR_{7.5}$	CSR*	Liquefac. Safety Factor	Volumetric Strain (%)	Induced Subsidence (in.)	p (tsf)	G_{max} (tsf)	τ_{av} (tsf)	Shear Strain γ	Strain E_{15}	Strain Enc	Dry Sand Subsidence (in.)
								0.000																						
5.0	9	6	1	98	10	3.0	6.0	0.147	0.147	1.00	1.70	0.75	1.00	11.2	49	1.1	12.4	1.00	0.135	0.275	**Non-Liq.**	0.00	**0.00**							
5.5	9	6	1	98	10	5.0	8.0	0.245	0.245	0.99	1.70	0.75	1.00	11.2	49	1.1	12.4	1.00	0.135	0.273	**Non-Liq.**	0.00	**0.00**							
10.0	26	16	1	115	10	6.0	9.0	0.298	0.298	0.99	1.70	0.75	1.00	32.5	84	1.1	33.6	1.00	0.844	0.272	**3.10**	0.00	**0.00**							
15.0	12	8	1	136	4	11.0	14.0	0.597	0.565	0.97	1.37	0.78	1.00	12.6	52	0.0	12.6	1.00	0.1	0.281	**0.49**	2.21	**1.33**							
20.0	41	26	1	138	10	16.0	19.0	0.938	0.751	0.94	1.19	0.88	1.00	41.9	95	1.1	43.1	1.00	13.0	0.325	**>10**	0.00	**0.00**							
25.0	45	28	1	112	10	21.0	24.0	1.269	0.926	0.92	1.07	0.94	1.00	44.5	98	1.1	45.6	1.00	42.919	0.347	**>10**	0.00	**0.00**							
28.0	24	15	1	125	10	26.0	29.0	1.555	1.055	0.89	1.00	0.99	1.00	23.3	71	1.1	24.5	1.00	0.278	0.362	**0.77**	1.18	**0.42**							
35.0	21	13	1	129	10	31.0	34.0	1.872	1.217	0.86	0.93	1.00	1.00	19.2	65	1.1	20.3	0.98	0.210	0.373	**0.56**	1.52	**1.27**							
40.0	29	18	1	126	10	36.0	39.0	2.193	1.382	0.84	0.88	1.00	1.00	24.8	73	1.1	26.0	0.96	0.316	0.381	**0.83**	1.09	**0.65**							
46.0	15	9	1	100	10	41.0	44.0	2.495	1.528	0.81	0.83	1.00	1.00	12.2	52	1.1	13.4	0.96	0.143	0.376	**0.38**	2.10	**1.51**							
51.5	82	52	1	124	10	51.0	54.0	3.055	1.776	0.75	0.77	1.00	1.00	62.0	100	1.1	63.1	1.00	#######	0.354	**>10**	0.00	**0.00**							

NCEER (1997) & Idriss (2004) Curves of Liquefaction Resistance



Post-Liquefaction Volumetric Strain
Ref: Tokimatsu & Seed (1987)



$N_{1(60)}$ = C_N*C_E*C_B*C_R*C_S*N
C_R = 0.75 for Rod lengths < 3m, 1.0 for > 10m
= min(1,max(0.75,1.4666-2.556/(z)$^{0.5}$))
C_N = min(1.7,(pa/σ'v)^[0.784-0.0768*min(46,$N_{1(60)}$)$^{0.5}$]
C_S = max(1.1,min(1.3,1+$N_{1(60)}$/100)) for SPT without liners
MSF = min(1.8,6.9*exp(-M/4)-0.058)
z = Depth (m)
rd = exp[(-1.012-1.126sin(z/11.73+5.133))+(0.106+0.118sin(z/11.28+5.14
pa = 1 atm = 101 KPa = 1.058 tsf
$\Delta N_{1(60)}$ = exp[1.63+9.7/FC-(15.7/FC)^2]
$N_{1(60)CS}$ = $N_{1(60)CS}$ + $\Delta N_{1(60)}$
Kσ = min(1,1-min(0.3,1/(18.9-17.3Dr))*ln(p'o/1.058))
Dr = $(N_{1(60)}/46)^{0.5}$

CSReq = 0.65*PGA*(po/p'o)*rd
CSR* = CSReq/MSF/Kσ
$CRR_{7.5}$ = exp((N/14.1)+(N/126)^2-(N/23.6)^3+(N/25.4)^4-2.8)
N = $N_{1(60)CS}$

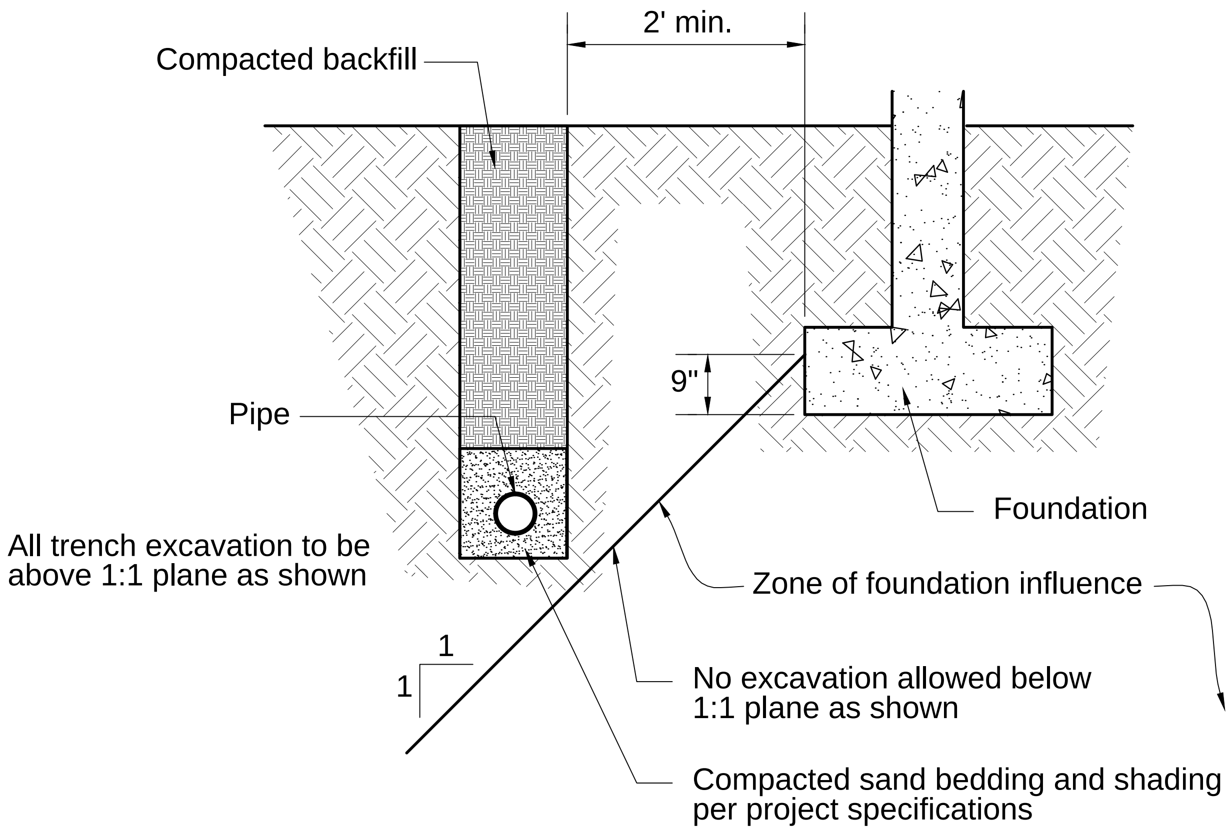
SF = $CRR_{7.5,1atm}$/CSR*

p = 0.67*po
τ_{av} = 0.65*PGA*po*rd
G_{max} = 447*$N_{1(60)CS}^{(1/3)}$*$p^{0.5}$ -sand, 10
ρ = γ/g
Vso = $(G/\rho)^{0.5}$
a = 0.0389*(p/1)+0.124
b = 6400*$(p/1)^{(-0.6)}$
γ = [1+a*EXP(b*τ_{av}/G_{max})]/[(1+a)*τ_{av}
E_{15} = γ*$(N_{1(60)CS}/20)^{-1.2}$
Nc = $(MAG-4)^{2.17}$
E_{nc} = $(Nc/15)^{0.45}$*E15
S = 2*H*E_{nc}

APPENDIX F

Typical Detail A: Pipe Parallel to Foundations

TYPICAL DETAIL A
PIPE PLACED PARALLEL TO FOUNDATION



SCHEMATIC ONLY
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