

Appendix G

Geotechnical Report

**REVISED GEOTECHNICAL ENGINEERING REPORT UPDATE
GOLETA BUSINESS CENTER BUILDING
(FORMELY NAMED SYWEST DEVELOPMENT COMMERCIAL BUILDING)
907 SOUTH KELLOGG AVENUE
GOLETA, CALIFORNIA**

January 27, 2023

Prepared for:

Mr. Tracy LaTray
SyWest Development

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January 27, 2023

File No.: 305544-001

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PROJECT: GOLETA BUSINESS CENTER BUILDING
(FORMERLY NAMED SYWEST DEVELOPMENT COMMERCIAL BUILDING)
907 SOUTH KELLOGG AVENUE, GOLETA, CALIFORNIA

SUBJECT: Revised Geotechnical Engineering Report Update

REF: 1) Environmental Review Peer Review Memorandum for the SyWest Industrial Building, 907 South Kellogg Avenue, Goleta, California, by Rincon Consultants, Inc., dated January 23, 2023, Project No. 22-13188

2) Geotechnical Engineering Report Update, Goleta Business Center Building, Formerly Named SyWest Development Commercial Building, 907 South Kellogg Avenue, Goleta, California, by Earth Systems Pacific, dated August 24, 2022, Doc. No. 2208-054.SER

Dear Mr. LaTray:

In accordance with your request, this revised geotechnical engineering report update has been prepared for use in the development of plans and specifications for the Goleta Business Center Building project. The project is located at 907 South Kellogg Avenue in the City of Goleta, California. This project was formally named SyWest Development Commercial Building.

The purpose of the revision to Reference 2 is to address the recommendations for report revisions presented by Rincon Consultants, Inc. in their environmental peer review letter to the City of Goleta (Reference 1).

Reference 2 was originally written to comply with the considerations of the 2019 Edition of the California Building Code (CBC); however, the requirements of the 2022 CBC will now govern the project. This report update is intended to generally comply with the 2022 CBC and to replace Reference 2; however, all the field and laboratory test data from Reference 2 has been included within this report.



Preliminary geotechnical recommendations for site preparation, grading, utility trenches, foundations, retaining walls, slabs-on-grade and exterior flatwork, pavement sections, drainage and maintenance, and construction observation and testing are presented herein. An electronic copy of this report is being furnished for your use.

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

Doug Dunham, GE
Associate Engineer

Doc. No.: 2301-039.SER/In





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1.0 INTRODUCTION

The Goleta Business Center Building along with its associated surface and subsurface improvements are planned within a property located at 907 South Kellogg Avenue in the City of Goleta, California. The general area where this project is planned within this property is referred to herein as "the site," and the site is shown on the Exploration Location Map presented in Appendix A.

We understand the following about the project. This approximately 71,600 square-foot commercial building will be one-story, will utilize Portland cement concrete (PCC) tilt-up walls and slab-on-grade, and the roof will be of wood and steel frame construction. Masonry or PCC retaining walls less than 6 feet in height may also be constructed. No other structures are planned. For the purposes of this report, maximum line loads of 3 kips/foot (3,000 lbs./foot), and maximum point loads of 30 kips (30,000 lbs.) were assumed.

We have assumed surface improvements will consist of hot mix asphalt (HMA) and/or PCC pavement placed over aggregate base (AB) for vehicle use, and PCC flatwork for pedestrian use. Subsurface improvements are assumed to be the underground conduits associated with the municipal utilities that will provide services for the site. Low Impact Development (LID) drainage disposal improvements are planned at the site; however, the location(s) and type(s) are unknown. No on-site effluent disposal systems are planned. Accordingly, LID drainage disposal improvements and effluent disposal systems are not within our scope of work and are not included within this report.

We understand the site is approximately 4 to 6 feet below final grades; therefore, import soils will be needed to develop the building and surface improvement areas, to improve access, and to improve drainage.

2.0 SCOPE OF SERVICES

The scope of work for the revised geotechnical engineering report update included a general site reconnaissance, review of Reference 2, geotechnical analysis of data, and preparation of this



report. The report and subsequent preliminary geotechnical recommendations were based, in part, on information provided by the client.

This report and preliminary geotechnical recommendations are intended to comply with the considerations of Sections 1803.1 through 1803.6, J104.3 and J104.4 (CBSC, 2022), and common geotechnical engineering practice in this area under similar conditions at this time. The 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.

Preliminary geotechnical recommendations for site preparation, grading, utility trenches, foundations, retaining walls, slabs-on-grade and exterior flatwork, pavement sections, drainage and maintenance, and construction observation and testing are presented to guide the development of project plans and specifications. It is our intent that this report be used exclusively by the client to form the geotechnical basis of the design of the project and in the preparation of plans and specifications. Application beyond this intent is strictly at the user's risk. If future property owners wish to use this report, such use will be allowed to the extent the report is applicable, only if the user agrees to be bound by the same contractual conditions as the original client, or contractual conditions that may be applicable at the time of the report use.

This report does not address 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, dewatering, shoring, temporary slope angles, construction means and methods, etc. Analyses of the soil for asbestos (either man-made or naturally occurring), radioisotopes, mold or other microbial content, hydrocarbons, lead, and/or other chemical properties (except for geotechnical corrosivity) are beyond the scope of this report. Ancillary features such as temporary access roads, fencing and site walls, flag and light poles, signage, LID drainage disposal improvements, effluent disposal systems; and nonstructural fills are not within our scope and are also not addressed in this report.



As there may be unresolved geotechnical issues with respect to this project, the geotechnical engineer should be retained to provide consultation as the design progresses, and to review project plans as they near completion to assist in verifying that pertinent geotechnical 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 modified or verified by the geotechnical engineer 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 in the field during construction, and the recommendations have been verified as appropriate or are modified by the geotechnical engineer in writing.

3.0 SITE SETTING

The site is located at 907 South Kellogg Avenue which is in the southeastern sector of the City of Goleta, California. A driveway off South Kellogg Avenue will provide access to the site. The approximate central site coordinates and elevation from the Google Earth website (Google, 2022) are latitude 34.4263 degrees north, longitude 119.8270 degrees west, and 8 feet above mean sea level.

The site is just east of the Santa Barbara Airport in a commercial/industrial area and is in the northern half of an existing but closed two screen "Drive-In" movie theater complex. The site currently supports a masonry movie projector building, and surface and subsurface improvements. The site is mostly devoid of vegetation except in the general areas next to the movie screen and the southeast boundary along the entrance driveway which are landscaped. San Jose Creek is aligned near the southeast side of the entrance driveway. Topographically, the site ranges from relatively flat along the perimeter and the various driveway aisles but is gently sloping where the vehicles park on graded berms to view the elevated movie screen. Surface drainage is by sheet flow.



4.0 FIELD INVESTIGATION AND LABORATORY ANALYSIS

On June 21 and 22, 2017, five exploratory borings were drilled at the site to depths of approximately 26.5 feet and 51.5 feet below the existing ground surface (bgs). The borings were drilled with a Mobile Drill Model B-53 drill rig, equipped with 6-inch outside diameter hollow stem auger and an automatic trip hammer for sampling. The approximate locations of the exploratory borings are shown on the Exploration Location Map presented in Appendix A.

Soils encountered in the exploratory borings were categorized and logged in general accordance with the Unified Soil Classification System and ASTM D 2488-09a. Copies of the boring logs and a Boring Log Legend are presented 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 cobbles or boulders, 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 descriptions that vary somewhat from the legend.

As the borings were drilled, soil samples were obtained using a ring-lined barrel sampler (ASTM D3550-01/07 with shoe similar to D2937-10). Standard penetration tests (SPT) were also performed in the borings (ASTM D1586-11) at selected depths. Bulk soil samples were obtained from the auger cuttings.

Ring samples were tested for bulk density (ASTM D2937-10, as modified for ring liners). One bulk sample was tested for maximum density and optimum moisture content (ASTM D1557-12). A direct shear test (ASTM D3080/D3080M-11) was conducted on the same bulk sample after it was remolded to approximately 90 percent of maximum dry density. Particle size tests (ASTM D422-63/07 and D1140-14) were conducted on three SPT bulk samples. Two one-dimensional consolidation tests (ASTM D2435/D2435M-11) were performed on ring samples. Two bulk samples were also sent to CERCO Analytical, Inc. of Concord, California for use in preparing a corrosion evaluation report. The evaluation and associated test results are for use by the



architect/engineer in determining appropriate corrosion mitigation measures. The laboratory test results and the corrosion evaluation report prepared by CERCO Analytical, Inc. are presented in Appendices B and C, respectively.

On June 28, 2017, four cone penetrometer test (CPT) soundings were also performed at the site. The CPT soundings were advanced to a depth of approximately 60 feet bgs. The approximate locations of the CPT soundings are also shown on the Exploration Location Map presented in Appendix A. The CPT soundings were conducted in general accordance with ASTM D5778-12 and D3441-16 using an electric cone penetrometer. Copies of the graphical CPT data can also be found in Appendix A.

5.0 GENERAL SUBSURFACE PROFILE

The general subsurface profile observed within the borings and encountered in the CPT soundings consisted of fill soils overlying alluvial soils that were further underlain with estuary deposit soils. All these soils exist as layered sands, or layered sands and clays. The sands were in a moist to wet condition with a loose consistency; the clays were moist to wet and very soft to medium stiff. Variable amounts of shells were observed within the estuary deposit soil layers. Subsurface water was encountered at depths ranging from approximately 3 to 7.5 feet below the existing ground surface within all the borings. Please refer to the boring logs and graphical CPT data for a more complete description of the subsurface conditions.

Based on the subsurface soil profile described above and the sampler blow counts, the Site Class per Chapter 20 Table 20.3-1 (ASCE, 2017) is "E - a Soft Clay Soil Profile."

6.0 FAULTS

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 documented 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 (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 north branch of the Red Mountain fault located approximately 10 miles east of the site; the south branch of the Red Mountain fault is located offshore approximately 3.4 miles south of the site, but this branch is not included in an A-P Zone (CGS, 2002 and USGS, 2013). There are several possible Holocene and late Quaternary faults within the region. The Mission Ridge fault system is the closest mapped fault to the site, with the westernmost fault of this system, the More Ranch fault, located approximately 690 feet south of the site (Minor et. al., 2007). Other regional faults include the Red Mountain, Pitas Point, Oak Ridge, Santa Ynez (west), Los Alamos, and San Andreas faults. Several of the regional faults considered capable of generating strong shaking at the site are discussed below.

Mission Ridge Fault System

The Mission Ridge fault system, which includes the More Ranch, Mission Ridge, Arroyo Parida, and Santa Ana faults make up an approximately 45 mile long, essentially continuous fault system extending from Goleta to Ojai along the southern flank of the Santa Ynez Mountains. The seismogenic plane is modeled by the USGS 0.3 miles north of the site (2013). The Mission Ridge fault system has likely ruptured during the Holocene epoch but lacks sufficient exposure to be included in an A-P Zone (Gurrola and Keller, 2003). The Mission Ridge fault system is reported to be capable of a magnitude 6.97 earthquake (USGS, 2014).



Red Mountain Fault

The late Quaternary to Holocene north branch Red Mountain fault, modeled approximately 3.4 miles south of the site, is a north dipping reverse fault with an onshore length of approximately 10 miles. The on-shore portion in the closest fault to the site included in an A-P Zone. The south branch Red Mountain fault is located off-shore approximately 3.4 miles south of the site. The Red Mountain fault is estimated to be capable of a magnitude 7.41 earthquake (USGS, 2014).

Pitas Point Fault

The Pitas Point fault is a north dipping reverse fault with a length of approximately 12 miles. It has likely ruptured during the Holocene epoch but lacks the exposure to be included in and A-P Zone. The fault is modeled approximately 6.4 miles south of the site and is considered capable of earthquakes up to magnitude 7.21 (USGS, 2014).

Oak Ridge Fault

The Oak Ridge fault is a segmented fault with on-shore and off-shore segments. The off-shore segment is located approximately 8.2 miles south of the site and is estimated to be capable of a magnitude 6.91 earthquake (USGS, 2014).

Santa Ynez Fault

The Santa Ynez has a length of at least 80 miles and is divided into east and west subsections. It is a south dipping reverse fault that forms part of the northern boundary of the Santa Ynez Mountains. The Santa Ynez (West) fault is approximately 8.2 miles north of the site and is considered capable of earthquakes up to magnitude 7.11 (USGS, 2014).

Los Alamos Fault

The Los Alamos fault is included in an A-P Zone at Zaca Creek and it is located approximately 9.3 miles northwest of the site. It is a northwest trending fault that is approximately 17 miles long. The southern end of the fault merges with the northern end of the West Baseline fault and is estimated to be capable of a magnitude 6.96 earthquake (USGS, 2014).



San Andreas Fault

The San Andreas Fault is considered the most active fault in the region; it lies about 43 miles northeast of the site. The 1857 Fort Tejon earthquake of approximate magnitude 7.9 was one of several large earthquakes identified in the paleoseismic record to have occurred on this fault. 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 may cause an earthquake of magnitude 8.0 or more.

7.0 GROUND MOTION ANALYSES

In accordance with the CBC (CBSC, 2022) and ASCE (2017, 2018, 2021), an assessment was made to determine the need for employing “Site-Specific Ground Motion Procedures” to calculate the ground motion parameters for the project. Based on our site evaluation, the subsurface characteristics are those of Seismic Site Class “E” (Soft Clay Soil Profile) as defined by Table 20.3-1 (ASCE, 2017), Site Classification, based on SPT blow counts and information from the CPT soundings.

In accordance with Chapter 20 (ASCE, 2017), the Seismic Site Class is “E, a Soft Clay Soil Profile,” as stipulated in Section 20.3.2 and Table 20.3-1. The mapped ground motion values were obtained from the Structural Engineers Association of California website (SEAOC, 2022) using the ASCE 7-16 (2017) for Site Class E. The value S_s is 2.348 which is greater than 1.0, and the S_1 value is 0.824 which is greater than 0.2; therefore, per Section 11.4.8 (ASCE, 2017), and Supplement 3 (ASCE, 2021), the project requires site-specific ground motion analyses. We performed a site-specific ground motion hazard analysis to develop the seismic design parameters in accordance with Sections 11.4.8, 21.2, Supplement 1, and Supplement 3 (ASCE 2017, 2018, 2021).

A risk-targeted maximum considered earthquake (MCE_R) modeling procedure was performed in accordance with ASCE 7-16 (ASCE, 2017) including a Probabilistic Seismic Hazard Analysis (PSHA) and a Deterministic Seismic Hazard Analysis (DSHA). These analyses are based on knowledge of the regional tectonic setting, geology, and seismicity. A PSHA using ground motion data from the USGS Unified Hazard Tool (USGS, 2022) and a DSHA using the Third California Earthquake Rupture



Forecast (UCERF3) fault model (USGS, 2013) and NGA-West2 ground motion prediction equations (PEER, 2015) were used as described in Section 21.2.1.1 (Method 1) (ASCE, 2017) to estimate the peak ground motion corresponding to the uniform hazards earthquake and MCE_R which has a 2% probability of being exceeded in 50 years.

Our DSHA analysis compared two earthquake scenarios and compared the calculated site-specific ground response anticipated to be generated. We considered an earthquake on the Mission Ridge-Arroyo Parida-Santa Ana fault with a potential magnitude of 6.97 at a distance of 0.3 miles, and an earthquake on the Red Mountain fault with a potential magnitude of 7.41 at a distance of 3.4 miles. The fault parameters that we considered in our analysis are shown on Table D-1 – Fault Parameters presented in Appendix D, and were modified in accordance with the USGS on-line Earthquake Scenario Map (USGS, 2014). The Site Specific Spectral Response Values listed in Table D-4 are based on the Red Mountain fault located 3.4 miles south of the site dipping north. We specified a 56-degree dip and hanging-wall conditions consistent with UCERF3 Fault Models 3.1 and 3.2 for the attenuation relationships. The primary seismic risks at the site are from earthquakes generated by the local Mission Ridge-Arroyo Parida-Santa Ana, Red Mountain, Oak Ridge, Santa Ynez (West), Los Alamos, Pitas Point and San Andreas faults. Although these listed faults are thought to potentially generate the most severe seismic shaking at the site any regional fault could produce seismic shaking.

The CBC (CBSC, 2022) seismic design criteria are based on a Design Earthquake that produces ground motion $2/3$ of the lesser of an earthquake with 2 percent probability of occurrence in 50 years, or maximum 84th percentile of the mean deterministic MCE. The deterministic spectra calculated with NGA-West2 ground motion prediction equations are shown on Table D-3 – Deterministic Spectral Response Values presented in Appendix D.

Seismic Design Category

Section 1613.2.5 (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... Other structures



shall be assigned to Design Category D. The mapped S_1 for the site is 0.824 which is greater than 0.75; therefore, the site should be assigned to the seismic design category E.

Seismic Design Parameters

This site may be subject to strong ground shaking due to potential fault movements along regional faults including the Red Mountain and Mission Ridge-Arroyo Parida-Santa Ana fault, whose proximity and magnitude potential was considered during our site-specific analysis. Engineered design and earthquake-resistant construction increase safety and allow development of seismically active areas. The minimum seismic design should comply with the CBC (CBSC, 2022) and ASCE 7-16 (ASCE, 2017). The resulting seismic coefficients considering Seismic Site Class E are summarized below. The results of our site-specific ground motion analysis are shown on Table D-4 – Site Specific Spectral Response Values presented in Appendix D.

SITE SPECIFIC SEISMIC PARAMETERS (2019 CBC/ASCE 7-16) (Method 1)

Seismic Design Category	E
Seismic Site Class	E
Mapped and Code Based Ground Motion	
Short Period Spectral Response, S_s	2.348 g
1 second mapped Spectral Response, S_1	0.824 g
Design Earthquake Ground Motion	
Short Period Spectral Response, S_{DS}	1.323 g
1 second Spectral Response, S_{D1}	1.961 g
Peak Ground Acceleration (PGA_M)	0.913 g
MCE Spectral Response Acceleration	
Short Period Spectral Response, S_{MS}	1.985 g
1 Second Period Spectral Response, S_{M1}	2.941 g
Site Amplification Factors	
Short Period Site Coefficient, F_a	1.00
1 Second Period Site Coefficient, F_v	4.00
Vertical Site Coefficient, C_v	1.50
Risk Coefficient (Short Period), C_{RS}	0.876
Risk Coefficient (1 Second Period), C_{R1}	0.875



Acceleration values provided are estimates only. Vertical accelerations are typically 1/3 to 2/3 of the horizontal accelerations but can equal or exceed the horizontal accelerations depending upon the fault type, local site effects and amplification.

Alternatively, two exceptions are stipulated in ASCE 7-16 Supplement 3 (ASCE, 2021) that permits the use of conservative design values. The general procedure values may be applicable depending on project parameters and design methods. The structural engineer shall verify that the exception applies before using the general procedure seismic design values. General procedure seismic design values are listed on Table D-5 presented in Appendix D. We have assumed that the fundamental period of the building will be 0.5 seconds or less. A site response analysis is required if the structure's fundamental period is greater than 0.5 seconds in accordance with Sections 20.2 and 20.3 (ASCE, 2017).

8.0 CONCLUSIONS

In our opinion, the site is suitable from a geotechnical standpoint for the project as described in the "Introduction" section of this report, provided the recommendations contained herein are implemented into the design and construction and that the potential risk of damage due to liquefaction is understood. The client has indicated that smaller isolated structures such as site walls, fencing, signage, etc., and surface improvements such as vehicle pavement and pedestrian flatwork should be considered sacrificial with respect to effects of liquefaction as the cost of protecting these items in a similar fashion to the building's foundation system generally outweighs the benefits of protection.

The upper site soils were judged to be generally nonexpansive; therefore, no special measures with respect to expansive soils are considered necessary. Assuming the site is designed and prepared in accordance with the "Preliminary Geotechnical Recommendations" section of this report, the planned structures may be supported by shallow foundation systems.

The primary geotechnical engineering concerns at this site are the potential for strong ground shaking, the potential for static settlement, the excavation characteristics of the soils, the



suitability of the soils for use as fill and backfill, the stability of the soils during grading, the erodible nature of the soils, the corrosive nature of the soils, the potential for surface ground rupture, the potential for liquefaction and seismically induced settlement of dry sand, and the type of shallow foundation systems.

Strong Ground Shaking

The site is in a region of high seismic activity with the potential for large seismic events that could generate strong ground shaking. The CBC (CBSC, 2022) requires that seismic loads are considered in structural design. A seismic analysis was undertaken to provide seismic acceleration design parameters; the results are presented in the “Ground Motion Analyses” section of this report for use by others in the structural design process.

Static Settlement Potential

Static settlement (total and differential) can occur when foundations and surface improvements span soil materials having variable moisture and density characteristics. Such a situation could stress and possibly damage foundations and surface improvements often resulting in severe cracks and displacement. To reduce this settlement potential, it is necessary for all foundations and surface improvements to bear on material that is as uniform as practicable. A program of overexcavation, scarification, moisture conditioning, and compaction of the upper soils in the building and the surface improvement areas is recommended to provide more uniform soil moisture and density and appropriate support.

Soil Excavation Characteristics

The soils are anticipated to be excavatable with conventional earthmoving equipment; however, the stability of excavations and the presence of shallow groundwater at the site are a concern. Based on our preliminary testing, the soils are considered to be “Type C” soils per the Cal/OSHA classification system (Cal/OSHA, 2019), and layers of flowing sands may be encountered near or below any groundwater surface. This classification should be verified by the contractor’s “Competent Person” at the time of construction. Excavation sloping or shoring and dewatering will be needed to safely work in and to restrict the size of the excavations.



Dewatering can cause the settlement of surrounding ground and improvements; therefore, the groundwater should be lowered no more than is necessary and for no longer than is necessary during construction. A program of monitoring the surrounding areas, the structures and improvements, and the discharge should be implemented where dewatering is utilized.

Soil Suitability for Use as Fill and Backfill

We anticipate that the majority, if not all, of the soils excavated at the site will be acceptable from a geotechnical viewpoint for reuse as compacted fill and backfill. However, special requirements for utility trench bedding and shading per the specifications of City of Goleta, the conduit manufacturer, and the utility companies should be anticipated.

Soil Stability During Grading

The site soils may be susceptible to temporary high soil moisture conditions especially during or soon after the rainy season. Attempting to compact the soils in an overly moist condition may promote unstable conditions in the form of pumping, yielding, shearing, and/or rutting. Therefore, the construction schedule should allow adequate time during grading for aerating and drying the soils to near optimum moisture content prior to compaction. Once the appropriate moisture content is achieved, the soils should then be compacted as recommended. If unstable conditions occur, the geotechnical engineer should be consulted to provide recommendations for correction of the conditions.

Soil Erosion

The surface soils are highly erodible. Stabilization of surface soils, particularly those disturbed during construction, by vegetation or other means *during* and *following* construction is essential to reduce erosion damage. Care should be taken to establish and maintain proper drainage around the structures.

Soil Corrosivity

Based on the testing and evaluation performed by CERCO Analytical, Inc., the two soil samples tested were classified as “moderately corrosive and corrosive” to certain construction materials



that will be in contact with these soils. The architect/engineer should refer to the CERCO Analytical, Inc. report presented in Appendix C for use in determining appropriate mitigation measures for the soil's corrosivity.

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 in any State Earthquake Fault Zones (Bryant and Hart, 2007) and there are no mapped faults crossing or trending towards the site. The closest mapped fault to the site is the inferred trace of the More Ranch fault located approximately 690 feet south of the site; therefore, the potential for surface fault rupture to occur at the site is considered low.

Liquefaction and Seismically Induced Settlement of Dry Sand

Liquefaction is the loss of soil strength caused by a significant seismic event. It occurs primarily in loose, fine to medium-grained sands, and in very soft to medium stiff silts that are saturated by groundwater. During a major earthquake, the saturated sands and silts tend to compress and the water filled void spaces between the soil particles decrease in volume. This causes the pore water pressure to build up within the soils. Then if the water does drain away rapidly, the soils may lose their strength and transition into a liquefied state.

Seismically induced settlement of dry sand is also caused by a significant seismic event and may occur in lower density sand and silt soils that are not saturated by groundwater. During a major earthquake, the air filled void spaces between the unsaturated soil particles tend to compress which translates to a decrease in volume or settlement.

In order to estimate the potential for liquefaction and seismically induced settlement of dry sand and their relative effects on the site, we reviewed the boring data and utilized methods suggested by the Guidelines for Evaluating and Mitigating Seismic Hazards in California, Special Publication 117a (CDMG, 2008). Considering the soil types and densities, and the shallow groundwater conditions encountered at the site, there is a potential for both liquefaction and seismically induced settlement dry sand to occur.



To further understand the effects of liquefaction and seismically induced settlement of dry sand, we performed analyses of the CPT data using the PGA_M of 0.913 g (from the "Ground Motion Analysis" section of this report), and mean earthquake magnitude from all sources of 6.72 (USGS, 2022), and an assumed groundwater depth of approximately 5 feet below the existing ground surface. Based on the analyses, total and differential dynamic settlement due to liquefaction may approach approximately 4 inches and 2 inches, respectively. As part of the liquefaction evaluation, seismically induced settlement of dry sand was also analyzed. Based on that part of the analyses, total and differential seismically induced settlement of dry sand may approach 2 inches and 1-inch, respectively. In summary, total and differential settlement attributed to dynamic settlement in conjunction with seismically induced settlement of dry sand could reach 6 inches and 3 inches, respectively.

Based on further review of the liquefaction analyses, it is our opinion there is a potential for lateral spreading; loss of soil bearing strength is not anticipated for the following reasons. The assessment for loss of soil bearing strength was developed by assuming the existing grades at the site will be raised by approximately 4 to 6 feet and that the existing soils and the planned import soils will be prepared and graded as recommended later in this report. This grading should create at least 10 feet of non-liquefiable soils overlying the potentially liquefiable soils that appear to liquefy in relatively thin discontinuous layers.

Since the potential for liquefaction exists at the site, lateral spreading was also addressed due to nearby San Jose Creek which is approximately 150 feet southeast of the building. San Jose Creek flows parallel to and is on the southeast side of the entrance driveway. Lateral spreading movement may occur when a soil mass slides laterally on liquefied soils moving downslope or towards a free face. The topographic conditions at the site are relatively flat; however, there is a free face along San Jose Creek. Accordingly, the magnitude of lateral spreading was estimated utilizing the methods presented in the Prediction of Liquefaction Induced Lateral Spreading with Free Face Conditions (ASCE, 2002). Based on our analyses and considering the discontinuous



nature of the potentially liquefiable soils, the site could move laterally approximately 6 inches towards San Jose Creek.

Considering the estimated magnitudes of the liquefaction induced dynamic settlement, seismically induced settlement of dry sand and lateral spreading listed above, recommendations are presented in the both the "Grading" and "Foundations" sections of this report to accomplish the following:

- Significantly reduce and possibly eliminate both the total and differential seismically settlement of dry sand.
- Reduce the magnitude of liquefaction induced differential dynamic settlement to a more tolerable level; however, those mitigation measures will have little or no effect on reducing the magnitude of the total dynamic settlement of 4 inches.
- Reduce the potential effects of liquefaction and seismically induced settlement of dry sand to an acceptable level of risk per the California Code of Regulations Title 14, Section 3721(a). The level of risk is that the building would essentially remain intact to the extent that the occupants could get out safely, preserving life and limb; however, repairs to the building and other isolated structures and/or their respective foundation systems may be needed. Repairs to surface and subsurface improvements should also be anticipated.

Provided the mitigation measures are implemented, we estimate that the magnitude of the total and differential settlement attributed to liquefaction induced dynamic settlement in conjunction with seismically induced settlement of dry sand will be on the order of approximately 4 inches and 1-inch, respectively. The building and utility connections will need to be designed to accommodate this level of seismic settlement and the 6 inches of lateral movement towards San Jose Creek.

If it is desired to further reduce the potential repercussions of liquefaction and seismically induced settlement of dry sand on the planned structures and improvements at the site,



additional options should be considered. Ground modification techniques such as vibro-replacement stone columns is a mitigation measure which can densify the subsurface soils over a broad area, thereby substantially reducing the potential effects of liquefaction and seismically induced settlement of dry sand across the treated area to the structures and improvements within that treated area.

There are certainly other options available. Additional geotechnical analysis beyond the scope of this report and value engineering by the design team would be needed to determine the most cost-effective approach to further reduce the potential repercussions of liquefaction at the site.

Type of Shallow Foundation Systems

The building and other isolated structures can be supported by shallow foundation systems based on the assumption that the site is designed, prepared, and graded in accordance with the "Preliminary Geotechnical Recommendations" section of this report. However, considering the anticipated potential total and differential settlement due to liquefaction of approximately 4 inches and 1-inch, respectively, and the lateral movement of approximately 5 inches, the shallow foundations systems should consist of either of the following.

The building should be supported by a rigid mat foundation underlain by a series of geogrids covered with AB. The isolated structures (walls, fencing, signage, etc.) considered sacrificial with respect to effects of liquefaction can be supported by conventional continuous and spread footings interconnected by grade beams. Recommendations for both these shallow foundation systems are presented in the "Foundations" section of this report.

9.0 PRELIMINARY GEOTECHNICAL RECOMMENDATIONS

The following recommendations are applicable to the project as described in the "Introduction" section of this report. If additional stories, basements, retaining wall taller than 6 feet, stacked retaining walls, or other such features are incorporated into site development, the geotechnical engineer should be contacted for individual assessment.



Definitions

Unless otherwise noted, the following definitions are used in these recommendations. Where specific terms are not defined, common definitions used in the construction industry are intended.

- **Building Area:** The building area is defined as the area within and extending a minimum of 5 feet beyond the perimeter of the foundations. The building area also includes the foundation areas (plus 5 feet to each side) of any ancillary structure that will be rigidly attached to the main structure and is expected to perform in the same manner as the main structure. Such structures could include covered walkways, covered work areas, patio covers, arbors, etc.
- **Surface Improvement Area:** The area within and extending a minimum of 1-foot beyond the perimeter of the surface improvement.
- **Scarified:** Ripping the exposed soil surface in two orthogonal directions to a minimum depth of 12 inches.
- **Moisture Conditioning:** Adjusting the soil moisture to optimum moisture content or slightly above, prior to the application of compaction effort.
- **Compacted or Recompacted:** Soils placed in level lifts not exceeding 8 inches in loose thickness, and compacted to a minimum of 90 percent of maximum dry density. A minimum of 95 percent will be required in the upper 1-foot of subgrade below vehicle pavement and in all AB. The standard tests used to define maximum dry density and field density should be ASTM D1557-12 (2021) and ASTM D6938-17, respectively, or by other methods acceptable to the geotechnical engineer and the governing jurisdiction.
- **Nonexpansive Material:** Nonexpansive material is defined as being a coarse-grained soil (ASTM D2487-17) and having an expansion index of 10 or less (ASTM D4829-21).



Site Preparation

1. The existing ground surface in the building and surface improvement areas should be prepared for construction by removing existing structures, improvements, vegetation, tree stumps, large roots, debris, and other deleterious material. Any existing fill soils should be completely removed and replaced as compacted fill. Any existing utilities that will not remain in service should be removed or properly abandoned. The appropriate method of utility 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, and extending below the recommended overexcavation depth should be immediately called to the attention of the geotechnical engineer. No fill should be placed unless the geotechnical engineer has observed the underlying soil.

Grading

1. Following site preparation, the soil within the building and improvement areas should be removed to a level plane to a minimum depth of 2 feet bgs. During construction, locally deeper removals may be recommended based on field conditions. The resulting soil surface should then be scarified, moisture conditioned, and compacted prior to placing any fill soil.
2. Following site preparation, the soils in fill areas beyond the building and surface improvement areas should be removed to a minimum depth of 2 feet bgs. During construction, locally deeper removals may be recommended based on field conditions. The resulting soil surface should then be scarified, moisture conditioned, and compacted prior to placing any fill soil.
3. Voids created by dislodging cobbles and/or debris during scarification should be backfilled and compacted, and the dislodged materials should be removed from the area of work.



4. On-site material and approved import materials may be used as general fill and up to 4 feet below the deepest mat foundation element within the building area of the building. All imported soil should be nonexpansive. Proposed imported soils should be evaluated by the geotechnical engineer before being used, and on an intermittent basis during placement on the site.
5. All materials used as fill should be cleaned of any debris and rocks larger than 6 inches in diameter. No rocks larger than 3 inches in diameter should be used within the upper 3 feet of finish grade. 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.
6. Under the mat foundation for the building, Tensar TX 160 triaxial geogrid or an approved equivalent should be placed on a level plane 4 feet below the bottom of the deepest foundation element within the building area. The geogrid should also extend vertically up the 4-foot sides of the excavation. Two feet of moisture conditioned and compacted Class 2 AB (Caltrans, 2018) should then be placed to a level plane on top of the first geogrid within the building area. Another or second layer of Tensar TX 160 triaxial geogrid or an approved equivalent should be placed on the top of the first layer of AB within the building area; however, it does not need to extend up the sides of the excavation. Another 2 feet of moisture conditioned and compacted AB should then be placed to a level plane on the geogrid within the building area. The bottom layer of geogrid that was extended up the sides of the excavation should then be folded down on top of the second layer of compacted AB such that it creates a minimum 5-foot overlap on top of the compacted AB surface. Previously excavated soil or Class 2 AB may then be placed as moisture conditioned and compacted fill back up to pad grade. The geogrid should be placed and overlapped as recommended by the manufacturer.
7. Fill slopes should be keyed and benched into competent soil as generally shown on the Typical Bench and Keyway Detail presented in Appendix E. The geotechnical engineer



should approve all keyways and benches. The keyway should be a minimum of 10 feet wide.

8. Slopes under normal conditions should be constructed at 2 to 1 (horizontal to vertical) or flatter inclinations. Slopes subject to inundation should be constructed at 3 to 1 or flatter inclinations.
9. The on-site soils are estimated to shrink by approximately 15 to 20 percent when prepared and graded as recommended above.

Utility Trenches

1. Unless otherwise recommended, utility trenches adjacent to foundations should not be excavated within the zone of foundation influence, as shown on Typical Detail A presented in Appendix F.
2. Utilities that will pass beneath a foundation should be placed with properly compacted utility trench backfill, and the foundation should be designed to span the trench.
3. Utility infrastructure and connections to the buildings should be flexible enough to handle the combination of total and differential dynamic settlement of 4 inches and 1-inch, respectively, and the 6 inches of lateral movement towards San Jose Creek.
4. A select, noncorrosive, granular, easily compacted material should be used as bedding and shading immediately around utilities. Generally, the soil found at the site may be used for trench backfill above the select material.
5. Utility trench backfill should be moisture conditioned and compacted; however, the Engineering Design Standards (SBC, 2011) requires a minimum compaction of 95 percent of maximum dry density in trench backfill in existing or future public roadway areas. A minimum of 95 percent of maximum dry density should also be obtained where trench backfill comprises the upper 1-foot of subgrade beneath PCC pavement, and in all AB. A minimum of 85 percent of maximum dry density will generally be sufficient where trench



backfill is located in landscaped or other unimproved areas, where settlement of trench backfill would not be detrimental.

6. Jetting of trench backfill should generally not be allowed as a means of backfill densification. However, to aid in encasing utility conduits, particularly corrugated conduits and multiple closely spaced conduits in a single trench, jetting or flooding may be useful. Jetting or flooding should only be attempted with extreme caution, and any jetting or flooding operation should be subject to review by the geotechnical engineer.
7. Long-term settlement of properly compacted imported sand should be assumed to be about 0.25 to 0.5 percent of the depth of the backfill. Where trenches are backfilled with site soils, the anticipated settlement would be about twice that of imported sand. Improvements that are constructed over or near trenches should be designed to accommodate the potential for settlement.
8. The Corrosion Evaluation Report prepared by CERCO Analytical, Inc. and presented in Appendix C should be used by the architect/engineer in specifying appropriate corrosion protection measures for the utility improvements.
9. The recommendations of this section are minimums only, and may be superseded by the architect/engineer based upon the soil corrosivity, or the requirements of the pipe manufacturer, the utility companies, or the governing jurisdiction.

Foundations

Mat Foundation

1. The building should be supported by a rigid mat foundation that is further underlain by a system of geogrids covered with compacted Class 2 AB (Caltrans, 2018). The mat foundation should consist of a structural, moment, and shear resisting slab; or an integrated system of structural slabs and moment and shear resisting grade beams. Due to the specialized design required for the mat foundation system, embedment depth,



reinforcement, and moisture intrusion prevention measures are left to the discretion of the architect/engineer.

2. The mat foundation should be designed using a maximum allowable bearing capacity of 2,000 psf dead plus live loads. The mat foundation can also be designed using modulus of subgrade reaction (K_{30}) of 10 pci. Maximum total and differential settlement under static conditions are expected to be on the order of 1-inch and 1/2-inch across the foundation footprint, respectively. Foundations should also be designed to accommodate the total and differential dynamic settlement of 4 inches and 1-inch, respectively, and the 6 inches of lateral movement towards San Jose Creek.

Conventional Foundations

3. Conventional continuous and spread footings bearing on compacted soil may be used to support the isolated structures. All spread footings should be interconnected by grade beams on at least two sides. Footings and grade beams should have a minimum depth of 12 inches below lowest adjacent grade; however, footing and grade beam dimensions should also conform to the applicable requirements of Section 1809 (CBSC, 2022). All spread footings should be a minimum of 2 feet square.
4. Footing and grade beam reinforcement should be in accordance with the requirements of the architect/engineer; minimum continuous footing and grade beam reinforcement should consist of two No. 4 rebars, one near the top and one near the bottom of the footing.
5. Conventional foundations should be designed using a maximum allowable bearing capacity of 2,000 psf dead plus live loads. Using these criterion, total and differential settlement is expected to be on the order of 3/4-inch and 1/4-inch in 40 feet, respectively.

Foundations, General

6. Lateral loads may be resisted by soil friction and by passive resistance of the soil or AB acting on foundations. Lateral capacity is based on the assumption that backfill adjacent



to foundations is properly compacted. A passive equivalent fluid pressure of 400 pcf and a coefficient of friction of 0.40 for the soil or 0.45 for the AB may be used in design. No factors of safety, load factors, and/or other factors have been applied to any of the values.

7. The allowable bearing capacity and/or modulus of subgrade reaction may be increased by one-third when transient loads such as wind or seismicity are included if the structural engineer determines they are allowed per Sections 1605.3.1 and 1605.3.2 (CBSC, 2022). The foundations should be designed using the seismic information presented in the "Seismic Design Parameters" listed in the "Ground Motion Analyses" section of this report.
8. Foundation excavations should be observed by the geotechnical engineer prior to rebar and PCC placement. Footing excavations should be thoroughly moistened prior to PCC placement and no desiccation cracks should be present.
9. The Corrosion Evaluation Report prepared by CERCO Analytical, Inc. and presented in Appendix C should be used by the architect/engineer in specifying appropriate corrosion protection measures for all foundation elements.

Retaining Walls

1. Retaining walls connected to the building should be supported by the mat foundation; however, isolated retaining walls or walls not connected to the building may be supported by a conventional foundation system underlain compacted fill. Conventional foundations for retaining walls should have a minimum depth of 12 inches below lowest adjacent grade not including the keyway. It is assumed that retaining walls will not exceed 6 feet in height.
2. As we have assumed that retaining wall heights will not exceed a height of 6 feet, seismic design per Section 1803.5.12.1 (CBSC, 2022) is not required. If retaining walls will retain more than 6 feet of soil, seismic design will be required by the geotechnical engineer.



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

Active equivalent fluid pressure

(native soil, imported sand or gravel backfill) 35 pcf

At-rest equivalent fluid pressure

(native soil, imported sand or gravel backfill) 55 pcf

Passive equivalent fluid pressure (compacted fill) 400 pcf

4. No surcharges are taken into consideration in the above values. The maximum toe pressure is an *allowable* value to which a factor of safety has been applied. No factors of safety, load factors, and/or other factors have been applied to any of the remaining values.
5. The above pressures 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 degree of slope inclination.
6. The active and at-rest values presented above are for drained conditions. Consequently, retaining walls should be drained with rigid perforated pipe encased in a free draining gravel blanket. The pipe should be placed with the perforations 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. The upper foot should be backfilled with on-site soil except in areas where a slab or pavement will abut the top of the wall. In such cases, the gravel backfill should extend up to the material that supports the slab or pavement. To reduce infiltration of the soil into the gravel, a permeable synthetic fabric conforming to the Standard Specifications (Caltrans, 2018) Section 96-1.02B – Class “C,” should be placed between the two. Manufactured geocomposite wall drains conforming to the Standard Specifications (Caltrans, 2018) Section 96-1.02C are acceptable



alternatives to the use of gravel, provided that they are installed in accordance with the recommendations of the manufacturer. Where drainage can be properly controlled, weep holes on maximum 4-foot centers may be used in lieu of perforated pipe. A filter fabric as described above should be placed between the weep holes and the drain gravel.

7. Retaining walls where moisture transmission through the wall would be undesirable should be thoroughly waterproofed in accordance with the specifications of the architect/engineer.
8. 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.

Slabs-on-Grade and Exterior Flatwork

Design Considerations

1. Conventional interior light-duty PCC slabs-on-grade and exterior flatwork should have a minimum thickness of 4 full inches; the thickness of heavy-duty slabs and exterior flatwork should be specified by the architect/engineer. Slabs-on-grade should be doweled to footings and grade beams with dowels.
2. Reinforcement size, placement, and dowels should be as directed by the architect/engineer. Light-duty exterior pedestrian flatwork should be reinforced, at a minimum, with No. 3 rebar at 18 inches on-center each way. Heavy-duty slabs and flatwork should have minimum rebar sizing and spacing that meets the criteria of American Concrete Institute (ACI) 318 (ACI, 2014). A modulus of subgrade reaction (K_{30}) of 100 psi/inch may be used in the design of heavy-duty slabs-on-grade founded on



compacted native soil. The modulus of subgrade reaction (K_{30}) may be increased to 150 psi/inch if the slab is underlain with a minimum of 6 inches of compacted Class 2 aggregate base (AB) (Caltrans, 2018), and to 200 psi/inch if the slab is underlain with a minimum of 12 inches of compacted Class 2 AB.

Moisture Vapor Transmission Considerations

3. 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, the 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.
4. Where specified, vapor retarders should conform to ASTM Standard E1745-17. 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 its installation and placement of slab reinforcement and concrete.
5. Several recent studies, including those of ACI 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 PCC in direct contact with a Class "A" vapor retarder, particularly during wet weather construction. However, placing the PCC directly on the vapor retarder requires special attention to using the proper vapor retarder, a very low water-cement ratio in the PCC mix, and special finishing and curing techniques.
6. Another option for vapor protection would be the *use of vapor-inhibiting admixtures in the slab PCC mix and/or application of a sealer* to the surface of the slab. This would also



require special PCC mixes and placement procedures, depending upon the recommendations of the admixture or sealer manufacturer.

7. A third 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 compacted soil material. 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 D2487-17) of which less than 3 percent passes the No. 200 sieve.

Construction Considerations

8. If 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.
9. Regardless of the underslab vapor retarder selected, proper installation of the retarder 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. All seams must be properly lapped, and all seams and utility penetrations properly sealed in accordance with the vapor retarder manufacturer's recommendations and ASTM E1643-18. At the 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.



10. In conventional construction, it is common to use 4 to 6 inches of sand beneath exterior light-duty PCC slabs-on-grade and exterior flatwork. Another measure that can be taken to reduce the risk of movement of flatwork is to provide thickened edges or grade beams around the perimeters of the flatwork. The thickened edges or grade beams could be up to 6 to 12 inches deep with the deeper edges or grade beams providing better protection. At a minimum, the thickened edge or grade beam should be reinforced by two No. 4 rebar, one near the top and one near the bottom.
11. It is recognized that the measures discussed above for protecting exterior flatwork are expensive; possibly more expensive than simply replacing the flatwork that has heaved and/or cracked. Accordingly, the above measures for protecting exterior flatwork should be implemented unless the client's preference is to replace exterior flatwork that is damaged over time. Therefore, the degree to which the exterior flatwork is protected from expansive soil damage is left to the discretion of the client.
12. Flatwork should be constructed with frequent joints to allow articulation as flatwork moves in response to seasonal moisture and/or temperature variations causing minor expansion and contraction of the soil, or variable bearing conditions. The soil in the subgrade should be moistened to at least optimum moisture content and no desiccation cracks should be present prior to casting the flatwork.
13. Where maintaining the elevation of the flatwork is desired, the flatwork should be doweled to the perimeter foundation as specified by the architect/engineer. 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.
14. To reduce shrinkage cracks in PCC, the PCC aggregates should be of appropriate size and proportion, the water/cement ratio should be low, the PCC should be properly placed and finished, contraction joints should be installed, and the PCC should be properly cured. PCC



materials, placement, and curing specifications should be at the direction of the architect/engineer. The Guide for Concrete Floor and Slab Construction (ACI, 2015) is suggested as a resource for the architect/engineer in preparing such specifications.

Pavement Sections

The following preliminary pavement sections are based on an assumed R-value of 25 and should be used for cost estimating purposes only. Accordingly, the soil exposed at the access road/driveway subgrade should be tested during construction for R-value to verify that these preliminary pavement sections are appropriate; otherwise, revised pavement sections should be prepared. Pavement design sections are provided for assumed Traffic Indices (TI) of 4.5, 5.0, 5.5, 6.0, 6.5, and 7.0. Determination of the appropriate TI for specific areas is left to others. The pavement sections were calculated in accordance with the Highway Design Manual (Caltrans, 2020). The calculated AB and HMA thickness are for compacted material. Normal Caltrans construction tolerances should apply.

Preliminary Pavement Sections

R-value	TI	HMA (inches)	Class 2 AB (inches)
15	4.5	2.50	6.0
	5.0	2.75	7.0
	5.5	3.00	8.0
	6.0	3.25	9.0
	6.5	3.75	10.0
	7.0	4.00	10.5

1. The upper 12 inches of subgrade and all AB should be compacted to a minimum of 95 percent of maximum dry density.
2. Subgrade and AB should be firm and unyielding when proof-rolled by heavy rubber-tired equipment prior to paving.
3. Where HMA will lie within 5 feet of landscape or drainage improvements, the HMA should be separated from these improvements by deepened curbs or other means that will



reduce the potential for moisture fluctuations in the soils beneath the HMA and improve the stability of the curbs.

4. Finished HMA surfaces should slope toward drainage facilities such that rapid runoff will occur and no ponding is allowed on or adjacent to the HMA.

Drainage and Maintenance

1. Per Section 1804.4 (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 foundation, slopes, and other improvements during construction. Where this is not practicable due to property lines, other improvements, etc., swales with improved surfaces, area drains, or other drainage facilities, should be used to collect and discharge runoff.
2. To reduce the potential for planter drainage from 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 foundations that would not otherwise freely drain.
3. The building roof should be fitted with roof gutters. Runoff from flatwork, roof gutters, downspouts, planter drains, area drains, etc. should discharge in a nonerosive manner away from foundations and other improvements in accordance with the requirements of the governing agencies.
4. The on-site soils are highly erodible; stabilization of soils disturbed during construction by vegetation or other means *during* and *following* construction, is essential to reduce 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. To reduce migration of surface drainage into the subgrade, maintenance of pavement areas is critical. Any cracks that develop in the pavement should be promptly sealed.
6. The owner or site maintenance personnel should periodically observe the areas within and around the site for indications of rodent activity and soil instability. The owner or site maintenance personnel should also implement an aggressive program for controlling the rodent activity in the general area.

Construction Observation and Testing

1. It must be recognized that the recommendations contained in this report are based on a review of Reference 2 and relies on the continuity of the subsurface conditions encountered. It is assumed that the geotechnical engineer will be retained to provide consultation during the design phase, to review final plans once they are available, to interpret this report during construction, and to provide construction monitoring in the form of testing and observation.
2. At a minimum, the geotechnical engineer should be retained to provide:
 - Review of final grading, utility, and foundation plans
 - Professional observation during grading, foundation excavations, and trench backfill
 - Oversight of compaction testing during grading
 - Oversight of special inspection during grading
3. Special inspection of grading and backfill should be provided as per Section 1705.6 and Table 1705.6 (CBSC, 2022). The special inspector should be under the direction of the geotechnical engineer. It is our opinion that none of the grading construction is of a nature that should warrant continuous special inspection; periodic special inspection should suffice. Subject to approval by the Building Official, the exception to continuous special inspection is described in Section 1704.2 (CBSC, 2022) and should be specified by



the architect/engineer and periodic special inspection of the following items should be provided by the special inspector.

- Stripping and clearing of vegetation
 - Overexcavation to the recommended depths
 - Scarification, moisture conditioning, and compaction of the soil
 - Geogrid placement and AB placement and compaction
 - Fill quality, placement, and compaction
 - Utility trench backfill
 - Retaining wall drains and backfill
 - Foundation excavations
 - Subgrade and AB compaction and proofrolling
4. A program of quality control should be developed prior to beginning grading. The contractor or project manager should determine any additional inspection items required by the architect/engineer or the governing jurisdiction.
 5. Locations and frequency of compaction tests should be as 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.
 6. A preconstruction conference among the owner, the geotechnical engineer, the City of Goleta, the special inspector, the architect/engineer, and contractors is recommended to discuss planned construction procedures and quality control requirements.
 7. The geotechnical engineer should be notified at least 48 hours prior to beginning construction operations. 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.

10.0 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 of this report. 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 development type or location 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 shall comply with the CBC (CBSC, 2022) and the requirements of the governing jurisdiction.

The preliminary recommendations of this report are based upon the geotechnical conditions encountered at the site and may be augmented by additional requirements of the architect/engineer, or by additional recommendations provided by this firm based on conditions exposed at the time of construction.

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.



TECHNICAL REFERENCES

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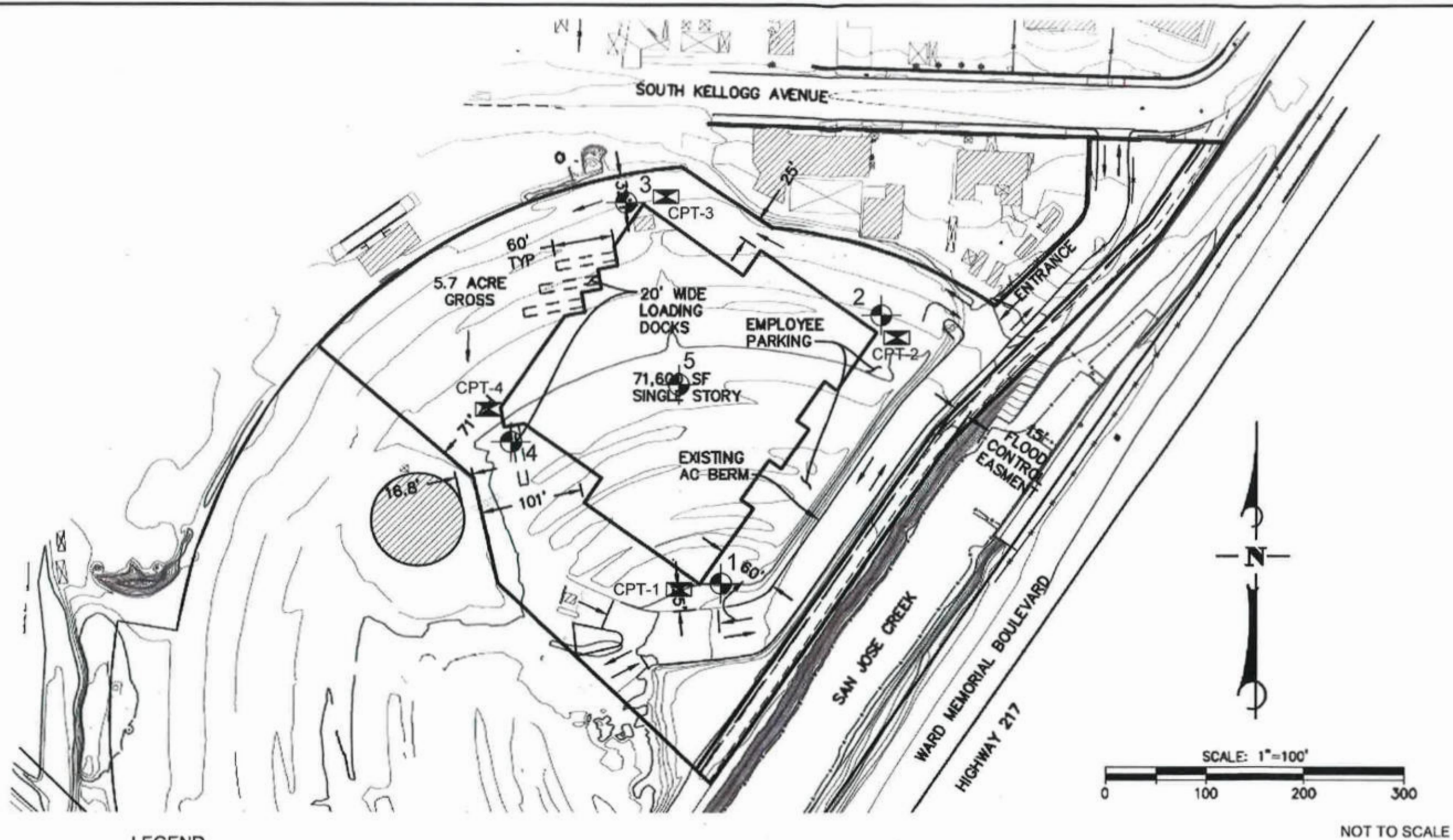
APPENDIX A

Exploration Location Map

Boring Log Legend

Boring Logs

Graphical CPT Data



Earth Systems

2049 Preisker Lane, Suite E, Santa Maria, CA. 93454
 www.earthsystems.com
 (805) 928-2991

EXPLORATION LOCATION MAP
 SYWEST DEVELOPMENT COMMERCIAL BUILDING
 907 South Kellogg Avenue
 Goleta, California

Date
 July 21, 2017
 Project No.
 SL-18090-SA

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			FINE GRAINED SOILS		
BLOWS/FOOT		DESCRIPTIVE TERM	BLOWS/FOOT		DESCRIPTIVE TERM
SPT	CA SAMPLER		SPT	CA SAMPLER	
0-10	0-16	LOOSE	0-2	0-3	VERY SOFT
11-30	17-50	MEDIUM DENSE	3-4	4-7	SOFT
31-50	51-83	DENSE	5-8	8-13	MEDIUM STIFF
OVER 50	OVER 83	VERY DENSE	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
FRESH	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"
INTENSELY 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



Earth Systems Pacific

LOGGED BY: R. Wagner
DRILL RIG: Mobile B-53
AUGER TYPE: 6" Hollow Stem

Boring No. 2

PAGE 1 OF 1
JOB NO.: SL-18090-SA
DATE: 06/21/2017

DEPTH (feet)	USCS CLASS	SYMBOL	SYWEST DEVELOPMENT COMMERCIAL BUILDING 907 South Kellogg Avenue Goleta, California SOIL DESCRIPTION	SAMPLE DATA				
				INTERVAL (feet)	SAMPLE TYPE	DRY DENSITY (pcf)	MOISTURE (%)	BLOWS PER 6 IN.
0 - 1 - 2 - 3 - 4	SM		SILTY SAND: brown, loose, moist, trace clay (Fill)	1-4				
4 - 5 - 6 - 7 - 8 - 9 - 10 - 11 - 12 - 13 - 14	SM		SILTY SAND: brown, loose, wet (Alluvium) thin lense of clay	5.0-6.5		95.3	26.4	1 1 2
14 - 15 - 16 - 17 - 18 - 19 - 20 - 21 - 22 - 23 - 24 - 25 - 26	CL		SANDY LEAN CLAY: dark blue/gray, very soft, wet, trace fine shells, trace silt (Estuary Deposits)	15.0-16.5		85.9	33.6	2 1 0
	SP		POORLY GRADED SAND: dark blue/gray, loose, wet, interbedded with thin discontinuous lenses of silt and clay, trace to some fine shells	20.0-21.5				4 4 6
				25.0-26.5		No Return		2 3 4
			End of Boring @ 26.5' Subsurface Water Encountered @ 4.0'					

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

LOGGED BY: R. Wagner
DRILL RIG: Mobile B-53
AUGER TYPE: 6" Hollow Stem

Boring No. 3

PAGE 1 OF 1

JOB NO.: SL-18090-SA

DATE: 06/21/2017

DEPTH (feet)	USCS CLASS	SYMBOL	SYWEST DEVELOPMENT COMMERCIAL BUILDING 907 South Kellogg Avenue Goleta, California SOIL DESCRIPTION	SAMPLE DATA				
				INTERVAL (feet)	SAMPLE TYPE	DRY DENSITY (pcf)	MOISTURE (%)	BLOWS PER 6 IN.
0 1 2 3 4	SM		SILTY SAND: brown, loose, moist, trace clay (Fill)					
5 6 7	CL		SANDY LEAN CLAY: gray brown, medium stiff, moist (Alluvium)	5.0-6.5	■	104.0	19.6	2 4 7
8 9 10 11 12 13	SM		SILTY SAND: brown, loose, wet	5-7	○			
14 15 16 17 18 19	SM		SILTY SAND: dark blue/gray, loose, wet, interbedded with thin discontinuous lenses of silt and clay, trace to some fine shells (Estuary Deposits)	10.0-11.5	■	100.4	21.5	3 4 7
20 21 22 23 24	CL		LEAN CLAY: dark blue/gray, very soft, wet, trace fine shells, some silt	15.0-16.5	■	108.4	17.9	3 5 8
25 26 27 28	SP		POORLY GRADED SAND: dark blue/gray, loose, wet, interbedded with thin discontinuous lenses of silt and clay, trace to some fine shells	20.0-21.5	●			0 0 0
			End of Boring @ 26.5' Subsurface Water Encountered @ 7.5'	25.0-26.5	●	No Return		0 0 1

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

LOGGED BY: R. Wagner
DRILL RIG: Mobile B-53
AUGER TYPE: 6" Hollow Stem

Boring No. 4

PAGE 1 OF 1

JOB NO.: SL-18090-SA

DATE: 06/22/2017

DEPTH (feet)	USCS CLASS	SYMBOL	SYWEST DEVELOPMENT COMMERCIAL BUILDING 907 South Kellogg Avenue Goleta, California	SAMPLE DATA				
			SOIL DESCRIPTION	INTERVAL (feet)	SAMPLE TYPE	DRY DENSITY (pcf)	MOISTURE (%)	BLOWS PER 6 IN.
0 1 2 3 4	SP		POORLY GRADED SAND: brown, loose, moist, trace clay (Fill)	5.0-6.5		100.7	25.2	2 4 5
5 6 7 8	SM		SILTY SAND: brown, loose, very moist (Alluvium) wet					
9 10 11 12 13 14 15 16 17 18 19 20	SP		POORLY GRADED SAND: dark blue/gray, loose, wet, interbedded with thin discontinuous lenses of silt and clay, trace to some fine shells (Estuary Deposits) thin lense of clay ~ 12" thick	10.0-11.5		93.5	25.2	0 1 4
21 22 23 24 25 26	CL		SANDY LEAN CLAY: dark blue/gray, soft, wet, trace fine shells, trace silt	15.0-16.5		No Return		3 2 1
20.0-21.5								3 2 2
25.0-26.5	SP		POORLY GRADED SAND: dark blue/gray, loose, wet, interbedded with thin discontinuous lenses of silt and clay, trace to some fine shells	25.0-26.5				3 2 3
			End of Boring @ 26.5' Subsurface Water Encountered @ 5.5'					

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

LOGGED BY: R. Wagner
 DRILL RIG: Mobile B-53
 AUGER TYPE: 6" Hollow Stem

Boring No. 5

PAGE 1 OF 2
 JOB NO.: SL-18090-SA
 DATE: 06/22/2017

DEPTH (feet)	USCS CLASS	SYMBOL	SYWEST DEVELOPMENT COMMERCIAL BUILDING 907 South Kellogg Avenue Goleta, California	SAMPLE DATA				
			SOIL DESCRIPTION	INTERVAL (feet)	SAMPLE TYPE	DRY DENSITY (pcf)	MOISTURE (%)	BLOWS PER 6 IN.
0 - 1 - 2 - 3 - 4 - 5 - 6 - 7 - 8 - 9 - 10 - 11 - 12 - 13 - 14 - 15 - 16 - 17 - 18 - 19 - 20 - 21 - 22 - 23 - 24 - 25 - 26 -	SP		POORLY GRADED SAND: brown, loose, moist, trace clay (Fill)					
			some clay					
	SM		SILTY SAND: brown, loose, very moist (Alluvium)	5.0-6.5		No Return		2 4 5
			wet					
	CL		SANDY LEAN CLAY: dark blue/gray, soft, wet, trace fine shells, trace silt (Estuary Deposits)	10.0-11.5		No Return		0 2 3
				15.0-16.5		99.2	22.7	2 2 2
	SP		POORLY GRADED SAND: dark blue/gray, loose, wet, interbedded with thin discontinuous lenses of silt and clay, trace to some fine shells	20.0-21.5		No Return		2 3 4
				25.0-26.5				1 2 4

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

LOGGED BY: R. Wagner
DRILL RIG: Mobile B-53
AUGER TYPE: 6" Hollow Stem

Boring No. 5
PAGE 2 OF 2
JOB NO.: SL-18090-SA
DATE: 06/22/2017

DEPTH (feet)	USCS CLASS	SYMBOL	SYWEST DEVELOPMENT COMMERCIAL BUILDING 907 South Kellogg Avenue Goleta, California SOIL DESCRIPTION	SAMPLE DATA				
				INTERVAL (feet)	SAMPLE TYPE	DRY DENSITY (pcf)	MOISTURE (%)	BLOWS PER 6 IN.
27	SP		POORLY GRADED SAND: as above					
28								
29								
30				30.0-31.5	●	No Return		1 3 3
31								
32								
33								
34								
35			trace silt	35.0-36.5	●			1 3 4
36								
37	CL							
38								
39								
40			SANDY LEAN CLAY: dark blue/gray, medium stiff, wet	40.0-41.5	●			2 2 3
41								
42								
43								
44								
45				45.0-46.5	●	No Return		2 1 4
46			POORLY GRADED SAND: dark blue/gray, loose, wet, interbedded with thin discontinuous lenses of silt and clay, trace to some fine shells					
47	SP							
48								
49								
50				50.0-51.5	●			3 3 2
51								
52			End of Boring @ 51.5'					
53			Subsurface Water Encountered @ 5.5'					

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.



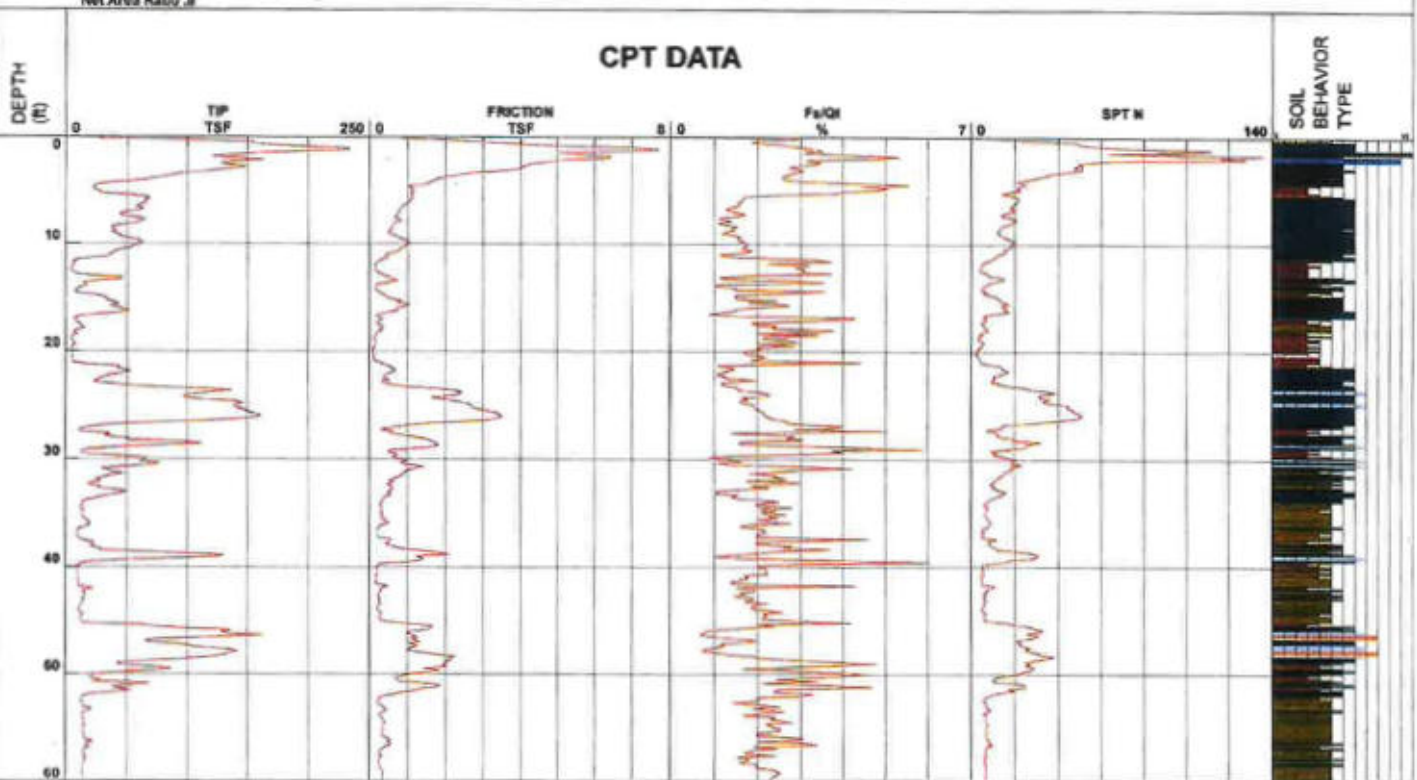
Earth Systems Pacific

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Hole Number CPT-03 Date and Time 6/28/2017 9:22:52 AM
EST GW Depth During Test 5.00 ft

Filename SDF112.cpt
GPS
Maximum Depth 60.53 ft

Net Area Ratio, β

CPT DATA



- | | | | |
|----------------------------|-------------------------------|------------------------------|----------------------------------|
| 1 - sensitive fine grained | 4 - silty clay to clay | 7 - silty sand to sandy silt | 10 - gravelly sand to sand |
| 2 - organic material | 5 - clayey silt to silty clay | 8 - sand to silty sand | 11 - very stiff fine grained (*) |
| 3 - clay | 6 - sandy silt to clayey silt | 9 - sand | 12 - sand to clayey sand (*) |

Cone Size 10cm squared

*) Soil behavior type and SPT based on data from USC-1983



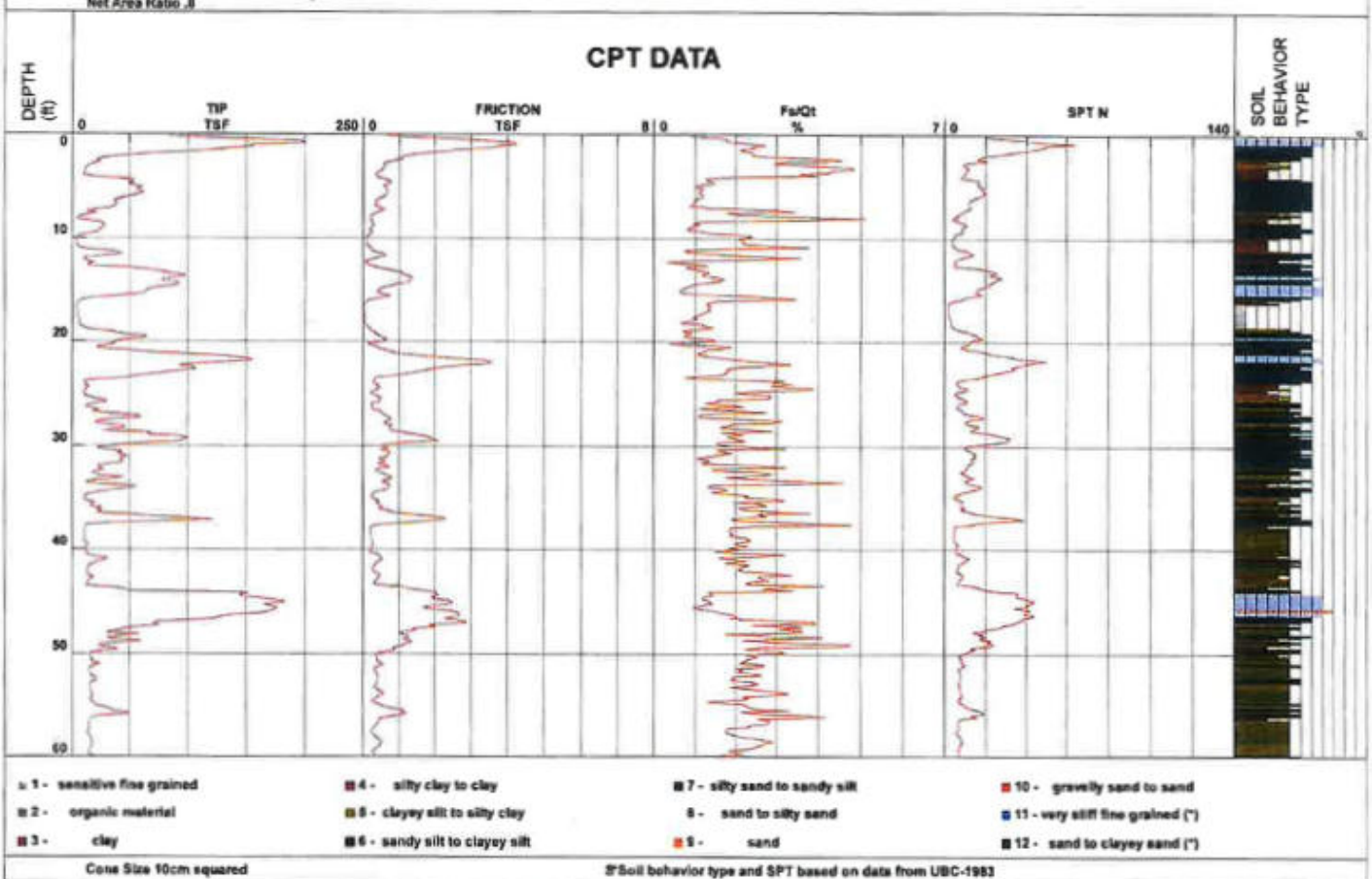
Earth Systems Pacific

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EST GW Depth During Test 5.00 ft

Filename SDF(113).cpt
GPS
Maximum Depth 60.83 ft

Net Area Ratio .8

CPT DATA





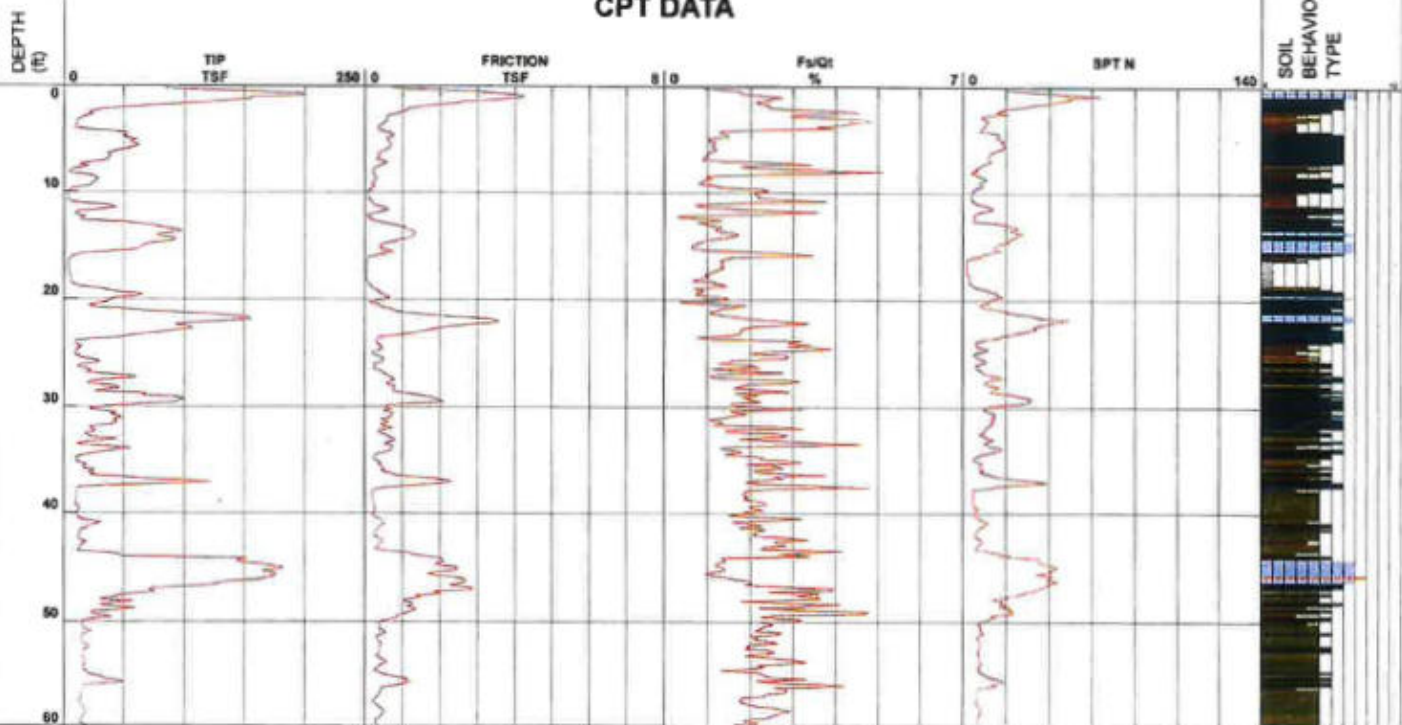
Earth Systems Pacific

Project SyWest Development Commercial Building Operator RC AD
Job Number SL-18090-SA Cone Number D001379
Hole Number CPT-04 Date and Time 6/26/2017 10:05:58 AM
EST GW Depth During Test 5.00 ft

Filename SDF(113).cpt
GPS
Maximum Depth 60.63 ft

Net Area Ratio .8

CPT DATA



- | | | | |
|----------------------------|-------------------------------|------------------------------|----------------------------------|
| 1 - sensitive fine grained | 4 - silty clay to clay | 7 - silty sand to sandy silt | 10 - gravelly sand to sand |
| 2 - organic material | 5 - clayey silt to silty clay | 8 - sand to silty sand | 11 - very stiff fine grained (*) |
| 3 - clay | 6 - sandy silt to clayey silt | 9 - sand | 12 - sand to clayey sand (*) |

Cone Size 10cm squared

SPT Soil behavior type and SPT based on data from UBC-1983

APPENDIX B

Laboratory Test Results



SyWest Development Commercial Building

SL-18090-SA

BULK DENSITY TEST RESULTS

ASTM D 2937-17 (modified for ring liners)

July 29, 2017

BORING NO.	DEPTH feet	MOISTURE CONTENT, %	WET DENSITY, pcf	DRY DENSITY, pcf
1	11.0 - 11.5	23.7	126.1	101.9
1	16.0 - 16.5	22.5	123.7	100.9
2	6.0 - 6.5	26.4	120.5	95.3
2	16.0 - 16.5	33.6	114.7	85.9
3	6.0 - 6.5	19.6	124.4	104.0
3	11.0 - 11.5	21.5	121.9	100.4
3	16.0 - 16.5	17.9	127.8	108.4
4	6.0 - 6.5	25.2	126.1	100.7
4	11.0 - 11.5	25.2	117.1	93.5
5	16.0 - 16.5	22.7	121.7	99.2



SyWest Development Commercial Building

SL-18090-SA

MOISTURE-DENSITY COMPACTION TEST

ASTM D 1557-12 (Modified)

PROCEDURE USED: A

July 29, 2017

PREPARATION METHOD: Moist

Boring #2 @ 1.0 - 4.0'

RAMMER TYPE: Mechanical

Silty Sand (SM)

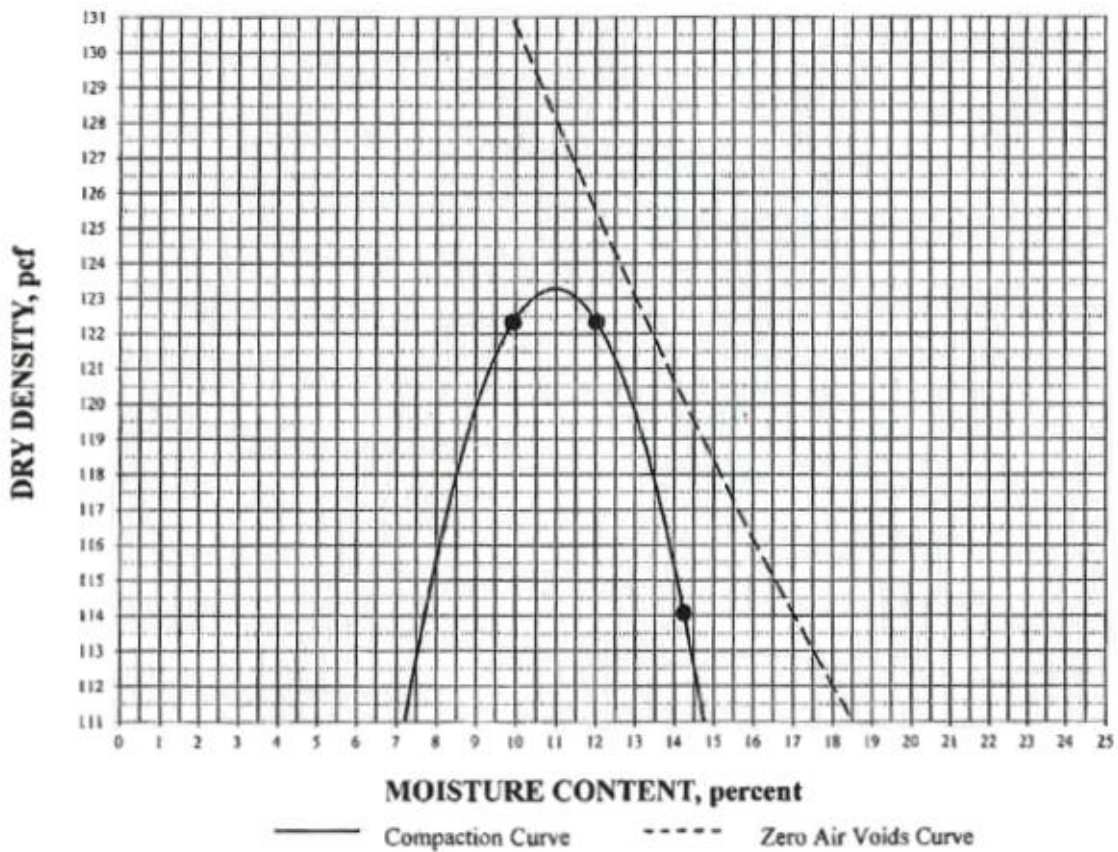
SPECIFIC GRAVITY: 2.65 (assumed)

SIEVE DATA:

Sieve Size	% Retained (Cumulative)
3/4"	0
3/8"	0
#4	1

MAXIMUM DRY DENSITY: 123.3 pcf

OPTIMUM MOISTURE: 11.0%





DIRECT SHEAR

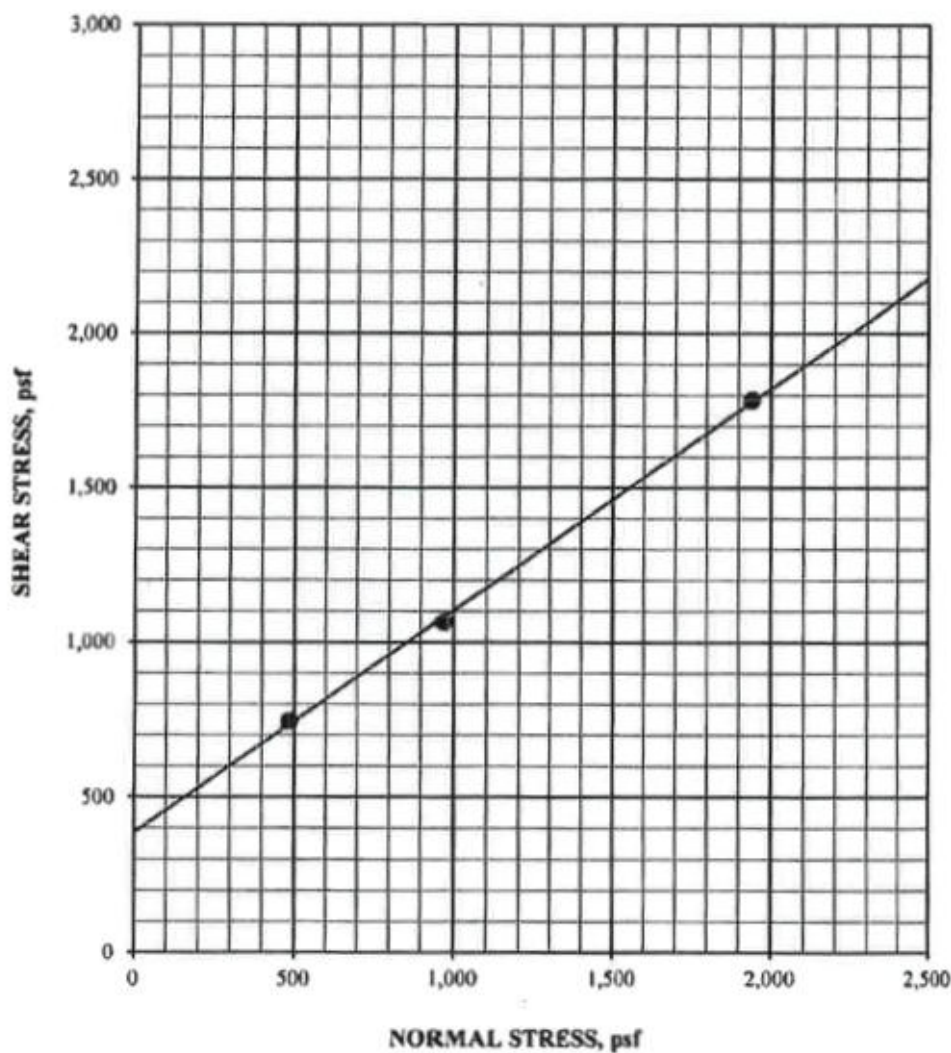
ASTM D 3080/D3080M-11 (modified for consolidated, undrained conditions)

July 29, 2017

Boring #2 @ 1.0 - 4.0'
Silty Sand (SM)
Compacted to 90% RC, saturated

INITIAL DRY DENSITY: 111.0 pcf
INITIAL MOISTURE CONTENT: 11.0 %
PEAK SHEAR ANGLE (ϕ): 36°
COHESION (C): 386 psf

SHEAR vs. NORMAL STRESS



**DIRECT SHEAR** continued

ASTM D 3080/D3080M-11 (modified for consolidated, undrained conditions)

Boring #2 @ 1.0 - 4.0'

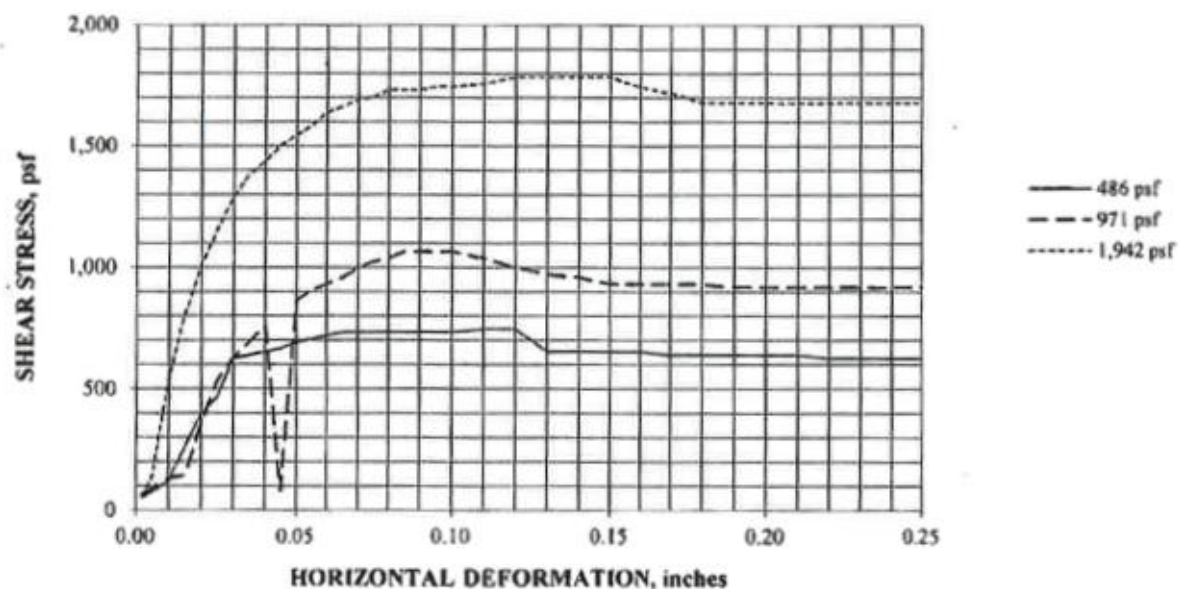
July 29, 2017

Silty Sand (SM)

Compacted to 90% RC, saturated

SPECIFIC GRAVITY: 2.65 (assumed)

SAMPLE NO.:	1	2	3	AVERAGE
INITIAL				
WATER CONTENT, %	11.0	11.0	11.0	11.0
DRY DENSITY, pcf	111.0	111.0	111.0	111.0
SATURATION, %	59.4	59.4	59.4	59.4
VOID RATIO	0.490	0.490	0.490	0.490
DIAMETER, inches	2.410	2.410	2.410	
HEIGHT, inches	1.00	1.00	1.00	
AT TEST				
WATER CONTENT, %	18.2	17.6	16.9	
DRY DENSITY, pcf	111.5	113.1	118.9	
SATURATION, %	100.0	100.0	100.0	
VOID RATIO	0.483	0.462	0.391	
HEIGHT, inches	1.00	0.98	0.93	





SyWest Development Commercial Building

SL-18090-SA

PARTICLE SIZE ANALYSIS

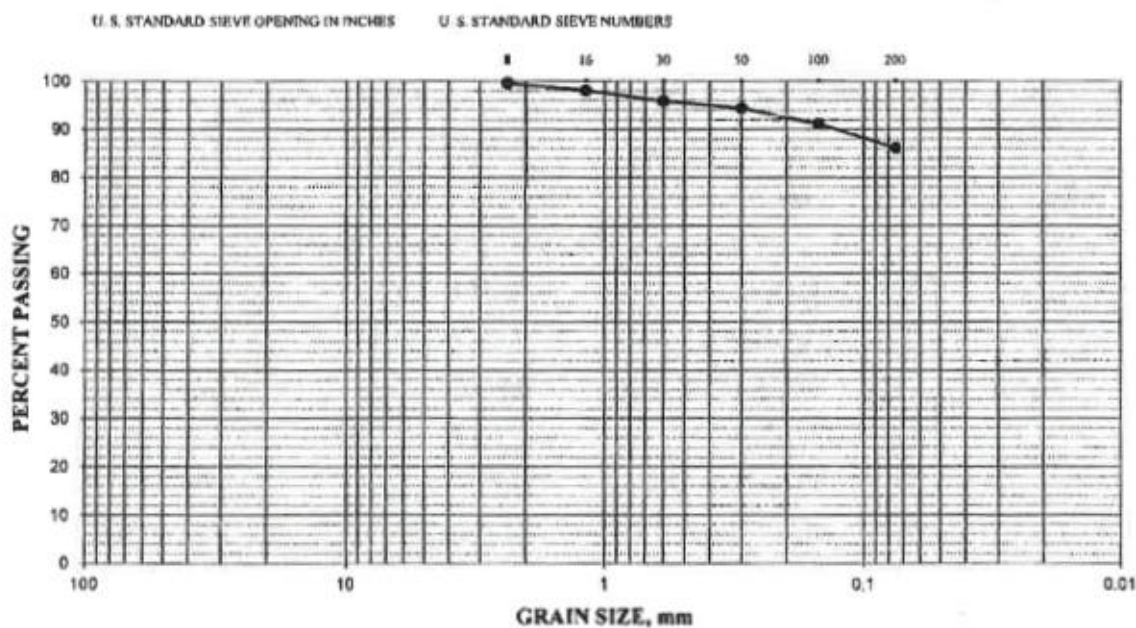
ASTM D 422-63/07; D 1140-17

Boring #3 @ 20.0 - 21.5'

July 29, 2017

Lean Clay (CL)

Sieve size	% Retained	% Passing
#8 (2.36-mm)	0	100
#16 (1.18-mm)	2	98
#30 (600- μ m)	4	96
#50 (300- μ m)	6	94
#100 (150- μ m)	9	91
#200 (75- μ m)	14	86





SyWest Development Commercial Building

SL-18090-SA

PARTICLE SIZE ANALYSIS

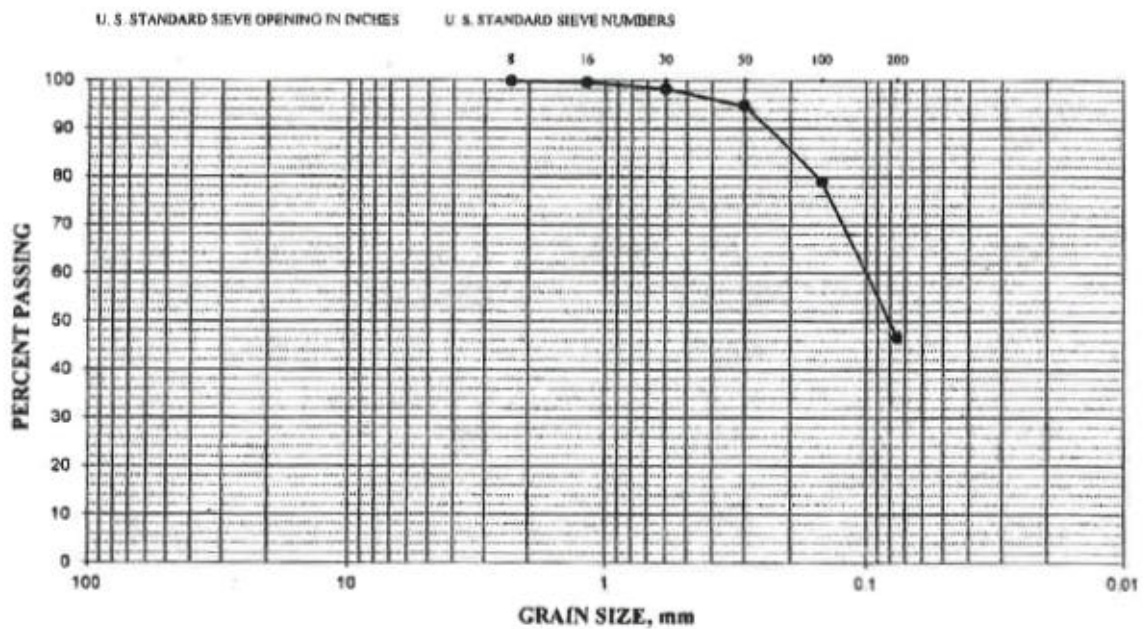
ASTM D 422-63/07; D 1140-17

Boring #5 @ 25.0 - 26.5'

July 29, 2017

Silty Sand (SM)

Sieve size	% Retained	% Passing
#8 (2.36-mm)	0	100
#16 (1.18-mm)	0	100
#30 (600- μ m)	2	98
#50 (300- μ m)	5	95
#100 (150- μ m)	21	79
#200 (75- μ m)	53	47





SyWest Development Commercial Building

SL-18090-SA

PARTICLE SIZE ANALYSIS

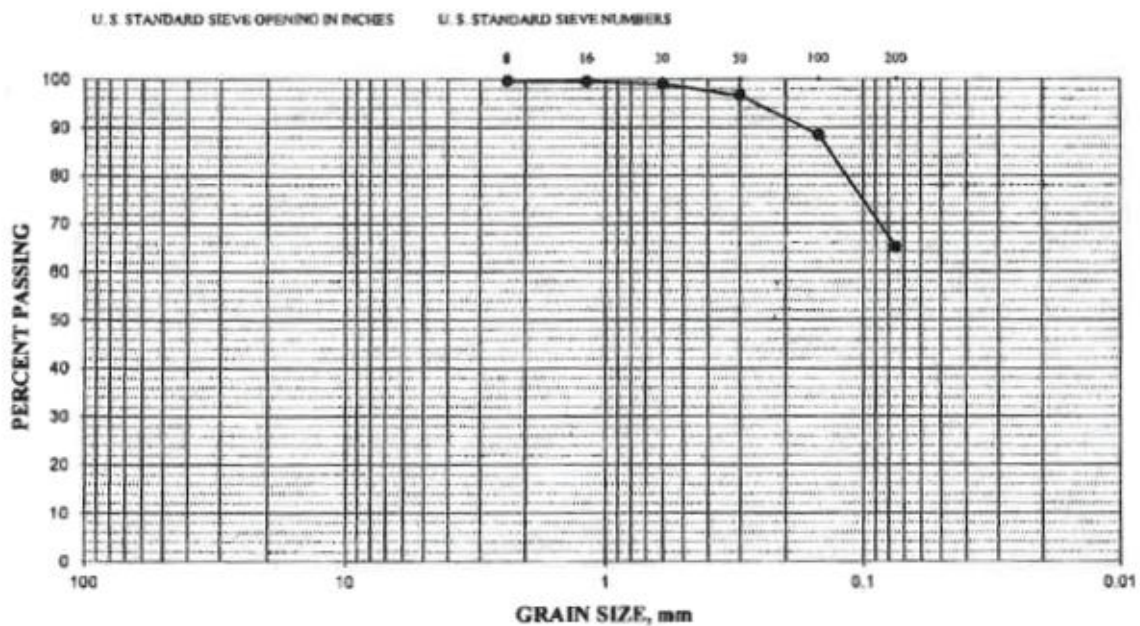
ASTM D 422-63/07; D 1140-17

Boring #5 @ 40.0 - 41.5'

July 29, 2017

Sandy Lean Clay (CL)

Sieve size	% Retained	% Passing
#8 (2.36-mm)	0	100
#16 (1.18-mm)	0	100
#30 (600- μ m)	1	99
#50 (300- μ m)	3	97
#100 (150- μ m)	12	88
#200 (75- μ m)	35	65





SyWest Development Commercial Building

SL-18090-SA

CONSOLIDATION TEST

ASTM D 2435/D2435M-11

July 29, 2017

Boring #2 @ 6.0 - 6.5'

Silty Sand (SM)

Ring Sample

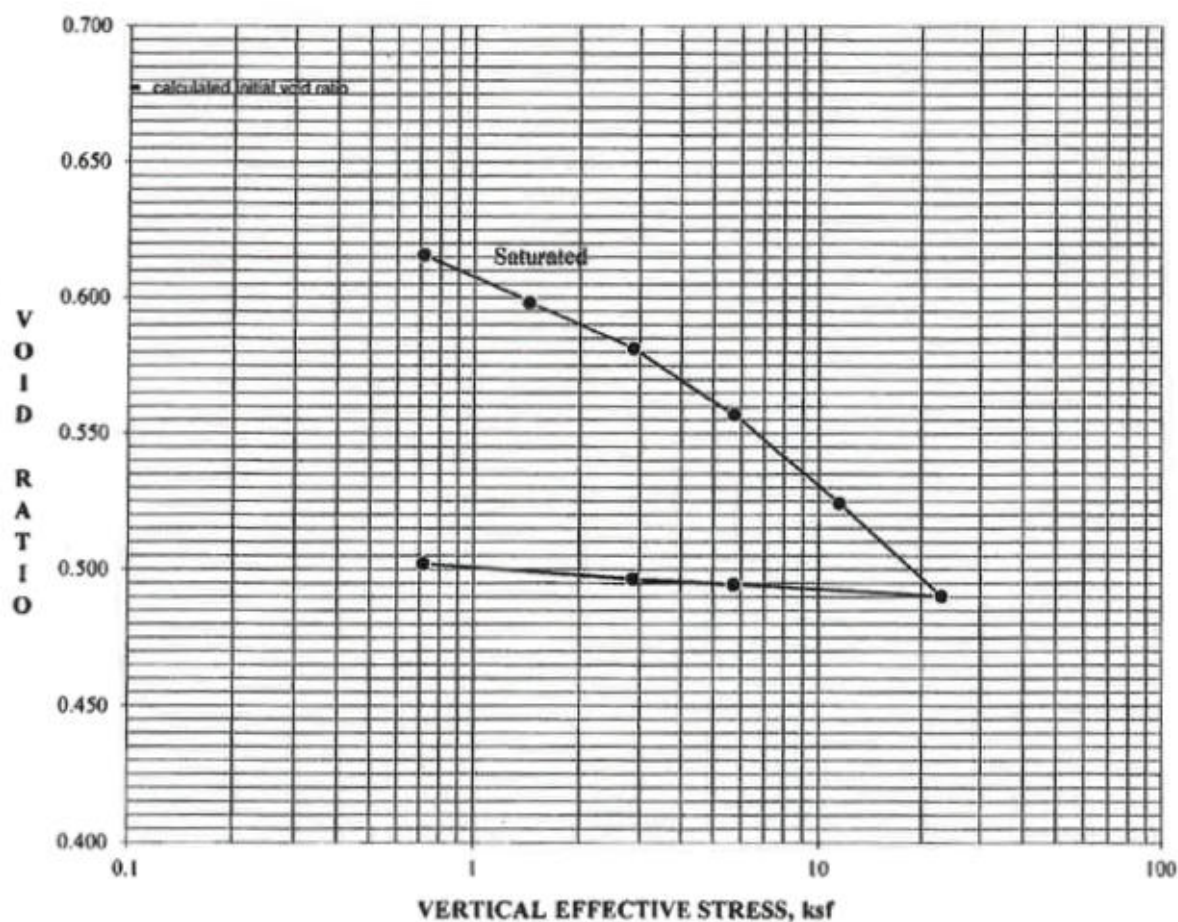
DRY DENSITY: 98.6 pcf

MOISTURE CONTENT: 26.4%

SPECIFIC GRAVITY: 2.65 (assumed)

INITIAL VOID RATIO: 0.677

VOID RATIO vs. NORMAL PRESSURE DIAGRAM





SyWest Development Commercial Building

SL-18090-SA

CONSOLIDATION TEST

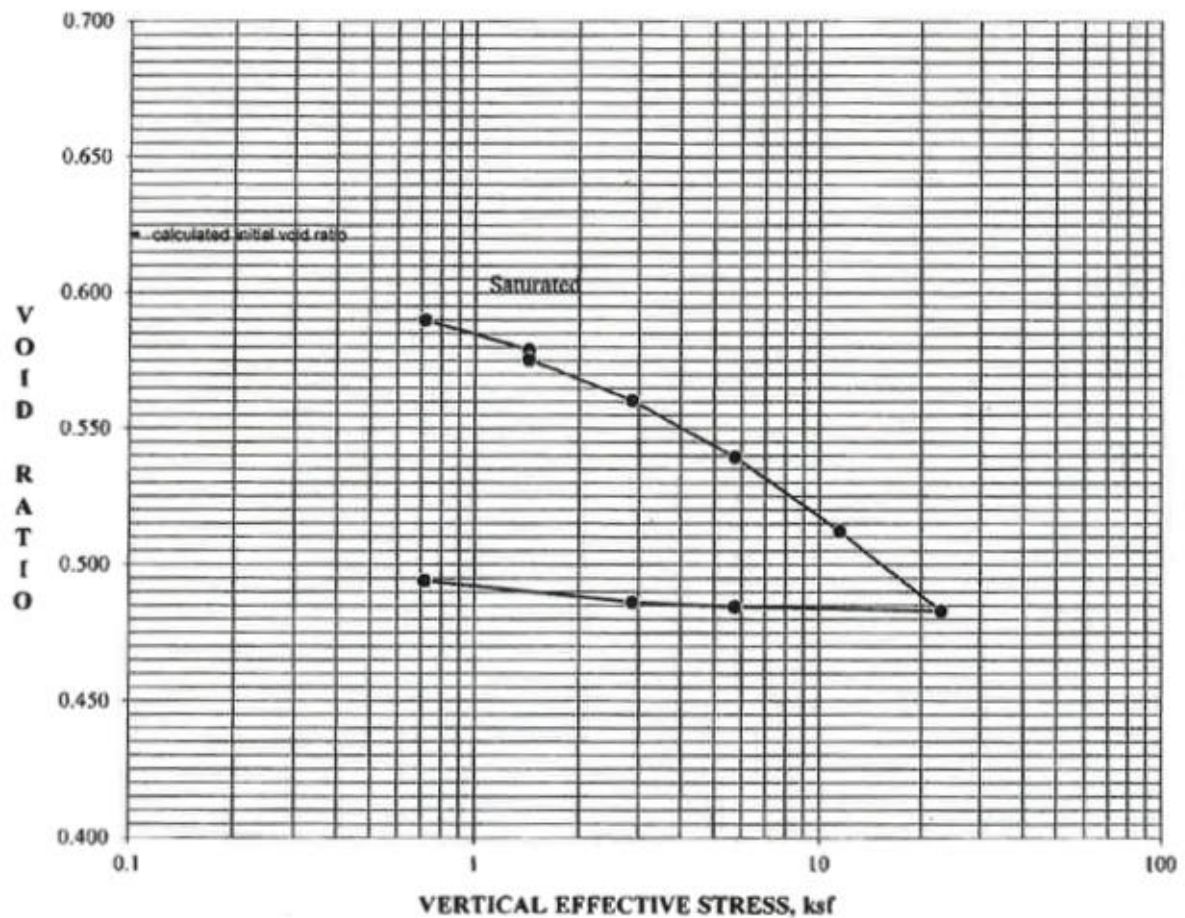
ASTM D 2435/D2435M-11

July 29, 2017

Boring #3 @ 6.0 - 6.5'
Sandy Lean Clay (CL)
Ring Sample

DRY DENSITY: 104.0 pcf
MOISTURE CONTENT: 19.6%
SPECIFIC GRAVITY: 2.70 (assumed)
INITIAL VOID RATIO: 0.621

VOID RATIO vs. NORMAL PRESSURE DIAGRAM



APPENDIX C

Corrosion Evaluation Report
Prepared by CERCO Analytical, Inc.



1100 Willow Pass Court, Suite A
Concord, CA 94520-1006

925 462 2771 Fax 925 462 2775

www.cercoanalytical.com

14 July, 2017

Job No. 1707017
Cust. No. 12651

Mr. Doug Dunham
Earth Systems Pacific
2049 Preisker Lane, Suite E
Santa Maria, CA 93454

Subject: Project No.: SL-18090-SA
Project Name: Sywest Development Commercial Bldg.
Corrosivity Analysis – CalTrans Test Methods

Dear Mr. Dunham:

Pursuant to your request, CERCO Analytical has analyzed the soil sample submitted on July 6, 2017. Based on the analytical results, a brief corrosivity evaluation is enclosed for your consideration.

Based upon the resistivity measurements, sample 001 is classified as "moderately corrosive" and sample 002 is classified as "corrosive". All buried iron, steel, cast iron, ductile iron, galvanized steel and dielectric coated steel or iron should be properly protected against corrosion depending upon the critical nature of the structure. All buried metallic pressure piping such as ductile iron firewater pipelines should be protected against corrosion.

The chloride ion concentrations range from 30 to 46 mg/kg. Because the chloride ion concentrations are less than 300 mg/kg, they are determined to be insufficient to attack steel embedded in a concrete mortar coating.

The sulfate ion concentrations range from 68 to 190 mg/kg and are determined to be insufficient to damage reinforced concrete structures and cement mortar-coated steel at these locations.

The pH of the soils range from 7.97 to 8.13 which does not present corrosion problems for buried iron, steel, mortar-coated steel and reinforced concrete structures.

This corrosivity evaluation is based on general corrosion engineering standards and is non-specific in nature. For specific long-term corrosion control design recommendations or consultation, please call *JDH Corrosion Consultants, Inc.* at (925) 927-6630.

We appreciate the opportunity of working with you on this project. If you have any questions, or if you require further information, please do not hesitate to contact us.

Very truly yours,
CERCO ANALYTICAL, INC.

A handwritten signature in black ink, appearing to read 'J. Darby Howard, Jr.', followed by the word 'for' in a smaller, cursive script.
J. Darby Howard, Jr., P.E.
President

JDH/jdl
Enclosure

Client: Earth Systems Pacific
 Client's Project No.: FL-18090-SA
 Client's Project Name: Sywest Development Commercial Building
 Date Sampled: 22-Jun-17
 Date Received: 6-Jul-17
 Matrix: Soil
 Authorization: Letter dated June 23, 2017



1100 Willow Pass Court, Suite A
 Concord, CA 94520-1006
 925 462 2771 Fax: 925 462 2775
 www.cercoanalytical.com

Date of Report: 14-Jul-2017

Job/Sample No.	Sample I.D.	Moisture (%)	pH	Min. Resistivity (ohms-cm)**	Sulfide (mg/kg)*	Chloride (mg/kg)*	Sulfate (mg/kg)*
1707017-001	Boring No. 2 @ 1'-4'	-	7.97	2,300	-	46	68
1707017-002	Boring No. 3 @ 5'-7'	-	8.13	1,400	-	30	190

Method:	CT 226 ^(a)	CT 643 ^(b)	CT 643 ^(b)	-	CT 422 ^(c)	CT 417 ^(c)
Reporting Limit:	-	-	-	50	15	15
Date Analyzed:	-	12-Jul-2017	13-Jul-2017	-	12-Jul-2017	12-Jul-2017

Cheryl McMillen
 Laboratory Director

* Results Reported on an "As Received" Basis

(a) Rev. July 2010

(b) Rev. June 2007

(c) Rev. November 2006

N.D. - None Detected

Quality Control Summary - All laboratory quality control parameters were found to be within established limits

Page No. 1

APPENDIX D

Table D-1 – Fault Parameters

Table D-2 – Historical Earthquakes in Vicinity of Project Site

Table D-3 – Deterministic Spectral Response Values

Table D-4 – Site Specific Spectral Response Values

Table D-5 – General Procedure Seismic Design Values

Table D-1
Fault Parameters

Fault Section Name	Distance		Upper Sels.	Lower Sels.	Avg Dip	Avg Dip	Avg Rake	Trace Length	Fault Type	Mean Mag	Mean Return interval	Slip Rate
	(miles)	(km)	Depth (km)	Depth (km)	Angle (deg.)	Direction (deg.)	(deg.)	(km)			(years)	(mm/yr)
Mission Ridge-Arroyo Parida-Santa Ana FM3.1, 3.	0.3	0.5	0.0	7.6	70	176	90	69	B	6.8		0.4
Red Mountain FM3.1, 3.2	3.4	5.5	0.0	14.1	56	2	90	101	B	7.4		2
North Channel FM3.2	5.9	9.4	1.1	4.5	26	10	90	51	B	6.7		1
Pitas Point (Upper) FM3.2	6.4	10.3	1.4	10.0	42	15	90	35	B	6.8		1
Oak Ridge (Offshore), west extension FM3.2	8.2	13.2	0.0	3.1	67	195	na	28	B'	6.1		
Santa Ynez (West) FM3.1, 3.2	8.2	13.2	0.0	9.2	70	182	0	80	B	6.9		2
San Luis Range (So Margin) FM3.2	9.3	15.0	0.0	12.0	45	37	90	115	B	7.1		0.2
Los Alamos extension FM3.1, 3.2	9.3	15.0	0.0	12.0	30	na	na	22	B'	6.8		
Santa Ynez River FM3.1, 3.2	9.5	15.3	0.0	12.0	70	na	na	73	B'	7.1		
Pitas Point (Lower, West), FM 3.1	10.7	17.2	1.5	8.8	13	3	90	35	B	7.2		2.5
Channel Islands Western Deep Ramp FM3.1, 3.2	10.8	17.4	4.8	12.5	21	204	90	62	B'	7.3		
Santa Ynez (East) FM3.1, 3.2	12.0	19.3	0.0	13.3	70	172	0	68	B	7.2		2
Oak Ridge (Offshore) FM3.2	14.7	23.7	0.0	7.9	32	180	90	38	B	6.9		3
Ventura-Pitas Point FM3.1, 3.2	16.0	25.8	1.0	15.0	64	353	60	59	B	6.9		1
Pitas Point (Lower)-Montalvo, FM3.1	17.9	28.8	0.4	12.7	16	359	90	30	B	7.3		2.5
Big Pine (West) FM3.1, 3.2	20.4	32.8	0.0	11.0	50	2	na	18	B'	6.5		
Los Alamos 2011 CFM FM3.1, 3.2	21.0	33.8	0.0	12.0	30	na	na	27	B'	6.9		
Lions Head 2011 CFM FM3.1, 3.2	22.0	35.4	0.0	12.0	75	29	90	65	B	6.7		0.02
Santa Cruz Island FM3.1, 3.2	26.4	42.5	0.0	13.3	90	188	30	69	B	7.1		1
East Huasna 2011 CFM FM3.1, 3.2	26.9	43.3	0.0	15.0	90	na	na	74	B'	7.2		
Pine Mtn FM3.1, 3.2	28.1	45.2	0.0	16.3	45	5	na	62	B'	7.3		
San Luis Range 2011 CFM, FM3.1	28.2	45.4	0.0	12.0	52	na	na	79	B'	7.2		
Santa Rosa Island FM3.1, 3.2	29.0	46.7	0.0	8.7	90	1	30	58	B	6.8		1
Ozena FM3.1, 3.2	30.0	48.3	0.0	13.9	33	na	na	41	B'	7.2		
Sisar FM3.1, 3.2	30.7	49.5	0.0	17.4	29	168	na	20	B'	7.0		
Channel Islands Thrust FM3.1, 3.2	31.4	50.5	5.0	12.3	20	354	90	59	B	7.3		1.5
Big Pine (Central) FM3.1, 3.2	31.5	50.7	0.0	6.6	76	167	na	23	B'	6.3		
South Cuyama FM3.1, 3.2	32.4	52.1	0.0	13.9	33	210	na	83	B'	7.5		
Santa Cruz Catalina Ridge Alt2 FM3.2	36.1	58.1	0.0	11.0	90	38	na	137	B'	7.3		
Santa Cruz Catalina Ridge alt 1, FM3.1	36.1	58.1	0.0	11.0	90	na	na	114	B'	7.2		
Oak Ridge (Onshore) FM3.1, 3.2	36.9	59.4	1.0	19.4	65	159	90	50	B	7.2		4
Casmalia 2011 CFM	37.2	59.9	0.0	12.0	75	na	na	48	B'	6.9		
San Cayetano FM3.1, 3.2	38.1	61.3	0.0	16.0	42	3	90	42	B	7.2		6
Morales (West) FM3.1, 3.2	38.3	61.7	0.0	8.6	32	49	na	28	B'	6.8		
Malibu Coast (Extension), alt 1, FM3.1	38.6	62.2	0.0	7.8	74	4	30	35	B'	6.5		
Malibu Coast (Extension), alt 2 FM3.2	38.6	62.2	0.0	16.6	74	4	30	35	B'	6.9		
Morales (East) FM3.1, 3.2	39.1	62.9	0.0	8.6	32	14	na	18	B'	6.6		
Big Pine (East) FM3.1, 3.2	40.3	64.9	0.0	14.3	73	338	na	23	B'	6.6		
San Andreas (Big Bend) FM3.1, 3.2	42.9	69.0	0.0	15.1	90	198	180	50	A	6.8	89	3.5
San Andreas (Carrizo) rev FM3.1, 3.2	43.7	70.3	0.0	15.1	90	224	180	59	A	6.8	89	3.5

Reference: USGS OFR 2013-1165 (CGS SP 228)

Based on Site Coordinates of 34.4263 Latitude, -119.827 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 Ellsworths-B and Hanks & Bakun moment area relationship.

Site Coordinates: 34.426 N 119.827 W

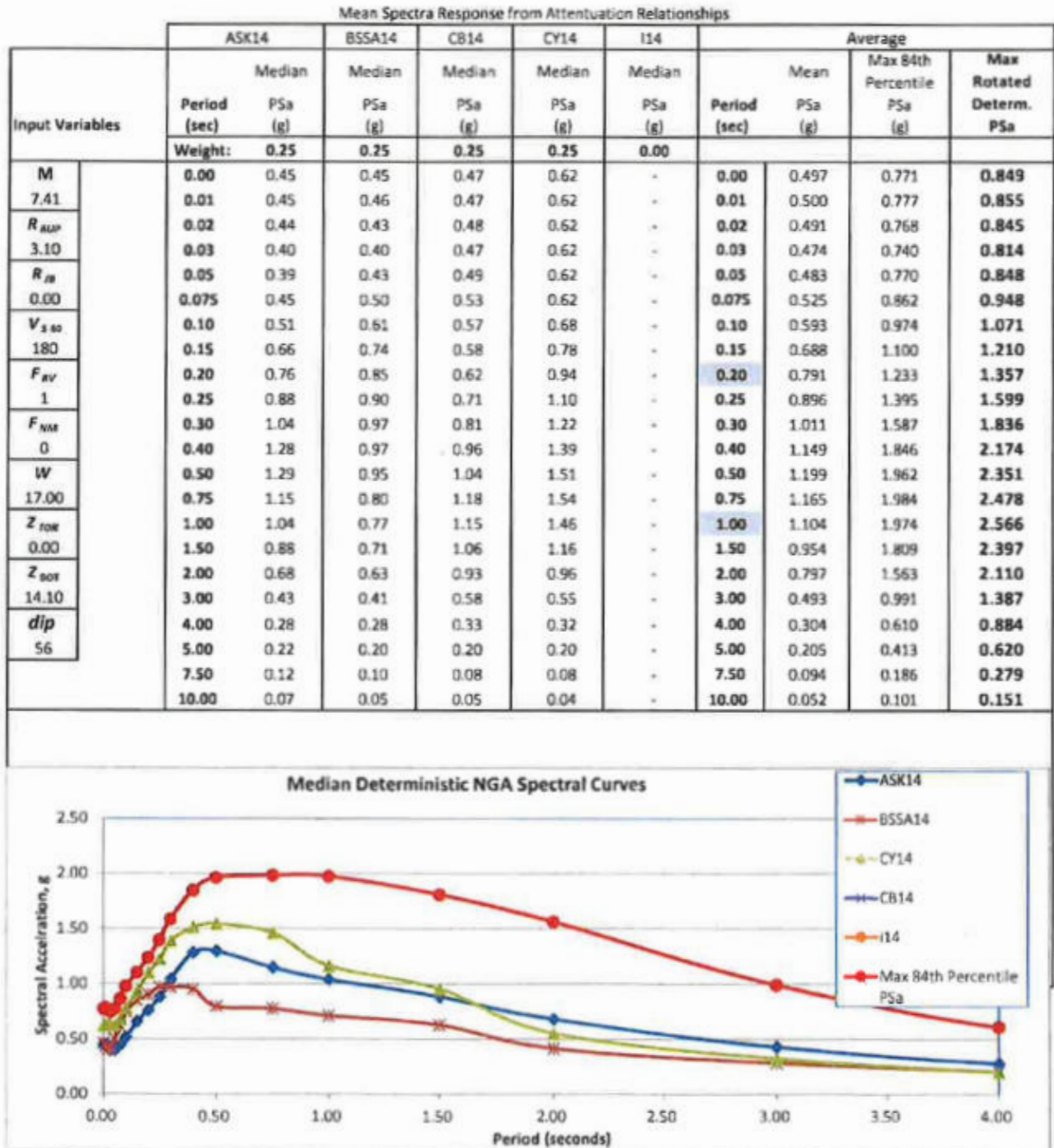
Table D-2
Historical Earthquakes in Vicinity of Project Site, $M \geq 5.0$

Day	Year	Epicenter		Distance from Site (mi)	Magnitude M_W
		Latitude (Degrees)	Longitude		
5/31	1854	34.45	119.70	7.4	6.0
3/25	1806	34.40	119.70	7.5	5.5
1/1	1821	34.55	119.85	8.6	6.3
6/29	1925	34.30	119.80	8.9	6.8
8/13	1978	34.31	119.71	10.4	5.8
12/21	1812	34.20	119.80	15.7	7.1
7/1	1941	34.18	119.67	19.2	5.9
6/29	1926	34.50	119.50	19.3	5.5
8/5	1930	34.50	119.50	19.3	5.0
7/5	1968	34.16	119.68	20.2	5.0
2/18	1926	34.30	119.50	20.6	5.5
1/12	*1915	34.60	120.20	24.4	5.7
7/31	1902	34.70	120.30	32.9	5.8
5/19	1893	34.10	119.40	33.2	5.8
8/6	1973	33.95	119.47	38.7	5.1
4/5	2018	33.84	119.73	41.1	5.3
7/28	1902	34.80	120.40	41.5	5.8
4/7	1885	35.00	119.50	43.7	5.5
9/5	1883	34.90	119.30	44.3	5.8
2/16	1919	34.90	119.20	48.3	5.7
2/21	1973	33.99	119.05	53.6	5.9
10/23	1916	34.85	119.00	55.3	5.5
4/16	2005	35.03	119.18	55.5	5.2
9/24	1827	34.00	119.00	55.6	6.0
1/12	1954	34.99	119.06	58.4	5.7
7/21	1952	35.00	119.07	58.4	6.3
9/21	1941	34.87	118.93	59.5	5.1
8/7	1952	35.05	119.10	59.6	5.0
7/21	1952	34.98	119.02	59.7	7.5
5/23	1954	34.98	119.00	60.5	5.0
11/4	*1927	34.60	120.90	62.2	7.1
7/23	1952	34.96	118.92	63.3	5.1
1/19	1994	34.38	118.71	63.7	5.3
1/18	1994	34.38	118.70	64.3	5.2
1/17	1994	34.33	118.70	64.6	5.8
5/28	1993	35.15	119.10	64.8	5.2

From full earthquake catalog in USGS OFR 2008-1437h as updated with current events through 2021. For events with an asterisk, alternate solutions are given in the OFR. Ordered By Closest Event. Maximum 40 Closest Events Shown.

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 - Chiou - Youngs (2014), and Idriss (2013)



34.4263 -119.827 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	Max Rotated Probab. 2% in 50 year MCE Spectrum	Max Rotated 84th Percentile Determ. MCE Spectrum	Determ. Lower Limit MCE Spectrum	Determ. MCE Spectrum	Site Specific MCE, Ground Response (SaM)	Site Specific MCE Spectrum Comparator	2019 CBC MCE Spectrum	Site Specific Design Spectrum (Sa)	2019 CBC Design Spectrum
	(1) 2475-year (ASCE 21.2.1)	(2) 2475-year (ASCE 21.2.1.1)	(3) (ASCE 21.2.2)	(4) (10 * 100-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	0.799	0.770	0.849	0.849	0.849	0.770	0.770	0.939	0.513	0.626
0.05	0.988	0.952	0.848	0.848	0.848	0.848	0.952	1.190	0.635	0.793
0.10	1.178	1.135	1.071	1.071	1.071	1.071	1.153	1.441	0.769	0.961
0.15	1.394	1.343	1.210	1.210	1.210	1.210	1.354	1.692	0.902	1.128
0.20	1.548	1.492	1.357	1.357	1.357	1.357	1.554	1.943	1.036	1.295
0.30	1.847	1.872	1.836	1.836	1.836	1.836	1.878	2.348	1.252	1.565
0.40	1.991	2.053	2.174	2.174	2.174	2.053	2.053	2.348	1.369	1.565
0.50	2.102	2.205	2.351	2.351	2.351	2.205	2.205	2.348	1.470	1.565
0.75	1.932	2.112	2.478	2.478	2.478	2.112	2.112	2.348	1.408	1.565
1.00	1.811	2.059	2.566	2.566	2.566	2.059	2.059	2.348	1.373	1.565
1.50	1.480	1.716	2.397	2.397	2.397	1.716	1.758	2.197	1.172	1.465
2.00	1.245	1.471	2.110	2.110	2.110	1.471	1.471	1.648	0.980	1.099
3.00	0.781	0.957	1.387	1.387	1.387	0.957	0.957	1.099	0.638	0.732
4.00	0.499	0.632	0.884	0.884	0.884	0.632	0.659	0.824	0.439	0.549
5.00	0.349	0.458	0.620	0.620	0.620	0.458	0.527	0.659	0.352	0.439
8.00	0.145	0.190	0.338	0.338	0.338	0.190	0.330	0.412	0.220	0.275
10.00	0.100	0.131	0.151	0.151	0.151	0.131	0.264	0.264	0.176	0.176

C_{es}: 0.876C_{st}: 0.875Site Specific T₀: 0.296 = 0.2 * S_{0.4}/S_{0.5}Site Specific T₁: 1.482 = S_{0.4}/S_{0.5}

The value of F₀ used in Column (3) is defined
 within ASCE 21.2.2 Supplement 1. This F₀ value
 only applies within Column (3).

Probabilistic spectrum from 2014 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

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>).

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

Calculation Utilized ASCE 7-16, Section 21.2.1.1 - Method 1

Short-Period Seismic Design Category:	1-Second Period Seismic Design Category:
E	E

Vertical Coefficient (C _v)
1.50

1 g = 980.6 cm/sec² = 32.2 ft/sec²PSV [ft/sec] = 32.2(S_v)/T(2p)

Deterministic Fault Parameters			
Red Mountain FM3.1, 3.2	R _{0.8} (km)	0	
Magnitude	7.41	R _{0.1} (km)	3.10
Distance (km)	5.5	Z ₁₀₀ (km)	0
Width (km)	17	Z _{0.01} (km)	14.1
Dip (Deg.)	56	V ₅₂₀ (m/s)	180

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

Site Coefficients

F _{PGA}	1.10
F _s	1.00
F ₀	4.00

Mapped MCE Acceleration Values

PGA	1.037	g
S ₁	2.348	g
S _{0.1}	0.824	g

Seismic Site Class

E

Risk Category

II

Site-Specific

Design Acceleration Values

PGA _M	0.913	g
S _{0.5}	1.323	g
S _{0.1}	1.961	g

Site-Specific

MCE, 5% damped, Spectral Response
Acceleration Parameter

S _{M1}	1.985	g
S _{M2}	2.941	g

Table D-5 - General Procedure Seismic Design Values

2019 California Building Code (CBC) (ASCE 7-16) Seismic Design Parameters

(Values presented should only be used by a Structural Engineer to determine if the exception in 11.4.8 (ASCE 7-16) can be used)

Seismic Design Category	E	CBC Reference	ASCE 7-16 Reference
Site Class	E	Table 1613.5.6	Table 11.6-2
Latitude:	34.426	Table 1613.5.2	Table 20.3-1
Longitude:	-119.827		
<u>Maximum Considered Earthquake (MCE) Ground Motion</u>			
Short Period Spectral Response S_s	2.348 g	Figure 1613.5	Figure 22-1
1 second Spectral Response S_1	0.824 g	Figure 1613.5	Figure 22-2
Site Coefficient F_a	1.20 **	Table 1613.5.3(1)	Table 11.4-1
Site Coefficient F_v	2.00	Table 1613.5.3(2)	Table 11-4.2
S_{MS}	2.818 g	$= F_a * S_s$	
S_{M1}	1.648 g	$= F_v * S_1$	

**Exception of ASCE7-16, Section 11.4.8, Exception Note 1 Applied as Site Class is E, $S_s \geq 1.0$, and therefore F_a was taken to be equal to that of Site Class C.Design Earthquake Ground Motion

Short Period Spectral Response S_{DS}	1.878 g	$= 2/3 * S_{MS}$
1 second Spectral Response S_{D1}	1.099 g	$= 2/3 * S_{M1}$

Site Specific Evaluation May Be Required Due to Site Class = D or E and 11.4.8. The Presented S_{DS} and S_{D1} are NOT Valid Unless the Exception of ASCE7-16, Section 11.4.8 AppliesSite Specific Evaluation May Be Required Due to Site Class = E and 11.4.8. The Presented S_{DS} and S_{D1} are NOT Valid Unless the Exception of ASCE7-16, Section 11.4.8 Applies

T_0	0.12 sec	$= 0.2 * S_{D1} / S_{DS}$
T_s (11.4.8 ASCE 7-16 Exception Assumed)	0.58 sec	$= S_{D1} / S_{DS}$
Risk Category	II	Table 1604.5
Seismic Importance Factor	1.00	
F_{PGA}	1.10	
PGA_M	1.14	
Vertical Coefficient (C_v)	1.50	Table 11.9-1

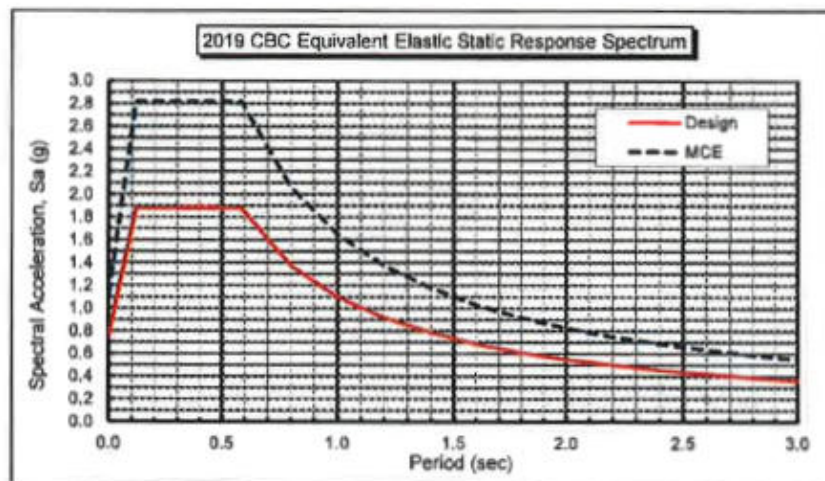
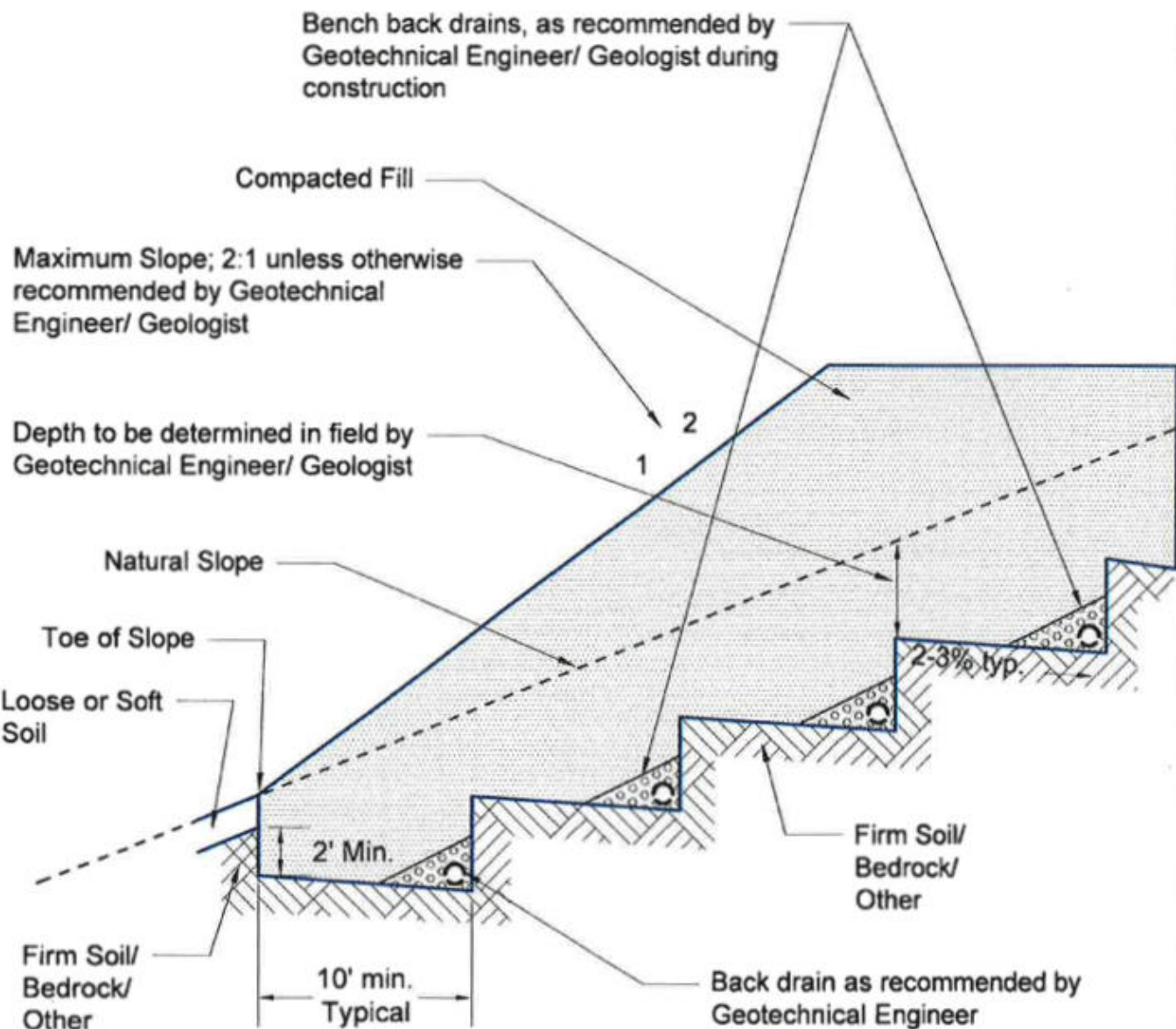


Table 11.5-1 Design

Period T (sec)	S_a (g)
0.00	0.751
0.05	1.233
0.12	1.878
0.58	1.878
0.80	1.373
1.00	1.099
1.20	0.916
1.40	0.785
1.60	0.687
1.80	0.610
2.00	0.549
2.20	0.499
2.40	0.458
2.60	0.423
2.80	0.392
3.00	0.366

APPENDIX E

Typical Bench and Keyway Detail



SCHEMATIC ONLY
NOT TO SCALE



Earth Systems Pacific

2049 Preisker Lane, Suite E, Santa Maria, CA 93454
www.earthsystems.com
(805) 928-2991

TYPICAL BENCH AND KEYWAY DETAIL
GOLETA BUSINESS CENTER BUILDING
907 South Kellogg Avenue
Goleta, California

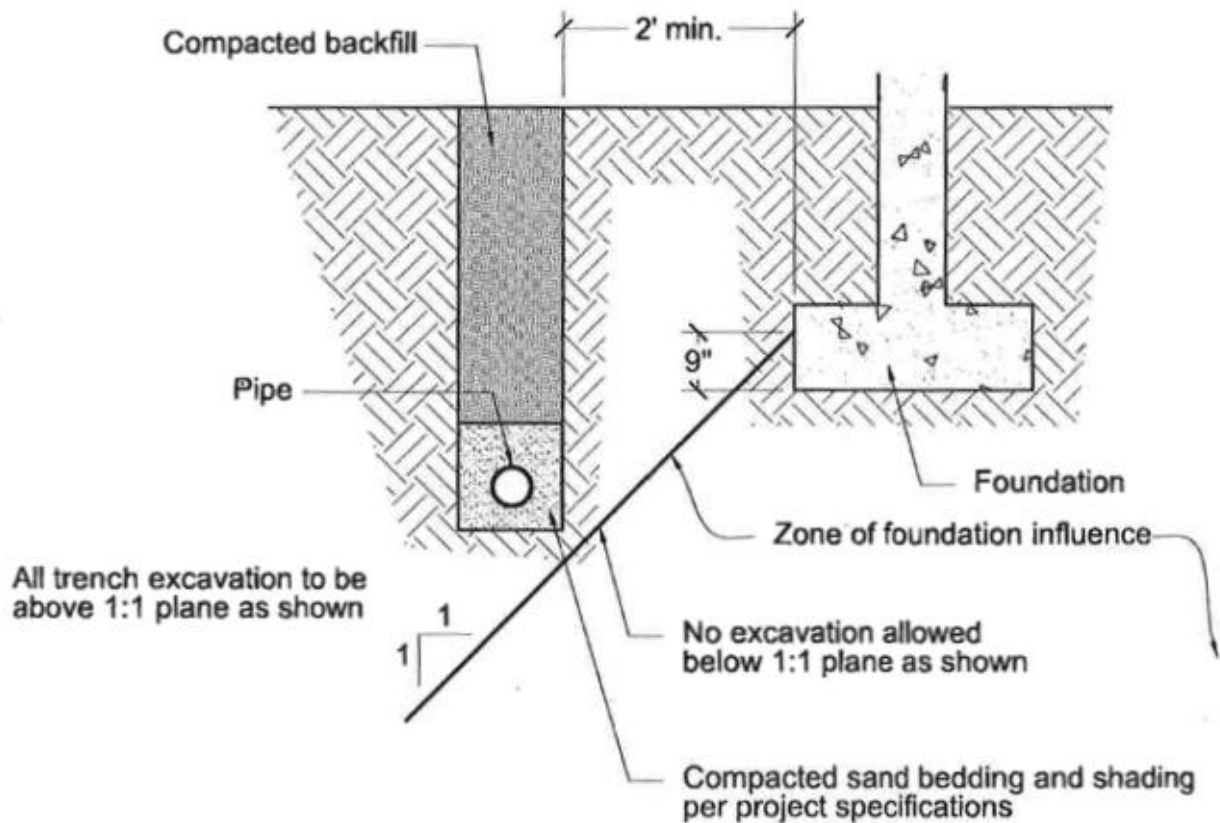
Date
August 24, 2022

Project No.
305544-001

APPENDIX F

Typical Detail A: Pipe Placed Parallel to Foundations

TYPICAL DETAIL A: PIPE PLACED PARALLEL TO FOUNDATIONS



SCHEMATIC ONLY
NOT TO SCALE



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