Riverside County Flood Control and
Water Conservation District
1995 Market Street
Riverside, California 92501

Geotechnical Engineering Report
Proposed Building 6 Modular Upgrade
1995 Market Street
Riverside, Riverside County, California

November 30, 2020

File No.: 302451-002
Doc. No.: 20-11-712

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Attention: Mr. Komy Ghods, PE

Subject: **Geotechnical Engineering Report**

Project: **Proposed Building 6 Modular Upgrade**
1995 Market Street
Riverside, Riverside County, California

Earth Systems Pacific (Earth Systems) is pleased to submit this geotechnical engineering report for the referenced project located at 1995 Market Street in Riverside, Riverside County, California. This report presents our findings and recommendations for site grading and foundation design, incorporating the information provided to our office. The site is suitable for the proposed development, provided the recommendations in this report are followed in design and construction. This report should stand as a whole, and no part of the report should be excerpted or used to the exclusion of any other part.

This report completes our scope of services in accordance with our proposal (PER-20-9-006) with an authorization date of September 29, 2020. Other geotechnical related services that may be required, such as plan reviews, responses to agency inquiries, and grading observation and testing are additional services and will be billed according to the Fee Schedule in effect at the time services are provided. Unless requested in writing, the Client is responsible to distribute the report to the appropriate governing agency and other members of the design team. Please review the Limitations (Section 6) of this report as they are vital to the understanding of this report.

We appreciate the opportunity to provide our professional services. Please contact our office if there are any questions or comments concerning this report or its recommendations.

Respectfully submitted,
Earth Systems Pacific

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Geotechnical Engineering Report
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Section 1
INTRODUCTION

1.1 Project Description

This geotechnical engineering report has been prepared for the proposed Building 6 Modular Upgrade located at 1995 Market Street in Riverside, Riverside County, California, see Plate 1 (Site Vicinity Map). Based upon the preliminary site plan provided, we understand the proposed improvements will consist of removing an old modular building and installing a newer, larger building. The existing modular is approximately 36 by 60 feet and sits on shallow retaining wall type foundations. The proposed new modular will be approximately 42 by 74 feet on a similar retaining wall foundation with raised wood floor. Appurtenant site work is anticipated to include underground utilities, hardscape, parking and driveway improvements, and landscaping. Preliminary grading and foundation plans, and appurtenant site work plans were not available at the time that this report was prepared. We have assumed site grades will be similar in elevation to the surrounding street grades (+-2 feet). The proposed site layout along with our exploration locations is presented in Plate 2 in Appendix A.

We have assumed the building will be one story in height of prefabricated steel and wood construction, founded on shallow wall foundations. No below grade basement levels are anticipated. Column loads are anticipated not to exceed approximately 24 kips for spread footings and 3 kip/lf for continuous footing loads. Preliminary design loading was not provided by the structural engineer. If actual structural loading exceeds these assumed values, we will need to re-evaluate the given recommendations.

1.2 Site Description

The project site is located at 1995 Market Street, in Riverside, Riverside County, California. Coordinates near the center of the proposed building are 34.0039°N/117.3785°W. Site elevation is approximately 807 feet (msl) based on Google Earth. Access to the site is via Market Street, north of the 60 Freeway.

Topographically, the site is generally flat and level. The site elevation is approximately 807 feet. Drainage is by sheet flow to the west. Existing improvements include a modular trailer approximately 36 by 60 feet that sits on shallow foundations with a raised floor. Concrete sidewalks and parking lots are present all around the existing building, as well as landscaping and mature trees (see Plate 2). Underground utilities are abundant in the proposed improvement area.

1.3 Site Reconnaissance

Earth Systems personnel visited the site on October 16, 2020, and have also reviewed select historic aerial photographs of the project site (Google Images and "Historic Aerials" between

1938 and 2020). Based on our review of these historical photographs, it appears that the existing building was constructed between 1994 and 1995.

1.4 Purpose and Scope of Services

The purpose for our services was to evaluate the site soil and geologic conditions at our exploration locations and to provide professional opinions and recommendations, from a geologic and geotechnical point of view, regarding the proposed development of the site. We understand that the proposed site improvements will be developed under the regulation of the current California Building Code (2019). The scope of services included:

Task 1 - Literature and Photograph Reviews

We reviewed select geologic and geotechnical literature pertaining to the project. This included a review of various hazard, fault, and geologic maps prepared by the California Geological Survey (CGS), the U.S. Geological Survey (USGS), Riverside County and other governmental agencies as they relate to the project area. Select historical aerial photographs were reviewed using Google Earth Pro and Historical Aerials website. The aerial photographs reviewed are listed in the References section of this report.

Task 2 – Utility Clearance, USA Dig Alert

Each of our proposed field exploration locations was located and marked in the field and cleared with known utility lines as identified by Underground Service Alert (USA), "Dig Alert" and Riverside Flood personnel. Our exploration locations were located in the field by pacing with sighting from landmarks identified on the project site plan.

Task 3 – Field Exploration

We evaluated the general subsurface conditions at the site by drilling four small-diameter hollow-stem auger borings, to depths from approximately 21½ feet to 45½ feet below the existing ground surface (bgs). The field exploration also included a visual site reconnaissance of the project area and immediate surroundings. Plate 2 shows the approximate locations of the borings.

Task 4 – Laboratory Testing

Laboratory tests were performed on selected samples to evaluate the physical characteristics of the materials encountered during our field exploration. Laboratory testing included moisture content, dry unit weight, maximum dry density/optimum moisture content, sieve analysis, consolidation/collapse potential, and Expansion Index. The testing was performed in general accordance with American Society for Testing and Materials (ASTM) or appropriate test procedures. Selected samples were also tested for a preliminary screening level of corrosion potential (pH, electrical resistivity, water-soluble sulfates and water-soluble chlorides).

Task 5 – Analysis and Report

Earth Systems analyzed the field data obtained, performed engineering analyses, and provided recommended design parameters for earthwork and foundations for the structure as described within. Our report includes:

- A description of the proposed project including a site plan showing the approximate boring locations;
- A description of the surface and subsurface site conditions including groundwater conditions, as encountered in our field exploration;
- A description of the site geologic setting and possible associated geology-related hazards, including liquefaction, subsidence, and seismic settlement analysis;
- A discussion of regional geology and site seismicity;
- A description of local and regional active faults, their distances from the site, their potential for future earthquakes;
- A discussion of other geologic hazards such as ground shaking, landslides, flooding, and tsunamis;
- A discussion of site conditions, including the geotechnical suitability of the site for the general type of construction proposed;
- A seismic analysis including recommendations for geotechnical seismic design coefficients and soil profile type in accordance with the 2019 California Building Code;
- Recommendations for imported fill for use in compacted fills;
- Recommendations for foundation design including parameters for shallow foundations and subgrade preparation;
- Anticipated total and differential settlements for the recommended foundation system;
- Recommendations for site preparation, earthwork, and fill compaction specifications;
- Discussion of anticipated excavation conditions;
- Recommendations for underground utility trench backfill;
- Recommendations for stability of temporary trench excavations;
- Recommendations for slabs-on-grade, including recommendations for reducing the potential for moisture transmission through interior slabs;
- Recommendations for collapsible or expansive soils (if applicable);
- Recommendations for asphalt concrete and Portland cement concrete parking and drives;
- A discussion of the corrosion potential of the near-surface soils encountered during our field exploration;
- An appendix, which includes a summary of the field exploration (computer generated boring logs) and laboratory testing program (computer generated plots).

Not Contained in This Report: Although available through Earth Systems, the current geotechnical scope of our services does not include:

- An environmental Phase 1 assessment.
- A study for the presence or absence of wetlands, hazardous or toxic materials in the soil, surface water, groundwater, or air on, below, or adjacent to the subject property, nor exploration or delineation of past development or remaining structures/debris.

The client did not direct Earth Systems to provide any service to investigate or detect the presence of moisture, mold, or other biological contaminates in or around any structure, or any service that was designed or intended to prevent or lower the risk or the occurrence of the amplification of the same. Client is hereby informed that mold is ubiquitous to the environment, with mold amplification occurring when building materials are impacted by moisture. Site conditions are outside of Earth Systems' control, and mold amplification will likely occur or continue to occur in the presence of moisture. As such, Earth Systems cannot and shall not be held responsible for the occurrence or recurrence of mold.

Section 2
METHODS OF EXPLORATION AND TESTING

2.1 Field Exploration

The subsurface exploration program consisted of advancing four exploratory borings. The borings were drilled to depths ranging from approximately 21½ to 45½ feet bgs using a Mobile B-61 truck-mounted drill rig equipped with 8-inch hollow-stem augers provided by CalPac Drilling of Calimesa, California. The borings were advanced to observe soil profiles and obtain samples for laboratory testing. Drill mud was utilized if needed to control caving and heaving in the borehole. The approximate boring locations are shown on Plate 2, in Appendix A. The locations shown are approximate, established by consumer grade Global Positioning System (GPS) accurate to ± 15 feet in conjunction with pacing based upon the control provided.

Staff from Earth Systems maintained a log of the subsurface conditions encountered and obtained samples for visual observation, classification and laboratory testing. Subsurface conditions encountered in the borings were categorized and logged in general accordance with the Unified Soil Classification System (USCS) and ASTM D 2487 and 2488 (current edition). Our typical sampling interval within the borings was approximately every 2½ or 5 feet to the full depth explored; however, sampling intervals were adjusted depending on the materials encountered onsite. Samples were obtained within the test borings using a Modified California (MC) ring sampler (ASTM D 3550 with those similar to ASTM D 1586). The MC sampler has an approximate 3-inch outside diameter and a 2.4-inch inside diameter. The ring sampler was mounted on a drill rod and driven using a rig-mounted 140-pound automatic hammer falling for a height of 30 inches. The number of blows necessary to drive the MC sampler within the borings was recorded.

Bulk samples of the soil materials were obtained from the drill auger cuttings, representing a mixture of soils encountered at the depths noted. The depth to groundwater, if any, was measured in the boreholes. Following drilling, sampling, and logging, the borings were plugged, backfilled with the cuttings and tamped upon completion. Our field exploration was provided under the direction of a State of California Registered Geotechnical Engineer from our firm.

Design parameters provided by Earth Systems in this report have considered an estimated 79% hammer efficiency based on test data provided by the drilling subcontractor and accepted SP 117A criteria. Since the MC sampler was used in our field exploration to collect ring samples, the N-values using the California sampler can be roughly correlated to SPT N-values using a conversion factor that may vary from about 0.5 to 0.7. In general, a conversion factor of approximately 0.63 from a study at the Port of Los Angeles (Zueger and McNeilan, 1998 per SP 117A) is considered satisfactory. A value of 0.63 was applied in our calculations for this project.

The final logs of the borings represent our interpretation of the contents of the field logs and the results of laboratory testing performed on the samples obtained during the subsurface exploration. The final logs are included in Appendix A of this report. The stratification lines represent the approximate boundaries between soil types, although the transitions may be gradual. In reviewing the logs and legend, the reader should recognize the legend is intended as a guideline only, and there are a number of conditions that may influence the soil characteristics observed during drilling. These include, but are not limited to cementation, oversize rocks, variations in soil moisture, presence of groundwater, and other factors.

The boring logs present field blowcounts per 6 inches of driven embedment (or portion thereof) for a total driven depth attempted of 18 inches. The blow counts on the logs are uncorrected (i.e. not corrected for overburden, sampling, etc.). Consequently, the user must correct the blow counts per standard methodology if they are to be used for design and exercise judgment in interpreting soil characteristics, possibly resulting in soil descriptions that vary somewhat from the legend.

2.2 Laboratory Testing

Samples were reviewed along with field logs to select those that would be analyzed further. Those selected for laboratory testing include, but were not limited to, soils that would be exposed and those deemed to be within the influence of the proposed structures. Test results are presented in graphic and tabular form in Appendix B of this report. Testing was performed in general accordance with American Society for Testing and Materials (ASTM) or other appropriate test procedure. A selected sample was also tested for a screening level of corrosion potential (pH, electrical resistivity, water-soluble sulfates, and water-soluble chlorides). Earth Systems does not practice corrosion engineering; however, these test results may be used by a qualified corrosion engineer in designing an appropriate corrosion control plan for the project.

Our testing program consisted of the following:

- Density and Moisture Content of select samples of the site soils (ASTM D 2937 & 2216).
- Maximum Dry Density/Optimum Moisture Content tests to evaluate the moisture-density relationship of typical soils encountered (ASTM D 1557).
- Particle Size Analysis to classify and evaluate soil composition. The gradation characteristics of selected samples were made by wash sieve procedures (ASTM D 1140).
- Consolidation and Collapse Potential to evaluate the compressibility and hydroconsolidation (collapse) potential of the soil upon wetting (ASTM D 5333).
- Direct Shear to evaluate the relative frictional strength of the surficial slope soils. Specimens were in a saturated condition prior to and during testing and were sheared under normal loads ranging from 1.0 to 4.0 kips per square foot (ASTM D 3080).
- Expansion Index tests to evaluate the expansive nature of the soil. The samples were surcharged under 144 pounds per square foot at moisture contents of near 50% saturation. Samples were then submerged in water for 24 hours and the amount of expansion was recorded with a dial indicator (ASTM D 4829).
- Chemical Analyses (Soluble Sulfates and Chlorides (ASTM D 4327), pH (ASTM G 187), and Electrical Resistivity/Conductivity (APHA 2320-B) to evaluate the potential for adverse effects of the soil on concrete and steel.
- One-Dimensional Swell or Settlement Potential of Cohesive Soils to determine heave or settlement and swell pressures (ASTM D 4546).

Section 3
DISCUSSION

3.1 Soil Conditions

The field exploration indicates that site soils consist generally of silty sand, well graded sand, poorly graded sands, poorly graded sand with silt, as well as isolated lean clay, and fat clay (from approximately 7 to 10 feet below the ground surface), (Unified Soils Classification System symbols of SM, SW, SP, SP-SM, CL, and CH) to the maximum depth of exploration of 45½ feet below the ground surface. The coarse-grained soils encountered were found to be loose to very dense. The fine-grained soils encountered were found to be soft to firm. The boring logs provided in Appendix A includes more detailed descriptions of the soils encountered. Site soils are classified as Type C in accordance with Cal OSHA.

3.2 Groundwater

Groundwater or perched water was encountered during our field exploration at depths ranging from approximately 15 to 20 feet bgs. The site is located within the Upper Santa Ana watershed that includes the Chino basin and Santa Ana River. The historic high depth to groundwater in the area is believed to be about 12-17 feet based on information from the Western Municipal Water District Cooperative Well Measuring Program (2009).

Nearby State monitoring wells were researched for their recent and historic well readings. The following is a summary of our findings for the two wells closest to the site.

- State Well No. 02S05W10P001S is located approximately 1 mile northwest of the project site. The surface elevation of this well is approximately 860 feet and the groundwater readings as measured from 2011 to 2020 varied from 773 to 778 feet.
- State Well No. 02S05W23F001S is located approximately 1.4 miles southeast of the project site. The surface elevation of this well is approximately 843 feet and the groundwater readings as measured from 2011 to 2020 varied from 769 to 763 feet.

Based on the above data, groundwater is not anticipated to be encountered during construction. The historic groundwater depth is estimated to be approximately 15 feet deep at the site (approximal water surface elevation of 792 feet). Fluctuations of the groundwater level and localized zones of increased soil moisture content should be anticipated during and following the rainy season or from irrigation.

3.3 Collapse Potential/Consolidation Potential

Collapsible soil deposits generally exist in regions of moisture deficiency. Collapsible soils are generally defined as soils that have potential to suddenly decrease in volume upon increase in moisture content even without an increase in external loads. Soils susceptible to collapse include loess, weakly cemented sands and silts where the cementing agent is soluble (e.g. soluble gypsum, halite), valley alluvial deposits within semi-arid to arid climate, and certain granite residual soils above the groundwater table. In arid climatic regions, granular soils may have a potential to collapse upon wetting. Collapse (hydro-consolidation) may occur when the soils are lubricated or the soluble cements (carbonates) in the soil matrix dissolve, causing the soil to densify from its loose configuration from deposition.

The degree of collapse of a soil can be defined by the Collapse Potential (CP) value, which is expressed as a percent of collapse of the sample using the Collapse Potential Test (ASTM Standard Test Method D 5333). Based on the Naval Facilities Engineering Command (NAVFAC) Design Manual 7.1, the severity of collapse potential is commonly evaluated by the following Table 1, Collapse Potential Values.

Table 1
Collapse Potential Values

Collapse Potential Value	Severity of Problem
0-1%	No Problem
1-5%	Moderate Problem
5-10%	Trouble
10-20%	Severe Trouble
> 20%	Very Severe Trouble

Table 1 can be combined with other factors such as the probability of ground wetting to occur on-site and the extent or depth of potential collapsible soil zone to evaluate the potential hazard by collapsible soil at a specific site. A hazard ranking system associated with collapsible soil as developed by Hunt (1984) is presented in Table 2, Collapsible Soil Hazard Ranking System.

Table 2
Collapsible Soil Hazard Ranking System

Degree of Hazard	Definition of Hazard
No Hazard	No hazard exists where the potential collapse magnitudes are non-existent under any condition of ground wetting.
Low Hazard	Low hazards exist where the potential collapse magnitudes are small and tolerable, or the probability of significant ground wetting is low.
Moderate Hazard	Moderate hazards exist where the potential collapse magnitudes are undesirable, or the probability of substantial ground wetting is low, or the occurrence of the collapsible unit is limited.
High Hazard	High hazard exists where potential collapse magnitudes are undesirably high and the probability of occurrence is high.

The results of collapse potential tests performed on four selected samples from depths ranging from 5 to 15 feet below the ground surface indicated a collapse potential on the order of 0.6 to 1.0 percent. Based on the above criteria and our field and laboratory findings, we estimate the potential for collapse is “No Problem”.

3.4 Expansive Soils

Expansive soils are characterized by their ability to undergo significant volume change (shrink or swell) due to variations in moisture content. Changes in soil moisture content can result from rainfall, landscape irrigation, utility leakage, roof drainage, perched groundwater, drought, or other factors, and may cause unacceptable settlement or heave of structures, concrete slabs supported-on-grade, or pavements supported over these materials. Depending on the extent and location below finished subgrade, expansive soils can have a detrimental effect on structures. The majority of soils encountered are granular and sandy, and based on visual classification “very low” in Expansion Index; however, based on our laboratory test at boring B-2, the Expansion Index of the on-site fine-grained fat clay soils encountered from 7 to 10 feet below the ground surface (bgs) and 10 to 15 feet bgs at boring B-3 is generally “high”. This is as defined by ASTM D 4829 and the 2019 California Building Code based on approximate 50% saturation levels. The samples from these depths are currently saturated insitu and exhibited no swell upon testing. Earth Systems then performed additional lab testing for heave pressures on samples air dried back to approximately 50% saturation and found the samples tested obtained a heave pressure of approximately 750 to 1000 pounds per square foot (psf). This heave pressure was found to be approximately the overburden stress, and this limits the amount of expansive movement if the proposed pad grade elevation is approximately the same as existing.

Testing and/or observation of the subgrade soils during grading of the building pad and at footing grade should be performed to further evaluate the expansion potential and confirm or modify the recommendations presented herein.

3.5 Corrosion Potential

One sample of the near-surface soils within the site was tested for potential corrosion of concrete and ferrous metals. Soils in the upper 0 to 5 feet were tested as a blended (composite) sample. The tests were conducted in general accordance with the ASTM Standard Test Methods to evaluate pH, resistivity, and water-soluble sulfate and chloride content. The test results are presented in Appendix B. These tests should be considered as only an indicator of corrosivity for the samples tested. Other earth materials found on site may be more, less, or of a similar corrosive nature.

Water-soluble sulfates in soil can react adversely with concrete. ACI 318 provides the relationship between corrosivity to concrete and sulfate concentration, presented in the table below:

Table 3

Water-Soluble Sulfate in Soil (ppm)	Corrosivity to Concrete
0-1,000	Negligible
1,000 – 2,000	Moderate
2,000 – 20,000	Severe
Over 20,000	Very Severe

In general, the lower the pH (the more acidic the environment), the higher the soil corrosivity will be with respect to ferrous structures and utilities. As soil pH increases above 7 (the neutral value), the soil is increasingly more alkaline and less corrosive to buried steel structures, due to protective surface films, which form on steel in high pH environments. A pH between 5 and 8.5

is generally considered relatively passive from a corrosion standpoint. High chloride levels tend to reduce soil resistivity and break down otherwise protective surface deposits, which can result in corrosion of buried steel or reinforced concrete structures. Soil resistivity is a measure of how easily electrical current flows through soils and is the most influential factor. Based on the findings of studies presented in ASTM STP 1013 titled "Effects of Soil Characteristics on Corrosion" (ASTM, 1989), the approximate relationship between soil resistivity and soil corrosivity was developed as shown in Table 4.

Table 4

Soil Resistivity (Ohm-cm)	Corrosivity to Ferrous Metals
0 to 900	Very Severely Corrosive
900 to 2,300	Severely Corrosive
2,300 to 5,000	Moderately Corrosive
5,000 to 10,000	Mildly Corrosive
10,000 to >100,000	Very Mildly Corrosive

Test results show a pH value of 7.9, chloride content of 13 ppm, sulfate content of 34 ppm and minimum resistivity of 6,000 Ohm-cm. Although Earth Systems does not practice corrosion engineering, the corrosion values from the soils tested are normally considered as being mildly corrosive to buried metals and as possessing a "negligible" exposure to sulfate attack for concrete as defined in American Concrete Institute (ACI, 2011) 318, Section 4.3. The results of chemical testing have been provided in Appendix B. The above values can potentially change based on several factors, such as importing soil from another job site and the quality of construction water used during grading and subsequent landscape irrigation.

3.6 Geologic Setting

Regional Geology: The site is situated in the north-central area of the landward portion of the Peninsular Ranges Geomorphic Province of California. The Peninsular Ranges Province is a distinct geomorphic region characterized as a complex series of northwest-southeast oriented mountain ranges and valleys generally sub-parallel to faults composing the San Andreas rift zone. The Peninsular Ranges Province is further described by sub-units, which include the Perris Block, the San Ana Mountains, and the San Jacinto Mountains.

The Perris Block is characterized as a broad area of intermixed valleys and low mountain ranges situated between the Elsinore and San Jacinto fault zones. In the Riverside area, the regional geomorphology is dominated by the Elsinore and San Jacinto fault zones, Jurupa Mountains, Pedley Hills, Chino basin, San Bernardino Valley, and Santa Ana River. The project site is located within the north-central portion of the Perris Block in an area of elevated older alluvial fans adjacent to the Santa Ana River.

Regional earth units consist predominantly of igneous rocks of the southern California batholith, Mesozoic metamorphic rocks of the Jurupa series, and Quaternary alluvium. Regional active and potentially active faults in the vicinity of the project site include the San Jacinto, Elsinore, Cucamonga, and San Andreas fault zones. A Regional Geologic map is presented as Plate 3 in appendix A.

Local Geology: The site is located within the Santa Ana River valley on younger fluvial sediments associated with deposition by the Santa Ana River. Lithologic materials within and adjacent to the Santa Ana River include intermixed intrusive igneous rocks of the southern California batholith overlain by Quaternary sediments, including Pleistocene older alluvium, and Holocene wash deposits. Younger fluvial/stream channel deposits composed of intermixed sand, gravel, clay, and silt underlie the site.

No major faults have been mapped within the project limits. The site is not located within a currently designated Alquist-Priolo Earthquake Fault Zone or County fault zone. The nearest mapped Holocene-active or Quaternary well-defined/sufficiently active faults are segments of the San Jacinto fault located approximately 5 to 6 miles northeast of the site. The Fontana seismic zone is located northwest of the site.

3.7 Geologic Hazards

Geologic hazards that may affect the region include seismic hazards (ground shaking, surface fault rupture, soil liquefaction, and other secondary earthquake-related hazards), slope instability, flooding, ground subsidence, and erosion. A discussion follows on the specific hazards to this site.

Seismic Sources: Several active faults or seismic zones lie within 38 miles of the project site as shown on Table A-1 in Appendix A. The primary seismic hazard to the site is strong ground shaking from earthquakes along the Elsinore, San Jacinto, Cucamonga, and San Andreas fault zones.

Surface Fault Rupture: The project site does not lie within a currently delineated State of California, Alquist-Priolo Earthquake Fault Zone (CGS, 2018). Well-delineated fault lines cross through this region as shown on California Geological Survey (CGS) maps (Jennings, 2010); however, no active faults are mapped in the immediate vicinity of the site. Therefore, active fault rupture is unlikely to occur at the project site. While fault rupture would most likely occur along previously established fault traces, future fault rupture could occur at other locations. No prominent aerial photograph lineaments were noted on select historical aerial photographs what would be suggestive of active faulting on or near the project site.

Historic Seismicity: The project area is in the seismically active southern California where approximately 35 earthquakes of magnitude 5.5 or greater have occurred within 60 miles of the park, usually originating on or near the San Andreas, San Jacinto, or Elsinore faults. These include the 1812 Wrightwood, 1894 Lytle Creek, 1899 San Jacinto, 1910 Elsinore (Glen Ivy Hot Springs), 1918 San Jacinto, and 1923 North San Jacinto earthquakes.

Of significance are the multiple earthquake events along the San Jacinto fault at the turn of the century in 1892, 1894, 1899, and 1923. Additional earthquakes in the region along this fault zone occurred in 1937 and 1954 suggesting that the San Jacinto fault is a significant source of large to major earthquakes. Of interest, the only significant historic earthquake along the local Elsinore fault was in 1910.

Seismic Risk: The primary seismic risk at the site is a potential earthquake along the Holocene-active Elsinore, San Jacinto, Cucamonga, and San Andreas fault. While accurate earthquake predictions are not possible, various agencies have conducted statistical risk analyses. In 2013, the California Geological Survey (CGS) and the United States Geological Survey (USGS) presented new earthquake forecasts for California (USGS UCERF3). We have used these maps in our evaluation of the seismic risk at the site. The recent Working Group of California Earthquake Probabilities (WGCEP, 2014) estimated a 14 percent conditional probability that a magnitude 6.7 or greater earthquake may occur in 30 years (2014 as base year) along the nearby San Jacinto fault. The revised estimate for a 7+ magnitude earthquake along the nearby San Bernardino segment San Andreas fault is about 29%.

Soil Liquefaction and Lateral Spreading: Liquefaction is the loss of soil strength from sudden shock (usually earthquake shaking), causing the soil to become a fluid mass. Liquefaction describes a phenomenon in which saturated soil loses shear strength and deforms as a result of increased pore water pressure induced by strong ground shaking during an earthquake. Dissipation of the excess pore pressures will produce volume changes within the liquefied soil layer, which can cause settlement. Shear strength reduction combined with inertial forces from the ground motion may also result in lateral migration (lateral spreading). Factors known to influence liquefaction include soil type, structure, grain size, relative density, confining pressure, depth to groundwater, and the intensity and duration of ground shaking. Soils most susceptible to liquefaction are saturated, loose sandy soils and low plasticity clay and silt.

In general, for the effects of liquefaction to be manifested at the surface, groundwater levels must be within 50 feet of the ground surface and the soils within the saturated zone must also be susceptible to liquefaction. We consider the potential for liquefaction to occur at this site as moderate to high because historic groundwater is generally less than 50 feet below the ground surface. We used a Magnitude Earthquake of 7.8 on the San Jacinto fault zone having a peak ground acceleration of 0.63g. Liquefaction output considering historic groundwater levels are presented in Appendix A. Results indicate a liquefaction potential at depths greater than 15 feet with estimated liquefaction induced settlement of 1.6 inches at borings B-2/B-3 composite for the liquefied portion. The dry settlement portion evaluated at full PGA_M is 1.2 inches which may be overstated (see following section). Clay soils onsite were generally plastic and would have a Plasticity Index greater than 7 (Boulanger and Idriss, 2006) exhibiting “clay like” properties in terms of liquefaction susceptibility. Due to near saturated water content, these clays could cyclically soften.

Dry Seismic Settlement: The amount of dry seismic settlement is dependent on relative density of the soil, ground motion, and earthquake duration. In accordance with current CGS policy (Earth Systems discussion with Jennifer Thornburg, CGS May 2014), we used a site peak ground acceleration of ⅔ $PGA_{M.}$ The reasoning for using ⅔ PGA_M has stemmed from the limitations of the Pradel (modified Tokimatsu and Seed) method of calculating dry seismic settlement. This issue manifested during the code change from ASCE 7-05 to ASCE 7-10 (2013) and has been continued into the ASCE 7-16 document, where it was no longer codified to evaluate liquefaction and seismic settlement as a design value (S_{DS}/2.5) but to evaluate it at full PGA. PGA_M equated to a maximum considered value and the design spectral response, $S_{DS,}$ is equal to ⅔ S_{MS}, where S_{MS} is the Site Class adjusted maximum considered earthquake value. When using the full PGA_M in Pradel’s method, (Pradel’s method is the Standard of Care methodology of calculating dry sand settlement), the potential settlements increased dramatically resulting in unreasonable amounts of assumed settlement (stress/strain curves defaulted to 10 percent vertical strain as high acceleration) which did not correlate with field observations of earthquake events. Given these unreasonably high potential settlements and after much discussion with agencies and CGS, the PGA used for dry seismic analysis was returned to a "design" value in practice. This is on the basis that since dry seismic shaking will typically not cause bearing failure (like liquefaction will) but is only settlement related deformation. Earth Systems is in concurrence with this methodology and thus the method utilized for our analysis and that ⅔ PGA_M may be used as an appropriate acceleration in the analysis of seismic settlement of unsaturated soils.

In accordance with the above discussion, we used a site peak ground acceleration of ⅔ PGA_M (where PGA_M = 0.63) and an earthquake magnitude of 7.8 to evaluate dry seismic settlement potential. The design peak ground acceleration values were obtained from the SEAOC/OSHPD online application (https://seismicmaps.org/). Based upon methods presented by Tokimatsu and Seed (1987), the potential for seismically induced dry settlement of soils above the groundwater table for the full soil column height (47 feet should the water table drop) was calculated in our deep boring at the site and estimated to be 0.6 inches in borings B-2/B-3 composite. Seismic settlement is based on post grading recommendations stated in Section 5.1.

Vertical Settlement from Liquefaction and Dry Seismic Analysis: Due to the general uniformity of the soils encountered, seismic settlement is expected to occur on an areal basis and as such per Special Publication 117A (CGS, 2008), the differential settlement potential is estimated to be approximately ½ of the total combined settlement for liquefaction and dry seismic settlement. Half of the total at full PGA_M is 1.4 inches and approximately 1.1 inches at design PGA_M for the dry portion. These valves are less than ¼ of the differential thresholds of ASCE7-16, Section 12.13.9 exception. This considers soil remediation as recommended in Section 5.1. Per ASCE7-16 Section 12, Table 12.13-3, the estimated differential settlement for Type I/II structures is within threshold limits (0.015L) which is estimated at 7.2 inches. Due to the depth and thin susceptible layer as well as recommended remedial grading, bearing failure potential is considered low.

Lateral Spreading: The potential for liquefaction induced lateral spreading under the proposed project is considered high for structures near boring B-3. We used Zhang al. et (2004) to provide screening level analysis for this project and determine if structural or/and geotechnical mitigation measures are required. This method requires the use of liquefaction analysis and its Displacement Potential Index (DPI), the horizontal distance between project and toe of descending slope, and vertical height between the project and toe of descending slope. As such the table below provides the estimated potential areas and estimated lateral displacements:

Table 5

Estimated Lateral Displacement (D_H)

Boring No.	DPI (unitless)	Length (feet)	Height (feet)	D_H (in)
B-3	0.67	540	10	26

Typically, only structural mitigation is acceptable when total settlements are small. ASCE7-16 presents a threshold of 18 inches for lateral spread; however, we consider the potential to occur low. Per California specific SP117A (2008, page 54), Youd (1989), citing data from Japan, suggests that structural mitigation may be acceptable where displacements of less than one foot horizontal and less than four inches vertical are predicted. Therefore, for this paper, large-scale ground displacements are defined as those that exceed 1-3 feet horizontally and 4-6 inches vertically. As shown in discussion for liquefaction and Table 5 above, lateral displacements is within the 1-3 feet horizontal requirement and is less than the 4-6 inch vertical settlement. This geotechnical estimate for horizontal and vertical displacements may be interpreted for small displacements; therefore, structural design alone should be acceptable. Earth System will recommend ties with grade beams or per the structural engineer recommendations; please see Section 5.4 of this report.

Fissuring and Ground Subsidence: The Riverside County Parcel report indicates that the site is within a "Susceptible" potential subsidence area. In areas of fairly uniform thickness of alluvium, fissures are thought to be the result of tensional stress near the ground surface and generally occur near the margins of the areas of maximum subsidence. Surface runoff and erosion of the incipient fissures augment the appearance and size of the fissures.

Changes in pumping regimes can affect localized groundwater depths, related cones of depression, and associated subsidence such that the prediction of where fissures might occur in the future is difficult. In the project area, groundwater depths remain fairly deep and we consider the current subsidence potential very low. However, in the event of future nearby aggressive groundwater pumping and utilization, the occurrence of deep subsidence cannot be ruled out. Changes in regional groundwater pumping could result in areal subsidence. The risk of areal subsidence in the future is more a function of whether groundwater recharge continues and/or over-drafting stops, than geologic processes, and therefore the future risk cannot be predicted or quantified from a geotechnical perspective.

Seismic Hazard Zones: This portion of Riverside County has not been mapped for the California Seismic Hazard Mapping Act (Ca. PRC 2690 to 2699) for earthquake faults, liquefaction or slope instability.

3.7.1 Other Geologic Hazards

Landslides and Slope Instability: The site is relatively flat and slopes are anticipated to be less than 5 feet high. Therefore, potential hazards from static slope instability, landslides, or debris flows are considered very low. The site is protected from the ephemeral Santa Ana river by an earthen levee. This levee appears maintained and we assume meets flood control capability.

Flooding: The project site lies in an area designated as an area with reduced flood risk due to levee, Zone X: "Areas of 0.2% annual chance floodplain; areas of 1% annual chance flood with average depths of less than 1 foot or with drainage areas less than 1 square mile; and areas protected by levees from 1% annual chance flood." This project area and Zone X are identified on FEMA Map No.: 06065C0705G, Panel 45 of 3805, Map Revised August 28, 2008. Appropriate project design by the project civil engineer, construction, and maintenance can minimize the site sheet flooding potential.

Seiches: Seiching is defined as a periodic oscillation of liquid within a container or reservoir. Its period is determined by the resonant characteristics of the container, as controlled by its physical dimensions. Lake Evans is located to the south approximately 2,200 feet of the site and down gradient. However, it is likely any flooding associated with lake related flooding would follow existing local drainage/roadway improvements, such that the impact to the site would be negligible.

Section 4
CONCLUSIONS

The following is a summary of our conclusions and professional opinions based on the data obtained from a review of selected technical literature and the field explorations.

General: Based on our field exploration, laboratory testing, and geotechnical analyses conducted for this study, it is our professional opinion that the site is suitable, from a geotechnical and geologic standpoint, for construction as proposed, provided the recommendations presented in this report are incorporated into project design and construction.

The recommendations presented in this report may change pending a review of final grading plans and foundation plans. Recommendations presented in this report should not be extrapolated to other areas or be used for other projects (beyond those expressly identified within) without our prior review and comment.

Geotechnical Constraints and Mitigation:

- The primary geologic hazard is the potential for moderate to severe ground shaking from earthquakes originating on regional southern California faults. A major earthquake originating on the nearby segments of the San Jacinto, Cucamonga, Elsinore, and San Andreas fault zones and other associated faults would be the critical seismic events that may affect the site within the design life. Engineered design and earthquake-resistant construction increase safety and allow development within seismic areas.
- The underlying geologic condition for seismic design is Site Class F. The site is approximately 6 miles from a Type A seismic source. The minimum seismic design should comply with the 2019 edition of the California Building Code.
- The site is not within a currently designated Alquist-Priolo Earthquake Fault Zone. No known faults or lineaments cross the site. Therefore, the potential for surface fault rupture at the site is considered very low.
- The potential for liquefaction settlement hazards are considered moderate to high for this project. The site is not within an area of significant documented areal subsidence.
- Other geologic hazards, including flooding, and landslides, are considered low potential on this site.
- Based on current conditions, groundwater is not anticipated to be encountered during construction, however wet soils will likely be encountered.
- Much of the existing on-site alluvial soils are very low in Expansion Index; however, an isolated layer of expansive soil was found at a depth of 7 to 10 feet bgs. Soils during grading should be suitable for location under structures or hardscape after remedial grading. Building structure recommendations provided within are based upon using very low expansion potential fill material.
- Laboratory testing of one sample showed potentially “mild” corrosivity to buried metallic elements and “negligible” for sulfate exposure to concrete. See Section 3.5 for further

information. Site soils should be reviewed by an engineer competent in corrosion evaluation.

- In our professional opinion, structures can be supported on interconnected shallow foundations bearing on a zone of properly prepared and compacted soils placed as recommended in Section 5.1. The recommendations that follow are based on “very low” expansion category soils.

Section 5
RECOMMENDATIONS

5.1 Site Development – Grading

A representative of Earth Systems should observe site clearing, grading, and the bottoms of excavations before placing fill. Local variations in soil conditions may warrant increasing or decreasing the depth of recompaction and over-excavation. Proper geotechnical observation and testing during construction is imperative to allow the geotechnical engineer the opportunity to verify assumptions made during the design process, to verify that our geotechnical recommendations have been properly interpreted and implemented during construction and is required by the 2019 California Building Code. Preventative measures to reduce seasonal flooding and erosion should be incorporated into site grading plans. Dust control should also be implemented during construction. Site grading should be in strict compliance with the requirements of the local Air Quality Management District.

Observation of fill placement by the Geotechnical Engineer of Record should be in conformance with Section 17 of the 2019 California Building Code. California Building Code requires full time observation by the geotechnical consultant during site grading (fill placement). Therefore, we recommend that Earth Systems be retained during the construction of the proposed improvements to provide testing and observe compliance with the design concepts and geotechnical recommendations, and to allow design changes in the event that subsurface conditions or methods of construction differ from those assumed while completing this study. Additionally, the California Building Code requires the testing agency to be employed by the project owner or representative (i.e. architect) to avoid a conflict of interest if employed by the contractor. Unless noted otherwise, grading should be performed in general accordance with Appendix J of the 2019 CBC.

<u>Clearing and Grubbing</u>: At the start of site grading, existing vegetation, trees (including the entire rootball), large roots, overly wet and/or soft soil, undocumented fill, pavements, foundations, construction debris, septic tanks, leach fields, deleterious material, trash, and abandoned underground utilities should be removed from the proposed building areas. Organic growth should be stripped off the surface and removed from the construction area. Areas disturbed during demolition and clearing should be properly backfilled and compacted as described below.

Buried utilities may be located in the vicinity of the planned structures and within other areas of the project site. All buried structures which are removed should have the resultant excavation backfilled with soil compacted as engineered fill described herein or with a minimum 2-sack cement-sand slurry (150 psi minimum compressive strength, collect 4 samples at placement) approved by the project geotechnical engineer. Abandoned utilities should be removed entirely, or pressure-filled with concrete or grout and be capped. Abandoned buried utilities structures, or foundations should not extend under building limits.

After stripping and grubbing operations, areas to receive fill should be stripped of loose or soft earth materials until a firm subgrade is exposed, as evaluated by the geotechnical engineer or geologist (or their representative). Before the placement of fill or after cut, the existing surface soils within the building pads and improvement areas should be over-excavated as follows:

Building Pad Preparation: The existing soils within the building pad and foundation areas should be over-excavated a minimum of 4 feet below existing grade or 3 feet below the bottom of the footings, whichever is lower. The over-excavation should extend laterally a distance of 4 feet from the outside edge of the footings and include any shallow footing areas such as canopies, covered walkways, etc., where possible. The exposed undisturbed subgrade bottom should be observed and tested by the geotechnical engineer or his representative to verify an in-place density of the subgrade is at or greater than 85% relative compaction per ASTM D 1557 or soils are firm (as determined by the geotechnical engineer or his representative). Deeper over-excavation may be recommended if the required in-place density is not achieved, soils are not firm, or undocumented fill exists.

The approved bottom of the over-excavation should then be scarified 12 inches; moisture conditioned to near optimum moisture content, and recompacted to at least 90% relative compaction (ASTM D 1557) prior to fill placement. On the compacted bottom of the overexcavation, one layer of Tensar TX160 or Terrafix BX1200, geogrid or direct equivalent should be placed and be pulled snug. The geogrid should wrap up the side walls a minimum of 3 feet. Moisture conditioned and compacted engineered fill should then be placed to finish subgrade elevation in suitable compacted lifts. Compaction should be to at least 90% relative compaction. Compaction should be verified by testing.

Auxiliary Structures Subgrade Preparation: Auxiliary structures such as out-structures, or retaining walls, etc. should have the foundation subgrade prepared similar to the building pad recommendations given above. The over-excavation should extend horizontally for 2 feet beyond the outer edge. The exposed soils should then be moisture conditioned to near optimum moisture content, and recompacted to at least 90 percent relative compaction (ASTM D 1557). Moisture conditioned, engineered fill may then be placed to finished subgrade in suitable, compacted lifts. Compaction should be verified by testing.

Subgrade Preparation: In areas to receive fill not supporting structures or hardscape, the native subgrade should be scarified; moisture conditioned and compacted to at least 90% relative compaction (ASTM D 1557) for a depth of 1 foot below existing grade, or finished subgrade, whichever is deeper. Compaction should be verified by testing.

Pavement and Hardscape Area Preparation: In street, drive, permanent parking, and hardscape areas the subgrade should be over-excavated a minimum depth of one foot below existing grade or finish grade (whichever is deeper). The excavation bottom should be scarified 8 inches, moisture conditioned to near or over optimum moisture content and be recompacted to at least 90% relative compaction. Engineered fill should then be moisture conditioned, placed in suitable lifts, and compacted to a minimum of 90% relative compaction to finish grade, with the upper 1 foot compacted to at least 95% relative compaction in parking and drive areas. Compacted fill should be placed to finish subgrade elevation. Compaction should be verified by testing.

Retention Basin and Infiltrator Bottom Preparation: Compaction effort should be kept to a minimum at retention basin bottom areas and bottom areas used for any infiltrators (except under foundations). The subgrade below the bottom of basins and infiltrator bottoms should be compacted to approximately 85% relative compaction. Side slopes and any other fill or foundation subgrade should be compacted to at least 90% relative compaction. Slope construction should be per this report. Loose rock, such as pea gravel or open graded rock placed in the basin bottoms does not require compaction testing, but should be placed in lifts no greater than 2 feet and consolidated by thoroughly wetting and consolidating by passes with heavy equipment (such as a loader with full bucket or full water truck) until firm such that none to minimal deformation (less than ½ inch) occurs under the weight of passing of equipment. Basins and infiltrators are recommended to be located at least 30 feet from foundations.

Slope Construction: Please see Section 5.5 for detailed slope preparation recommendations.

All over-excavations should extend to a depth where the project geologist, engineer or his representative has deemed the exposed soils as being suitable for receiving compacted fill. The materials exposed at the bottom of excavations should be observed by a geotechnical engineer or geologist from our office prior to the placement of any compacted fill soils to verify that all old fill is removed. Additional removals may be required as a result of observation and/or testing of the exposed subgrade subsequent to the required over-excavation.

Engineered Fill Soils: The existing fill and native soils when processed appropriately are considered to be suitable for use as engineered fill. Engineered fill should be generally free from expansive soil (Expansive defined as Expansive Index (EI) greater than 20), vegetation, trash, large roots, overly wet and/or soft soil, clods larger than 3 inches, construction debris, oversized rock (greater than 6 inches) and other deleterious material as determined by the geotechnical engineer or his representative. Deleterious materials should be hauled offsite. Engineered fill soils should have a “very low” Expansion Index.

Engineered fill (and any import) should be placed in maximum 8-inch lifts (loose) and compacted to at least 90 percent relative compaction (ASTM D 1557) near its optimum moisture content prior to placement of a subsequent loose lift. Within pavement areas, the upper 12 inches of subgrade should be compacted to at least 95 percent relative compaction (ASTM D 1557). Compaction should be verified by testing. Rocks larger than 6 inches in greatest dimension should be removed from fill or backfill material.

Imported fill soils should be “very low” expansion potential granular soils meeting the USCS classifications of ML (as pre-approved by the geotechnical engineer), SM, SP-SM, or SW-SM with a maximum rock size of 3 inches and 5 to 35-percent passing the No. 200 sieve (unless otherwise approved by the geotechnical engineer). The geotechnical engineer should evaluate the import fill soils before hauling to the site. However, because of the potential variations within the borrow source, import soil will not be prequalified by Earth Systems.

A program of compaction testing, including frequency and method of test, should be developed by the project geotechnical engineer at the time of grading. Acceptable methods of testing may include Nuclear methods such as those outlined in ASTM D 6938 (Standard Test Methods for In-Place Density and Water Content of Soil and Soil-Aggregate by Nuclear Methods). Alternative methods may include methods outlined in ASTM D 1556 (Standard Test Method for Density and Unit Weight of Soil in Place by the Sand-Cone Method) or correlation probing with a hand probe.

All soils should be moisture conditioned prior to application of compactive effort and prior to foundation, slab-on-grade and pavement placement. Moisture conditioning of soils refers to adjusting the soil moisture to or just above optimum moisture content. If the soils are overly moist so that instability occurs, or if the minimum recommended compaction cannot be readily achieved, it may be necessary to aerate to dry the soil to optimum moisture content or use other means to address soft soils (as approved by the geotechnical engineer prior to use).

Shrinkage and Oversize Loss: The shrinkage factor for earthwork for the alluvial soil materials is expected to range from 10 to 20 percent for the upper excavated or scarified *site* soils based upon evaluation of 10 in-place densities (one standard deviation = 5, 95% Confidence Interval). This estimate is based on compactive effort to achieve a weighted average relative compaction of about 93 percent.

Shrinkage is highly dependent on and may vary with contractor methods for compaction. Losses from site clearing, oversize rock removal, and removal of existing site improvements, as well as the addition of excavated soil (footings, piers, etc.) may significantly affect earthwork quantity calculations and should be considered.

Dust Control: The proposed site lies within an area of high potential for wind erosion. The site soils have a fine-grained component of their composition. As such, exposed soil surfaces may be subject to disturbed fine particulate matter (PM_{10}) which can create airborne dust if the soil surface or roadways are not maintained. During construction, watering the soil surface can reduce airborne dust. Alternatively, a dust control palliative may be spray applied to the soil surface to act as a tackifier which contains loose soil particles. Palliatives must be reapplied periodically as they weather and degrade. Further guidance for dust palliatives can be found in reviewing the United States Department of Agriculture publication *Dust Palliative Selection and Application Guide*, Document No. 9977-1207-SDTDC. The recommended soil input parameters are Plasticity Index <3, and fines content 20-30 percent.

5.2 Excavations and Shoring

Excavations should be made in accordance with Cal/OSHA requirements. Using the Cal/OSHA standards and general soil information obtained from the field exploration, classification of the near surface on-site soils will likely be characterized as Type C. Actual classification of site specific soil type per Cal/OSHA specifications as they pertain to trench safety should be based on real-time observations and determinations of exposed soils by the contractors *Competent Person* (as defined by OSHA) during grading and trenching operations.

Our site exploration and knowledge of the general area indicates there is a moderate potential for caving and sloughing of site excavations (over excavation areas, utilities, footings, etc.) due to dry and also overly moist/wet conditions. Where excavations in soils over 4 feet deep are planned, lateral bracing or appropriate cut slopes of 1.5:1 (horizontal/vertical) should be provided. No surcharge loads from stockpiled soils or construction materials should be allowed within a horizontal distance measured from the top of the excavation slope and equal to the depth of the excavation. Excavations should be protected from water flow over the exposed surface and saturation.

Excavations which parallel structures, pavements, or other flatwork, should be planned so that they do not extend into a plane having a downward slope of 1:1 (horizontal: vertical) from the bottom edge of the footings, pavements, or flatwork. Shoring or other excavation techniques may be required where these recommendations cannot be satisfied due to space limitations or foundation layout. Where overexcavation will be performed adjacent to existing structures, alternating ABC slot cutting techniques may be used as pre-approved by the project geotechnical engineer. Slot cuts should have a maximum width of 5 feet.

Shoring: Shoring may be required where soil conditions, space, or other restrictions do not allow a sloped excavation or slot cutting is not an option. A braced or cantilevered shoring system may be used. Trench boxes should not be placed below or within the pipe zone elevation as their removal may loosen compacted backfill. Positive trench shoring may be required (jacks and plates).

A temporary cantilevered shoring system should be designed to resist an active earth pressure equivalent to a fluid weighing as shown in the table below. Braced or restrained excavations above the groundwater table should be designed to resist a uniform horizontal equivalent soil pressure as presented in the table below.

Table 6
Temporary Cantilevered and Braced Shoring System Parameters

Equivalent Fluid Pressure pounds per cubic foot (pcf)	
Cantilevered	**Braced**
38	58

The values provided above assume a level ground surface adjacent to the top of the shoring and do not include a factor of safety. Fifty percent of an areal surcharge placed adjacent to the shoring may be assumed to act as an additional uniform horizontal pressure against the shoring. Special cases such as combinations of slopes and shoring or other surcharge loads may require an increase in the design values recommended above. These conditions should be evaluated by the project geotechnical or shoring engineer on a case-by-case basis. Retaining walls subjected to traffic loads should include a uniform surcharge load equivalent to at least 240 psf for auto or delivery truck (2 axle) traffic kept at least 3 feet from the back of the wall. Retaining walls with closer traffic or heavier traffic loads should be designed for a 450 psf surcharge load. Retaining walls should be designed with a minimum factor of safety of 1.5.

The wall pressures above the groundwater do not include hydrostatic pressures; it is assumed that drainage will be provided. If drainage is not provided, shoring extending below the groundwater level should be evaluated on a case-by-case basis.

Cantilevered shoring must extend to a sufficient depth below the excavation bottom to provide the required lateral resistance. We recommend required embedment depths be determined using methods for evaluating sheet pile walls and based on the principles of force and moment equilibrium. For this method, the allowable passive pressure against shoring, which extends below the level of excavation, may be assumed to be equivalent to a fluid weighing 300 pcf. Additionally, we recommend a factor of safety of at least 1.2 be applied to the calculated embedment depth and that passive pressure be limited to 2,000 psf.

The contractor should be responsible for the structural design and safety of all temporary shoring systems. The contractor should carefully review the exploration logs in this report, and perform their own assessment of potential construction difficulties, and methods should be selected accordingly. Shoring should be sealed to prevent the piping of soil material and potential soil loss conditions which can cause settlement. The method of excavation and support is ultimately left to the contractor with guidance and restrictions provided by the designer and owner. We recommend that existing structures be monitored for both vertical and horizontal movement.

The method of excavation and support is ultimately left to the contractor with guidance and restrictions provided by the designer and owner. A representative from our firm should be present during grading operations to monitor site conditions; substantiate proper use of materials; evaluate compaction operations; and verify that the recommendations contained herein are met.

5.3 Utility Trenches

Backfill of utilities within roads or public rights-of-way should be placed in conformance with the requirements of the governing agency (water district, public works department, etc.). Utility trench backfill within private property should be placed in conformance with the provisions of this report. Backfill operations should be observed and tested to monitor compliance with these recommendations.

<u>Trench Width and Vertical Loads on Pipelines</u>: Vertical loads to the pipeline are highly dependent upon the geometry of the trench. In general, the narrower the trench is at the top of the pipe/conduit with respect to the diameter of the conduit, the less vertical load is applied to the conduit. This is because as the trench backfill and bedding compress or consolidate over time, the weight of the soil mass is partially offset by the frictional resistance along the trench sidewalls. In addition, the type of bedding supporting the pipeline affects the bearing strength of the conduit. This is accounted by a load factor that is multiplied to the design strength of the conduit. The pipe manufacturer recommendations for trench installation and maximum width should be followed to reduce the potential for overloading the pipe due to excess backfill load.

<u>Pipe Subgrade and Bedding</u>: Pipeline subgrade should be compacted to a minimum of 90% relative compaction (ASTM D 1557) or to a firm condition as evaluated by the geotechnical engineer or his representative for a depth of 6 inches below any bedding. Bedding material shall consist of sand 100 percent passing a No. 4 sieve and less than 5 percent fines (passing a No. 200 sieve), and a sand equivalent of 30 or more or as approved by the project inspector and geotechnical engineer. The unprocessed native soils are not typical of that used for bedding and import will be required if needed.

<u>Pipe-Zone, Trench–Zone, Trench Backfill and Compaction</u>: Backfill of utilities should be placed in conformance with the requirements of the specifications. Backfill of utilities within roads or public right-of-ways should be placed in conformance with the requirements of the governing agency (water district, public works department, etc.).

Pipe zone backfill material (the pipe area from the bedding to 12 inches above the top of pipe) may consist of native soils screened to a ¾” maximum particle size or import sand (as described above for bedding) as dictated by the pipe designer or manufacturer. The pipe zone backfill material should be placed in maximum 8-inch lifts (loose) and compacted near its optimum moisture content prior to the placement of subsequent lifts. Pipe zone backfill should be compacted to a minimum of 90% relative compaction (ASTM D 1557) or to a firm condition as evaluated by the geotechnical engineer or his representative. Compaction should be assured in the pipe haunches.

The native sandy soil is suitable for use as trench zone and street zone (and manholes) backfill (from the top of pipe zone up to finished grade), provided it is free of significant organic or deleterious matter and oversize materials. This backfill shall contain no particles larger than 3 inches in greatest dimension. The final backfill material should be placed in maximum 8-inch lifts (loose) and compacted to at least 90% relative compaction (ASTM D 1557) near its optimum moisture content for the trench zone and 95% for the street zone (upper 12 inches) where below pavement. Compaction should be verified by testing.

Backfill materials should be brought up at substantially the same rate on both sides of the pipe or conduit. Reduction of the lift thickness may be necessary to achieve the above recommended compaction. Care should be taken to not overstress the piping during compaction operations. Mechanical compaction is recommended; ponding or jetting is not recommended.

Alternatively, if the utility cannot accommodate the increased stress, or if compaction is difficult, we recommend the pipe be encased by at least 1 foot of 2-sack cement-sand slurry (at least 1 foot as measured from the top of pipe). Backfill operations should be observed and tested to monitor compliance with these recommendations.

In general, coarse-grained sand and/or gap graded gravel (i.e. ¾-inch rock or pea-gravel, etc.) should not be used for pipe or trench zone backfill due to the potential for soil migration into the relatively large void spaces present in this type of material and water seepage along trenches backfilled with coarse-grained sand and/or gravel. Gravel should be separated from backfill with a filter fabric such as Mirafi 140N or equivalent as approved by the soils engineer. Water seepage or soil migration will cause settlement of the overlying soils.

Compaction should be verified by testing. Backfill operations should be observed and tested to monitor compliance with these recommendations. Trench backfill compacted per these requirements can be expected to settle 0.1 to 0.3 percent of the trench depth. This can cause an elevation difference between backfilled trenches and the surrounding soil or pavement. Increased relative compaction can reduce settlement if the potentials presented are not acceptable. The geotechnical engineer should be consulted on a case-by-case basis to provide further recommendations to reduce the settlement potential.

STRUCTURES

In our professional opinion, structure foundations can be supported on shallow foundations bearing on a zone of properly prepared and compacted soils placed as recommended in Section 5.1. The recommendations that follow are based on “very low” expansion category soils.

5.4 Foundations

Footing design of widths, depths, and reinforcing are the responsibility of the Structural Engineer, considering the structural loading and the geotechnical parameters given in this report. A minimum footing depth of 12 to 18 inches (below lowest adjacent grade) should be maintained and considers a "very low" Expansion Index soil. Lowest adjacent grade is the lowest grade within 4 feet laterally of the footing edge. A representative of Earth Systems should observe foundation excavations to verify compaction (minimum 90% per ASTM D 1557) before placement of reinforcing steel or concrete. Loose soil or construction debris should be removed from footing excavations before placement of concrete. *All footing excavations should be probed for uniformity. Soft or loose zones should be excavated and recompacted to finish foundation bottom subgrade. The bottom of all foundations should be tested to confirm compaction effort and moisture contents as stated in Section 5.1 of this report are met.* The moisture contents should be at least the indicated moisture content 24 hours prior to and immediately prior to placing concrete for a depth of at least 12 inches below the foundation subgrade. If the moisture condition is less than indicated, it shall be brought up to or above the indicated moisture content.

Minimum Slope Setback for Foundations: Earth Systems recommends a minimum setback distance of 5 feet. The 2019 California Building Code provides setback distances for foundations along slopes. Setback distances are measured differently for foundations located above the slope and those located below the slope. For foundations located at the top of the slope, the measurement is taken horizontally from the outside face of the foundation footing to the face of the slope. For foundations located below the slope, the horizontal distance is measured from the face of the structure foundation to the toe of the slope. For slopes steeper than 1(H):1(V), please contact Earth System for these setbacks with submittal of detailed information using plan form.

Conventional Spread Foundations Tied by Grade Beams: Allowable soil bearing pressures are given below for foundations bearing on recompacted soils as described in Section 5.1. Allowable bearing pressures are net (weight of footing and soil surcharge may be neglected).

- Continuous foundations, 1 foot minimum and 2-foot maximum widths and 12-inch minimum depth below grade:

  1,500 psf for dead plus design live loads

- Pad foundations, 2 x 2-foot minimum and 3.5 x 3.5-foot maximum in plan and 18 inches below grade:

  2,000 psf for dead plus design live loads

A one-third (⅓) increase in the allowable bearing pressure may be used when calculating resistance to wind or seismic loads.

If the anticipated loads exceed the estimated values stated in Section 1.1 (24 kips for isolated footings and 3 kip/linear-ft for continuous footings), the geotechnical engineer must reevaluate the allowable bearing values as the allowable bearing was controlled by the allowable total differential settlement from seismic, collapse, and static loads. Underground utilities should be designed for an anticipated settlement within the building areas.

The spacing between any large spread footings should be evaluated by the geotechnical engineer during the plan review stage to confirm or modify the settlement estimates and bearing capacity due to large footings and the influences from adjacent footings. A preliminary analysis suggests spacing the footings (adjacent edge to adjacent edge) a lateral distance from one another of the width of the largest footing from any adjacent footing, such that influence effects are minor.

Maximum foundation sizes given above are based on settlement due to Dead + Live loads. Transient loads such as earthquake or wind loads are not subject to the stated size limitations; however, the allowable bearing pressure (including ⅓ increase) should be followed considering the relevant foundation sizes given above.

Minimum Foundation Reinforcement: Minimum reinforcement should be provided by the structural engineer to accommodate the settlement potentials presented within. Section 12.13.8.2 and 12.13.9.2.1 (as applicable) of ASCE7-16 may be considered for design of ties. Compacted fill as described within can be considered a very dense granular soil for confinement. Minimum reinforcement for continuous wall and spread footings should be four, No. 5 steel reinforcing bars, two placed near the top and two placed near the bottom of the footing. This reinforcing is not intended to supersede any structural requirements provided by the structural engineer. Foundations that use grade beam footings (minimum 12” X 12” wide) to tie isolated footings to continuous footings or other footings are recommended.

An average modulus of subgrade reaction, k, of 150 pounds per cubic inch (pci) can be used to design lightly loaded footings, beams, pavement, and slabs founded upon compacted fill. Other foundations such as mat slabs, will require the use of differing modulus of subgrade reaction values than used for lightly loaded slabs. Please contact Earth Systems for k values used for mat foundations.

The table below is based upon the above presented allowable, short term, and ultimate bearing pressures. Values may be increased by the provisions given above. Short Term allowable bearing may use the values presented below (based on Allowable Stress Design) or be based on Code mandated structural reductions, whichever is less. Ultimate bearing capacities consider a factor of safety of 3 (ASD design) to control settlement (4,500 to 6,000 psf ultimate) and a safety factor of 2.25 on transient loads (2,000 to 2,667 psf). Ultimate bearing to soil failure depends on foundation size and could be greater than 6,000 psf.

Table 7

	Allowable Bearing Capacity (psf) (FS = 3)	**Short Term (Wind/Seismic) (FS = 2.25)**	**Ultimate Bearing Capacity (FS = 1)**
Continuous Foundations	1,500	2,000	4,500
Isolated Pad Foundations	2,000	2,667	6,000

FS = Factor of Safety

Footings should be designed and reinforced by the structural engineer for the specific loading, or settlement, conditions defined herein.

Stepped foundations should be designed in accordance with the 2019 CBC. CBC 2019 and ACI Section 4.3, Table 4.3.1 should be followed for recommended cement type, water cement ratio, and compressive strength. Seismic Design Category for compressive strength determination is ‘D’. Due to the negligible sulfates in the site soils, normal cements may be used and should be proportioned in accordance with ACI recommendations considering the time of year for placement. Hot weather proportions should be used during high ambient heat days during placement and curing.

Expected Structure Settlement: Estimated total static settlement should be less than 1 inch, based on footings founded remediated or firm soils as recommended within. Differential static settlement between exterior and interior bearing members should be less than ½ inch and may be designed using a distortion angle of 1:480, which meets geotechnical guidelines per Riverside County.

Considering static and seismic settlements, we estimate a total settlement of 1 inch + 2.8 inch = 3.8 inches. Differential settlement over 40 feet should be half the total settlement (1.9 inches). Expressed as an angular distortion, this is approximately 1:250. Settlement will not result in the complete loss of soil support, but will be manifested as a tilting of the structure over the applied distance.

Settlement calculations are presented in Appendix A. The actual settlement of large spread footings should be evaluated by the geotechnical engineer during the plan review stage based on the actual column loads to confirm or modify the settlement estimates presented.

Earthquake Performance Statement

Depending upon the extent of structural and geotechnical design of foundations, exterior flatwork, walls, utilities, roadways, and other similar site improvements, some damage due to seismic events will occur. Note that all of southern California in general is in earthquake country. Site developments in southern California are typically not designed to mitigate anticipated seismic events without some damage. In fact, the Building Code is intended to provide Life-Safety performance, not complete damage-free design. In other words, some damage from earthquakes in the form of structural damage, settlement, cracking, and disruption of utilities is expected and that repair after an earthquake event will likely be required. It is not the current standard of care for site developers to fully mitigate all anticipated earthquake induced hazards. It is incumbent on the developer to advise the end-users of the project of the anticipated hazards in the form of disclosure statements during the initial and subsequent purchase processes.

According to literature from Robert W. Day, doors and windows may stick at distortion angles between 1:240 and 1:175. In this situation, a human being could be put in a life-threatening situation. Therefore, Earth Systems recommends the maximum distortion angle using all the settlement conditions including seismic settlements be 1:240. For all settlement conditions excluding seismic settlement, the structure’s maximum distortion angle should be the Riverside County’s required 1:480.

Earth Systems should review the foundation plan for conformance with this report. Distortion angles provided above are based on lengths between foundation being at least 40 feet and the foundations are not tied together. If the structural engineer ties all the foundations together, then Earth Systems can provide the individual settlements, but the structural engineer must use those settlements to determine sufficient stiffness of the foundations.

<u>Minor Deep Foundations</u>: Although no specific elements were identified by the architect, for miscellaneous structural components such as light poles, gate posts, temporary retaining walls, and flag poles, may be supported on cast-in-place piles, or direct embed in drilled holes filled with concrete, and the design be based on parameters presented in the subsequent sections of this report. Construction employing poles or posts may utilize design methods presented in Section 1807A of the CBC for Silty Sand (CL and ML) material class. For designs utilizing allowable frictional resistance, Earth Systems recommends the use of Section 1810.3.3.1.4 and .5 of the CBC. For piles with an axial load, these design methods apply for piles spaced at least 3 pile diameters center to center for axial loads as graded in accordance with Section 5.1. Piles spaced closer than these limits could have soil strength reduction and should be evaluated on a case-by-case basis by geotechnical engineer.

<u>For piers founded in areas with soil at the surface, an additional 1.5 feet should be added to the calculated pile embedment due to the potential effects of long-term surficial disturbance and erosion</u>. Additionally, where piers are constructed adjacent to the tops of slopes, there should be a minimum distance between the top of the slope and the closest edge of the pier of H/3, where 'H' is the height of the slope, otherwise a lateral resistance reduction must be applied. For piers founded closer than a distance H/3 to the crest or within the slope area itself, the calculated lateral resistance of the soil should be reduced by 30 percent. The above recommendations have considered slopes no steeper than 2:1 (horizontal:vertical). Steeper slopes will require additional analysis and may change the recommendations presented.

Drilled piers should have a minimum 3 inches of clearance between the embedded post and the soil side wall to allow for adequate placement and flow of concrete.

Drill holes may end up oversize. Casing or other means may be required in a drilled hole. Any "slough" or loose soils at the bottom of the shaft must be removed or tamped prior to setting rebar cages and placing concrete. Extreme care must be exercised to carefully position reinforcing steel cages and place concrete without disturbing the sidewalls of the drilled shafts. We recommend centralizers be used to positively locate rebar cages within the pier shaft. It is recommended that pier excavations that have not received concrete, not be left open and concrete should be placed immediately. Caving is a very high concern.

Normally, drilled pier excavations should be made without the use of water. If necessary, water may be used to facilitate removal of cuttings unless it aggravates caving problems. Added water that may accumulate at the bottom of the hole should be removed from the drilled hole prior to placing the concrete. Sidewalls which have softened from the addition of water should be cleaned of the soft/loose material. Each excavation should be completed in a continuous operation and the concrete should be placed without undue delay. The contractor should use appropriate means to clean the bottom of the excavation so that no loose material is present at the base of the pier. We do not recommend overdrilling beyond specified pier tip elevations to eliminate the need for bottom cleaning in order to account for slough or loose materials at the excavation bottom. To reduce the potential for caving and sidewall sloughing which may contaminate concrete during placement, and segregation, concrete should be placed by tremie methods and not directly chute-dumped into the hole.

Where casing is used with drilled holes and cannot be withdrawn, the skin friction capacity is theoretically reduced, as are passive resistance and stiffness. The amount of reduction is subject to assessment by the geotechnical consultant. The use of casing with drilled holes should be approved prior to use by the geotechnical engineer.

If casing is required, it should be withdrawn as the concrete is being placed, maintaining a 3-foot minimum head of concrete within the casing. This is to prevent reduction in the diameter of the drilled shaft due to earth pressure on the fresh concrete and to prevent extraneous material from falling in from the sides and mixing with the concrete. Concrete placement should continue in this manner until suitable concrete extends to the top of the excavation or forms. The upper eight feet of the pier should be consolidated by vibratory means.

Pier capacity is greatly dependent on the soil conditions at the location of the pier and upon contractor means and methods of placement. It is recommended that drilling operations and concrete placement be performed in the continuous presence of the geotechnical consultant or his representative to confirm that suitable materials for pier support are penetrated, that the dimensions of the installed piers meet the design dimensions, and that the installation has been performed as specified by the 2019 California Building Code. Observation during drilling is required by the 2019 California Building Code on a full-time basis by the geotechnical engineer or his representative. If subsurface conditions noted during drilled pier installation are significantly different than those encountered in our borings, it may be necessary to adjust the overall length of the pier.

Prior to the placement of steel, and again prior to and during the placement of concrete, the excavation must be examined by the geotechnical consultant before proceeding with construction. The contractor should provide all aid and assistance required by the geotechnical and geologic consultants for field monitoring of the drilled pier operations.

Piers are accepted or rejected based on visual observation and testing during construction. The contractor should not allow nor cause any of this work to be permanently enclosed or covered up until it has been observed, tested, and accepted by the geotechnical engineer and all legally constituted authorities having jurisdiction.

5.5 Slope Construction

Slopes are not generally proposed for this project; however, minor slopes (less than 5 feet in height) may be constructed. Compacted fill slopes protected against erosion (per approved methods such as significant planting, facing, or erosion blankets, etc.) may be constructed at 2:1 (horizontal: vertical) or flatter inclinations. Unprotected slopes with exposed soils or compacted fill at the surface should be expected to require repair after heavy nuisance or storm runoff due to significant erosion. Slope recommendations may change pending a more in-depth geotechnical evaluation once design plans are developed. Slopes used as nuisance or storm drainage channel slopes should be no steeper than 3:1 and protected with heavy 12" minimum Rip-Rap.

Compacted fill should be placed at near optimum moisture content and compacted to a minimum 90 percent of the maximum dry unit weight, as measured in relation to ASTM D 1557 test procedures. The exposed face of any cut or fill slope (upper 12 inches) should have a minimum relative compaction of 90 percent, as measured in relation to ASTM D 1557 test procedures, and be compacted at near optimum moisture content. Due to the erodible site soils, slope faces should be protected with facing or densely spaced vegetation to reduce the erosion potential.

Surficial Slope Failures: All slopes will be exposed to weathering, resulting in decomposition of surficial earth materials, thus potentially reducing shear strength properties of the surficial soils. In addition, these slopes become increasingly susceptible to rodent burrowing. As these slopes deteriorate, they can be expected to become susceptible to surficial instability such as soil slumps, erosion, soil creep, and debris flows. Development areas immediately adjacent to ascending or descending slopes should address future surficial sloughing of soil material and erosion. Such measures may include debris fences, slope facing, catchment areas or walls, diversion ditches or berms, soil planting, velocity reducers or other techniques to contain soil material away from developed areas and reduce erosion. Additionally, foundations should be set back at least 5 feet from the edge of slope or as per the 2019 CBC, whichever is greater.

Operation and maintenance inspections should be done after a significant rainfall event and on a time-based criteria (annually or less) to evaluate distress such as erosion, slope condition, rodent infestation burrows, etc. Inspections should be recorded and photographs taken to document current conditions. The repair procedure should outline a plan for fixing and maintaining surficial slope failures, erosional areas, gullies, animal burrows, etc. Repair methods could consist of excavating and infilling with compacted soil erosional features, track walking the slope faces with heavy equipment, as determined by the type and size of repair. These repairs should be performed in a prompt manner after their occurrence. Slope inclinations should be maintained and a maintenance program should include identifying areas where slopes begin to steepen. Where future maintenance is not possible, slopes should be faced to reduce the erosion and degradation potential.

Slope faces are highly erodible even if compacted and will gradually erode and move down slope presenting maintenance issues and debris deposited in drainage devices and flatwork areas. The minimum material necessary to support landscaping should be specified by the landscape consultant (typically less than 6 inches).

5.6 Slabs-on-Grade

Subgrade: Concrete slabs-on-grade and flatwork should be supported by compacted and moisture conditioned soil placed in accordance with Section 5.1 of this report. The moisture content below slabs should be at least optimum moisture content 24 hours prior to and immediately prior to placing concrete for a depth 12 inches. If the moisture condition is less than indicated, it shall be brought up to or above the indicated moisture content.

Vapor Retarder: In areas of moisture-sensitive floor coverings, coatings, adhesives, underlayment, goods or equipment stored in direct contact with the top of the slab, bare slabs, humidity controlled environments, or climate-controlled cooled environments, an appropriate vapor retarder that maintains a permeance of 0.01 perms or less after ASTM E1745's mandatory conditioning tests should be installed to reduce moisture transmission from the subgrade soil to the slab. For these areas, a vapor retarder (Stego wrap 15-mil thickness or equal) should underlie the floor slabs. If a Class A vapor retarder (ASTM E 1745) is specified, the retarder can be placed directly on non-expansive soil, and be covered with a minimum 2 inches of clean sand.

Clean sand is defined as well or poorly-graded sand (ASTM D 2488) of which less than 5 percent passes the No. 200 sieve and all the material passes a No. 4 sieve. The site soils do not fulfill the criteria to be considered clean sand. Alternatively, the slab designer may consider the use of other vapor retarder systems that are recommended by the American Concrete Institute.

Low-slump concrete should be used to help reduce the potential for concrete shrinkage. The effectiveness of the membrane is dependent upon its quality, the method of overlapping, its protection during construction, the successful sealing of the membrane around utility lines, and sealing the membrane at perimeter terminations and of all penetrations. Capillary breaks, if any, beneath slabs should consist of a minimum of at least four inches of permeable base material (Caltrans) with the following specified gradation.

Table 8
Percent Passing Sieve Size

Sieve Size	**Percent Passing**
1 inch	100
¾ Inch	90-100
3/8 Inch	40-100
#4	25-40
#8	18-33
#30	5-15
#50	0-7
#200	0-3

Where vapor retarders are placed directly on a gravel capillary break, they should be a minimum of 15 mil thickness.

Where concrete is placed directly on the vapor retarder “plastic”, proper curing techniques are essential to minimizing the potential of slab edge curl and shrinkage cracking. The edges of slabs can curl upward because of differential shrinkage when the top of the slab dries to lower moisture content than the bottom of the slab. Curling and cracking are caused by the difference in drying shrinkage between the top and bottom of the slab. Curling and cracking can be exacerbated by hot weather, or dry condition concrete placement, even with proper curing techniques.

The following minimum slab recommendations are intended to address geotechnical concerns such as potential variations of the subgrade and are not to be construed as superseding any structural design. A design engineer should be retained to provide building specific systems to

handle subgrade moisture to ensure compliance with SB800 with regards to moisture and moisture vapor.

Slab Thickness and Reinforcement: Structure slabs should be a minimum of 4 inches in actual thickness and be reinforced with # 3 bars at 18 inches on center both ways. Slabs in contact with earth should use closer joints to control cracking or be thickened to allow adequate earth to rebar clearance. Reinforcing bars should extend at least 40 bar diameters into the footings and slabs. Concrete slabs-on-grade and flatwork should be supported by compacted and moisture conditioned soil placed in accordance with this report. If slabs are structural, they should be designed for the specific settlement conditions presented within.

Slab thickness and reinforcement of slabs-on-grade are contingent on the recommendations of the structural engineer or architect and the Expansion Index of the supporting soil. Based upon our findings, a modulus of subgrade reaction of approximately 150 pounds per cubic inch can be used in concrete lightly loaded (not mat) slab design for the expected compacted subgrade. Mat slab design will require differing modulus values. ACI Section 4.3, Table 4.3.1 should be followed for recommended cement type, water cement ratio, and compressive strength.

If heavily loaded flatwork is proposed (forklift drive areas, heavy racking, etc.), the actual thickness should be designed by the structural engineer utilizing techniques of the American Concrete Institute (ACI) and may be greater than 4 inches in thickness. Concrete floor slabs may either be monolithically placed with the foundations or doweled (No. 4 bar embedded at least 40 bar diameters) after footing placement. The thickness and reinforcing given are not intended to supersede any structural requirements provided by the structural engineer. The project architect or concrete inspector should continually observe all reinforcing steel in slabs during placement of concrete to check for proper location within the slab. The minimum concrete rebar cover should be as per the project architect or structural engineer.

Slab-On-Grade Control Joints: Control joints should be provided in all regular concrete slabs-on-grade at a maximum spacing of 26 to 36 times the slab thickness (12 feet maximum on-center each way, 4 to 6 feet for sidewalks) as recommended by American Concrete Institute [ACI] guidelines. All joints should form approximately square patterns to reduce the potential for randomly oriented shrinkage cracks. Control joints in the slabs should be tooled at the time of the concrete placement or saw cut (¼ of slab depth) as soon as practical but not more than 8 hours from concrete placement.

Construction (cold) joints should consist of thickened butt joints with ¾-inch dowels at 18 inches on center embedded per ACI or a thickened keyed-joint to resist vertical deflection at the joint. All control joints in exterior flatwork should be sealed to reduce the potential of moisture or foreign material intrusion. These procedures will reduce the potential for randomly oriented cracks, but may not prevent them from occurring.

Curing and Quality Control: The contractor should take precautions to reduce the potential of curling and cracking of slabs in this arid desert region using proper batching, placement, and curing methods. Curing is highly affected by temperature, wind, and humidity.

Quality control procedures should be used, including trial batch mix designs, batch plant inspection, and on-site special inspection and testing. Curing should be in accordance with ACI recommendations contained in ACI 211, 304, 305, 308, 309, and 318. Additionally, the concrete should be vibrated during placement. Concrete should be wet cured for at least 7 days with burlap or plastic and not allowed to dry out to minimize surface cracking.

5.7 Seismic Design Criteria

This site is subject to strong ground shaking due to potential fault movements along regional faults including the San Jacinto and San Andreas fault zones. Engineered design and earthquake-resistant construction increase safety and allow development of seismic areas. The minimum seismic design should comply with the 2019 edition of the California Building Code and ASCE 7-16 using the seismic coefficients given in the table below. General Procedure seismic parameters are presented below per ASCE7-16 exception, considering a Site Class D (based on Vs shear wave velocity) for structures not greater than 0.5 seconds in period. Structures greater than 0.5 seconds in period will require a Site Specific Seismic evaluation and the values presented below are not valid (ASCE7-16, Section 20.3.1). For foundations described within, site soils are not subject to bearing failure.

2019 CBC (ASCE 7-16) Seismic Parameters

Seismic Risk Category:	II
Site Class:	D
Maximum Considered Earthquake [MCE] Ground Motion	
Short Period Spectral Response S_s:	1.500 g
1 second Spectral Response, S_1:	0.600 g
Code Design Earthquake Ground Motion	
Short Period Spectral Response, S_{DS}	1.000 g
1 second Spectral Response, S_{D1}	0.680 g
Peak Ground Acceleration (PGA_M)	0.63 g

The intent of the CBC lateral force requirements is to provide a structural design that will resist collapse to provide reasonable life safety from a major earthquake but may experience some structural and nonstructural damage. A fundamental tenet of seismic design is that inelastic yielding is allowed to adapt to the seismic demand on the structure. In other words, *damage is allowed*. The CBC lateral force requirements should be considered a *minimum* design. The owner and the designer may evaluate the level of risk and performance that is acceptable. Performance based criteria could be set in the design. The design engineer should exercise special care so that all components of the design are fully met with attention to providing a continuous load path. An adequate quality assurance and control program is urged during project construction to verify that the design plans and good construction practices are followed. This is especially important for sites lying close to the major seismic sources.

Estimated peak horizontal site accelerations are based upon a probabilistic analysis (2 percent probability of occurrence in 50 years) is approximately 0.8 g for a stiff soil site. Actual accelerations may be more or less than estimated. Vertical accelerations are typically ⅓ to ⅔ of the horizontal accelerations, but can equal or exceed the horizontal accelerations, depending upon the local site effects and amplification.

5.8 Driveways and Parking Areas

Pavement structural sections for associated drive areas including recommendations for standard asphalt concrete, and Portland cement concrete are provided below and are based upon on-site soils as described in Section 5.1. Soils differing from those described will require differing pavement sections. The appropriate pavement section depends primarily on the shear strength of the subgrade soil exposed after grading in the near finished subgrade elevation and the anticipated traffic over the useful life of the pavement. R-value testing or observation of subgrade soils should be performed of near finished subgrade elevation soils to verify and/or modify the preliminary pavement sections presented within this report.

Pavement Area Preparation: In street, drive, and parking areas, the exposed subgrade should be overexcavated as recommended in Section 5.1, moisture conditioned, and compacted. Compaction should be verified by testing. Aggregate base should be compacted to a minimum 95% relative compaction (ASTM D 1557).

Automobile Traffic and Parking Areas: Pavement sections presented in the following table for automobile type traffic areas and are based on an assumed R-value and current Caltrans design procedures. Traffic Indices (TI) of 5 and 7 were used to facilitate the design of asphalt concrete pavements for parking and main drives, including fire lanes. The fire lane calculation assumed a conservative traffic flow of one fire truck per day entering and exiting the site on the same path (20 year life cycle), and a maximum loading of an 88,000 lb Tandem Axle apparatus (approximate 12,000 lb front axle load and two 34,000 lb rear axles loads) which is based upon the *Emergency Vehicle Size and Weight Regulation Guideline*, dated November 22, 2011, prepared by the Fire Apparatus Manufacturers' Association.

Based on the above stated traffic pattern and apparatus loads, a Traffic Index of 4.6 is calculated for fire lanes. For comparison, a 40 year fire lane life cycle analysis results in a Traffic Index of 5. The TI's assumed below should be reviewed by the project Civil Engineer to evaluate the suitability for this project. All design should be based upon an appropriately selected traffic index. Changes in the traffic indices will affect the corresponding pavement section.

Table 9
Preliminary Flexible Pavement Section Recommendations
On-site/Interior Automobile Drive Areas

R-Value of Subgrade Soils - 45 (Assumed) Design Method – CALTRANS

Traffic Index (Assumed)*	Pavement Use	Flexible Pavements**	
		Asphaltic Concrete Thickness (inches)	Aggregate Base Thickness (inches)
5	Parking Areas & Fire Lanes***	3.0	4.0
7	Main Drive Areas	4.0	5.5

*The presented Traffic Indices should be confirmed by the project civil engineer. Changes to the Traffic Index will result in a differing pavement section required.

**Pavement Sections were calculated using Caltrans software CalFP Version 1.5.

***Where fire lanes will be a part of a main drive use with other traffic, busses, or trucks, the Main Drive Area pavement section should be used.

- Subsequent to utility installation, the entire pavement (including PCC) final subgrade should be scarified 12 inches, moisture conditioned to near optimum moisture content, and compacted to a minimum 95% relative compaction immediately prior (within a few days) to the placement and compaction of aggregate base to re-establish proper moisture content and compaction in site soils.
- Subgrade soils and aggregate base should be in a stable, non-pumping condition at the time of placement and compaction. Exposed subgrades should be proof-rolled to verify the absence of soft or unstable zones.
- Aggregate base materials should be compacted at near optimum moisture content to at least 95 percent relative compaction (ASTM D 1557) and should conform to Caltrans Class II criteria. Standard Specifications for Public Works Construction "Greenbook: standards (Crushed Aggregate Base Class) may be used in lieu of Caltrans. Compaction efforts should include rubber tire proof-rolling of the aggregate base with heavy compaction-specific equipment (i.e. fully loaded water trucks).
- All concrete curbs separating pavement from landscaped areas should extend at least 6 inches into the subgrade soils to reduce the potential for movement of moisture into the aggregate base layer (this reduces the risk of pavement failures due to subsurface water originating from landscaped areas).
- Asphaltic concrete should be ½-in. or ¾-in. grading and compacted to a minimum of 95% of the 75-below Marshall density (ASTM D 1559) or equivalent.
- Within the structural pavement section areas, positive drainage (both surface and subsurface) should be provided. In no instance should water be allowed to pond on the pavement. Roadway performance depends greatly on how well runoff water drains from the site. This drainage should be maintained both during construction and over the entire life of the project.

- Proper methods, such as hot-sealing or caulking, should be employed to limit water infiltration into the pavement base course and/or subgrade at construction/expansion joints and/or between existing and reconstructed asphalt concrete sections (if any). Water infiltration could lead to premature pavement failure.
- To reduce the potential for detrimental settlement, excess soil material, and/or fill material removed during any footing or utility trench excavation, should not be spread or placed over compacted finished grade soils unless subsequently compacted to at least 90% of the maximum dry unit weight, as evaluated by ASTM D 1557 test procedure, at near optimum moisture content, or 95% if placed under areas designated for pavement.
- Where new roadways will be installed against existing roadways, the repaired asphalt concrete pavement section should be designed and constructed to have at least the pavement and aggregate base section as the original pavement section thickness (for both AC and base) or upon the newly calculated pavement sections presented within, whichever is greater.
- Pavement designs assume that heavy construction traffic will not be allowed on base cap or finished pavement sections.

5.9 Surface and Subsurface Site Drainage and Maintenance

Positive drainage should be maintained away from the structures and underside of structures (5 percent for 10 feet minimum) to prevent ponding and subsequent saturation of the foundation soils. Gutters and downspouts in conjunction with a 1 to 2% hardscape grade can be considered as a means to convey water away from foundations if increased fall is not provided. Drainage should be maintained for paved areas. Water should not pond on or near paved areas or foundations. Ponded water can saturate subgrade soils and lead to pavement failure. The following recommendations are provided in regard to site drainage and structure performance:

- Water control and conveyance is a critical aspect of project design. It is highly recommended that landscape irrigation or other sources of water be collected and conducted to an approved drainage device. Landscaping grades should be lowered and sloped such that water drains to appropriate collection and disposal areas. All runoff water should be controlled, collected, and drained into proper drain outlets. Control methods may include curbing, ribbon gutters, 'V' ditches, or other suitable containment and redirection devices.
- It is highly recommended that landscape irrigation or other sources of water be collected and conducted to an approved drainage device. Site drainage should be devised such that runoff should be directed away from the tops of all graded slopes. Water should not freely flow over slopes or retaining wall faces. Diversion and conveyance structures which can accommodate water and eroded soil should be constructed at the tops and toes of all slopes. Lined swales at the top and bottom of slopes, and at the top of retaining walls are recommended.
- In no instance should water be allowed to flow or pond against structures, slabs or foundations or flow over unprotected slope faces. Adequate provisions should be employed to control and limit moisture changes in the subgrade beneath foundations or structures to reduce the potential for soil saturation. Landscape borders should not act as traps for water within landscape areas. Potential sources of water such as piping, drains, over-spray broken

sprinklers, etc, should be frequently examined. Any such leakage, over-spray, or plugging should be immediately repaired.

- Maintenance of drainage systems and infiltration structures can be the most critical element in determining the success of a design. They must be protected and maintained from sediment-laden water both during and after construction to prevent clogging of the surficial soils any filter medium. The potential for clogging can be reduced by pre-treating structure inflow through the installation of maintainable forebays, biofilters, or sedimentation chambers. In addition, sediment, leaves, and debris must be removed from inlets and traps on a regular basis. Since these and other factors (such as varying soil conditions) may affect the rate of water infiltration, it is imperative to apply a conservative factor of safety [FOS] to unfactored Basic Percolation/Infiltration Rates to provide a reliable basis for design. In order to account not only for the unknown factors above but also for changes of conditions during the use of the structures such as potential clogging effects due to washing in of soil fines, a FOS between 3 and 10 should be applied to lower infiltration rates.
- The factor of safety should be selected by the project drainage engineer and may be dependent on agency guidelines and the presence of testing, filters, and sedimentation structures. If these measures are provided, the factor of safety can be reduced.
- The drainage pattern should be established at the time of final grading and maintained throughout the life of the project. Additionally, drainage structures should be maintained (including the de-clogging of piping, basin bottom scarification, soil crust removal, etc.) throughout their design life. Maintenance of these structures should be incorporated into the facility operation and maintenance manual. Structural performance is dependent on many drainage-related factors such as landscaping, irrigation, lateral drainage patterns and other improvements.
- Infiltrating structures (basins, infiltrators, trenches, dry wells, seepage pits, leach lines, etc.) should be located at least 30 feet from settlement sensitive structures.

Section 6
LIMITATIONS AND ADDITIONAL SERVICES

6.1 Uniformity of Conditions and Limitations

Our findings and recommendations in this report are based on selected points of field exploration, laboratory testing, and our understanding of the proposed project. Furthermore, our findings and recommendations are based on the assumption that soil conditions do not vary significantly from those found at specific exploratory locations. Variations in soil or groundwater conditions could exist between and beyond the exploration points. The nature and extent of these variations may not become evident until construction. Variations in soil or groundwater may require additional studies, consultation, and possible revisions to our recommendations.

The planning and construction process is an integral design component with respect to the geotechnical aspects of this project. Because geotechnical engineering is an inexact science due to the variability of natural processes and because we sample only a small portion of the soil and material affecting the performance of the proposed structure, unanticipated or changed conditions can be disclosed during demolition and construction. Proper geotechnical observation and testing during construction is imperative to allow the geotechnical engineer the opportunity to verify assumptions made during the design process and to verify that our geotechnical recommendations have been properly interpreted and implemented during construction. Therefore, we recommend that Earth Systems be retained during the construction of the proposed improvements to observe compliance with the design concepts and geotechnical recommendations, and to allow design changes in the event that subsurface conditions or methods of construction differ from those assumed while completing this study. If we are not accorded the privilege of performing this review, we can assume no responsibility for misinterpretation or the applicability of our recommendations. The above services can be provided in accordance with our current Fee Schedule.

Our evaluation of subsurface conditions at the site has considered subgrade soil and groundwater conditions present at the time of our study. The influence(s) of post-construction changes to these conditions such as introduction or removal of water into or from the subsurface will likely influence future performance of the proposed project. It should be recognized that definition and evaluation of subsurface conditions are difficult. Judgments leading to conclusions and recommendations are generally made with incomplete knowledge of the subsurface conditions due to the limitation of data from field studies. The availability and broadening of knowledge and professional standards applicable to engineering services are continually evolving. As such, our services are intended to provide the Client with a source of professional advice, opinions and recommendations based on the information available as applicable to the project location and scope. If the scope of the proposed construction changes from that described in this report, the conclusions and recommendations contained in this report are not considered valid unless the changes are reviewed, and the conclusions of this report are modified or approved in writing by Earth Systems.

Findings of this report are valid as of the issued date of the report. However, changes in conditions of a property can occur with passage of time, whether they are from natural processes or works of man, on this or adjoining properties. In addition, changes in applicable standards occur, whether they result from legislation or broadening of knowledge. Accordingly, findings of this report may be invalidated wholly or partially by changes outside our control. Therefore, this report is subject to review and should not be relied upon after a period of one year.

This report is issued with the understanding that the owner or the owner's representative has the responsibility to bring the information and recommendations contained herein to the attention of the architect and engineers for the project so that they are incorporated into the plans and specifications for the project. The owner or the owner's representative also has the responsibility to verify that the general contractor and all subcontractors follow such recommendations. It is further understood that the owner or the owner's representative is responsible for submittal of this report to the appropriate governing agencies.

Earth Systems has striven to provide our services in accordance with generally accepted geotechnical engineering practices in this locality at this time. No warranty or guarantee, express or implied, is made. This report was prepared for the exclusive use of the Client and the Client's authorized agents.

Earth Systems should be provided the opportunity for a general review of final design and specifications in order that earthwork and foundation recommendations may be properly interpreted and implemented in the design and specifications. If Earth Systems is not accorded the privilege of making this recommended review, we can assume no responsibility for misinterpretation of our recommendations. The owner or the owner's representative has the responsibility to provide the final plans requiring review to Earth Systems' attention so that we may perform our review.

Any party other than the client who wishes to use this report shall notify Earth Systems of such intended use. Based on the intended use of the report, Earth Systems may require that additional work be performed and that an updated report be issued. Non-compliance with any of these requirements by the client or anyone else will release Earth Systems from any liability resulting from the use of this report by any unauthorized party.

In addition, if there are any changes in the field to the plans and specifications, the Client must obtain written approval from Earth Systems' engineer that such changes do not affect our recommendations. Failure to do so will vitiate Earth Systems' recommendations.

Although available through Earth Systems, the current scope of our services does not include an environmental assessment or an investigation for the presence or absence of wetlands, hazardous or toxic materials in the soil, surface water, groundwater, or air on, below, or adjacent to the subject property.

6.2 Additional Services

This report is based on the assumption that an adequate program of client consultation, construction monitoring, and testing will be performed during the final design and construction phases to check compliance with these recommendations. Maintaining Earth Systems as the geotechnical consultant from beginning to end of the project will provide continuity of services.

The geotechnical engineering firm providing tests and observations shall assume the responsibility of Geotechnical Engineer of Record.

Construction monitoring and testing would be additional services provided by our firm. The costs of these services are not included in our present fee arrangements, but can be obtained from our office. The recommended review, tests, and observations include, but are not necessarily limited to, the following:

- Consultation during the final design stages of the project;
- A review of the building and grading plans to observe that recommendations of our report have been properly implemented into the design;
- Observation and testing during site preparation, grading, and placement of engineered fill as required by CBC Sections 17 and Appendix J or local grading ordinances;
- Consultation as needed during construction.

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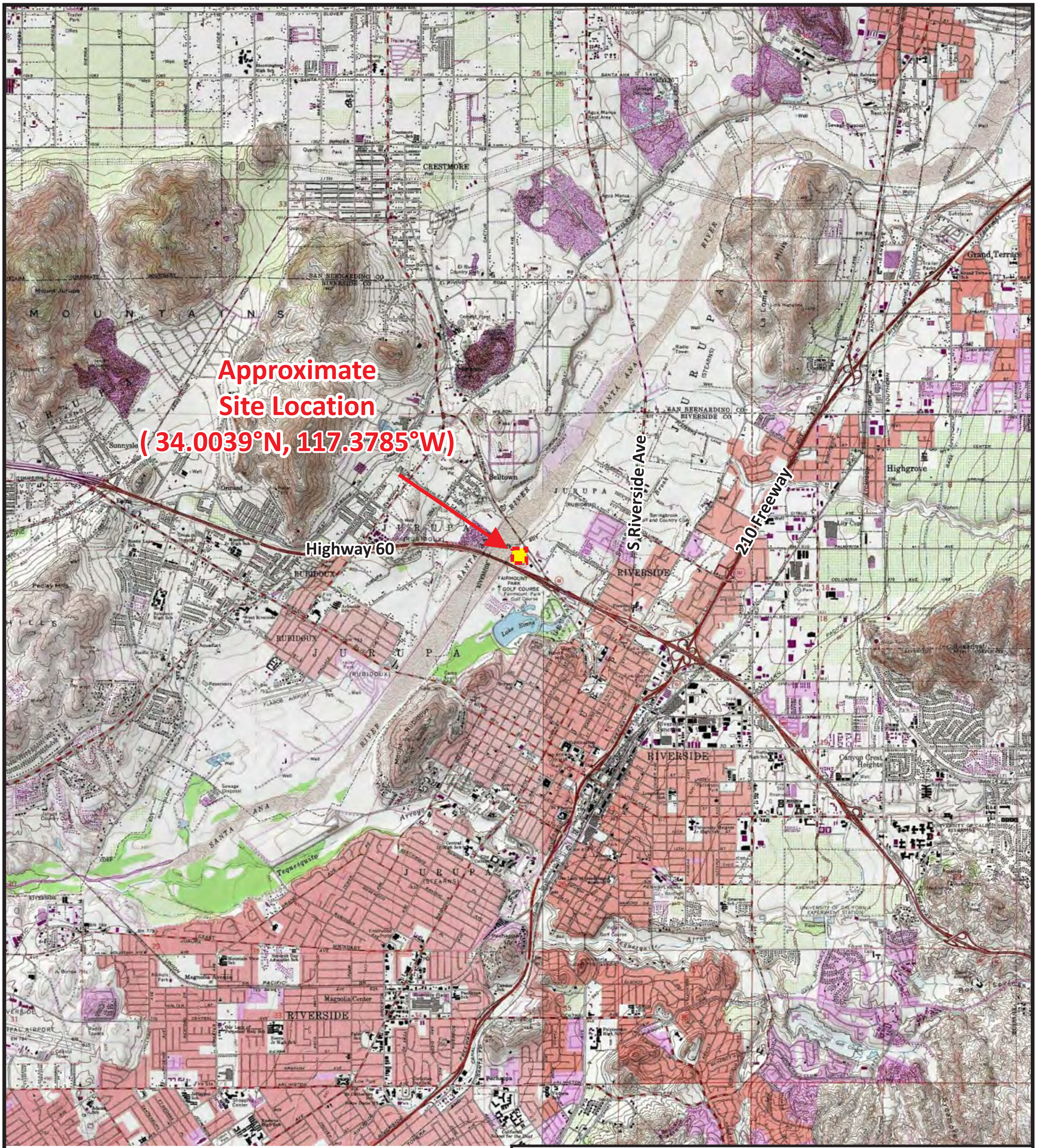
Aerial Photographs:

Google Earth: 1993-2020

Historic Aerials: 1938-2016

APPENDIX A

Plate 1 – Site Vicinity Map
Plate 2 – Boring Location Map
Plate 3 – Regional Geologic Map
Table A-1 Fault Parameters
Table A-2 Historic Earthquakes
2019 CBC Seismic Design Paramters
Terms and Symbols Used on Boring Logs
Soil Classification System
Log of Borings (4 pages)
Site Class Estimator (1 page)
Liquefaction and Dry Seismic Settlement (2 pages)
Continuous Footing Settlement (1 page)
Spread Footing Settlement (1 page)



Source: Google Earth satellite image with USGS topographic map overlay.

LEGEND

Approximate Site Location

Approximate Scale: 1" = 1 Mile

0 1 Mile 2 Miles

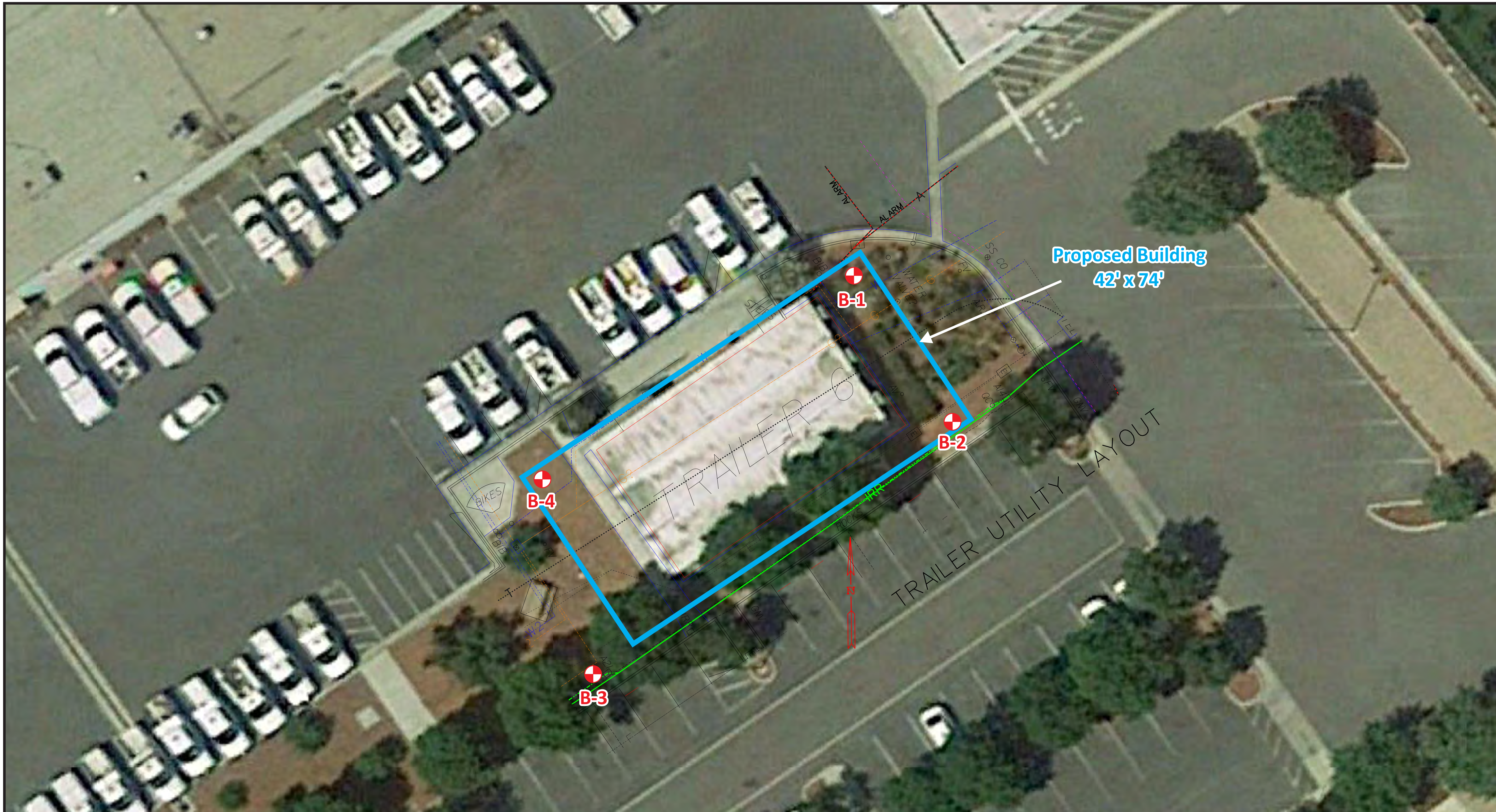
N

Plate 1
Site Vicinity Map

Proposed Building 6 Modular Upgrade
1995 Market Street
Riverside, Riverside County, California

Earth Systems

11/30/2020	File No.: 302451-002



N

Approximate Scale: 1" = 20'

0' 20' 40'

Boring Coordinates	
#	Lat_Lon_NAD83
B-1	34.00402N,117.37839W
B-2	34.00393N,117.37832W
B-3	34.00377N,117.37858W
B-4	34.0039N,117.37862W

LEGEND

B-4 Approximate Exploration Locations

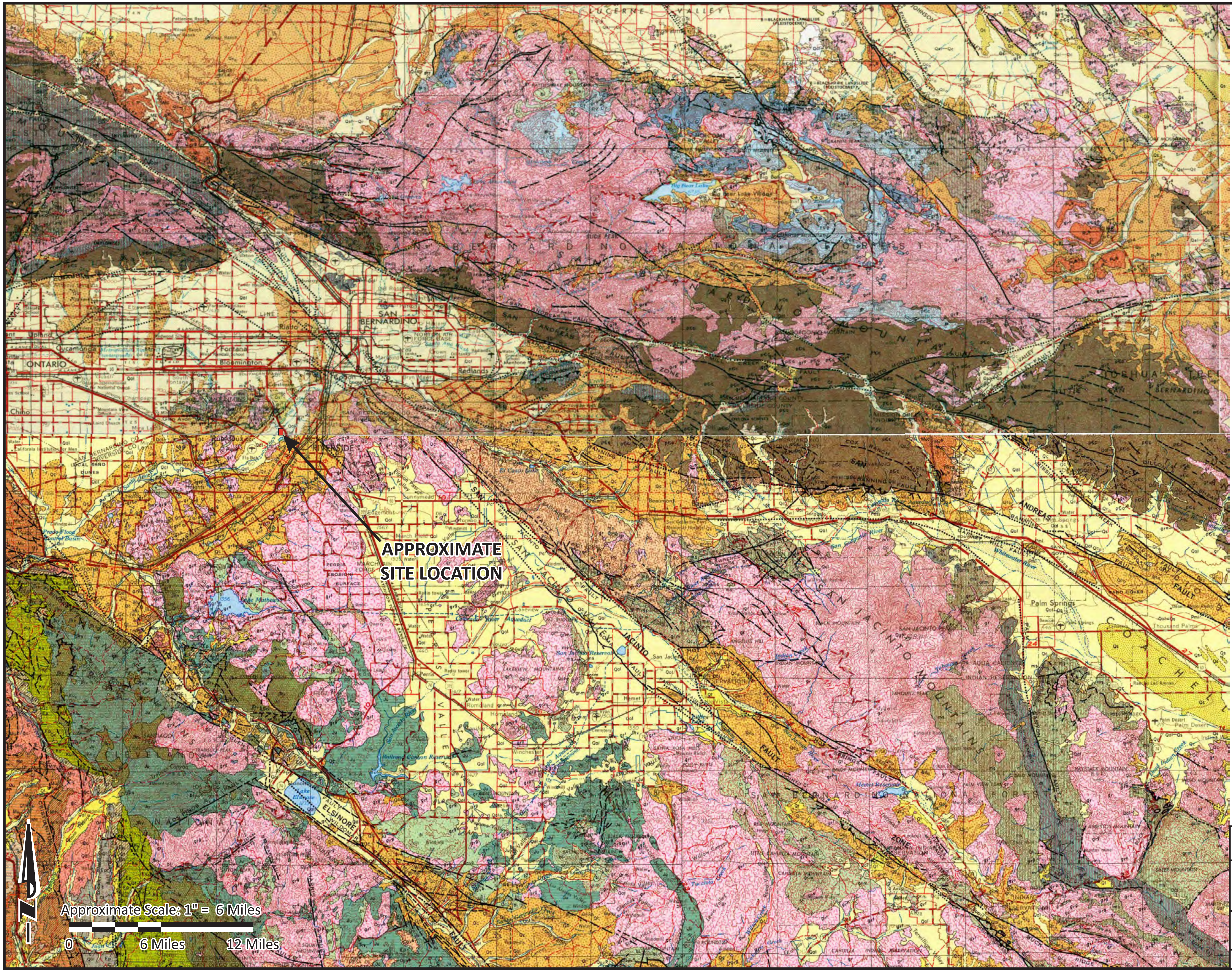
Plate 2
Exploration Location Map

Proposed Building 6 Modular Upgrade
1995 Market Street
Riverside, Riverside County, California

Earth Systems

11/30/2020 File No.: 302451-002

Source: Google Earth satellite image dated 8/24/2018 with Utility Map Overlay.



LEGEND

Qs	Dune sand
Qal	Alluvium
Ql	Lake deposits
Qg	Glacial deposits
Qt	River terrace deposits
Qm	Pleistocene marine and marine terrace deposits
Qpv	Pleistocene volcanic rocks Qpv^r-rhyolite Qpv^a-andesite Qpv^b-basalt Qpv^p-pyroclastic rocks
Qc	Pleistocene nonmarine
QP	Plio-Pleistocene nonmarine
✳	Quaternary and/or Pliocene cinder cones
Pc	Undivided Pliocene nonmarine
Pu	Upper Pliocene marine
Pmlc	Middle and/or lower Pliocene nonmarine
Pml	Middle and/or lower Pliocene marine
Pv	Pliocene volcanic rocks Pv^r-rhyolite Pv^a-andesite Pv^b-basalt Pv^p-pyroclastic rocks
Mc	Undivided Miocene nonmarine
Muc	Upper Miocene nonmarine
Mu	Upper Miocene marine
Mm	Middle Miocene marine
Ml	Lower Miocene marine
Mv	Miocene volcanic rocks Mv^r-rhyolite Mv^a-andesite Mv^b-basalt Mv^p-pyroclastic rocks

Φc	Oligocene nonmarine
Ec	Eocene nonmarine
E	Eocene marine
Ep	Paleocene marine
Tc	Tertiary nonmarine
Tm	Tertiary marine
Ti	Tertiary intrusive (hypabyssal) rocks Ti^r-rhyolite Ti^a-andesite Ti^b-basalt
Tl	Tertiary lake deposits
Tv	Tertiary volcanic rocks Tv^r-rhyolite Tv^a-andesite Tv^b-basalt Tv^p-pyroclastic rocks
Ku	Upper Cretaceous marine
Ju	Upper Jurasssic marine
gr	Mesozoic granitic rocks
bi	Mesozoic basic intrusive rocks
ub	Mesozoic ultrabasic intrusive rocks
JR̄v	Jurassic-Triassic metavolcanic rocks
m ls	Pre-Cretaceous metamorphic rocks (ls=limestone)
ms	Pre-Cretaceous metasedimentary rocks
mv	Pre-Cretaceous metavolcanic rocks
gr-m	Pre-Cenozoic granitic and metamorphic rocks
pЄc	Precambrian igneous and metamorphic rock complex
pЄ	Undivided Precambrian metamorphic rocks pCg=gneiss pCs=schist pCls=limestone and/or dolomite
pЄgr	Undivided Precambrian granitic rocks

Source: CGS Geologic Map of California, Santa Ana & San Bernardino

Plate 3
Regional Geologic Map

Proposed Building 6 Modular Upgrade
1995 Market Street
Riverside, Riverside County, California

Earth Systems

11/30/2020	File No.: 302451-002

Table A-1
Fault Parameters

Fault Section Name	Distance (miles)	Distance (km)	Upper Seis. Depth (km)	Lower Seis. Depth (km)	Avg Dip Angle (deg.)	Avg Dip Direction (deg.)	Avg Rake (deg.)	Trace Length (km)	Fault Type	Mean Mag	Mean Return Interval (years)	Slip Rate (mm/yr)
San Jacinto (San Bernardino) FM3.1, 3.2	5.7	9.2	0.0	16.1	90	225	180	37	A	**7.6**	219	17
San Jacinto (Lytle Creek connector) FM3.1, 3.2	7.4	11.9	0.0	16.1	90	na	na	23	B'	**6.7**		
Fontana (Seismicity) FM3.1, 3.2	7.9	12.7	0.0	16.3	80	313	na	24	B'	**6.7**		
San Jacinto (San Jacinto Valley) rev FM3.1, 3.2	8.1	13.1	0.0	16.1	90	223	180	18	A	**7.6**	219	18
Cucamonga FM3.1, 3.2	12.5	20.1	0.0	7.8	45	347	90	28	B	**6.6**		5
San Andreas (San Bernardino N) FM3.1, 3.2	13.1	21.1	0.0	12.8	90	212	180	35	A	**7.5**	103	27
San Andreas, (North Branch, Mill Creek) FM3.1, 3	13.1	21.1	0.0	18.2	76	204	180	106	A	**7.6**	219	34
San Andreas (San Bernardino S) FM3.1, 3.2	13.5	21.7	0.0	12.8	90	210	180	43	A	**7.6**	150	29
Chino, alt 1, FM3.1	16.3	26.3	0.0	9.0	50	236	150	24	B	**6.6**		1
Elsinore (Glen Ivy) rev FM3.1, 3.2	16.4	26.4	0.0	13.2	90	218	180	26	A	**7.0**	222	3
Chino, alt 2, FM3.2	16.4	26.4	0.0	13.4	65	234	150	29	B	**6.7**		1
Whittier, alt 1, FM3.1	17.1	27.5	0.0	12.4	70	24	150	46	A	**7.1**	530	4
Whittier, alt 2 FM3.2	17.1	27.5	0.0	14.1	75	24	150	46	A	**6.9**	165	4
San Gorgonio Pass FM3.1, 3.2	17.9	28.8	0.0	18.5	60	11	na	29	B'	**6.9**		
San Jacinto (San Jacinto Valley, stepover)	17.9	28.8	0.0	16.1	90	224	180	24	A	**7.6**	219	9
San Jacinto (Stepovers Combined) FM3.1, 3.2	17.9	28.8	0.0	16.5	90	229	180	25	A	**7.5**	110	4
Cleghorn FM3.1, 3.2	19.2	30.9	0.0	15.5	90	187	0	25	B	**6.7**		3
San Jose FM3.1, 3.2	19.4	31.2	0.0	15.8	74	334	30	20	B	**6.6**		0.5
Yorba Linda FM3.1, 3.2	20.2	32.4	0.0	13.3	90	153	na	18	B'	**6.5**		
Elsinore (Glen Ivy stepover) FM3.1, 3.2	22.0	35.4	0.0	13.2	90	216	180	11	A	**7.1**	322	15
Elsinore (Stepovers Combined) FM3.1, 3.2	22.0	35.4	0.0	13.7	90	224	180	12	A	**7.6**	725	5
Sierra Madre FM3.1, 3.2	22.3	35.8	0.0	14.2	53	19	90	57	B	**7.2**		2
North Frontal (West) FM3.1, 3.2	22.5	36.2	0.0	15.7	49	171	90	50	B	**7.2**		1
San Gabriel (Extension) FM3.1, 3.2	23.0	37.0	0.0	14.7	61	6	180	62	B'	**7.2**		
San Andreas (Mojave S) FM3.1, 3.2	23.7	38.1	0.0	13.1	90	206	180	98	A	**7.7**	102	34
Peralta Hills FM3.1, 3.2	23.7	38.2	0.3	14.0	50	3	na	14	B'	**6.5**		
Richfield FM3.1, 3.2	25.8	41.6	2.5	12.9	28	353	na	6	B'	**6.2**		
Mission Creek FM3.1, 3.2	27.5	44.2	0.0	17.7	65	5	180	31	B'	**6.9**		
Elsinore (Temecula) rev FM3.1, 3.2	27.7	44.5	0.0	14.2	90	230	180	40	A	**7.4**	431	3
Puente Hills, FM3.1	28.5	45.9	5.0	13.0	25	20	90	44	B	**7.1**		0.7
Puente Hills (Coyote Hills) FM3.2	28.9	46.6	2.8	14.6	26	358	90	17	B	**6.8**		0.7
Clamshell-Sawpit FM3.1, 3.2	31.3	50.4	0.0	14.0	50	334	90	16	B	**6.6**		0.5
San Joaquin Hills FM3.1, 3.2	32.1	51.6	2.0	12.4	23	204	90	27	B	**7.0**		0.5
San Jacinto (Anza) rev FM3.1, 3.2	32.4	52.1	0.0	16.8	90	216	180	46	A	**7.6**	219	17
San Andreas (San Gorgonio Pass-Garnet HIll) FM	33.2	53.5	0.0	12.8	58	20	180	56	A	**7.6**	219	24
Elysian Park (Lower, CFM) FM3.1, 3.2	34.3	55.2	10.0	14.7	22	33	na	41	B'	**6.8**		
Anaheim FM 3.1, 3.2	36.0	58.0	3.8	14.2	71	45	na	16	B'	**6.3**		
Raymond FM3.1, 3.2	36.7	59.0	0.0	15.6	79	348	60	22	B	**6.7**		1.5
Puente Hills (Santa Fe Springs) FM3.2	37.0	59.5	2.8	15.0	29	347	90	11	B	**6.6**		0.7
Pinto Mtn FM3.1, 3.2	37.8	60.9	0.0	15.5	90	175	0	74	B	**7.2**		2.5

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

Based on Site Coordinates of 34.0039 Latitude, -117.3785 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 magntude is average of Ellworths-B and Hanks & Bakun moment area relationship.

Site Coordinates: 34.004 N 117.379 W

Table A-2
Historic Earthquakes in Vicinity of Project Site, M >= 5.5

Day	*Year*	*Epicenter Latittude (Degrees)*	*Longitude*	*Distance from Site (mi)*	*Magnitude* M_W
7/23	*1923	34.00	117.25	7.4	**6.2**
12/16	1858	34.20	117.40	13.6	**6.0**
7/22	1899	34.20	117.40	13.6	**5.9**
6/14	1892	34.20	117.50	15.2	**5.5**
2/28	1990	34.14	117.70	20.6	**5.7**
9/20	*1907	34.20	117.10	20.9	**5.8**
5/15	1910	33.70	117.40	21.0	**6.0**
7/22	1899	34.30	117.50	21.6	**6.4**
11/22	1880	34.00	117.00	21.7	**5.5**
12/19	1880	34.00	117.00	21.7	**5.9**
7/30	1894	34.30	117.60	24.0	**6.2**
10/16	1999	34.24	117.04	25.3	**5.6**
12/25	*1899	33.80	117.00	25.9	**6.7**
4/21	*1918	33.75	117.00	27.9	**6.8**
12/8	1812	34.37	117.65	29.7	**7.5**
1/16	1930	34.20	116.90	30.5	**5.5**
6/28	1992	34.16	116.85	32.1	**5.5**
8/28	1889	34.20	117.90	32.7	**5.6**
6/28	1992	34.20	116.83	34.2	**6.5**
7/28	1769	34.00	118.00	35.6	**6.0**
2/7	1889	34.10	116.70	39.4	**5.6**
6/28	1991	34.27	117.99	39.5	**5.6**
10/1	1987	34.06	118.08	40.3	**5.9**
7/11	1855	34.10	118.10	41.8	**6.0**
3/11	1933	33.64	117.97	42.2	**6.4**
4/99	1803	34.20	118.10	43.4	**5.5**
7/8	1986	34.00	116.61	44.0	**6.0**
6/6	1918	33.60	116.70	47.9	**5.5**
10/2	1928	33.60	116.70	47.9	**5.5**
1/16	1857	34.52	118.04	51.9	**6.3**
6/28	1992	34.20	116.44	55.3	**7.3**
6/28	1992	34.13	116.41	56.1	**5.8**
6/29	1992	34.10	116.40	56.4	**5.7**
3/15	1979	34.33	116.44	58.2	**5.5**
4/23	1992	33.96	116.32	60.7	**6.2**
6/28	1992	34.12	116.32	61.1	**5.7**
4/11	1910	33.50	116.50	61.3	**5.8**
2/9	1971	34.42	118.37	63.5	**6.6**
2/9	1971	34.42	118.37	63.5	**5.8**
2/9	1971	34.42	118.37	63.5	**5.8**

From full earthquake catalog in USGS OFR 2008-1437h as updated with current events through 2019. For events with an asterisk, alternate solutions are given in

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)

			CBC Reference	ASCE 7-16 Reference
Seismic Design Category		**D**		
Site Class		**D** *	Table 1613.5.6	Table 11.6-1
Latitude:		34.004	Table 1613.5.2	Table 20.3-1
Longitude:		-117.379		
Maximum Considered Earthquake (MCE) Ground Motion				
Short Period Spectral Reponse	S_S	**1.500 g**	Figure 1613.5	Figure 22-1
1 second Spectral Response	S_1	**0.600 g**	Figure 1613.5	Figure 22-2
Site Coefficient	F_a	1.00	Table 1613.5.3(1)	Table 11.4-1
Site Coefficient	F_v	1.70	Table 1613.5.3(2)	Table 11-4.2
	S_{MS}	1.500 g	$= F_a*S_S$	
	S_{M1}	1.020 g	$= F_v*S_1$	
Design Earthquake Ground Motion				
Short Period Spectral Reponse	S_{DS}	**1.000 g**	$= 2/3*S_{MS}$	
1 second Spectral Response	S_{D1}	**0.680 g**	$= 2/3*S_{M1}$	

*Site Class F applies. Site Class D may be used for Structure Periods less than 0.5 seconds

Site Specific Evaluation May Be Required Due to Site Class = D or E and S1>=0.2. The Presented SDS and SD1 are NOT Valid Unless the Exception of ASCE7-16, Section 11.4.8 Applies

To	0.14 sec	$= 0.2*S_{D1}/S_{DS}$
Ts (11.4.8 ASCE 7-16 Exception Assumed)	0.68 sec	$= S_{D1}/S_{DS}$
Risk Category	II	Table 1604.5
Seismic Importance Factor	1.00	
F_{PGA}	1.10	
PGA_M	**0.63**	
Vertical Coefficient (C_V)	1.40	Table 11.9-1

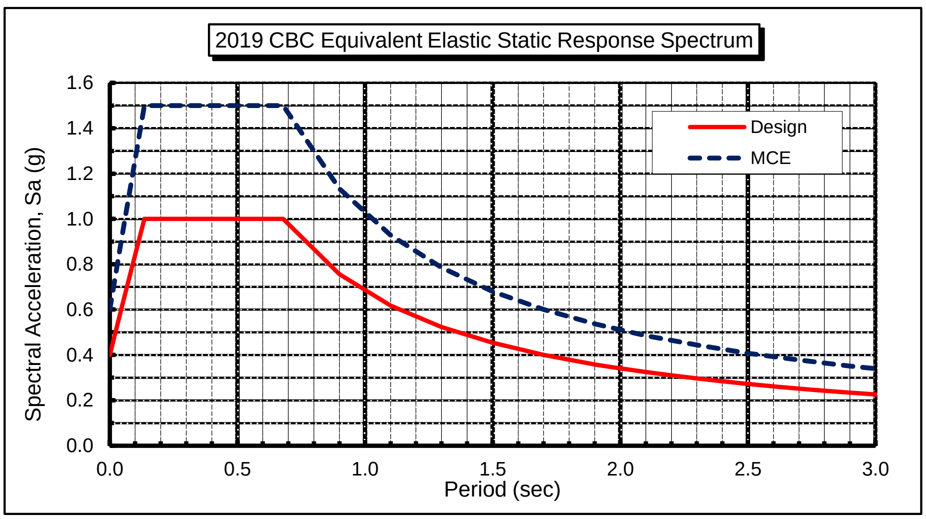


Table 11.5-1 Design

Period T (sec)	Sa (g)
0.00	0.400
0.05	0.621
0.14	1.000
0.68	1.000
0.90	0.756
1.10	0.618
1.30	0.523
1.50	0.453
1.70	0.400
1.90	0.358
2.10	0.324
2.30	0.296
2.50	0.272
2.70	0.252
2.90	0.234
3.10	0.219

DESCRIPTIVE SOIL CLASSIFICATION

Soil classification is based on ASTM Designations D 2487 and D 2488 (Unified Soil Classification System). Information on each boring log is a compilation of subsurface conditions obtained from the field as well as from laboratory testing of selected samples. The indicated boundaries between strata on the boring logs are approximate only and may be transitional.

SOIL GRAIN SIZE

U.S. STANDARD SIEVE

12” 3” 3/4” 4 10 40 200

BOULDERS	COBBLES	GRAVEL		SAND			SILT	CLAY
		COARSE	FINE	COARSE	MEDIUM	FINE		

305 76.2 19.1 4.76 2.00 0.42 0.074 0.002

SOIL GRAIN SIZE IN MILLIMETERS

RELATIVE DENSITY OF GRANULAR SOILS (GRAVELS, SANDS, AND NON-PLASTIC SILTS)

Very Loose	*N=0-4	RD=0-30	Easily push a 1/2-inch reinforcing rod by hand
Loose	N=5-10	RD=30-50	Push a 1/2-inch reinforcing rod by hand
Medium Dense	N=11-30	RD=50-70	Easily drive a 1/2-inch reinforcing rod with hammer
Dense	N=31-50	RD=70-90	Drive a 1/2-inch reinforcing rod 1 foot with difficulty by a hammer
Very Dense	N>50	RD=90-100	Drive a 1/2-inch reinforcing rod a few inches with hammer

*N=Blows per foot in the Standard Penetration Test at 60% theoretical energy. For the 3-inch diameter Modified California sampler,140-pound weight, multiply the blow count by 0.63 (about 2/3) to estimate N. If automatic hammer is used, multiply a factor of 1.3 to 1.5 to estimate N. RD=Relative Density (%). C=Undrained shear strength (cohesion).

CONSISTENCY OF COHESIVE SOILS (CLAY OR CLAYEY SOILS)

Very Soft	*N=0-1	*C=0-250 psf	Squeezes between fingers
Soft	N=2-4	C=250-500 psf	Easily molded by finger pressure
Medium Stiff	N=5-8	C=500-1000 psf	Molded by strong finger pressure
Stiff	N=9-15	C=1000-2000 psf	Dented by strong finger pressure
Very Stiff	N=16-30	C=2000-4000 psf	Dented slightly by finger pressure
Hard	N>30	C>4000	Dented slightly by a pencil point or thumbnail

MOISTURE DENSITY

Moisture Condition: An observational term; dry, damp, moist, wet, saturated.

Moisture Content: The weight of water in a sample divided by the weight of dry soil in the soil sample expressed as a percentage.

Dry Density: The pounds of dry soil in a cubic foot.

MOISTURE CONDITION

Dry.....................Absence of moisture, dusty, dry to the touch
Damp................Slight indication of moisture
Moist.................Color change with short period of air exposure (granular soil)
Below optimum moisture content (cohesive soil)
Wet....................High degree of saturation by visual and touch (granular soil)
Above optimum moisture content (cohesive soil)
Saturated..........Free surface water

RELATIVE PROPORTIONS

Trace.............minor amount (<5%)
with/some......significant amount
modifier/and...sufficient amount to influence material behavior (Typically >30%)

PLASTICITY

DESCRIPTION	FIELD TEST
Nonplastic	A 1/8 in. (3-mm) thread cannot be rolled at any moisture content.
Low	The thread can barely be rolled.
Medium	The thread is easy to roll and not much time is required to reach the plastic limit.
High	The thread can be rerolled several times after reaching the plastic limit.

LOG KEY SYMBOLS

Bulk, Bag or Grab Sample

Standard Penetration Split Spoon Sampler (2” outside diameter)

Modified California Sampler (3” outside diameter)

No Recovery

GROUNDWATER LEVEL

Water Level (measured or after drilling)

Water Level (during drilling)

Terms and Symbols Used on Boring Logs

Earth Systems

MAJOR DIVISIONS			GRAPHIC SYMBOL	LETTER SYMBOL	TYPICAL DESCRIPTIONS
COARSE GRAINED SOILS More than 50% of material is <u>larger</u> than No. 200 sieve size	GRAVEL AND GRAVELLY SOILS More than 50% of coarse fraction <u>retained</u> on No. 4 sieve	CLEAN GRAVELS		GW	Well-graded gravels, gravel-sand mixtures, little or no fines
				GP	Poorly-graded gravels, gravel-sand mixtures. Little or no fines
		GRAVELS WITH FINES		GM	Silty gravels, gravel-sand-silt mixtures
				GC	Clayey gravels, gravel-sand-clay mixtures
	SAND AND SANDY SOILS More than 50% of coarse fraction <u>passing</u> No. 4 sieve	CLEAN SAND (Little or no fines)		SW	Well-graded sands, gravelly sands, little or no fines
				SP	Poorly-graded sands, gravelly sands, little or no fines
		SAND WITH FINES (appreciable amount of fines)		SM	Silty sands, sand-silt mixtures
				SC	Clayey sands, sand-clay mixtures
FINE-GRAINED SOILS More than 50% of material is <u>smaller</u> than No. 200 sieve size	SILTS AND CLAYS	LIQUID LIMIT <u>LESS</u> THAN 50		ML	Inorganic silts and very fine sands, rock flour, silty low 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
		LIQUID LIMIT <u>GREATER</u> THAN 50		MH	Inorganic silty, micaceous, or diatomaceous fine sand or silty soils
				CH	Inorganic clays of high plasticity, fat clays
				OH	Organic clays of medium to high plasticity, organic silts
HIGHLY ORGANIC SOILS				PT	Peat, humus, swamp soils with high organic contents
VARIOUS SOILS AND MAN MADE MATERIALS					Fill Materials
MAN MADE MATERIALS					Asphalt and concrete

Soil Classification System

Earth Systems

Earth Systems

1680 Illinois Ave., Suite 20, Perris, CA 92571
Phone (951) 928-9799

Boring No. B-1

Project Name: Proposed Building 6 Modular Upgrade
Project Number 302451-002
Boring Location: See Plate 2

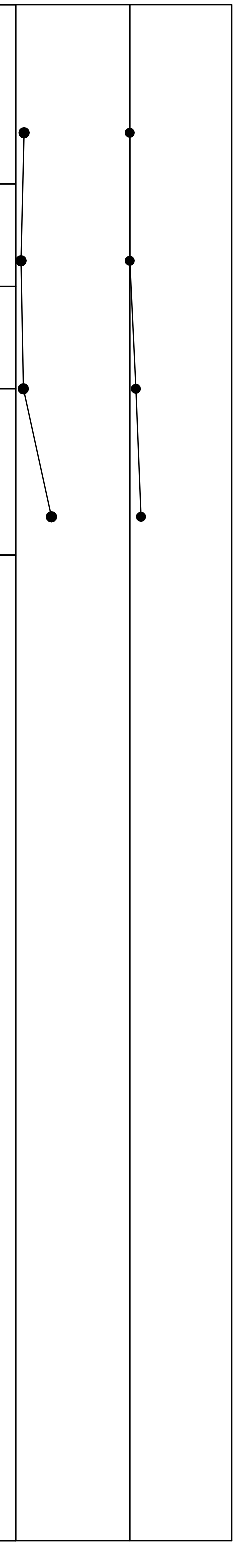
Drilling Date: October 26, 2020
Drilling Method: Mobile B-61 w/autohammer
Drill Type: 8" HSA
Logged By: AL

Description of Units

Note: The stratification lines shown represent the approximate boundary between soil and/or rock types and the transition may be gradational.

Depth (Ft.)	Sample Type (Bulk / SPT / MOD Calif.)	Penetration Resistance (Blows/6")	Symbol	USCS	Dry Density (pcf)	Moisture Content (%)	Description of Units
0				SW			WELL GRADED SAND: olive brown, loose, damp, fine to coarse grained
5	MOD Calif.	4,5,6			98	3	
				CH			SANDY FAT CLAY: olive gray, firm, wet, fine grained sand
10	MOD Calif.	1,3,4			79	49	
				SM			SILTY SAND: olive brown, loose, moist, fine grained sand
15	MOD Calif.	6,6,4		SW	106	21	WELL GRADED SAND: olive brown, medium dense, wet, fine to coarse grained sand
20	MOD Calif.	8,20,27			111	18	dense, wet

Graphic Trend
Blow Count Dry Density



Boring ended at 21-1/2 feet
Backfilled with cuttings and plug
Groundwater encountered at 15 feet

Earth Systems

1680 Illinois Ave., Suite 20, Perris, CA 92571
Phone (951) 928-9799

Boring No. B-2

Project Name: Proposed Building 6 Modular Upgrade
Project Number 302451-002
Boring Location: See Plate 2

Drilling Date: October 26, 2020
Drilling Method: Mobile B-61 w/autohammer
Drill Type: 8" HSA
Logged By: AL

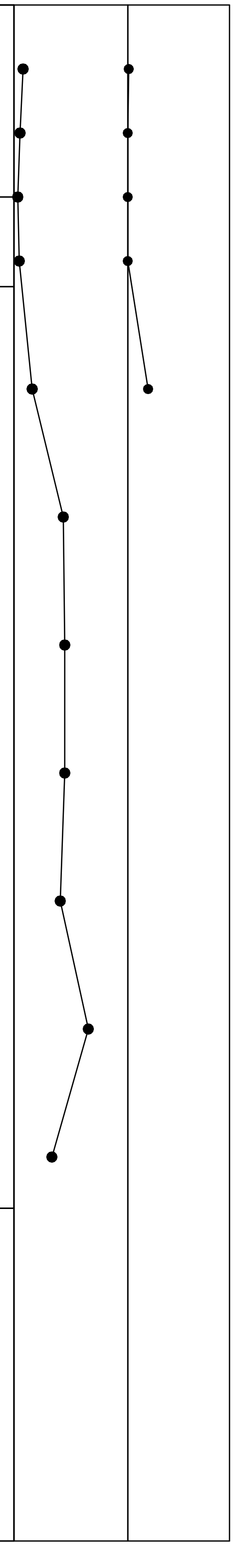
Description of Units

Note: The stratification lines shown represent the approximate boundary between soil and/or rock types and the transition may be gradational.

Depth (Ft.)	Sample Type (Bulk / SPT / MOD Calif.)	Penetration Resistance (Blows/6")	USCS	Dry Density (pcf)	Moisture Content (%)	Description of Units
0	Bulk (0–4.5)		SM			SILTY SAND: light brown, loose, damp, fine grained sand
2.5	MOD Calif.	3,5,7		101	9	
5	MOD Calif.	3,3,5		94	6	
7.5	MOD Calif.	1,2,3	CH	74	49	SANDY FAT CLAY: olive brown, soft, wet, fine grained sand
10	MOD Calif.	3,3,4		74	47	firm
11			SW			WELL GRADED SAND: gray brown, medium dense, moist, fine to coarse grained sand, trace gravel
15	MOD Calif.	7,11,13		120	5	
20	MOD Calif.	16,27,38				very dense, wet
25	MOD Calif.	17,50/6"				
30	MOD Calif.	17,27,40				
35	MOD Calif.	15,26,35				no recovery, log from cuttings
40	MOD Calif.	4,8,50/5"				very dense, 3 rings collected near shoe, partial recovery
45	MOD Calif.	50/6"				no recovery, log from cuttings

Refusal at 45-1/2 feet, due to hard drilling
Backfilled with cuttings and plug
Groundwater encountered at 20 feet

Graphic Trend
Blow Count Dry Density



Earth Systems
1680 Illinois Ave., Suite 20, Perris, CA 92571
Phone (951) 928-9799

Boring No. B-3

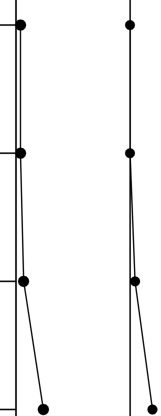
Project Name: Proposed Building 6 Modular Upgrade
Project Number 302451-002
Boring Location: See Plate 2

Drilling Date: October 26, 2020
Drilling Method: Mobile B-61 w/autohammer
Drill Type: 8" HSA
Logged By: AL

Depth (Ft.)	Sample Type (Bulk / SPT / MOD Calif.)	Penetration Resistance (Blows/6")	Symbol	USCS	Dry Density (pcf)	Moisture Content (%)	Description of Units
0				SP-SM			SAND WITH SILT: olive gray, loose, damp, fine grained sand
5	MOD Calif.	2,3,3		SM	96	5	SILTY SAND: olive gray, loose, damp, fine grained sand
10	MOD Calif.	1,2,4		CH	82	39	SANDY FAT CLAY: olive gray, soft, very moist, no free water, fine grained sand
15	MOD Calif.	1,6,4		SM	105	21	SILTY SAND: olive gray, loose, wet, fine grained sand
20	MOD Calif.	1,8,28		SP-SM	122	13	POORLY GRADED SAND WITH SILT: olive gray, medium dense, wet, fine to medium grained sand

Note: The stratification lines shown represent the approximate boundary between soil and/or rock types and the transition may be gradational.

Graphic Trend
Blow Count Dry Density



Total depth 21-1/2 feet
Backfilled with cuttings and plug
Groundwater encountered at 15 feet

Earth Systems

1680 Illinois Ave., Suite 20, Perris, CA 92571
Phone (951) 928-9799

Boring No. **B-4**

Project Name: Proposed Building 6 Modular Upgrade
Project Number 302451-002
Boring Location: See Plate 2

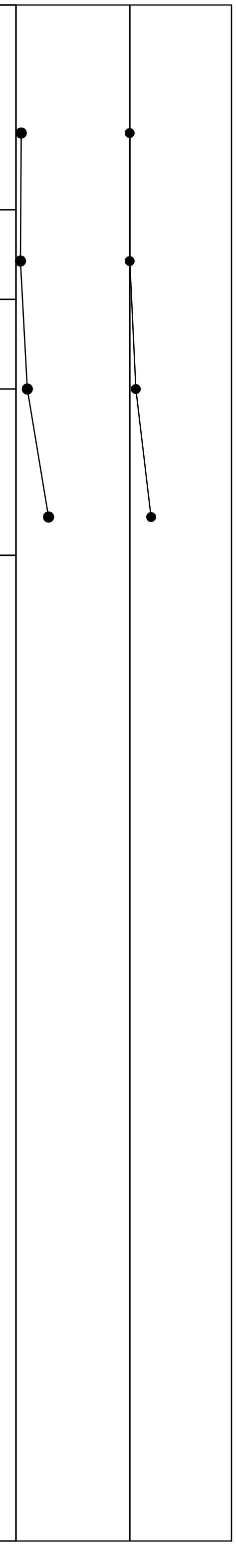
Drilling Date: October 26, 2020
Drilling Method: Mobile B-61 w/autohammer
Drill Type: 8" HSA
Logged By: AL

Description of Units

Note: The stratification lines shown represent the approximate boundary between soil and/or rock types and the transition may be gradational.

Depth (Ft.)	Sample Type (Bulk / SPT / MOD Calif.)	Penetration Resistance (Blows/6")	Symbol	USCS	Dry Density (pcf)	Moisture Content (%)	Description of Units
0	Bulk			SW			WELL GRADED SAND: olive brown, loose, damp, fine to coarse grained sand
5	MOD Calif.	3,3,4			98	2	
				CL			SANDY LEAN CLAY: olive brown, soft, moist, fine grained sand
10	MOD Calif.	2,3,3			93	17	
				SM			SILTY SAND: olive brown, very loose, moist, fine grained sand
15	MOD Calif.	1,5,10		SW	106	7	WELL GRADED SAND: olive brown, medium dense, damp, fine to coarse grained sand, alternating lenses of silt
20	MOD Calif.	10,17,26			121	14	olive gray, dense, wet

Graphic Trend
Blow Count Dry Density



Total depth 21-1/2 feet
Backfilled with cuttings and plug
Groundwater encountered at 16-1/2 feet

Boring No.	B-2	Project and Number	Proposed Building 6 N	302451-002

ESSW Field Staff	
Drilling Company	CalPac
Drilling Method	6-8" H S A / HSA Inner Diameter: 3"
Site Latitude (North)	Decimal Degrees: 34.0039
Site Longitude (West)	Decimal Degrees: 117.3785
Date Drilled	10/26/2020
Hammer Weight (lbs)	140
Hammer Drop (inches)	30
Hammer Efficiency (E_M)	79
Borehole Correction (Cb)*	1
*inside diameter of Hollow Stem Auger	
Sampler Correction Mod Cal to SPT	0.63
Sampler Liner Correction (Cs)	1.2 Applied if SPT Sampler Used; 1.0 Applied if Cal Sampler Used
Rod Length Above Ground (ft)	3
Depth to Estimate Vs Over (ft)*	100
*Caltrans Estimation Method	
*N_{sub} Value Desired For Column 6	70

*Only Used for Calculating Nsub otherwise not used by program (i.e.N50, N70, N80, etc)

Calculation Results

Ave. SPT N60HE-value (blows/ft)	19 (Based on Upper 45 feet)
Ave. Shear Wave Velocity (ft/sec)	870 (Based on Upper 45 feet)
Soil Profile Type (Site Class)	D (Based on Upper 45 feet)
Ave. Friction Angle (degrees)	35 (Based on Upper 45 feet)
Estimated Shear Wave Velocity ** Based on Depth Less than 100' ft	867 (ft/sec Upper 100 feet)
Soil Profile Type (Site Class)**	D Based on Ave. Shear Wave Velocity (ft/sec) 264 (m/sec Upper 100 feet)
Ave. Field SPT N-value (blows/ft)	14.2 (Based on Upper 45 feet)
Ave. Field SPT N-value (blows/ft)	29.3 (Based on Upper 100 feet)
Soil Profile Type (Site Class)**	D Based on Ave. Field Blow Count 29 (Upper 100 feet)

	Equipment variable	Typical Correction (%/100)
Energy ratio (Skempton, 1986)	Donut Hammer	0.50 to 1.00
	Safety Hammer	0.70 to 1.20
	Automatic-Trip Donut-type Hammer	0.80 to 1.30

→ Hammer energy as related to the standard 60% delivered energy, i.e. a 72% hammer has and energy ratio of 1.2, i.e. (72/60=1.2)

1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
Bottom of Layer Depth (ft)	Blow Count***	Type of Sampler	d_i (feet)	N_{60} (blows/ft)	N70 (blows/ft)	N_{60HE} (blows/ft)	V_{si}** (m/sec)	V_{si} (ft/sec)	Φ_i (degrees)	d_i/N_{60HEi}	d_i/V_{si}	d_i/Φ_i	Consistency if Coarse Grained (Based on ASTM and Corrected for N60)	Consistency if Fine Grained (Based on ASTM and Corrected for N60)
2.5	12	c	2.5	7.47	6.40	9.95	195.70	641.89	29.22	0.25116	0.00389	0.085548	Loose	Firm
5.0	8	c	2.5	4.98	4.27	6.64	173.99	570.68	27.72	0.37673	0.00438	0.090195	Loose	Firm
7.5	5	c	2.5	3.11	2.67	4.15	151.82	497.96	26.18	0.60277	0.00502	0.095498	Very Loose	Soft
10.0	7	c	2.5	4.35	3.73	5.81	167.38	549.00	27.26	0.43055	0.00455	0.091713	Loose	Firm
15.0	24	c	5.0	16.92	14.50	19.91	239.27	784.80	32.24	0.25116	0.00637	0.155106	Medium Dense	Very Stiff
20.0	65	c	5.0	51.22	43.90	53.92	319.42	1047.71	37.75	0.09273	0.00477	0.132463	Very Dense	Hard
25.0	67	c	5.0	52.80	45.26	55.58	322.24	1056.96	37.94	0.08997	0.00473	0.131789	Very Dense	Hard
30.0	67	c	5.0	55.58	47.64	55.58	322.24	1056.96	37.94	0.08997	0.00473	0.131789	Very Dense	Hard
35.0	61	c	5.0	50.60	43.37	50.60	313.59	1028.59	37.35	0.09882	0.00486	0.133881	Very Dense	Hard
40.0	100	c	5.0	82.95	71.10	82.95	361.93	1187.13	40.66	0.06028	0.00421	0.12298	Very Dense	Hard
45.0	100	c	5.0	82.95	71.10	82.95	361.93	1187.13	40.66	0.06028	0.00421	0.12298	Very Dense	Hard
		Total:	45.0	"d" Feet				Total:		2.40440	0.05174	1.293943		

**Used When Boring Depths are less than 100 feet to estimate Shear Wave Velocity over 100 feet. Caltrans Geotechnical Services Design Manual, Version 1.0, August 2009 using N60HE corrected only for Hammer Energy (Empirical Calculation)

*** Uncorrected blowcount not to exceed 100 blows as entry per CBC

Consistency classification based upon ASCE 1996

BOREHOLE, SAMPLER, AND ROD CORRECTION FACTORS

Factor	Equipment Variables	Value
Borehole diameter factor, C_B	2.5 - 4.5 in (65 - 115 mm)	1.00
	6 in (150 mm)	1.05
	8 in (200 mm)	1.15
Sampling method factor, C_S	Standard sampler	1.00
	Sampler without liner	1.20
Rod length factor, C_R	10 - 13 ft (3 - 4 m)	0.75
	13 - 20 ft (4 - 6 m)	0.85
	20 - 30 ft (6 - 10 m)	0.95
	> 30 ft (> 10 m)	1.00

Adapted from Skempton (1986).

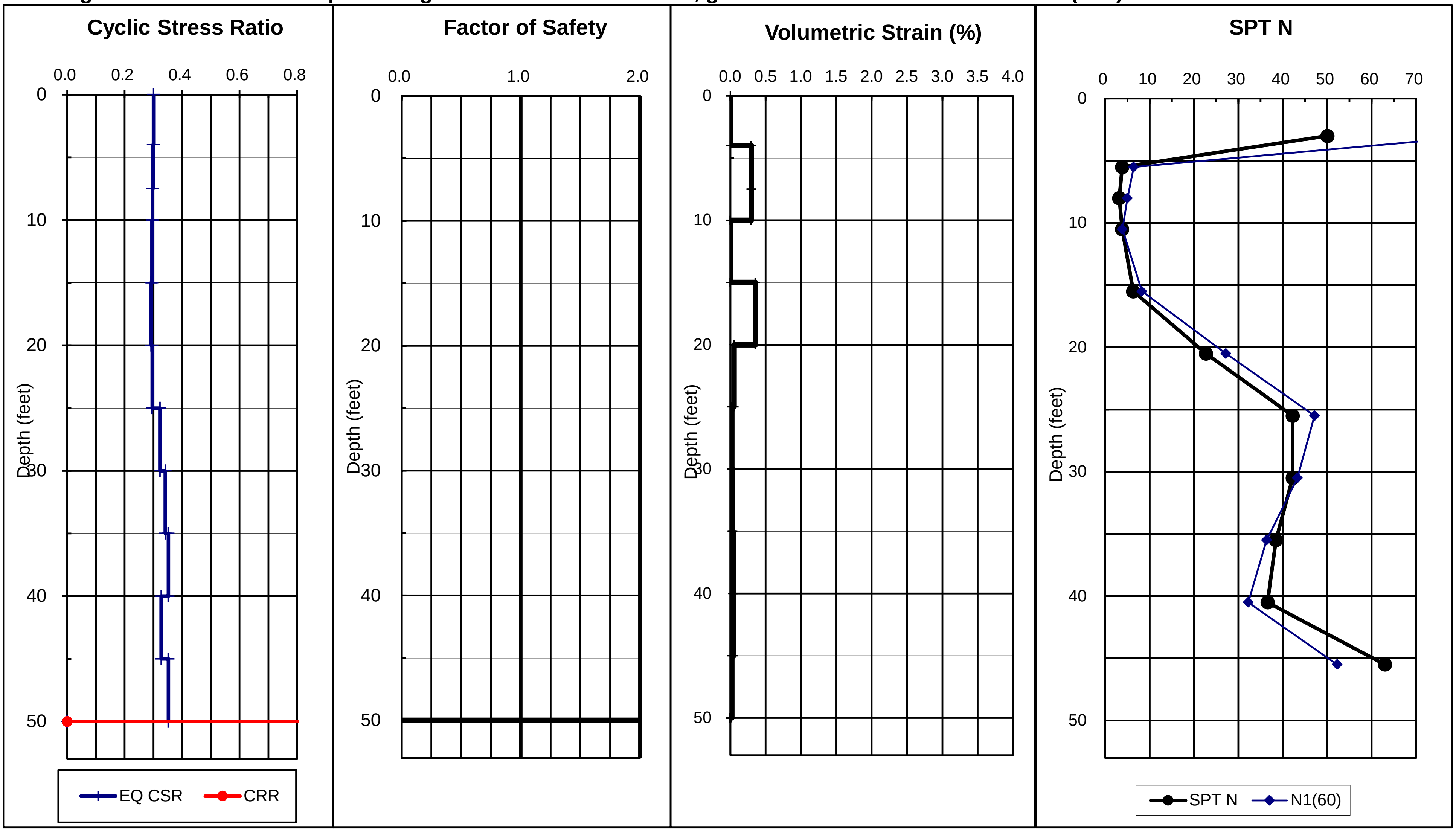
Spreadsheet Version 2.6, 2019: Prepared by Kevin L. Paul, PE, GE

EARTH SYSTEMS - EVALUATION OF LIQUEFACTION POTENTIAL AND INDUCED SUBSIDENCE

Proposed Building 6 Modular Upgrade **Project No: 302451-002** 1996/1998 NCEER Method

Ground Compaction Remediated to 5 foot depth

Boring: B-2 **Earthquake Magnitude: 7.8** **PGA, g: 0.42** **Calc GWT (feet): 50**



Total Thickness of Liquefiable Layers: 0.0 feet

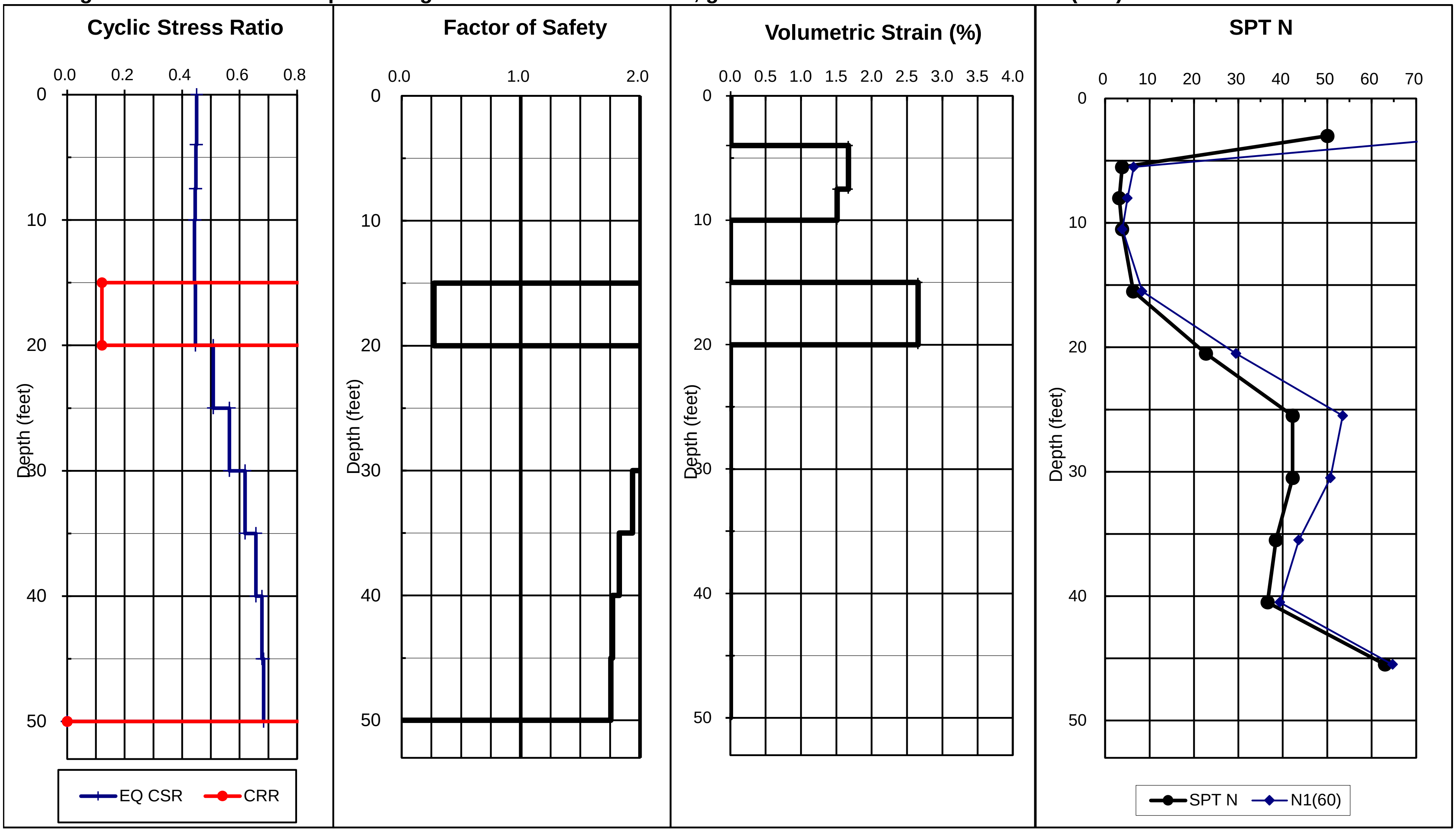
Estimated Total Ground Subsidence: 0.6 inches

EARTH SYSTEMS - EVALUATION OF LIQUEFACTION POTENTIAL AND INDUCED SUBSIDENCE

Proposed Building 6 Modular Upgrade **Project No: 302451-002** 1996/1998 NCEER Method

Ground Compaction Remediated to 5 foot depth

Boring: B-2 **Earthquake Magnitude: 7.8** **PGA, g: 0.63** **Calc GWT (feet): 15**



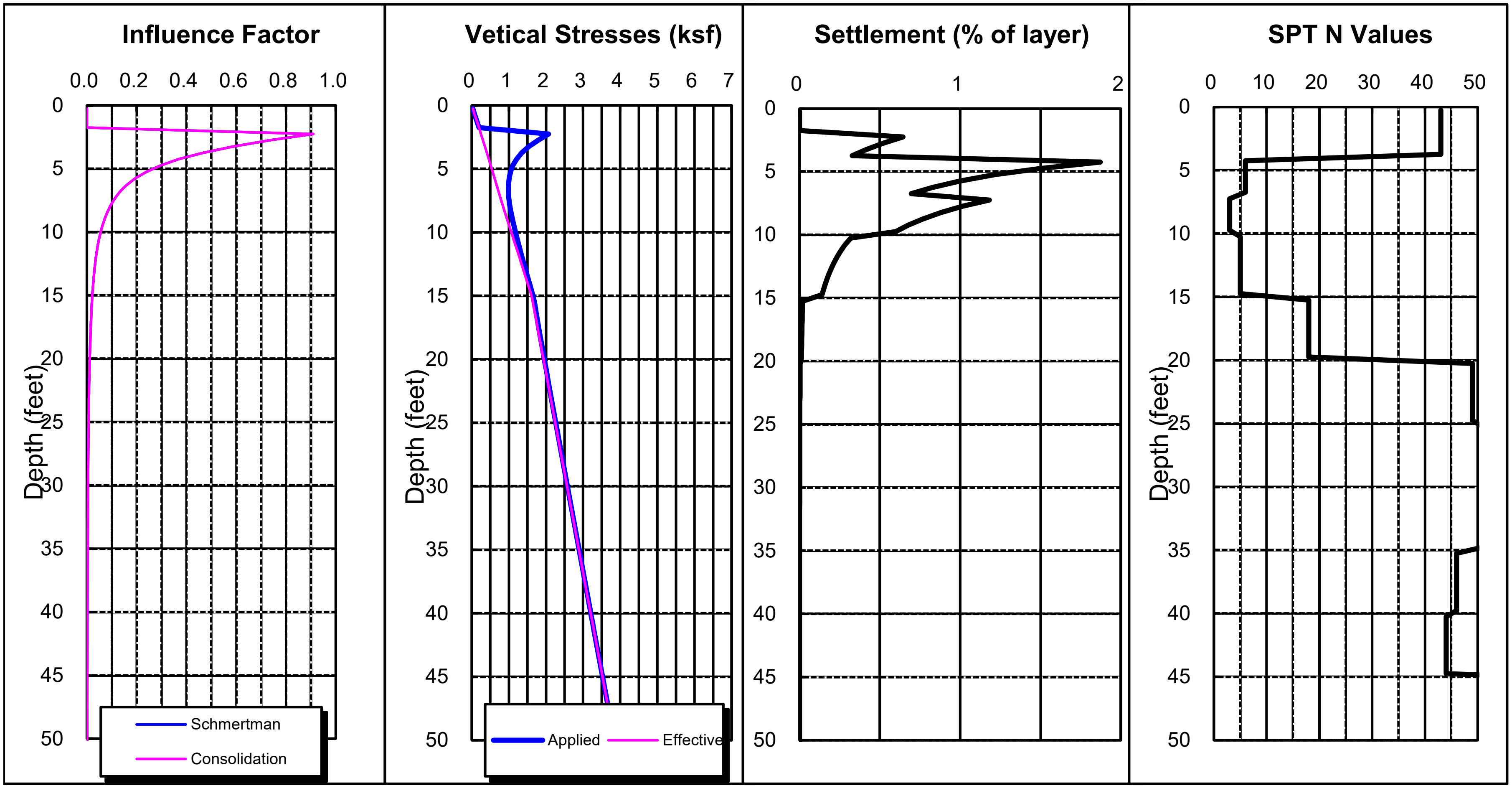
Total Thickness of Liquefiable Layers: 5.0 feet **Estimated Total Ground Subsidence: 2.8 inches**

EARTH SYSTEMS - SETTLEMENT ANALYSES

Proposed Building 6 Modular Upgrade

302451-002

Width, ft: 3.5 | Length, ft: 3.5 | Net pressure, ksf: 2.00 | **Settlement, inches: 1.0**



Load, Q: 25 kips

Embedment, feet: 2.0

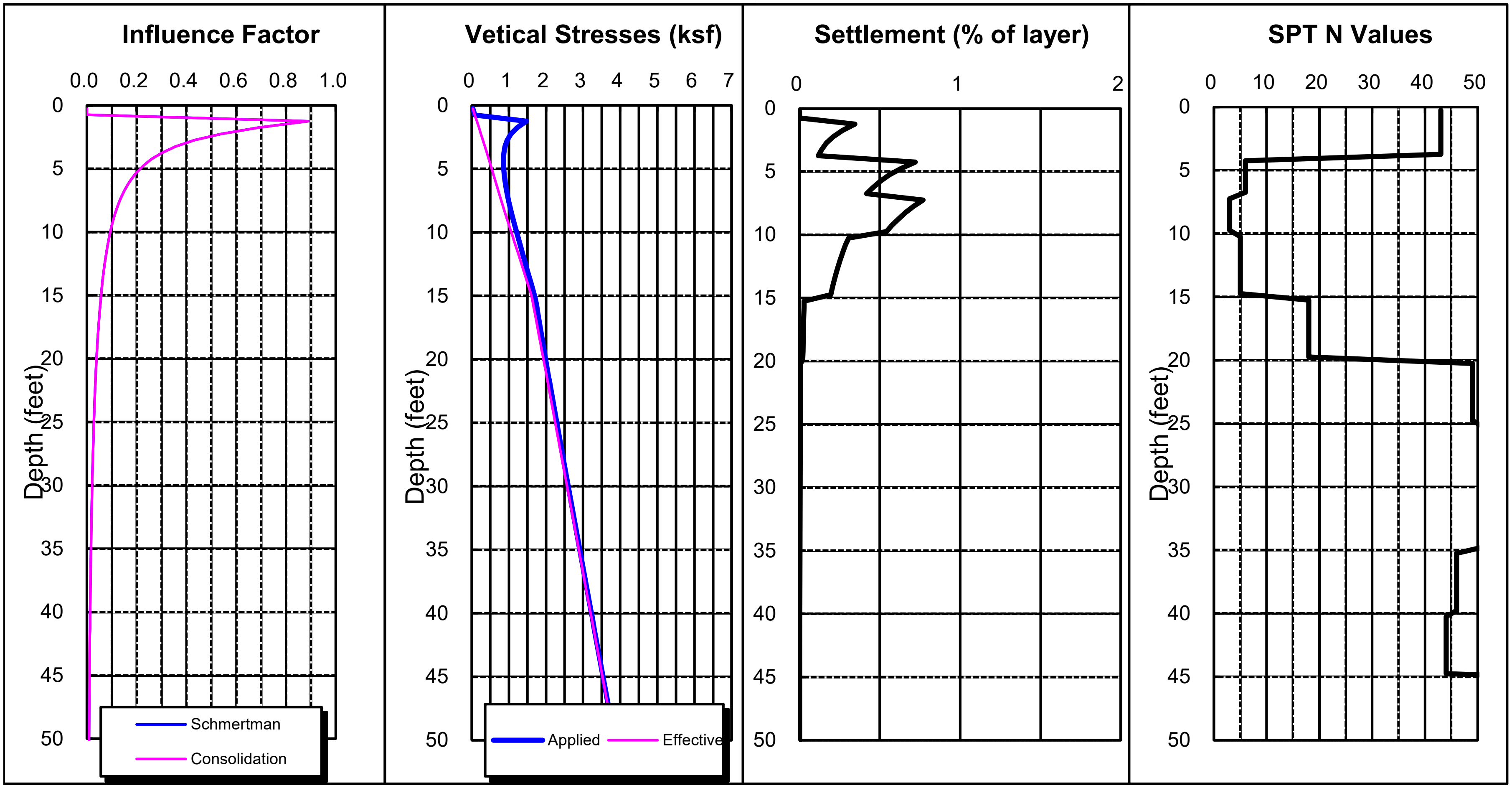
Boring: B-2

EARTH SYSTEMS - SETTLEMENT ANALYSES

Proposed Building 6 Modular Upgrade

302451-002

Width, ft: 2.0 Length, ft: 40.0 Net pressure, ksf: 1.50 **Settlement, inches: 0.7**



Load, Q: 3 kpf Embedment, feet: 1.0 **Boring: B-2**

APPENDIX B

Laboratory Test Results

UNIT DENSITIES AND MOISTURE CONTENT ASTM D2937 & D2216

Job Name: Proposed Building 6 Mod

Sample Location	Depth (feet)	Unit Dry Density (pcf)	Moisture Content (%)	USCS Group Symbol
B1	5	98.4	2.6	SW
B1	10	78.7	48.6	CH
B1	15	105.6	21.0	SW
B1	20	111.3	18.0	SW
B2	2.5	100.9	8.8	SM
B2	5	93.6	5.7	SM
B2	7.5	73.8	48.5	CH
B2	10	74.0	46.9	CH/SW
B2	15	120.1	4.6	SW
B3	5	95.6	5.1	SM
B3	10	82.2	39.2	CH
B3	15	105.4	21.0	SM
B3	20	122.1	13.0	SP-SM
B4	5	98.1	2.3	SW
B4	10	93.1	17.3	CL
B4	15	105.6	7.3	SW
B4	20	121.3	14.0	SW

File No.: 302451-002 11/30/2020

Job Name: Proposed Building 6 Mod

Lab Number: 20-196

ASTM D-1140 or Earth Systems Method (circle one)

AMOUNT PASSING NO. 200 SIEVE (Earth Systems Method Transfers Sample until water runs clear)

Sample Location	Depth (feet)	Fines Content (%)	USCS Group Symbol	Soaking Time
B1	10	18.8	SM	10

File No.: 302451-002
Lab No.: 20-196

November 30, 2020

CONSOLIDATION TEST **ASTM D 2435 & D 5333**

Proposed Building 6 Mod
B2 @ 7 1/2 feet

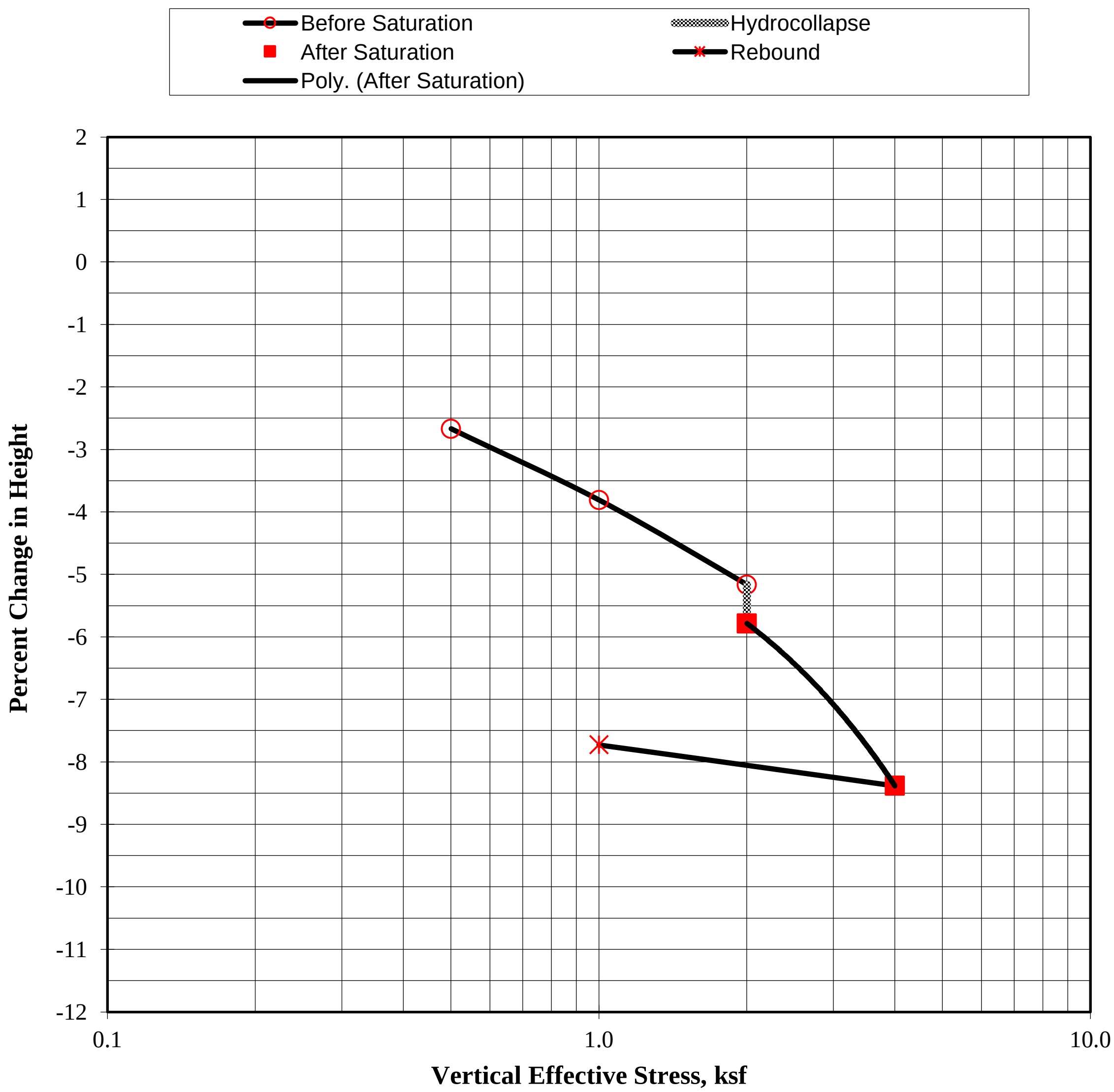
Sandy Fat Clay (CH)

Ring Sample

Initial Dry Density: 73.7 pcf
Initial Moisture: 42.9%
Specific Gravity: 2.67
Initial Void Ratio: 1.261

Hydrocollapse: 0.6% @ 2.0 ksf

% Change in Height vs Normal Pressure Diagram



File No.: 302451-002
Lab No.: 20-196

November 30, 2020

CONSOLIDATION TEST **ASTM D 2435 & D 5333**

Proposed Building 6 Mod
B3 @ 5 feet

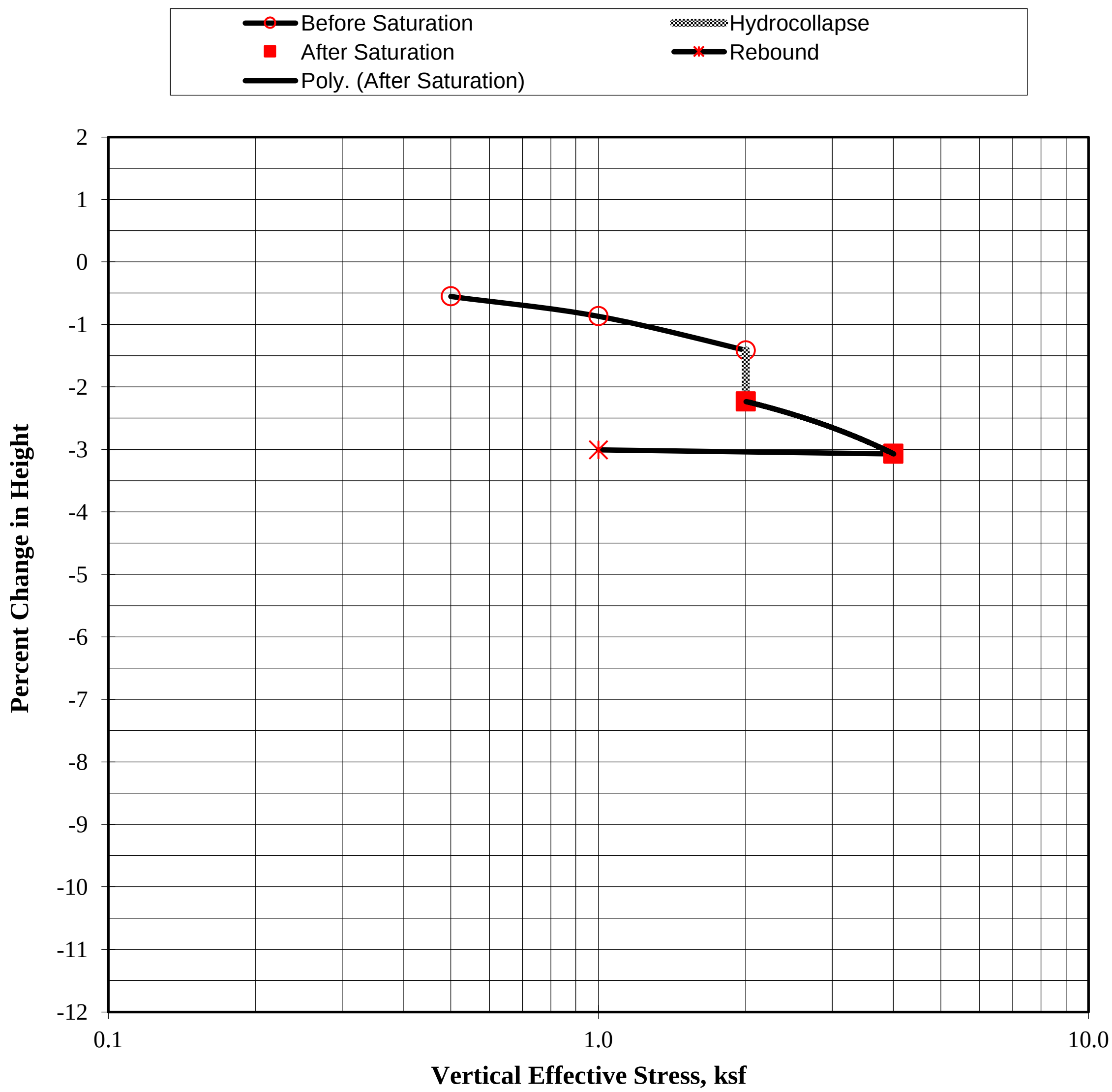
Silty Sand (SM)

Ring Sample

Initial Dry Density: 89.5 pcf
Initial Moisture: 9.0%
Specific Gravity: 2.67
Initial Void Ratio: 0.862

Hydrocollapse: 0.8% @ 2.0 ksf

% Change in Height vs Normal Pressure Diagram



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CONSOLIDATION TEST

ASTM D 2435 & D 5333

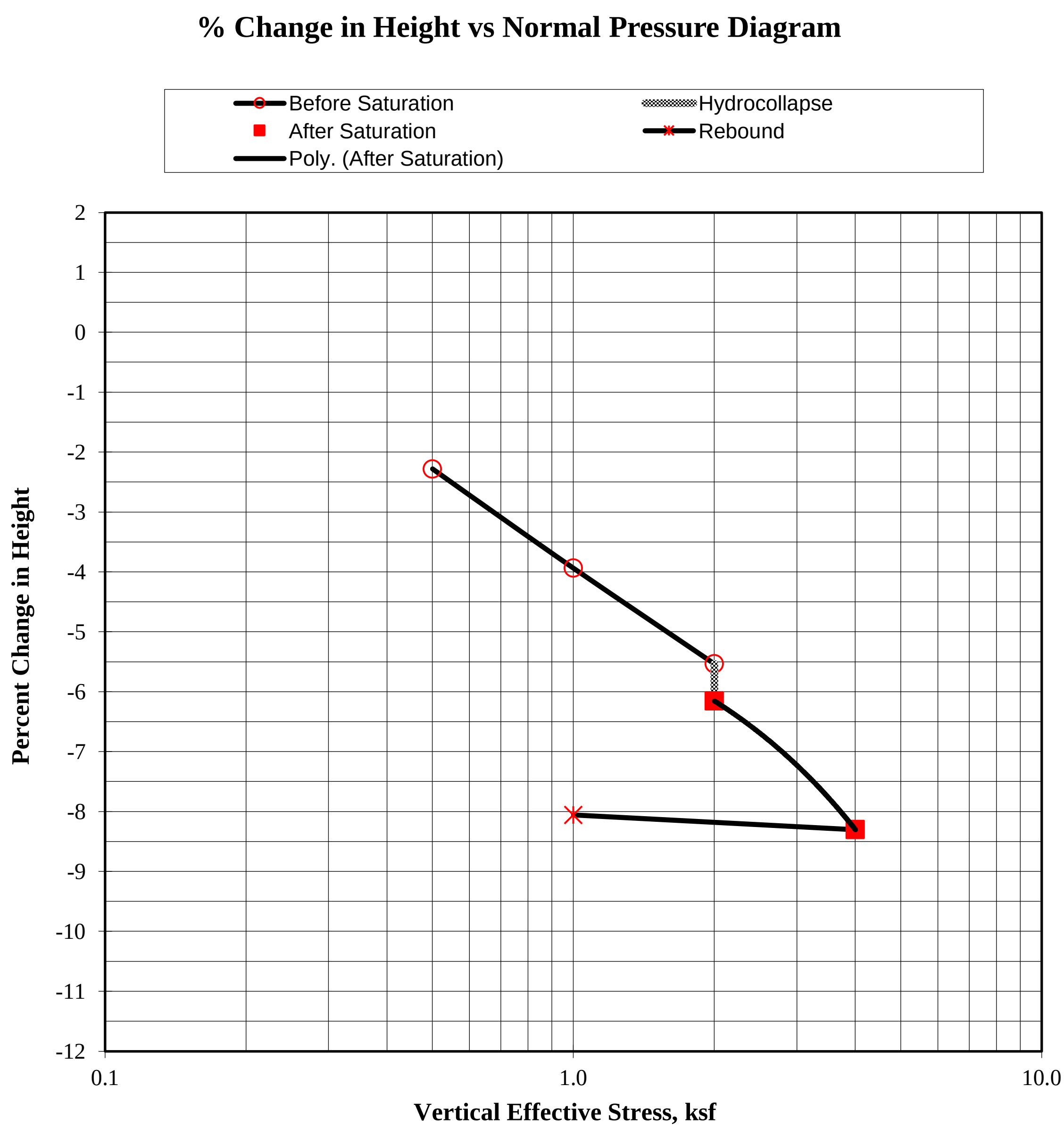
Proposed Building 6 Mod
B3 @ 10 feet

Sandy Fat Clay (CH)

Ring Sample

Initial Dry Density: 80.9 pcf
Initial Moisture: 38.0%
Specific Gravity: 2.67
Initial Void Ratio: 1.060

Hydrocollapse: 0.6% @ 2.0 ksf



File No.: 302451-002
Lab No.: 20-196

November 30, 2020

CONSOLIDATION TEST **ASTM D 2435 & D 5333**

Proposed Building 6 Mod
B3 @ 15 feet

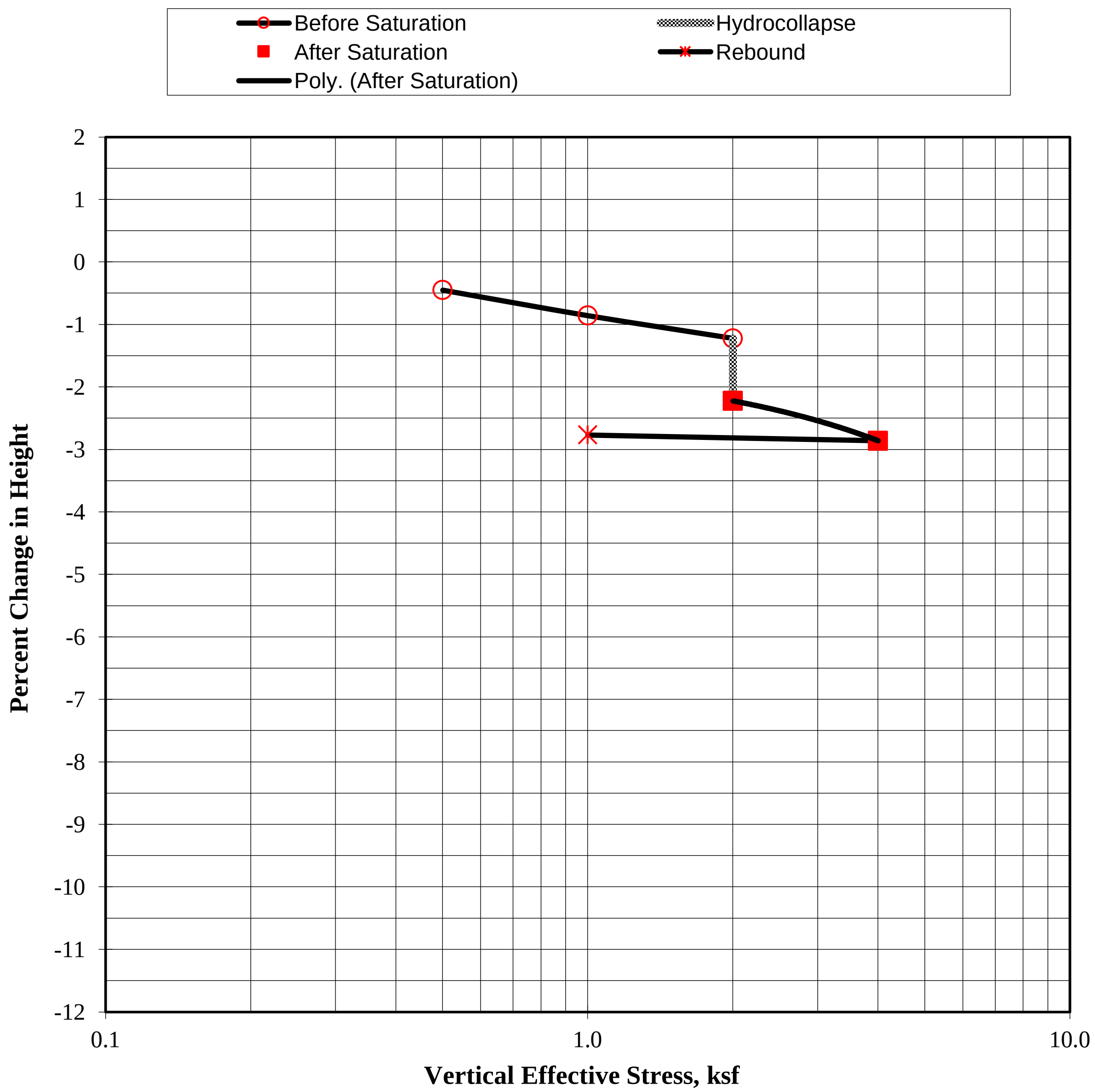
Silty Sand (SM)

Ring Sample

Initial Dry Density: 98.3 pcf
Initial Moisture: 7.5%
Specific Gravity: 2.67
Initial Void Ratio: 0.696

Hydrocollapse: 1.0% @ 2.0 ksf

% Change in Height vs Normal Pressure Diagram



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EXPANSION INDEX **ASTM D-4829**

Job Name: Proposed Building 6 Mod
Sample ID: B2 @ 7 1/2 feet
Soil Description: Sandy Fat Clay (CH)

Initial Moisture, %:	16.3	
Initial Compacted Dry Density, pcf:	90.9	
Initial Saturation, %:	52	*
Final Moisture, %:	43.4	
Volumetric Swell, %:	9.6	
Expansion Index, EI:	**98**	**High**

EI	ASTM Classification
0-20	Very Low
21-50	Low
51-90	Medium
91-130	High
>130	Very High

File No.: 302451-002
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MAXIMUM DRY DENSITY / OPTIMUM MOISTURE ASTM D 1557 (Modified)

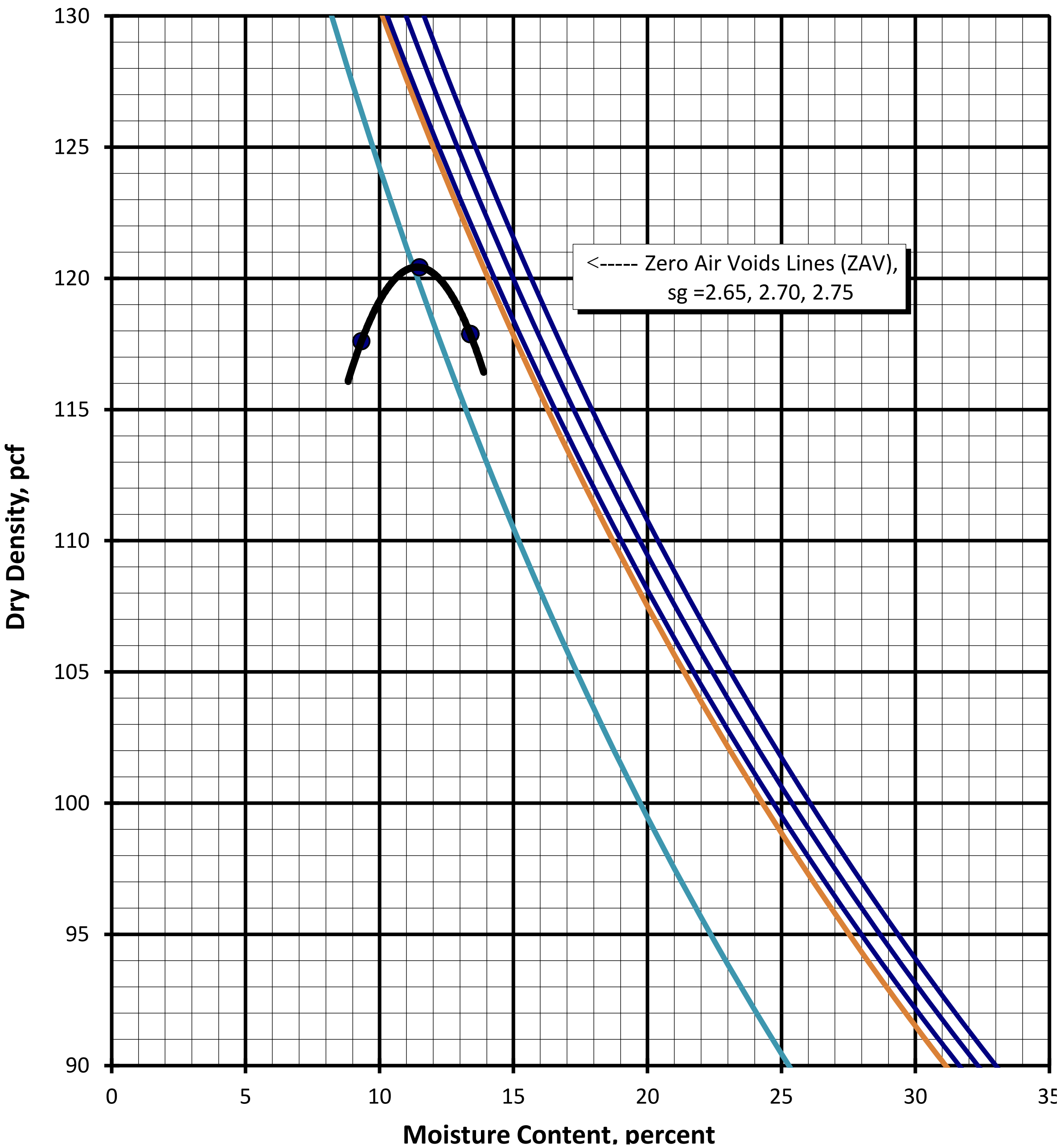
Job Name: Proposed Building 6 Mod
Sample ID: 1
Location: B2 @ 0-5 feet

Procedure Used: A
Preparation Method: Moist
Rammer Type: Mechanical

Description: Silty Sand (SM)

Maximum Dry Density: **120.4 pcf**
Optimum Moisture: **11.5%**
Corrected for Oversize (ASTM D4718)

Sieve Size	% Retained (Cumulative)
3/4"	0.2
3/8"	3.6
#4	8.0



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SOIL CHEMICAL ANALYSES

Job Name:	Proposed Building 6 Mod
Job No.:	302451-002
Sample ID:	B2
Sample Location:	0-5 feet
Resistivity (Units)	
as-received (ohm-cm)	12,800
saturated (ohm-cm)	6,000
pH	7.9
Electrical Conductivity (mS/cm)	0.12
Chemical Analyses	
Cations	
calcium Ca^{2+} (mg/kg)	104
magnesium Mg^{2+} (mg/kg)	11
sodium Na^{1+} (mg/kg)	31
potassium K^{1+} (mg/kg)	16
Anions	
carbonate CO_3^{2-} (mg/kg)	45
bicarbonate HCO_3^{1-}(mg/kg)	192
fluoride F^{1-} (mg/kg)	5.1
chloride Cl^{1-} (mg/kg)	13
sulfate SO_4^{2-} (mg/kg)	34
phosphate PO_4^{3-} (mg/kg)	ND
Other Tests	
ammonium NH_4^{1+} (mg/kg)	ND
nitrate NO_3^{1-} (mg/kg)	16
sulfide S^{2-} (qual)	na
Redox (mV)	na

Note: Tests performed by Subcontract Laboratory:
HDR Engineering, Inc.
431 West Baseline Road
Claremont, California 91711 Tel: (909) 962-5485

mg/kg = milligrams per kilogram (parts per million) of dry soil.
Redox = oxidation-reduction potential in millivolts
ND = not detected
na = not analyzed
T.O.P. = top of pipe

Resistivity per ASTM G187, Cations per ASTM D6919, Anions per ASTM D4327, and Alkalinity per APHA 2320-B. Electrical conductivity in millisiemens/cm and chemical analyses were made on a 1:5 soil-to-water extract.

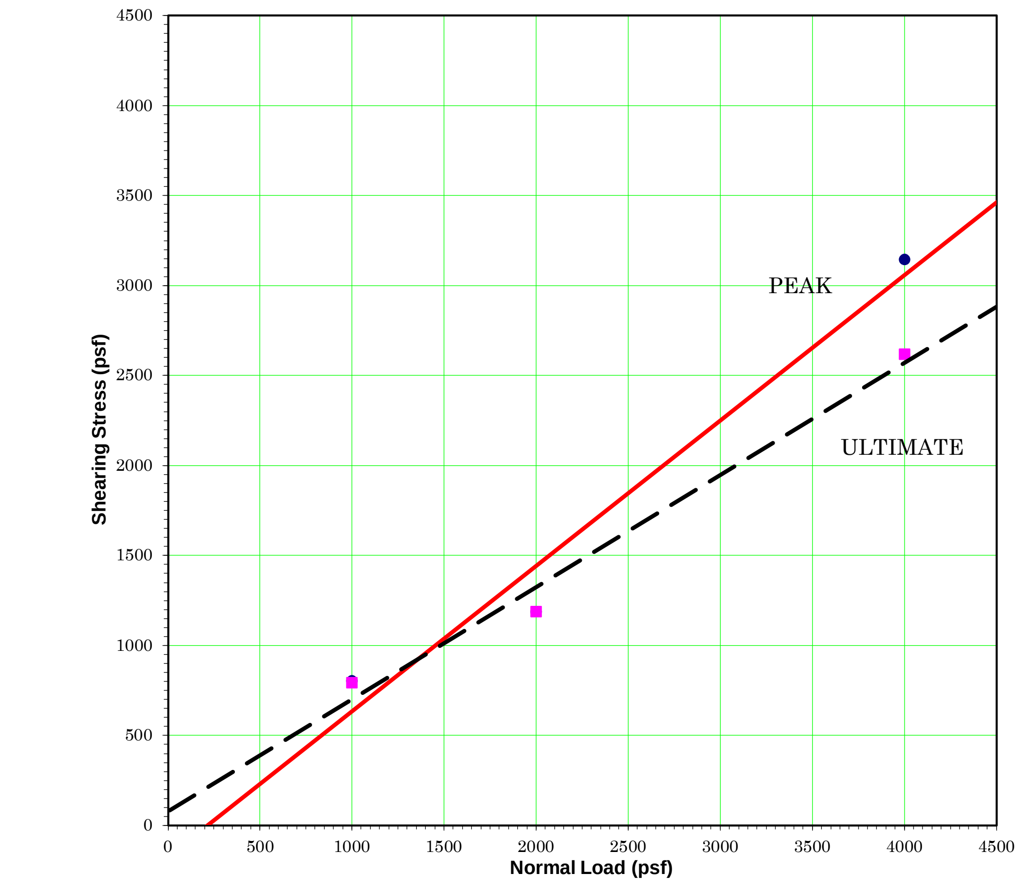
General Guidelines for Soil Corrosivity

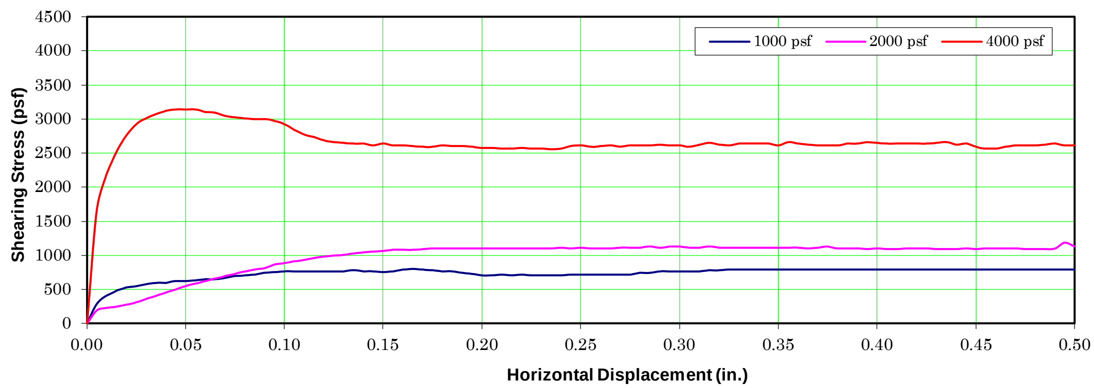
Chemical Agent	Amount in Soil	Degree of Corrosivity
Soluble	0 -1,000 mg/Kg (ppm) [0-.1%]	Low
Sulfates[1]	1,000 - 2,000 mg/Kg (ppm) [0.1-0.2%]	Moderate
	2,000 - 20,000 mg/Kg (ppm) [0.2-2.0%]	Severe
	> 20,000 mg/Kg (ppm) [>2.0%]	Very Severe
Resistivity[2]	0- 900 ohm-cm	Very Severely Corrosive
(Saturated)	900 to 2,300 ohm-cm	Severely Corrosive
	2,300 to 5,000 ohm-cm	Moderately Corrosive
	5,000-10,000 ohm-cm	Mildly Corrosive
	10,000+ ohm-cm	Progressively Less Corrosive

1 - General corrosivity to concrete elements. American Concrete Institute (ACI) Water Soluble Sulfate in Soil by Weight, ACI 318, Tables 4.2.2 - Exposure Conditions and Table 4.3.1 - Requirements for Concrete Exposed to Sulfate-Containing Solutions. *It is recommended that concrete be proportioned in accordance with the requirements of the two ACI tables listed above (4.2.2 and 4.3.1). The current ACI should be referred to for further information.*

2 - General corrosivity to metallic elements (iron, steel, etc.). Although no standard has been developed and accepted by corrosion engineering organizations, it is generally agreed that the classification shown above, or other similar classifications, reflect soil corrosivity. Source: Corrosionsource.com. The classification presented is excerpted from ASTM STP 1013 titled "Effects of Soil Characteristics on Corrosion" (February, 1989)

3 - Earth Systems does not practice corrosion engineering. Results should be reviewed by an engineer competent in corrosion evaluation, especially in regard to nitrites and ammonium.





DIRECT SHEAR DATA*

Sample Location: B2 @ 0-5 feet

Material: Silty Sand (SM)

Remolded to 90% of Max Dry Density (pcf): 108.4

	Initial	Final
Moisture Content (%):	11.5	15.4
Saturation (%):	80	100
	Peak	Ultimate
ϕ Angle of Friction (degrees):	39	32
c Cohesive Strength (psf):	0	70

Test Type: Peak and Ulitimate

Shear Rate (in/min): 0.007

* Test Method: ASTM D-3080

DIRECT SHEAR TEST

Proposed Building 6 Mod

Riverside, California

Earth Systems Pacific

11/30/2020	302451-002