Appendix B
Geotechnical Report
(Available for review at City Hall)

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Attention: Rich Malacoff, AICP, Principal Planner

Project: **Proposed Santa Cruz Project (Six Lots)**
NWC of Calle Santa Cruz and Calle De Ricardo
Palm Springs, Riverside County, California

Subject: **Geotechnical Engineering Report**

Earth Systems Pacific [Earth Systems] is pleased to submit this geotechnical engineering report for a proposed development located at the northwest corner Calle Santa Cruz and Calle De Ricardo in Palm Springs, Riverside County, California. The intent of this report is to provide geotechnical information for the development on six (6) lots as shown in Figure 1, page 1.

This report completes the geotechnical report scope of services in accordance with our agreement (BER 23-9-002) with an authorization date of September 19, 2023. Other geotechnical related services that may be required, such as plan reviews, responses to agency inquiries, and grading observation are additional services and will be billed according to the agreed upon 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 it is vital to the understanding of this report.
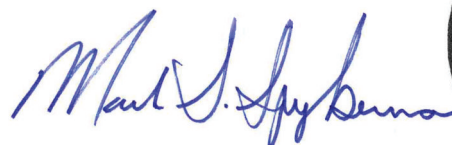
We appreciate the opportunity to provide our professional services. Please contact our office if there are any questions or comments concerning this report or its recommendations.
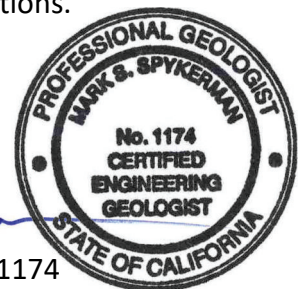
Respectfully submitted,
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NWC of Calle Santa Cruz and Calle De Ricardo
Palm Springs, Riverside County, California

Section 1
INTRODUCTION

1.1 Project Information

This geotechnical engineering report has been prepared for a proposed project located on the northeast corner of Calle Santa Cruz and Calle De Ricardo in Palm Springs, Riverside County, California. Six lots are approximated in Figure 1 below and we were provided Assessor Parcel Numbers (APN): 680-072-022, 680-072-021, 680-072-020, 680-072-019, 680-072-014, and 680-072-013. Based on the APN, the smaller areas are 0.14 acres each and the larger lots are 0.27 acres each providing a total area of all six lots of approximately 1.1 acres.

At the time of drafting this report, additional information was not provided to Earth Systems; therefore, we made assumptions to the project, and they are summarized here as the basis for the recommendations in this report. We anticipate that the structures will be one to two stories and will be of concrete, masonry, metal, wood, or light frame construction supported on conventional shallow foundations. We assume loadings will not be more than 54 kips for column footings and 3 kip per linear foot (klp) for wall foundations, see Section 5.3 for additional details. As the basis for the foundation recommendations, all loading is assumed to be dead plus actual live load. If actual structures or structural loading differs or exceeds these assumed values, we will need to reevaluate the given recommendations. We have assumed no basement levels. Site grading is anticipated to achieve finished grade with cut, fill, and slopes anticipated to be less than five feet, excluding remedial grading.

Figure 1   Google Overview of Six Lots

1.2 Site Description

We understand the proposed project is located on vacant land south of Ramon Road and west of

Gene Autry Trail in Palm Springs, Riverside County, California. The accessor parcel numbers (APNs) are provided in the Section "Project Information". One latitude and longitude of the project area that is generally nearest to San Andreas fault is approximately 33.8157°N, 116.4986°W. The project is bounded by Ramon Road to the north, vacant land to the east, Calle De Ricardo the south, and Calle Santa Cruz to the west.

Per the APN report, the total gross area is 1.1 acres. There is not a defined access to the site as of the preparation of this report. Topographically, the site is relatively flat and based on google imagery the site elevations range from approximately 384 feet above mean sea level to 387 feet above mean sea level. Drainage is assumed to be sheet flow to the southwest flowing onto the public roadways, see Plate 1 and 2 in the Appendix A.

The lot is currently vacant with minor desert vegetation and alluvium deposits (sand and gravel). Fugitive dust preventive material (green shown in Figure 1) was observed at the site. We researched past use of the site via select documents and we did find past use history. As will be shown in site reconnaissance, the southeast area once had a structure on it. During our site visits, loose sandy surface soil was evident.

We performed a Dig Alert procedure for our borings and all borings were cleared of existing utilities. We did find Dig Alert marks running along the middle of the project shown in the red line in Figure 1.

1.3   Purpose and Scope of Services

The purpose for our services was to evaluate the site soil and groundwater conditions at our exploration locations and to provide professional opinions and recommendations regarding the proposed development.

The scope of services included:

Task 1 – Literature and Photograph Reviews: We began our services by reviewing select geologic and geotechnical literature pertaining to the project. This included a review of various hazard, fault and geologic maps prepared by the U.S. Geological Survey, California Geological Survey, Riverside County, and other governmental agencies as they relate to the site area. We also reviewed select historical aerial photographs.

Task 2 – Utility Clearance, USA Dig Alert: Earth Systems pre-marked each proposed exploration locations for underground utility search companies' verification. On September 20, 2023, exploration locations were marked, and 811 Dig Alert was contacted and informed of our intent to explore the site. Earth Systems obtained a positive response from Dig Alert on September 27, 2023. The client specific boring locations are shown in the Exploration Map found in Appendix A.

Task 3 – Field Exploration: The on-site soil profiles were explored by means of drilling and sampling within four (4) exploratory borings. The borings extended from approximately 11½ to 51½ feet below the ground surface. The exposed soil profiles were examined relative to soil conditions and the presence or absence of groundwater. Samples of the surface and subsurface materials were taken at various intervals, logged by our representative, and returned to our laboratory.

<u>Task 4 – Laboratory Testing</u>: Laboratory tests were performed on selected soil samples obtained from the site. Geotechnical testing included moisture content, dry unit weight, maximum density/optimum moisture content, sieve analysis, consolidation/collapse potential, and R-Value. Testing was performed in general accordance with American Society for Testing and Materials [ASTM] or appropriate test procedure. Selected samples were 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 engineer in designing an appropriate corrosion control plan for the project.

<u>Task 5 – Analysis and Report</u>: This report was prepared summarizing our geologic and geotechnical findings in accordance with the 2022 California Building Code. This 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.

➢ 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 "General Procedure" seismic analysis including geotechnical seismic design coefficients in accordance with the 2022 CBC.

➢ Recommendations for imported fill (if required) for use in compacted fills.

➢ Recommendations for foundation design including parameters for shallow foundations and building pad and subgrade preparation.

➢ Anticipated total and differential settlements for the recommended foundation system.

➢ Preliminary recommendations for the mitigation of seismic induced settlement.

➢ Recommendations for lateral earth pressures (active, at-rest, and passive) for retaining walls, including drainage requirements, coefficients of friction and seismic earth pressures.

➢ Recommendations for site preparation, earthwork, and fill compaction specifications.

➢ Discussion of anticipated excavation conditions, including shrinkage and/or bulking.

➢ Recommendations for underground utility trench backfill and import soils.

➢ Recommendations for stability of temporary trench excavations.

➢ Recommendations for slabs-on-grade (building slabs and walkways), including recommendations for reducing the potential for moisture transmission through interior slabs.

➢ Asphalt concrete pavement and Portland cement driveway recommendations for auto, driveways, and fire lanes (Traffic Index values of 5 for parking areas and 7 for driveways and 5 for fire lanes).

➢ Recommendations for collapsible or expansive soils (if applicable).

➢ A discussion of the corrosion potential of the near-surface soils encountered during our field exploration.

➢ An appendix, which will include a summary of the field exploration and laboratory testing program.

Section 2
METHODS OF EXPLORATION AND TESTING

2.1   Field Exploration

The subsurface exploration program for the subject site included advancing four (4) geotechnical exploratory borings on October 4, 2023. The borings were drilled to depths ranging from about 11½ to 51½ feet below the existing ground surface to observe soil profiles, groundwater, and obtain samples for laboratory testing. The geotechnical borings were drilled using approximately 8-inch outside diameter hollow-stem augers, powered by a Mobile B-61, truck-mounted drill rig, operated by California Pacific Drilling of Calimesa, California, under subcontract to Earth Systems. The boring locations are shown on the Exploration Location Map, Plate 2, in Appendix A. The locations shown are approximate, established by pacing and GPS locating (+/-15 feet) based upon landmarks and the provided plans.

Earth Systems staff maintained a log of the subsurface conditions encountered and obtained samples for visual observation, classification and laboratory testing. Soils were logged in general accordance with the Unified Soil Classification System. Our typical sampling interval within the borings was approximately every 2½ to 5 feet to the full depth explored; however, sampling intervals were adjusted depending on the materials encountered. Samples were obtained within the test borings using a Standard Penetration [SPT] sampler (ASTM D 1586) and a Modified California [MC] ring sampler (ASTM D 3550 with those similar to ASTM D 1586). The SPT sampler has a 2-inch outside diameter and a 1.38-inch inside diameter. The MC sampler has a 3-inch outside diameter and a 2.4-inch inside diameter. Samplers were mounted to the end of screwed drill rod and were driven using a 140-pound automatic hammer falling 30 inches.

Design parameters provided by Earth Systems in this report have considered an estimated approximate 80% hammer efficiency for the B-61 drill rig based on data provided by the drilling subcontractor. The number of blows necessary to drive either a SPT sampler or a MC type ring sampler within the borings was recorded. 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 SP117A) is considered satisfactory. A value of 0.63 was applied in our calculations for this project.

Bulk samples of the soil materials were obtained from the drill auger cuttings, representing a mixture of soils encountered at the depths noted. Following drilling, sampling, and logging the borings were backfilled with native cuttings and tamped upon completion.

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 gradational. In reviewing the boring logs and legend, the reader should recognize that the legend is intended as a guideline only, and there are a number of conditions that may influence the soil characteristics as observed during drilling. These include, but are not limited to, the presence of cobbles or boulders, cementation, variations in soil moisture, presence of groundwater, and

other factors. The logs present field blowcounts per 6 inches of driven embedment (or portion thereof) for a total driven depth attempted of 18 inches. The blowcounts are uncorrected (i.e. not corrected for overburden, sampling, etc.). Consequently, the user must correct the blowcounts 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. Selected samples were 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 collected (ASTM D 2937 & 2216).

- Maximum density 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 sieve analysis procedures (ASTM D 6913).

- Consolidation/Collapse Potential to evaluate the compressibility and hydroconsolidation (collapse) potential of the soil upon wetting (ASTM D 5333).

- Screening Level Chemical Analyses (Soluble Sulfates and Chlorides (ASTM D 4327), pH (ASTM D 1293), and Electrical Resistivity/Conductivity (ASTM D 1125) to evaluate the potential for adverse effects of the soil on concrete and steel.

- R-Value to evaluate the empirical shear strength of the site soils in relation to pavement loading characteristics (California Test 301).

Section 3
DISCUSSION

3.1   Soil Conditions

The field exploration indicates that site soils consist predominantly of Poorly Graded Sand with Silt, Poorly Graded Sand, Silty Sand and to a lesser degree Well Graded Sand (Unified Soils Classification System symbols of SP-SM, SP, SM and SW) to the maximum depth of exploration of 51½ feet below the ground surface. Some gravel was encountered and in boring B4 at 20 feet a large gravel fragment was found in the sampler—it was considered a cobble that the sampler went through. In general, most of the site is covered with naturally deposited soils that consist of Quaternary alluvium and dune sand). Some undocumented fill (af) is present at the southeast corner of the site and where demolition of previous structures were present. In Section 3.3 "Reconnaissance" we found past use of the site that had a building on it. Appendix A presents the boring logs which have more detail.

Consistency: Coarse grained soil (sand and gravel) was observed at this site. The observed sandy soils in the upper 5 feet were generally loose to medium dense. From 5 to 15 feet below the ground surface, consistency was generally "medium dense". Beyond 15 feet the soil consistency varied between medium dense and very dense to the deepest depth explored (51½ feet). Although fine grained soil layers (silts and clays) were not found, at boring B-4 at a depth of 50 feet, silt and clay lenses having thickness of approximately ⅛ inch were observed.

Densities and Saturation: For the upper 5 feet of soil profile, laboratory dry density testing averaged 103 pounds per cubic foot (pcf) and varied from 102 to 104 (pcf). Saturation levels varied from 9 to 20 percent by weight. Note, 85 percent is typically considered nearly saturated and a sign of a recent water table. Fifty (50) percent is near optimum moisture content.

For the soil profile below a 5-foot depth, laboratory dry density testing averaged 104 pounds per cubic foot (pcf) and varied from 101 to 107 (pcf). Saturation levels varied from 2 to 11 percent by weight.

Fugitive Dust: Blow sand was observed on the site. The site lies within a recognized blow sand hazard area. Fine particulate matter (PM_{10}) can create an air quality hazard if dust is blowing. Watering the surface, planting grass or landscaping, or placing hardscape normally mitigates this hazard.

3.2   Groundwater

Earth Systems reviewed both current and historic groundwater levels near the project site. For this report, we used information dated back to 1968 for use as historic information. We also provide a brief discussion of the moisture contents of the soils found during the exploration and the ability of water features to produce a perched water table.

Field Exploration Information: Free groundwater was not encountered at our explorations conducted on October 4, 2023; observations indicated that moisture contents varied from dry to damp. We performed 17 moisture content tests of the soil samples recovered and obtained values varying between 0% to 5% at depths ranging between 2½ feet and 51½ feet below the

ground surface (bgs). The average moisture content was 2 percent, note the maximum density optimum moisture content is approximately 12 percent. Perched water conditions were not observed.

Nearby Well Readings: We researched wells in the vicinity of the project using http://www.water.ca.gov/waterdatalibrary/ and found a well (04S05E09B001S) located approximately 3½ miles northeast of the project site. The well head elevation of 398 feet mean sea level (MSL), and groundwater elevations that varied from 161 to 209 feet MSL (or 237 to 189 feet below ground surface) between 2011 and February 2021, respectively. The subject site has a surface elevation approximately 385 feet above MSL. As such, the depth to groundwater is more than 150 feet below the ground surface based on the researched data and has been since 2011.
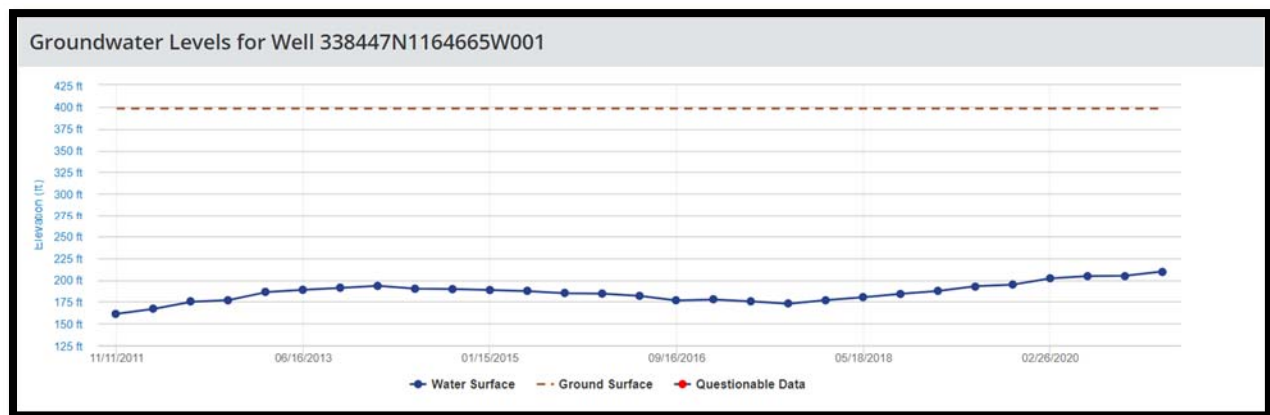
Figure 2   State Well Monitoring Data from year 2011 to 2021

Historic Groundwater Table: Earth Systems researched historic groundwater maps for the Coachella Valley. Based on a map published in 1964 (DWR, 1964), the approximate historic groundwater elevation at the project area is approximately 205 feet MSL (DWR, 1964). Per Section 1.2 of this report the project site elevation is approximately 385 AMSL. The difference between historic groundwater and project elevation is approximately 180 feet. Therefore, based on historic groundwater table, the depth to groundwater is more than 150 feet below the ground surface (bgs).

Estimated Project Groundwater Depth: Based on the data provided above, Earth Systems estimates the highest groundwater depth from the "Historic Groundwater Table" and nearby well data to be more than 150 feet below the ground surface. Water levels may fluctuate with precipitation, irrigation, drainage, regional pumping from wells, site grading, and nearby faults.

3.3   Site Reconnaissance

Earth Systems personnel visited the site on multiple days: September 20 and October 4, 2023. Earth Systems personnel also reviewed select historic aerial photographs of the project site. A summary of our findings is presented below:

Field Observations:

1. Disturbed area (probable grading) along the southeast portion was observed.
2. Fugitive dust control stabilizer was observed on the site.
3. Loose material at surface, see photo with tire tracks.
4. Dig alert painted notes on the ground showing underground utilities.

Figure 3   Photo taken October 4, 2023, at Northeast Corner of Site

Historical aerial photographs:

a. A 1959 aerial photo shows:
   o A structure existing on the southeast portion of the site.
   o The Palm Springs Airport Exists.
b. A 1972 aerial photo shows:
   o A structure exists east of the structure noted in the 1959 photo.
c. A 1996 to 2002 aerial photo shows the structures identified in 1959 and 1972, see Appendix.
d. A 2005 aerial photo shows the structures identified in 1959 and 1972 photos have been removed.
e. 2019 aerial photo shows grading has been performed on the site and green fugitive dust control stabilizer has been applied to the surface.
f. 2021 aerial photo shows additional grading has disturbed the surface and green fugitive dust control stabilizer has been applied to the surface.

3.4   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 can be 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 12.5 feet below the ground surface indicated a collapse potential on the order of 0.4 to 0.9 percent. The goal of the collapse testing was to identify soils and densities where the potential for collapse decreased to accepted levels if the test can produce samples tested that are relatively undisturbed. This accepted level is defined as where on-site soils had collapse potential less than 1% or the estimated relative compaction is greater or equal to 85 percent compaction, which is the typical standard of care based on the above Table 1 (1%) or where soil collapse becomes a concern for structural soils (Less than 108 pcf dry density) (County of Los Angeles, 2013).

Based on the field and laboratory testing performed, Earth Systems provides key items of interest that supports Earth Systems recommendations regarding collapse potential at this site:

1. Two soil types (SP-SM and SM) were tested for collapse.

2. Soils are generally granular in nature and no significant cementation was observed. These soils were found to be disturbed very easily.

3. Pinhole voids were not observed.

4. Soil collapse can be directly related to in-place density (relative compaction).

5. Some samples had dry densities of less than 108 pounds per cubic feet having a collapse potential of less than 1 percent; however, this 108 pcf relates to a maximum density of approximately 127 pcf.

6. The project soil samples have a maximum density of 113.7 pcf; therefore, actual dry densities of 97 pcf should produce a potential collapse potential of no problem.

Based on the above criteria and our field and laboratory findings, we estimate there is a "No Problem" collapse potential from soil layers between 2½ and 15 ft bgs. Assuming the grading is accomplished according to Section 5.1 of this report (4-foot over-excavation), we estimate the collapse potential is low.

3.5   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. Based on visual classification of the sandy soils encountered, the Expansion Index of the onsite upper soils is "very low", as defined by ASTM D 4829. Samples of building pad soils should be observed and/or tested during grading to confirm or modify these findings.

3.6   Corrosion Potential

One sample of the near-surface soils was tested for potential corrosion of concrete and ferrous metals. The test was conducted in general accordance with ASTM procedures 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" (February 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 (presented in Appendix B) show pH value of 7.9, chloride content of 7 ppm, sulfate content of 21 ppm and minimum resistivity of 10,000 Ohm-cm. Although Earth Systems does not

practice corrosion engineering, the corrosion values from the soil 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] 318, Section 4.3. As such, we recommend an engineer competent in corrosion mitigation review these results and design corrosion protection appropriately. The onsite values can potentially change based on several factors, such as importing soil from another job site and the quality of water used during construction and subsequent landscape irrigation.

3.7   Geologic Setting

Regional Geology: The site lies along the southwest margin of the northern-most Coachella Valley, a part of the Colorado Desert geomorphic province. Just to the west are the San Jacinto Mountains which are part of the Peninsular Ranges geomorphic province. A significant feature within the Colorado Desert geomorphic province is the Salton Trough. The Salton Trough is a large northwest-trending structural depression that extends approximately 180 miles from the San Gorgonio Pass to the Gulf of California. Much of this depression in the area of the Salton Sea is below sea level.

The Coachella Valley forms the northerly part of the Salton Trough and contains a thick sequence of Miocene to Holocene sedimentary deposits. Mountains surrounding the Coachella Valley include the Little San Bernardino Mountains on the northeast, foothills of the San Bernardino Mountains on the northwest, and the San Jacinto and Santa Rosa Mountains on the southwest. These mountains expose primarily Precambrian metamorphic and Mesozoic granitic rocks. Within the immediate site area, native geologic lithologic units consist of a mix of younger (Holocene) alluvium, fluvial deposits, and dune sand.

Active faults in the immediate vicinity (within 25 miles) of the site include the San Andreas, San Jacinto, Pinto Mountain, Landers, Burnt Mountain, and Eureka Peak faults (See Table A-1). The closest active faults are multiple traces of the San Andreas fault zone including the Garnet Hill, Banning, and the Mission Creek faults that traverse along the northeast margin of the valley. The site does not lie within a currently designated Alquist-Priolo Earthquake Fault zone or Riverside County designated fault zone. The buried, and in-active Palm Canyon fault is located approximately 2.6 miles west of the site.

Local Geology: The project site is located within the northwest portion of the Coachella Valley, near the confluence of Palm Canyon with the Coachella Valley and the Whitewater River. The steep eastern foothills of the San Jacinto Mountains are about three miles to the west and southwest.   The site is approximately one mile southwest of the Whitewater River channel and is geologically located on the distal eastern margin of the Tahquitz and Palm Canyon alluvial fans. Sediments within this area consist of fine- to coarse-grained sands and gravels with interbedded clays, silts, gravels, and cobbles of aeolian (wind-blown) and alluvial (water-laid) origin. The site is not located in a frost zone.

The project site is located in a mapped area where surficially, a mix of alluvial fan deposits associated with deposition of the Tahquitz Canyon and Palm Canyon alluvial fans and adjacent fluvial deposits of the White Water River.   Aeolian (dune) sand deposits are layered between the alluvial deposits.

Palm Canyon is a fault controlled geomorphic feature largely influenced by differential erosion along the Palm Canyon fault which trends to the north-northwest towards Palm Springs. The fault appears to disrupt only the Triassic metamorphic rocks and is mapped as being buried by alluvial deposits of Palm Springs. The Palm Canyon fault is not considered "active" by State definition and is not zoned within a currently designated Alquist-Priolo Earthquake Fault Zone.

3.8   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.

3.8.1   Primary Seismic Hazards

Seismic Sources: Several active faults or seismic zones lie within 50 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 regional faults including the San Andreas and San Jacinto faults.

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] Fault Activity Map (2010); however, no active faults are mapped in the immediate vicinity of the site (Plate 3). The closest active faults are traces/segments of the San Andreas fault zone, including the Garnet Hill, Banning, and Mission Creek faults, located approximately 3 to 8 miles northeast of the site. The mapped buried/conjectural trace of the Palm Canyon fault is located 2.6 miles west of the project, although this fault, if actually present, is not considered active. 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.

Review of select aerial photographs reveal that the site is located in an area predominated by alluvial fan development from Palm Canyon and other secondary drainages from the San Jacinto Mountains. No evidence of fault related lineaments were noted.

Historic Seismicity: The site is located within an active seismic area in southern California where large numbers of earthquakes are recorded each year. Approximately 40 magnitude 5.5 or greater earthquakes have occurred within 60 miles of the project since 1856 (See Table A-2 for an abbreviated list of historic earthquakes).

Six notable historic seismic events (5.9 M or greater) have significantly affected the Coachella Valley in the last 100 years. They are as follows:

- *Desert Hot Springs Earthquake* – On December 4, 1948, a magnitude 6.5 M_L (6.0M_W) earthquake occurred east of Desert Hot Springs. This event was strongly felt in the Palm Springs area.
- *Palm Springs Earthquake* – A magnitude 5.9 M_L (6.2M_W) earthquake occurred on July 8, 1986 in the Painted Hills, causing minor surface creep of the Banning segment of the San Andreas

fault. This event was strongly felt in the Palm Springs area and caused structural damage, as well as injuries.

- *Joshua Tree Earthquake* – On April 22, 1992, a magnitude 6.1 M_L (6.1M_W) earthquake occurred in the mountains 9 miles east of Desert Hot Springs. Structural damage and minor injuries occurred in the Coachella Valley as a result of this earthquake.

- *Landers and Big Bear Earthquakes* – Early on June 28, 1992, a magnitude 7.5 M_S (7.3M_W) earthquake occurred near Landers, the largest seismic event in Southern California for 40 years. Surface rupture occurred just south of the town of Yucca Valley and extended some 43 miles toward Barstow. About three hours later, a magnitude 6.6 M_S (6.4M_W) earthquake occurred near Big Bear Lake. No significant structural damage from these earthquakes was reported in the Palm Springs area.

- *Hector Mine Earthquake* – On October 16, 1999, a magnitude 7.1 M_W earthquake occurred on the Lavic Lake and Bullion Mountain faults north of Twentynine Palms. While this event was widely felt, no significant structural damage has been reported in the Coachella Valley.

Seismic Risk: 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 which estimate a 24 to 28% conditional probability that a magnitude 6.7 or greater earthquake may occur in 30 years (2014 as base year) along the nearby San Andreas fault (San Gorgonio Pass-Garnet Hill subsection). For the nearest segment (Anza) of the San Jacinto fault, the conditional probability for a similar magnitude earthquake is about 14%. Recent estimates suggest a nearly 98% probability of a nearby magnitude 5 earthquake in the next 50 years.

The primary seismic risk at the site is a potential earthquake along the San Andreas fault that is about 3 to 8 miles from the site and is considered as fault Type A per the CGS. Geologists believe that the San Andreas fault has characteristic earthquakes that result from rupture of each fault segment. The estimated mean characteristic earthquake is magnitude 7.7 for the Southern Segment of the fault (USGS, 2002). However, recent paleoseismic data and standard of practice suggests a maximum magnitude of 7.9 be used for analysis, assuming a multi-segment rupture event.

The Coachella segments have the longest elapsed time since rupture of any part of the San Andreas fault. The last rupture occurred about 1680 AD, based on dating by the USGS near Indio (WGCEP, 2008). This segment has also ruptured on about 1020, 1300, and 1450 AD, with an average recurrence interval of about 220 years. The San Andreas fault may rupture in multiple segments, producing a higher magnitude earthquake. Recent paleoseismic studies suggest the San Bernardino Mountain Segment to the north and the Coachella Segment may have ruptured together in 1450 and 1690 AD (WGCEP, 1995).

Secondary seismic hazards related to ground shaking include soil liquefaction, ground subsidence, tsunamis, and seiches. Other hazards may include flooding and slope instability. The site is far inland, so the hazard from tsunamis is non-existent.

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. At the present time, several water storage/treatment reservoirs are located up and downgradient and to west and south of the site. Existing drainages and residential improvements would direct overflow or tank discharge away from the project, and flood control channels would dissipate or redirect the flow. Reservoir failure is dependent upon the structural design and integrity of the reservoirs, which is beyond the scope of our services. In addition adjacent properties do include pools. Seiching of pool water will probably be contained within the immediate area of the pools. Hazards from seiches and reservoir failure are considered low at this time, assuming a catastrophic failure of the regional water storage reservoirs.

Slope Stability: The site is relatively flat and a precise grading plan was not ready for review at this time. Potential hazards from slope instability, landslides are considered low, Debris flow hazards are considered low. Erosion is considered a possibility due to seasonal precipitation events.

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. From Section 3.2, our project groundwater level we used is greater than 50 feet below the ground surface. Also, the site is within a "MODERATE" liquefaction hazard zone as defined by Riverside County (Geographic Information Services, 2023) and parcel report. Having a historic groundwater level deeper than 50 feet below the ground surface, the potential for liquefaction is low and analysis is not typically required. Lateral spreading is caused by liquefaction. Since liquefaction has a low potential, lateral spreading has a low potential.

The design peak ground acceleration value was obtained from the USGS online application (seismicmaps.org) on October 20, 2023, see Section 5.6. We obtained a $PGA_M = 0.84g$. Because the groundwater was deeper than 50 feet below the ground surface, liquefaction However, we analyzed for dry seismic settlement and determined it potential, please see the following section.

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 (⅔ x0.84) and an earthquake magnitude of 7.9 to evaluate dry seismic settlement potential, also see Section 5.6. The design peak ground acceleration values were obtained from the USGS online application (seismicmaps.org). Based upon methods presented

by Tokimatsu and Seed (1987), the potential for seismically induced dry settlement of soils above the 50 feet full soil column height was estimated for boring B-4. Boring B-4 was explored to 51½ feet below the ground surface (bgs) and analyzed for the top 50 feet. Its settlement was approximately ¾ inch. Per SP117A, for similar soils, differential seismic settlement may be estimated to be half the total settlement results. Based on half of the total dry seismic settlement, the differential settlement is estimated at ⅜ inch for boring B-4.

These estimates are based on the grading recommendations found in Section 5.1 of this report. Due to the general uniformity of the soils encountered, seismic settlement is expected to occur on an areal basis as per Special Publication 117 (2008).

3.8.2   Other Geologic Hazards

Other seismic hazards related to ground shaking include tsunamis, seiches, ground subsidence, and flooding.

Tsunamis: The site is far inland, so the hazard from tsunamis is non-existent.

Seiches: No water storage tanks are located immediately upgradient and near to the project site or are within a close enough distance to allow potential water intrusion from failure. The extent of flooding on this project is dependent upon local drainage patterns and diversionary effects of existing improvements.

Slope Instability: Soils are cohesionless and surficially unstable in sloping configurations. See Section 5.7 for additional information on surficially stable slopes.

Ground Subsidence: The site is not within an area of known study of subsidence (USGS zone of subsidence monitoring in the Coachella Valley, Sneed, 2014 and 2020). 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 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.

Flooding: The project site appears to lie in a Zone X floodplain as identified on FEMA Panel 06065C1559G eff 8/28/2008. The civil engineer should contact FEMA to confirm Zone X. The project site is in an area where sheet flooding, debris flow, and erosion do occur. Appropriate project design by the civil engineer, construction, and maintenance can minimize the site sheet and debris flooding and erosion potential.
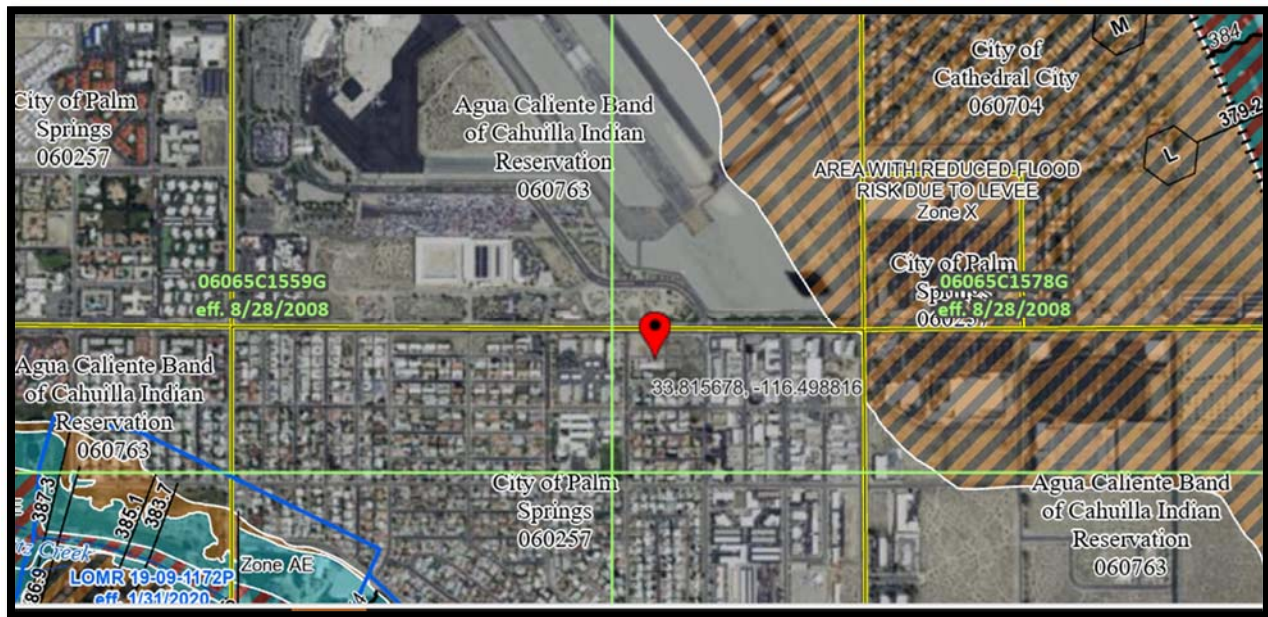
Figure 4   FEMA Map of Site

3.9   Frost Depth

The site's potential frost penetration depth is less than 5 inches, based on Department of Commerce information.

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 site evaluation.

➢ From a geotechnical perspective, the site is suitable for the proposed development, provided the recommendations in this report are followed in the design and construction of this project. Concrete masonry units (CMU), concrete, or wood and metal light frame construction are considered equally feasible from a geotechnical perspective.

➢ The primary geologic hazard is severe ground shaking from earthquakes originating on local and regional faults. A major earthquake above magnitude 7 or greater originating on the local segment of the San Andreas fault zone would be the critical seismic event that may affect the site within the design life of the proposed development. Engineered design and earthquake-resistant construction increase safety and allow development of seismic areas.

➢ The underlying geologic condition for seismic design is Site Class D based on ASCE7-16 Exceptions (see Section 5.6 for more detail). The site is about three miles southwest of Type A seismic sources of the San Andreas' fault zone, as defined by the California Geological Survey. A qualified professional should design any permanent structure constructed on the site. The *minimum* seismic design should comply with the 2022 edition of the California Building Code.

➢ Our analysis indicates that the expected design level seismic shaking could cause dry sand settlement (upper 50 foot dry soil profile).

➢ Other geologic hazards, including fault rupture, lateral spreading, tsunamis, seiches, and ground subsidence are considered to have a low or negligible potential to occur onsite.

➢ The site is susceptible to erosion and flooding.

➢ The upper site soils are very loose from wind blown deposition, minor grading disturbance, structure demolition, and possible underground improvements.

➢ Drywells or septic systems (pits) are possible onsite given the past site development. Removal and backfill of these buried items will be required if unearthed during grading.

➢ The upper soils were dry to damp with moisture generally less than 2 percent.

➢ 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 Competent Person during grading and trenching operations.

➢ The soils are highly susceptible to water erosion. Preventative measures to reduce seasonal flooding and erosion should be incorporated into site grading plans and include any slopes. Dust control should also be implemented during construction. Site grading should be in strict compliance with the requirements of the South Coast Air Quality Management District [SCAQMD].

➢ Site soils are cohesionless and susceptible to surficial instability in a sloped condition.

➢ The site soils are generally "very low" in Expansion Index as defined by ASTM D 4829. Samples of building pad soils should be evaluated during grading to confirm or modify these findings.

➢ The corrosion values from the soil 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) 318, Section 4.3.

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 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 2022 California Building Code. Observation of fill placement by the Geotechnical Engineer of Record should be in conformance with Section 17 of the 2022 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 our previous study. Additionally, the California Building Codes 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.

Clearing and Grubbing: At the start of site grading, existing asphalt and concrete debris, existing concrete capped drywell, septic systems (if existing), existing vegetation, pavement (if found), irrigation systems, undocumented fill, construction debris, trash, and underground utilities should be removed from the proposed building pads and improvement areas. Oversize material, trash, debris, vegetation (greater than 1% organic content), etc. should be removed prior to use as engineered fill.

Buried utilities and pits may exist 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 approved by the project geotechnical engineer. Abandoned utilities should be removed entirely, or pressure-filled with concrete or grout and be capped. Buried utilities should not extend under building limits. Subsequent to stripping and grubbing operations, areas to receive fill should be stripped of loose or soft earth materials until a uniform, firm subgrade is exposed, as evaluated by the geotechnical engineer, geologist, or their representative. Prior to the placement of fill or subsequent to cut, the existing surface soils within the building pads and improvement areas should be over-excavated as follows:

Moisture Conditioning of In-Situ Soils: Result of moisture testing the existing soils indicated moisture contents near zero percent. Caving of soil walls was also noticed during our exploration. Prior to clearing and grubbing and excavation, the contractor should moisture condition the native soil to depth of 5 feet below the existing surface or 1 foot below the bottom of footing or utility, whichever is deeper. Moisture conditioning should be defined as a moisture content being at or near optimum moisture content (within 2 percent). Moisture conditioning can be approved at the discretion of the geotechnical engineer of record or his representative. The contractor or

project manager shall contact the geotechnical engineer of record when this moisture depth has been achieved and provide test pits for the geotechnical engineer to sample for moisture contents.

The area is found in a high-risk wind-blown area. It's highly recommended to protect any excavated soils during and after grading has commenced for that day.

Building Pad Preparation: Due to the loose relative density of shallow soils in the upper soil profile, the existing soils within the building pad and foundation areas (including pier/pile foundation areas) should be over-excavated a minimum of 4 feet below existing or finished grade, or 3 feet below the bottom of shallow foundations, whichever is lower. The over-excavation should extend for at least 5 feet beyond the outer edge of the building pad and include all exterior footings or slabs and include any overhead canopy/walkway areas as well as pier foundation areas or the depth of overexcavation, whichever is deeper and be equal depth across the pad bottom. The exposed and undisturbed bottom of the over-excavation should be observed and tested by the geotechnical engineer or their representative to verify that an in-place density of the over-excavation subgrade bottom is at or greater than 85% relative compaction 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 or soils are not firm. Once the bottom subgrade is attained and approved, the surface should be scarified an additional 12 inches, moisture conditioned to near optimum moisture for an additional 1 foot and recompacted to a minimum of 90% compaction relative to ASTM D 1557. Moisture conditioned and compacted fill should be placed to finish subgrade as engineered fill.

Auxiliary Structures Subgrade Preparation: Auxiliary structures such as masonry walls, trash enclosures, or retaining walls should have the foundation subgrade prepared similar to the building pad recommendations given above. The lateral extent of the over-excavation need only extend 2 feet beyond the face of the footing and include all exterior footings or slabs, and also any overhead canopy/walkway areas, where possible. All footing excavations, prior to bottom recompaction, should be probed for uniformity. Soft or loose zones should be excavated and recompacted to finish foundation bottom subgrade. Footing bottom compaction testing should confirm at least 90% relative compaction.

NOTE: Due to property line constraints and any adjacent site improvements, shoring may be necessary to achieve the remedial grading depths for known improvements and unknown improvements encountered near the property lines.

Subgrade Preparation: In areas to receive fill not supporting structures, or supporting lightly loaded hardscape such as sidewalks (i.e. no vehicle traffic), the subgrade should be over-excavated moisture conditioned to near optimum moisture, and compacted to at least 90% relative compaction (ASTM D 1557) for a depth of two feet below existing or finished subgrade, whichever is lower. Compaction should be verified by testing.

Pavement Area Preparation: In street, drive, and permanent parking areas, the subgrade should be over-excavated, scarified, moisture conditioned to near optimum moisture, and compacted to at least 90% relative compaction (ASTM D 1557) for a depth of three feet below existing grade or finish grade (whichever is deeper). Engineered fill (as described below) should then be

moisture conditioned, placed in suitable lifts, and compacted to a minimum of 90% relative compaction, with the upper 12 inches of finish subgrade compacted to at least 95% relative compaction. Within paver areas, the upper 12 inches of finish subgrade should be compacted to at least 98% relative compaction. Compacted fill should be placed to finish subgrade elevation as engineered fill. Compaction should be verified by testing.

<u>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.</u>

<u>Engineered Fill Soils</u>: The overexcavated and native soil is suitable for use as engineered fill and utility trench backfill provided it is free of significant organic or deleterious matter (less than 1%), debris, concrete, and oversize rock. Construction debris, concrete, asphalt, organic material, etc. is not suitable for placement within fill. These materials should be hauled offsite.

Within areas to receive foundations and slabs-on-grade the fill should be at least "very low" in Expansion Index. Fill soils should have a classification of SP, SM, or SP-SM. Fill should be placed in maximum 8-inch lifts (loose thickness), moisture conditioned to near optimum moisture content (typically between -2 and 2 percent of optimum moisture) and compacted to at least 90 percent relative compaction in general accordance with ASTM D 1557 (current edition) prior to the placement of subsequent lifts. Compaction should be verified by testing. In general, rocks larger than 6 inches in greatest dimension should be removed from fill or backfill material. All soils should be moisture conditioned prior to application of compactive effort. Moisture conditioning of soils refers to adjusting the soil moisture to 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 (such as blending or punching aggregate into the exposed subgrade.

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 test 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) or correlated hand-probing.

Imported fill soils (if needed) should be very low to low expansion potential granular soils meeting the Unified Soil Classification System (USCS) classifications of 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 or his representative should evaluate the import fill soils before hauling to the site.

<u>Shrinkage and Bulking</u>: The shrinkage factor for soils is expected to range from 4 to 6 percent (%) for the upper excavated or scarified *site* soils. This estimate is based on compactive effort to achieve an average relative compaction of about 93%.

The average computed shrinkage is 5%. Shrinkage and construction related subsidence are highly dependent on and may vary with contractor methods for compaction. Losses from site clearing, oversize material, and removal of existing site improvements may affect earthwork quantity calculations and should be considered.

Surcharge Load Restrictions: No fill or other surcharge loads shall be placed adjacent to any building or structure unless such building or structure is capable of withstanding the additional loads caused by the fill or the surcharge. Footings or foundations that will be affected by any excavation shall be underpinned or otherwise protected against settlement and shall be protected against detrimental lateral or vertical movement, or both.

> **Exception:** Minor grading for landscaping purposes shall be permitted where done with walk-behind equipment, where the grade is not increased more than 1 foot from original design grade or where approved by the building official.

5.2   Excavations and Utility Trenches

Excavations should be made in accordance with OSHA requirements. Using the 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 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 very high potential for caving and slaking of site excavations (over-excavation areas, utilities, footings, etc.). Where excavations over 4 feet deep are planned lateral bracing or appropriate cut slopes of 1.5:1 (horizontal/vertical) should be provided. Prewatering should be required to improve stability; however, boring sidewall collapse were still observed using moisture conditioning so precaution is mandatory. 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. Soils are susceptible to caving such that shallower excavated slopes may be required for site safety.

Where excavations will reduce support from any foundation, a registered design professional shall prepare an assessment for the structure as determined from examination of the structure, the review of available design documents and, if necessary, excavation of test pits. The registered design professional shall determine the requirements for underpinning and protection and prepare site-specific plans, details and sequence of work for submission. Such support shall be provided by underpinning, sheeting and bracing, or by other means acceptable to the building official.

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 1.5:1 (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 over-excavation will be performed adjacent to existing structures, ABC

slot cutting may be used if it can be demonstrated the loose soils encountered remain stable during excavation and replacement.

Temporary Shoring: Shoring may be required where soil conditions, space or other restrictions do not allow a sloped excavation. A braced or cantilevered shoring system may be used. A temporary cantilevered shoring system should be designed to resist an active earth pressure equivalent to a fluid weighing 45 pounds per cubic foot (pcf). Braced or restrained excavations above the groundwater table should be designed to resist a uniform horizontal equivalent soil pressure of 65 pounds per cubic foot (pcf). 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 a 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 engineer 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 boring logs in this report, and perform their own assessment of potential construction difficulties, and methods should be selected accordingly. The method of excavation and support is ultimately left to the contractor.

A representative from our firm should be present during all site demolition and clearing and grading operations to monitor site conditions; substantiate proper use of materials; evaluate compaction operations; and verify that the recommendations contained herein are met.

Utilities and Trenches: 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.). Utility trench backfill within private property should be placed in conformance with the provisions of this report for engineered fill. In general, service lines extending inside of property may be backfilled with native soils compacted to a minimum of 90% relative compaction per ASTM D 1557. Backfill operations should be observed and tested to monitor compliance with these recommendations. The trench bottom should be in a firm condition prior to placing pipe, bedding, or fill.

Under pavement sections, the upper 12 inches of trench backfill using native on-site soil below the pavement section should be compacted to at least 95 percent relative compaction (ASTM D 1557). 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. Mechanical compaction is recommended; ponding or jetting is not recommended.

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/conduit 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. Loss of soil may cause damaging settlement. Filter fabric, such as Mirafi 140N should separate gravel from native or native derived soils. NOTE: Rocks greater than 3 inches in diameter should not be incorporated within utility trench backfill.

5.3   Foundations

In our professional opinion, foundations for the structures proposed (as presented within) could be supported on shallow foundations bearing in properly prepared and compacted soils placed as recommended in Section 5.1. The following recommendations are based on "very low" expansion category soils in the upper 5 feet of subgrade and the moisture condition of soils below final pad grade as explained in Section 5.1 of this report. Soils which are found to be more expansive than a "very low" Expansion Index will require differing foundation requirements which should be provided on a case by case basis.

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 inches (15 inches for two-story, 24 inches for three-story) below lowest adjacent <u>soil</u> grade should be maintained (lowest grade within 3 feet laterally as measured from the foundation bottom) for wall foundations, see minimum embedment depths below. Other overburden such as concrete, slurry, etc. is not considered suitable to account for footing embedment. Earth Systems should be retained to observe foundation excavations before placement of reinforcing steel or concrete. Loose soil or construction debris should be removed from footing excavations before placement of concrete. <u>After excavation, foundation bottoms should be compacted to at least 90% relative compaction.</u>

<u>Slope Setback for Foundations</u>: Earth Systems recommends a minimum setback distance of 5 feet. The 2022 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 to the bottom 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.

<u>Conventional Spread Foundations</u>: 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 wall foundations, 12-inch minimum width (24 inches maximum width) and minimum 12 inches below grade (maximum 30 inches depth):

> 1,500 pounds per square foot (psf) for dead plus design live loads

Allowable increases of 500 psf for each additional 0.5 foot of footing depth may be used up to a maximum value of 3,000 psf. No increase of bearing pressure for increase of footing width.

➢ Pad foundations, 2 x 2 foot minimum and 6 x 6 foot maximum in plan and 18 inches minimum below grade (maximum 36 inches):

> 2,000 psf for dead plus design live loads

Allowable increases of 500 psf for each additional 0.5 foot of footing depth may be used up to a maximum value of 3,500 psf. No increase of bearing pressure for increase of footing width.

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 maximum values provided in Section 1.1, the geotechnical engineer must reevaluate the allowable bearing values. Underground utilities should be designed for an anticipated settlement of 1 inch of loading settlement and 1 inch seismic settlement within building areas, see additional information below "Estimated Settlements".

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 twice the width of the largest footing from any adjacent footing, such that influence effects are minor.

Maximum foundation sizes given above are based on allowable bearing pressures. 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.

An average modulus of subgrade reaction, k, of 200 pounds per cubic inch (pci) can be used to design lightly loaded footings 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.

Minimum Foundation Reinforcement: Minimum reinforcement should be provided by the structural engineer to accommodate the settlement potentials presented within. Minimum reinforcement for continuous wall footings should be two, No. 4 steel reinforcing bars, one placed near the top and one placed near the bottom of the footing. This reinforcing is not intended to supersede any structural requirements provided by the structural engineer.

Estimated Settlements for Foundations: Estimated total static load settlement and collapse settlement should be less than 1 inch, based on footings found on firm soils as recommended. The total estimated differential settlement for the static loading settlement is estimated to be ½ inch. As such, static and differential settlement applied over a typical foundation distance of 40 feet, we recommend the structural engineer design for the standard angular distortion of 1:480,

which is normally defined as a tolerable level for typical buildings (County of Riverside, 2000, page 39).

The total estimated differential settlement for the combined static (1 inch) and seismic (¾ inch) settlement is estimated to be half of the estimated total for each case or ⅞ inch. As such, considering both static and seismic differential settlement applied over a typical foundation distance of 40 feet, we recommend the structural engineer design for the angular distortion of 1:480, which is defined as a tolerable level for typical buildings (County of Riverside, 2000, page 39) without structural improvement.

Earthquake Performance Statement: Depending upon the extent of structural and geotechnical design of exterior flatwork, walls, utilities, roadways, and other similar site improvements, some damage due to seismic events will occur. We recommend a standard statement for purchasers of the property and within title reports that seismic induced damage may 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. For all settlement conditions excluding seismic settlement, the structure's maximum distortion angle should be the Riverside County's required 1:480.

5.4   Slabs-on-Grade

Subgrade: Concrete slabs-on-grade and flatwork should be supported by compacted soil placed in accordance with Section 5.1 of this report.

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 4 inches of permeable base material with the following specified gradation.

Table 5
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, excessive cement content, 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: Slab thickness and reinforcement of slabs-on-grade are contingent on the recommendations of the structural engineer or architect. Based upon our findings, a modulus of subgrade reaction of approximately 200 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.

Concrete slabs and flatwork should be a minimum of 5 inches thick (actual, not nominal). If heavily loaded flatwork is proposed (forklift, etc.), the actual thickness should be designed by the structural engineer utilizing techniques of the American Concrete Institute (ACI) and may be greater than 5 inches in thickness. Pavement slab thickness is presented in Section 5.8 for rubber

tire vehicles. We suggest the concrete slabs be reinforced with a minimum of No. 3 rebar at 16-inch centers, both horizontal directions, placed at slab mid-height to resist cracking. 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, location, and reinforcing given are not intended to supersede any structural or corrosion 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. These slab recommendations are based on the shallow surface soils having an Expansive Index of "Very Low", and prior to placement of concrete, the subgrade is pre-saturated and compacted as recommended within.

A minimum concrete gap of three (3) inches should be provided around the steel reinforcing and the edge of the formwork or surfaces. Reinforcing steel should be placed upon centralizers rather than lifted into place during placement. Where the reinforcing steel does not have adequate cover, it will corrode and can fracture the cured concrete and produce unsightly rust discoloration when exposed to site soils and water.

Slab-On-Grade Control Joints: Control joints should be provided in all regular concrete slabs-on-grade at a maximum spacing between 26 and 36 times the slab thickness and around all penetrations (12 feet maximum on-center, each way) as recommended by American Concrete Institute [ACI] guidelines. Control joints should be provided in all concrete slabs-on-grade at a maximum spacing of approximately 4 to 6 feet for sidewalks. For decorative slabs, closer joints are recommended, with slabs also isolated from foundations and penetrations via spacer strips and joint cuts. 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 ½-inch dowels at 18 inches on center embedded per ACI or a thickened keyed-joint to resist vertical deflection at the joint. Dowels are not a replacement for improperly cured concrete which can experience slab curl and joint and edge offset. Proper wet curing is critical. All control joints in 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. Hot Weather Concreting guidelines per ACI 305 should be followed.

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 lightly vibrated during placement. Concrete should be continuously covered and wet cured for at least 7 days with burlap or plastic and not allowed to dry out to minimize surface cracking and curling.

5.5   Retaining Walls and Lateral Earth Pressures (granular backfill)

<u>Retaining Structures</u>:

- Retaining walls should be designed for an active soil pressure equivalent to a fluid density of 41 pcf. The active lateral earth pressures are for horizontal (level) backfills using the recommended compacted on-site native soils on flexible walls that are free to rotate at least 0.1 percent of the wall height. Walls, which are restrained against movement or rotation at the top, should be designed for an at-rest equivalent fluid pressure of 62 pcf. The lateral earth pressure values for level backfill are provided for walls backfilled with drainage materials and existing on-site soils.

- In addition to the active or at rest soil pressure, the proposed retaining structures should be designed to include forces from dynamic (seismic) earth pressure (Atik and Sitar, 2010).   Dynamic pressures are additive to active and at-rest earth pressure (following their distribution) and should be considered as 51 pcf for flexible walls, and 67 pcf for rigid walls. Seismic pressures are based on PGAM of 0.84g, Friction Soil Angle (φ) of 31°, and a maximum dry density of 125 pcf. A factor of safety of 1.5 should be used in stability analysis except for dynamic earth pressure where a factor of safety of 1.2 is acceptable.

- Retaining wall foundations should be placed upon compacted fill described in Section 5.1.

- A backdrain or an equivalent system of backfill drainage should be incorporated into the wall design, whereby the collected water is conveyed to an approved point of discharge. Free draining soils may use weep holes. Design should be in accordance with the 2022 California Building Code. Drain rock should be wrapped in filter fabric such as Mirafi 140N as a minimum. Backfill immediately behind the retaining structure should be a free-draining granular material such as the sandy on site soils. Waterproofing should be according to the designer's specifications. Water should not be allowed to pond or infiltrate near the top of the wall. To accomplish this, the final backfill grade should be such that water is diverted away from retaining walls.

- Compaction on the retained side of the wall within a horizontal distance equal to one wall height (to a maximum of 6 feet) should be performed by hand-operated or other lightweight compaction equipment (minimum 90% compaction relative to ASTM D 1557 at near optimum moisture content). This is intended to reduce potential locked-in lateral pressures caused by compaction with heavy grading equipment or dislodging modular block type walls.

- The above recommended values do not include compaction or truck-induced wall pressures. Care must be taken during the compaction operation not to overstress the wall. Heavy construction equipment should be maintained a distance of at least 3 feet away from the walls while the backfill soils are placed. Upward sloping backfill or surcharge loads from nearby footings can create larger lateral pressures. Should any walls be considered for retaining sloped backfill or placed next to foundations, our office should be contacted for recommended design parameters.   Surcharge loads should be considered if they exist within a zone between the face of the wall and a plane projected 45 degrees upward from the base of the wall. The increase in lateral earth pressure

should be taken as 50% of the surcharge load within this zone. Retaining walls subjected to traffic loads should include a uniform surcharge load equivalent of 240 psf for auto and 450 psf for truck traffic located at least 3 feet from the wall back edge. Closer loads will impart greater pressures on the wall. Retaining walls should be designed with a minimum factor of safety of 1.5.

<u>Frictional and Lateral Coefficients</u>:

- Resistance to lateral loads (including those due to wind or seismic forces) may be provided by frictional resistance between the bottom of concrete foundations and the underlying soil, and by passive soil pressure against the foundations. An allowable coefficient of friction (Factor of Safety = 1.2) of 0.35 may be used between cast-in-place concrete foundations and slabs and the underlying soil. An allowable coefficient of friction of 0.30 may be used between pre-cast or formed concrete foundations and slabs and the underlying soil.

- Compacted native allowable passive pressure may be taken as equivalent to the pressure exerted by a fluid weighing 300 pounds per cubic foot (pcf) or psf/ft, which includes a 1.5 Factor of Safety. Vertical uplift resistance may consider a soil unit weight of 100 pounds per cubic foot. The upper 1 foot of soil should not be considered when calculating passive pressure unless confined by overlying asphalt concrete pavement or Portland cement concrete slab. Where post or foundations are constructed in unremediated areas (i.e. no over-excavation) the passive resistance (including a safety factor of 1.5) should be limited to 2,000 psf and have the upper 3 feet of soil neglected. <u>The soils pressures presented have considered onsite soils</u>. Testing or observation should be performed during grading by the soils engineer or his representative to confirm or revise the presented values.

- Lateral passive pressures may be increased by 1/3 for temporary wind or seismic forces.

- Construction employing poles or posts (i.e. lamp posts) may utilize deep foundation (pile/pier) design methods presented in Sections 1806 and 1807.3 of the CBC for Silty Sand (SM) material class for soils overexcavated as recommended in Section 5.1.

- The passive resistance of the subsurface soils will diminish or be non-existent if trench sidewalls slough, cave, or are over widened during or following excavations. If this condition is encountered, our firm should be notified to review the condition and provide remedial recommendations, if warranted. For foundations setback as per this report from the face of slopes, the full passive pressure may be utilized.

- Temporary backcuts for retaining wall construction should be no steeper than 1.5:1 (H:V).

5.6   Seismic Design Criteria

This site is subject to strong ground shaking due to potential fault movements along regional faults including San Andreas and San Jacinto fault zones. Engineered design and earthquake-resistant construction increase safety and allow development of seismic areas. The minimum seismic design should comply with the 2022 edition of the California Building Code and ASCE 7-16 (Supplement 3) using the seismic coefficients given in the table below, which assume

Exception 11.4.8 in ASCE7-16 applies. The site was <u>found not to have</u> liquefiable soil. Based on this section, the site is classified as Site Class D. If these exceptions do not apply, the structural engineer should contact Earth Systems for a Site-Specific Ground Motion Analysis, as the values below would no longer be valid.

Note to the Structural Engineer: the seismic coefficients in Table 6, below, apply to the general procedure for determining seismic coefficients. In other words, the seismic coefficients were <u>not</u> determined by a Site-Specific Ground Motion Analysis, which is allowed if ASCE 7-16's Section 11.4.8 Supplement 3 Exceptions and the Exceptions of 20.3.1 are accepted by the structural engineer.

According to the current procedure as outlined in ASCE 7-16, Supplement 3, a ground motion hazard analysis shall be performed in accordance with Section 21.2 for structures:

1) On Site Class D sites with S_1 greater than or equal to 0.2.

> EXCEPTION Item 1: A ground motion hazard analysis is not required where the value of the parameter S_{M1} determined by Eq. (11.4-2) is increased by 50% for all applications of S_{M1} in this Standard. The resulting value of the parameter S_{D1} determined by Eq. (11.4-4) shall be used for all applications of S_{D1} in this Standard.

During plan review of the foundation, **Earth Systems will request a letter from the structural engineer** stating ASCE 7-16's Section 11.4.8 Exception 1 and 20.3.1 period applies to the structures applicable to this report's Table 6 below, which contains the modified S_{m1} and resultant modified S_{D1} values. The structural engineer and client should contact Earth Systems if ASCE 7-16's Exceptions do not apply to the structure/s and request Earth Systems to perform a Site-Specific Ground Motion Hazard Analysis.

General Procedure for seismic parameters is presented below considering a Site Class D shear wave velocity (results in Appendix A). Values were obtained from a web site (https://seismicmaps.org/) using a coordinate location of Latitude 33.8157°N and Longitude 116.4986°W. The structural design engineer should use the most conservative results based of the specific building design and spectral response.

Table 6
2022 CBC (ASCE 7-16) Seismic Parameters

Site Class:	D
Risk Category:	II
Seismic Design Category	D

Maximum Considered Earthquake [MCE] Ground Motion

Short Period Spectral Response S_s:	1.759g
1 second Spectral Response, S_1:	0.729g

Code Design Earthquake Ground Motion

F_a	1.00
F_v	1.70
F_{PGA}	1.10
S_{MS}	1.759g
$S_{M1 \text{ (unmodified)}}$	1.239g
$S_{M1 \text{ (ASCE 7-16 Supplement 3 Modified)}}$	1.859g
Short Period Spectral Response, S_{DS}	1.173g
1 second Spectral Response, $S_{D1 \text{ (unmodified)}}$	0.826g
*1 second Spectral Response, $S_{D1 \text{ (ASCE 7-16 Supplement 3 Modified)}}$	1.239g
Peak Ground Acceleration (PGA_M) Eq 11.8-1	0.84g

* Only if Exceptions is used, see Section 5.6 text, otherwise Site
Specific Analysis required.

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.

Spectral accelerations will exceed one g. Actual acceleration 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.

5.7   Slope Construction

Slopes are not generally proposed for this project; however, minor slopes (less than 5 feet in height) may be constructed. Site soils are highly susceptible to erosion. Slopes protected against erosion (per approved methods such as significant planting, large rip-rap, facing, or erosion blankets, etc.) should be constructed at 3:1 (horizontal: vertical) or flatter inclinations except for slopes used as nuisance or storm drainage channel slopes. Unprotected slopes with <u>exposed</u> native soils at the surface should be expected to require repair after heavy nuisance or storm runoff occurs due to significant erosion. The above slope recommendations may change pending a more in-depth geotechnical evaluation once design plans are developed.

Where new slopes will be constructed against existing slopes, a series of level benches and keyways should be provided to seat the compacted fill. The benches should be a minimum of 5 feet in width and be constructed at approximately 2-foot vertical intervals or as dictated by topographic conditions, and be constructed in accordance with the California Building Code. Slopes should be constructed at inclinations no steeper than 3:1 (horizontal: vertical) such that the slope is comprised of fully compacted soil exposed at the surface. Such methods may include overfilling during construction and cutting back to expose a fully compacted soil, or track-walking or grid-rolling. 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 density of 90 percent of the maximum dry unit weight, as measured in relation to ASTM D 1557 test procedures, and be compacted at near optimum moisture content. Due to the highly erodible site soils, slope faces should be protected with facing or densely spaced vegetation to reduce the erosion potential.

5.7.1   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. Such measures may include catchment areas or walls, ditches, soil planting, facing, or other techniques to contain soil material. An erosion control mat as the final slope facing layer can be used.

<u>Slope Maintenance</u>: Site soils are highly susceptible to erosion. Unprotected slopes with exposed native or native derived soils at the surface should be expected to require repair after heavy nuisance or storm runoff occurs due to significant erosion. 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. Fill should be placed and compacted as recommended. These repairs should be performed in a prompt manner after their occurrence. Design slope inclinations should be

maintained, and a maintenance program should include identifying areas where slopes begin to steepen. Due to the highly erodible site soils, slope faces should be protected with facing or densely spaced vegetation to reduce the erosion potential.

5.8   Streets, 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 within. 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 a tested R-value, observed soil types likely present after grading, 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 20,000 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 7
Preliminary Flexible Pavement Section Recommendations
On-site/Interior Automobile Drive Areas

R-Value of Subgrade Soils – (73 Tested, 50 used)   Design Method – CALTRANS

Traffic Index (Assumed)*	Pavement Use	Flexible Pavements**	
		Asphaltic Concrete Thickness (inches)	Aggregate Base Thickness (inches)
5	Parking Areas & Fire Lanes***	3	4
7	Main Drive Areas	4	4½
9	Tractor Trailer Drives	5½	6½

*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 or Tractor Trailer Drive use with other traffic, busses, or trucks, the thicker pavement section should be used.

Conventional, rigid pavements, i.e. Portland cement concrete (PCC) pavements, are recommended in areas that will be subject to relatively high static wheel loads and/or heavy vehicle loading and unloading and turning areas (i.e. truck/bus lanes/trash enclosure truck aprons). This is due to rutting and shoving that can occur due to the heavy vehicle loads and the repetitious set path which is followed at the bus/delivery trucks areas where the same wheel track and stopping occurs generally in the same spot each time. The vehicle load combined with hot summer asphalt (AC) concrete causes the upper surface of the AC to creep forming ruts in conjunction with the braking and accelerating forces which shove the AC. Turning forces also do the same.

The pavement section below is based upon the American Concrete Institute (ACI) *Guide for Construction of Concrete Parking Lots, ACI 330R*, and the assumptions outlined below.

Table 8
Preliminary Portland Cement Concrete Pavement Sections
(R Value of 55 has Equivalent CBR of 12)

Area	Minimum Pavement PCC Thickness (inches)	Minimum 28 Day Flexural Strength (psi)	Concrete Compressive Strength (psi)
Truck/Bus Access or Loading/Unloading Areas (Traffic Category C, ADTT =100)	6½	500	3,250
Tractor Trailer Drives (Traffic Category D)	7.0	500	3,250

Should the actual traffic category vary from those assumed and listed above, these sections should be modified. All above recommended preliminary pavement sections are contingent on the following recommendations being implemented during construction:

- Pavement should be placed upon compacted fill processed as described in Section 5.1. The upper 12 inches of subgrade soils beneath the asphalt concrete and conventional PCC pavement subgrade section should be compacted to a minimum of 95% relative compaction (ASTM D 1557).

- <u>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.</u> Subgrade soils should be surface watered prior to the placement of aggregate base.

- 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) (98% for below pavers) 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-blow Marshall density (ASTM D 1559) or equivalent.

- Portland cement concrete pavements should be constructed with transverse joints at maximum spacing of 12 feet. A thickened edge should be used where possible and, as a minimum, where concrete pavements abut asphalt pavements. The thickened edge should be 1.2 times the thickness of the pavement (8 inches for a 6½-inch pavement), and should taper back to the PCC thickness over a horizontal distance on the order of 3 feet.

- All longitudinal or transverse control joints should be constructed by hand forming or placing pre-molded filler such as "zip strips." Expansion joints should be used to isolate fixed objects abutting or within the pavement area.

The expansion joint should extend the full depth of the PCC pavement. Joints should run continuously and extend through integral curbs and thickened edges. We recommend that joint layout be adjusted to coincide with the corners of objects and structures. In addition, the following is recommended for concrete pavements:

   1.   Slope pavement at least ½ percent to provide drainage;

2.   Provide rough surface texture for traction;

3.   Cure PCC concrete with curing compound or keep continuously moist for a minimum of seven days;

4.   Keep all traffic off concrete until PCC compressive strength exceeds 2,000 pounds per square inch (truck traffic should be limited until the concrete meets the design strength (3,250 psi); and

5.   Consideration should be given to having PCC construction joints keyed or using slip dowels on 24-inch centers to strengthen control and construction joints. Dowels placed within dowel baskets should be incorporated into the concrete at each saw-cut control joint (i.e. dowel baskets and dowels are set in place prior to placement of concrete).

- Portland cement concrete placement and curing should, at a minimum, be in accordance with the American Concrete Institute [ACI] recommendations contained in ACI 211, 304, 305, 308, 309, and 318.

- 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 the base cap or finished pavement sections.

5.9   Site Drainage and Maintenance

Positive drainage should be maintained away from the structure foundations (5 percent for 10 feet minimum) to prevent ponding, water intrusion, and subsequent saturation of the foundation soils. Gutters and downspouts in conjunction with a 2 percent (%) hardscape grade sloped away from structures draining to area drains can be considered as a means to convey water away from foundations if increased fall is not provided. A minimum 2% fall is required by the 2022 CBC.

There is an exception where less than 2% grade is allowable if the area is for a door landing or ramp per ADA regulations.

Drainage should be maintained for paved areas. Water should not pond on or near paved areas, slabs, or foundations. Ponded water can saturate subgrade soils and lead to pavement, slab, flooring, and foundation failure. The following recommendations are provided in regard to site drainage and structure performance:

- 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.

- 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. Diversion and conveyance structures which can accommodate water and eroded soil should be constructed at the tops and toes of all slopes. Lined swales or berms at the top and bottom of slopes are recommended.

- Applied irrigation to maintain landscaping should be controlled to the minimum volume and frequency necessary to sustain plant material. Excess and frequent watering could lead to saturated areas and standing water which can cause slab and foundation distress. The irrigation system designer should consider these conditions in their design and control irrigation accordingly.

- 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 and intrusion. 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.

- 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) throughout their design life. Structural performance is dependent on many drainage-related factors such as landscaping, irrigation, lateral drainage patterns and other improvements. Cleanout should be provided in drainage piping.

- 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 the 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 12 should be applied to lower design 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 filters and sedimentation structures. If these measures are provided, the factor of safety can be reduced.

- We recommend drywells or infiltrating structure be located at least 30 feet from structure foundations, slopes, or settlement sensitive features.

- Water should not be allowed to pond within 10 feet of building foundations. The civil engineer producing the grading plan is notified that a gap made of clean sand is typically found between the vapor barrier and bottom of slabs or foundations. If water intrusion into this gap is possible the civil engineer shall either provide for no ponding or an impervious material to prevent water intrusion into the sand gap.

- The above recommendations are recommended to be available as part of the homeowner documents such that homeowner improvements can be designed and maintained to allow proper drainage. Homeowner improvements which do not follow these recommendations are at increased risk of causing home moisture intrusion and water damage.

When grades of 5 percent for 10 feet away from foundations are not possible, the CBC allows for alternative drainage systems (Section 1804.3). Earth Systems experience has found that alternative methods may include impervious surface material (such as concrete and asphalt), area drains, and horizontal moisture barriers. Please contact Earth Systems for additional information or if the project has difficulty making grades.
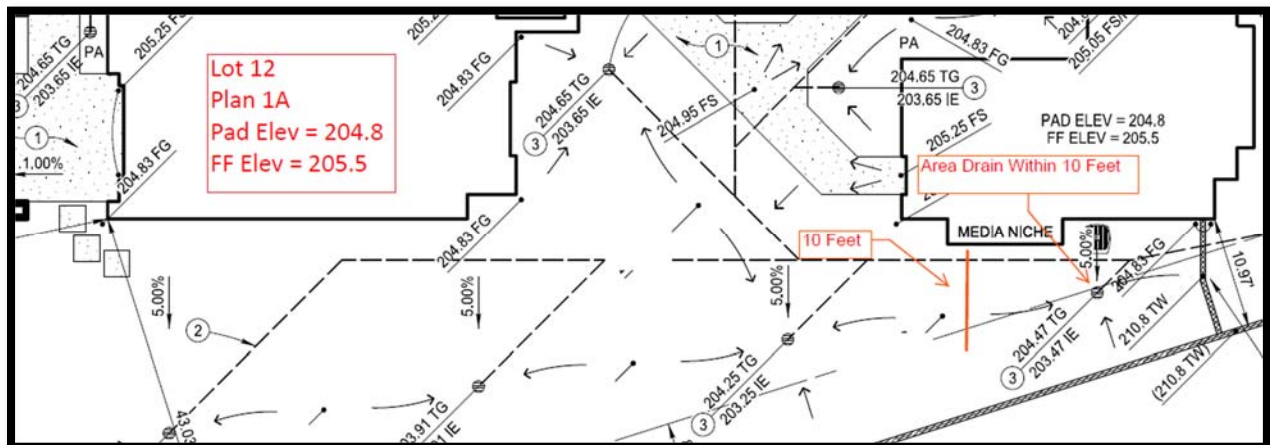
Figure 5 Example of Area Drains Used Within 10 Feet of Foundations When Slope < 5% in 10 Feet.

Section 6
LIMITATIONS AND ADDITIONAL SERVICES

6.1   Uniformity of Conditions and Limitations

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. The magnitude of the introduction or removal, and the effect on the surface and subsurface soils is currently unknown.

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. Earth Systems is not providing civil, structural, drainage, architectural, etc., or any portion of the actual design. We make no representation as to the accuracy of the dimensions, measurements, calculations, or any portion of the design. Recommendations contained in this report are based on our field observations and subsurface explorations, select published documents (referenced), and our present knowledge of the proposed construction. 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 and are strictly for the client. 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. Land use, site conditions (both on site and off site) or other factors may change over time, and additional work may be required with the passage of time.

If during construction, soil conditions are encountered which differ from those described herein, we should be notified immediately in order that a review may be made and any supplemental recommendations provided. In such an event, the contractor should promptly notify the owner so that Earth Systems geotechnical engineer can be contacted to confirm those conditions. We recommend the contractor describe the nature and extent of the differing conditions in writing and that the construction contract include provisions for dealing with differing conditions. Contingency funds should be reserved for potential problems during earthwork and foundation construction.

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.

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 reviewed for applicability and conformance to the current design and incorporated into the plans for the project. The owner or the owner's representative also has the responsibility to take the necessary steps to see 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.

Grading and compaction operations should be performed in conjunction with observation and testing. The recommendations provided in this report are based on the assumption that Earth Systems will be retained to provide observation during the construction phase to evaluate our recommendations in relation to the apparent site conditions at that time. If we are not accorded this observation, Earth Systems assumes no responsibility for the suitability of our recommendations. 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. These services will be performed on a time and expense basis in accordance with our agreed upon fee schedule once we are authorized and contracted to proceed.

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.

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 System 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.* 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 and is required by the 2022 California Building Code. 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 our previous study. Additionally, the California Building Codes

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.

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 or local grading ordinances.

- Consultation as needed during construction.

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Appendices as cited are attached and complete this report.

REFERENCES

Al Atik, L., and Sitar, N., 2010, Seismic Earth Pressures on Cantilever Retaining Structures, Journal of Geotechnical and Geoenvironmental Engineering, ASCE.

American Association of State Highway and Transportation Officials, 1993, AASHTO Guide for Design of Pavement Structures 1993.

American Concrete Institute (1992) "Guide for Design and Construction of Concrete Parking Lots (ACI 330R.01) (Reapproved 1997)."

American Society of Civil Engineers [ASCE]/Structural Engineering Institute [SEI], 2017, Minimum Design Loads and Associated Criteria for Buildings and Other Structures, ASCE 7-16, including Supplements 1 to 3.

American Society for Testing Materials, 1989, STP 1013, Effects of Soil Characteristics on Corrosion (February, 1989).

Boulanger, R. W., and Idriss, I. M., 2006, Liquefaction susceptibility criteria for silts and clays, J. Geotechnical and Geoenvironmental Eng., ASCE 132 (11), 1413-1426.

Bowles, J.E., 1988, Foundation Analysis and Design, Fourth Edition, McGraw-Hill Book Company.

California Geologic Survey-SP117A, 2008, Guidelines for Evaluating and Mitigating Seismic Hazards in California.

California Geological Survey, 2018, Earthquake Fault Zones, Special Publication 42, Revised 2018.

County of Riverside, 2000, Technical Guidelines for Review of Geotechnical and Geologic Reports, Transportation and Land Management Agency, http://rctlma.org/Portals/4/Handouts/Grading/technical_guide_geotechnical_geologic_reports.pdf

Dept. of the Navy, 1986, NAVFAC DM 7.01: Soil Mechanics, Naval Facilities Engineering Command, Alexandria, Virginia.

Dept. of the Navy, 1986, NAVFAC DM 7.02: Foundations and Earth Structures, Naval Facilities Engineering Command, Alexandria, Virginia.

Department of Water Resources (DWR), 2023, Groundwater Level Data, http://well.water.ca.gov/gw/.

Dept. of Water Resources, 1964, Bulletin No. 108.

FEMA, 2023, Map Service Center website http://msc.fema.gov/

Field, E.H., Biasi, G.P., Bird, P., Dawson, T.E., Felzer, K.R., Jackson, D.D., Johnson, K.M., Jordan, T.H., Madden, C., Michael, A.J., Milner, K.R., Page, M.T., Parsons, T., Powers, P.M., Shaw, B.E., Thatcher, W.R., Weldon, R.J., II, and Zeng, Y., 2013, Uniform California earthquake rupture forecast, version 3 (UCERF3)—The time-independent model: U.S. Geological Survey Open-File Report 2013–1165, 97 p., California Geological Survey Special Report 228, and Southern California Earthquake Center Publication 1792, http://pubs.usgs.gov/of/2013/1165/.

Google Earth, Historical Satellite Photographs, (1985 to 2023).

International Code Council [ICC], 2022, California Building Code, 2022 Edition.

Jennings, C.W, 2010, Fault Activity Map of California and Adjacent Areas: California Division of Mines and Geology, Geological Data Map No. 6, scale 1:750,000.

Riverside County Flood Control and Water Conservation District, 2011, Design Handbook for Low Impact Development Best Management Practices.

Riverside County Transportation and Land Management Agency, 2013, Riverside County Land Information Service, http://mmc.rivcoit.org/MMC_Public/Custom/disclaimer/Default.htm (Parcel Reports performed for APN Nos. 694-060-011/012)

Sneed, Michelle, et al, 2014, Land Subsidence, Groundwater Levels, and Geology in the Coachella Valley, California, United States Geological Survey, Scientific Investigations Report 2014-5075.

Sneed, Michelle and Brandt T. Justin, 2020, Detection and Measurement of Land Subsidence and Uplift Using Global Positioning System Surveys and Interferometric Synthetic Aperture radar, Coachella Valley, California, 2010-17; Scientific Investigations Report 2020-5093; United States Geological Survey (USGS), 86 pages, digital copy.

Southern California Earthquake Center (S.C.E.C.), 1999, Recommended Procedures for Implementation of DMG Special Publication 117, Guidelines for Analyzing and Mitigating Liquefaction in California: available at web site: http://www.scecdc.scec.org.

Structural Engineers Association of California (SEAOC)/California's Office of Statewide Health Planning and Development (OSHPD), 2022, U.S. Seismic Design Maps Website, https://seismicmaps.org/.

Tokimatsu, K, and Seed, H.B., 1987, Evaluation of Settlements in Sands Due To Earthquake Shaking, ASCE, Journal of Geotechnical Engineering, Vol. 113, No. 8, August 1987.

United States Geologic Survey, 2023, Design Maps website: http://ehp3-earthquake.wr.usgs.gov/designmaps/.

United States Geological Survey, 2014, Land Subsidence, Groundwater Levels, and Geology in the Coachella Valley, California, 1993-2010, Scientific Investigations Report 2014-5075

Wallace, R. E., 1990, the San Andreas Fault System, California: U.S. Geological Survey Professional Paper 1515, 283 p.

Youd, T.L., and Garris, C.T., 1995, Liquefaction-Induced Ground-Surface Disruption, Journal of Geotechnical Engineering, Vol. 121, No. 11, November 1995.

Youd, T.L., and Idriss, I.M., 2001, Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER/NSF Workshops on Evaluation of Liquefaction Resistance of Soils, Journal of Geotechnical and Geoenvironmental Engineering, Vol. 127, No. 10, October 2001.

Youd, T.L., Hansen, C.M., and Bartlett, S.F., 2002, Revised Multilinear Regression Equations for Prediction of Lateral Spread Displacement, Journal of Geotechnical and Geoenvironmental Engineering, Vol. 128, No. 12, December 2002.

Zhang, G., Robertson, P.K., and Brachman, R.W.I. (2004). "Estimating Liquefaction-Induced Lateral Displacements Using the Standard Penetration Test or Cone Penetration Test. "J. Geotech. Geoenviron. Eng., 130(8): 861-871.

APPENDIX A

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

LEGEND

▢ Approximate Site Boundary

Approximate Scale: 1" = 1 Mile

0   1 Mile   2 Miles

N

Plate 1
Site Location Map

Santa Cruz Project (Six Lots)
NEC of Calle Santa Cruz and Calle De Ricardo
Palm Springs, Riverside County, California

Earth Systems

11/14/2023 | File No.: 306336-002

Ramon Road

Calle Santa Cruz

⊕ B-2

⊕ B-3   ⊕ B-4

⊕ B-1

Calle De Ricardo

Source: Google Earth satellite image dated 10/31/2022.

Plate 2
Exploration Location Map

Santa Cruz Project (Six Lots)
NEC of Calle Santa Cruz and Calle De Ricardo
Palm Springs, Riverside County, California

Earth Systems

11/14/2023   File No.: 306336-002

LEGEND

Symbol	Description		Symbol	Description
Qs	Dune sand		oc	Oligocene nonmarine
Qal	Alluvium		Ec	Eocene nonmarine
Ql	Lake deposits		E	Eocene marine
Qg	Glacial deposits		Ep	Paleocene marine
Qt	River terrace deposits		Tc	Tertiary nonmarine
Qm	Pleistocene marine and marine terrace deposits		T	Tertiary marine
Qpv	Pleistocene volcanic rocks Qpv¹-rhyolite Qpv⁴-andesite Qpv³-basalt Qpv²-pyroclastic rocks		Ti	Tertiary intrusive (hypabyssal) rocks Ti¹-rhyolite Ti⁴-andesite Ti³-basalt
Qc	Pleistocene nonmarine		Tl	Tertiary lake deposits
QP	Plio-Pleistocene nonmarine		Tv	Tertiary volcanic rocks Tv¹-rhyolite Tv⁴-andesite Tv³-basalt Tv²-pyroclastic rocks
	Quaternary and/or Pliocene cinder cones		Ku	Upper Cretaceous marine
Pc	Undivided Pliocene nonmarine		Ju	Upper Jurassic marine
Pu	Upper Pliocene marine		gr	Mesozoic granitic rocks
Pmlc	Middle and/or lower Pliocene nonmarine		bi	Mesozoic basic intrusive rocks
Pml	Middle and/or lower Pliocene marine		ub	Mesozoic ultrabasic intrusive rocks
Pv	Pliocene volcanic rocks Pv¹-rhyolite Pv⁴-andesite Pv³-basalt Pv²-pyroclastic rocks		Jtv	Jurassic-Triassic metavolcanic rocks
Mc	Undivided Miocene nonmarine		ls	Pre-Cretaceous metamorphic rocks (ls=limestone)
Muc	Upper Miocene nonmarine		ms	Pre-Cretaceous metasedimentary rocks
Mu	Upper Miocene marine		mv	Pre-Cretaceous metavolcanic rocks
Mm	Middle Miocene marine		gr-m	Pre-Cenozoic granitic and metamorphic rocks
Ml	Lower Miocene marine		pCc	Precambrian igneous and metamorphic rock complex
Mv	Miocene volcanic rocks Mv¹-rhyolite Mv⁴-andesite Mv³-basalt Mv²-pyroclastic rocks		pCc	Undivided Precambrian metamorphic rocks pCg=gneiss pCs=schist pCls=limestone and/or dolomite
			pCgr	Undivided Precambrian granitic rocks

— Fault: Dashed where approximate, dotted where concealed

Source: USGS Geologic Map of California, Santa Ana & San Bernardino sheets.

Approximate Site Location

Approximate Scale: 1" = 6 Miles

0   6 Miles   12 Miles

Plate 3
Regional Geologic Map

Santa Cruz Project (Six Lots)
NEC of Calle Santa Cruz and Calle De Ricardo
Palm Springs, Riverside County, California

Earth Systems

11/14/2023   File No.: 306336-002
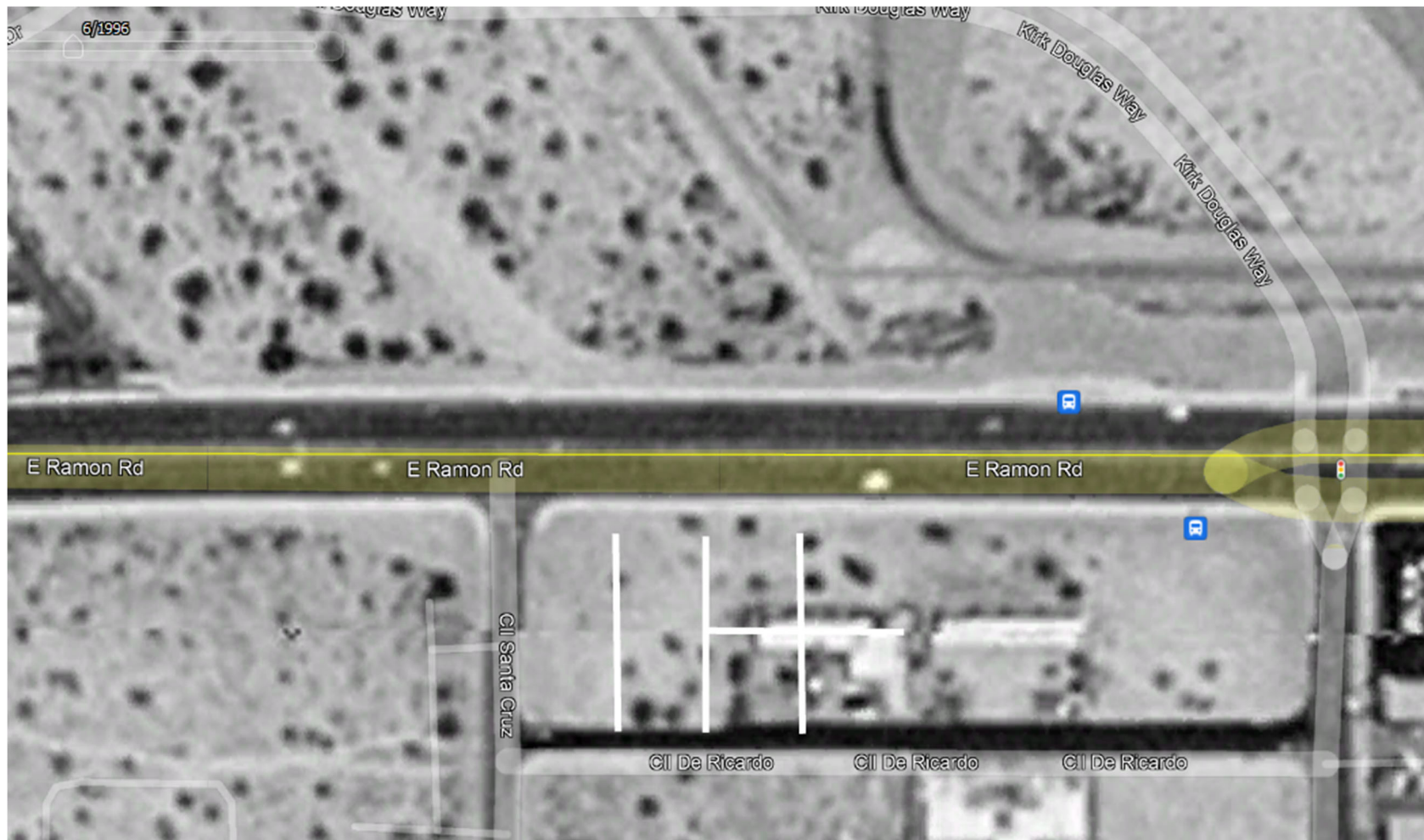
Figure 1   1996 Aerial Photo Showing Structure Southeast Portion of Project
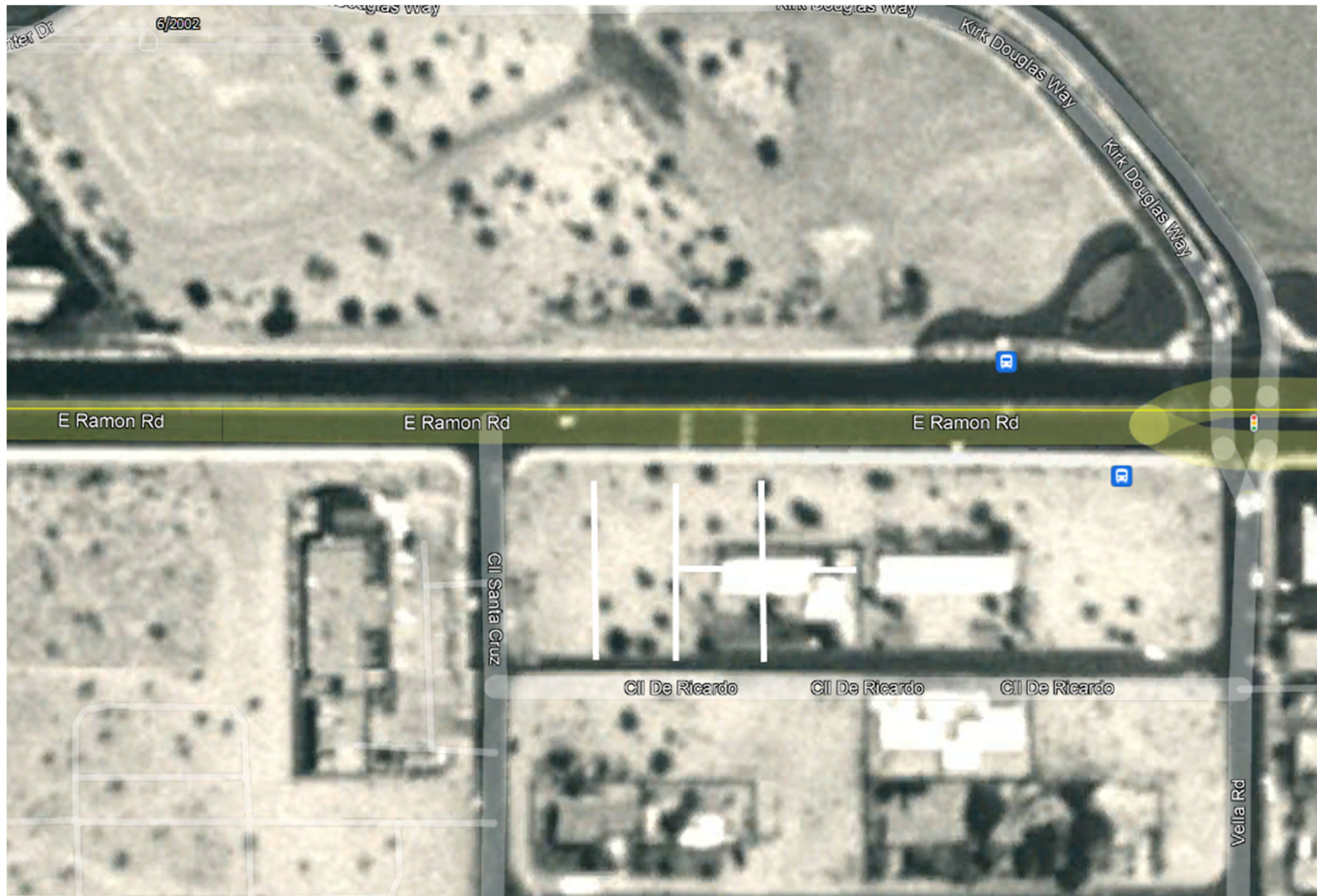
Figure 2   2002 Aerial Photo Building Still on Site
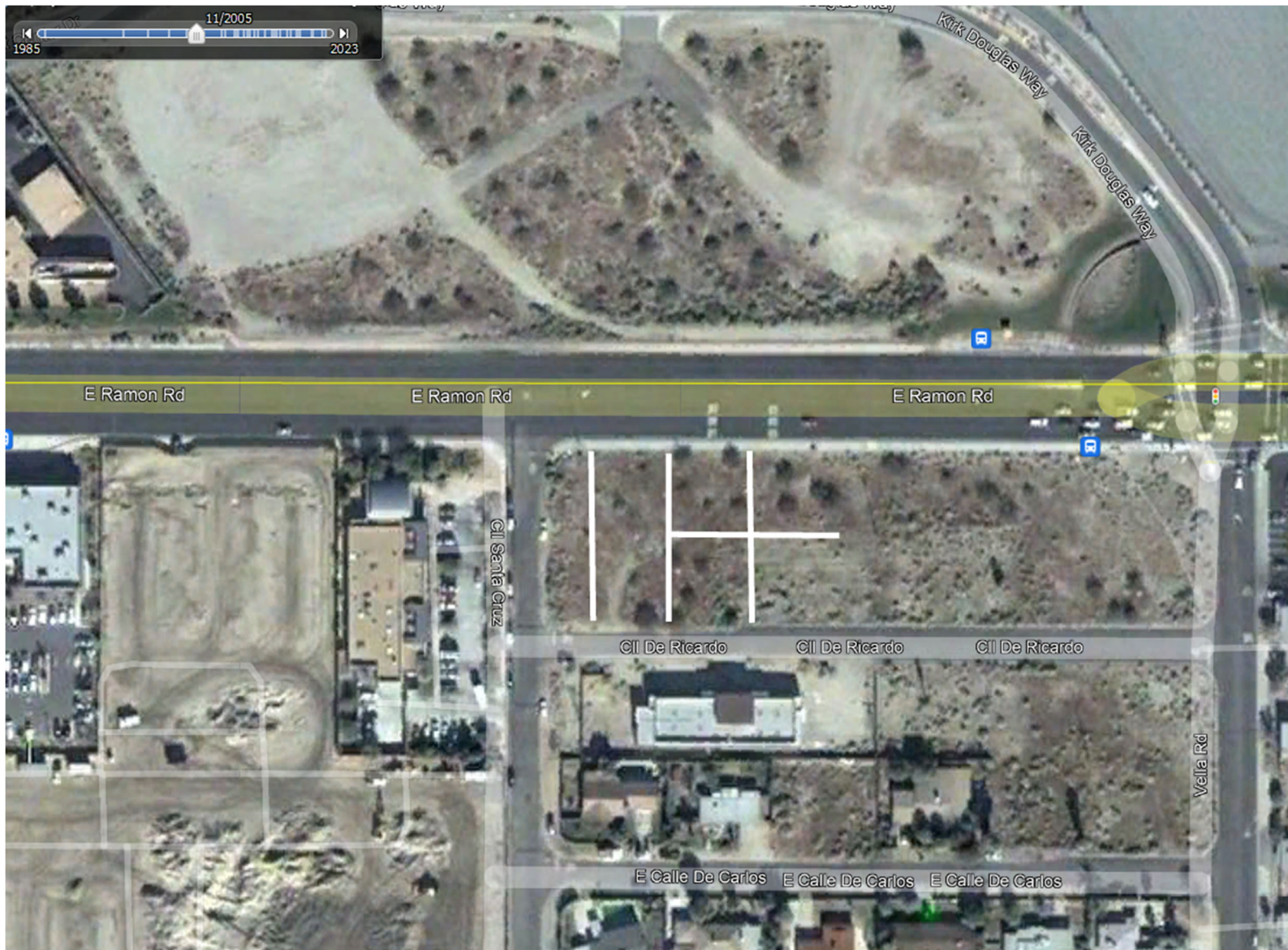
Figure 3   2005 Aerial Photo Showing Structure Removed

Figure 4   2017 Aerial Photo
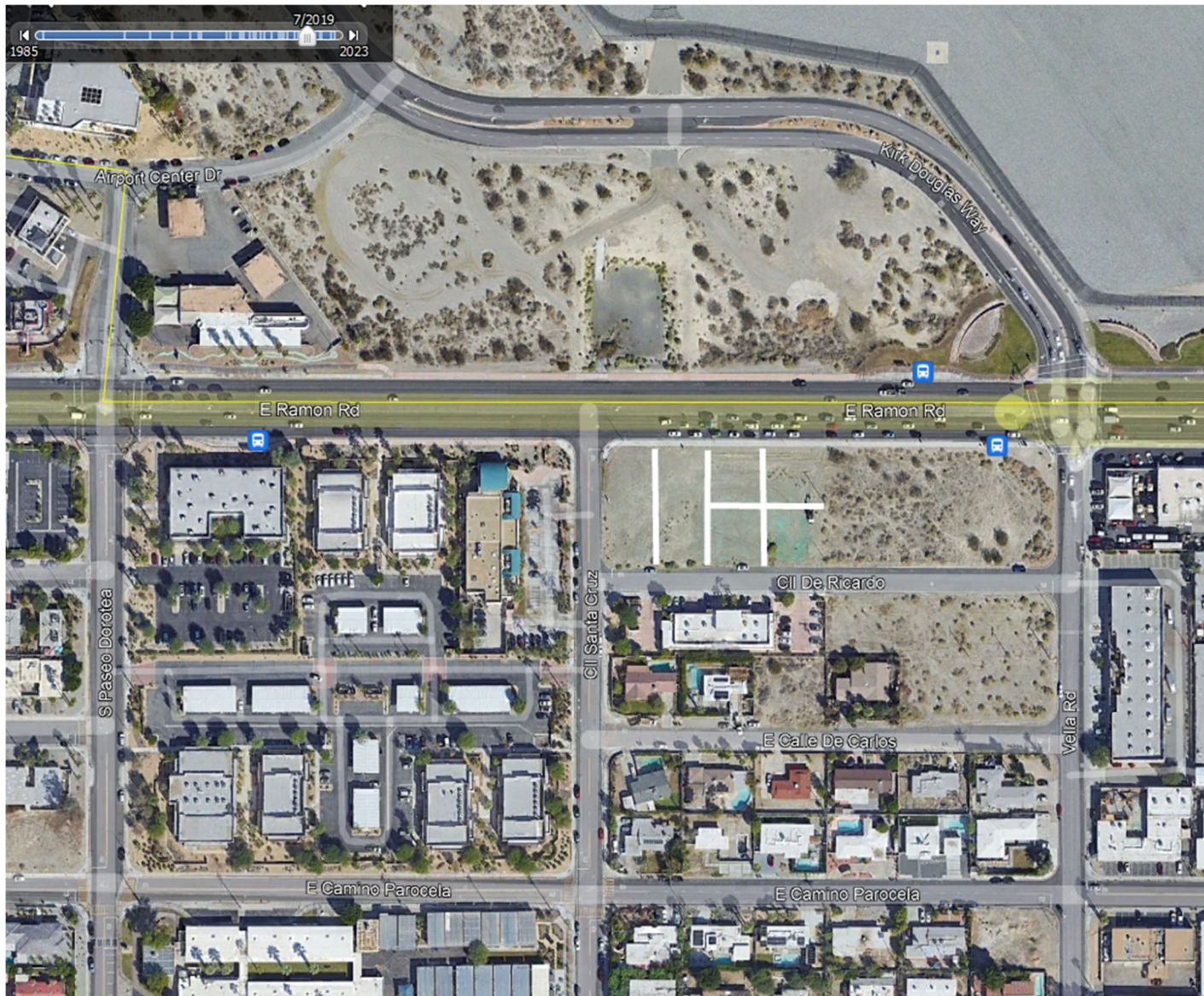
Figure 5   2019 Aerial Photo Showing Grading and Use of Fugitive Dust Glue
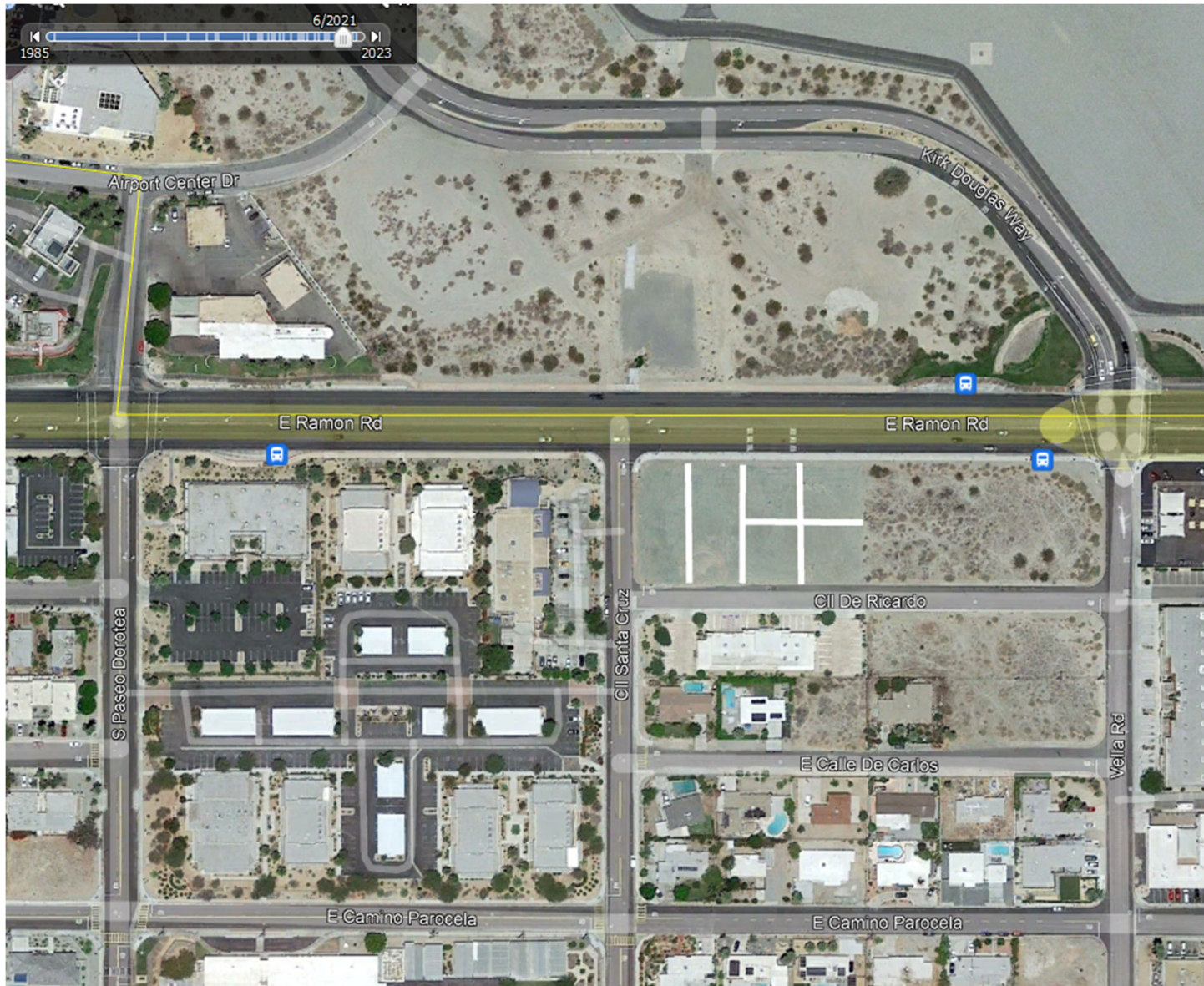
Figure 6 2021 Aerial Photo Showing Additional Grading and Use of Fugitive Dust Control Glue
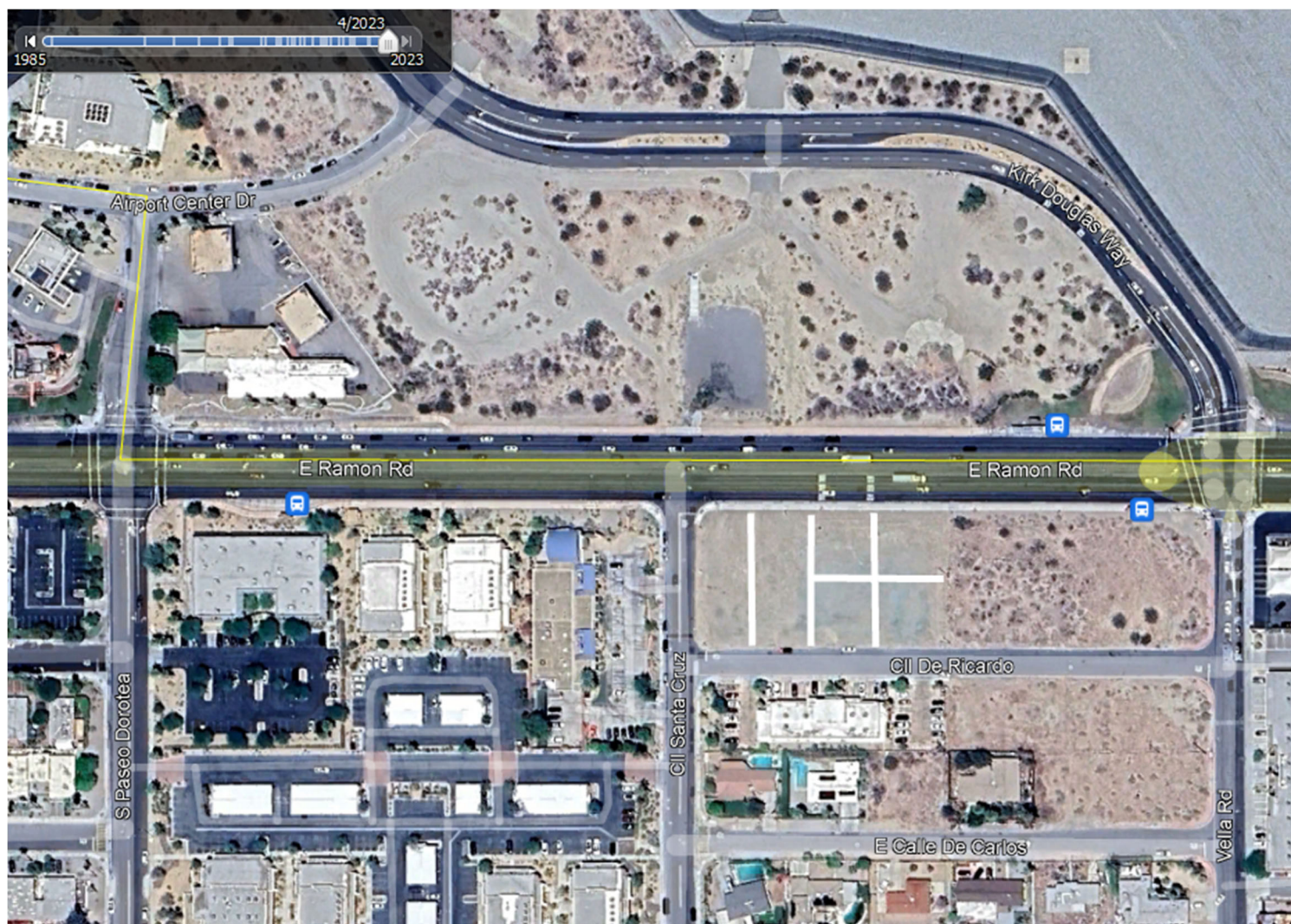
Figure 7   2023 Aerial Photo

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 Andreas (San Gorgonio Pass-Garnet HIll) FM3	3.9	6.2	0.0	12.8	58	20	180	56	A	7.6	219	24
San Andreas, (North Branch, Mill Creek) FM3.1, 3	7.6	12.2	0.0	18.2	76	204	180	106	A	7.6	219	34
Burnt Mtn FM3.1, 3.2	10.3	16.6	0.0	15.9	67	265	180	21	B	6.7		0.6
Eureka Peak FM3.1, 3.2	13.4	21.6	0.0	15.0	90	75	180	19	B	6.6		0.6
Mission Creek FM3.1, 3.2	13.7	22.0	0.0	17.7	65	5	180	31	B'	6.9		
Joshua Tree (Seismicity) FM3.1, 3.2	13.9	22.4	0.0	13.3	90	271	na	17	B'	6.5		
San Andreas (Coachella) rev FM3.1, 3.2	14.6	23.5	0.0	11.1	90	224	180	69	A	6.8	89	9
Blue Cut FM3.1, 3.2	14.7	23.7	0.0	13.1	90	177	na	79	B'	7.1		
San Gorgonio Pass FM3.1, 3.2	18.3	29.4	0.0	18.5	60	11	na	29	B'	6.9		
Pinto Mtn FM3.1, 3.2	18.8	30.2	0.0	15.5	90	175	0	74	B	7.2		2.5
San Jacinto (Anza) rev FM3.1, 3.2	18.8	30.2	0.0	16.8	90	216	180	46	A	7.6	219	17
San Andreas (San Bernardino S) FM3.1, 3.2	20.9	33.6	0.0	12.8	90	210	180	43	A	7.6	150	29
San Jacinto (Clark) rev FM3.1, 3.2	22.5	36.3	0.0	16.8	90	214	180	47	A	7.6	219	17
San Jacinto (San Jacinto Valley, stepover)	23.6	37.9	0.0	16.1	90	224	180	24	A	7.6	219	9
San Jacinto (Coyote Creek) FM3.1, 3.2	24.1	38.8	0.0	15.9	90	223	180	43	A	7.6	219	17
San Jacinto (Stepovers Combined) FM3.1, 3.2	24.5	39.4	0.0	16.5	90	229	180	25	A	7.5	110	4
Johnson Valley (No) 2011 rev FM3.1, 3.2	24.7	39.8	0.0	15.9	90	51	180	52	B	6.8		0.6
Homestead Valley FM3.1, 3.2	26.0	41.9	0.0	15.9	90	na	na	46	B'	7.0		
Emerson-Copper Mtn FM3.1, 3.2	29.8	47.9	0.0	14.1	90	51	180	54	B	7.0		0.6
Calico-Hidalgo FM3.1, 3.2	31.6	50.9	0.0	13.9	90	52	180	117	B	7.4		1.8
Kickapoo FM3.1, 3.2	34.1	54.9	0.0	15.1	90	na	na	6	B'	6.1		
North Frontal (East) FM3.1, 3.2	34.2	55.0	0.0	16.6	41	187	90	28	B	6.9		0.5
San Jacinto (San Jacinto Valley) rev FM3.1, 3.2	34.4	55.4	0.0	16.1	90	223	180	18	A	7.6	219	18
Pisgah-Bullion Mtn-Mesquite Lk FM3.1, 3.2	35.6	57.3	0.0	13.1	90	60	180	91	B	7.3		0.8
Earthquake Valley (No Ext.) FM3.1, 3.2	36.6	58.8	0.0	18.8	90	221	180	33	B'	6.9		
Lenwood-Lockhart-Old Woman Springs FM3.1, 3	37.8	60.8	0.0	13.2	90	43	180	145	B	7.5		0.9
Helendale-So Lockhart FM3.1, 3.2	39.5	63.6	0.0	12.8	90	51	180	114	B	7.4		0.6
Elsinore (Temecula) rev FM3.1, 3.2	41.9	67.4	0.0	14.2	90	230	180	40	A	7.4	431	3
Elsinore (Julian) FM3.1, 3.2	42.0	67.7	0.0	18.8	84	36	180	75	A	7.6	725	2.5
North Frontal (West) FM3.1, 3.2	43.3	69.7	0.0	15.7	49	171	90	50	B	7.2		1
Earthquake Valley FM3.1, 3.2	44.1	71.0	0.0	18.8	90	217	180	20	B	6.7		2
Bullion Mountains FM3.1, 3.2	44.3	71.4	0.0	13.0	90	na	na	38	B'	6.8		
San Jacinto (San Bernardino) FM3.1, 3.2	44.5	71.7	0.0	16.1	90	225	180	37	A	7.6	219	17
San Jacinto (Borrego) FM3.1, 3.2	45.3	72.9	0.0	13.1	90	223	180	34	A	7.6	219	17
Elsinore (Glen Ivy stepover) FM3.1, 3.2	46.4	74.7	0.0	13.2	90	216	180	11	A	7.1	322	15
Elsinore (Stepovers Combined) FM3.1, 3.2	47.1	75.8	0.0	13.7	90	224	180	12	A	7.6	725	5
San Andreas (San Bernardino N) FM3.1, 3.2	47.4	76.3	0.0	12.8	90	212	180	35	A	7.5	103	27
Cleghorn Pass FM3.1, 3.2	47.5	76.4	0.0	13.0	90	na	na	20	B'	6.5		
Hector Mine FM3.1, 3.2	48.6	78.3	0.0	14.6	90	246	na	28	B'	6.7		
Cleghorn Lake FM3.1, 3.2	50.1	80.6	0.0	13.0	90	na	na	29	B'	6.7		

Reference: USGS OFR 2013-1165 (CGS SP 228)   Based on Site Coordinates of 33.8156 Latitude, -116.4986 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: 33.816   N   116.499 W

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

Day	Year	Epicenter Latittude (Degrees)	Longitude	Distance from Site (mi)	Magnitude M_W
6/29	1992	33.87	116.27	13.6	**5.5**
7/24	1947	34.02	116.48	14.2	**5.3**
7/8	1986	34.00	116.61	14.2	**6.0**
4/23	1992	33.96	116.32	14.3	**6.2**
6/30	1992	34.00	116.36	15.0	**5.1**
12/16	1988	33.98	116.68	15.4	**5.0**
7/25	1947	34.05	116.42	16.8	**5.2**
6/28	1992	34.06	116.47	17.0	**5.0**
6/12	1944	34.01	116.69	17.3	**5.2**
6/12	1944	34.00	116.72	18.0	**5.1**
8/21	1993	34.03	116.32	18.0	**5.0**
9/15	1992	34.06	116.36	18.6	**5.2**
6/6	1918	33.60	116.70	18.9	**5.5**
10/2	1928	33.60	116.70	18.9	**5.5**
5/18	1940	34.05	116.30	19.8	**5.2**
12/4	1948	34.00	116.23	20.0	**6.0**
6/28	1992	34.10	116.42	20.1	**5.0**
6/12	2005	33.53	116.57	20.1	**5.2**
6/29	1992	34.10	116.40	20.4	**5.7**
6/29	1992	34.10	116.39	20.6	**5.4**
2/25	1980	33.52	116.55	20.6	**5.1**
10/31	2001	33.51	116.51	21.1	**5.1**
6/28	1992	34.12	116.43	21.4	**5.5**
5/18	1940	34.08	116.30	21.5	**5.3**
4/11	1910	33.50	116.50	21.8	**5.8**
6/28	1992	34.13	116.41	22.3	**5.8**
2/7	1889	34.10	116.70	22.8	**5.6**
6/28	1992	34.12	116.32	23.4	**5.7**
3/25	1937	33.46	116.44	24.8	**5.6**
6/10	2016	33.43	116.44	26.7	**5.2**
6/28	1992	34.20	116.44	26.8	**7.3**
7/7	2010	33.42	116.49	27.3	**5.4**
12/25	*1899	33.80	117.00	28.8	**6.7**
4/21	*1918	33.75	117.00	29.1	**6.8**
2/9	*1890	33.40	116.30	30.9	**6.8**
6/28	1992	34.16	116.85	31.1	**5.5**
4/3	1926	34.00	116.00	31.3	**5.5**
11/22	1880	34.00	117.00	31.4	**5.5**
12/19	1880	34.00	117.00	31.4	**5.9**
9/23	1963	33.66	117.02	31.8	**5.3**

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

Table A-1 - General Procedure Seismic Design Values

2022 California Building Code (CBC) (ASCE 7-16, Supplement 3) 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:	33.816		Table 1613.5.2	Table 20.3-1
Longitude:	-116.499			

Maximum Considered Earthquake (MCE) Ground Motion

				CBC Reference	ASCE 7-16 Reference
Short Period Spectral Reponse	S_S	**1.759 g**		Figure 1613.5	Figure 22-1
1 second Spectral Response	S_1	**0.729 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}	1.759 g	$= F_a \times S_S$		
	S_{M1}	1.239 g	$= F_v \times S_1$		
ASCE7-16 Supplement 3, Exception 1*	$S_{M1}*$	1.859 g	$= S_{M1} \times 1.5$	Exception 1, 11.4.8, Supplement 3	

Design Earthquake Ground Motion

Short Period Spectral Reponse	S_{DS}	**1.173 g**	$= 2/3 \times S_{MS}$	
Unadjusted 1 second Spectral Response	S_{D1}	**0.826 g**	$= 2/3 \times S_{M1}$	
Modified 1 second Spectral Response*	$S_{D1}*$	**1.239 g**	$= 2/3 \times S_{M1}*$	*Exception 1, 11.4.8, Supplement 3

Site Specific Evaluation May Be Required for Site Class=D or E & S1>=0.2. The Above SDS and SD1 are NOT Valid Unless the Exception of ASCE7-16 Supplement 3, Section 11.4.8 Applies & the SM1* & SD1* Values are used for Site Class D.

To	0.14	sec	$= 0.2 * S_{D1}/S_{DS}$
Ts (11.4.8 ASCE 7-16 Exception Assumed)	0.70	sec	$= S_{D1}/S_{DS}$
Risk Category	II		Table 1604.5
Seismic Importance Factor	1.00		
F_{PGA}	1.10		
PGA_M	**0.84**		
Vertical Coefficient (C_V)	1.45		Table 11.9-1

2022 CBC Equivalent Elastic Static Response Spectrum

Table 11.5-1	Design
Period	Sa
T (sec)	(g)
0.00	0.469
0.05	0.719
0.14	1.173
0.70	1.173
0.90	0.918
1.10	0.751
1.30	0.636
1.50	0.551
1.70	0.486
1.90	0.435
2.10	0.393
2.30	0.359
2.50	0.330
2.70	0.306
2.90	0.285
3.10	0.267
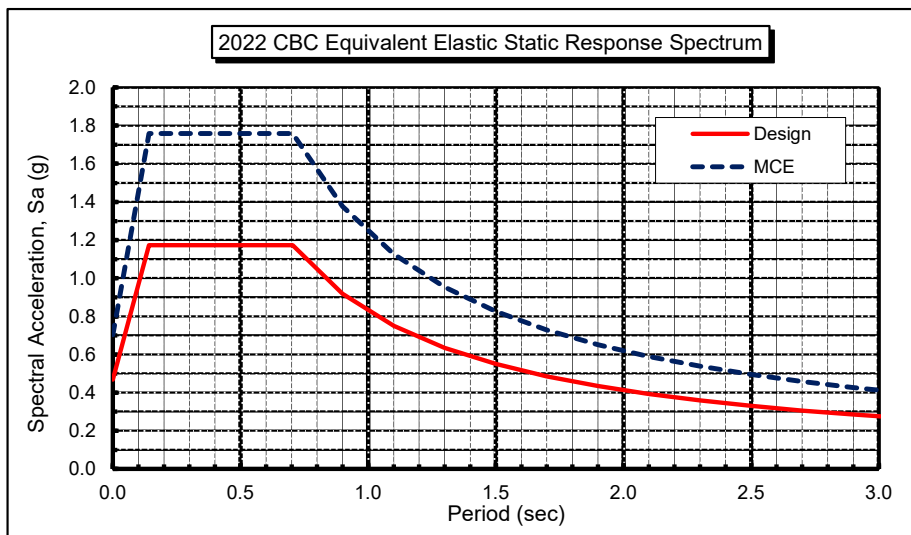
DESCRIPTIVE SOIL CLASSIFICATION

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
Firm	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
Slightly Moist.....Very quick (less than 1 minute) color change when exposed to air (granular soil), Below optimum (granular)
Moist.................Color change with period of air exposure (granular soil) Below optimum moisture content (cohesive soil)
Very Moist.........High degree of saturation by visual and touch (granular soil) Above optimum moisture content (cohesive soil), No free water
Wet...................Free surface water

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.

RELATIVE PROPORTIONS

Trace.............minor amount (<5%)
some.............significant amount
with................(Typically greater than 15%)
modifier/and...sufficient amount to influence material behavior (Typically >30%)

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)
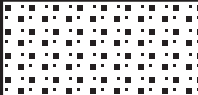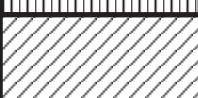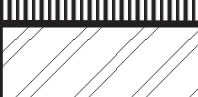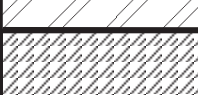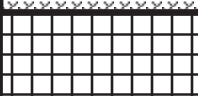
Terms and Symbols Used on Boring Logs

Earth Systems

MAJOR DIVISIONS			GRAPHIC SYMBOL	LETTER SYMBOL	TYPICAL DESCRIPTIONS
COARSE GRAINED SOILS More than 50% of material is _larger_ than No. 200 sieve size	GRAVEL AND GRAVELLY SOILS More than 50% of coarse fraction _retained_ 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 _passing_ 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 _smaller_ than No. 200 sieve size	SILTS AND CLAYS	LIQUID LIMIT _LESS_ 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 _GREATER_ 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

Boring No. B-1

Project Name: Santa Cruz Project (Six Lots)
Project Number: 306336-002
Boring Location: See Plate 2

Drilling Date: October 4, 2023
Drilling Method: B-61 CME w/auto hammer
Drill Type: 8" HSA
Logged By: TC

Depth (Ft.)	Sample Type — Bulk	Sample Type — SPT	Sample Type — MOD Calif.	Penetration Resistance (Blows/6")	Symbol	USCS	Dry Density (pcf)	Moisture Content (%)	Description of Units	Graphic Trend Blow Count / Dry Density
0						SP-SM			**POORLY GRADED SAND WITH SILT:** olive brown, moist, loose, fine to medium grained sand	
				4,4,7			102	5		
5				4,6,11			102	3	damp, medium dense	
				6,7,9					no recovery	
10				6,14,16			104	0	dry, fine grained sand	
				5,6,9						
15				5,7,11			104	1		
20				7,11,15		SP		2	**POORLY GRADED SAND WITH GRAVEL:** light olive brown, dry, dense, fine to coarse grained sand	

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

Page 1 of 1

Boring completed at 21-1/2 feet
No groundwater encountered
Backfilled with cuttings

Earth Systems

Boring No. B-2

Project Name: Santa Cruz Project (Six Lots)
Project Number: 306336-002
Boring Location: See Plate 2

Drilling Date: October 4, 2023
Drilling Method: B-61 CME w/auto hammer
Drill Type: 8" HSA
Logged By: TC

Depth (Ft.)	Sample Type (Bulk / SPT / MOD Calif.)	Penetration Resistance (Blows/6")	Symbol	USCS	Dry Density (pcf)	Moisture Content (%)	Description of Units	
				SP-SM			**POORLY GRADED SAND WITH SILT:** olive brown, moist, loose, fine to medium grained sand	Page 1 of 1
		3,5,7			103	3	damp	
5		4,7,10					no recovery, medium dense	
		6,9,12		SP-SM			**POORLY GRADED SAND WITH SILT:** light olive brown, dry, medium dense, fine grained sand	
10		4,7,8						
		6,7,12		SM	104	0	**SILTY SAND:** light olive brown, dry, medium dense, fine grained sand	
15		5,6,7						
20		10,16,30		SW			**WELL GRADED SAND WITH GRAVEL:** light olive brown, dry, dense, fine to coarse grained sand, fine gravel	

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

Boring completed at 21-1/2 feet
No groundwater encountered
Backfilled with cuttings

Earth Systems

Boring No. B-3

Project Name: Santa Cruz Project (Six Lots)
Project Number: 306336-002
Boring Location: See Plate 2

Drilling Date: October 4, 2023
Drilling Method: B-61 CME w/auto hammer
Drill Type: 8" HSA
Logged By: TC

Depth (Ft.)	Sample Type			Penetration Resistance (Blows/6")	Symbol	USCS	Dry Density (pcf)	Moisture Content (%)	Description of Units
	Bulk	SPT	MOD Calif.						

Description of Units

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

Page 1 of 1

Graphic Trend
Blow Count Dry Density

Depth	SPT	Dry Density	Moisture	USCS	Description
~2.5	5,6,11	104	4	SP-SM	POORLY GRADED SAND WITH SILT: olive brown, moist, loose, fine to medium grained sand / meduim dense, damp
~5.5	4,6,10	104	3		
~10.5	6,9,12	104	1	SP	POORLY GRADED SAND: light olive brown, damp, medium dense, fine to medium grained sand, rust mottling stain 1/8 only one

Boring completed at 11-1/2 feet
No groundwater encountered
Backfilled with cuttings

Earth Systems

Boring No. B-4

Project Name: Santa Cruz Project (Six Lots)
Project Number: 306336-002
Boring Location: See Plate 2

Drilling Date: October 4, 2023
Drilling Method: B-61 CME w/auto hammer
Drill Type: 8" HSA
Logged By: TC

Description of Units

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

Page 1 of 1

Graphic Trend
Blow Count Dry Density

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			POORLY GRADED SAND WITH SILT: light olive brown, damp, loose, fine to medium grained sand
	MOD	6,8,10			104	2	medium dense
5	MOD	5,8,11			103	2	
	MOD	5,8,12			105	1	
10	MOD	6,9,13			107	1	
	SPT	3,4,5					
15	MOD	5,9,12			101	3	fine grained sand
20	SPT	12,22,24		SP-SM			POORLY GRADED SAND WITH SILT AND GRAVEL: light olive brown, damp, very dense, fine grained sand, fine gravel, possible cobble
25	MOD	10,15,22		SP			POORLY GRADED SAND WITH GRAVEL: light olive brown, damp, medium dense, fine to coarse grained sand, fine gravel
30	SPT	11,13,19					no recovery
35		12,21,38				1	no recovery, switching to SPT, fine to coarse grained sand, cutting at 40 - 45, dense, dry
40	SPT	16,25,28		SP			POORLY GRADED SAND WITH GRAVEL: light olive brown, dry, very dense, fine to coarse grained sand, fine gravel
45	MOD	12,19,26		SW		1	WELL GRADED SAND WITH GRAVEL: light olive brown, dry, dense, fine to coarse grained sand, fine gravel
50	SPT	10,11,13		SM			SILTY SAND: light olive brown, moist, dense, fine grained sand, mottling or strip coloing 1/16" thick silt and clay layering
55							Boring completed at 51-1/2 feet
							No groundwater encountered
							Backfilled with cuttings
60							

Boring No.	B-4	Project and Number	Santa Cruz Project	306336-002

ESSW Field Staff — Notes: Soils Remediated t Upper 5 feet

Drilling Company

Drilling Method — 6-8" H S A | HSA Inner Diameter | 3"

Site Latitude (North) — Decimal Degrees

Site Longitude (West) — Decimal Degrees

Date Drilled

Hammer Weight (lbs) — 140

Hammer Drop (inches) — 30

Hammer Efficiency (E_M) — 80

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 — 60
*Only Used for Calculating Nsub otherwise not used by program (i.e.N50, N70, N80, etc)

Calculation Results

Ave. SPT N_{60HE}-value (blows/ft)
32
(Based on Upper 50 feet)

Ave. Shear Wave Velocity (ft/sec)
926
(Based on Upper 50 feet)

Soil Profile Type (Site Class)
D
(Based on Upper 50 feet)

Ave. Friction Angle (degrees)
35
(Based on Upper 50 feet)

Estimated Shear Wave Velocity **
Based on Depth Less than 100' ft
911 (ft/sec Upper 100 feet)

Soil Profile Type (Site Class)**
D
Based on
Ave. Shear Wave Velocity (ft/sec)
278 (m/sec Upper 100 feet)

Ave. Field SPT N-value (blows/ft)
24.1
(Based on Upper 50 feet)

Ave. Field SPT N-value (blows/ft)
27.8
(Based on Upper 100 feet)

Soil Profile Type (Site Class)**
D
Based on
Ave. Field Blow Count
28 (Upper 100 feet)

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

Energy ratio (Skempton, 1986)

→ 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)	N60 (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	57	c	2.5	35.91	35.91	47.88	308.61	1012.24	37.00	0.05221	0.00247	0.067559	Dense	Hard
	5.0	57	c	2.5	35.91	35.91	47.88	308.61	1012.24	37.00	0.05221	0.00247	0.067559	Dense	Hard
	7.5	20	c	2.5	12.60	12.60	16.80	227.77	747.10	31.44	0.14881	0.00335	0.07951	Medium Dense	Stiff
	10.0	22	c	2.5	13.86	13.86	18.48	234.16	768.04	31.88	0.13528	0.00326	0.07841	Medium Dense	Stiff
	12.5	9	s	2.5	12.24	12.24	12.00	206.60	677.65	29.98	0.20833	0.00369	0.083393	Medium Dense	Stiff
	15.0	21	c	2.5	14.99	14.99	17.64	231.02	757.75	31.67	0.14172	0.00330	0.078947	Medium Dense	Stiff
	20.0	46	s	5.0	69.92	69.92	61.33	331.59	1087.61	38.58	0.08152	0.00460	0.129602	Very Dense	Hard
	25.0	37	c	5.0	29.53	29.53	31.08	272.26	893.02	34.51	0.16088	0.00560	0.144891	Very Dense	Very Stiff
	30.0	32	s	5.0	51.20	51.20	42.67	298.46	978.96	36.31	0.11719	0.00511	0.137708	Very Dense	Hard
	35.0	59	c	5.0	49.56	49.56	49.56	311.71	1022.42	37.22	0.10089	0.00489	0.134345	Dense	Hard
	40.0	53	s	5.0	84.80	84.80	70.67	345.49	1133.21	39.53	0.07075	0.00441	0.12648	Very Dense	Hard
	45.0	45	c	5.0	37.80	37.80	37.80	288.16	945.18	35.60	0.13228	0.00529	0.140444	Dense	Hard
	50.0	24	s	5.0	38.40	38.40	32.00	274.57	900.60	34.67	0.15625	0.00555	0.144227	Dense	Hard

Total: 50.0 "d" Feet   Total: 1.55833 | 0.05398 | 1.413074

**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

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

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).

EARTH SYSTEMS - EVALUATION OF LIQUEFACTION POTENTIAL AND INDUCED SUBSIDENCE

Santa Cruz Project   **Project No: 306336-002**   1996/1998 NCEER Method

Ground Compaction Remediated to 4 foot depth

Boring: B-4   **Earthquake Magnitude: 7.9**   **PGA, g: 0.56**   **Calc GWT (feet):   100**
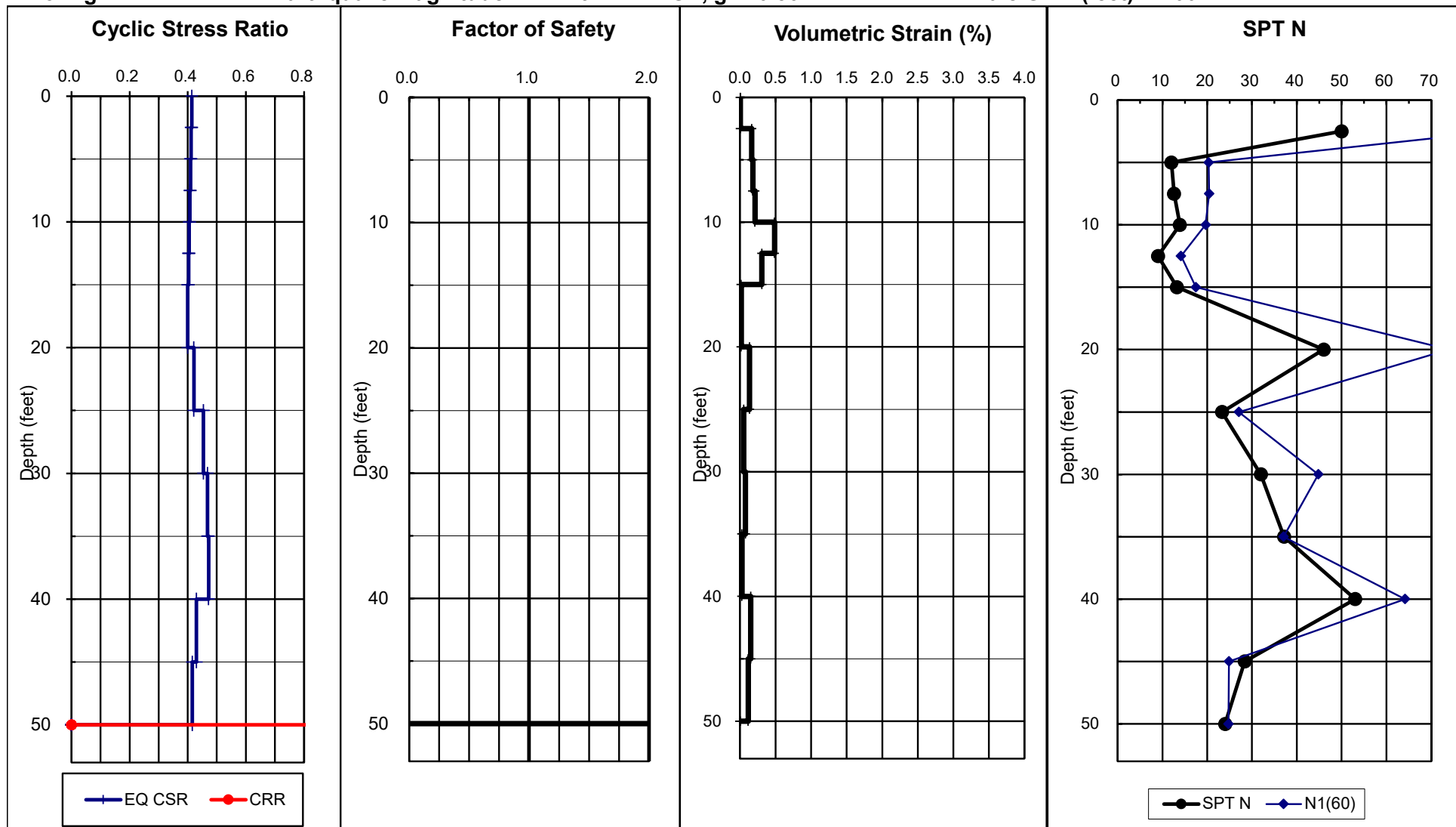
Total Thickness of Liquefiable Layers: 0.0 feet   **Estimated Total Ground Subsidence: 0.8 inches**

EARTH SYSTEMS - SETTLEMENT ANALYSES
Santa Cruz Project Six Lots

306336-002

Width, ft: 6.0   Length, ft: 6.0   Net pressure, ksf: 3.50   **Settlement, inches: 0.7**
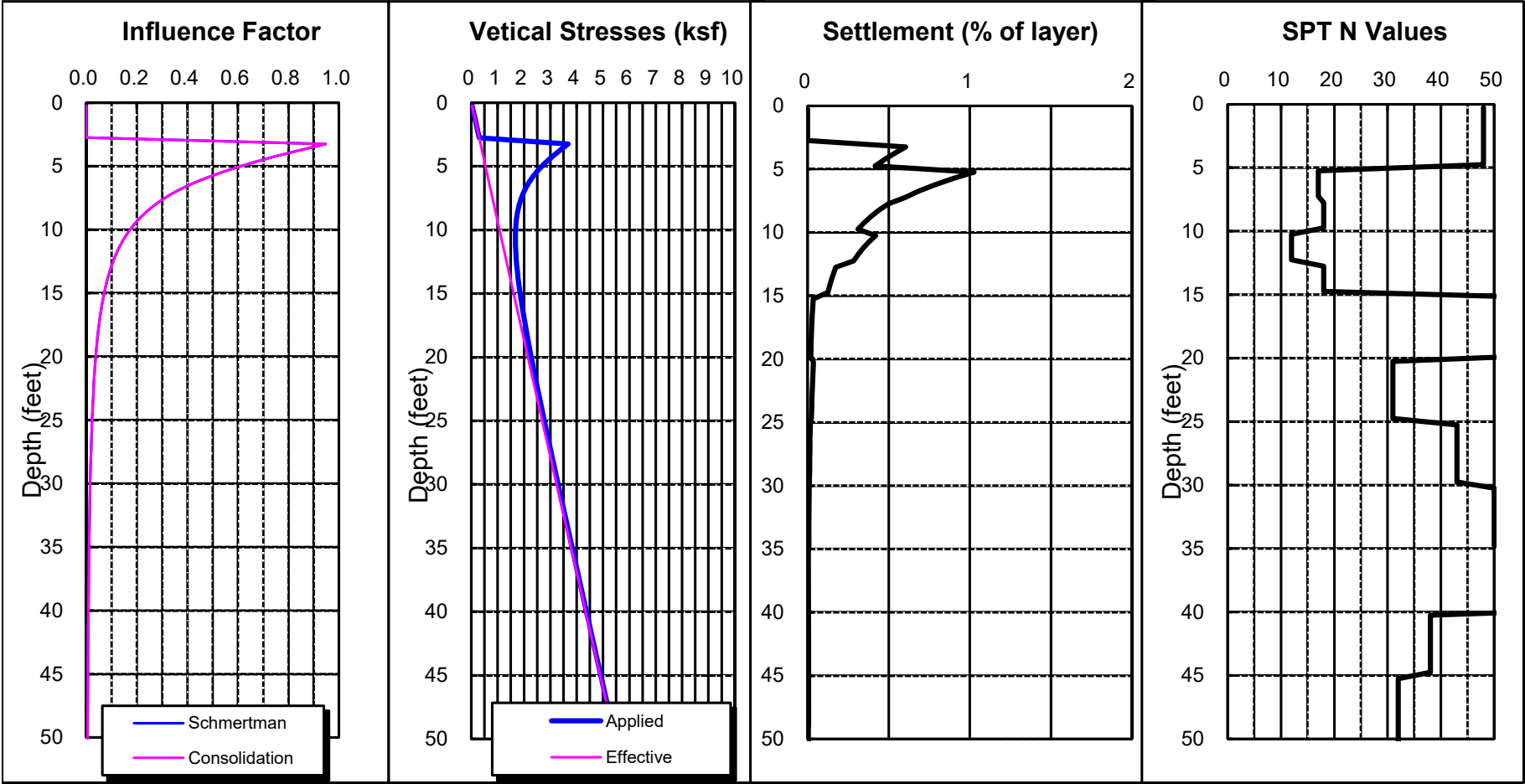
Load, Q: 126 kips   Embedment, feet: 3.0   **Boring: B-4**

EARTH SYSTEMS - SETTLEMENT ANALYSES
Santa Cruz Project Six Lots

306336-002

Width, ft: 2.0   Length, ft: 40.0   Net pressure, ksf: 3.00   **Settlement, inches: 0.3**

Influence Factor

Vetical Stresses (ksf)

Settlement (% of layer)

SPT N Values

Depth (feet)

- Schmertman
- Consolidation

- Applied
- Effective
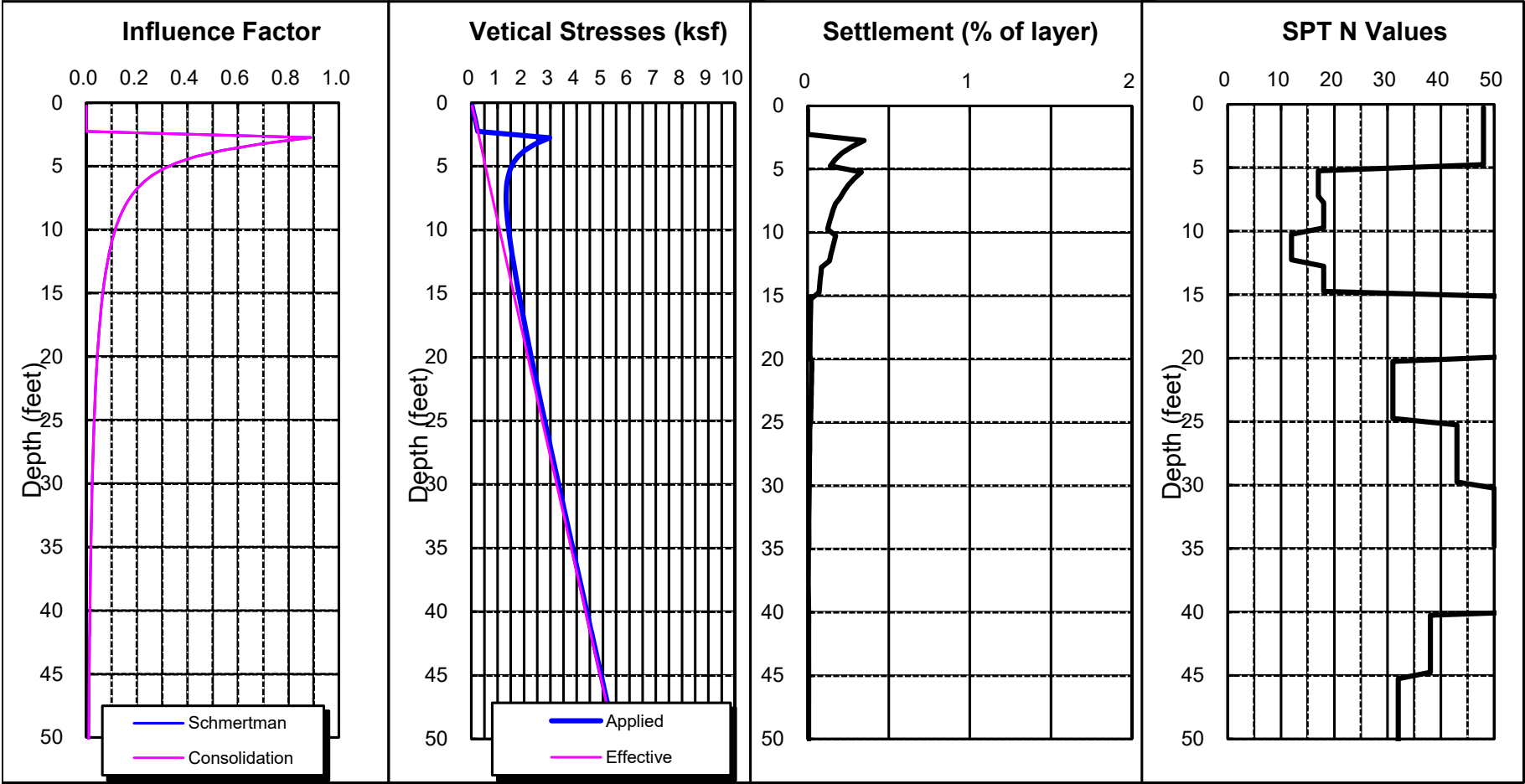
Load, Q: 6 kpf   Embedment, feet: 2.5   **Boring: B-4**

APPENDIX B
Lab Results

Job Name: Santa Cruz Project

Sample Location	Depth (feet)	Unit Dry Density (pcf)	Moisture Content (%)	USCS Group Symbol
B-1	2.5	102	5	SP-SM
B-1	5	102	3	SP-SM
B-1	10	104	0	SP-SM
B-1	15	104	1	SP-SM
B-1	20	---	2	SP
B-2	2.5	103	3	SP-SM
B-2	12.5	104	0	SM
B-3	2.5	104	4	SP-SM
B-3	5	104	3	SP-SM
B-3	10	104	1	SP
B-4	2.5	104	2	SP-SM
B-4	5	103	2	SP-SM
B-4	7.5	105	1	SP-SM
B-4	10	107	1	SP-SM
B-4	15	101	3	SP-SM
B-4	35	---	1	SP
B-4	45	---	1	SW

Job Name: Santa Cruz Project
Sample ID: B-4 @ 0-5'
Description: Poorly Graded Sand with Silt (SP-SM)

Sieve Size	% Passing	
3"	100	-
2"	100	-
1-1/2"	100	-
1"	100	-
3/4"	100	-
1/2"	100	-
3/8"	99	-
#4	98	-
#10	97	-
#16	96	-
#30	89	-
#40	73	-
#100	25	-
#200	8.5	-
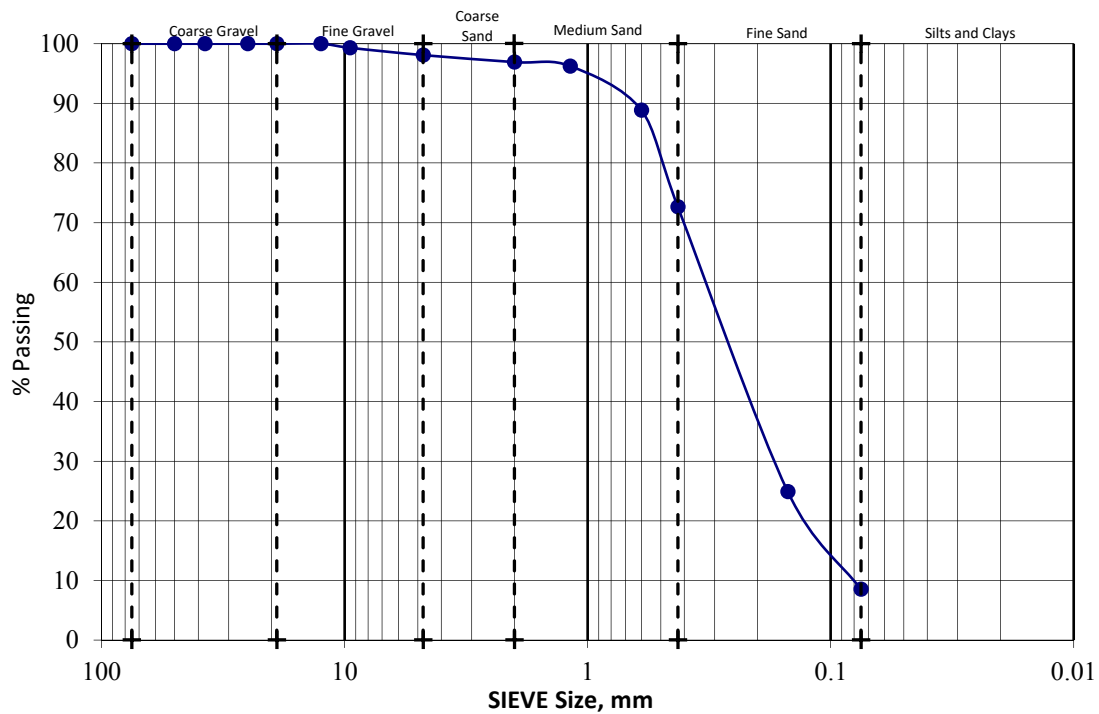
FM= 1.50

% Coarse Gravel:	0	% Coarse Sand:	1				
% Fine Gravel:	2	% Medium Sand:	24		Cu:	4.04	
		% Fine Sand:	64		Cc:	1.091	Gradation
% Total Gravel	2	% Total Sand	90	% Fines:	8.5		Poorly Graded

File No.: 306336-002

Job Name: Santa Cruz Project

Lab Number: 23-293

November 14, 2023

AMOUNT PASSING NO. 200 SIEVE

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

(Earth Systems Method Transfers Sample until water runs clear)

Sample Location	Depth (feet)	Fines Content (%)	USCS Group Symbol	Soaking Time
B-2	15	18.8	SM	10
B-4	7.5	5.7	SP-SM	10

EARTH SYSTEMS PACIFIC

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

Santa Cruz Project   Initial Dry Density: 101.8 pcf
B-1 @ 5'   Initial Moisture: 1.1%
Poorly Graded Sand with Silt (SP-   Specific Gravity: 2.67
SM)   Initial Void Ratio: 0.637
Ring Sample

                                                Hydrocollapse: 0.9% @ 2.0 ksf

% Change in Height vs Normal Pressure Diagram
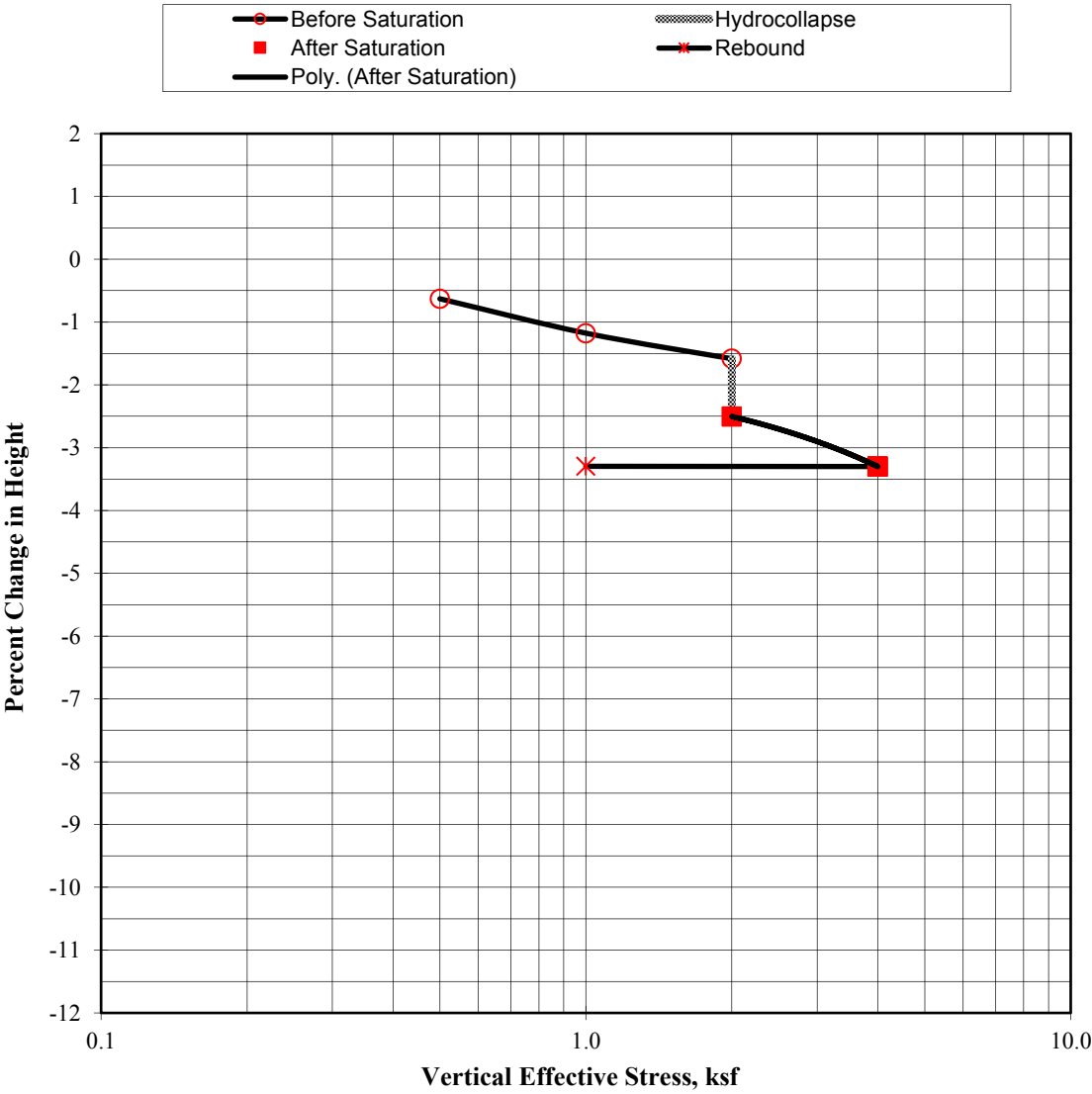
CONSOLIDATION TEST   **ASTM D 2435 & D 5333**

Santa Cruz Project   Initial Dry Density: 103.8 pcf

B-2 @ 12.5'   Initial Moisture: 1.9%

Silty Sand (SM)   Specific Gravity: 2.67

                                      Initial Void Ratio: 0.605

Ring Sample

                                  Hydrocollapse: 0.6% @ 2.0 ksf

% Change in Height vs Normal Pressure Diagram

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

Santa Cruz Project   Initial Dry Density: 104.5 pcf
B-4 @ 7.5'   Initial Moisture: 1.1%
Poorly Graded Sand with Silt (SP-   Specific Gravity: 2.67
SM)   Initial Void Ratio: 0.594
Ring Sample

                                   Hydrocollapse: 0.5% @ 2.0 ksf

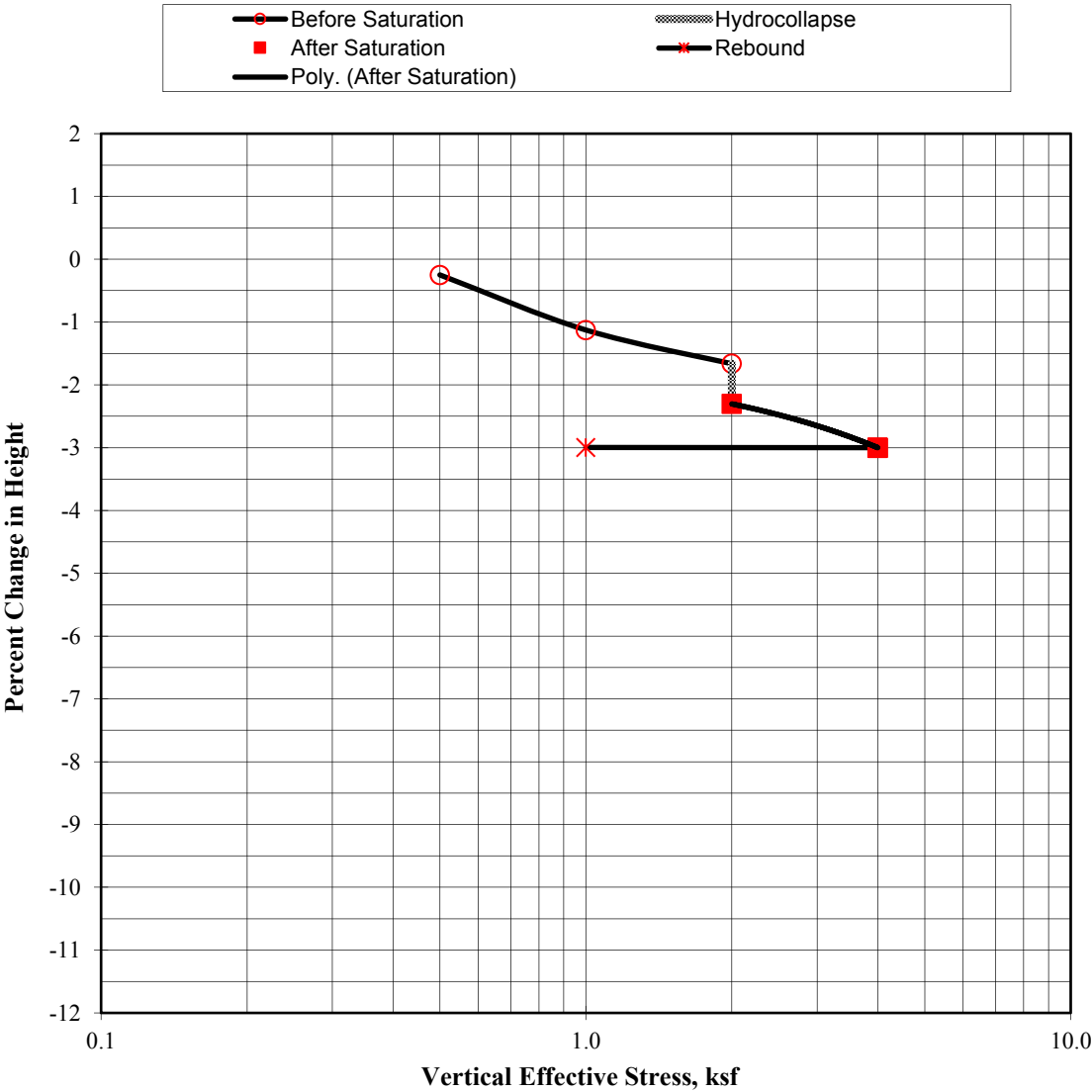### % Change in Height vs Normal Pressure Diagram
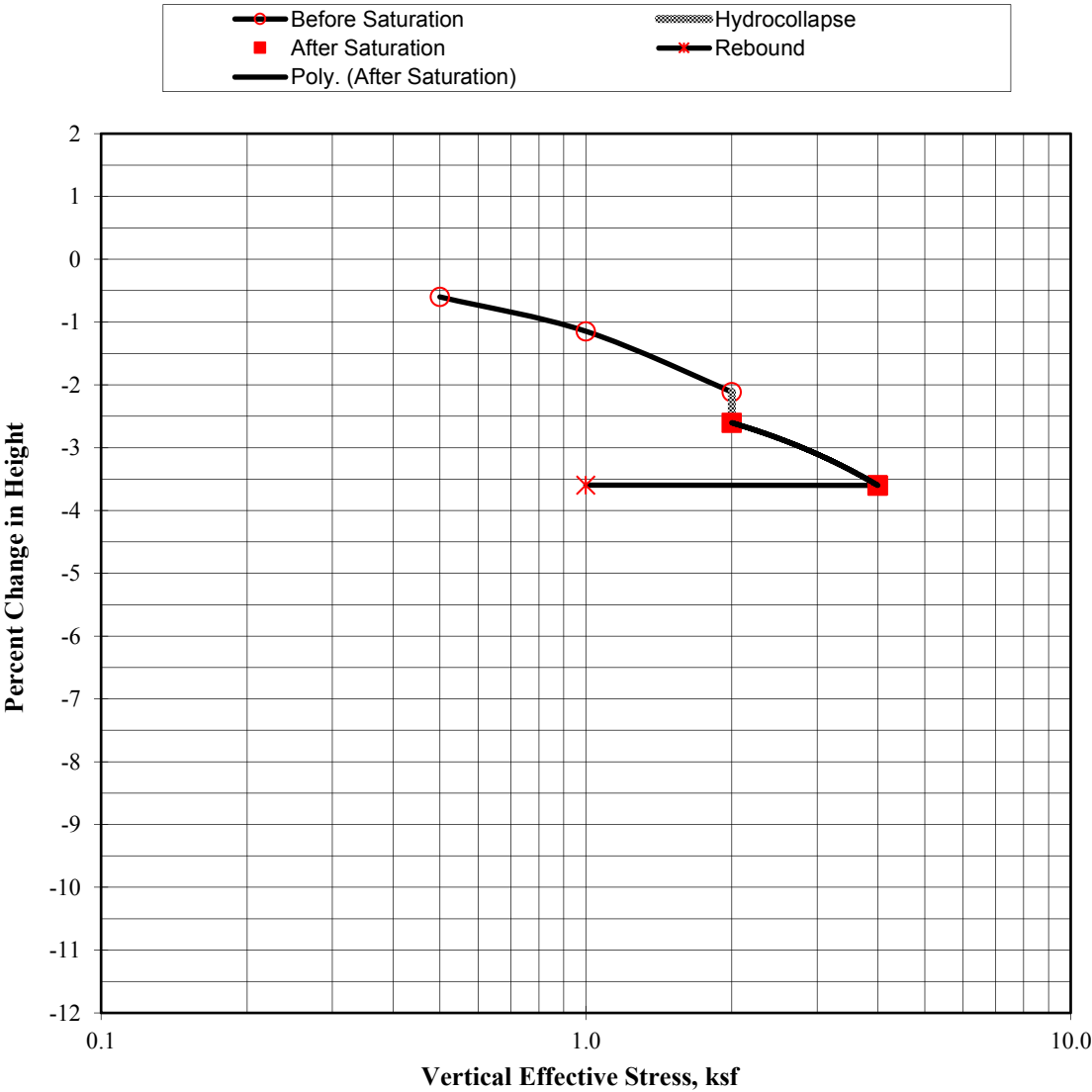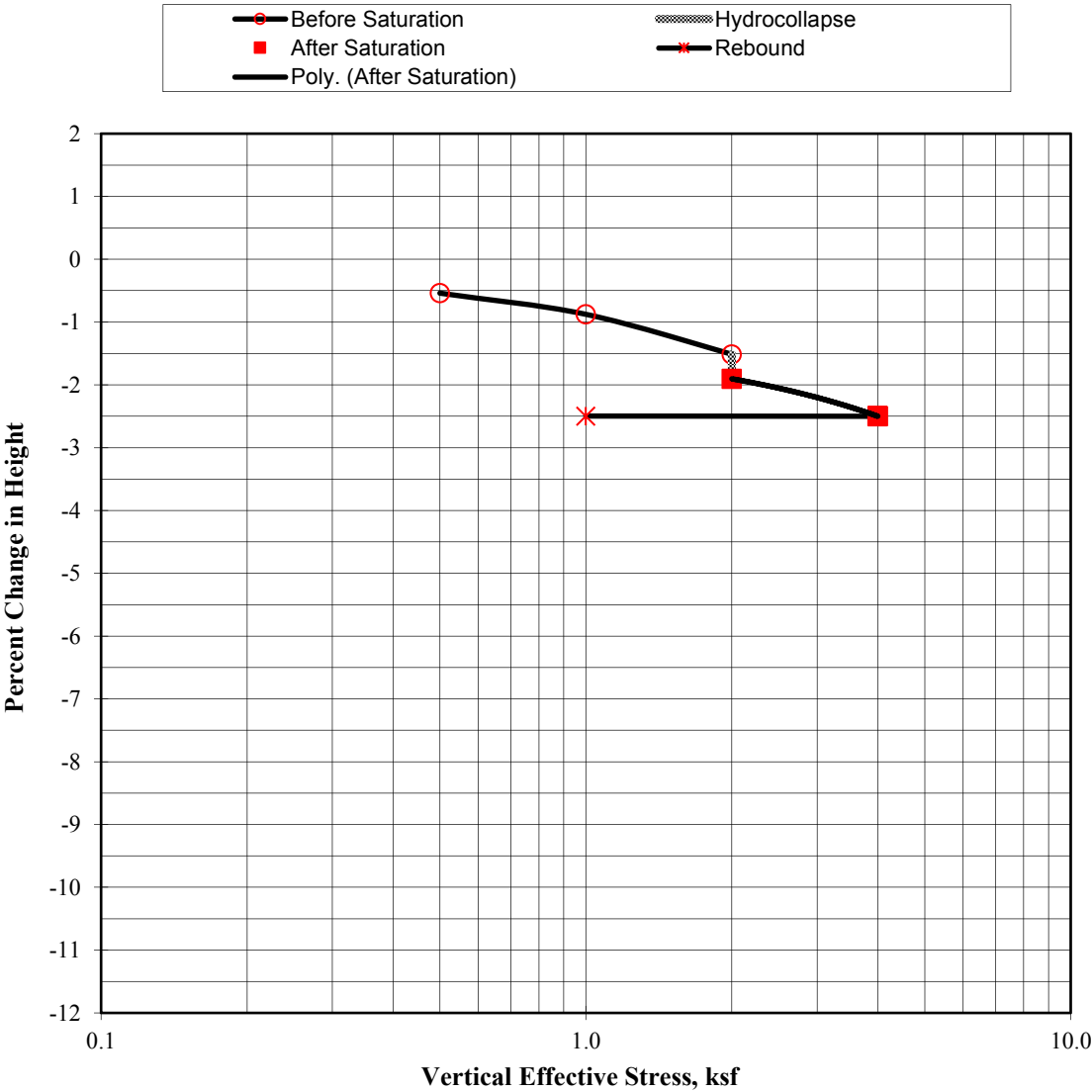
CONSOLIDATION TEST   **ASTM D 2435 & D 5333**

Santa Cruz Project   Initial Dry Density: 106.2 pcf
B-4 @ 10'   Initial Moisture: 0.9%
Poorly Graded Sand with Silt (SP-   Specific Gravity: 2.67
SM)   Initial Void Ratio: 0.569
Ring Sample

                                         Hydrocollapse: 0.4% @ 2.0 ksf

% Change in Height vs Normal Pressure Diagram

Legend:
- Before Saturation
- After Saturation
- Poly. (After Saturation)
- Hydrocollapse
- Rebound

X-axis: Vertical Effective Stress, ksf
Y-axis: Percent Change in Height

MAXIMUM DRY DENSITY / OPTIMUM MOISTURE   ASTM D 1557 (Modified)

Job Name: Santa Cruz Project
Sample ID: 0
Location: B-4 @ 0-5'

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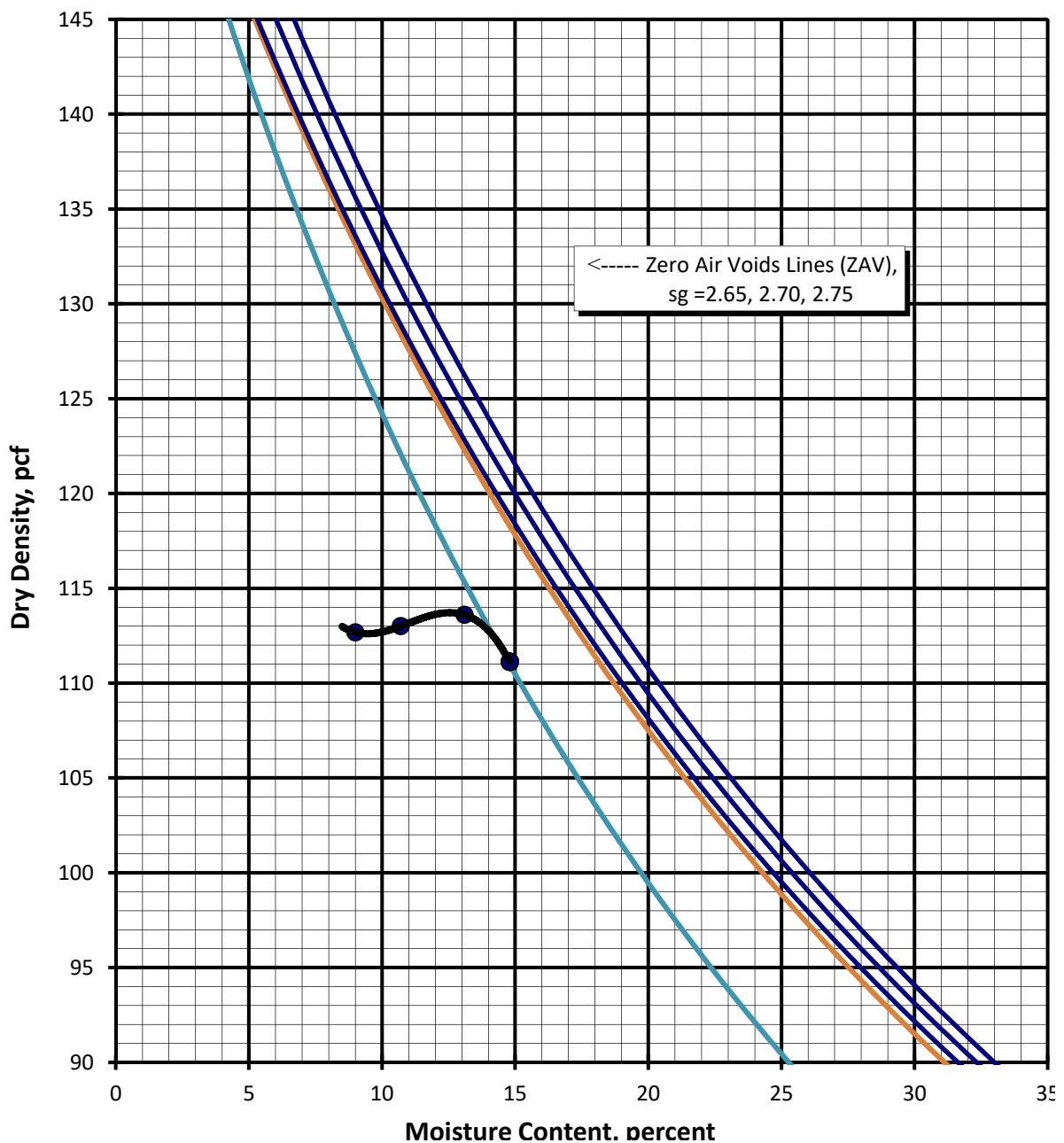Rammer Type: Mechanical

Description: Poorly Graded Sand with Silt (SP-SM)

Sieve Size	% Retained (Cumulative)
3/4"	8.3
3/8"	9.6
#4	10.9

Maximum Dry Density: **113.7 pcf**
Optimum Moisture: **12.6%**
Corrected for Oversize (ASTM D4718)

<----- Zero Air Voids Lines (ZAV), sg =2.65, 2.70, 2.75

Dry Density, pcf

Moisture Content, percent

SOIL CHEMICAL ANALYSES

Job Name: Santa Cruz Project
Job No.: 306336-002
Sample ID:

Sample Location: B-4 @ 0-5'

Resistivity (Units)
as-received (ohm-cm)   96,000
saturated (ohm-cm)   10,000
pH   7.9
Electrical Conductivity (mS/cm)   0.07

Chemical Analyses
Cations
calcium Ca^{2+} (mg/kg)   55
magnesium Mg^{2+} (mg/kg)   2
sodium Na^{1+} (mg/kg)   18
potassium K^{1+} (mg/kg)   14
ammonium NH41+ (mg/kg)   ND

Anions
carbonate CO_3^{2-} (mg/kg)   ND
bicarbonate HCO_3^{1-} (mg/kg)   101
fluoride F^{1-} (mg/kg)   2
chloride Cl^{1-} (mg/kg)   7
sulfate SO_4^{2-} (mg/kg)   21
nitrate NO31- (mg/kg)   4
phosphate PO_4^{3-} (mg/kg)   4

Other Tests
sulfide S^{2-} (qual)   na
Redox (mV)   na

Note: Tests performed by Subcontract Laboratory: mg/kg = milligrams per kilogram (parts per million) of dry soil.

HDR Engineering, Inc.   Redox = oxidation-reduction potential in millivolts

431 West Baseline Road   ND = not detected

Claremont, California 91711 Tel: (909) 962-5485   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.

EXUDATION PRESSURE CHART
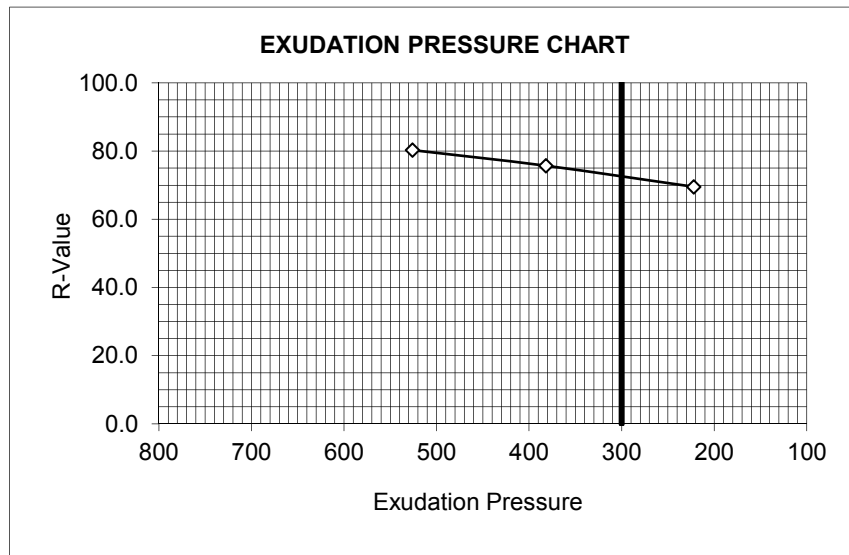
JOB NAME: Santa Cruz Project
SAMPLE I. D.:   B4 @ 0-5 feet
SOIL DESCRIPTION: Poorly Graded Sand with Silt (SP-SM)

SPECIMEN NUMBER	D	E	F
EXUDATION PRESSURE	526	382	222
RESISTANCE VALUE	80.3	75.7	69.5
EXPANSION DIAL(0.0001")	0	0	0
EXPANSION PRESSURE (PSF)	0.0	0.0	0.0
% MOISTURE AT TEST	10.2	10.7	11.0
DRY DENSITY AT TEST	118.1	118.1	118.4

R-VALUE @ 300 PSI EXUDATION | 73 |

Based on Traffic Index = 8.00 & Gravel Factor = 1.34

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