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Geotechnical Engineering Report

ECS Southwest, LLP

Geotechnical Engineering Report

Proposed Maverik Gas Station and Convenience Store

Golf Center Drive and Avenue 45
Indio, California

ECS Project Number 80:1035

July 7, 2023

ECS SOUTHWEST, LLP
"One Firm. One Mission."
Geotechnical • Construction Materials • Environmental • Facilities

July 7, 2023

Mr. Zach Michels
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7217 Watson Road
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ECS Project No. 80:1035

Reference: Geotechnical Engineering Report
Proposed Maverik Gas Station and Convenience Store
Golf Center Drive and Avenue 45
Indio, California

Dear Mr. Michels:

ECS Southwest, LLP (ECS) and our subconsultant engineer Earth Systems have completed the subsurface exploration and geotechnical engineering analyses for the above-referenced project. Our services were performed in general accordance with our agreed to scope of work. The enclosed report presents our understanding of the geotechnical aspects of the project, the results of the field exploration conducted, and our geotechnical design and construction recommendations for the project. The seepage testing results will be submitted separately as soon as it is completed.

It has been our pleasure to be of service to you during this phase of this project. We would appreciate the opportunity to remain involved during the continuation of the design phase, and we would like to provide our services during construction phase operations as well to verify subsurface conditions assumed for this report. Should you have any questions concerning the information contained in this report, or if we can be of further assistance to you, please contact us.

Respectfully submitted,

ECS Southwest, LLP

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Geotechnical Engineering Report and Infiltration Testing
Proposed Indio California Maverik Store No.: TBD
Golf Center Parkway and Avenue 45 (APN 611-330-025-9)
Indio, Riverside County, California

July 3, 2023

File No.: 306043-001
Doc. No.: 23-06-702

EARTH SYSTEMS

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July 3, 2023

File No.: 306043-001
Doc No.: 23-06-702

ECS Southwest, LLP
3033 Kellway Drive
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Attention: Stephen Geraci, P.E., CHMM

Project: **Proposed Indio California Maverik Store No.: TBD**
Northeast Corner Golf Center Parkway and Avenue 45
Indio, Riverside County, California

Subject: **Geotechnical Engineering Report and Infiltration Testing**

Earth Systems Pacific [Earth Systems] is pleased to submit this geotechnical engineering report and infiltration testing for the proposed development located at the northeast corner of Golf Center Parkway (aka Highway 111) and Avenue 45 in Indio, Riverside County, California. The intent of this report is to provide geotechnical and storm water infiltration information for the development of a new Maverik gas station and store. Earth Systems also provided services for percolation testing for private wastewater disposal improvements using 20-foot-deep seepage pits. The percolation report is provided via separate cover (see Earth Systems, 2023, Percolation Testing).

This report completes our geotechnical and infiltration testing scope of services in accordance with our agreement (BER 23-4-001) with an authorization date of April 28, 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. This report is for the use of ECS Southwest and Maverik for the subject site. 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,
EARTH SYSTEMS PACIFIC

No. 60302
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Proposed Indio California Maverik Store No.: TBD
Northeast Corner of Golf Center Parkway and Avenue 45
Indio, Riverside County, California

Section 1
INTRODUCTION

1.1 Project Information

This geotechnical engineering report with infiltration testing has been prepared for the proposed improvements for a Maverik Gas Station and Store #TBD located at the northeast corner of Golf Center Parkway (aka Highway 111) and Avenue 45 in Indio, Riverside County, California. We understand that site development will include a 5,600 square foot store, fueling canopy, underground fuel storage tanks, biodiesel mixing vault, air station, trash enclosure, generator, RV dump area, cat scale, two drive entrances, parking area, and truck route.

Architectural, civil, and structural plans were not available during the preparation of this report. We anticipate that the structures will be one to two stories and will be of masonry, metal, wood, or light frame construction supported on conventional shallow foundations. We assume loadings will not be more than 50 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 structural loading 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.

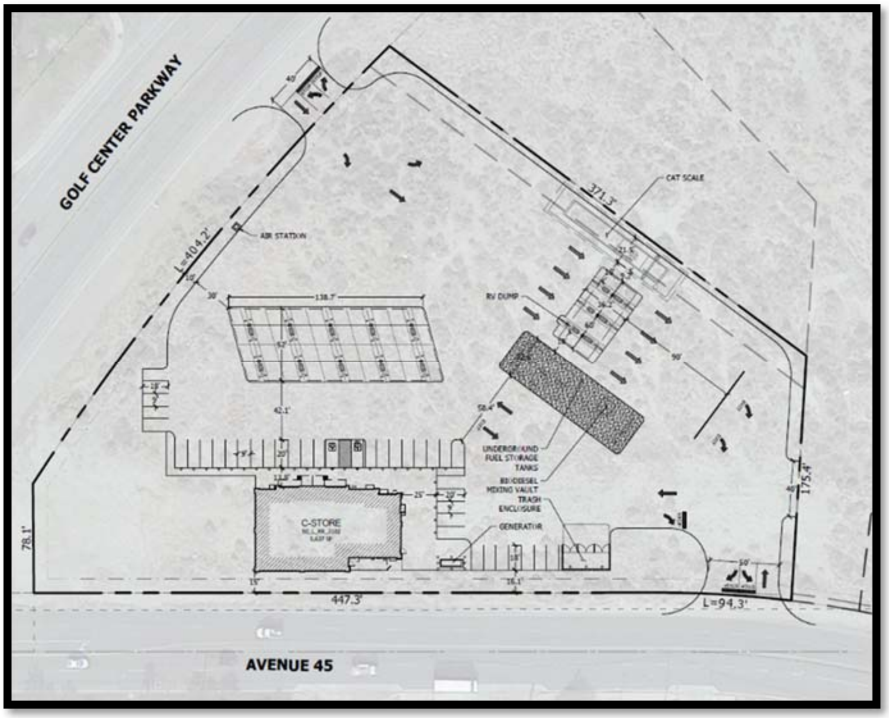

Figure 1 Preliminary Site Plan Showing Proposed Improvements.

1.2 Site Description

The project's legal address is Accessor Parcel Number APN 611-330-025-9 in Indio, Riverside County, California. Per a client provided site plan, the parcel area has a gross area of 3.34 acres and a building area of 5,637 square feet. Access to the lot is via Avenue 45 that does not have curbing. One latitude and longitude of the proposed building area nearest to adjacent faulting (1.5 miles northeast) is approximately 33.7229°N, 116.2043°W.

Topographically, the site is relatively flat. Per the Riverside County APN report, the site elevation is approximately -28 feet above mean sea level. Drainage is assumed by sheet flow toward Avenue 45.

The lot is currently vacant with desert vegetation. We researched past use of the site via select documents and we did not find past use history. During our site visits, loose sandy surface soil was evident by rubber tire vehicles getting stuck in the loose sand. Firm silts and medium dense silty sands are also present. Also undocumented fill was found at the site. The fill appears to be silty sand with gravel, similar to Class II aggregate base. An unusual concrete and block structure was found at the eastern portion of the site, see Figure 3. That structure appears to be a historic drywell or seepage pit; however, it was sealed, and we could not tell if it was deep or just the top portion of a seepage pit that was dumped at the site. Based on the underground utility request, mark out was placed along the ground from a pole located near Avenue 45 toward the Whitewater Channel, see Figure 3.

Currently the site is bounded by a Avenue 45 to the south, Golf Center Parkway to the west, vacant land and Whitewater Channel to the north, and vacant land and industrial buildings to the east. Along the project's easterly perimeter, a small cardboard type house was found. No existing power poles were observed in the public right of ways; however, at the top of the Whitewater Channel westerly slope are power poles trending in an east and west direction.

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:

<u>Task 1 – Literature and Photograph Reviews</u>: 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.

<u>Task 2 – Utility Clearance, USA Dig Alert</u>: Earth Systems pre-marked each proposed exploration locations for underground utility search companies' verification. On April 28, 2023, Underground Service Alert was contacted and informed of our intent to explore the site. Earth Systems obtained a positive response from Dig Alert on May 14, 2023. The client specific boring locations

are shown in the Exploration Map found in the Appendix A. Boring exploration points were determined based on a client provided document entitled "Conceptual Site Plan", see Plate 2.

Task 3 – Field Exploration: The on-site soil profile was explored by means of drilling and sampling within eight (8) exploratory borings. The borings extended from approximately 15 to 70 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.

Task 4 – Laboratory Testing: 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, Atterberg Limits, consolidation/collapse potential, Expansion Index, direct shear strength, 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.

Task 5 – Analysis and Report: 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).
- Recommendations for location specific storm water infiltration rates allowed by Riverside County Whitewater River Region.
- Preliminary recommendations for underground storage tanks.
- 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.

Task 6 – Infiltration Testing for Surface BMPs: Two shallow pits, about 6 to 12 inches below existing grade, were hand dug and used for infiltration testing. These pits extended vertically below the existing ground surface to represent the bottom of the stormwater BMP infiltration system.

Infiltration testing was performed in general accordance with Riverside County Flood Control and Water Conservation District (RCFCWCD), 2011, Design Handbook for Low Impact Development Best Management Practices. Infiltration testing consisted of field testing in general accordance with American Standard Test Methods (ASTM D3385). The double-ring infiltrometer method consist of driving two open cylinders, one inside the other, into the ground, partially filling the rings with water or other liquid, and then maintaining the liquid at a constant level. The volume of liquid added to the inner ring, to maintain the liquid level constant is the measure of the volume of liquid that infiltrates the soil. The volume infiltrated during timed intervals is converted to an incremental infiltration, usually expressed in inches per hour.

Task 7 – Percolation Testing for Onsite Waste Water Treatment: This task was performed and documented under separate cover.

Section 2
METHODS OF EXPLORATION AND TESTING

2.1 Field Exploration

The subsurface exploration program for the subject site included advancing six (6) geotechnical exploratory borings and two (2) seeepage pit testing borings on May 22 and June 6, 2023. The borings were drilled to depths ranging from about 15 to 71½ feet below the existing ground surface to observe soil profiles, obtain samples for laboratory testing, and to perform seepage pit percolation testing. The geotechnical borings were drilled using 8-inch outside diameter hollow-stem augers, powered by a Mobile B-61, truck-mounted drill rig, and a Limited Access Rig (LAR) 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.

The second day of drilling was required because the B-61 rubber tired rig could not traverse two locations due to existing very loose sand on the way to borings B-3 and B-5. On the second day the LAR, which is equipped with rubber tire tracks, performed the explorations at B-3. Since boring B-5 was selected as a pavement exploration to the entrance on Golf Center Parkway, we moved that boring closer to the public right of way.

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 72% hammer efficiency for the B-61 drill rig and an approximate 68% hammer efficiency for the LAR 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.

In addition to the six geotechnical and two septic pit borings, two (2) double ring infiltrometer pits were excavated by hand using shovels and driving the rings into the ground. The excavation was shallow and made invert depths between 6 and 12 inches.

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.

- Expansion Index tests to evaluate the expansive nature of the soil. The samples were surcharged under 144 pounds per square foot at moisture content of near 50% saturation. Samples were then submerged in water for 24 hours and the amount of expansion was recorded with a dial indicator (ASTM D 4829).
- R-Value to evaluate the empirical shear strength of the site soils in relation to pavement loading characteristics (California Test 301). R-Value data was converted to CBR using American Concrete Institute (ACI) tabled data for use in parking lot Portland cement concrete pavement design (ACI, 1992).

Section 3
DISCUSSION

3.1 Soil Conditions

The field exploration indicates that site soils consist predominantly of lean silts with varying sand, silty sand, poorly graded sand with silt, poorly graded sand, and lean clays (Unified Soils Classification System symbols of ML, SM, SP-SM, SP, and CL) to the maximum depth of exploration of 71½ feet below the ground surface. In general, most of the site is covered with naturally deposited soils that consist of interbedded aeolian (wind deposited) and alluvial (water deposited), and lacustrine (lake) sediments (geologic types Qs, Qal, and Ql). Some undocumented fill (AF) is present in areas of past disturbance (see Section 3.3), but is generally undifferentiated from the native soil.

The observed native sandy soils in the upper 10 feet were generally very loose to medium dense. Fills are not compact. The sandy soils were medium dense to very dense at lower depths to approximately 70 feet.

Fine grained soil layers (mostly lean silts) were found in borings and at depths explored. They ranged in consistency from soft to very stiff. Although the clays were interbedded with sands and silts at shallow depths, thick lean clay layers were identified at depths starting at 35 feet bgs. However, clay layers were not observed consistently from boring to boring. Saturation levels above 85 percent were not consistent until groundwater was reached at a depth of approximately 60 feet bgs (see boring B-2).

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.

Mottling: Mottling was observed as shallow as 7½ feet and 15 feet below the ground surface, see borings B-6, B-2 and B-4. The presence of mottling is a characteristic specified by some agencies to estimate past groundwater levels.

Field Exploration Information: Free groundwater was encountered in boring B-2 during our exploration conducted on May 22, 2023; The groundwater was observed at a depth of 59½ feet below the ground surface. Very moist soil appeared at a depth of 25 feet below the ground surface (bgs) at boring B-1; however, very moist soils were not continued until groundwater was observed at 59½ feet. We performed 26 moisture content tests of the soil samples recovered and obtained values varying between 0.5% to 33% at depths ranging between 2½ feet and 71½ feet below the ground surface (bgs). The average moisture content was 5.5 percent. The top 10

foot soil profile had a moisture content varying from 0 to 10 percent moisture content.
Nearby Well Information: We researched the California Department of Water Resources (DWR) groundwater database and found one well (Local Well 337345N1162245W001) located approximately 1.4 miles northwest of the project site. That well had readings taken on September 27, 2021, and indicates the groundwater elevation is approximately 79.92 feet below mean sea level (MSL). From Section 1.2, the project low elevation is 28 feet below MSL. Based on State Well readings, the estimated groundwater depth at the site is approximately 51.9 feet below the ground surface. This is very close to the actual reading we found at boring B-2 showing a groundwater depth of 59½ feet below the ground surface.

Historic Groundwater Information: From observation of a 1961 Ground Water Basin Subdivisions and Contours of Ground Water Levels Map (see Figure 2 below), published by the Resources Agency of California Department of Water Resources Southern District (Department of Water Resources Bulletin 108), the historic groundwater contour nearest the project is between contour -30 and -40 feet. Using an elevation of -38 feet near the site, the historic groundwater is at a depth of approximately 10 feet below the ground surface.

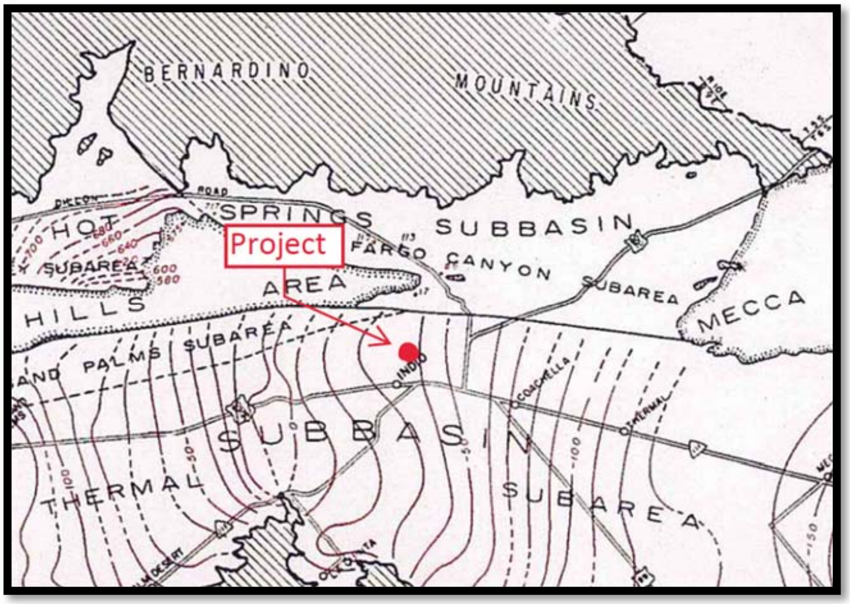

Figure 2 Historic Groundwater Map

Estimated Project Groundwater Depth: Based on the data provided above, Earth Systems estimates the highest groundwater depth from the "Historic Groundwater Table" as the worst case. A depth of 7½ feet below the ground surface will be used to set the groundwater depth for the liquefaction analysis. Groundwater 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 throughout this geotechnical commission. Earth Systems personnel also reviewed select historic aerial photographs of the project site. An aerial image was taken on April 28, 2023, and is used to show select observations, see Figure 3 below. A summary of our findings is presented below:

Field Observations:

1. Artificial fill areas containing gravel and asphalt grindings (see approximate boundaries observed in yellow).
2. Possible dry well location observed. Man-made block with mortar and what appears to be concrete filled. However, other blocks were found north of this location.
3. Cardboard and plywood-built objects (possible squatter location).
4. A possible underground conduit flagged during Dig Alert procedures.

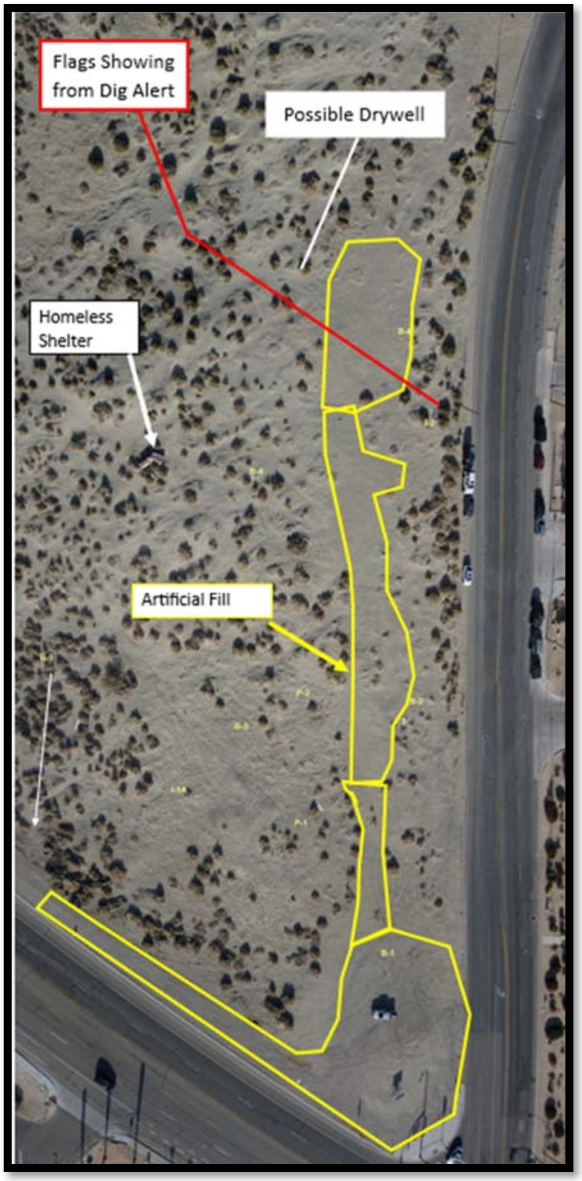

Figure 3 Aerial View Taken April 28, 2023.

Historical aerial photographs:

A. 1956 Aerial Photo: The site appears untouched with minor desert vegetation. A natural channel or row of vegetation appears to cross the westerly portion of the site in a northwest to southeast direction. This lineament is likely a reflection of a receding shoreline of ancient Lake Cahuilla. Golf Center Parkway does not exist yet. However, Avenue 45 is being used as a roadway.

B. 1972 Aerial Photo: The site still appears untouched. Golf Center Parkway appears to be under construction or is a dirt road. Avenue 45 appears to be a paved roadway. The drainage channel discussed in 1956 observations is no longer visible in this aerial.

C. 1977 Aerial Photo: Golf Center Parkway appears to be a paved roadway. Some shoulder work may have encroached into the property adjacent to Golf Center Parkway.

D. 1984 Aerial Photo: the undocumented fill area along Avenue 45 and explained in Section 3.1, appears in this aerial. At the projects' proposed entrance site along Golf Center Parkway is round white anomaly measuring with a diameter of approximately the width of Golf Center Parkway pavement.

E. 2005 Google Aerial: shows vegetation except where artificial fill area is located.

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 (hydroconsolidation) 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 total sample using the Collapse Potential Test (ASTM Standard Test Method D 5333). Based on the Naval Facilities Engineering Command (NAVFAC) Design Manual 7.1, the severity of collapse potential is commonly evaluated by the following Table 1, Collapse Potential Values.

Table 1
Collapse Potential Values

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

The results of four (4) collapse potential tests performed on selected samples from different depths throughout the project site indicated a range of collapse potential on the order of 0.6 to 0.9 percent at applied vertical stresses of 2,000 psf. These results suggest the potential for collapse is “no problem”. Collapse related settlement is estimated to be minor.

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. The Expansion Index of the onsite upper soils is “very low” for silts and sands, and “low” for clay (CL) soils 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

Two samples of the near-surface soils were tested for potential to corrosion of concrete and ferrous metals. The tests were 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 2

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 3.

Table 3

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 values of 7.0 and 7.2 chloride contents of 72 and 129 ppm, sulfate contents of 136 and 146 ppm and minimum resistivity of 880 and 1,440 Ohm-cm. Although Earth Systems does not practice corrosion engineering, the corrosion values from the soil tested are normally considered as being “Severely” and “Very Severely” 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 near the northeast margin of the central Coachella Valley, a part of the Colorado Desert geomorphic province. To the northeast are the Little San Bernardino Mountains which are part of the Peninsular Ranges geomorphic province. A significant feature within the Colorado Desert geomorphic province is the Salton Trough, 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) wind-blown sand, alluvium, and lake deposits of the ancient Lake Cahuilla.

Active faults in the immediate vicinity (within 30 miles) of the site include the San Andreas, San Jacinto, Blue Cut, Pinto Mountain, Burnt Mountain, and Eureka Peak faults (See Table A-1). The closest active faults are multiple traces of the San Andreas fault zone 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.

Local Geology: The project site is located within the central portion of the Coachella Valley within the limits of mapped lakebed deposits associated with the ancient Lake Cahuilla (ancestral Salton Sea). The Little San Bernardino Mountains are located to the northeast. The site is southwest and adjacent to the Whitewater River channel. Sediments within this area consist of fine- to medium-grained sands with interbedded clays, silts of alluvial, aeolian (wind-blown) and lacustrine (lake) origins.

The project site is located in a mapped area where surficially, a mix of alluvial fan deposits, dunes sands, and lake deposits are prevalent. Thin deposits of artificial fill resulting from site modification are also present across the site and are generally undifferentiated from the underlying native deposits.

No active faults are currently mapped in the immediate project vicinity. The closest mapped Holocene-active faults are segments of the San Andreas fault located approximately 1.6 miles northeast of the project site.

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 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 fault, San Jacinto fault, and faults associated with the Eastern California shear zone.

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. The closest active faults are traces/segments of the San Andreas fault zone, located approximately 1.5 miles northeast of the site. Therefore, active fault rupture is unlikely to occur at the project site. While fault rupture would most likely occur along previously established fault traces, future fault rupture could occur at other locations.

Review of select aerial photographs reveal that the site is located in an area predominated by alluvial fan, lineal dune (wind) patterns, and regressive shoreline features associated with ancient Lake Cahuilla. No significant evidence of fault related lineaments was 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 38 miles of the project since 1856 (See Table A-2).

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 Indio 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 Indio 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 Indio 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 37% 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. For a magnitude 8 earthquake locally on the San Andreas fault, the probability is approximately 8% for the same time period. 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 1.5 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 standard of practice suggests a maximum magnitude of 7.9 be used for analysis, assuming a multi-segment rupture event and paleo seismic data.

The local segment has 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 include flooding and slope instability. The site is far inland, so the hazard from tsunamis is non-existent. At the present time, no water storage tanks and ponds are located in the immediate vicinity of the site. Therefore, localized flooding hazards from seiches is considered a low potential. The site is adjacent to the Whitewater Channel and associated levee. Flooding is a potential should levee breach occur. Base flood elevation is approximately -34 to -35.5 feet (see Section 3.8.2).

Site Acceleration and Seismic Coefficients: In developing site-specific seismic design criteria, the characteristics of the earth units underlying the site are an important input to evaluate the site response at a given site. Based on the results of our 2023 field exploration at the site, the project site is underlain by stiff and medium dense alluvium and lacustrine deposits with an estimated average shear wave velocity of about 263 m/s (average blowcount of 26 for the upper 100 feet of soil) based on our deep boring at the site; however, liquefaction is a potential. Based on the above information, we classify the site soil profile for site response as Site Class F due to potential for liquefaction, according to ASCE 7-16, and D based on blowcount. The D characterization is defined as a soil profile consisting of stiff soil with shear wave velocities between 180 and 360 m/s, and field blowcount of 15 to 50 blows/ft.

Earthquake horizontal peak ground motions of approximately 0.98 g is estimated based upon a 2% probability of exceedance in 50 years based on the US Seismic Design Maps website. Acceleration values provided are estimates only. Actual spectral acceleration values may be more or less than those provided and could exceed 1 g assuming a maximum considered earthquake event occurs on the nearby San Andreas fault. Vertical accelerations are typically 1/3 to 2/3 of the horizontal accelerations, but can equal or exceed the horizontal accelerations depending upon the local site effects and amplification.

2022 CBC Seismic Coefficients: The seismic and site coefficients given in Chapter 16 of the 2022 California Building Code are provided in Section 5.6 of this report.

3.8.2 Other Hazards

Slope Stability: The site is relatively flat and a precise grading plan was not ready for review at this time. Therefore, potential hazards from slope instability, landslides, or debris flows are considered low assuming the planned development is sufficiently setback from the Whitewater River channel top of slopes, at least 40 feet.

3.8.3 Secondary Hazards

Secondary seismic hazards related to ground shaking include soil liquefaction, ground subsidence, tsunamis, and seiches.

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.

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 7½ feet below the ground surface. Also, the site is within a “HIGH” liquefaction hazard zone as defined by Riverside County (Geographic Information Services, 2023) and parcel report. As such, the potential for liquefaction was analyzed.

The design peak ground acceleration value was obtained from the USGS online application (seismicmaps.org) on June 9, 2023, see Section 5.6. We obtained a PGA_M = 1.08g. Liquefaction output considering groundwater at 7½ feet below the ground surface (bgs) are presented in Appendix A. Results indicate a liquefaction potential for a 50-foot soil profile (boring B-2) is 1.9 inches. However, the remaining boring profiles of 35 feet (normalized to utilize the lower 15 feet of B-2), had settlement varying from 0 inches to 1 inch.

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 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 historic groundwater table (7½ feet bgs) for the full soil column height (7½ feet) was estimated for borings B-1 to B-5 as 0 inches. If considering a 50 foot dry soil column, results indicate dry seismic settlement on the order of 0.6 to 1 inch.

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 and as such per Special Publication 117 (2008).

Differential Seismic Settlement: Per SP117A, differential seismic settlement is estimated to be the difference between boring settlement results. For the dry seismic case, on the order of ½ inch considering current conditions, and 1.9 inches considering the historic groundwater condition for liquefaction.

3.8.4 Other Geologic Hazards

Slope Instability: The site is relative flat. Therefore, potential hazards from slope instability, landslides, or debris flows are considered very low. Soils are cohesionless and surficially unstable in sloping configurations.

The Whitewater Channel is found northeast of the project. The project’s north boundary line is over 300 feet from the top of Whitewater Channel slope closest to the project. The Whitewater Channel has a depth of approximately 25 feet below the project elevation. The channel slopes are also protected with a concrete facing. Work is currently ongoing along the channel slope. Because of the project distance from the Whitewater Channel slope and the lack of continuous clay layers, there is a low potential for slope stability issues. Additionally, a screening level lateral spreading analysis indicates a low potential for lateral spread (SP117A) as N160 blowcount values in the liquefaction zone are greater than 15 for susceptible soils.

Ground Subsidence: The site is not within an area of known damaging subsidence as the subsidence is generally occurring on an areal basis (USGS zone of subsidence monitoring in the Coachella Valley, Sneed, 2014 and 2020). In areas of fairly uniform thickness of alluvium, fissures are thought to be the result of tensional stress near the ground surface and generally occur near the margins of the areas of maximum subsidence. Surface runoff and erosion of the incipient fissures augment the appearance and size of the fissures. Streets, curbs, sidewalks, and paved parking lots were observed for evidence of linear cracks with or without vertical offsets. Typical pavement cracking, relating to asphalt concrete shrinkage and heat expansion and contraction was noted, but no significant evidence of linear cracking along the peripheries of the project or within adjacent streets was observed that would be suggestive of tensional stresses or fissuring related to differential areal subsidence.

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 lies in areas outside the 0.2% annual chance floodplain Zone X as identified on FEMA Panel 06065C2252H eff 3/6/2018, and has an adjacent levee. The project site may be in an area where sheet flooding could occur, however local drainage is controlled by public streets and storm drain improvements with curb and gutter, storm drains, and underdrains. Appropriate project design by the civil engineer, construction, and maintenance can minimize the site sheet 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.

3.10 Near Surface Infiltration Testing

As indicated in Section 2.1 of this report, two shallow exploratory test pits were excavated at the project site. Test pit locations are shown on the Exploration Location Map, in Appendix A, see I-1A and I-2.

To evaluate the soils encountered, two infiltration tests were performed. Locations were selected at random as storm water infiltration BMP locations were not determined by the BMP designer prior to our testing. Testing was performed at a depth of 6 to 12-inches below existing grade. The infiltration testing was performed with double-ring infiltrometers, following the general guidelines contained in ASTM D3385, *Standard Test Method for Infiltration Rate of Soils in Field Using Double-Ring Infiltrometer*. Double ring test procedures simulate the low water head conditions typically present during shallow infiltration.

For each test location, the test elevation was reached by excavating the soil using hand shovels and the inner and outer rings of the test apparatus was driven into the ground an additional four to six inches, approximately. As necessary, powdered bentonite was placed around the edges of the rings in order to create a watertight seal. Care was taken to not alter the structure of the soil during hand excavation. Per ASTM test procedure, potable water was used to evaluate the basic infiltration rate. The tests were performed for a period of at least 6 hours or a stabilized infiltration rate. The soils encountered at each test location and the results of the infiltration testing are presented in Table 4 below.

Table 4
Infiltration Test Results

Test Location	Test Description	USCS Soil Description	Estimated Unfactored Approximate Infiltration Rate*	Estimated Unfactored Approximate Percolation Rate**
I-1A	Double Ring Infiltrometer	Silty Sand	1.2 inches/hour	17 gallons/sf/day
I-2	Double Ring Infiltrometer	Silty Sand	2.0 inches/hour	30 gallons/sf/day

*Field Values, no factor of safety applied. Typical factors of safety range from 3 to 10 depending on the type of system which will be designed using the field values and depending on the level of pre-treatment and influent to be discharged into the basins per Riverside County BMP Design Manual.

** In small trenches, there may be sidewall infiltration that can play a role as well. In those cases, in/hr is not a good indicator of infiltration, and a unit of gallons/square feet/day typically can be used to specify percolation.

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.
- Settlement limited to 1 inch total and ¾ inch differential over 40 feet are considered feasible at the site without extensive ground modification. However, deep depth of seismically induced settlement exceeds these limits. The client's structural engineer may consider the use of stiffened foundations and slabs to address the settlement potential.
- 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 1.5 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 (above the groundwater table) and liquefaction settlement (below the historic groundwater table).
- Other geologic hazards, including fault rupture, lateral spreading, tsunamis, seiches, slope instability, flooding, and ground subsidence are considered to have a low or negligible potential to occur onsite.
- The site soils are disturbed from artificial/undocumented fill placement, and possible underground improvement.
- The site soils in the upper 6 feet are generally loose (Section 3.1). As such, moisture conditioning, overexcavation, and recompaction is recommended to provide a firm bearing surface.
- Surface site soils have a very high potential for wind-driven migration. Additionally, filter fabric should separate any gap-graded (such as pea gravel) backfill from native soils or fill composed of native soils.
- The upper soils were dry to damp with moistures 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.
- Free groundwater was encountered in the borings during exploration. Groundwater levels may fluctuate with precipitation, irrigation, drainage, regional pumping from wells, and site grading.
- The site soils are generally “very low” to “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 “very severely” 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.

Artificial/undocumented fill was encountered during the exploration in and outside of the building footprints from approximate depths of 1 to 2 feet (see Figure 3). Additional removals to chase and remove deeper undocumented fills may be required during grading. The locations shown in Figure 3 do not limit undocumented fill locations. During grading (including Clear and Grubbing), the soils engineer or representative shall be employed to observe over-excavations and approve bottoms and lateral extents. The extents shall be defined during construction.

Buried utilities 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 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 up to only 2 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.

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 should be over-excavated a minimum of 6 feet below existing or finished grade, or 3 feet below the bottom of foundations, whichever is lower. The over-excavation should extend for at least 6 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). 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.

Tank Pad Grading Recommendations: Tank pad grading should be similar to the building pad preparation, except the over-excavation should extend a minimum of 2 feet below the bottom of tank foundation and should extend 3 feet laterally from the most outer face of the tank, including second skins if provided per the tank designer. The backfill shall be non-expansive soils (SM, SP-SM, and ML).

The native backfill can easily migrate into gap graded backfills such as pea gravel. All gap graded backfill adjacent to native soils should be wrapped in filter fabric such as Mirafi 140N as a minimum. Also, if the contractor wishes to test for native migration into contractor selected backfill, please contact Earth Systems. Backfill immediately behind the tank should be a free-draining granular material.

Tank Buoyancy Considerations: We understand underground storage tanks (USTs) will be installed on site for probable fuel storage and storm drain storage. Plans were not provided to Earth Systems at the time of this report. The USTs and possible storm water storage will require an excavation depth below final grade. Overexcavation below the tank foundation should follow Tank Pad Grading above. Earth Systems should review UST plans prior to approval of this overexcavation recommendation.

Although existing groundwater level is estimated to be around 59½ feet below the ground surface, silt layers and clay lenses, which are capable of preventing water flow, were observed in the underground tank area and historic groundwater levels are only 7½ feet. Groundwater recharge is also occurring and planned for the Coachella Valley area. Potential water source leakage or infiltration could cause a localized perched water table condition. Tank design should include buoyancy evaluation. For seismic events buoyancy calculation should use the wet density of the surrounding soils, which Earth Systems estimates to be approximately 100 pounds per cubic foot (pcf). Please contact Earth Systems for additional information on ground anchor design recommendations if applicable.

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 adjacent site improvements, shoring may be necessary to achieve the remedial grading depths for known improvements (C store and Cat Scale) and unknown improvements encountered near the property lines.

Underground Stormwater Infiltration Bottom Preparation: Compaction effort should be kept to a minimum at bottom areas used for infiltrators (except under foundations). The bottom of basins and infiltrator bottoms should be compacted to approximately 85% relative compaction. Side slopes and any other fill should be compacted to at least 90% relative compaction. Slope construction should be per Section 5.7 of this report.

Earth Systems recommends locating underground storm chambers at least 30 feet from building foundations—please note some Coachella Valley agencies require greater setback depending on plan layout. If this setback cannot be obtained, Earth Systems recommends the use of moisture barriers or other mitigation measures. Another mitigation method could be the use of geo grid under foundations that are proposed near underground storage facilities. Please contact Earth Systems for additional information.

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. Compaction should be verified by testing.

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

Engineered Fill Soils and Rock: 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, SP-SM, or ML. 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.

Where UST’s (tanks) are backfilled with gravel (pea, ¾”, etc.), the gravel should be enveloped in filter fabric as recommended earlier in Section 5.1 for Tank Grading. Gravel backfill should also be moistened and densified using mechanical equipment. Maximum lift thickness should be 8 inches for plate compactors and 2 feet for walk behind vibratory compactor, per the approval of the geotechnical engineer. Excavator wheel compactors are not recommended unless fit with a vibratory plate.

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. Gravel backfill densification may be evaluated by observation of effort, settlement, and using unit weight bucket relative density in conjunction with nuclear gauge backscatter (minimum 95% compaction).

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 should evaluate the import fill soils before hauling to the site.

Shrinkage and Bulking: The shrinkage factor for soils is expected to range from 11 to 27 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%.

Based upon 10 in-place densities evaluated, the average computed shrinkage is 18% with one standard deviation of 4.7%. 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 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 to the geotechnical engineer 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 1,500 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" to "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" to "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 soil grade should be maintained (lowest grade within 3 feet laterally as measured from the foundation bottom). 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. After excavation, foundation bottoms should be compacted to at least 90% relative compaction.

Slope Setback for Foundations: Earth Systems recommends a minimum setback distance of 5 feet (40 feet from Whitewater Channel slopes). 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.

Conventional Spread Foundations: 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 24 inches depth):

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

  Allowable increases of 300 psf for each additional 0.5 foot of footing depth may be used up to a maximum value of 2,100 psf. No increase of bearing pressure for increase of footing width.

- ➢ Pad foundations, 2 x 2 foot minimum and 5 x 5 foot maximum in plan and 18 inches minimum below grade (maximum 30 inches):

  2,000 psf for dead plus design live loads

  Allowable increases of 400 psf for each additional 0.5 foot of footing depth may be used up to a maximum value of 2,400 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 2 inches 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 150 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 four, No. 4 steel reinforcing bars, two placed near the top and two placed near the bottom of the footing. This reinforcing is not intended to supersede any structural requirements provided by the structural engineer.

Estimated Settlements for Foundations: Estimated total static load settlement and collapse settlement should be less than 1 inch, based on footings founded 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 and seismic settlement is estimated to be half of the estimated total for each case or 1½ inches. 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 1:320, which is not defined as a tolerable level for typical buildings (County of Riverside, 2000, page 39) without structural improvement. The structural engineer should refer to ASCE7-16, Section 12.13.9.1 and .2, as well as additional sections cited, considering 1.5 inches of vertical differential settlement over 40 feet and no lateral spreading, for foundation design considering seismic settlement.

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. Therefore, Earth Systems recommends the maximum distortion angle using all the settlement conditions including seismic settlements be 1:240. For all settlement conditions excluding seismic settlement, the structure's maximum distortion angle should be the Riverside County's required 1:480.

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, 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 150 pounds per cubic inch can be used in concrete lightly loaded (not mat) slab design for the expected compacted subgrade. Mat slab design will require differing modulus values.

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” to “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)

Retaining Walls:

- 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 wall 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 81 pcf for flexible walls, and 99 pcf for rigid walls. Seismic pressures are based on PGAM of 1.08g, 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.

Frictional and Lateral Coefficients:

- 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 1,000 psf and have the upper 3 feet of soil neglected. The soils pressures presented have considered onsite soils. 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 design methods presented in Sections 1806 and 1807.3 of the CBC for Silt (ML) 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 ASCE 7-16 applies. Additionally, the site is liquefiable and Site is Class F. Section 20.3.1 of ASCE 7-16 allows an exception for General Procedure seismic evaluation if the structures have a fundamental period less than 0.5 seconds, and the site is not subject to bearing failure. Based on that section, the site is classified as Site Class D for those structures less than 0.5 seconds period. The site is not subject to bearing failure for foundations as described within. 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 not 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.

Additionally, 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 33.7229°N and Longitude -116.2043°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	E
Maximum Considered Earthquake [MCE] Ground Motion	
Short Period Spectral Response S_s:	2.272 g
1 second Spectral Response, S_1:	0.964 g
Code Design Earthquake Ground Motion	
F_a	1.00
F_v	1.70
F_{PGA}	1.10
S_{MS}	2.272g
$S_{M1\ (unmodified)}$	1.639g
$S_{M1\ (ASCE\ 7\text{-}16\ Supplement\ 3\ Modified)}$	2.458g
Short Period Spectral Response, S_{DS}	1.515g
1 second Spectral Response, $S_{D1\ (unmodified)}$	1.093 g
1 second Spectral Response, $S_{D1\ (ASCE\ 7\text{-}16\ Supplement\ 3\ Modified)}$	1.639 g
Peak Ground Acceleration (PGA_M) Eq 11.8-1	1.08 g

* Only if Exceptions apply, see Section 5.6 text.

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, 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 exposed 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.

Slope Maintenance: 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 – (57 Tested, 40 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	4½
7	Main Drive Areas	4	7
9	Tractor Trailer Drives	5½	9½

*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). 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 40 has Equivalent CBR of 7.5)

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)	7.0	500	3,250
Alternate Truck/Bus Access or Loading/Unloading Areas (Traffic Category C, ADTT =100)	6” PCC Over 4” of CAB*	500	3,250
Tractor Trailer Drives	8.0	500	3,250

*-Crushed Aggregate Base Per the Greenbook

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:

- For PCC placed on aggregate base, please contact Earth Systems for additional analysis.
- 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 section should be compacted to a minimum of 95% relative compaction (ASTM D 1557).
- Subsequent to utility installation, the entire pavement (including PCC) final subgrade should be scarified 12 inches, moisture conditioned to near optimum moisture content, and compacted to a minimum 95% relative compaction immediately prior (within a few days) to the placement and compaction of aggregate base to re-establish proper moisture content and compaction in site soils. Subgrade soils 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 (8½ inches for a 7.0-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 10 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.

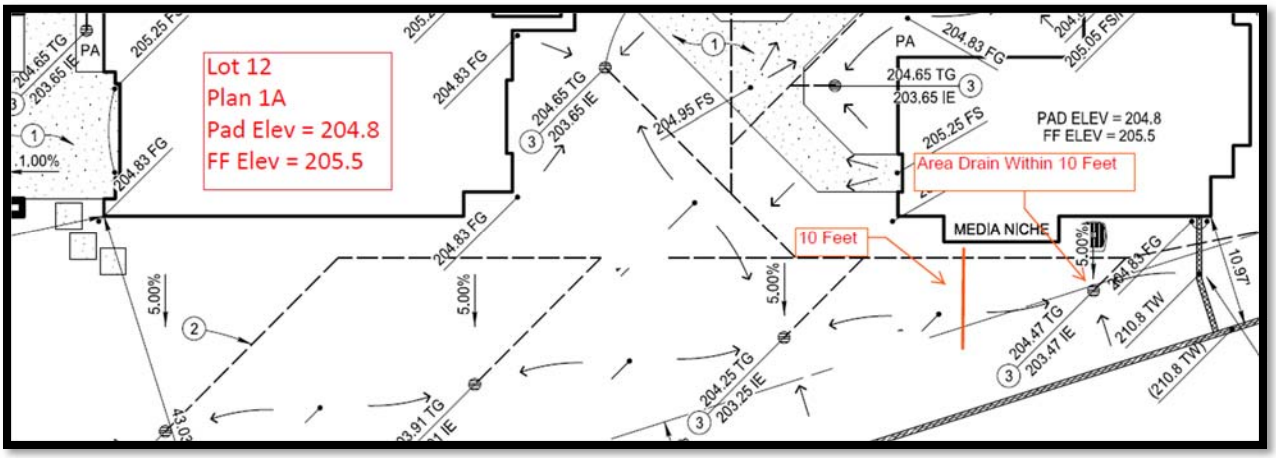

Figure 5 Example of Area Drains Used Within 10 Feet of Foundations When Slope < 5% in 10 Feet.

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. 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.

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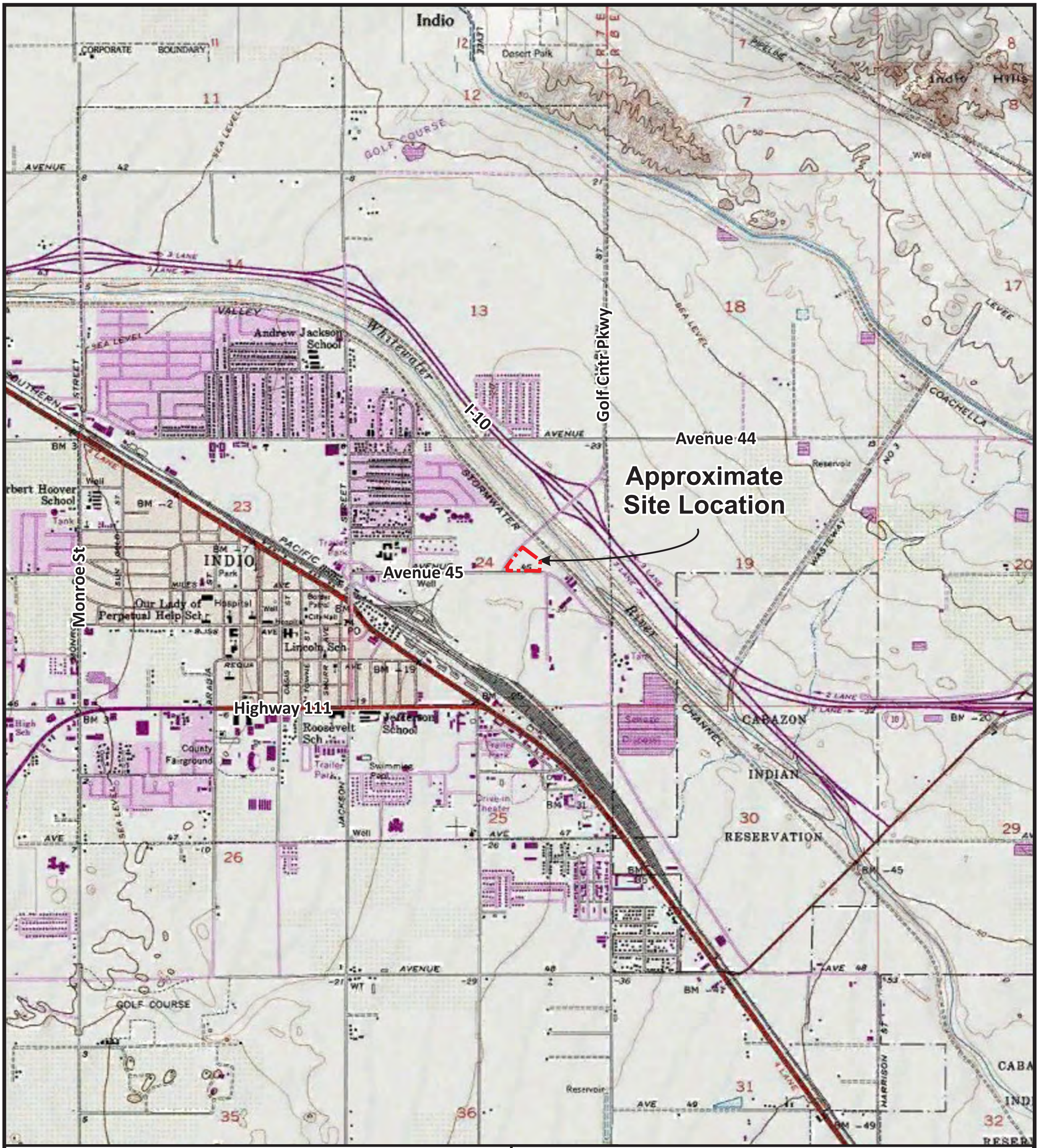
APPENDIX A



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

LEGEND

⬚ Approximate Site Location

Approximate Scale: 1" = 1/2 Mile

0 1/2 Mile 1 Mile

N

Plate 1
Site Location Map

Proposed Indio Maverik Store
Indio Center Drive & Avenue 45
Indio, Riverside County, California

Earth Systems

7/3/2023

File No.: 306043-001

LEGEND

◐ **B-6** Approximate Exploration Locations

◐ **P-2** Approximate Percolation Test Locations

◐ **I-2** Approximate Infiltration Test Locations

N

Approximate Scale: 1" = 200'

0 200' 400'

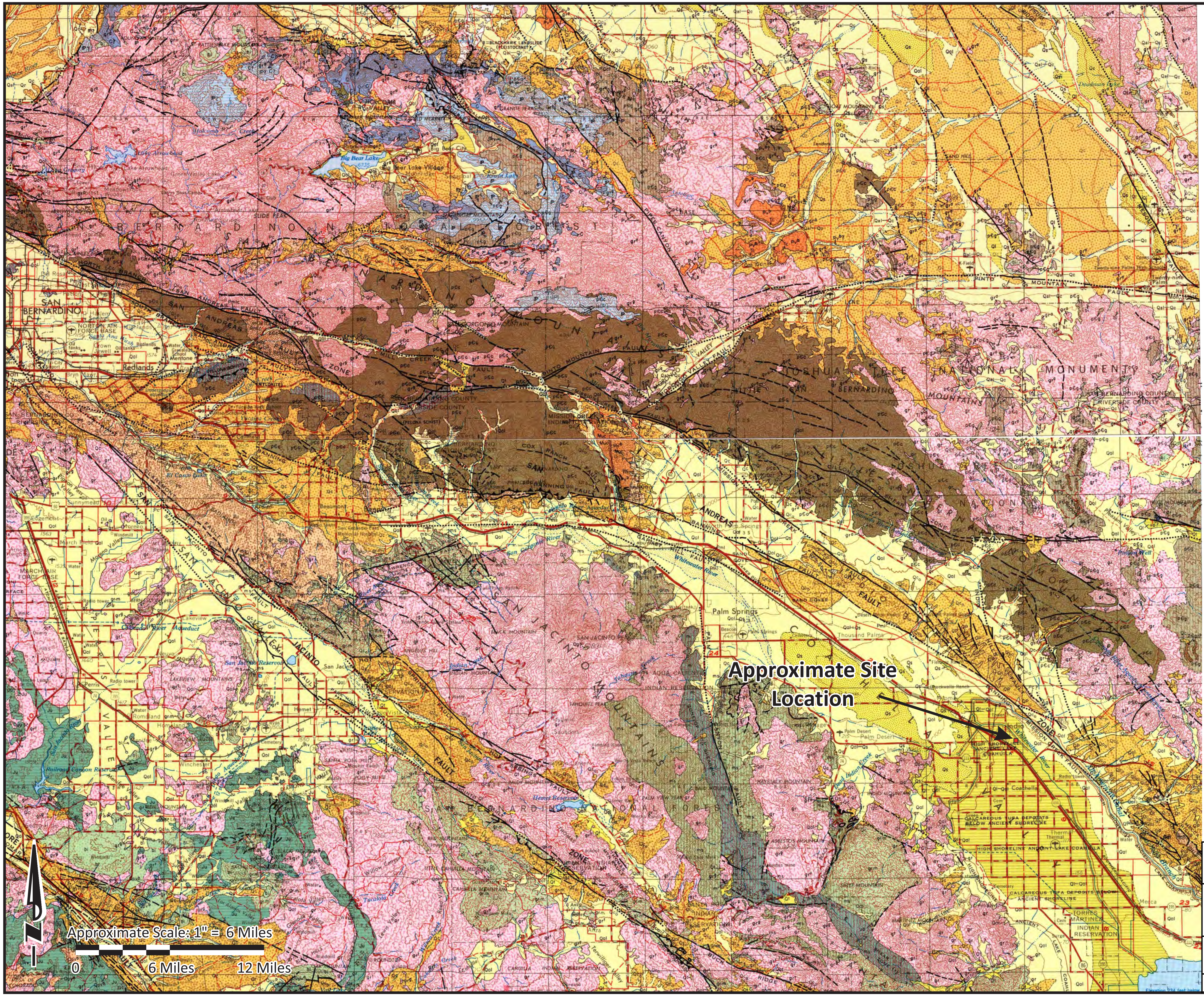
Source: Google Earth satellite image dated 6/11/2021, with Conceptual Site Plan 09 overlay dated 3/27/2023.

Plate 2
Exploration Location Map

Proposed Indio Maverik Store
Indio Center Drive & Avenue 45
Indio, Riverside County, California

Earth Systems

7/3/2023 File No.: 306043-001



LEGEND

Qs	Dune sand
Qal	Alluvium
Ql	Lake deposits
Qg	Glacial deposits
Qt	River terrace deposits
Qm	Pleistocene marine and marine terrace deposits
Qpv	Pleistocene volcanic rocks Qpv^r-rhyolite Qpv^a-andesite Qpv^b-basalt Qpv^p-pyroclastic rocks
Qc	Pleistocene nonmarine
QP	Plio-Pleistocene nonmarine
✳	Quaternary and/or Pliocene cinder cones
Pc	Undivided Pliocene nonmarine
Pu	Upper Pliocene marine
Pmlc	Middle and/or lower Pliocene nonmarine
Pml	Middle and/or lower Pliocene marine
Pv	Pliocene volcanic rocks Pv^r-rhyolite Pv^a-andesite Pv^b-basalt Pv^p-pyroclastic rocks
Mc	Undivided Miocene nonmarine
Muc	Upper Miocene nonmarine
Mu	Upper Miocene marine
Mm	Middle Miocene marine
Ml	Lower Miocene marine
Mv	Miocene volcanic rocks Mv^r-rhyolite Mv^a-andesite Mv^b-basalt Mv^p-pyroclastic rocks
Φc	Oligocene nonmarine
Ec	Eocene nonmarine
E	Eocene marine
Ep	Paleocene marine
Tc	Tertiary nonmarine
Tm	Tertiary marine
Ti	Tertiary intrusive (hypabyssal) rocks Ti^r-rhyolite Ti^a-andesite Ti^b-basalt
Tl	Tertiary lake deposits
Tv	Tertiary volcanic rocks Tv^r-rhyolite Tv^a-andesite Tv^b-basalt Tv^p-pyroclastic rocks
Ku	Upper Cretaceous marine
Ju	Upper Jurasssic marine
gr	Mesozoic granitic rocks
bi	Mesozoic basic intrusive rocks
ub	Mesozoic ultrabasic intrusive rocks
JRv	Jurassic-Triassic metavolcanic rocks
m ls	Pre-Cretaceous metamorphic rocks (ls=limestone)
ms	Pre-Cretaceous metasedimentary rocks
mv	Pre-Cretaceous metavolcanic rocks
gr-m	Pre-Cenozoic granitic and metamorphic rocks
pCc	Precambrian igneous and metamorphic rock complex
pC	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.

Plate 3
Regional Geologic Map

Proposed Indio Maverik Store
Indio Center Drive & Avenue 45
Indio, Riverside County, California

Earth Systems

7/3/2023
File No.: 306043-001

Table A-1
Fault Parameters

Fault Section Name	**Distance**		Upper Seis. Depth	Lower Seis. Depth	Avg Dip Angle	Avg Dip Direction	Avg Rake	Trace Length	Fault Type	**Mean Mag**	Mean Return Interval	Slip Rate
	(miles)	(km)	(km)	(km)	(deg.)	(deg.)	(deg.)	(km)			(years)	(mm/yr)
San Andreas (Coachella) rev FM3.1, 3.2	1.5	2.5	0.0	11.1	90	224	180	69	A	**6.8**	89	9
San Andreas (San Gorgonio Pass-Garnet HIll) FM	5.1	8.2	0.0	12.8	58	20	180	56	A	**7.6**	219	24
San Andreas, (North Branch, Mill Creek) FM3.1,	5.1	8.2	0.0	18.2	76	204	180	106	A	**7.6**	219	34
Blue Cut FM3.1, 3.2	12.5	20.0	0.0	13.1	90	177	na	79	B'	**7.1**		
Joshua Tree (Seismicity) FM3.1, 3.2	15.5	24.9	0.0	13.3	90	271	na	17	B'	**6.5**		
Burnt Mtn FM3.1, 3.2	18.1	29.2	0.0	15.9	67	265	180	21	B	**6.7**		0.6
Eureka Peak FM3.1, 3.2	18.2	29.3	0.0	15.0	90	75	180	19	B	**6.6**		0.6
San Jacinto (Clark) rev FM3.1, 3.2	23.8	38.3	0.0	16.8	90	214	180	47	A	**7.6**	219	17
San Jacinto (Anza) rev FM3.1, 3.2	24.0	38.6	0.0	16.8	90	216	180	46	A	**7.6**	219	17
San Jacinto (Coyote Creek) FM3.1, 3.2	25.9	41.7	0.0	15.9	90	223	180	43	A	**7.6**	219	17
Mission Creek FM3.1, 3.2	28.7	46.1	0.0	17.7	65	5	180	31	B'	**6.9**		
Pinto Mtn FM3.1, 3.2	29.1	46.9	0.0	15.5	90	175	0	74	B	**7.2**		2.5
Emerson-Copper Mtn FM3.1, 3.2	30.1	48.4	0.0	14.1	90	51	180	54	B	**7.0**		0.6
Pisgah-Bullion Mtn-Mesquite Lk FM3.1, 3.2	30.2	48.5	0.0	13.1	90	60	180	91	B	**7.3**		0.8
Homestead Valley FM3.1, 3.2	30.8	49.6	0.0	15.9	90	na	na	46	B'	**7.0**		
Calico-Hidalgo FM3.1, 3.2	31.2	50.2	0.0	13.9	90	52	180	117	B	**7.4**		1.8
Brawley (Seismic Zone), alt 1, FM3.1	33.1	53.2	0.0	13.2	90	250	na	60	B'	**7.0**		
Johnson Valley (No) 2011 rev FM3.1, 3.2	33.2	53.4	0.0	15.9	90	51	180	52	B	**6.8**		0.6
San Jacinto (Borrego) FM3.1, 3.2	35.6	57.4	0.0	13.1	90	223	180	34	A	**7.6**	219	17
San Gorgonio Pass FM3.1, 3.2	36.2	58.3	0.0	18.5	60	11	na	29	B'	**6.9**		
Brawley (Seismic Zone), alt 2, FM3.2	38.8	62.4	0.0	13.2	90	250	na	61	B'	**7.0**		
San Andreas (San Bernardino S) FM3.1, 3.2	38.9	62.6	0.0	12.8	90	210	180	43	A	**7.6**	150	29
San Jacinto (San Jacinto Valley, stepover)	40.2	64.7	0.0	16.1	90	224	180	24	A	**7.6**	219	9
San Jacinto (Stepovers Combined) FM3.1, 3.2	40.8	65.6	0.0	16.5	90	229	180	25	A	**7.5**	110	4
Cleghorn Pass FM3.1, 3.2	41.6	67.0	0.0	13.0	90	na	na	20	B'	**6.5**		
Cleghorn Lake FM3.1, 3.2	41.8	67.3	0.0	13.0	90	na	na	29	B'	**6.7**		
Bullion Mountains FM3.1, 3.2	41.9	67.5	0.0	13.0	90	na	na	38	B'	**6.8**		
Earthquake Valley (No Ext.) FM3.1, 3.2	42.5	68.3	0.0	18.8	90	221	180	33	B'	**6.9**		
Kickapoo FM3.1, 3.2	42.9	69.0	0.0	15.1	90	na	na	6	B'	**6.1**		
Earthquake Valley FM3.1, 3.2	43.4	69.9	0.0	18.8	90	217	180	20	B	**6.7**		2
Sheephole FM3.1, 3.2	44.5	71.5	0.0	13.0	90	na	na	15	B'	**6.4**		
North Frontal (East) FM3.1, 3.2	44.5	71.7	0.0	16.6	41	187	90	28	B	**6.9**		0.5
Earthquake Valley (So Ext) FM3.1, 3.2	46.5	74.9	0.0	18.8	90	204	180	9	B'	**6.3**		
Elsinore (Julian) FM3.1, 3.2	46.6	75.1	0.0	18.8	84	36	180	75	A	**7.6**	725	2.5
Elmore Ranch FM3.1, 3.2	46.7	75.2	0.0	11.4	90	310	0	29	B	**6.6**		1
Lenwood-Lockhart-Old Woman Springs FM3.1, 3	50.0	80.5	0.0	13.2	90	43	180	145	B	**7.5**		0.9
Superstition Hills FM3.1, 3.2	50.8	81.7	0.0	12.6	90	220	180	36	A	**7.4**	199	9
San Jacinto (Superstition Mtn) FM3.1, 3.2	52.1	83.8	0.0	12.4	90	210	180	26	A	**7.4**	199	14
Elsinore (Coyote Mountain) FM3.1, 3.2	52.2	84.0	0.0	13.2	82	35	180	39	A	**7.1**	322	15
San Jacinto (San Jacinto Valley) rev FM3.1, 3.2	52.3	84.2	0.0	16.1	90	223	180	18	A	**7.6**	219	18

Reference: USGS OFR 2013-1165 (CGS SP 228) Based on Site Coordinates of 33.7229 Latitude, -116.2043 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: 33.723 N 116.204 W

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

		Epicenter		*Distance*	
		Latittude	*Longitude*	*from*	*Magnitude*
Day	*Year*	*(Degrees)*		*Site (mi)*	M_W
6/29	1992	33.87	116.27	10.8	**5.5**
4/23	1992	33.96	116.32	17.7	**6.2**
12/4	1948	34.00	116.23	19.2	**6.0**
6/30	1992	34.00	116.36	21.1	**5.1**
8/21	1993	34.03	116.32	22.2	**5.0**
4/3	1926	34.00	116.00	22.4	**5.5**
3/25	1937	33.46	116.44	22.7	**5.6**
4/11	1910	33.50	116.50	22.9	**5.8**
10/31	2001	33.51	116.51	22.9	**5.1**
2/9	*1890	33.40	116.30	23.0	**6.8**
5/18	1940	34.05	116.30	23.2	**5.2**
2/25	1980	33.52	116.55	24.3	**5.1**
6/10	2016	33.43	116.44	24.3	**5.2**
6/12	2005	33.53	116.57	24.9	**5.2**
9/15	1992	34.06	116.36	24.9	**5.2**
5/18	1940	34.08	116.30	25.3	**5.3**
7/25	1947	34.05	116.42	25.7	**5.2**
7/24	1947	34.02	116.48	25.9	**5.3**
7/7	2010	33.42	116.49	26.6	**5.4**
6/28	1992	34.06	116.47	27.8	**5.0**
6/29	1992	34.10	116.39	28.1	**5.4**
6/28	1992	34.12	116.32	28.2	**5.7**
6/29	1992	34.10	116.40	28.3	**5.7**
6/28	1992	34.10	116.42	28.8	**5.0**
6/6	1918	33.60	116.70	29.7	**5.5**
10/2	1928	33.60	116.70	29.7	**5.5**
7/8	1986	34.00	116.61	30.1	**6.0**
6/28	1992	34.12	116.43	30.3	**5.5**
6/28	1992	34.13	116.41	30.5	**5.8**
3/19	1954	33.29	116.07	30.9	**6.4**
12/16	1988	33.98	116.68	32.5	**5.0**
6/12	1944	34.01	116.69	34.2	**5.2**
6/12	1944	34.00	116.72	35.2	**5.1**
4/28	1969	33.23	116.37	35.3	**5.5**
6/28	1992	34.20	116.44	35.6	**7.3**
5/2	1949	33.99	115.67	35.8	**5.7**
5/28	*1892	33.20	116.20	36.1	**6.5**
9/30	1916	33.20	116.10	36.6	**5.7**
3/19	1954	33.20	116.09	36.7	**5.5**
2/7	1889	34.10	116.70	38.5	**5.6**

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-3 - 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)

Seismic Design Category		**E**	CBC Reference	ASCE 7-16 Reference
Site Class		**D**	Table 1613.5.6	Table 11.6-2
Latitude:		33.723	Table 1613.5.2	Table 20.3-1
Longitude:		-116.204		
Maximum Considered Earthquake (MCE) Ground Motion				
Short Period Spectral Reponse	S_S	**2.272 g**	Figure 1613.5	Figure 22-1
1 second Spectral Response	S_1	**0.964 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}	2.272 g	= F_axS_S	
	S_{M1}	1.639 g	= F_vxS_1	
ASCE7-16 Supplement 3, Exception 1*	S_{M1*}	2.458 g	=S_{M1}x1.5	Exception 1, 11.4.8, Supplement 3
Design Earthquake Ground Motion				
Short Period Spectral Reponse	S_{DS}	**1.515 g**	= 2/3xS_{MS}	
Unadjusted 1 second Spectral Response	S_{D1}	**1.093 g**	= 2/3xS_{M1}	
Modified 1 second Spectral Response*	S_{D1*}	**1.639 g**	= 2/3xS_{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)	0.72 sec	= S_{D1}/S_{DS}
Risk Category	II	Table 1604.5
Seismic Importance Factor	1.00	
F_{PGA}	1.10	
PGA_M	**1.08**	
Vertical Coefficient (C_V)	1.50	Table 11.9-1

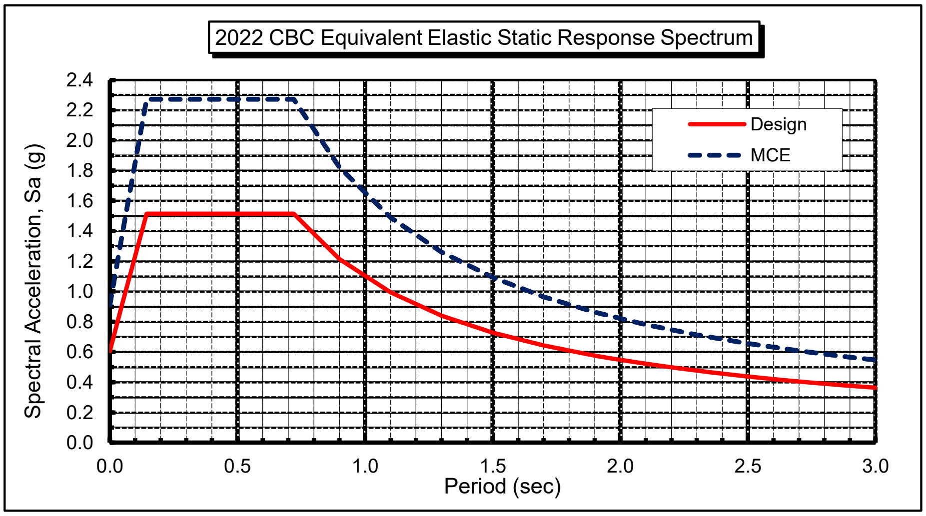
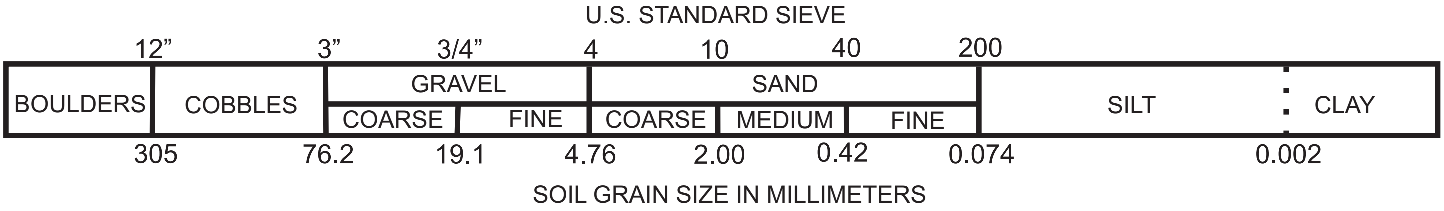


Table 11.5-1	Design
Period T (sec)	Sa (g)
0.00	0.606
0.05	0.921
0.14	1.515
0.72	1.515
0.90	1.214
1.10	0.993
1.30	0.840
1.50	0.728
1.70	0.643
1.90	0.575
2.10	0.520
2.30	0.475
2.50	0.437
2.70	0.405
2.90	0.377
3.10	0.352

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



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.

GROUNDWATER LEVEL

▼	Water Level (measured or after drilling)
▽	Water Level (during drilling)

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

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

Boring No. B-1

Project Name: Indio Maverik Store
Project Number: 306043-001
Boring Location: See Plate 2

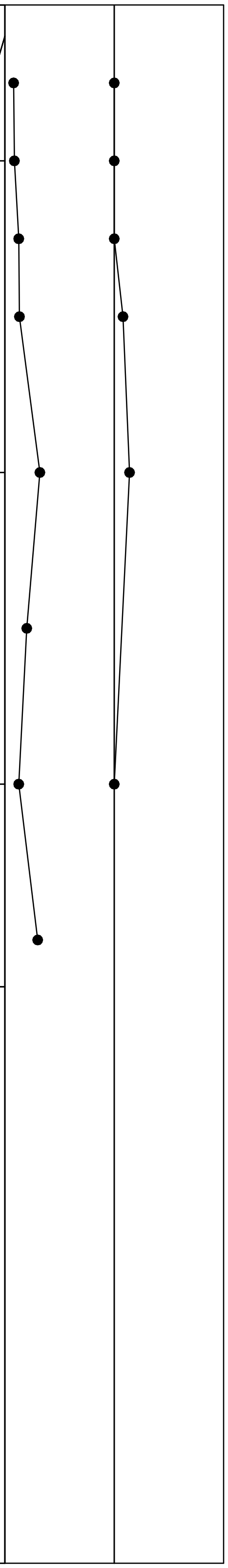
Drilling Date: 5/22/23
Drilling Method: Mobile B-61 w/auto hammer
Drill Type: 8" HSA
Logged By: Julian G.

Page 1 of 1

Description of Units

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

Depth (Ft.)	Penetration Resistance (Blows/6")	USCS	Dry Density (pcf)	Moisture Content (%)	Description of Units
0		SP			GRAVELLY SAND: grey brown, very loose, dry, fine to coarse grained sand, asphalt grindings (AF*)
	5,6,6	SM	96	1	SILTY SAND: tan brown, loose, dry, fine to medium grained sand, lenses of silt, trace root hairs, slight porosity
5	4,5,8	ML	94	1	SILT WITH SAND: olive brown, firm, dry, fine grained sand, trace of shells, moderately porous
	7,8,11		95	1	
					stiff
10	6,9,11		104	4	damp, no porosity
15	9,13,25	ML	107	10	SANDY SILT: olive brown, very stiff, slightly moist, fine grained sand, interbedded with equal layers of silty sand
20	9,12,18				
25	7,8,11	CL	89	33	SANDY LEAN CLAY: olive brown, stiff, very moist, mottled, interbedded with layers of silt, cemented nodules
30	6,17,26				hard
35					
40					
45					
50					



*AF - Artificial Fill
Boring completed at 31-1/2 feet
No groundwater encountered
Backfilled with cuttings

Boring No. B-2

Project Name: Indio Maverik Store
Project Number: 306043-001
Boring Location: See Plate 2

Drilling Date: 5/22/23
Drilling Method: Mobile B-61 w/auto hammer
Drill Type: 8" HSA
Logged By: Julian G.

Page 1 of 1

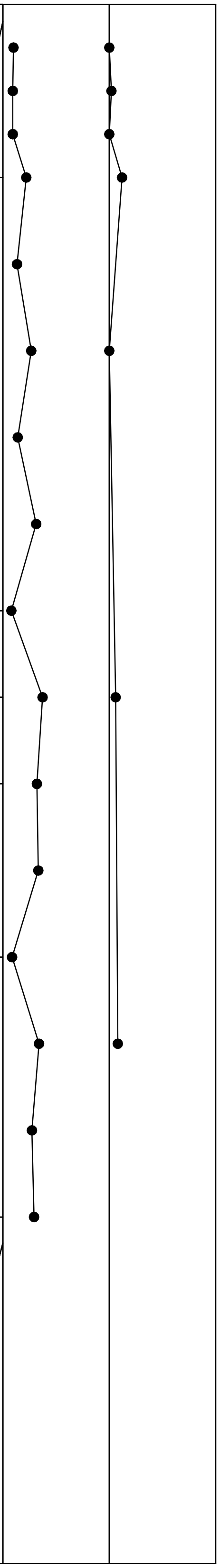
Description of Units

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

Depth (Ft.)	Bulk	SPT	MOD Calif.	Penetration Resistance (Blows/6")	USCS	Dry Density (pcf)	Moisture Content (%)	Description of Units
0	■				SP			POORLY GRADED SAND: light grey, very loose, dry, fine grained sand
	■		■	5,6,9	SP-SM	99	0	POORLY GRADED SAND WITH SILT: light olive brown, loose, dry to moist, fine grained sand
5	■		■	4,5,9		101	6	
			■	3,5,8		85	10	
10			■	8,11,22	ML	106	7	SANDY SILT: light olive brown, firm, damp, fine grained sand, some small shells
15		◧		7,7,13				with carbonate stringers and small nodules
20			■	9,14,26		99	7	interbedded silt, silty sand, fine sand
25		◧		7,8,13				interbedded with layers of clay
30			■	10,21,26				hard
35		◧		3,5,7	CL			LEAN CLAY: olive brown, stiff, moist
40			■	12,25,31	SM	103	2	SILTY SAND: light olive brown, dense, damp, fine grained sand
45		◧		11,19,29	SP			POORLY GRADED SAND: light olive brown, very dense, damp, fine grained sand, mica
50			■	21,50,50/3"				varies fine to medium and fine to coarse
55		◧		4,5,8	CL			SANDY LEAN CLAY: olive brown, very stiff, very moist, fine grained sand
60	▼		■	4,15,36		104	22	hard, wet
65		◧		8,14,27				olive grey
70		◧		8,13,31	SM			SILTY SAND: gray, dense, wet, fine to medium grained sand
75								
80								
85								
90								

Boring completed at 71-1/2 feet
Groundwater encountered at 59-1/2 feet
Backfilled with cuttings

Graphic Trend
Blow Count Dry Density



Boring No. B-3

Project Name: Indio Maverik Store
Project Number: 306043-001
Boring Location: See Plate 2

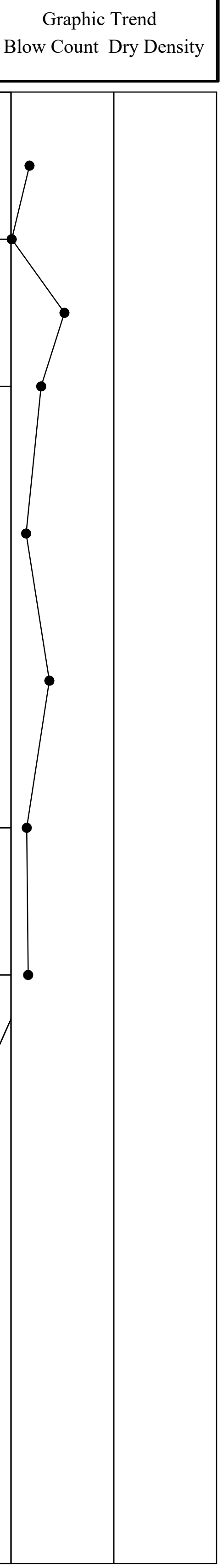
Drilling Date: 6/6/2023
Drilling Method: Track Rig
Drill Type: 8" HSA
Logged By: Julian G.

Page 1 of 1

Description of Units

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

Depth (Ft.)	Sample Type Bulk	Sample Type SPT	Sample Type MOD Calif.	Penetration Resistance (Blows/6")	Symbol	USCS	Dry Density (pcf)	Moisture Content (%)	Description of Units
0	■					SM			SILTY SAND: light grey brown, very loose, dry, fine to medium grained sand
			■	7,11,16					
5			■	5,9,12		ML			SILT WITH SAND: light grey brown, loose, dry, fine grained sand, slight porosity
			■	9,30,45					
10			■	12,16,28		SM			SILTY SAND: light tan brown, dense, dry, traces of small shells
15		■		9,9,13					no shells
20			■	11,24,32					
25		■		6,11,12		ML			SANDY SILT: brown, hard, moist, fine grained sand, slight mottling, interbedded with equal layers of lean clay with sand
30		■		12,12,13		SM			SILTY SAND: gray brown, dense, moist, fine grained sand
35									
40									
45									Boring completed at 31-1/2 feet No groundwater encountered Backfilled with cuttings
50									



Earth Systems

1680 Illinois Ave., Perris, CA 92571
Phone (951) 928-9799, Fax (951) 928-9948

Boring No. B-4

Project Name: Indio Maverik Store
Project Number: 306043-001
Boring Location: See Plate 2

Drilling Date: 5/22/23
Drilling Method: Mobile B-61 w/auto hammer
Drill Type: 8" HSA
Logged By: Julian G.

Page 1 of 1

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 Note: The stratification lines shown represent the approximate boundary between soil and/or rock types and the transition may be gradational.
0						SM			SILTY SAND: light gray brown, very loose, dry, fine grained sand, with mica
			■	5,3,3			88	1	
5			■	3,5,10		ML	94	2	SANDY SILT: olive brown, stiff, dry, fine grained sand, with mica, traces of small shells, slightly porous
			■	9,9,10			88	4	damp
10			■	7,10,17			98	3	no porosity
15		◧		7,7,9					interbedded with silt, lenses of lean clay, very stiff, damp
20			■	5,14,20			104	4	
25		◧		6,10,15					
30			■	10,28,50/5"		SM	112	1	SILTY SAND: olive brown, very dense, dry, fine grained sand
35									
40									
45									Boring completed at 31-1/2 feet No groundwater encountered Backfilled with cuttings
50									

Graphic Trend
Blow Count Dry Density

Boring No. B-5

Project Name: Indio Maverik Store
Project Number: 306043-001
Boring Location: See Plate 2

Drilling Date: 5/22/23
Drilling Method: Mobile B-61 w/auto hammer
Drill Type: 8" HSA
Logged By: Julian G.

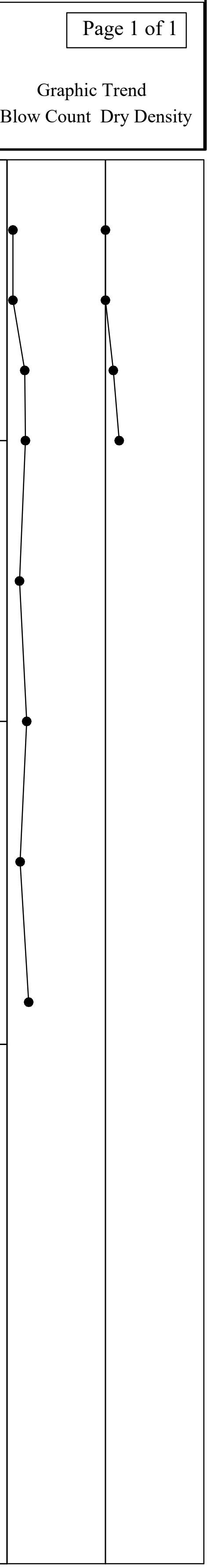
Page 1 of 1

Description of Units

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

Depth (Ft.)	Sample Type Bulk	Sample Type SPT	Sample Type MOD Calif.	Penetration Resistance (Blows/6")	USCS	Dry Density (pcf)	Moisture Content (%)	Description of Units
0	■				ML			SANDY SILT: tan brown, firm, dry, fine grained sand, mica
	■		■	3,4,5		94	2	soft
5	■		■	3,3,6		96	7	olive brown, firm, moist, small shells, slight carbonate staining
	■		■	3,10,17		104	3	very stiff
10			■	7,13,15	SP-SM	107	3	POORLY GRADED SAND WITH SILT: tan brown, medium dense, dry, fine grained sand, interbedded with silty sand, lean clay
15		◧		6,8,11				
20			■	6,12,19	ML			SILT: olive brown, very stiff, damp, interbedded with thin layer of lean clay and silty sand, clay layers are mottled, mica
25		◧		4,8,12				
30			■	5,13,20				
35								
40								
45								
50								

Boring completed at 31-1/2 feet
No groundwater encountered
Backfilled with cuttings



Earth Systems
1680 Illinois Ave., Perris, CA 92571
Phone (951) 928-9799, Fax (951) 928-9948

Boring No. B-6

Project Name: Indio Maverik Store
Project Number: 306043-001
Boring Location: See Plate 2

Drilling Date: 5/22/23
Drilling Method: Mobile B-61 w/auto hammer
Drill Type: 8" HSA
Logged By: Julian G.

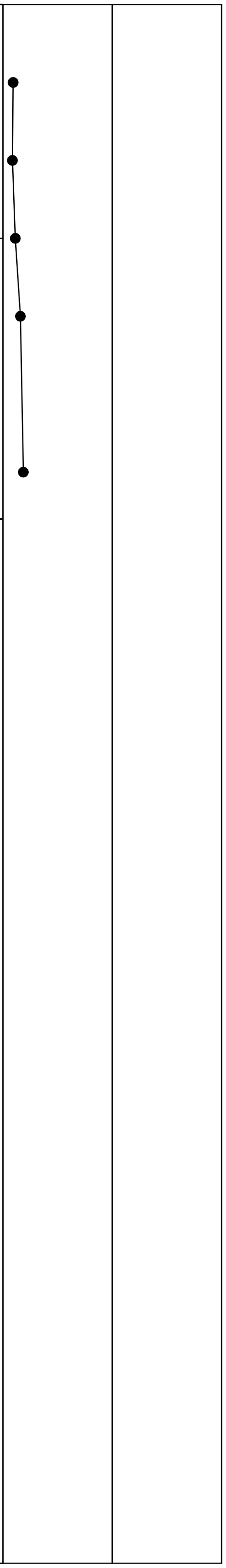
Page 1 of 1

Description of Units

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

Depth (Ft.)	Sample Type: Bulk	Sample Type: SPT	Sample Type: MOD Calif.	Penetration Resistance (Blows/6")	Symbol	USCS	Dry Density (pcf)	Moisture Content (%)	Description of Units
0						SM			SILTY SAND: light olive brown, loose, dry, fine to medium grainded sand, trace of gravel 1 inch
			■	5,6,8			91	2	
5			■	3,5,8			97	1	olive brown, micaceous
			■	6,8,9		ML	92	6	SILT: olive brown, stiff, damp, trace small shells and, roots, slightly porous
10			■	5,8,16					very stiff, mottled with carbonate stringers, no porosity
15		◧		9,12,16					moist, hard, alternating layers of silty sand
20									
25									
30									
35									
40									
45									Boring completed at 16-1/2 feet No groundwater encountered Backfilled with cuttings
50									

Graphic Trend
Blow Count Dry Density



Earth Systems

1680 Illinois Ave., Perris, CA 92571
Phone (951) 928-9799, Fax (951) 928-9948

Boring No. P-1

Project Name: Indio Maverik Store
Project Number: 306043-001
Boring Location: See Plate 2

Drilling Date: 5/22/23
Drilling Method: Mobile B-61 w/auto hammer
Drill Type: 8" HSA
Logged By: Julian G.

Page 1 of 1

Description of Units

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

Depth (Ft.)	Sample Type Bulk	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	Graphic Trend Dry Density
0						SM			SILTY SAND: light tan brown, very loose, dry, fine grained sand, with mica		
5						ML			SANDY SILT: olive brown, dry, fine grained sand, with mica		
10											
15											
20			■	10,13,23					mottled, medium grained sand		
25											
30											
35											
40											
45									pipe, sock and rock installed Perc Test completed at 21-1/2 feet No groundwater encountered Backfilled with cuttings		
50											

Boring No. P-2

Project Name: Indio Maverik Store
Project Number: 306043-001
Boring Location: See Plate 2

Drilling Date: 5/22/23
Drilling Method: Mobile B-61 w/auto hammer
Drill Type: 8" HSA
Logged By: Julian G.

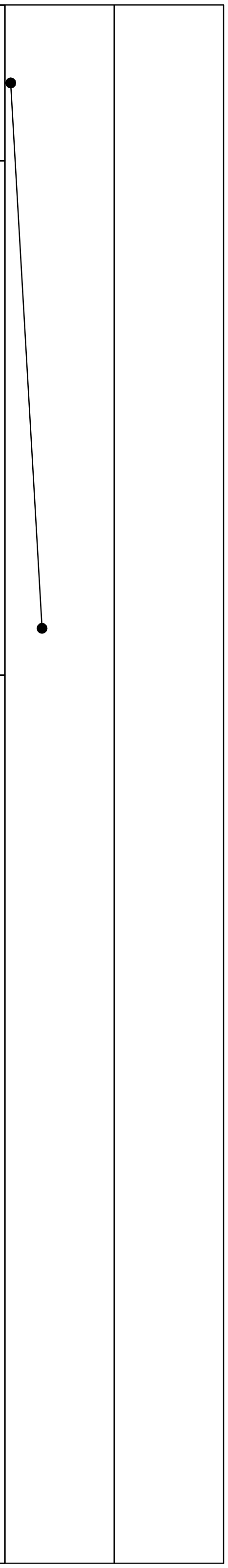
Page 1 of 1

Description of Units

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

Depth (Ft.)	Sample Type: Bulk	Sample Type: SPT	Sample Type: MOD Calif.	Penetration Resistance (Blows/6")	Symbol	USCS	Dry Density (pcf)	Moisture Content (%)	Description of Units
0		■		3,4,4		SM			SILTY SAND: light tan brown, very loose, dry, with mica
5						ML			SANDY SILT: olive brown, damp, fine grained sand, traces of small shells and mica
20			■	11,18,33					mottled, very dense
45									(Pipe, sock and rock installed) Perc Test completed at 21-1/2 feet No groundwater encountered Backfilled with cuttings
50									

Graphic Trend
Blow Count Dry Density



Boring No.	B-2	**Project and Number**	Indio Maverik	306043-001
		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	**Calculation Results**
5/22/2023	**Ave. SPT N60HE-value (blows/ft)**
	25
	(Based on Upper 70 feet)
Hammer Weight (lbs)	**Ave. Shear Wave Velocity (ft/sec)**
140 `	871
	(Based on Upper 70 feet)
Hammer Drop (inches)	**Soil Profile Type (Site Class)**
30	D
	(Based on Upper 70 feet)
Hammer Efficiency (E_M)	**Ave. Friction Angle (degrees)**
72	34
	(Based on Upper 70 feet)
Borehole Correction (Cb)*	**Estimated Shear Wave Velocity ****
1	**Based on Depth Less than 100' ft**
*inside diameter of Hollow Stem Auger	**863 (ft/sec Upper 100 feet)**
	Soil Profile Type (Site Class)**
Sampler Correction Mod Cal to SPT	**D**
0.63	**Based on**
	Ave. Shear Wave Velocity (ft/sec)
	263 (m/sec Upper 100 feet)
Sampler Liner Correction (Cs)	
1.2 Applied if SPT Sampler Used	
1.0 Applied if Cal Sampler Used	**Ave. Field SPT N-value (blows/ft)**
	20.9
Rod Length Above Ground (ft)	(Based on Upper 70 feet)
3	
	Ave. Field SPT N-value (blows/ft)
Depth to Estimate Vs Over (ft)*	**26.4**
100	(Based on Upper 100 feet)
*Caltrans Estimation Method	
	Soil Profile Type (Site Class)**
***N_{sub} Value Desired For Column 6**	**D**
60	**Based on**
*Only Used for Calculating Nsub otherwise not used by program (i.e.N50, N70, N80, etc)	**Ave. Field Blow Count**
	26 (Upper 100 feet)

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

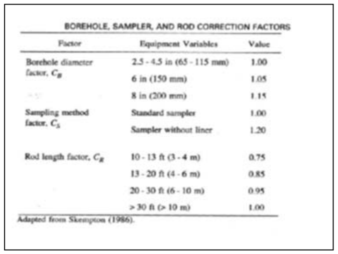
→ Hammer energy as related to the standard 60% delivered energy, i.e. a 72% hammer has and energy ratio of 1.2, i.e. (72/60=1.2)

1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
Bottom of Layer Depth (ft)	**Blow Count*****	**Type of Sampler**	**d_i (feet)**	**N_{60} (blows/ft)**	**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	15	c	2.5	8.51	8.51	11.34	203.24	666.62	29.75	0.22046	0.00375	0.084046	Loose	Stiff
5.0	14	c	2.5	7.94	7.94	10.58	199.21	653.41	29.47	0.23621	0.00383	0.084841	Loose	Firm
7.5	13	c	2.5	7.37	7.37	9.83	194.98	639.52	29.17	0.25438	0.00391	0.085695	Loose	Firm
10.0	33	c	2.5	18.71	18.71	24.95	255.45	837.88	33.35	0.10021	0.00298	0.074959	Medium Dense	Very Stiff
12.5	33	c	2.5	21.21	21.21	24.95	255.45	837.88	33.35	0.10021	0.00298	0.074959	Medium Dense	Very Stiff
15.0	20	s	2.5	20.40	20.40	24.00	252.60	828.52	33.16	0.10417	0.00302	0.075403	Medium Dense	Very Stiff
20.0	40	c	5.0	28.73	28.73	30.24	270.11	885.95	34.36	0.16534	0.00564	0.145516	Medium Dense	Very Stiff
25.0	21	s	5.0	23.94	23.94	25.20	256.20	840.32	33.40	0.19841	0.00595	0.149687	Medium Dense	Very Stiff
30.0	47	c	5.0	35.53	35.53	35.53	283.04	928.37	35.25	0.14072	0.00539	0.141846	Dense	Hard
35.0	12	s	5.0	14.40	14.40	14.40	217.82	714.44	30.75	0.34722	0.00700	0.162578	Medium Dense	Stiff
40.0	56	c	5.0	42.34	42.34	42.34	297.79	976.76	36.26	0.11810	0.00512	0.137883	Dense	Hard
45.0	48	s	5.0	57.60	57.60	57.60	325.60	1067.98	38.17	0.08681	0.00468	0.130994	Very Dense	Hard
50.0	100	c	5.0	75.60	75.60	75.60	352.32	1155.61	40.00	0.06614	0.00433	0.125002	Very Dense	Hard
55.0	13	s	5.0	15.60	15.60	15.60	222.93	731.22	31.11	0.32051	0.00684	0.16073	Medium Dense	Very Stiff
60.0	51	c	5.0	38.56	38.56	38.56	289.82	950.62	35.72	0.12968	0.00526	0.139995	Dense	Hard
65.0	41	s	5.0	49.20	49.20	49.20	311.05	1020.26	37.17	0.10163	0.00490	0.134508	Dense	Hard
70.0	44	s	5.0	52.80	52.80	52.80	317.49	1041.37	37.61	0.09470	0.00480	0.13293	Very Dense	Hard
		Total:	70.0	"d" Feet				Total:		2.78488	0.08038	2.041573		

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

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

Consistency classification based upon ASCE 1996



BOREHOLE, SAMPLER, AND ROD CORRECTION FACTORS

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

Adapted from Skempton (1986).

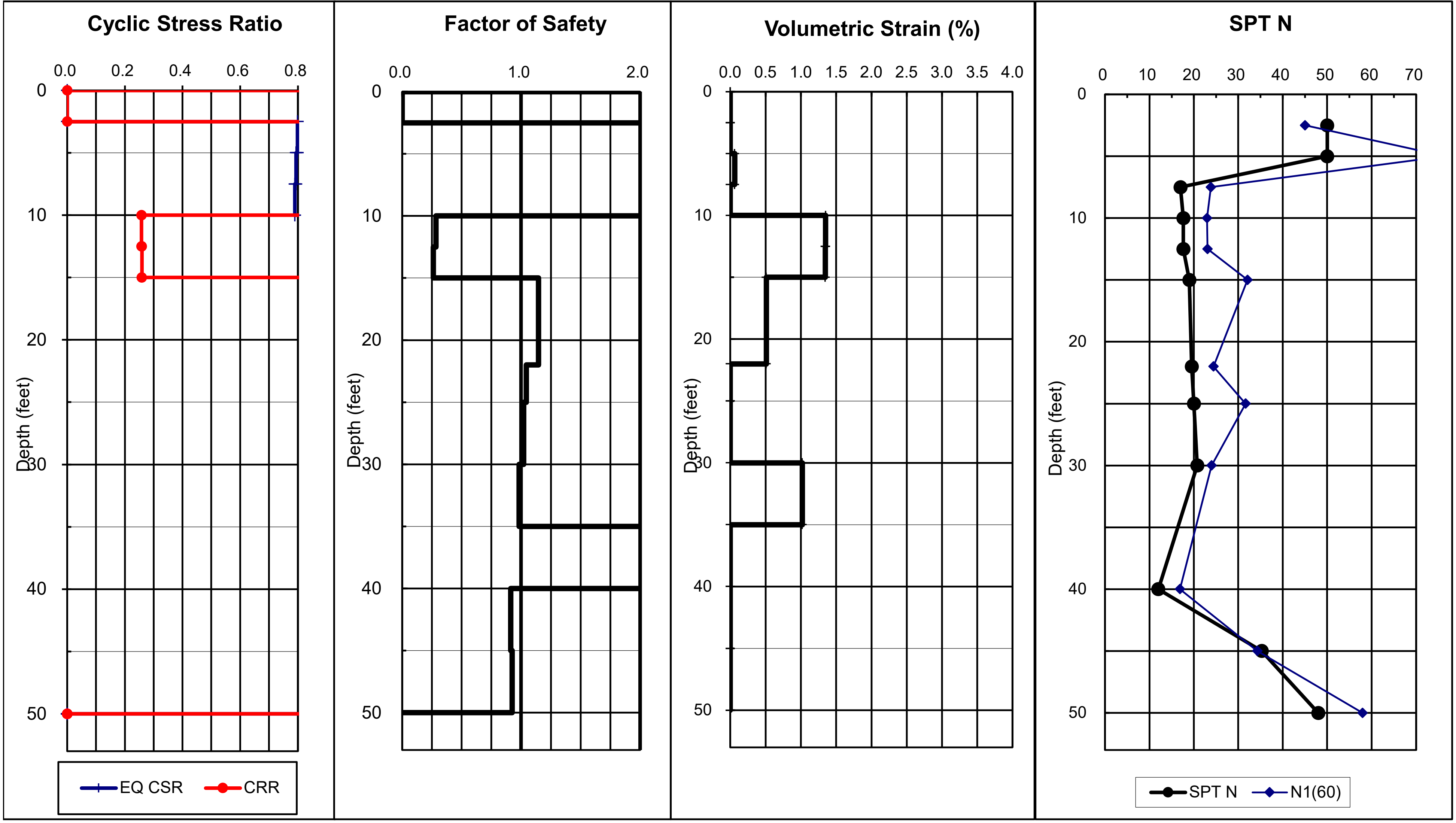
Spreadsheet Version 2.6, 2019: Prepared by Kevin L. Paul, PE, GE

EARTH SYSTEMS - EVALUATION OF LIQUEFACTION POTENTIAL AND INDUCED SUBSIDENCE

Indio Maverik Store **Project No: 306043-001** 1996/1998 NCEER Method

Ground Compaction Remediated to 8 foot depth

Boring: B-5 **Earthquake Magnitude: 7.9** **PGA, g: 1.08** **Calc GWT (feet): 7.5**



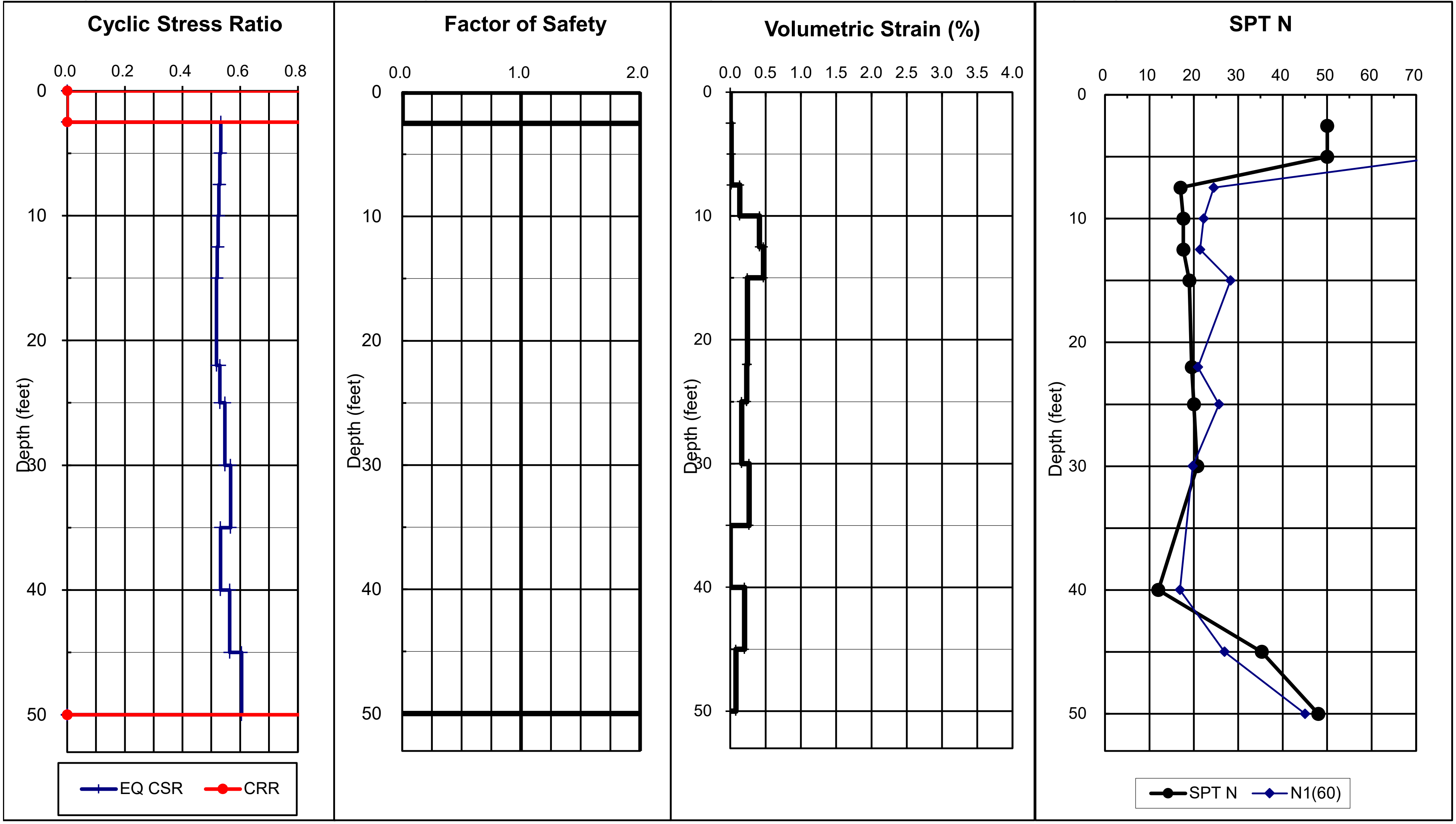
Total Thickness of Liquefiable Layers: 35.0 feet **Estimated Total Ground Subsidence: 1.9 inches**

EARTH SYSTEMS - EVALUATION OF LIQUEFACTION POTENTIAL AND INDUCED SUBSIDENCE

Indio Maverik Store **Project No: 306043-001** 1996/1998 NCEER Method

Ground Compaction Remediated to 8 foot depth

Boring: B-5 **Earthquake Magnitude: 7.9** **PGA, g: 0.72** **Calc GWT (feet): 50**



Total Thickness of Liquefiable Layers: 0.0 feet

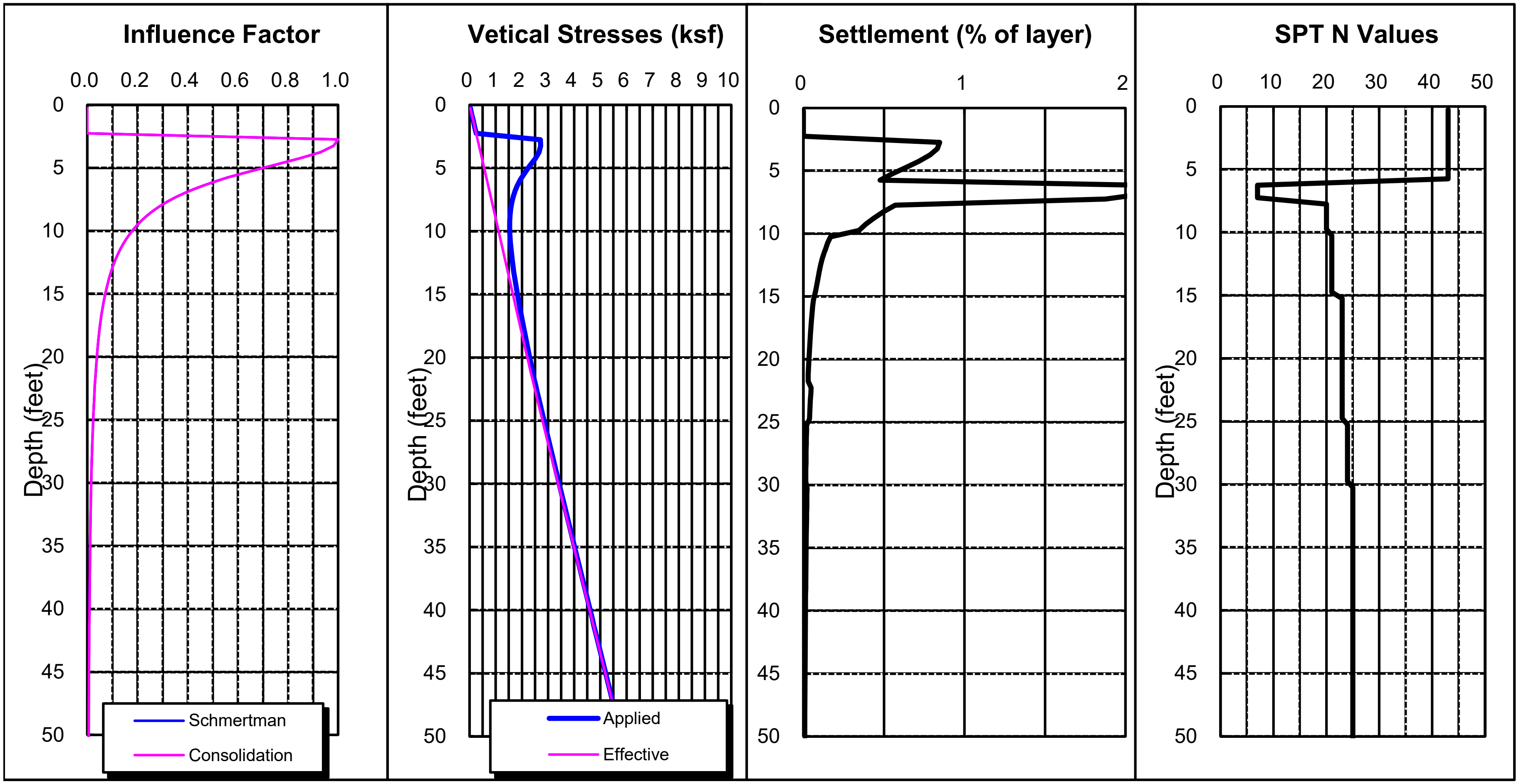
Estimated Total Ground Subsidence: 1.0 inches

EARTH SYSTEMS - SETTLEMENT ANALYSES

Indio Maverik Store Column Load

306043-001

Width, ft: 5.0 Length, ft: 5.0 Net pressure, ksf: 2.40 **Settlement, inches: 1.0**



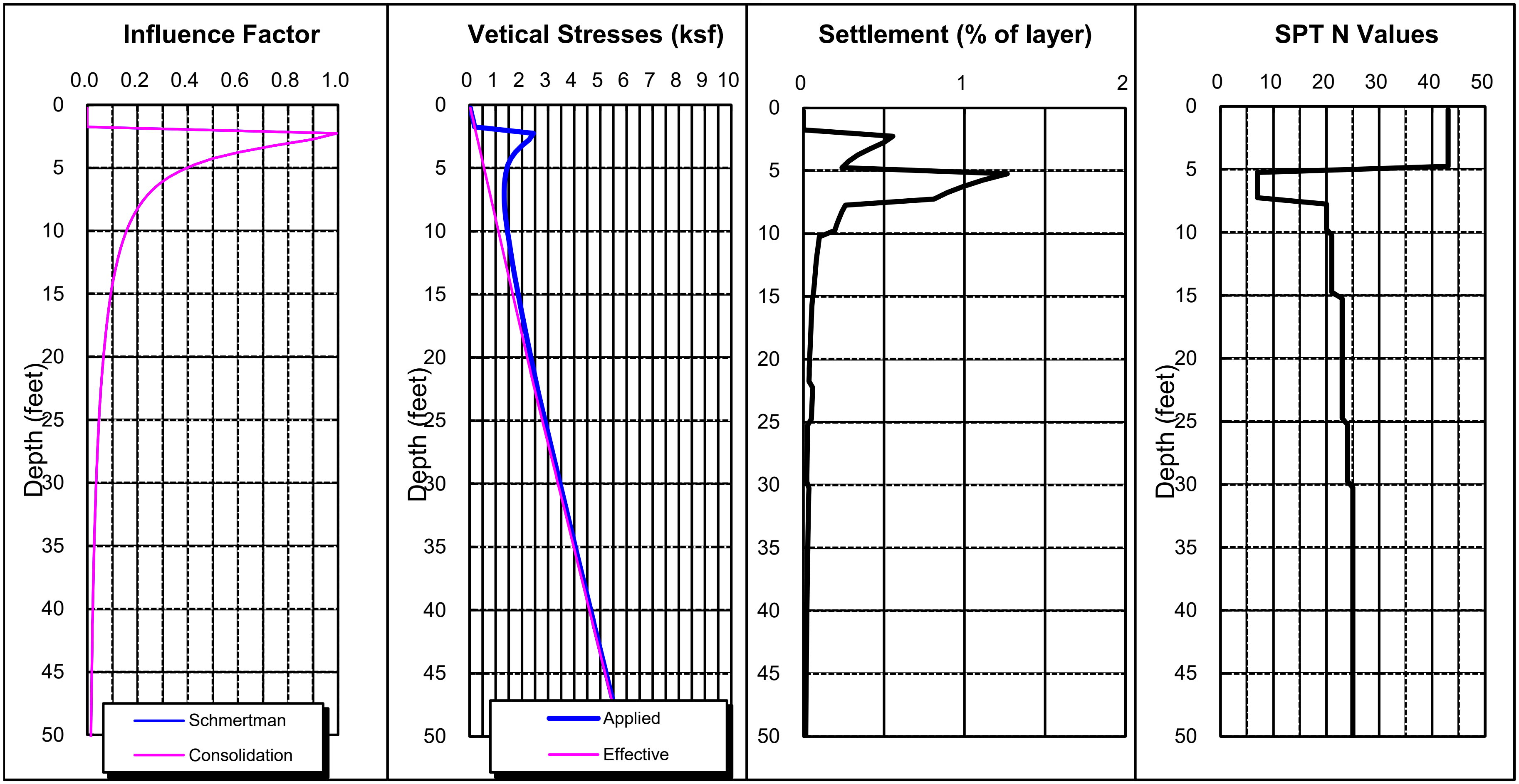
Load, Q: 60 kips Embedment, feet: 2.5 **Boring: B-5**

EARTH SYSTEMS - SETTLEMENT ANALYSES

Indio Maverik Store Wall Load

306043-001

Width, ft: 2.0 Length, ft: 40.0 Net pressure, ksf: 2.20 **Settlement, inches: 0.7**



Load, Q: 4 kpf Embedment, feet: 2.0 **Boring: B-5**

APPENDIX B

Laboratory Test Results

File No.: 306043-001 July 3, 2023

Lab No.: 23-144

UNIT DENSITIES AND MOISTURE CONTENT ASTM D2937 & D2216

Job Name: Indio Maverik Store

Sample Location	Depth (feet)	Unit Dry Density (pcf)	Moisture Content (%)	USCS Group Symbol
B-1	2.5	96	1	SM
B-1	5	94	1	ML
B-1	7.5	95	1	ML
B-1	10	104	4	ML
B-1	15	107	10	ML
B-1	25	89	33	CL
B-2	2.5	99	0	SP-SM
B-2	5	101	6	SP-SM
B-2	7.5	85	10	SP-SM
B-2	10	106	7	ML
B-2	20	99	7	ML
B-2	40	103	2	SM
B-2	60	104	22	CL
B-4	2.5	88	1	SM
B-4	5	94	2	ML
B-4	7.5	88	4	ML
B-4	10	98	3	ML
B-4	20	104	4	ML
B-4	30	112	1	SM

File No.: 306043-001 July 3, 2023

Lab No.: 23-144

UNIT DENSITIES AND MOISTURE CONTENT ASTM D2937 & D2216

Job Name: Indio Maverik Store

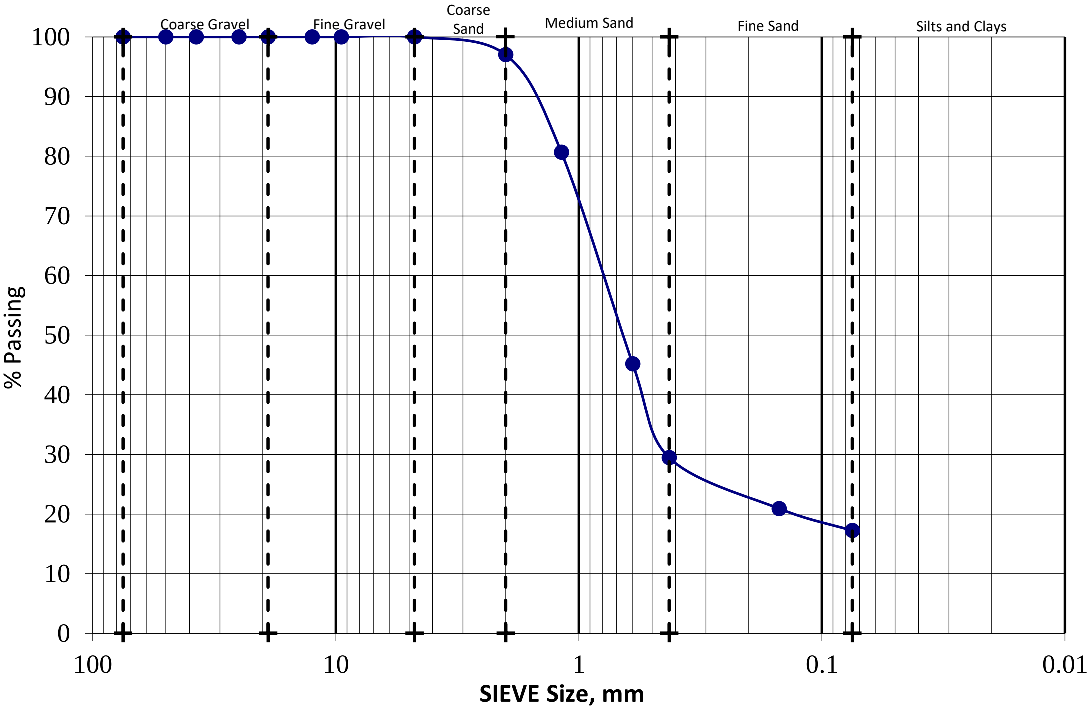
Sample Location	Depth (feet)	Unit Dry Density (pcf)	Moisture Content (%)	USCS Group Symbol
B-5	2.5	94	2	ML
B-5	5	96	7	ML
B-5	7.5	104	3	ML
B-5	10	107	3	SP-SM
B-6	2.5	91	2	SM
B-6	5	97	1	SM
B-6	7.5	92	6	ML

File No.: 306043-001 7/3/2023
Lab No.: 23-144

SIEVE ANALYSIS ASTM D6913

Job Name: Indio Maverik Store
Sample ID: B-1 @ 1-7'
Description: Silty Sand (SM)

Sieve Size	% Passing	
3"	100	-
2"	100	-
1-1/2"	100	-
1"	100	-
3/4"	100	-
1/2"	100	-
3/8"	100	-
#4	100	-
#10	97	-
#16	81	-
#30	45	-
#40	29	-
#100	21	-
#200	17.2	-



FM= 2.37

% Coarse Gravel:	0	% Coarse Sand:	3			
% Fine Gravel:	0	% Medium Sand:	68	Cu:	NA	
		% Fine Sand:	12	Cc:	NA	Gradation
% Total Gravel	0	% Total Sand	83	% Fines:	17.2	NA

File No.: 306043-001

July 3, 2023

Job Name: Indio Maverik Store

Lab Number: 23-144

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

AMOUNT PASSING NO. 200 SIEVE

(Earth Systems Method Transfers Sample until water runs clear)

Sample Location	Depth (feet)	Fines Content (%)	USCS Group Symbol	Soaking Time
B-2	7.5	10.0	SP-SM	10

File No.: 306043-001
Lab No.: 23-144

July 3, 2023

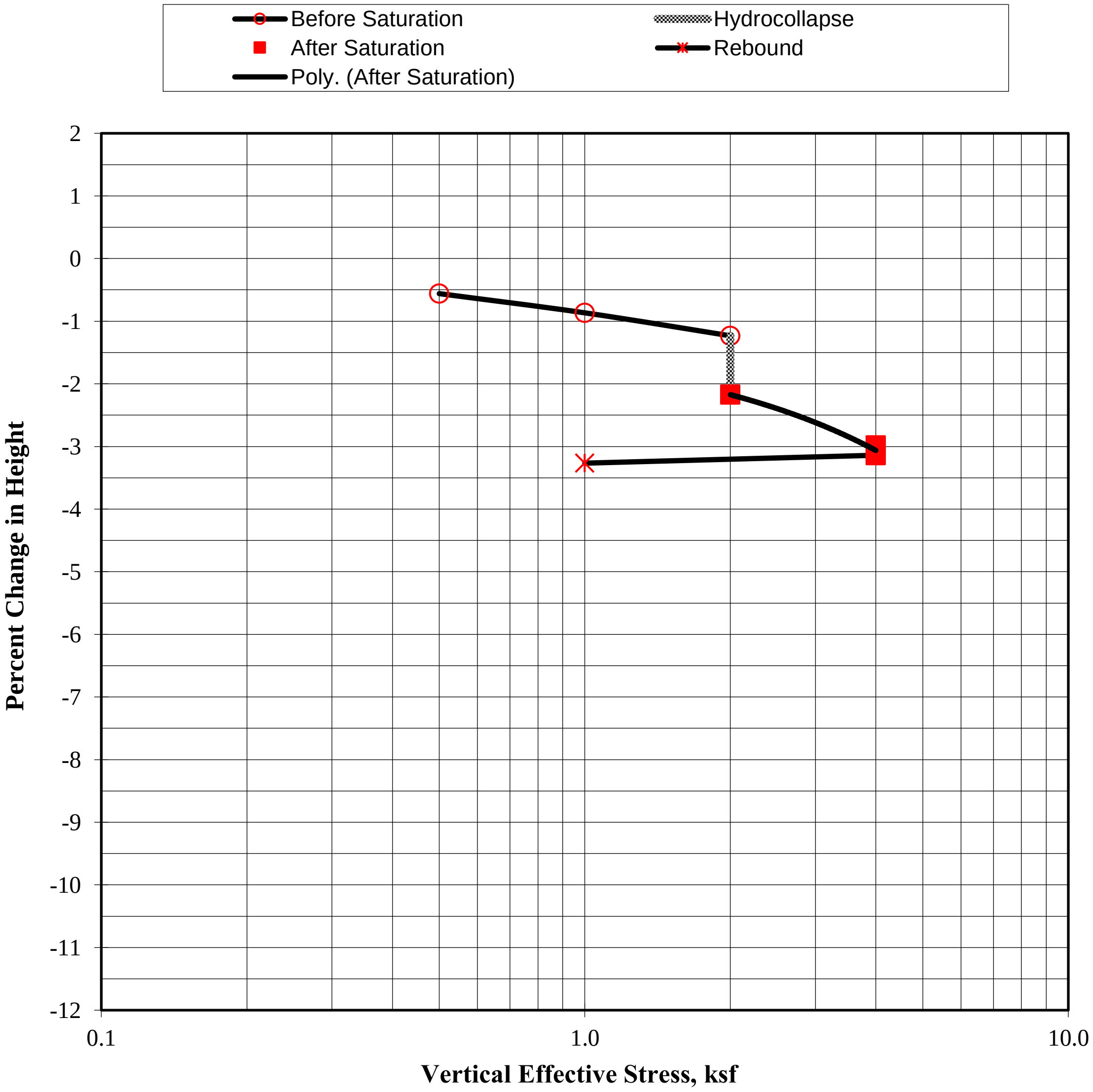
CONSOLIDATION TEST **ASTM D 2435 & D 5333**

Indio Maverik Store
B-2 @ 5'
Poorly Graded Sand with Silt (SP-SM)
Ring Sample

Initial Dry Density:	103.0 pcf
Initial Moisture:	2.5%
Specific Gravity:	2.67
Initial Void Ratio:	0.618

Hydrocollapse: 0.9% @ 2.0 ksf

% Change in Height vs Normal Pressure Diagram



File No.: 306043-001
Lab No.: 23-144

July 3, 2023

CONSOLIDATION TEST

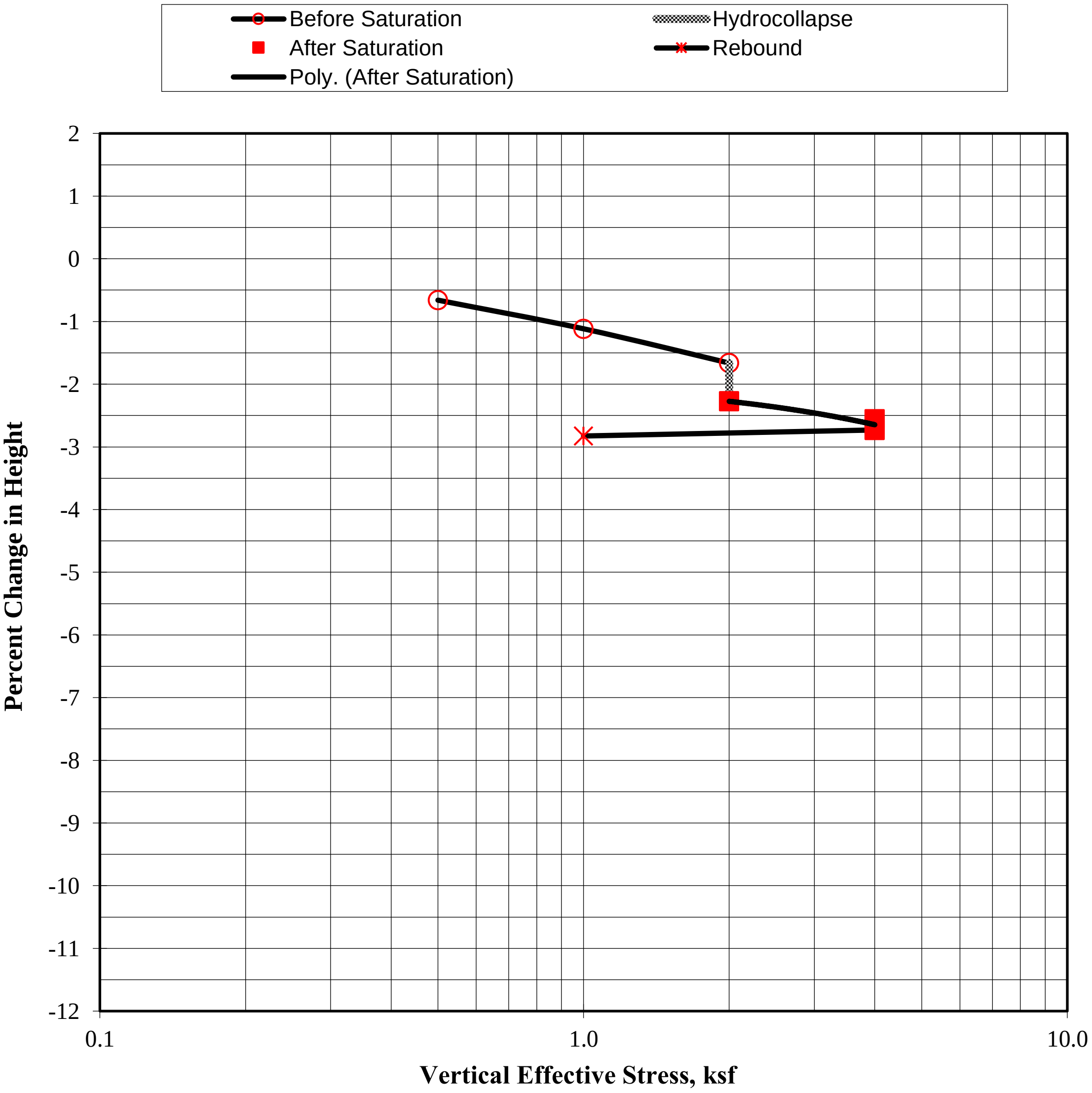
ASTM D 2435 & D 5333

Indio Maverik Store
B-2 @ 7.5'
Poorly Graded Sand with Silt (SP-SM)
Ring Sample

Initial Dry Density: 88.5 pcf
Initial Moisture: 1.7%
Specific Gravity: 2.67
Initial Void Ratio: 0.885

Hydrocollapse: 0.6% @ 2.0 ksf

% Change in Height vs Normal Pressure Diagram



File No.: 306043-001 July 3, 2023
Lab No.: 23-144

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

Indio Maverik Store
B-2 @ 10'

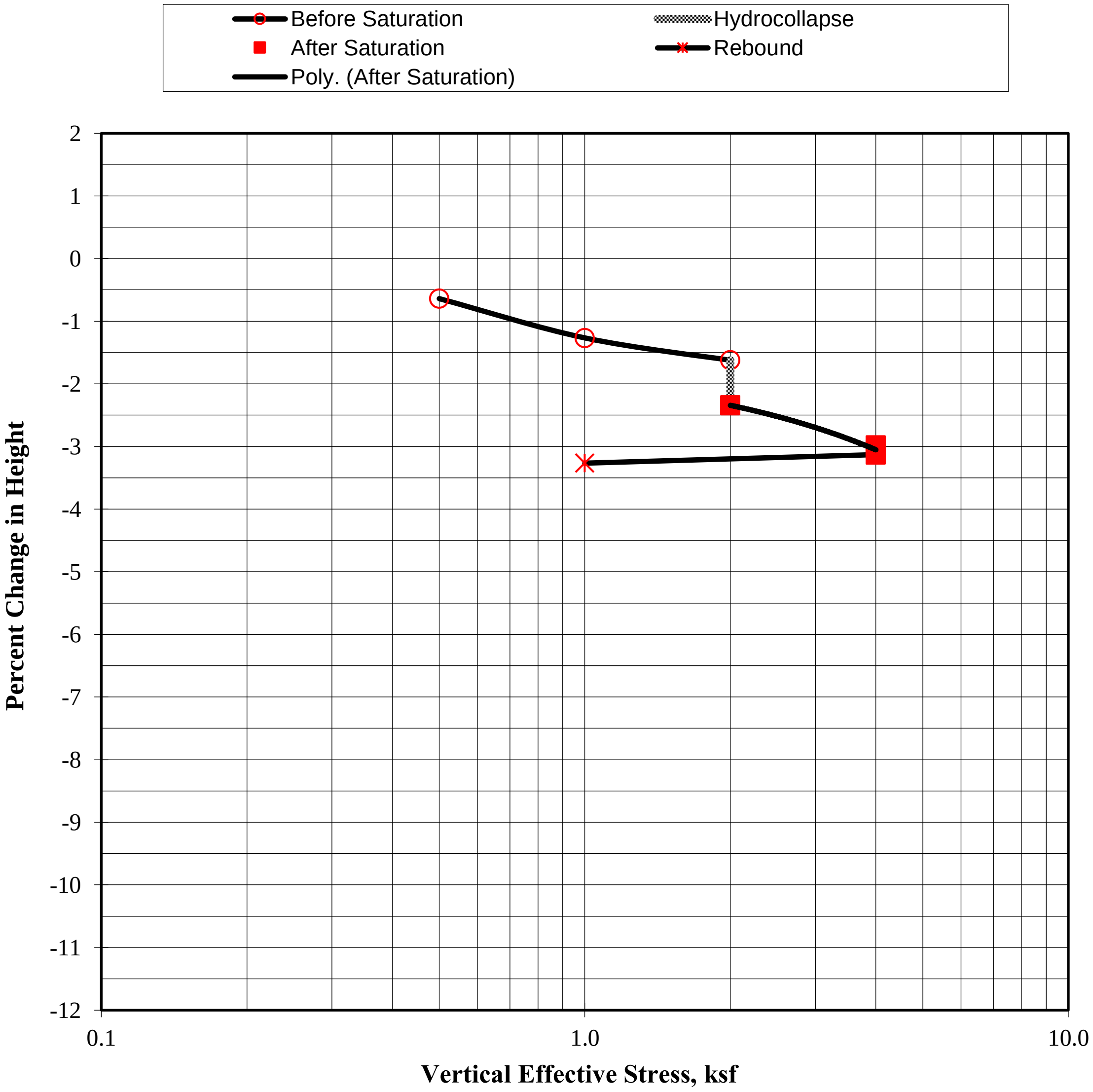
Sandy Silt (ML)

Ring Sample

Initial Dry Density:	92.6 pcf
Initial Moisture:	6.3%
Specific Gravity:	2.67
Initial Void Ratio:	0.800

Hydrocollapse: 0.7% @ 2.0 ksf

% Change in Height vs Normal Pressure Diagram



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CONSOLIDATION TEST

ASTM D 2435 & D 5333

Indio Maverik Store
B-2 @ 20'

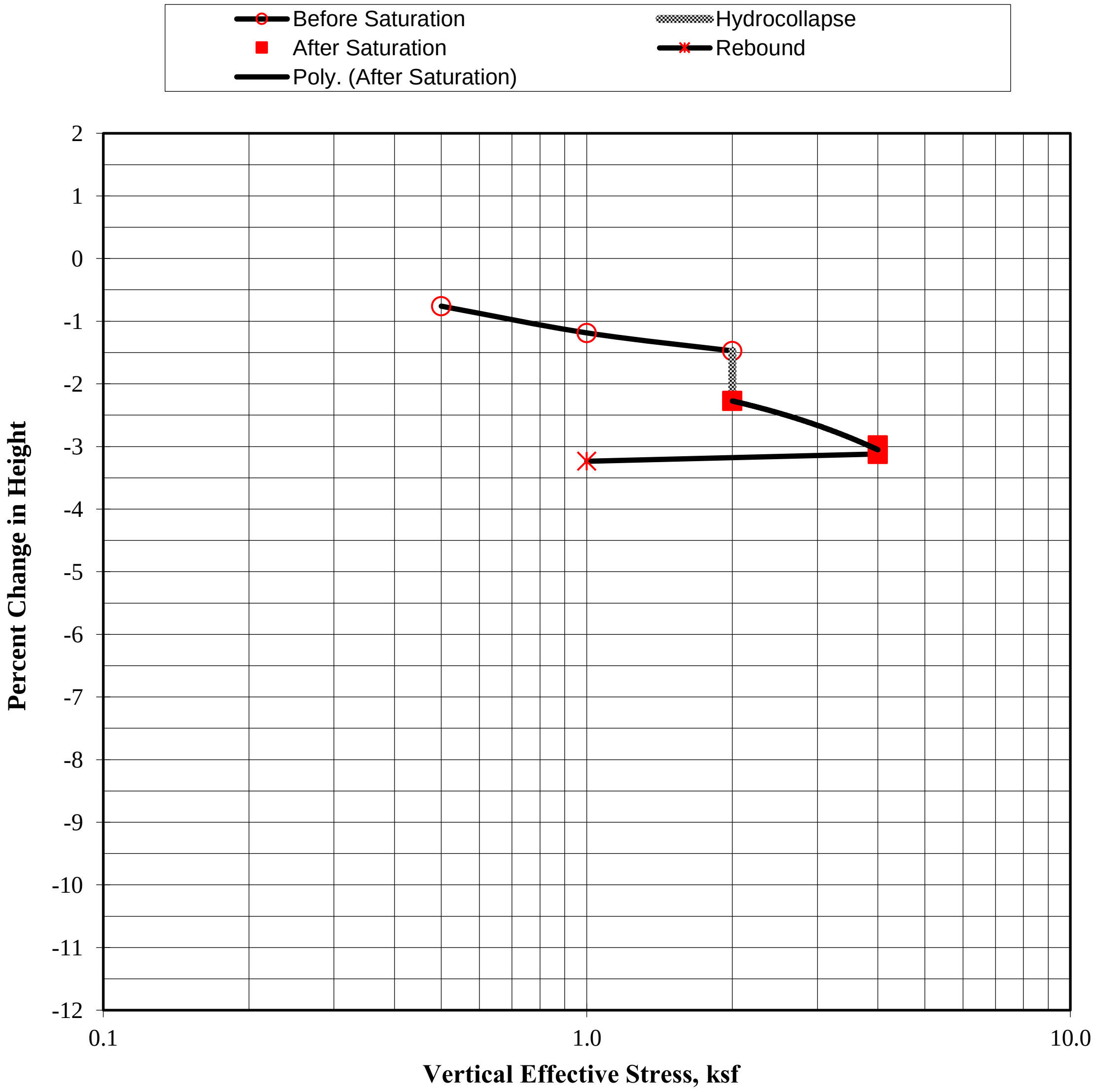
Sandy Silt (ML)

Ring Sample

Initial Dry Density: 98.4 pcf
Initial Moisture: 2.3%
Specific Gravity: 2.67
Initial Void Ratio: 0.694

Hydrocollapse: 0.8% @ 2.0 ksf

% Change in Height vs Normal Pressure Diagram



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EXPANSION INDEX **ASTM D-4829**

Job Name: Indio Maverik Store
Sample ID: B-1 @ 1-7'
Soil Description: Silty Sand (SM)

Initial Moisture, %:	7.4
Initial Compacted Dry Density, pcf:	109.4
Initial Saturation, %:	49
Final Moisture, %:	18.4
Volumetric Swell, %:	0.1

Expansion Index, EI: **1** **Very Low**

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

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EXPANSION INDEX **ASTM D-4829**

Job Name: Indio Maverik Store
Sample ID: B-4 @ 2.5'
Soil Description: Silty Sand (SM)

Initial Moisture, %:	9.2
Initial Compacted Dry Density, pcf:	111.2
Initial Saturation, %:	49
Final Moisture, %:	19.8
Volumetric Swell, %:	0.1

Expansion Index, EI: 0 Very Low

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

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Lab No.: 23-144

MAXIMUM DRY DENSITY / OPTIMUM MOISTURE

ASTM D 1557 (Modified)

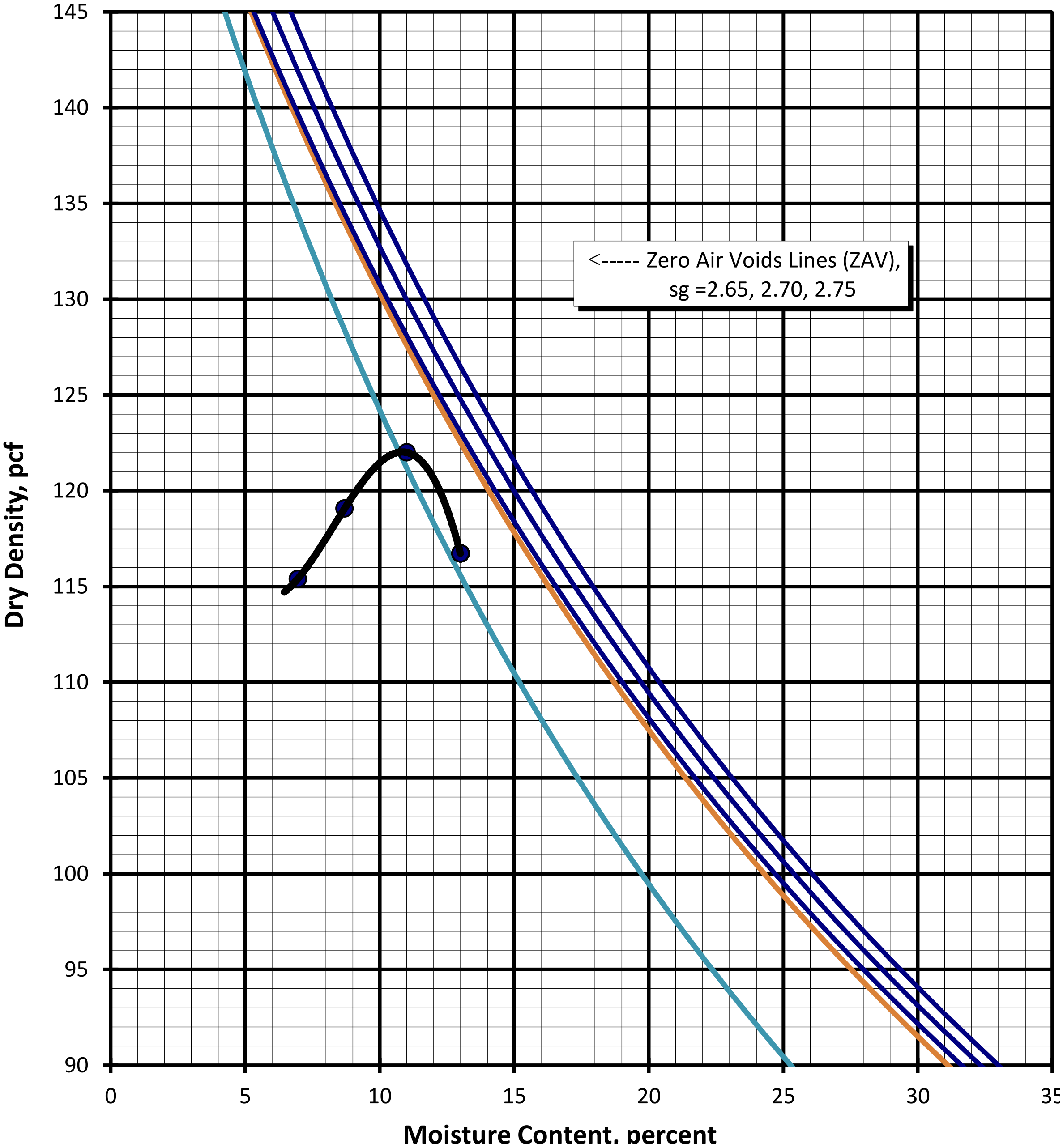
Job Name: Indio Maverik Store
Sample ID: B-1 @ 1-7'
Location: 0

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

Description: Silty Sand (SM)

Maximum Dry Density: **122 pcf**
Optimum Moisture: **10.8%**

Sieve Size	% Retained (Cumulative)
3/4"	0.0
3/8"	0.0
#4	1.0



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SOIL CHEMICAL ANALYSES

Job Name: Indio Maverik Store
Job No.: 306043-001
Sample ID:

Sample Location:	B-1 @ 1-9'	B-4 @ 2.5'
Resistivity (Units)		
as-received (ohm-cm)	>4,400,000	>4,400,000
saturated (ohm-cm)	880	1,440
pH	7.0	7.2
Electrical Conductivity (mS/cm)	0.34	0.24
Chemical Analyses		
Cations		
calcium Ca^{2+} (mg/kg)	132	113
magnesium Mg^{2+} (mg/kg)	10	5
sodium Na^{1+} (mg/kg)	123	81
potassium K^{1+} (mg/kg)	98	53
ammonium NH41+ (mg/kg)	ND	ND
Anions		
carbonate CO_3^{2-} (mg/kg)	ND	ND
bicarbonate HCO_3^{1-}(mg/kg)	143	70
fluoride F^{1-} (mg/kg)	ND	ND
chloride Cl^{1-} (mg/kg)	129	72
sulfate SO_4^{2-} (mg/kg)	146	136
nitrate NO31- (mg/kg)	56	41
phosphate PO_4^{3-} (mg/kg)	ND	ND
Other Tests		
sulfide S^{2-} (qual)	na	na
Redox (mV)	na	na

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

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

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

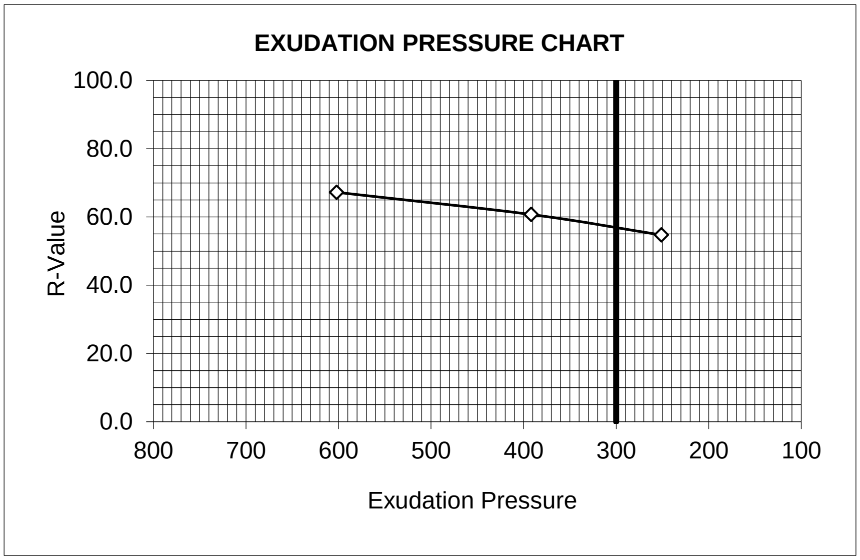
General Guidelines for Soil Corrosivity

Chemical Agent	Amount in Soil	Degree of Corrosivity
Soluble Sulfates[1]	0 -1,000 mg/Kg (ppm) [0-.1%]	Low
	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] (Saturated)	0- 900 ohm-cm	Very Severely Corrosive
	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.



JOB NAME: Indio Maverik Store
SAMPLE I. D.: B-5 @ 0-10'
SOIL DESCRIPTION: Sandy Silt (ML)

SPECIMEN NUMBER	D	E	F
EXUDATION PRESSURE	251	392	602
RESISTANCE VALUE	54.7	60.7	67.2
EXPANSION DIAL(0.0001")	0	0	0
EXPANSION PRESSURE (PSF)	0.0	0.0	0.0
% MOISTURE AT TEST	10.9	10.4	9.4
DRY DENSITY AT TEST	121.3	122.4	122.7

R-VALUE @ 300 PSI EXUDATION **57**

**Based on Traffic Index = 8.00 & Gravel Factor = 1.34*