Durban Development
106 Foster Avenue
Charlotte, NC 28203

**Geotechnical Engineering Report**
**Proposed Tractor Supply Company**
**Roy Rogers Drive East of Amargosa Road**
**Victorville, San Bernardino County, California**

July 26, 2024

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Durban Development
106 Foster Avenue
Charlotte, NC 28203

Attention: Mr. Stephen Knudsen

Subject: Geotechnical Engineering Report

Projects: **Proposed Tractor Supply Company**
Roy Rogers Drive East of Amargosa Road
Victorville, San Bernardino County, California

Earth Systems Pacific (Earth Systems) is pleased to present this Geotechnical Engineering Report for the proposed new Tractor Supply Company retail store located east of the existing WinCo Foods store east of Amargosa Road and north of Roy Rogers Drive in the city of Victorville, San Bernardino County, California. This report presents our findings and recommendations for site grading and foundation design, incorporating the information provided to our office. The site is suitable for the proposed development, provided the recommendations in this report are followed in design and construction. This report should stand as a whole, and no part of the report should be excerpted or used to the exclusion of any other part.

This report completes our scope of services in accordance with our proposal (BER-24-4-002) with an authorization date of May 17, 2024. Other geotechnical related services that may be required, such as plan reviews, responses to agency inquiries, and grading observation and testing are additional services and will be billed according to the Fee Schedule in effect at the time services are provided as per our authorized proposal. Unless requested in writing, the Client is responsible to distribute the report to the appropriate governing agency and other members of the design team. Please review the Limitations (Section 6) of this report as it is vital to the understanding of this report.

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

Respectfully submitted,
**EARTH SYSTEMS PACIFIC**

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**Geotechnical Engineering Report**
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**Victorville, San Bernardino County, California**

## Section 1.0
## INTRODUCTION

### 1.1 Project Description

This Geotechnical Engineering Report has been prepared for the proposed new Tractor Supply Company retail store located east of the existing WinCo store along Amargosa Road and Roy Rogers Drive in the city of Victorville, San Bernardino County, California, see Plate 1. Based on a client provided site plan, project improvements include approximately 44,000 square feet of indoor building area, receiving dock, vehicular parking, east-west access road, and a pylon sign along Roy Rogers Drive. Appurtenant site work is anticipated to include underground utilities, shallow retention basins, hardscape, driveway improvements, block walls, and landscaping. Preliminary grading and foundation plans, and appurtenant site work plans were not available at the time that this report was prepared. We have assumed site grades will be similar in elevation to the surrounding street grades (+-5 feet). The proposed site layout along with our exploration locations is presented on Plates 2a & 2b in Appendix A.

We have assumed the new buildings will be one to two-stories in extended wall height made of wood, metal, or light steel frame construction, founded on shallow permanent foundations. No below grade basement levels are anticipated. Column loads are anticipated not to exceed approximately 100 kips for spread footings and 3 kip/lf for continuous footing loads. Preliminary design loading was not provided by the structural engineer. If actual structural loading exceeds these assumed values, we will need to re-evaluate the given recommendations.

### 1.2 Site Description

The proposed new Tractor Supply Company retail store is located in the northeast quadrant of the intersection of Roy Rogers Drive and Amargosa Road. The project is located east of the existing WinCo store about 860 feet east of Amargosa Road in the city of Victorville, San Bernardino County, California. Based on the site plan provided and a San Bernardino GIS online service, the legal description of the land is identified as Accessor Pacel Number (APN) 3106-201-20-0000 and 3106-201-21-000 encompassing approximately 3.45 acres. Coordinates near the center of the proposed site are 34.5223°N/117.3274°W. Site elevations range from approximately 2,938 feet to 2,948 feet (msl) based on Google Earth. Access to the site is via Roy Rogers Drive; a paved improved street located adjacent to the southern boundary of the site.

Topographically, the site is generally flat to gently sloping ground with a descending cut slope in alluvium along the southeast margin of the site. The cut slope is about 6 to 8 feet high and graded at about a 2:1 (H:V) slope. The site is comprised of vacant unimproved land and is vegetated with a sparse covering of typical desert vegetation consisting of weeds, grasses, and bushes. Fill piles and disturbed soils are present in the eastern portion of the site. Localized areas of trash and debris exist. There was a soil stockpile offsite on the vacant lot located to the east. Drainage is by

sheet flow to the northeast. No evidence was noted of significant erosion at the time of our site visit.

### 1.3 Purpose and Scope of Services

The purpose for our services was to evaluate the site soil and geologic conditions at our exploration locations and to provide professional opinions and recommendations, from a geologic and geotechnical point of view, regarding the proposed development of the site. We understand that the proposed site improvements will be developed under the regulation of the current California Building Code (2022). The scope of services included:

Task 1 – Literature and Photograph Reviews

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

Task 2 – Utility Clearance, USA Dig Alert

Each of our proposed field exploration locations was located and marked in the field and cleared with known utility lines as identified by Underground Service Alert (USA), “Dig Alert”. Our exploration locations were located in the field by pacing with sighting from landmarks identified on the project site plan. Private utility locating was also performed for greater assurance of not encountering a buried utility at our boring locations.

Task 3 – Field Exploration

We evaluated the general subsurface conditions at the site by drilling nine small-diameter hollow-stem auger borings, to depths from approximately 6½ feet to 48½ feet below the existing ground surface (bgs). The field exploration also included a visual site reconnaissance of the project area and immediate surroundings. Plate 2a shows the approximate locations of the borings and planned improvements. A local geologic map is depicted as Plate 2b.

Task 4 – Laboratory Testing

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

Task 5 – Analysis and Report

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

- A description of the proposed project including a site plan showing the approximate boring locations;
- A description of the surface and of subsurface site conditions including groundwater conditions, as encountered in our recent field explorations.
- 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 and ASCE 7-16.
- Recommendations for foundation design including parameters for shallow foundations, pole foundations, and building pad and subgrade preparation if needed.
- Anticipated total and differential settlements for the recommended foundation system if needed.
- 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 slabs-on-grade (slabs and walkways), including recommendations for reducing the potential for moisture transmission through interior slabs.
- Roadway and parking asphalt concrete and Portland cement concrete recommendations for site access, parking areas, and truck aisles will be provided.
- Recommendations for collapsible or expansive soils (if applicable).
- A discussion of the corrosion potential of the near-surface soils encountered during our field exploration.
- An appendix, which will include a summary of the field exploration and laboratory testing program.

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

- A study for the presence or absence of wetlands, hazardous or toxic materials in the soil, surface water, groundwater, or air on, below, or adjacent to the subject property, nor exploration or delineation of past development or remaining structures/debris.
- The client did not direct Earth Systems to provide any service to investigate or detect the presence of moisture, mold, or other biological contaminates in or around any structure, or any service that was designed or intended to prevent or lower the risk or the occurrence of the amplification of the same. Client is hereby informed that mold is ubiquitous to the environment, with mold amplification occurring when building materials are impacted by moisture. Site conditions are outside of Earth Systems' control, and mold amplification will likely occur or continue to occur in the presence of moisture. As such, Earth Systems cannot and shall not be held responsible for the occurrence or recurrence of mold.

## Section 2.0
## METHODS OF EXPLORATION AND TESTING

### 2.1 Field Exploration

The subsurface exploration program consisted of advancing nine exploratory borings. The borings were drilled to depths ranging from approximately 6½ to 48½ feet bgs using a Mobile B-61 truck-mounted drill rig equipped with 6-inch hollow-stem augers provided by GP Drilling of Apple Valley, California. The borings were advanced to observe soil profiles and obtain samples for laboratory testing. The approximate boring locations are shown on Plate 2, in Appendix A.

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

Design parameters provided by Earth Systems in this report have considered an estimated 75% hammer efficiency based on data provided by the drilling subcontractor and limits per SP117A. The N-values (blow count) using the California sampler can be roughly correlated to SPT N-values using a conversion factor that may vary from about 0.5 to 0.7. In general, a conversion factor of approximately 0.63 from a study at the Port of Los Angeles (Zueger and McNeilan, 1998 per SP 117A) is considered satisfactory. A value of 0.63 was applied in our calculations for this project.

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

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

The boring logs present field blow counts per 6 inches of driven embedment (or portion thereof) for a total driven depth attempted of 18 inches. The blow counts on the logs are uncorrected (i.e. not corrected for overburden, sampling, etc.). Consequently, the user must correct the blow

counts per standard methodology if they are to be used for design and exercise judgment in interpreting soil characteristics, possibly resulting in soil descriptions that vary somewhat from the legend.

## 2.2 Laboratory Testing

Samples were reviewed along with field logs to select those that would be analyzed further. Those selected for laboratory testing include, but were not limited to, soils that would be exposed and those deemed to be within the influence of the proposed structures. Test results are presented in graphic and tabular form in Appendix B of this report. Testing was performed in general accordance with American Society for Testing and Materials (ASTM) or other appropriate test procedure. 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 (ASTM D 2937 & 2216).
- Maximum Dry Density/Optimum Moisture Content tests to evaluate the moisture-density relationship of typical soils encountered (ASTM D 1557).
- Particle Size Analysis to classify and evaluate soil composition. The gradation characteristics of selected samples were made by sieve analysis procedures (ASTM D 6913).
- Consolidation and Collapse Potential to evaluate the compressibility and hydroconsolidation (collapse) potential of the soil upon wetting (ASTM D 5333).
- Expansion Index tests to evaluate the expansive nature of the soil. The samples were surcharged under 144 pounds per square foot at moisture contents of near 50% saturation. Samples were then submerged in water for 24 hours and the amount of expansion was recorded with a dial indicator (ASTM D 4829).
- Screening Level Chemical Analyses (Soluble Sulfates and Chlorides (ASTM D 4327), pH (APHA 2320-B), and Electrical Resistivity/Conductivity (ASTM G 187) to evaluate the potential for adverse effects of the soil on concrete and steel.
- R-Value for pavement section analysis (CTM 301).

## Section 3.0
## DISCUSSION

### 3.1 Soil Conditions

The field exploration and literature review indicate that the site soils consist generally sporadic shallow fill or fill piles overlying native alluvial soils of sand with varying amounts of silt and silt with varying amounts of sand (Unified Soils Classification System symbols of SM, SP, and ML) to the maximum depth of exploration of 48½ feet below the ground surface. The underlying coarse-grained soils encountered were found to be loose to very dense. Groundwater was not encountered. Soils were generally "very low" in Expansion Index with isolated layers which could be defined as "low". The boring logs provided in Appendix A include more detailed descriptions of the soils encountered.

### 3.2 Groundwater

Groundwater or perched water was not encountered during our field exploration to the maximum depth of exploration of 48½ feet. Nearby State monitoring wells were researched for their recent and historic well readings. The following is a summary of our findings from three documented wells near the site.

- Well No. 05N04W08N001S is located approximately 0.5 miles to the north of the site. The surface elevation of the well is approximately 2,930 feet and groundwater elevation readings as measured from 2004 to 2014 varied from 2,699 to 2,730 feet.
- Well No. 05N04W16M002S is located approximately 0.6 miles east of the site. The surface elevation of this well is approximately 2,943 feet and the groundwater elevation readings as measured from 1957 were 2,787 feet.
- Well No. 05N05W13H002S is located approximately 1.2 miles to the west of the site. The surface elevation of this well is approximately 2,979 feet and the groundwater elevation readings as measured from 2006 were 2,759 feet.

Based on the above data, groundwater is not anticipated to be encountered during construction and is historically expected to be deeper than 150 feet at the site. Fluctuations of the groundwater level and localized zones of increased soil moisture content should be anticipated during and following the rainy season or from irrigation. Fluctuations in groundwater levels may occur due to variations in rainfall, regional climate, and other factors.

### 3.3 Collapse Potential

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

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

**Table 1**
**Collapse Potential Values**

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

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

**Table 2**
**Collapsible Soil Hazard Ranking System**

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

Based on recent collapse testing studies, we estimate there is a “moderate” collapse potential from soil layers between 2 to 15 ft bgs. The results of collapse potential tests performed on ten selected single ring samples from depths ranging from 2½ to 12½ feet below the ground surface indicated a collapse potential on the order of 1.5 to 2.9 percent. Collapse appears to be related to carbonate cementation as in-place densities were elevated and similar. Assuming the recommended grading is accomplished according to Section 5.1 of this report, we estimate the collapse potential is on the order of approximately 1.0 inch.

### 3.4 Expansive Soils

Expansive soils are characterized by their ability to undergo significant volume change (shrink or swell) due to variations in moisture content. Changes in soil moisture content can result from

rainfall, landscape irrigation, utility leakage, roof drainage, perched groundwater, drought, or other factors, and may cause unacceptable settlement or heave of structures, concrete slabs supported-on-grade, or pavements supported over these materials. Depending on the extent and location below finished subgrade, expansive soils can have a detrimental effect on structures. Based on our laboratory testing and experience with the project, the expansion potential of the on-site soils is generally "very low" for coarse grained shallow soils and "low" for shallow silts, as defined by ASTM D 4829 and the 2022 California Building Code.

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

### 3.5 Corrosion Potential

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

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

**Table 3**

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

In general, the lower the pH (the more acidic the environment), the higher the soil corrosivity will be with respect to ferrous structures and utilities. As soil pH increases above 7 (the neutral value), the soil is increasingly more alkaline and less corrosive to buried steel structures, due to protective surface films, which form on steel in high pH environments. A pH between 5 and 8.5 is generally considered relatively passive from a corrosion standpoint. High chloride levels tend to reduce soil resistivity and break down otherwise protective surface deposits, which can result in corrosion of buried steel or reinforced concrete structures. Soil resistivity is a measure of how easily electrical current flows through soils and is the most influential factor. Based on the findings of studies presented in ASTM STP 1013 titled "Effects of Soil Characteristics on Corrosion" (ASTM, 1989), the approximate relationship between soil resistivity and soil corrosivity was developed as shown in Table 4.

**Table 4**

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

Test results show a pH value of 8.4, chloride content of 5 ppm, sulfate content of 96 ppm and minimum resistivity of 3,680 Ohm-cm. Although Earth Systems does not practice corrosion engineering, the corrosion values from the soils tested are normally considered as being moderately corrosive to buried metals and as possessing a "negligible" exposure to sulfate attack for concrete as defined in American Concrete Institute (ACI, 2011) 318, Section 4.3. The results of chemical testing have been provided in Appendix B. The above values can potentially change based on several factors, such as importing soil from another job site and the quality of construction water used during grading and subsequent landscape irrigation.

### 3.6 Geologic Setting

Regional Geology: The project site is located at the southwestern edge of the Mojave Desert Geomorphic Province a broad interior region of isolated mountain ranges separated by desert plains and basins (see Plate 3). The Western Mojave Desert is a wedge-shaped structural block bounded on the north by the Garlock fault and along the southwest by the San Andreas rift zone. North of the Garlock fault are the Sierra Nevada and Tehachapi Mountains. Southwest of the San Andreas fault are the Transverse Ranges and coastal basins. Regional features of the western Mojave Desert include the alluvial-filled Antelope Valley, Victor Valley, San Gabriel and San Bernardino Mountains, the and the Mojave River. Victorville is within the high desert Victor Valley portion of the Western Mojave Desert, east of Palmdale and north of Cajon Pass. Lithologic units exposed in this area consist predominantly of Quaternary sediments, including both fluvial and alluvial fan deposits.

Local active faults are typically located along the margins of Victor Valley including the Frontal Fault system and San Andreas fault zone. Other faults within the Mojave Desert include the many faults within the Eastern California shear zone located north and east of the Victorville. The North Frontal Fault zone is located along the northern margin of the San Bernardino Mountains about 11 miles south of Victorville. The transform San Andreas rift zone, which forms the boundary between the North American and Pacific crustal plates, dominates the geology of the southwestern Mojave Desert and Victor Valley area, with the primary fault zone embedded within the adjacent San Gabriel and San Bernardino Mountains. The rift zone is an extensive zone of Holocene-active faults that extends from the Gulf of California to Cape Mendocino in northern California. The San Andreas fault, at its nearest point, is approximately 19 miles southwest of the site.

Faults associated with the Eastern California shear zone are located predominately within the eastern Mojave Desert and include the Helendale-Lockhart, Cleghorn, Lenwood, and Johnson Valley faults. Refer to Table A-1 in Appendix A for a listing of select local and regional Holocene-

active faults within approximately 50 miles of Victorville. No known Holocene-active faults exist within the project site boundaries.

Locally, the site is underlain by native older alluvial fan/river terrace deposits. These include interbedded gravel and sand associated with the deposition by the ancestral Mojave River

Local Geology: Victorville is located upon the generally flat or gently sloping alluvial and terrace deposits associated with alluvial fan deposition off the San Gabriel and San Bernardino Mountains and the Mojave River. Older alluvial terrace deposits associated with Pleistocene deposition by the Mojave River form elevated geomorphic surfaces generally above modern drainage grades. Locally, thick sequences of interbedded alluvium deposits underlie the site.

No active faults have been mapped across the site, with the nearest active faults being the North Frontal fault zone, Helendale, and San Andreas faults, located about 11, 13, and 19 miles respectively from the project. Native sediments underlying the site consist of fine- to coarse-grained sands with interbedded gravels and calcium carbonate (caliche), of fluvial and alluvial origin. Undocumented fills associated with adjacent site improvements are present, and generally undifferentiated from the native soils, except in stockpile areas where the fill is readily evident.

Regional Faulting: No active faults have been mapped within the project limits based upon local and regional select published geologic maps by the California Geological Survey (2010) or United States Geological Survey fault database (2006). The site is not located within a currently designated Alquist-Priolo Earthquake fault zone or City designated fault zone.

### 3.7 Geologic Hazards

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

Seismic Sources: Approximately 37 active faults or seismic zones lie within 50 miles of the project site (see Table A-1). The primary seismic hazard to the site is strong ground shaking from earthquakes along regional faults including the San Andreas fault, Garlock fault, Eastern California shear zone, and the many faults within the Los Angeles basin and Inland Empire. The Mojave and San Bernardino segments of the San Andreas fault are located approximately 19 miles southwest of the site.

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 the Western Mojave Desert as shown on California Geological Survey (CGS) maps (Jennings, 2010), but none are mapped within the Victorville city limits. 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.

Historic Seismicity: The site is located within an active seismic area in southern California where large numbers of earthquakes are recorded each year. Magnitudes that are above 6 and prior to accurate instrumental measurements (after 1933) are based on moment magnitudes ($M_W$). Magnitudes that are below 6 or earthquakes prior to 1933 are based on local magnitudes ($M_L$).

Approximately 37 earthquakes of magnitude 5.5 or greater have occurred within 60 miles of the site. Significant regional earthquakes include the 1812 Wrightwood, 1857 Fort Tejon, 1872 Owens Valley, 1952 Arvin-Tehachapi, 1971 San Fernando, 1992 Landers, and 1999 Hector Mine earthquakes. Strong ground motions were experienced with magnitudes ranging from approximately 5.9 to 7.8. Major historic earthquakes felt in the vicinity of Victorville have usually originated from faults located outside the area. With the exception of the 1812 Wrightwood and 1857 Fort Tejon earthquakes, these include the 1872 Owens Valley, 1952 Kern County, 1971 San Fernando, 1987 Whittier Narrows, 1992 Landers, 1994 Northridge, and 1999 Hector Mine earthquakes. Of recent note, the July 6, 2019 7.1 Ridgecrest earthquake was about 88 miles north-northwest of Victorville.

Historically, the San Andreas fault is responsible for two of the three great earthquakes experienced in California. These are the 1857 Fort Tejon and 1906 San Francisco earthquakes. Each event is credited with approximately 200 miles of surface rupture and horizontal displacements as great as 30 feet. Ground shaking was very intense and damage to man-made structures widespread. The 1857 rupture extended along the San Andreas fault from Parkfield to Cajon Pass and was felt throughout most of California. No significant earthquakes or fault movements have been attributed to this segment of the San Andreas fault since 1857. Prior to 1857, a strong earthquake that occurred in 1812 near Wrightwood, a small community in the eastern San Gabriel Mountains, originated on the San Andreas fault.

On March 26, 1872, the greatest recorded earthquake in the western United States, excluding Alaska, occurred along the Owens Valley fault near Lone Pine. The earthquake is estimated to have had a Richter magnitude of about 8.3, and significantly shook most of California.

In 1952, the White Wolf fault, located approximately 100 miles northwest of Victorville, was responsible for the Kern County earthquake. The earthquake registered 7.5 on the Richter Scale and did significant damage to the Bakersfield and Tehachapi areas. Considerable damage occurred to unreinforced masonry structures and to railroad tunnels located nearby.

The 1971 San Fernando earthquake resulted in extensive damage to structures in parts of San Fernando and the Santa Clarita Valley. The epicenter of the earthquake was located near Soledad Junction approximately 60 miles southwest of Victorville. Strong motion accelerographs recorded ground accelerations as high as 1.25 g at Pacoima Dam near the epicenter of the earthquake. Some structures designed in accordance with the Building Code in effect at the time were extensively damaged. Freeways, hospitals, schools, electrical facilities, water projects, and some residential structures sustained light to major damage as a result of severe ground shaking.

The major 1992 Landers/Big Bear earthquakes also shook the Victorville area. Damage in the Victor Valley was moderate. This earthquake was generated by a system of strike-slip faults in the mountain and desert areas about 55 miles east of Victorville. Structural damage and loss of life was limited primarily because of the remote area of occurrence.

The 1994 Northridge earthquake and related aftershocks significantly shook the Victorville area for 10 to 20 seconds. Like the Whittier Narrows earthquake, this event was produced by a buried thrust fault that underlies portions of the San Fernando Valley and the Santa Susana Mountains. The epicenter of the Northridge earthquake was approximately 70 miles southwest of Victorville.

Another major earthquake to affect the Victorville area was the 1999 Hector Mine earthquake. The epicenter of this magnitude 7.1 earthquake was approximately 60 miles east of northeast of Victorville. Approximately 26 miles of surface rupture occurred along the Lavic Lake fault and the central section of the Bullion fault.

The last major earthquake to affect the Victorville area was the July 2019 Ridgecrest earthquake. The epicenter of this magnitude 7.1 earthquake was approximately 88 miles north-northwest of the site. Surface rupture occurred near segments of the Little Lake fault zone near Ridgecrest.

Seismic intensity, a measure of earthquake effects, is typically described using the Modified Mercalli Scale. A description of damage based on the Modified Mercalli Scale is included in Table 1. For the above-mentioned earthquakes, Modified Mercalli Scale intensities ranged from IV to IX, with the 1857 Fort Tejon and 1992 Landers events generating the highest intensities.

From this analysis, it appears that the past maximum intensity in the Victorville area from major historical earthquakes due to regional faults is on the order of VIII-IX on the Modified Mercalli Scale. Anticipated intensities from a local 7+ magnitude earthquake along the nearby San Andreas fault is VIII-IX. Refer to Table A-1 in Appendix A for a list of active faults and their approximate distances from the site.

**Table 5**
**Modified Mercalli Intensity Scale of 1931[1], (1956 version)[2]**

Masonry A, B, C, D. To avoid ambiguity of language, the quality of masonry, brick or otherwise, is specified by the following lettering.

*Masonry A* Good workmanship, mortar, and design; reinforced, especially laterally and bound together by using steel, concrete, etc.; designed to resist lateral forces.

*Masonry B* Good workmanship and mortar; reinforced, but no designed in detail to resist lateral forces.

*Masonry C* Ordinary workmanship and mortar; no extreme weaknesses like failing to tie in at corners, but neither reinforced nor designed against horizontal forces.

*Masonry D* Weak materials, such as adobe; poor mortar; low standards of workmanship; weak horizontally.

| | |
|---|---|
| **I.** | **Not felt. Marginal and long-period effects of large earthquakes.** |
| **II.** | **Felt by persons at rest, on upper floors, or favorably placed.** |
| **III.** | **Felt indoors. Hanging objects swing. Vibration like passing of light trucks. Duration estimated. May not be recognized as an earthquake.** |
| **IV.** | **Hanging objects swing. Vibrations like passing of heavy trucks; or sensation of a jolt like a heavy ball striking the walls. Standing motor cars rock. Windows, dishes, doors rattle. Glasses clink. Crockery clashes. In the upper range of IV wooden walls and frame creak.** |
| **V** | **Felt outdoors; direction estimated. Sleepers wakened. Liquids disturbed, some spilled. Small unstable objects displaced or upset. Doors swing, close, open. Shutters, pictures move. Pendulum clocks stop, start, change rate.** |
| **VI.** | **Felt by all. Many frightened and run outdoors. Persons walk unsteadily. Windows, dishes, glassware broken. Knickknacks, books, etc., off shelves. Pictures off walls. Furniture moved or overturned. Weak plaster and masonry D cracked. Small bells ring (church, school). Trees, bushes shaken visibly, or heard to rustle.** |
| **VII.** | **Difficult to stand. Noticed by drivers of motor cars. Hanging objects quiver. Furniture broken. Damage to masonry D, including cracks. Weak chimneys broken at roof line. Fall of plaster, loose bricks, stones, tiles, cornices also unbraced parapets and architectural ornaments. Some cracks in masonry C. Waves on ponds; water turbid with mud. Small slides and caving in along sand or gravel banks. Large bells ring. Concrete irrigation ditches damaged.** |
| **VII.** | **Steering of motor cars affected. Damage to masonry C; partial collapse. Some damage to masonry B; none to masonry A. Fall of stucco and some masonry walls. Twisting, fall of chimneys, factory stacks, monuments, towers, elevated tanks. Frame houses moved on foundations if not bolted down; loose panel walls thrown out. Decayed piling broken off. Branches broken from trees. Changes in flow or temperature of springs and wells. Cracks in wet ground and on steep slopes.** |
| **IX.** | **General panic. Masonry D destroyed; masonry C heavily damaged, sometimes with complete collapse; masonry B seriously damaged. General damage to foundations. Frame structures, if not bolted, shifted off foundations Frames racked. Serious damage to reservoirs. Underground pipes broken. Conspicuous cracks in ground. In alluviated areas sand and mud ejected, earthquake fountains, sand craters.** |
| **X.** | **Most masonry and frame structures destroyed with their foundations. Some well-built wooden structures and bridges destroyed. Serious damage to dams, dikes, embankments. Large landslides. Water thrown on banks of canals, rivers, lakes, etc. Sand and mud shifted horizontally on beaches and flat land. Rails bent slightly.** |
| **XI.** | **Rails bent greatly. Underground pipelines completely out of service.** |
| **XII.** | **Damage nearly total. Large rock masses displaced. Lines of sight and level distorted. Objects thrown into the air.** |

[1]Original 1931 version in Wood, H.O., and Neumann, F., 1931, Modified Mercalli intensity scale of 1931: Seismological Society of America Bulletin, v. 53, no. 5, p. 979-987.

[2]1956 version prepared by Charles F. Richter, in Elementary Seismology, 1958, p. 137-138, W. H. Freeman & Co.

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. The recent Working Group of California Earthquake Probabilities (WGCEP, 2014) estimated a 43 percent conditional probability that a magnitude 6.7 or greater earthquake may occur in 30 years (2014 as base year) along the nearby Mojave segment of the San Andreas fault. The revised estimate for a 7.5+ magnitude earthquake along the local San Andreas fault is about 35%. For the nearby Frontal Fault zone, the probability of a 6.7 event in the same time period is estimated at about 2%.

The primary seismic risk at the site is a potential earthquake along the San Andreas fault that is about 19 miles southwest of 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 7.9 for the Mojave segment of the fault, which is being used for analysis. Historically, the San Andreas fault is responsible for two of the three great earthquakes experienced in the Southern California area. These are the 1812 Wrightwood and the 1857 Fort Tejon earthquakes. The 1857 rupture extended along the San Andreas fault from Parkfield to Cajon Pass and was felt throughout most of California. While the epicenter of this earthquake is assumed to be located near Parkfield, California, the fault rupture extended southeastward to the vicinity of Cajon Pass. No significant earthquakes or fault movements have been attributed to this segment of the San Andreas fault since 1857. A great earthquake that occurred in 1812 near Wrightwood in the eastern San Gabriel Mountains also originated on the San Andreas fault.

In general, Victorville is in a very high seismic risk area. Besides the San Andreas fault, other Holocene-active faults in southern California capable of generating significant seismic shaking include the San Jacinto fault near San Bernardino, the Garlock fault north of Mojave, and the many faults associated with the Eastern California shear zone. Approximately 5,400 earthquakes have been felt in Victorville since 1931 with a conditional probability of approximately 97% that a 5.0 magnitude earthquake will be felt in the next 50 years (Homefacts, 2024)

<u>Soil Liquefaction and Lateral Spreading</u>: 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, groundwater levels must be within 50 feet of the ground surface and the soils within the saturated zone must also be susceptible to liquefaction. Our exploration did not reveal a shallow groundwater table or a perched water table. Our research of nearby water well data and historic groundwater levels indicate both current and historic groundwater tables are greater than 50 feet below the ground surface. Since groundwater tables are greater than 50 feet bgs, and no perched layers were encountered, Earth Systems estimated the potential for liquefaction at the site is low.

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

In accordance with the above discussion, we used a site peak ground acceleration of ⅔ $PGA_M$ (where $PGA_M$ = 0.548) and an earthquake magnitude of 7.9 to evaluate dry seismic settlement potential. The design peak ground acceleration values were obtained from the SEAOC/OSHPD online application (https://seismicmaps.org/). Based upon methods presented by Tokimatsu and Seed (1987), the potential for seismically induced dry settlement of soils above the groundwater table for the full soil column height (48½ feet) was calculated in our deeper boring at the site and estimated to be 0.1 inches in boring B-1. Seismic settlement is based on post grading recommendations stated in Section 5.1.

Fissuring and Ground Subsidence: Subsidence is the gradual settling or sudden sinking of the Earth's surface due to subsurface movement of earth materials. It can result in the loss of aquifer storage, increased flooding, cracks and fissures at the land surface, damage to man-made structures, and associated economic costs. The site is not in an area of documented significant subsidence (SMAG 2024).

In areas of fairly uniform thickness of alluvium, fissures are thought to be the result of tensional stress near the ground surface and generally occur near the margins of the areas of maximum subsidence. Surface runoff and erosion of the incipient fissures augment the appearance and size of the fissures.

Changes in pumping regimes can affect localized groundwater depths, related cones of depression, and associated subsidence such that the prediction of where fissures might occur in the future is difficult. In the project area, groundwater depths remain fairly deep and we consider the current subsidence potential low. However, in the event of future nearby aggressive groundwater pumping and utilization, the occurrence of deep subsidence cannot be ruled out. Changes in regional groundwater pumping could result in areal subsidence. The risk of areal

subsidence in the future is more a function of whether groundwater recharge continues and/or over-drafting stops, than geologic processes, and therefore the future risk cannot be predicted or quantified from a geotechnical perspective.

Seismic Hazard Zones: This portion of San Bernardino County has been mapped for the California Seismic Hazard Mapping Act (Ca. PRC 2690 to 2699) for earthquake faults, liquefaction, and slope instability (CGS, 2005). These maps indicate the site is not currently within a designated hazard zone for these maps.

Site Acceleration and Seismic Coefficients: In developing site 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 evaluation at the site, the project site is underlain by some artificial fill overlying Quaternary older alluvial deposits. Based on the average field blowcount of 45 for the upper 50 feet of site soils (see Appendix A for output), the site classification for site response is Site Class D according to Table 20.3-1 of ASCE7-16. The D characterization is defined as a soil profile consisting of stiff soil with blowcounts between 15 to 50 blows per foot.

General Procedure: The Code seismic parameter $S_1$ is 0.447 g (< 0.75 g). The site is not within a designated Alquist-Priolo Earthquake Fault Zone or subject to bearing failure. Estimated shear wave velocities indicate a Site Class D. Seismic Design Category is D. Please see Section 5.8 for further discussion on the method to use for structural evaluation.

Earthquake horizontal peak ground motions of approximately 0.54 g is estimated based upon a 2% probability of exceedance in 50 years SEA 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 General Procedure seismic parameters given in Chapter 16 of the 2022 California Building Code are provided in Section 5.8 of this report.

### 3.7.1 Other Geologic Hazards

Other seismic hazards related to ground shaking include ground lurching, landslides, tsunamis, seiches, and flooding. Ground lurching is generally associated with fault rupture and liquefaction. As these two hazards are considered unlikely, it is Earth Systems opinion that the potential for ground lurching is also low. Due to the inland location of the site, hazards from tsunamis are considered nonexistent.

Landslides and Slope Instability: The site is relatively flat and proposed new slopes are anticipated to be less than 8 feet high and at shallow inclination as recommended. The site is not within a current Earthquake Induced Landslide Hazard Zones designated by the California Geological Survey. Therefore, potential hazards from slope instability, landslides, or debris flows are considered very low.

Flooding: The project site lies in an area designated as Zone X: “Areas determined to be outside the 0.2% annual chance floodplain.” This project area and Zone X are identified on FEMA Map No.: 06071C5815H, Panel 5815 of 9400, Map Revised August 28, 2008. Appropriate project civil design, construction, and maintenance can minimize the site sheet flooding potential.

Seiches: Seiching is defined as a periodic oscillation of liquid within a container or reservoir. Its period is determined by the resonant characteristics of the container, as controlled by its physical dimensions. Fluid storage tanks are not located near the project site, and no significant water storage facilities are currently located up gradient of the property. The site is not within a dam inundation zone. Therefore, it is Earth Systems opinion that significant hazards due to local reservoir failure (seiches) are currently considered very low.

**Section 4.0**
**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 our field exploration.

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

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

Geotechnical Constraints and Mitigation:

- The primary geologic hazard is moderate to severe ground shaking from earthquakes originating on regional faults. A major earthquake originating on the nearby segments of the San Andreas fault zone, and other regional faults would be the critical seismic event that may affect the site within the design life. Engineered design and earthquake-resistant construction increase safety and allow development within seismic areas.
- The underlying geologic condition for seismic design is Site Class D. The site is about 19 miles from a Type A seismic source 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.
- The site is not within a currently designated Alquist-Priolo Earthquake Fault Zone. No known faults cross the site. Therefore, the potential for surface fault rupture at the site is considered very low.
- The potential for liquefaction induced settlement is considered low for current groundwater conditions and for historic groundwater conditions for this project. The site is not within an area of significant documented areal subsidence distress.
- The potential for ground subsidence is considered low for this project under current conditions.
- The soils are susceptible to wind and water erosion.
- Other geologic hazards, including flooding, and landslides, are considered very low to low potential on this site.
- Based on current conditions, groundwater is not anticipated to be encountered during construction.
- Based on our laboratory testing, the Expansion Index of the onsite soil is “very low” as defined by ASTM D 4829, however silt zones do exist and if encountered could increase the expansion potential of the soil or fill. Soils should be graded to remove or blend silt soils to achieve a “very low” to “low” Expansion Index. Samples of soils should be tested or observed during grading to confirm or modify these findings.

- Laboratory testing of one sample showed potentially “moderate” corrosivity to buried metallic elements and “negligible” for sulfate exposure to concrete. See Section 3.5 for further information. Site soils should be reviewed by an engineer competent in corrosion evaluation and protection for all elements embedded or founded upon site soils.
- In our professional opinion, structure foundations can be supported on shallow foundations bearing on a zone of properly prepared and compacted soils placed as recommended in Section 5.1, or deep cast in drilled hole foundations for sign structures. The recommendations that follow are based on “very low” to “low” expansion category soils.

## Section 5.0
## RECOMMENDATIONS

### 5.1 Site Development and Grading

A representative of Earth Systems should observe site clearing, grading, and the bottoms of excavations before placing fill. Local variations in soil conditions may warrant increasing or decreasing the depth of recompaction and over-excavation. Proper geotechnical observation and testing during construction is imperative to allow the geotechnical engineer the opportunity to verify assumptions made during the design process, to verify that our geotechnical recommendations have been properly interpreted and implemented during construction and is required by the 2022 California Building Code. Preventative measures to reduce seasonal flooding and erosion should be incorporated into site grading plans. Dust control should also be implemented during construction. Site grading should be in strict compliance with the requirements of the local Air Quality Management District.

Observation of fill placement by the Geotechnical Engineer of Record should be in conformance with the 2022 California Building Code. The California Building Code requires full time observation by the geotechnical consultant during site grading (fill placement). Therefore, we recommend that Earth Systems be retained during the construction of the proposed improvements to provide testing and observe compliance with the design concepts and geotechnical recommendations, and to allow design changes in the event that subsurface conditions or methods of construction differ from those assumed while completing this study. Additionally, the California Building Code requires the testing agency to be employed by the project owner or representative (i.e., architect) to avoid a conflict of interest if employed by the contractor. Unless noted otherwise, grading should be performed in general accordance with the 2022 CBC.

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

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

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

Building Pad Preparation and Slope Grading: Due to the potentially collapsible soils in the upper soil profile, existing soils within the building pads, slab-on-grade, and foundation areas should be over-excavated a minimum of 6 feet below existing grade, below proposed grade, or 2 feet below the bottom of the footings or foundations, whichever is lower. The over-excavation should extend laterally a distance of 6 feet from the outside edge of the slab or footings, or the depth of overexcavation, whichever is deeper, where possible, as approved by the project geotechnical engineer. The exposed undisturbed subgrade bottom should be observed and tested by the geotechnical engineer or his representative to verify an in-place density of the subgrade is at or greater than 85% relative compaction per ASTM D 1557 or soils are firm (as determined by the geotechnical engineer or his representative). Deeper over-excavation may be recommended if the required in-place density is not achieved, soils are not firm, or undocumented fill exists.

The approved bottom of the over-excavation should then be scarified 12 inches; moisture conditioned to near optimum moisture content, and recompacted to at least 90% relative compaction (ASTM D 1557) prior to fill placement. Moisture conditioned and compacted engineered fill should then be placed to finish subgrade elevation in suitable compacted lifts. Compaction should be to at least 90% relative compaction prior to the placement of subsequent lifts with the final upper 12 inches of pad subgrade compacted to at least 95% relative compaction. Compaction should be verified by testing.

The adjacent property cut slope should be excavated and reconstructed as a buttress fill slope. The slope should be excavated a minimum of 5 feet below the slope face (normal to the face) and be reconstructed in a series of benches (2 horizontal feet for every 1 foot vertical) as compacted engineered fill. A key way should also be constructed at the slope toe. See Section J of the 2022 CBC and Section 5.5 within  for slope grading requirements.

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

Subgrade Preparation: In areas to receive fill not supporting structure, pavements, or hardscape the subgrade should be scarified; moisture conditioned and compacted to at least 90% relative compaction (ASTM D 1557) for a depth of 2 feet below existing grade or finished subgrade, whichever is deeper. Compaction should be verified by testing.

New Pavement, Hardscape, Area Preparation: In street, drive, permanent parking, and exterior hardscape areas, the subgrade should be over-excavated a minimum depth of 2 feet below existing grade or finish grade, or 2 feet below the slab bottom (whichever is deeper) for the applicable item described above. The excavation bottom should be scarified 8 inches, moisture conditioned to near or over optimum moisture content and be recompacted to at least 90% relative compaction. Engineered fill should then be moisture conditioned, placed in suitable lifts, and compacted to a minimum of 90% relative compaction prior to the placement of subsequent

lifts, with the upper 1 foot compacted to at least 95% relative compaction in parking and drive areas. Compacted fill should be placed to finish subgrade elevation.

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

Engineered Fill Soils: The existing native soils when processed appropriately are considered to be suitable for use as engineered fill. Much of the existing on-site soils are very low in Expansion Index and suitable for location under structures or hardscape after remedial grading.

Engineered fill should be generally free from expansive soil (Expansive in this case defined as Expansive Index (EI) greater than 50), vegetation, trash, large roots, overly wet and/or soft soil, clods larger than 3 inches, construction debris, oversized rock (greater than 6 inches) and other deleterious material as determined by the geotechnical engineer or his representative. Deleterious materials should be hauled offsite. Engineered fill soils should have a “very low” to “low” Expansion Index (Expansion Index less than 50).

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

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

Imported fill soils should be “very low” expansion potential granular soils meeting the USCS classifications of ML (as pre-approved by the geotechnical engineer), SM, SP-SM, or SW-SM with a maximum rock size of 3 inches and 5 to 35-percent passing the No. 200 sieve (unless otherwise approved by the geotechnical engineer). The geotechnical engineer should evaluate the import fill soils before hauling to the site. However, because of the potential variations within the borrow source, import soil will not be prequalified by Earth Systems. See also Section 5.3 for coarse grained backfill/fill recommendations and the use of filter fabric.

A program of compaction testing, including frequency and method of test, should be developed by the project geotechnical engineer at the time of grading. Acceptable methods of testing may include Nuclear methods such as those outlined in ASTM D 6938 (Standard Test Methods for In-Place Density and Water Content of Soil and Soil-Aggregate by Nuclear Methods). Alternative

methods may include methods outlined in ASTM D 1556 (Standard Test Method for Density and Unit Weight of Soil in Place by the Sand-Cone Method) or correlation probing with a hand probe.

Sand-Cement Slurry Backfill (Controlled Low Strength Material, CLSM): Where approved by the geotechnical engineer for use, sand-cement "slurry" (controlled low strength material (CLSM)), shall comply with the provisions of this report for the following:

1. Specifications for the preparation of the site prior to placement of the CLSM.
   - Remove any spoils, uncertified fill, loose material, debris and standing water from the existing excavations to a firm subgrade as approved by the geotechnical engineer or his representative.
   - A representative of Earth Systems should observe bottoms of excavations before placing CLSM.
   - CLSM shall not be placed on a sloping surface with a gradient steeper than 5:1 (H:V).
2. Specifications for the CLSM.
   - CLSM shall be ready-mixed by an ACI approved batch plant.
   - Use 2 to 3-sack sand-cement CLSM complying with Caltrans Standard Specification (2015) Section 19-3.02G Controlled Low-Strength Material or similar as approved by the geotechnical engineer. CLSM should have a low or non-shrinkage potential.
   - CLSM must have a 28-day compressive strength of at least 120 psi; however, it is cautioned that increased strength may not be excavatable.
3. Placement of CLSM After Item 1 above is Completed
   i. Place CLSM to the bottom of the foundation area or, in maximum 3-foot lifts (the minimum placement of CLSM should be 12 inches thickness), and allow the CLSM to cure at least for 24 hours or initial set, whichever is longer before placing a subsequent lift. Vibration and rodding of the entire lift should be accomplished during placement. Repeat as needed until the CLSM is within the bottom of the footing.
4. Laboratory or field test method(s) to be used to determine the compressive strength or bearing capacity of the CLSM.
   - Sample CLSM and fabricate test cylinder in accordance with ASTM D4832. Test for density in accordance with ASTM D6023 where there is concern. Test for compressive strength at 28 days in accordance with ASTM D4832.
5. Test methods for determining the acceptance of the CLSM in the field.
   - ASTM D4832-16, Preparation and Testing of Controlled Low Strength Material (CLSM) Test Cylinders.
   - ASTM D6023-07, Density (Unit Weight), Yield, Cement Content, and Air Content (Gravimetric) of Controlled Low-Strength Material (CLSM). At placement, wet density should be within 10% of the design wet unit weight.
6. Number and frequency of field tests required to determine compliance with items 4 and 5.

- Fabricate one set of 5 cylinders or cubes for compressive strength testing (average of three), and test for density for each 50 cubic yards or fraction thereof or as directed by the geotechnical engineer (GE) of record or the GE's representative.

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

## 5.2 Excavations and Shoring

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

Our site exploration and knowledge of the general area indicates there is a high potential for caving and sloughing of site excavations (over excavation areas, utilities, footings, etc.) due to potential dry to damp sandy soil conditions. Lateral bracing or appropriate cut slopes of 1.5:1 (horizontal/vertical) should be provided. No surcharge loads from stockpiled soils or construction materials should be allowed within a horizontal distance measured from the top of the excavation slope and equal to the depth of the excavation. Existing structures adjacent to excavation should be monitored and protected against caving, undermining, and settlement.

The borings were advanced with moderate effort within the existing on-site soils within the anticipated depths of excavation. Conventional equipment should be capable of performing shallow on-site excavations.

Excavations which parallel structures, pavements, or other flatwork, should be planned so that they do not extend into a plane having a downward slope of 1:1 (horizontal: vertical) from the bottom edge of the footings, pavements, or flatwork. Shoring or other excavation techniques may be required where these recommendations cannot be satisfied due to space limitations or foundation layout.

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. Trench boxes should not be placed below or within the pipe zone elevation as their removal may loosen compacted backfill. Positive trench shoring will likely be required (jacks and plates).

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

above the groundwater table should be designed to resist a uniform horizontal equivalent soil pressure as presented in the table below.

**Table 6**
**Temporary Cantilevered and Braced Shoring System Parameters**

| Equivalent Fluid Pressure pounds per cubic foot (pcf) | |
|---|---|
| **Cantilevered** | **Braced** |
| 31 | 50 |

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 (including construction equipment) placed adjacent to the shoring may be assumed to act as an additional uniform horizontal pressure against the shoring. Special cases such as combinations of slopes and shoring or other surcharge loads may require an increase in the design values recommended above. These conditions should be evaluated by the project geotechnical or shoring engineer on a case-by-case basis. Walls subjected to traffic loads should include a uniform surcharge load equivalent to at least 250 psf for auto or delivery truck (2 axle) traffic kept at least 3 feet from the back of the wall. Retaining walls with closer traffic or heavier non-construction HS20 type traffic loads should be designed for a 450 psf surcharge load. Construction equipment and aircraft loads should be considered on a case-by-case basis by the shoring designer.

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

Cantilevered shoring must extend to a sufficient depth below the excavation bottom to provide the required lateral resistance. We recommend required embedment depths be determined using methods for evaluating sheet pile walls and based on the principles of force and moment equilibrium. For this method, the allowable passive pressure against shoring, which extends below the level of excavation, may be assumed to be equivalent to a fluid weighing 275 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. Shoring should be sealed to prevent the piping of soil material and potential soil loss conditions which can cause settlement. The method of excavation and support is ultimately left to the contractor with guidance and restrictions provided by the designer and owner. We recommend that existing structures be monitored for both vertical and horizontal movement.

The 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 grading operations to monitor site conditions; substantiate proper use of materials; evaluate compaction operations; and verify that the recommendations contained herein are met.

### 5.3 Utility Trenches

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

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

Pipe Bedding: Pipeline subgrade should be compacted to a minimum of 90% relative compaction (ASTM D 1557) or be in a firm condition as evaluated by the geotechnical engineer or his representative for a depth of 6 inches below any bedding. Bedding, where specified by the manufacturer, material shall consist of sand 100 percent passing a No. 4 sieve and less than 5 percent fines (passing a No. 200 sieve) with a sand equivalent of 30 or more if jetted, and a fines content no greater than 15 percent with a sand equivalent of 30 or more if mechanically compacted, or as approved by the project inspector and geotechnical engineer. Bedding should be compacted to at least 90% relative compaction or firm (less than 2” insertion of a ½“-probe under typical 200 pound user weight).

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

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

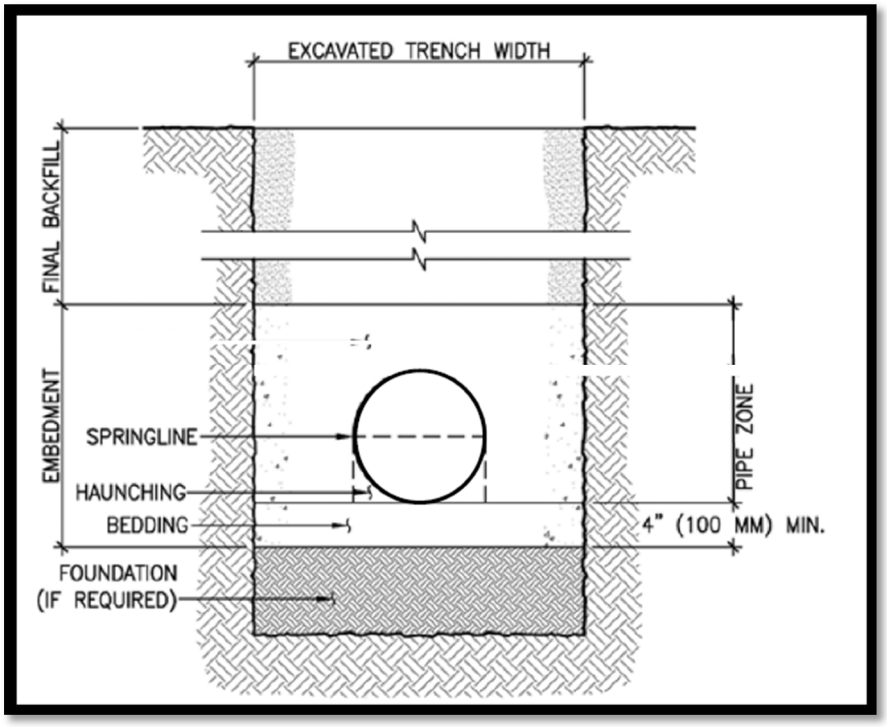


**Figure 1** Trench Cross Section

The native soil is suitable for use as trench zone (final backfill) and street zone (and manholes) backfill (from the top of pipe zone up to finished grade), provided it is free of significant organic or deleterious matter, have an Expansion Index of "very low" as evaluated by ASTM D 4829, and oversize materials (greater than 3" in maximum dimension) are removed. The final backfill material should be placed in maximum 8-inch lifts (loose) and be compacted to at least 90% relative compaction (ASTM D 1557) near its optimum moisture content for the trench zone and 95% for the street zone (upper 12 inches) below pavement or Portland cement concrete. Compaction should be verified by testing.

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

Alternatively, if the utility cannot accommodate the increased stress, or if compaction is difficult, we recommend the pipe or conduit be encased on all sides by at least 1 foot of 1-sack cement-sand slurry (at least 1 foot as measured from the top of pipe); Note: 2 sack slurry required if under structural improvements. Grouped conduit should be spaced adequately to allow soil compaction as required within or spaced a minimum of 2 inches edge to edge to allow adequate slurry inflow to encase the conduit(s). Slurry should be tamped, rodded, or vibrated to assure adequate flow. Backfill operations should be observed and tested to monitor compliance with these recommendations.

In general, coarse-grained sand and/or gap graded gravel (i.e. ¾-inch rock or pea-gravel, etc.) should not be used for pipe or trench zone backfill due to the potential for soil migration into the relatively large void spaces present in this type of material and water seepage along trenches

backfilled with coarse-grained sand and/or gravel. Water seepage or soil migration will cause settlement of the overlying soils. Rock backfill, where permitted by the geotechnical consultant, should be wrapped in filter fabric such as Mirafi 140N where there is contact with native or other fill soils.

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

**STRUCTURES**

In our professional opinion, site structure foundations, as described in Section 5.4.1 and 5.4.2 can be supported on shallow foundations bearing on a zone of properly prepared and compacted soils as described in Section 5.1.

## 5.4 Foundations

Footing design of widths, depths, and reinforcing are the responsibility of the Structural Engineer, considering the structural loading and the geotechnical parameters given in this report. A minimum footing depth of 18 inches (below lowest adjacent grade) should be maintained for continuous footings and 24 inches for isolated spread footings and considers a “very low” to “low” Expansion Index soil as per Section 5.1. Lowest adjacent grade is the lowest grade within 3 feet laterally of the footing edge. A representative of Earth Systems should observe foundation excavations to verify compaction (minimum 90% per ASTM D 1557) before placement of reinforcing steel or concrete. Loose soil or construction debris should be removed from footing excavations before placement of concrete. *All footing excavations should be probed for uniformity. Soft or loose zones should be excavated and recompacted to finish foundation bottom subgrade. The bottom of all foundations should be tested to confirm compaction effort and moisture contents as stated in Section 5.1 of this report are met.* The moisture contents should be at least the indicated moisture content 24 hours prior to and immediately prior to placing concrete for a depth of at least 12 inches below the foundation subgrade. If the moisture condition is less than indicated, it shall be brought up to or above the indicated moisture content.

5.4.1 Conventional Shallow 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 overburden may be neglected).

- Continuous wall foundations, 12-inch minimum and 24 inches maximum width and 18 inches minimum embedment (or per CBC, whichever is deeper), maximum 36 inches embedment (to bottom of footing):

  2,000 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 3,200 psf.

- Isolated pad foundations, 2 x 2-foot minimum in plan dimension, maximum 7 x 7 feet in plan dimension and 24-inch minimum depth, maximum 4 feet embedment:

  2,400 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 3,600 psf.

Maximum foundation sizes and allowable bearing pressures given above are based on a factor of safety of 3.0 due to Dead + sustained Live loads. In addition, settlements were evaluated to determine maximum allowable bearing stress and limitation of foundation sizing. 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. As the provided allowable bearing pressures are soil failure governed, a one-third (⅓) increase in the allowable bearing pressure can be used when calculating resistance to short term wind or seismic loads increase allowed if a lessor factor of safety is permitted (see Table 4).

The table below is based upon the above presented allowable, short term, and ultimate bearing pressures. Values may be increased by the provisions given above. Short Term allowable bearing may use the values presented below (based on Allowable Stress Design) or be based on Code mandated structural reductions, whichever is less. Ultimate bearing capacities consider a factor of safety of 1 (ASD design) and a safety factor of 2.25 on transient loads (1/3 increase). Ultimate bearing to soil failure depends on foundation size and type and could be much greater than 5,400 pounds per square foot (psf) for continuous footings and 6,600 psf for column footings (see Table 4).

**Table 7**
**Bearing Capacity Governed by Allowable Settlement - Continuous and Isolated Foundations**

| | Allowable Bearing Capacity (psf) (FS = 3) | Short Term (Wind/Seismic) (psf) (FS = 2.25) | Ultimate Bearing Capacity (psf) (FS = 1) |
|---|---|---|---|
| Continuous Foundations | 2,000 | 2,667 | 6,000 |
| Isolated Pad Foundations | 2,400 | 3,200 | 7,200 |

FS = Factor of Safety
# Allowable Bearing increase may be used as described above

Minimum Foundation Reinforcement: Minimum reinforcement should be provided by the structural engineer to accommodate the settlement potentials presented within.

A 1:1 (h:v) influence zone may be considered for footing design which extends downward and outward from the footing outside edges. Other structures or footings within this 1:1 projection should include surcharge loading and vice versa if other footings or structures project onto new footings.

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

Stepped foundations should be designed in accordance with the 2022 CBC. CBC 2022 and ACI Section 4.3, Table 4.3.1 should be followed for recommended cement type, water cement ratio, and compressive strength. Seismic Design Category for compressive strength determination is 'D'. Due to the potentially corrosive soils, a corrosion engineer should evaluate corrosion protection. Hot weather proportions should be used during high ambient heat days during placement and curing.

Expected Foundation and Fill Settlement: Estimated total static settlement (including collapse) should be less than 1½ inches, based on footings founded remediated or firm soils as recommended within. Differential static settlement between similarly loaded bearing members should be less than ¾ inch and may be designed using a distortion angle of 1:480, which meets typical geotechnical guidelines.

Seismic differential settlement could be on the order of 0.1 inches. Considering static and seismic differential settlements, we estimate a total differential settlement on the order of less than 1 inch. Expressed as an angular distortion, this is approximately 1:480.

Settlement calculations are presented in Appendix A. The actual settlement of large spread footings should be evaluated by the geotechnical engineer during the plan review stage based on the actual column loads to confirm or modify the settlement estimates presented. Settlement of compacted fill is calculated to be on the order of 0.20% of the total fill height.

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. 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 standard of practice for site developers to fully mitigate all anticipated earthquake induced hazards. This report is based on that understanding and Standard of Care.

According to literature form 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 or better. For all settlement

conditions excluding seismic settlement, the structure's maximum distortion angle should be 1:480 unless greater structural design is provided to accommodate the anticipated distortion.

Minimum Slope Setback for Shallow Foundations: Earth Systems recommends a minimum setback distance of +1 feet, where H is the height of the slope. 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 foundation to the toe of the slope. For pools and slopes steeper than 1(H):1(V), please contact Earth System for these setbacks with submittal of detailed information using plan form.

Pier Foundations: Where required, such as for lighting or light standards, or pylon signs, structures can be supported on cast-in-place piers, and the design be based on parameters presented within. Pier design, diameters, depths, and reinforcing are the responsibility of the project Structural Engineer, considering the structural loading and the geotechnical parameters given in this report. A representative of Earth Systems should observe foundation boreholes during excavation, and prior to placement of reinforcing steel or concrete (if any). Loose\soil, water, or construction debris should be removed from foundation excavations prior to placement of concrete.

Due to the small, estimated degree of settlement (less than 1 inch), it is estimated that the majority of load will be carried by side friction that will be mobilized on a pier. Therefore, the drilled pier should be designed as side friction pier without considering end bearing. Frictional and lateral resistance for uncased cast piles may be determined per CBC Section 1810.3.3.1.4 and .5 for silt (ML) soils as per Table 1806.2 and Section 1806.3. The lateral bearing pressure values (psf/ft) given in Table 1806.2 may be used for design. Additionally, where piers are constructed adjacent to the tops of slopes, there should be a minimum distance between the top of the slope and the closest edge of the pier of H/3, where 'H' is the height of the slope, otherwise a lateral resistance reduction must be applied. For piers for structures as described above, founded closer than a distance H/3 to the crest or within the slope area itself, the lateral resistance of the soil should be reduced by 30 percent. The above recommendations have considered slopes no steeper than 2:1 (horizontal:vertical). Steeper slopes will require additional analysis and may change the recommendations presented.

Since the piers are anticipated to be founded within compacted fill or native soil, potential axial load capacity reduction from negative skin friction/downdrag developed from seismic settlement of surrounding soils is anticipated to be nil. The allowable skin friction may be increased by ⅓ for short-term loading due to wind or seismic forces.

Where conditions have a pile located adjacent, but not connected, to a below grade wall, where piles are located at least 2.5 pile diameters from the nearest edge of pile to nearest wall edge considering axial pile loading and 5.5 pile diameters from the nearest edge of pile to nearest wall edge under lateral conditions for lateral loads towards the wall, there is no additional load that needs to be applied to the wall.

Where piers are proposed to be founded within areas protected from erosion by asphalt concrete pavement, the pile depth of embedment may be considered from the underlying soil surface. For piers founded in areas with native soil at the surface, an additional 1.5 feet should be added to the calculated pile embedment due to the potential effects of long-term surficial disturbance and erosion.

The piers are assumed to be a single isolated pile; however, additional piers may be incorporated to accommodate the design loads. If the piers are to be grouped, then the center-to-center spacing between piers should be at least 3 times the pier diameter. With such spacing, no group capacity reduction is considered necessary for the axial or lateral loading conditions. If grouped piers with spacing less than 3 times the pier diameter under axial conditions and 6 times the pier diameter under lateral conditions are required, or Lpile or other design will be utilized, our office can evaluate these conditions on a case-by-case basis.

We recommend a minimum Factor of Safety of 3 be applied to the axial downward load capacities calculated from the data presented. The allowable uplift capacity Factor of Safety should be at least 2 and may include the dead weight of the foundation concrete or driven pile.

Pier Installation: The onsite soils are expected to be excavatable with conventional power drilling. Drilled piers should have a minimum 3 inches of clearance between the embedded post and the soil side wall to allow for adequate placement and flow of concrete and be a maximum of 8 feet in depth, otherwise pre-approval will be required from the geotechnical engineer. To avoid a reduction in lateral and uplift resistance caused by variable subsurface conditions, we recommend that drawings instruct the contractor to notify the engineer if subsurface conditions are significantly different than encountered in our borings. Under these circumstances, it may be necessary to adjust the overall length of the pier.

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

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

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

Concrete Quality and Placement for Drilled Piers: Concrete placement should begin within four hours after completion of drilling. Concrete placement should be continuous without interruption and at such a rate that fresh concrete will not be deposited on concrete that has hardened sufficiently to form cold joints or planes of weakness.

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

Pier Inspections: Pier capacity is greatly dependent on the soil conditions at the location of the pier and upon contractor means and methods of placement. It is recommended that pier driving or drilled pier drilling operations and concrete placement be performed in the continuous presence of the geotechnical consultant or his representative to confirm that suitable materials for pier support are penetrated, that the dimensions of the installed piers meet the design dimensions, and that the installation has been performed as specified by the 2022 California Building Code. Observation during drilling or driving is required by the 2022 California Building Code on a full-time basis by the geotechnical engineer or his representative. If subsurface conditions noted during drilled pier installation are significantly different than those encountered in our borings, it may be necessary to adjust the overall length of the pier.

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

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

The contractor should review the logs of this report and perform their own assessment of potential construction difficulties. However, the logs are only an indicator of potential site conditions, and the contractor is ultimately left to their own experience in regard to potential site conditions.

Expected Pier Settlement: Based on the recommendation for compacted fill, the estimated total settlement should be less than one inch, based on piers founded on firm soils as recommended and the loads presented in Section 1.1. Differential total settlement between similarly loaded bearing members should be less than 1 inch over a distance of approximately 40 feet, expressed in a post-construction angular distortion ratio of 1:480 or better. At a minimum, foundations

should be designed by the structural engineer for the specific static and seismic settlement conditions presented.

### 5.5 Slope Construction

Slopes are not generally proposed for this project; however, minor slopes (less than 8 feet in height) may be constructed as per Section 5.1. Compacted fill slopes protected against erosion (per approved methods such as significant planting, facing, or erosion blankets, etc.) may be constructed at 2:1 (horizontal: vertical) or flatter inclinations. Unprotected slopes with exposed native soils or compacted fill at the surface should be expected to require repair after heavy nuisance or storm runoff due to significant erosion. Slope recommendations may change pending a more in-depth geotechnical evaluation once design plans are developed. Slopes used as nuisance or storm drainage channel slopes should be no steeper than 3:1 and protected with heavy 12" minimum Rip-Rap.

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

Surficial Slope Failures: All slopes will be exposed to weathering, resulting in decomposition of surficial earth materials, thus potentially reducing shear strength properties of the surficial soils. In addition, these slopes become increasingly susceptible to rodent burrowing. As these slopes deteriorate, they can be expected to become susceptible to surficial instability such as soil slumps, erosion, soil creep, and debris flows. Development areas immediately adjacent to ascending or descending slopes should address future surficial sloughing of soil material and erosion. Such measures may include debris fences, slope facing, catchment areas or walls, diversion ditches or berms, soil planting, velocity reducers or other techniques to contain soil material away from developed areas and reduce erosion. Additionally, foundations should be set back at least 5 feet from the edge of slope or as per the 2022 CBC, whichever is greater.

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

Slope faces are highly erodible even if compacted and will gradually erode and move down slope presenting maintenance issues and debris deposited in drainage devices and flatwork areas. The

minimum material necessary to support landscaping should be specified by the landscape consultant (typically less than 6 inches).

### 5.6 Slabs-on-Grade

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

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

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

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

**Table 8**
**Percent Passing Sieve Size**

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

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

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

***The following minimum slab recommendations are intended to address geotechnical concerns such as potential variations of the subgrade and are not to be construed as superseding any structural design. A design engineer should be retained to provide building specific systems to handle subgrade moisture to ensure compliance with SB800 with regards to moisture and moisture vapor.***

Slab Thickness and Reinforcement: Structure slabs should be a minimum of 5 inches in actual thickness and be reinforced with # 3 bars at 18 inches on center both ways. Flatwork slabs should be a minimum of 4 inches thick and be jointed to reduce cracking potential. Drive slabs should per Earth Systems trench and pavement report under separate cover. Slabs for forklift traffic should be designed by the structural engineer for these specific loading conditions. Slabs in contact with earth should use closer joints to control cracking or be thickened to allow adequate earth to rebar clearance. Reinforcing bars should extend at least 40 bar diameters into the footings and slabs. Concrete slabs-on-grade and flatwork should be supported by compacted and moisture conditioned soil placed in accordance with this report. If slabs are structural, they should be designed for the specific settlement conditions presented within.

Slab thickness and reinforcement of slabs-on-grade are contingent on the recommendations of the structural engineer or architect and the Expansion Index of the supporting soil. Based upon our findings, a modulus of subgrade reaction of approximately 100 pounds per cubic inch can be used in concrete lightly loaded (not mat) slab design for the expected compacted subgrade. Mat

slab design will require differing modulus values. ACI Section 4.3, Table 4.3.1 should be followed for recommended cement type, water cement ratio, and compressive strength.

If heavily loaded flatwork is proposed (forklift drive areas, heavy racking, etc.), the actual thickness should be designed by the structural engineer utilizing techniques of the American Concrete Institute (ACI) and may be greater than 5 inches in thickness. Concrete floor slabs may either be monolithically placed with the foundations or doweled (No. 4 bar embedded at least 40 bar diameters) after footing placement. The thickness and reinforcing given are not intended to supersede any structural requirements provided by the structural engineer. The project architect or concrete inspector should continually observe all reinforcing steel in slabs during placement of concrete to check for proper location within the slab. The minimum concrete rebar cover should be as per the project architect or structural engineer.

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

Construction (cold) joints, especially at door threshold exterior to interior slab transitions, should be reinforced with thickened butt joints with ¾-inch dowels at 12 inches on center embedded per ACI or a thickened keyed-joint to resist vertical deflection at the joint. All control joints in exterior flatwork should be sealed to reduce the potential of moisture or foreign material intrusion. These procedures will reduce the potential for randomly oriented cracks, but may not prevent them from occurring.

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

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

### 5.7 Retaining Walls and Lateral Earth Pressures

Retaining Walls: Walls which are restrained at the top such as retaining wall returns, below-grade walls and top of walls tied to floor slabs should be designed with “at rest” earth pressures. Retaining walls, free to tilt at the top 0.2 percent of the wall height, should be designed for “active” earth pressures.

The following list presents lateral earth pressures for use in wall design. The values are given as equivalent fluid pressures ***without*** surcharge loads or hydrostatic pressure. Clay or silt, non-free

draining soils are not suitable for wall backfill. Silty sand material may be used for backfill or free draining material should be imported as wall backfill within the zone of 45 degrees projected up from the wall bottom. For native or import free draining material, active and restrained walls equivalent fluid pressures are as follows:

- Retaining walls should be designed for an active soil pressure equivalent to a fluid density of 31 pcf. The active lateral earth pressures are for slope conditions similar to those on-site and backfills using the on-site soils on 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 50 pcf. Walls retaining sloped backfill of 3:1 or 2:1 should increase the presented fluid pressures by 5 pcf and 8 pcf, respectively. Earth Pressures for walls retaining other sloping backfill above the wall can be provided on a case-by-case basis by the geotechnical engineer. The lateral earth pressure values for level backfill are provided for walls backfilled with drainage materials and "very low" expansive soils.
- In addition to the active or at rest soil pressure, the proposed wall structures should be designed (where not excepted) to include forces from dynamic (seismic) earth pressure. Dynamic pressures are <u>additive</u> to active and at-rest earth pressures (and utilize their respective standard load distribution) and should be considered as an additional 13 pcf for flexible walls, and 25 pcf for rigid walls (Al Atik and Sitar, 2010, Mikola and Sitar, 2013). Seismic pressures are based on $PGA_M$ of 0.548 g (ASCE7-16), Friction Soil Angle of 36$^o$, and a maximum total moist fill density of 120 pcf.
- 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 rock, or surcharge loads from nearby footings can create larger lateral pressures. Should any walls be considered for retaining sloped backfill (or rock) 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 as described in Section 5.2. Retaining walls should be designed with a minimum factor of safety of 1.5.
- A backdrain or an equivalent system of backfill drainage should be incorporated into the retaining wall design, whereby the collected water is conveyed to an approved point of discharge. 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 or commercial drainage panels behind the wall which lead to an approved outlet system. 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 the retaining wall.
- 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 (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. Foundation subgrade preparation should be as specified in Section 5.1.

**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/slurry, and by ½ of the calculated passive soil pressure against the foundations (when combined). An allowable coefficient of friction 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
- Allowable passive pressure may be taken as equivalent to the pressure exerted by a fluid weighing 300 pounds per cubic foot (pcf). 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. The soils pressures presented have considered onsite fill soils. Testing or observation should be performed during grading by the soils engineer or his representative to confirm or revise the presented values.
- Passive resistance for thrust blocks bearing against firm natural soil or properly compacted backfill can be calculated using an equivalent fluid pressure of 300 pcf. The maximum passive resistance should not exceed 2,000 psf.
- Friction and soil pressure values (resistance) presented above are considered to have a factor of safety of 1.5 in relation to ultimate values (factor of safety = 1) and may be combined. The above values are not permitted to be increased by 1/3 due to short term loads such as wind or seismic forces.
- Construction employing poles or posts (i.e. lamp posts) may utilize design methods presented in Section 1807 of the CBC for Silt (ML) material class. The lateral bearing pressure values (psf/ft) given in Table 1806.2 may be increased by a factor of 1.5 for design if constructed in areas graded per Section 5:1; otherwise no increase is permitted.
- 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.

### 5.8 Seismic Design Criteria

This site is subject to strong ground shaking due to potential fault movements along regional faults including the San Andreas fault. 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. The site is not within an Alquist-Priolo or other hazard zone, is Site Class D, and the spectral acceleration $S_1$ is less than 0.75.

**Note to the Structural Engineer:** the seismic coefficients in Table 9, 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 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 applies to the structures applicable to this report's Table 5 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 34.5223°N and Longitude 117.3274°W. The structural design engineer should use the most conservative results based of the specific building design and spectral response.

**Table 9**
**2022 CBC (ASCE 7-16) Seismic Parameters**

| | |
|---|---|
| Site Class: | D |
| Risk Category: | II |
| Seismic Design Category | D |
| **Maximum Considered Earthquake [MCE] Ground Motion** | |
| Short Period Spectral Response $S_s$: | 1.155 g |
| 1 second Spectral Response, $S_1$: | 0.447 g |
| **Code Design Earthquake Ground Motion** | |
| $F_a$ | 1.04 |
| $F_v$ | 1.85 |
| $F_{PGA}$ | 1.103 |
| $S_{MS}$ | 1.199 g |
| $S_{M1\ (unmodified)}$ | 0.828 g |
| $S_{M1\ (ASCE\ 7\text{-}16\ Supplement\ 3\ Modified)}$ | 1.242 g |
| Short Period Spectral Response, $S_{DS}$ | 0.799 g |
| 1 second Spectral Response, $S_{D1\ (unmodified)}$ | 0.552 g |
| 1 second Spectral Response, $S_{D1\ (ASCE\ 7\text{-}16\ Supplement\ 3\ Modified)}$ | 0.828 g |
| Peak Ground Acceleration ($PGA_M$) Eq 11.8-1 | 0.548 g |

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

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.

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. Note that all of southern California in general is

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 standard of practice 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 development. This report is based on that understanding and current Standard of Care.

### 5.9 Driveways and Parking Areas

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

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

Traffic and Parking Areas: Pavement sections presented in the following table for automobile type traffic areas and are based on an assumed R-value and current Caltrans design procedures. Truck and Main Drive drives are also included. Traffic Indices (TI) of 5, 7, and 9 were used to facilitate the design of asphalt concrete pavements for parking and main drives, including fire lanes. The fire lane calculation assumed a conservative traffic flow of one fire truck per day entering and exiting the site on the same path (20 year life cycle), and a maximum loading of an 88,000 lb Tandem Axle apparatus (approximate 12,000 lb front axle load and two 34,000 lb rear axles loads) which is based upon the *Emergency Vehicle Size and Weight Regulation Guideline*, dated November 22, 2011, prepared by the Fire Apparatus Manufacturers' Association.

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

**Table 10**
**Preliminary Flexible Pavement Section Recommendations**
**On-site/Interior Automobile Drive Areas**

R-Value of Subgrade Soils - 39 (Tested) Design Method – CALTRANS

| Traffic Index (Assumed)* | Pavement Use | Flexible Pavements** # | |
|---|---|---|---|
| | | Asphaltic Concrete Thickness (inches) | Aggregate Base Thickness (inches) |
| 5 | Parking Areas & Fire Lanes*** | 3.0 | 4.0 |
| 7 | Parking Lot Main Drive Areas | 4.0 | 7.0 |
| 9 | Tractor Trailer Access and Main Drive Road/Entrance-Exit Roads | 5.5 | 9.5 |

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

**Pavement Sections were calculated using Caltrans methodology.

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

# City of Victorville Street Standards should be referred to where required.

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 and Trash Truck approach pads). This is due to rutting and shoving, as well as cracking, 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 11**
**Preliminary Portland Cement Concrete Pavement Sections**

| Area | Minimum Pavement PCC Thickness (inches) | Minimum 28 Day Flexural Strength (psi) | Concrete Compressive Strength (psi) |
|---|---|---|---|
| Tractor Trailer Drives and Trash Truck Access/Loading Pads (Traffic Category D, ADTT =700) | 8.0 | 550 | 3,250 |

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

- Subsequent to utility installation, the entire pavement (including PCC) final subgrade should be scarified 12 inches, moisture conditioned to near optimum moisture content, and compacted to a minimum 95% relative compaction immediately prior (within a few days) to the placement and compaction of aggregate base to re-establish proper moisture content and compaction in site soils.
- Subgrade soils and aggregate base should be in a stable, non-pumping condition at the time of placement and compaction. Exposed subgrades should be proof-rolled to verify the absence of soft or unstable zones.
- Aggregate base materials should be compacted at near optimum moisture content to at least 95 percent relative compaction (ASTM D 1557) and should conform to Caltrans Class II criteria. Standard Specifications for Public Works Construction "Greenbook: standards (Crushed Aggregate Base Class) may be used in lieu of Caltrans. Compaction efforts should include rubber tire proof-rolling of the aggregate base with heavy compaction-specific equipment (i.e. fully loaded water trucks).
- All concrete curbs separating pavement from landscaped areas should extend at least 6 inches into the subgrade soils to reduce the potential for movement of moisture into the aggregate base layer (this reduces the risk of pavement failures due to subsurface water originating from landscaped areas).
- 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-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.
- Asphaltic concrete should be ½-in. or ¾-in. grading and compacted to a minimum of 95% of the 75-below Marshall density (ASTM D 1559) or equivalent.
- Within the structural pavement section areas, positive drainage (both surface and subsurface) should be provided. In no instance should water be allowed to pond on the pavement. Roadway performance depends greatly on how well runoff water drains from the site. This drainage should be maintained both during construction and over the entire life of the project.
- Proper methods, such as hot-sealing or caulking, should be employed to limit water infiltration into the pavement base course and/or subgrade at construction/expansion joints and/or between existing and reconstructed asphalt concrete sections (if any). Water infiltration could lead to premature pavement failure.
- To reduce the potential for detrimental settlement, excess soil material, and/or fill material removed during any footing or utility trench excavation, should not be spread or placed over compacted finished grade soils unless subsequently compacted to at least 90% of the maximum dry unit weight, as evaluated by ASTM D 1557 test procedure, at near optimum moisture content, or 95% if placed under areas designated for pavement.
- Where new roadways will be installed against existing roadways, the repaired asphalt concrete pavement section should be designed and constructed to have at least the pavement and aggregate base section as the original pavement section thickness (for both AC and base) or upon the newly calculated pavement sections presented within, whichever is greater.
- **Pavement designs assume that heavy construction traffic will not be allowed on base cap or finished pavement sections**. **Failure to heed these recommendations will subject the pavement to increased potential for random cracking, distress, premature aging and failure.**

### 5.10 Surface and Subsurface Site Drainage and Maintenance

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

- Water control and conveyance is a critical aspect of project design. It is highly recommended that landscape irrigation or other sources of water be collected and conducted to an approved drainage device. Landscaping grades should be lowered and sloped such that water drains to appropriate collection and disposal areas. All runoff water should be controlled, collected, and drained into proper drain outlets. Control methods may include curbing, ribbon gutters, landscape vertical barriers in planters, 'V' ditches, or other suitable containment and redirection devices.
- It is highly recommended that landscape irrigation or other sources of water be collected and conducted to an approved drainage device. Site drainage should be devised such that runoff should be directed away from the tops of all graded slopes. Water should not freely flow over slopes or retaining wall faces. Diversion and conveyance structures which can accommodate water and eroded soil should be constructed at the tops and toes of all slopes. Lined swales at the top and bottom of slopes, and at the top of retaining walls are recommended.
- In no instance should water be allowed to flow or pond against structures, slabs or foundations or flow over unprotected slope faces. Adequate provisions should be employed to control and limit moisture changes in the subgrade beneath foundations or structures to reduce the potential for soil saturation. Landscape borders should not act as traps for water within landscape areas. Potential sources of water such as piping, drains, over-spray broken sprinklers, etc., should be frequently examined. Any such leakage, over-spray, or plugging should be immediately repaired.
- 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 saturation and standing water which increase the potential for soil expansion and failure. The irrigation system designer should consider these conditions in their design and control irrigation accordingly.
- Maintenance of drainage systems and infiltration structures can be the most critical element in determining the success of a design. They must be protected and maintained from sediment-laden water both during and after construction to prevent clogging of the surficial soils any filter medium. The potential for clogging can be reduced by pre-treating structure inflow through the installation of maintainable forebays, biofilters, or sedimentation chambers. In addition, sediment, leaves, and debris must be removed from inlets and traps on a regular basis. Since these and other factors (such as varying soil conditions) may affect the rate of water infiltration, it is imperative to apply a conservative factor of safety [FOS] to unfactored Basic Percolation/Infiltration Rates to provide a reliable basis for design. In order to account not only for the unknown factors above but also for changes of conditions during

the use of the structures such as potential clogging effects due to washing in of soil fines, a FOS between 3 and 12 should be applied to lower infiltration rates as per LA County guidelines.

- The factor of safety should be selected by the project drainage engineer and may be dependent on agency guidelines and the presence of testing, filters, and sedimentation structures. If these measures are provided, the factor of safety can be reduced.
- The drainage pattern should be established at the time of final grading and maintained throughout the life of the project. Additionally, drainage structures should be maintained (including the de-clogging of piping, basin bottom scarification, soil crust removal, etc.) throughout their design life. Maintenance of these structures should be incorporated into the facility operation and maintenance manual. Structural performance is dependent on many drainage-related factors such as landscaping, irrigation, lateral drainage patterns and other improvements.
- Drywells and any gravel fill should be separated from the native soils with filter fabric such as Mirafi 140N or directly similar (similarly approved by the geotechnical engineer).
- <u>Drywells and infiltration underground chambers/structures or basins/trenches should be located at least 50 feet from any structure or be setback outside a projection of 1:1 (h:v) from the projection upward from the bottom of any foundation</u>, whichever is greater.

## Section 6.0
## LIMITATIONS AND ADDITIONAL SERVICES

### 6.1 Uniformity of Conditions and Limitations

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

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

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

Findings of this report are valid as of the issued date of the report. However, changes in conditions of a property can occur with passage of time, whether they are from natural processes or works of man, on this or adjoining properties. In addition, changes in applicable standards occur, whether they result from legislation or broadening of knowledge. Accordingly, findings of

this report may be invalidated wholly or partially by changes outside our control. Therefore, this report is subject to review and should not be relied upon after a period of one year.

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

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

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

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

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

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

### 6.2 Additional Services

This report is based on the assumption that an adequate program of client consultation, construction monitoring, and testing will be performed during the final design and construction phases to check compliance with these recommendations. Maintaining Earth Systems as the geotechnical consultant from beginning to end of the project will provide continuity of services. ***The geotechnical engineering firm providing tests and observations shall assume the responsibility of Geotechnical Engineer of Record.***

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

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

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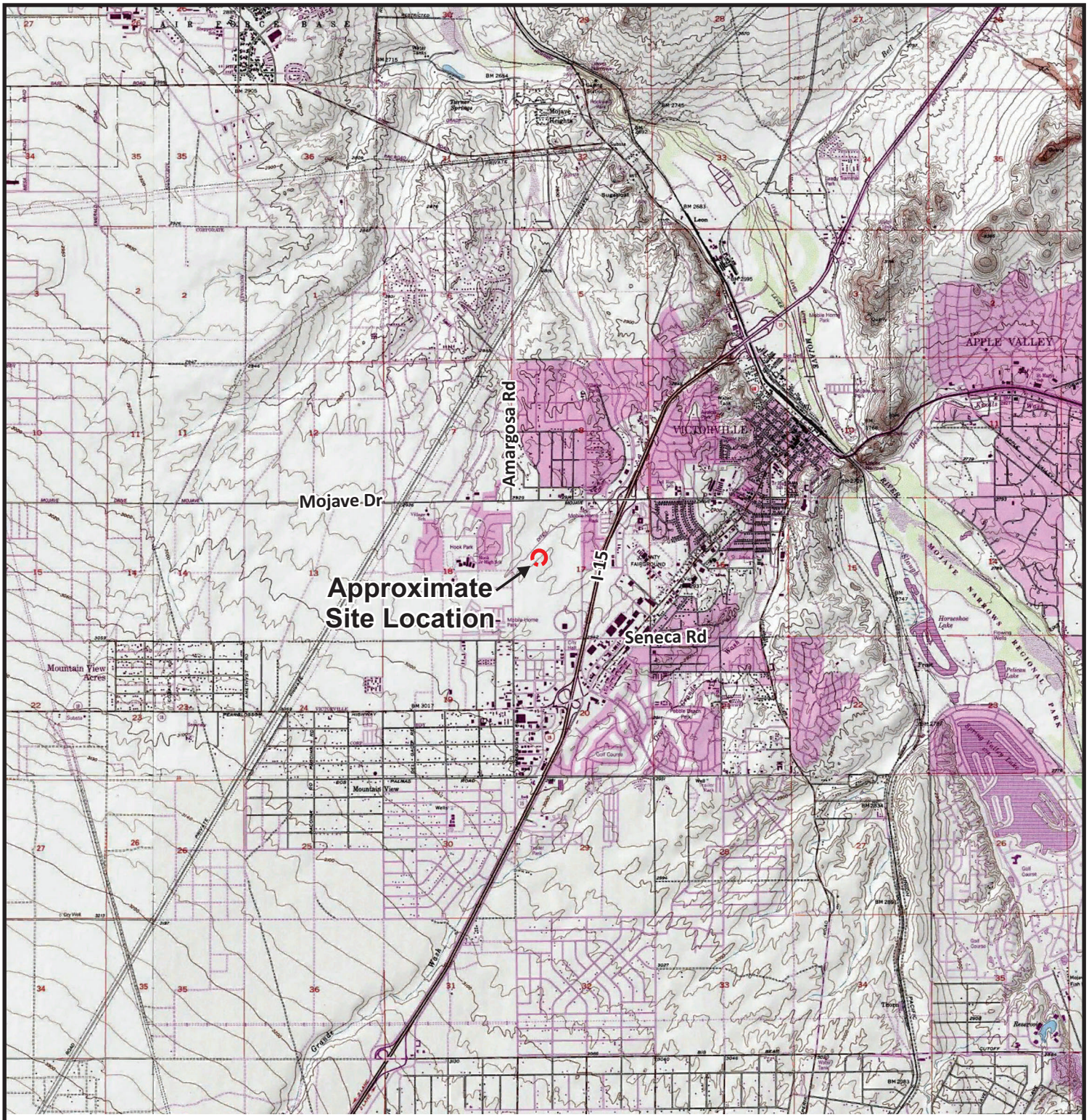
**APPENDIX A**

Plate 1 – Site Location Map
Plate 2a – Exploration Location Map
Plate 2b – Local Geologic Map
Plate 3 – Regional Geologic Map
Table A-1 Fault Parameters
Table A-2 Historic Earthquakes
Table A-3 2022 California Building Code Seismic - General Procedure
Terms and Symbols Used on Boring Logs
Soil Classification System
Logs of Geotechnical Borings (9 pages)
Site Class Estimate (1 page)
Dry Seismic Settlement (1 page)
Continuous Footing Settlement (1 page)
Spread Footing Settlement (1 page)




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

**LEGEND**

Approximate Site Location

Approximate Scale: 1" = 1 Mile

0 1 Mile 2 Miles

N

**Plate 1**
**Site Location Map**

Proposed Tractor Supply Company
Roy Rogers Drive
Victorville, San Bernardino County, California

Earth Systems

7/26/2024

File No.: 306687-002

## LEGEND

**B-9** Approximate Exploration Locations

Approximate Scale: 1" = 100'

0 100' 200'

Source: Google Earth satellite image dated 8/23/2022, with Durban Development Site Plan Overlay dated 12/14/23

**Plate 2a**
**Exploration Location Map**

Proposed Tractor Supply Company
Roy Rogers Drive
Victorville, San Bernardino County, California

Earth Systems

7/26/2024 | File No.: 306687-002

## LEGEND

B-9 Approximate Exploration Locations

af Undocumented Fill

Qoa Quaternary Older Alluvium

N

Approximate Scale: 1" = 100'

0 100' 200'

Source: Google Earth satellite image dated 8/23/2022

**Plate 2b**
**Local Geologic Map**

Proposed Tractor Supply Company
Roy Rogers Drive
Victorville, San Bernardino County, California

Earth Systems

| 7/26/2024 | File No.: 306687-002 |
|---|---|

## LEGEND

| | | | |
|---|---|---|---|
| Qs | Dune sand | Φc | Oligocene nonmarine |
| Qal | Alluvium | Ec | Eocene nonmarine |
| Ql | Lake deposits | E | Eocene marine |
| Qg | Glacial deposits | Ep | Paleocene marine |
| Qr | River terrace deposits | Tc | Tertiary nonmarine |
| Qm | Pleistocene marine and marine terrace deposits | Tm | Tertiary marine |
| Qpv | Pleistocene volcanic rocks<br>Qpv$^r$-rhyolite<br>Qpv$^a$-andesite<br>Qpv$^b$-basalt<br>Qpv$^p$-pyroclastic rocks | Ti | Tertiary intrusive (hypabyssal) rocks<br>Ti$^r$-rhyolite<br>Ti$^a$-andesite<br>Ti$^b$-basalt |
| Qc | Pleistocene nonmarine | Tl | Tertiary lake deposits |
| QP | Plio-Pleistocene nonmarine | Tv | Tertiary volcanic rocks<br>Tv$^r$-rhyolite<br>Tv$^a$-andesite<br>Tv$^b$-basalt<br>Tv$^p$-pyroclastic rocks |
| | Quaternary and/or Pliocene cinder cones | Ku | Upper Cretaceous marine |
| Pc | Undivided Pliocene nonmarine | Ju | Upper Jurasssic marine |
| Pu | Upper Pliocene marine | gr | Mesozoic granitic rocks |
| Pmlc | Middle and/or lower Pliocene nonmarine | bi | Mesozoic basic intrusive rocks |
| Pml | Middle and/or lower Pliocene marine | ub | Mesozoic ultrabasic intrusive rocks |
| Pv | Pliocene volcanic rocks<br>Pv$^r$-rhyolite<br>Pv$^a$-andesite<br>Pv$^b$-basalt<br>Pv$^p$-pyroclastic rocks | JTRv | Jurassic-Triassic metavolcanic rocks |
| Mc | Undivided Miocene nonmarine | TR is | Pre-Cretaceous metamorphic rocks (is=limestone) |
| Muc | Upper Miocene nonmarine | TR ms | Pre-Cretaceous metasedimentary rocks |
| Mu | Upper Miocene marine | TR mv | Pre-Cretaceous metavolcanic rocks |
| Mm | Middle Miocene marine | TR gr,m | Pre-Cenozoic granitic and metamorphic rocks |
| Ml | Lower Miocene marine | pCc | Precambrian igneous and metamorphic rock complex |
| Mv | Miocene volcanic rocks<br>Mv$^r$-rhyolite<br>Mv$^a$-andesite<br>Mv$^b$-basalt<br>Mv$^p$-pyroclastic rocks | pC | Undivided Precambrian metamorphic rocks<br>pCg=gneiss<br>pCs=schist<br>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 sheet, dated 1965 & USGS Geologic Map of California, Long Beach sheet, dated 1962

**Plate 3**
**Regional Geologic Map**

Proposed Tractor Supply Company
Roy Rogers Drive
Victorville, San Bernardino County, California

Earth Systems

| | |
|---|---|
| 7/26/2024 | File No.: 306687-002 |

**Table A-1**
**Fault Parameters**

| Fault Section Name | Distance (miles) | Distance (km) | Upper Seis. Depth (km) | Lower Seis. Depth (km) | Avg Dip Angle (deg.) | Avg Dip Direction (deg.) | Avg Rake (deg.) | Trace Length (km) | Fault Type | **Mean Mag** | Mean Return Interval (years) | Slip Rate (mm/yr) |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| North Frontal (West) FM3.1, 3.2 | 10.8 | 17.4 | 0.0 | 15.7 | 49 | 171 | 90 | 50 | B | **7.2** | | 1 |
| Helendale-So Lockhart FM3.1, 3.2 | 13.1 | 21.1 | 0.0 | 12.8 | 90 | 51 | 180 | 114 | B | **7.4** | | 0.6 |
| Cleghorn FM3.1, 3.2 | 15.8 | 25.4 | 0.0 | 15.5 | 90 | 187 | 0 | 25 | B | **6.7** | | 3 |
| San Andreas (San Bernardino N) FM3.1, 3.2 | 18.7 | 30.1 | 0.0 | 12.8 | 90 | 212 | 180 | 35 | A | **7.5** | 103 | 27 |
| San Andreas (Mojave S) FM3.1, 3.2 | 19.0 | 30.6 | 0.0 | 13.1 | 90 | 206 | 180 | 98 | A | **7.7** | 102 | 34 |
| San Jacinto (San Bernardino) FM3.1, 3.2 | 20.5 | 33.0 | 0.0 | 16.1 | 90 | 225 | 180 | 37 | A | **7.6** | 219 | 17 |
| San Jacinto (Lytle Creek connector) FM3.1, 3.2 | 21.5 | 34.5 | 0.0 | 16.1 | 90 | na | na | 23 | B' | **6.7** | | |
| San Andreas, (North Branch, Mill Creek) FM3.1, | 24.4 | 39.3 | 0.0 | 18.2 | 76 | 204 | 180 | 106 | A | **7.6** | 219 | 34 |
| Cucamonga FM3.1, 3.2 | 24.5 | 39.4 | 0.0 | 7.8 | 45 | 347 | 90 | 28 | B | **6.6** | | 5 |
| San Gabriel (Extension) FM3.1, 3.2 | 24.9 | 40.1 | 0.0 | 14.7 | 61 | 6 | 180 | 62 | B' | **7.2** | | |
| San Andreas (San Bernardino S) FM3.1, 3.2 | 26.4 | 42.5 | 0.0 | 12.8 | 90 | 210 | 180 | 43 | A | **7.6** | 150 | 29 |
| Fontana (Seismicity) FM3.1, 3.2 | 26.5 | 42.6 | 0.0 | 16.3 | 80 | 313 | na | 24 | B' | **6.7** | | |
| Lenwood-Lockhart-Old Woman Springs FM3.1, : | 27.5 | 44.3 | 0.0 | 13.2 | 90 | 43 | 180 | 145 | B | **7.5** | | 0.9 |
| North Frontal (East) FM3.1, 3.2 | 31.7 | 51.0 | 0.0 | 16.6 | 41 | 187 | 90 | 28 | B | **6.9** | | 0.5 |
| Camp Rock FM3.1, 3.2 | 32.2 | 51.9 | 0.0 | 15.1 | 90 | na | na | 45 | B' | **6.9** | | |
| Gravel Hills-Harper Lk FM3.1, 3.2 | 32.7 | 52.7 | 0.0 | 11.4 | 90 | 41 | 180 | 65 | B | **7.0** | | 0.7 |
| San Jose FM3.1, 3.2 | 35.0 | 56.3 | 0.0 | 15.8 | 74 | 334 | 30 | 20 | B | **6.6** | | 0.5 |
| San Jacinto (San Jacinto Valley) rev FM3.1, 3.2 | 35.3 | 56.8 | 0.0 | 16.1 | 90 | 223 | 180 | 18 | A | **7.6** | 219 | 18 |
| Clamshell-Sawpit FM3.1, 3.2 | 35.4 | 56.9 | 0.0 | 14.0 | 50 | 334 | 90 | 16 | B | **6.6** | | 0.5 |
| Johnson Valley (No) 2011 rev FM3.1, 3.2 | 35.9 | 57.7 | 0.0 | 15.9 | 90 | 51 | 180 | 52 | B | **6.8** | | 0.6 |
| Sierra Madre FM3.1, 3.2 | 36.2 | 58.3 | 0.0 | 14.2 | 53 | 19 | 90 | 57 | B | **7.2** | | 2 |
| Blackwater FM 3.1, 3.2 | 37.8 | 60.8 | 0.0 | 12.1 | 90 | 55 | 180 | 60 | B | **7.0** | | 0.5 |
| Calico-Hidalgo FM3.1, 3.2 | 39.6 | 63.7 | 0.0 | 13.9 | 90 | 52 | 180 | 117 | B | **7.4** | | 1.8 |
| San Gorgonio Pass FM3.1, 3.2 | 40.0 | 64.3 | 0.0 | 18.5 | 60 | 11 | na | 29 | B' | **6.9** | | |
| Chino, alt 2, FM3.2 | 41.3 | 66.5 | 0.0 | 13.4 | 65 | 234 | 150 | 29 | B | **6.7** | | 1 |
| Chino, alt 1, FM3.1 | 41.4 | 66.7 | 0.0 | 9.0 | 50 | 236 | 150 | 24 | B | **6.6** | | 1 |
| Paradise FM3.1, 3.2 | 41.7 | 67.1 | 0.0 | 13.4 | 90 | 255 | na | 55 | B' | **7.0** | | |
| Mission Creek FM3.1, 3.2 | 42.1 | 67.8 | 0.0 | 17.7 | 65 | 5 | 180 | 31 | B' | **6.9** | | |
| Coyote Lake FM3.1, 3.2 | 42.5 | 68.3 | 0.0 | 13.4 | 90 | 165 | na | 52 | B' | **7.0** | | |
| Manix-Afton Hills FM3.1, 3.2 | 42.7 | 68.8 | 0.0 | 13.2 | 90 | 161 | na | 59 | B' | **7.0** | | |
| San Jacinto (San Jacinto Valley, stepover) | 44.8 | 72.0 | 0.0 | 16.1 | 90 | 224 | 180 | 24 | A | **7.6** | 219 | 9 |
| San Jacinto (Stepovers Combined) FM3.1, 3.2 | 44.8 | 72.0 | 0.0 | 16.5 | 90 | 229 | 180 | 25 | A | **7.5** | 110 | 4 |
| Emerson-Copper Mtn FM3.1, 3.2 | 44.8 | 72.1 | 0.0 | 14.1 | 90 | 51 | 180 | 54 | B | **7.0** | | 0.6 |
| Homestead Valley FM3.1, 3.2 | 45.0 | 72.3 | 0.0 | 15.9 | 90 | na | na | 46 | B' | **7.0** | | |
| Raymond FM3.1, 3.2 | 45.2 | 72.7 | 0.0 | 15.6 | 79 | 348 | 60 | 22 | B | **6.7** | | 1.5 |
| Yorba Linda FM3.1, 3.2 | 46.6 | 74.9 | 0.0 | 13.3 | 90 | 153 | na | 18 | B' | **6.5** | | |
| Pinto Mtn FM3.1, 3.2 | 47.3 | 76.2 | 0.0 | 15.5 | 90 | 175 | 0 | 74 | B | **7.2** | | 2.5 |
| San Andreas (San Gorgonio Pass-Garnet HIll) FM | 49.5 | 79.6 | 0.0 | 12.8 | 58 | 20 | 180 | 56 | A | **7.6** | 219 | 24 |
| Whittier, alt 1, FM3.1 | 49.6 | 79.8 | 0.0 | 12.4 | 70 | 24 | 150 | 46 | A | **7.1** | 530 | 4 |
| Whittier, alt 2 FM3.2 | 49.6 | 79.8 | 0.0 | 14.1 | 75 | 24 | 150 | 46 | A | **6.9** | 165 | 4 |

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

Based on Site Coordinates of 34.52231 Latitude, -117.32743 Longitude

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

Site Coordinates: 34.522 N 117.327 W

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

| *Day* | *Year* | *Epicenter Latittude (Degrees)* | *Epicenter Longitude (Degrees)* | *Distance from Site (mi)* | *Magnitude* $M_W$ |
|---|---|---|---|---|---|
| 7/22 | 1899 | 34.30 | 117.50 | 18.2 | **6.4** |
| 9/12 | 1970 | 34.27 | 117.53 | 20.9 | **5.2** |
| 12/8 | 1812 | 34.37 | 117.65 | 21.2 | **7.5** |
| 7/30 | 1894 | 34.30 | 117.60 | 21.8 | **6.2** |
| 12/16 | 1858 | 34.20 | 117.40 | 22.6 | **6.0** |
| 7/22 | 1899 | 34.20 | 117.40 | 22.6 | **5.9** |
| 6/14 | 1892 | 34.20 | 117.50 | 24.3 | **5.5** |
| 10/16 | 1999 | 34.24 | 117.04 | 25.5 | **5.6** |
| 9/20 | *1907 | 34.20 | 117.10 | 25.8 | **5.8** |
| 12/4 | 1992 | 34.37 | 116.90 | 26.5 | **5.2** |
| 2/10 | 2001 | 34.29 | 116.95 | 26.8 | **5.1** |
| 8/29 | 1943 | 34.31 | 116.93 | 27.0 | **5.3** |
| 11/27 | 1992 | 34.34 | 116.90 | 27.4 | **5.3** |
| 6/28 | 1992 | 34.26 | 116.91 | 29.9 | **5.3** |
| 2/22 | 2003 | 34.31 | 116.85 | 30.9 | **5.2** |
| 1/16 | 1930 | 34.20 | 116.90 | 33.0 | **5.5** |
| 1/16 | 1930 | 34.18 | 116.92 | 33.1 | **5.1** |
| 2/28 | 1990 | 34.14 | 117.70 | 33.9 | **5.7** |
| 7/9 | 1992 | 34.24 | 116.84 | 33.9 | **5.3** |
| 8/17 | 1992 | 34.20 | 116.86 | 34.7 | **5.2** |
| 6/28 | 1992 | 34.20 | 116.83 | 36.0 | **6.5** |
| 7/23 | *1923 | 34.00 | 117.25 | 36.3 | **6.2** |
| 6/28 | 1992 | 34.16 | 116.85 | 37.0 | **5.5** |
| 9/5 | 1928 | 35.00 | 117.00 | 37.9 | **5.5** |
| 8/28 | 1889 | 34.20 | 117.90 | 39.5 | **5.6** |
| 1/16 | 1857 | 34.52 | 118.04 | 40.5 | **6.3** |
| 11/22 | 1880 | 34.00 | 117.00 | 40.6 | **5.5** |
| 12/19 | 1880 | 34.00 | 117.00 | 40.6 | **5.9** |
| 6/28 | 1991 | 34.27 | 117.99 | 41.6 | **5.6** |
| 3/18 | 1997 | 34.97 | 116.82 | 42.2 | **5.3** |
| 2/7 | 1889 | 34.10 | 116.70 | 46.2 | **5.6** |
| 7/29 | 2008 | 33.96 | 117.77 | 46.5 | **5.4** |
| 6/1 | 1975 | 34.51 | 116.49 | 47.7 | **5.3** |
| 6/28 | 1992 | 34.34 | 116.51 | 48.2 | **5.4** |
| 4/99 | 1803 | 34.20 | 118.10 | 49.3 | **5.5** |
| 6/12 | 1944 | 34.00 | 116.72 | 50.0 | **5.1** |
| 9/26 | 1929 | 34.83 | 116.52 | 50.5 | **5.1** |
| 6/12 | 1944 | 34.01 | 116.69 | 50.7 | **5.2** |
| 7/1 | 1992 | 34.33 | 116.46 | 51.2 | **5.3** |
| 3/15 | 1979 | 34.33 | 116.44 | 52.3 | **5.5** |

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**

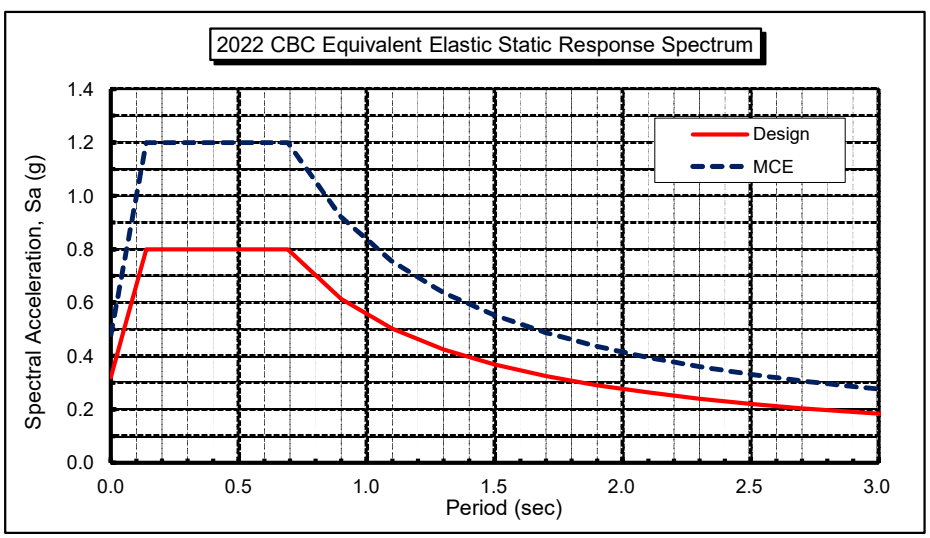
**2022 California Building Code (CBC) (ASCE 7-16, Supplement 3) Seismic Design Parameters**

**(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)**

| | | | | CBC Reference | ASCE 7-16 Reference |
|---|---|---|---|---|---|
| Seismic Design Category | | **D** | | | |
| Site Class | | **D** | | Table 1613.5.6 | Table 11.6-1 |
| Latitude: | | 34.522 | | Table 1613.5.2 | Table 20.3-1 |
| Longitude: | | -117.327 | | | |
| Maximum Considered Earthquake (MCE) Ground Motion | | | | | |
| Short Period Spectral Reponse | $S_S$ | **1.155** | **g** | Figure 1613.5 | Figure 22-1 |
| 1 second Spectral Response | $S_1$ | **0.447** | **g** | Figure 1613.5 | Figure 22-2 |
| Site Coefficient | $F_a$ | 1.04 | | Table 1613.5.3(1) | Table 11.4-1 |
| Site Coefficient | $F_v$ | 1.85 | | Table 1613.5.3(2) | Table 11-4.2 |
| | $S_{MS}$ | 1.199 | g | $= F_a x S_S$ | |
| | $S_{M1}$ | 0.828 | g | $= F_v x S_1$ | |
| ASCE7-16 Supplement 3, Exception 1* | $S_{M1*}$ | 1.242 | g | $=S_{M1} x 1.5$ | Exception 1, 11.4.8, Supplement 3 |
| Design Earthquake Ground Motion | | | | | |
| Short Period Spectral Reponse | $S_{DS}$ | **0.799** | **g** | $= 2/3 x S_{MS}$ | |
| Unadjusted 1 second Spectral Response | $S_{D1}$ | **0.552** | **g** | $= 2/3 x S_{M1}$ | |
| Modified 1 second Spectral Response* | $S_{D1*}$ | **0.828** | **g** | $= 2/3 x S_{M1}$* | *Exception 1, 11.4.8, Supplement 3 |

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

| | | | |
|---|---|---|---|
| To | 0.14 | sec | $= 0.2*S_{D1}/S_{DS}$ |
| Ts (11.4.8 ASCE 7-16 Exception Assumed) | 0.69 | sec | $= S_{D1}/S_{DS}$ |
| Risk Category | II | | Table 1604.5 |
| Seismic Importance Factor | 1.00 | | |
| $F_{PGA}$ | 1.10 | | |
| **$PGA_M$** | **0.55** | | |
| Vertical Coefficient ($C_V$) | 1.33 | | Table 11.9-1 |




| Table 11.5-1 Period T (sec) | Design Sa (g) |
|---|---|
| 0.00 | 0.320 |
| 0.05 | 0.493 |
| 0.14 | 0.799 |
| 0.69 | 0.799 |
| 0.90 | 0.614 |
| 1.10 | 0.502 |
| 1.30 | 0.425 |
| 1.50 | 0.368 |
| 1.70 | 0.325 |
| 1.90 | 0.291 |
| 2.10 | 0.263 |
| 2.30 | 0.240 |
| 2.50 | 0.221 |
| 2.70 | 0.205 |
| 2.90 | 0.190 |
| 3.10 | 0.178 |

## DESCRIPTIVE SOIL CLASSIFICATION

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

## SOIL GRAIN SIZE

U.S. STANDARD SIEVE

| 12” | 3” | 3/4” | 4 | 10 | 40 | 200 |
|---|---|---|---|---|---|---|

| BOULDERS | COBBLES | GRAVEL | | SAND | | | SILT | CLAY |
|---|---|---|---|---|---|---|---|---|
| | | COARSE | FINE | COARSE | MEDIUM | FINE | | |

| 305 | 76.2 | 19.1 | 4.76 | 2.00 | 0.42 | 0.074 | 0.002 |
|---|---|---|---|---|---|---|---|

SOIL GRAIN SIZE IN MILLIMETERS

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

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

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

## CONSISTENCY OF COHESIVE SOILS (CLAY OR CLAYEY SOILS)

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

## MOISTURE DENSITY

**Moisture Condition:** An observational term; dry, damp, moist, wet, saturated.
**Moisture Content:** The weight of water in a sample divided by the weight of dry soil in the soil sample expressed as a percentage.
**Dry Density:** The pounds of dry soil in a cubic foot.

## MOISTURE CONDITION

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

## PLASTICITY

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

## 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<br><br>More than 50% of material is larger than No. 200 sieve size | GRAVEL AND GRAVELLY SOILS<br><br>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<br><br>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<br><br>More than 50% of material is smaller than No. 200 sieve size | SILTS AND CLAYS | LIQUID LIMIT LESS THAN 50 | | ML | Inorganic silts and very fine sands, rock flour, silty low clayey fine sands or clayey silts with slight plasticity |
| | | | | CL | Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays |
| | | | | OL | Organic silts and organic silty clays of low plasticity |
| | | LIQUID LIMIT GREATER THAN 50 | | MH | Inorganic silty, micaceous, or diatomaceous fine sand or silty soils |
| | | | | CH | Inorganic clays of high plasticity, fat clays |
| | | | | OH | Organic clays of medium to high plasticity, organic silts |
| HIGHLY ORGANIC SOILS | | | | PT | Peat, humus, swamp soils with high organic contents |
| VARIOUS SOILS AND MAN MADE MATERIALS | | | | | Fill Materials |
| MAN MADE MATERIALS | | | | | Asphalt and concrete |

**Soil Classification System**

**Earth Systems**

**Earth Systems**

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

## Boring No. B-1

Project Name: Proposed Tractor Supply Company
Project Number 306687-002
Boring Location: See Plate 2

Drilling Date: June 17, 2024
Drilling Method: Mobile B-61 CME w/auto hammer
Drill Type: 8" HSA
Logged By: JG



| Depth (Ft.) | Sample Type (Bulk / SPT / MOD Calif.) | Penetration Resistance (Blows/6") | Symbol | USCS | Dry Density (pcf) | Moisture Content (%) | Description of Units |
|---|---|---|---|---|---|---|---|
| 0 | | | | SM | | | SILTY SAND: light brown, loose, damp, fine to medium grained sand, some coarse grained sand, and traces of roots, heavy calcium carbonate (CaCO3), white, cemented |
| | MOD Calif. | 29,50/6" | | | 116 | 3 | |
| 5 | MOD Calif. | 7,9,15 | | SP | 113 | 3 | SAND: light brown, medium dense, damp, fine to coarse grained sand, with traces of gravel to 3/8" |
| | MOD Calif. | 31,50 | | ML | 112 | 6 | SANDY SILT: light tan brown, very stiff, damp, fine to medium grained sand, with traces of CaCO3 |
| 10 | MOD Calif. | 26,50 | | | 114 | 5 | |
| | MOD Calif. | 27,50 | | SM | 114 | 6 | SILTY SAND: mottled tan/red brown, dense, damp, fine grained sand, slightly cemented |
| 15 | SPT / MOD Calif. | 18,23,25 | | SM | | | SILTY SAND: light tan brown, very dense, dry, fine to medium grained sand, no cementation |
| 20 | MOD Calif. | 25,25,29 | | | 115 | 3 | becomes slightly cemented, with lenses of fine to coarse grained sand, damp |
| 25 | SPT / MOD Calif. | 16,17,21 | | SP | | | SAND: light brown, medium dense, dry, fine to medium grained sand, with some coarse grained sand |
| 30 | MOD Calif. | 19,25,37 | | ML | 111 | 6 | SANDY SILT: light brown, hard, damp, fine grained sand, slightly cemented |
| 35 | SPT / MOD Calif. | 17,28,39 | | | | | |
| 40 | MOD Calif. | 19,29,35 | | SP | 111 | 2 | SAND: light brown, medium dense, dry, fine to coarse grained sand, traces of silt |
| 45 | SPT / MOD Calif. | 17,30,31 | | SM | | 8 | SILTY SAND: light brown, dense, damp, fine grained sand, traces of cemented nodules to 1/2" |
| | MOD Calif. | 28,50 | | ML | 110 | 5 | SILT: grey brown, hard, dry, fine grained sand, few iron oxide stains |
| 50 | | | | | | | very hard drilling - Refusal |
| 55 | | | | | | | Boring refusal at 48-1/2 feet<br>No groundwater encountered<br>Backfilled with cuttings |
| 60 | | | | | | | |

Graphic Trend
Blow Count Dry Density

**Earth Systems**

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Phone (951) 928-9799, Fax (951) 928-9948

## Boring No. B-2

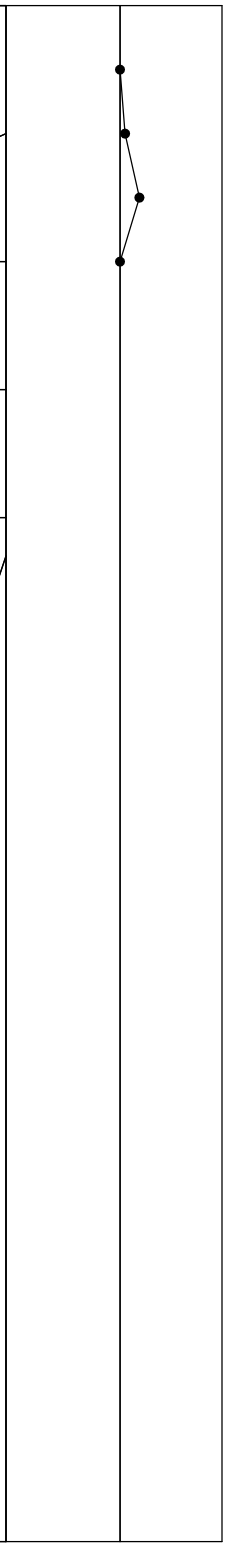
Project Name: Proposed Tractor Supply Company
Project Number 306687-002
Boring Location: See Plate 2

Drilling Date: June 17, 2024
Drilling Method: Mobile B-61 CME w/auto hammer
Drill Type: 8" HSA
Logged By: JG



| Depth (Ft.) | Sample Type (Bulk / SPT / MOD Calif.) | Penetration Resistance (Blows/6") | Symbol | USCS | Dry Density (pcf) | Moisture Content (%) | Description of Units |
|---|---|---|---|---|---|---|---|
| 0 | MOD Calif. | 50/6" | | SM | 96 | 4 | SILTY SAND: light brown, dense, dry, fine to medium grained sand, some coarse grained sand, traces of gravel to 1/2" and root hairs, moderately cemented with varying amounts of CaCO3 damp |
| 5 | MOD Calif. | 21,50 | | SM | 105 | 6 | SILTY SAND: very light gray brown, very dense, damp, fine grained sand, severely cemented with CaCO3 dry |
| | MOD Calif. | 15,27,40 | | | 119 | 2 | |
| 10 | MOD Calif. | 14,26,28 | | ML | 99 | 5 | SANDY SILT: very light grey brown, hard, damp, moderately cemented with CaCO3 |
| 15 | SPT / MOD Calif. | 11,17,19 | | ML | | | SANDY SILT: brown, hard, dry, fine grained sand |
| 20 | MOD Calif. | 17,30,39 | | SM | | | SILTY SAND: light brown, very dense, dry, fine grained sand |
| 25 | | | | | | | |
| 30 | | | | | | | |
| 35 | | | | | | | |
| 40 | | | | | | | |
| 45 | | | | | | | |
| 50 | | | | | | | |
| 55 | | | | | | | End of Boring at 21-1/2 feet<br>No groundwater encountered<br>Backfilled with cuttings |
| 60 | | | | | | | |

Graphic Trend
Blow Count Dry Density



**Earth Systems**

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## Boring No. B-3

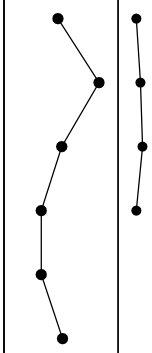
Project Name: Proposed Tractor Supply Company
Project Number 306687-002
Boring Location: See Plate 2

Drilling Date: June 17, 2024
Drilling Method: Mobile B-61 CME w/auto hammer
Drill Type: 8" HSA
Logged By: JG



| Depth (Ft.) | Sample Type (Bulk / SPT / MOD Calif.) | Penetration Resistance (Blows/6") | Symbol | USCS | Dry Density (pcf) | Moisture Content (%) | Description of Units | Graphic Trend (Blow Count / Dry Density) |
|---|---|---|---|---|---|---|---|---|
| 0 | | | | SM | | | SILTY SAND: light brown, very loose, dry, fine to medium grained sand, some coarse grained sand, and traces of gravel 3/8", roots to 1/4" | |
| | MOD Calif. | 20,28,43 | | | 118 | 4 | very dense, damp | |
| 5 | MOD Calif. | 24,37,48 | | | 122 | 3 | | |
| | MOD Calif. | 17,26,50 | | | 124 | 2 | dry | |
| 10 | MOD Calif. | 19,27,42 | | | 118 | 4 | damp | |
| | SPT | 14,22,27 | | | | | becomes fine grained sand | |
| 15 | SPT | 12,16,17 | | | | | becomes fine to medium grained sand | |
| 20 | | | | | | | | |
| 25 | | | | | | | | |
| 30 | | | | | | | | |
| 35 | | | | | | | | |
| 40 | | | | | | | | |
| 45 | | | | | | | | |
| 50 | | | | | | | | |
| 55 | | | | | | | End of Boring at 16-1/2 feet<br>No groundwater encountered<br>Backfilled with cuttings | |
| 60 | | | | | | | | |

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



**Earth Systems**

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## Boring No. B-4

Project Name: Proposed Tractor Supply Company
Project Number 306687-002
Boring Location: See Plate 2

Drilling Date: June 17, 2024
Drilling Method: Mobile B-61 CME w/auto hammer
Drill Type: 8" HSA
Logged By: JG



| Depth (Ft.) | Sample Type (Bulk / SPT / 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 brown, medium dense, dry, fine to medium grained sand, some coarse grained sand | | |
| | MOD Calif. | 9,18,30 | | ML | 112 | 5 | SANDY SILT: grey brown, very stiff, damp, fine grained sand, slightly cemented, traces of $CaCO_3$ | | |
| 5 | MOD Calif. | 16,25,30 | | | 116 | 4 | | | |
| 10 | | | | | | | | | |
| 15 | | | | | | | | | |
| 20 | | | | | | | | | |
| 25 | | | | | | | | | |
| 30 | | | | | | | | | |
| 35 | | | | | | | | | |
| 40 | | | | | | | | | |
| 45 | | | | | | | | | |
| 50 | | | | | | | | | |
| 55 | | | | | | | End of Boring at 6-1/2 feet<br>No groundwater encountered<br>Backfilled with cuttings | | |
| 60 | | | | | | | | | |

**Description of Units**

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

Graphic Trend: Blow Count | Dry Density

**Earth Systems**

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## Boring No. B-5

Project Name: Proposed Tractor Supply Comapny
Project Number 306687-002
Boring Location: See Plate 2

Drilling Date: June 17, 2024
Drilling Method: Mobile B-61 CME w/auto hammer
Drill Type: 8" HSA
Logged By: JG



| Depth (Ft.) | Sample Type (Bulk / SPT / MOD Calif.) | Penetration Resistance (Blows/6") | Symbol | USCS | Dry Density (pcf) | Moisture Content (%) | Description of Units |
|---|---|---|---|---|---|---|---|
| 0 | Bulk | | | SM | | | SILTY SAND: light brown, very dense, dry, fine to medium grained sand, some coarse grained sand, and traces of gravel to 1", with carbonate stringers |
| | MOD Calif. | 15,18,21 | | | 112 | 2 | |
| | | | | SP | | | SAND: light brown, very dense, dry, fine to coarse grained sand, with gravel to 1/2" and silt |
| 5 | MOD Calif. | 13,22,31 | | SM | 116 | 3 | |
| | | | | | | | SILTY SAND: white, very dense, damp, fine grained sand, cemented heavily with CaCO3 |
| 10 | | | | | | | |
| 15 | | | | | | | |
| 20 | | | | | | | |
| 25 | | | | | | | |
| 30 | | | | | | | |
| 35 | | | | | | | |
| 40 | | | | | | | |
| 45 | | | | | | | |
| 50 | | | | | | | |
| 55 | | | | | | | End of Boring at 6-1/2 feet<br>No groundwater encountered<br>Backfilled with cuttings |
| 60 | | | | | | | |

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

Graphic Trend
Blow Count Dry Density

**Earth Systems**

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Phone (951) 928-9799, Fax (951) 928-9948

## Boring No. B-6

Project Name: Proposed Tractor Supply Company
Project Number 306687-002
Boring Location: See Plate 2

Drilling Date: June 17, 2024
Drilling Method: Bobile B-61 CME w/auto hammer
Drill Type: 8" HSA
Logged By: JG

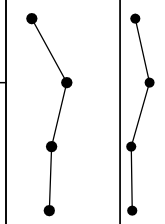


| Depth (Ft.) | Sample Type (Bulk / SPT / MOD Calif.) | Penetration Resistance (Blows/6") | Symbol | USCS | Dry Density (pcf) | Moisture Content (%) | Description of Units |
|---|---|---|---|---|---|---|---|
| 0 | Bulk | | | SM | | | SILTY SAND: light brown, loose, damp, fine to medium grained sand, some coarse grained sand, gravel to 1/2" |
| | MOD Calif. | 8,12,22 | | | 115 | 3 | becomes more fine, heavily cemented with CaCO3 |
| 5 | MOD Calif. | 30,50/5" | | ML | 109 | 7 | SANDY SILT: very light brown and white, hard, damp, fine grained sand, heavy CaCO3 cementation |
| | MOD Calif. | 17,24,36 | | | 111 | 6 | slight caliche stringers |
| 10 | MOD Calif. | 19,26,31 | | | 112 | 6 | light olive brown, less CaCO3 |
| 15 | | | | | | | |
| 20 | | | | | | | |
| 25 | | | | | | | |
| 30 | | | | | | | |
| 35 | | | | | | | |
| 40 | | | | | | | |
| 45 | | | | | | | |
| 50 | | | | | | | |
| 55 | | | | | | | End of Boring at 11-1/2 feet<br>No groundwater encountered<br>Backfilled with cuttings |
| 60 | | | | | | | |

### Description of Units

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

Graphic Trend



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## Boring No. B-7

Project Name: Proposed Tractor Supply Comapny
Project Number 306687-002
Boring Location: See Plate 2

Drilling Date: June 17, 2024
Drilling Method: Mobile B-61 CME w/auto hammer
Drill Type: 8" HSA
Logged By: JG



| Depth (Ft.) | Sample Type (Bulk / SPT / MOD Calif.) | Penetration Resistance (Blows/6") | Symbol | USCS | Dry Density (pcf) | Moisture Content (%) | Description of Units | Graphic Trend (Blow Count / Dry Density) |
|---|---|---|---|---|---|---|---|---|
| 0 | Bulk | | | SM | | | SILTY SAND: light brown, very loose, dry, fine to medium grained sand, with coarse grained sand | |
| | MOD Calif. | 28,50/4" | | ML | 100 | 6 | SANDY SILT: very light grey brown, hard, damp, fine grained sand, cemented with CaCO3 | |
| 5 | MOD Calif. | 21,50 | | ML | 100 | 4 | SANDY SILT: light brown, hard, damp, fine grained sand, with CaCO3 nodules and stringers | |
| 10 | | | | | | | | |
| 15 | | | | | | | | |
| 20 | | | | | | | | |
| 25 | | | | | | | | |
| 30 | | | | | | | | |
| 35 | | | | | | | | |
| 40 | | | | | | | | |
| 45 | | | | | | | | |
| 50 | | | | | | | | |
| 55 | | | | | | | End of Boring at 6-1/2 feet<br>No groundwater encountered<br>Backfilled with cuttings | |
| 60 | | | | | | | | |

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

**Earth Systems**

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## Boring No. B-8

Project Name: Proposed Tractor Supply Company
Project Number 306687-002
Boring Location: See Plate 2

Drilling Date: June 17, 2024
Drilling Method: Mobile B-61CME w/auto hammer
Drill Type: 8" HSA
Logged By: JG



| Depth (Ft.) | Sample Type (Bulk / SPT / MOD Calif.) | Penetration Resistance (Blows/6") | Symbol | USCS | Dry Density (pcf) | Moisture Content (%) | Description of Units |
|---|---|---|---|---|---|---|---|
| 0 | Bulk | | | SM | | | SILTY SAND: light brown, loose, dry, fine to medium grained sand, some coarse grained sand, possible Fill |
| | MOD Calif. | 16,22,32 | | SM | 105 | 3 | SILTY SAND: light brown, loose, damp, fine to medium grained sand, some coarse grained sand slightly cemented with CaCO3 stringers |
| 5 | MOD Calif. | 23,50 | | | 113 | 3 | |
| | MOD Calif. | 26,50 | | SP | 111 | 2 | SAND: light brown, medium dense, dry, fine to coarse grained sand, traces of gravel to 3/8" |
| 10 | MOD Calif. | 7,12,15 | | ML | 105 | 7 | SANDY SILT: light brown and white, hard, damp, fine grained sand, cemented with CaCO3, slight porosity, root hair |
| | MOD Calif. | 15,19,30 | | | | | |
| 15 | MOD Calif. | 15,50/4" | | | | | less cemented, no CaCO3 |
| | SPT | 30,31,35 | | SM | | | SILTY SAND: mottled olive/red brown, dense, dry, fine to medium grained sand, traces of coarse grained sand |
| 20 | MOD Calif. | 24,32,50 | | SM | | | SILTY SAND: olive brown, very dense, dry, fine grained sand, strongly cemented |
| 25 | SPT | 19,24,29 | | SP-SM | | | SAND WITH SILT: brown, very dense, dry, fine grained sand with silt<br>Refusal due to hard drilling |
| 30 | | | | | | | |
| 35 | | | | | | | |
| 40 | | | | | | | |
| 45 | | | | | | | |
| 50 | | | | | | | |
| 55 | | | | | | | Boring refusal at 26-1/2 feet<br>No groundwater encountered<br>Backfilled with cuttings |
| 60 | | | | | | | |

### Description of Units

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

Graphic Trend
Blow Count Dry Density

**Earth Systems**

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

## Boring No. B-9

Project Name: Proposed Tractor Supply Company
Project Number 306687-002
Boring Location: See Plate 2

Drilling Date: June 17, 2024
Drilling Method: Mobile B-61 CME w/auto hammer
Drill Type: 8" HSA
Logged By: JG

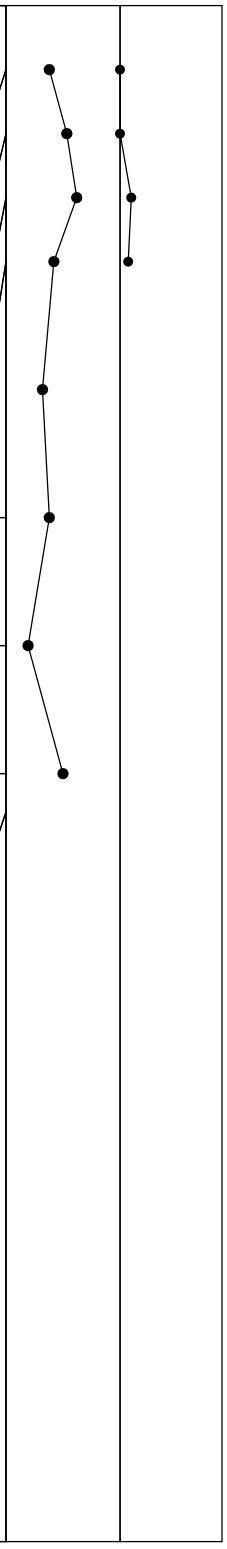


| Depth (Ft.) | Sample Type (Bulk / SPT / MOD Calif.) | Penetration Resistance (Blows/6") | Symbol | USCS | Dry Density (pcf) | Moisture Content (%) | Description of Units |
|---|---|---|---|---|---|---|---|
| 0 | Bulk | | | SM | | | SILTY SAND: light brown, very loose, dry, fine to medium grained sand, some coarse grained sand and gravel to 1/2", roots to 1/2" |
| | MOD Calif. | 7,18,39 | | SM | 100 | 5 | |
| 5 | MOD Calif. | 30,50/3" | | ML | 83 | 6 | SILTY SAND: light brown, loose, damp, fine grained sand, white layers of CaCO3 streaking |
| | MOD Calif. | 9,18,25 | | SP | 111 | 2 | SANDY SILT: very light grey brown, very stiff, damp, fine grained sand, moderately cemented |
| 10 | MOD Calif. | 15,29,34 | | ML | 108 | 2 | SAND: light brown, dense, dry, fine to coarse grained sand |
| 15 | SPT / MOD Calif. | 15,29,24 | | | | | SANDY SILT: light brown, hard, dry, fine to medium grained sand, some coarse grained sand |
| 20 | MOD Calif. | 20,25,32 | | SP | | | SAND: light brown, dense, dry, fine to coarse grained sand, with traces of silt |
| 25 | SPT / MOD Calif. | 10.13.16 | | ML | | | CLAYEY SILT: olive brown, very stiff, damp, fine grained sand |
| 30 | MOD Calif. | 20,31,44 | | SP | | | SAND: light brown, dense, dry, fine to coarse grained sand, gravel to 1/2" |
| 35 | | | | | | | |
| 40 | | | | | | | |
| 45 | | | | | | | |
| 50 | | | | | | | |
| 55 | | | | | | | End of Boring at 31-1/2 feet<br>No groundwater encountered<br>Backfilled with cuttings |
| 60 | | | | | | | |

### Description of Units

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

Graphic Trend
Blow Count Dry Density



| Boring No. | B-1 | Project and Number | Proposed Tractor Sup | 306687-002 |
|---|---|---|---|---|
| | | ESSW Field Staff | | |
| | | Drilling Company | GP Drilling | |
| | | Drilling Method | 6-8" H S A | HSA Inner Diameter 3" |
| | | Site Latitude (North) | Decimal Degrees | 34.5223°N |
| | | Site Longitude (West) | Decimal Degrees | 117.3274°W |

| | |
|---|---|
| Date Drilled | 6/17/2024 |
| Hammer Weight (lbs) | 140 |
| Hammer Drop (inches) | 30 |
| Hammer Efficiency ($E_M$) | 72 |
| Borehole Correction (Cb)* | 1 |
| *inside diameter of Hollow Stem Auger | |
| Sampler Correction Mod Cal to SPT | 0.63 |
| Sampler Liner Correction (Cs) | |
| 1.2 Applied if SPT Sampler Used | |
| 1.0 Applied if Cal Sampler Used | |
| Rod Length Above Ground (ft) | 3 |
| Depth to Estimate Vs Over (ft)* | 100 |
| *Caltrans Estimation Method | |
| *$N_{sub}$ Value Desired For Column 6 | 70 |
| *Only Used for Calculating Nsub otherwise not used by program (i.e.N50, N70, N80, etc) | |

| Calculation Results | |
|---|---|
| Ave. SPT $N_{60HE}$-value (blows/ft) | 46 |
| | (Based on Upper 47.5 feet) |
| Ave. Shear Wave Velocity (ft/sec) | 1017 |
| | (Based on Upper 47.5 feet) |
| Soil Profile Type (Site Class) | D |
| | (Based on Upper 47.5 feet) |
| Ave. Friction Angle (degrees) | 37 |
| | (Based on Upper 47.5 feet) |
| Estimated Shear Wave Velocity ** Based on Depth Less than 100' ft | 1024 (ft/sec Upper 100 feet) |
| Soil Profile Type (Site Class)** | D Based on Ave. Shear Wave Velocity (ft/sec) 312 (m/sec Upper 100 feet) |
| Ave. Field SPT N-value (blows/ft) | 38.1 |
| | (Based on Upper 47.5 feet) |
| Ave. Field SPT N-value (blows/ft) | 44.7 |
| | (Based on Upper 100 feet) |
| Soil Profile Type (Site Class)** | D Based on Ave. Field Blow Count 45 (Upper 100 feet) |

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

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

| 1 | 2 | 3 | 4 | 5 | 6 | 7 | 8 | 9 | 10 | 11 | 12 | 13 | 14 | 15 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Bottom of Layer Depth (ft) | Blow Count*** | Type of Sampler | $d_i$ (feet) | $N_{60}$ (blows/ft) | N70 (blows/ft) | $N_{60HE}$ (blows/ft) | $V_{si}$** (m/sec) | $V_{si}$ (ft/sec) | $\Phi_i$ (degrees) | $d_i/N_{60HEi}$ | $d_i/V_{si}$ | $d_i/\Phi_i$ | Consistency if Coarse Grained (Based on ASTM and Corrected for N60) | Consistency if Fine Grained (Based on ASTM and Corrected for N60) |
| 2.5 | 79 | c | 2.5 | 44.79 | 38.39 | 59.72 | 329.04 | 1079.25 | 38.41 | 0.04186 | 0.00232 | 0.065095 | Dense | Hard |
| 5.0 | 24 | c | 2.5 | 13.61 | 11.66 | 18.14 | 232.92 | 763.96 | 31.80 | 0.13779 | 0.00327 | 0.078622 | Medium Dense | Stiff |
| 7.5 | 81 | c | 2.5 | 45.93 | 39.37 | 61.24 | 331.43 | 1087.11 | 38.57 | 0.04083 | 0.00230 | 0.064818 | Dense | Hard |
| 10.0 | 76 | c | 2.5 | 43.09 | 36.94 | 57.46 | 325.37 | 1067.20 | 38.15 | 0.04351 | 0.00234 | 0.065525 | Dense | Hard |
| 12.5 | 77 | c | 2.5 | 49.48 | 42.41 | 58.21 | 326.60 | 1071.26 | 38.24 | 0.04295 | 0.00233 | 0.06538 | Dense | Hard |
| 15.0 | 48 | c | 2.5 | 30.84 | 26.44 | 36.29 | 284.77 | 934.05 | 35.37 | 0.06889 | 0.00268 | 0.070684 | Dense | Hard |
| 20.0 | 54 | c | 5.0 | 38.78 | 33.24 | 40.82 | 294.67 | 966.51 | 36.05 | 0.12248 | 0.00517 | 0.138704 | Dense | Hard |
| 25.0 | 38 | c | 5.0 | 27.29 | 23.39 | 28.73 | 266.12 | 872.87 | 34.09 | 0.17405 | 0.00573 | 0.146688 | Medium Dense | Very Stiff |
| 30.0 | 62 | s | 5.0 | 89.28 | 76.53 | 74.40 | 350.69 | 1150.26 | 39.89 | 0.06720 | 0.00435 | 0.125352 | Very Dense | Hard |
| 35.0 | 67 | c | 5.0 | 50.65 | 43.42 | 50.65 | 313.69 | 1028.90 | 37.35 | 0.09871 | 0.00486 | 0.133858 | Very Dense | Hard |
| 40.0 | 64 | s | 5.0 | 92.16 | 78.99 | 76.80 | 353.93 | 1160.90 | 40.11 | 0.06510 | 0.00431 | 0.124658 | Very Dense | Hard |
| 45.0 | 61 | c | 5.0 | 46.12 | 39.53 | 46.12 | 305.27 | 1001.28 | 36.78 | 0.10842 | 0.00499 | 0.135959 | Dense | Hard |
| 47.5 | 78 | s | 2.5 | 112.32 | 96.27 | 93.60 | 374.83 | 1229.45 | 41.54 | 0.02671 | 0.00203 | 0.060183 | Very Dense | Hard |
| | | Total: | 47.5 | "d" Feet | | | | Total: | | 1.03850 | 0.04668 | 1.275526 | | |

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

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

Consistency classification based upon ASCE 1996

BOREHOLE, SAMPLER, AND ROD CORRECTION FACTORS

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

Adapted from Skempton (1986).

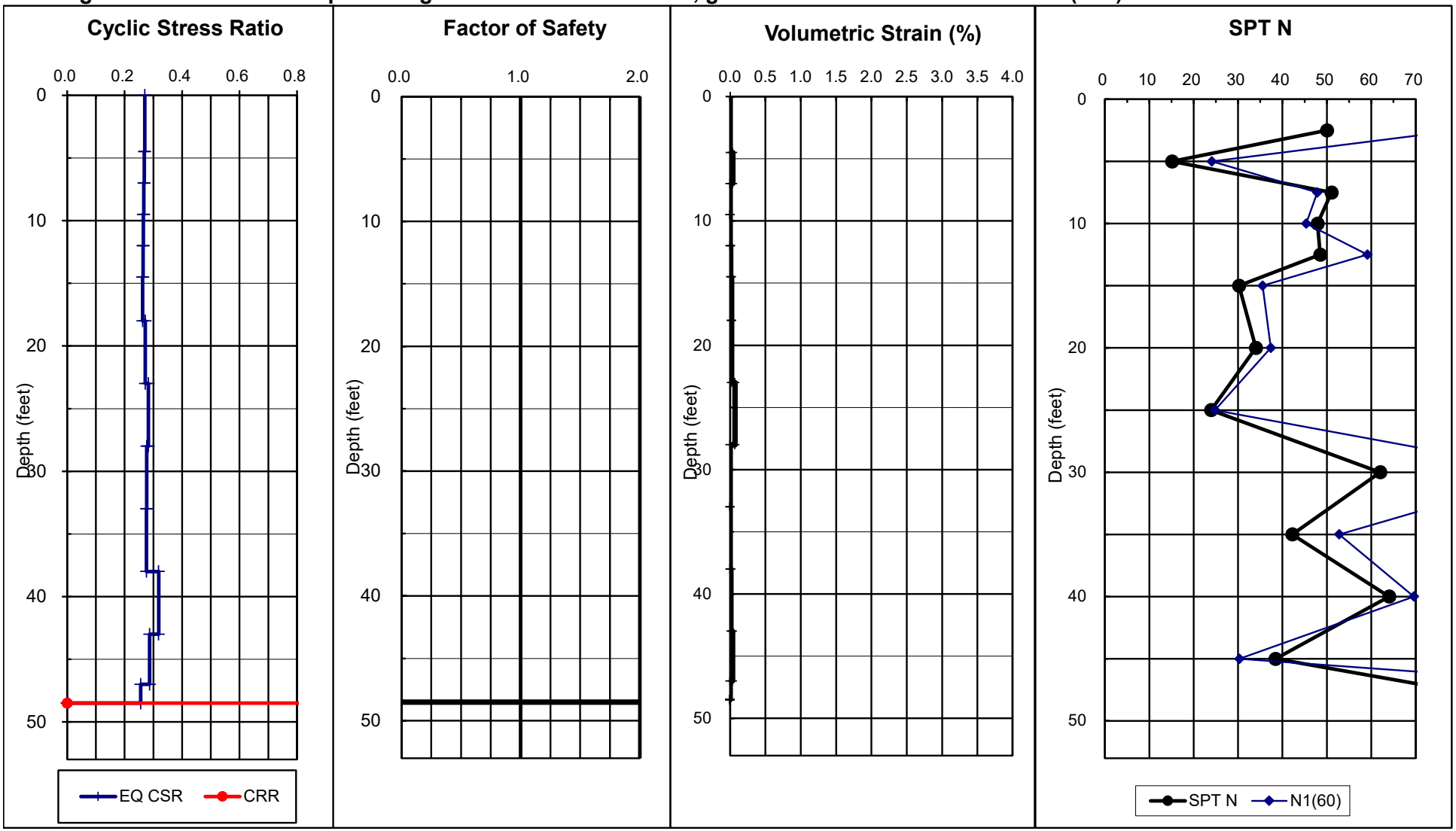
Spreadsheet Version 2.6, 2019: Prepared by Kevin L. Paul, PE, GE

**EARTH SYSTEMS - EVALUATION OF LIQUEFACTION POTENTIAL AND INDUCED SUBSIDENCE**

**Proposed Tractor Supply Company** **Project No: 306687-002** 1996/1998 NCEER Method

Ground Compaction Remediated to 5 foot depth

**Boring: B-1** **Earthquake Magnitude: 7.9** **PGA, g: 0.37** **Calc GWT (feet): 50**



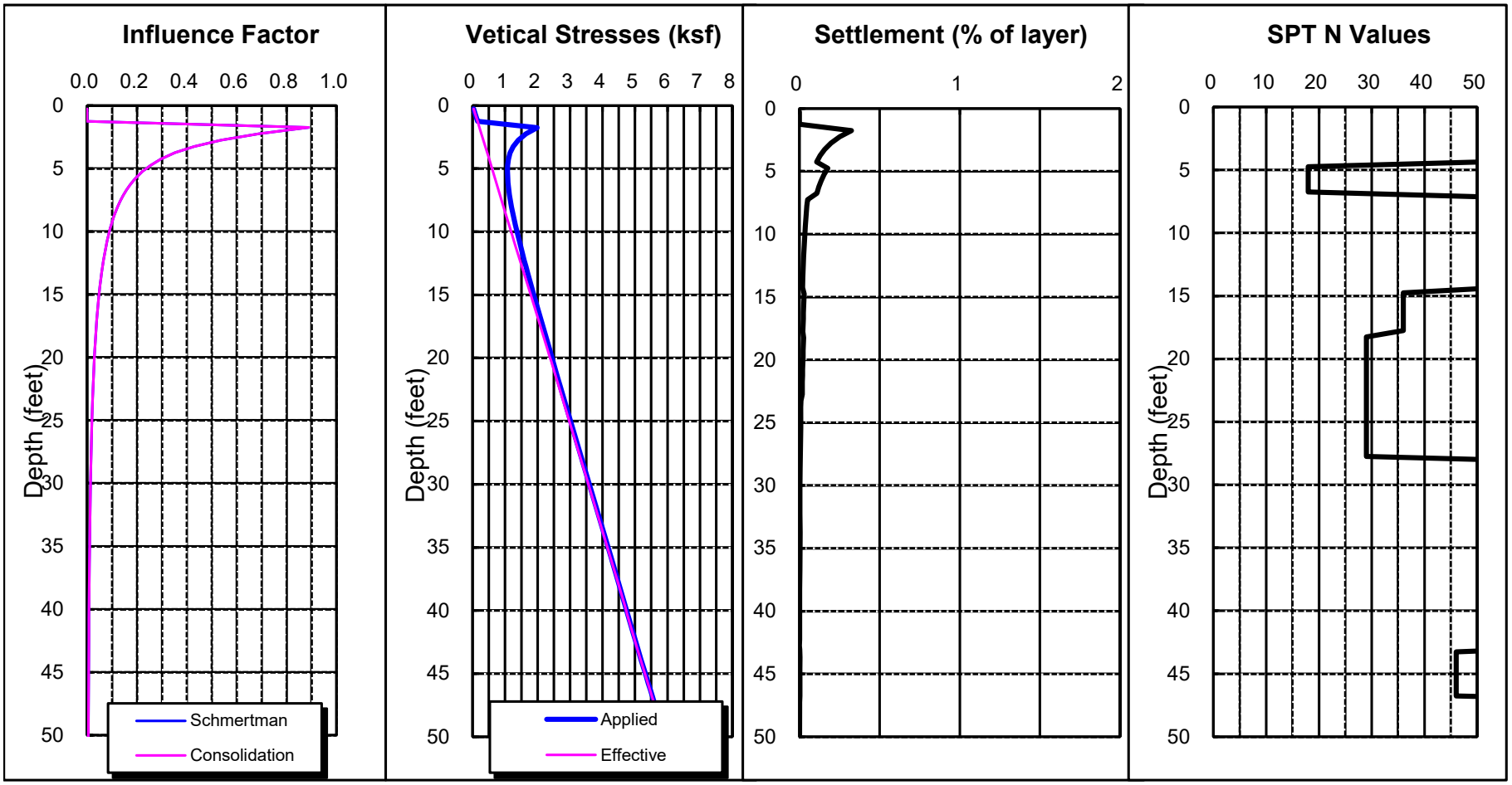

**Total Thickness of Liquefiable Layers: 0.0 feet** **Estimated Total Ground Subsidence: 0.1 inches**

**EARTH SYSTEMS - SETTLEMENT ANALYSES**

Proposed Tractor Supply Company

306687-002

Width, ft: 2.0 Length, ft: 20.0 Net pressure, ksf: 2.00 **Settlement, inches: 0.2**



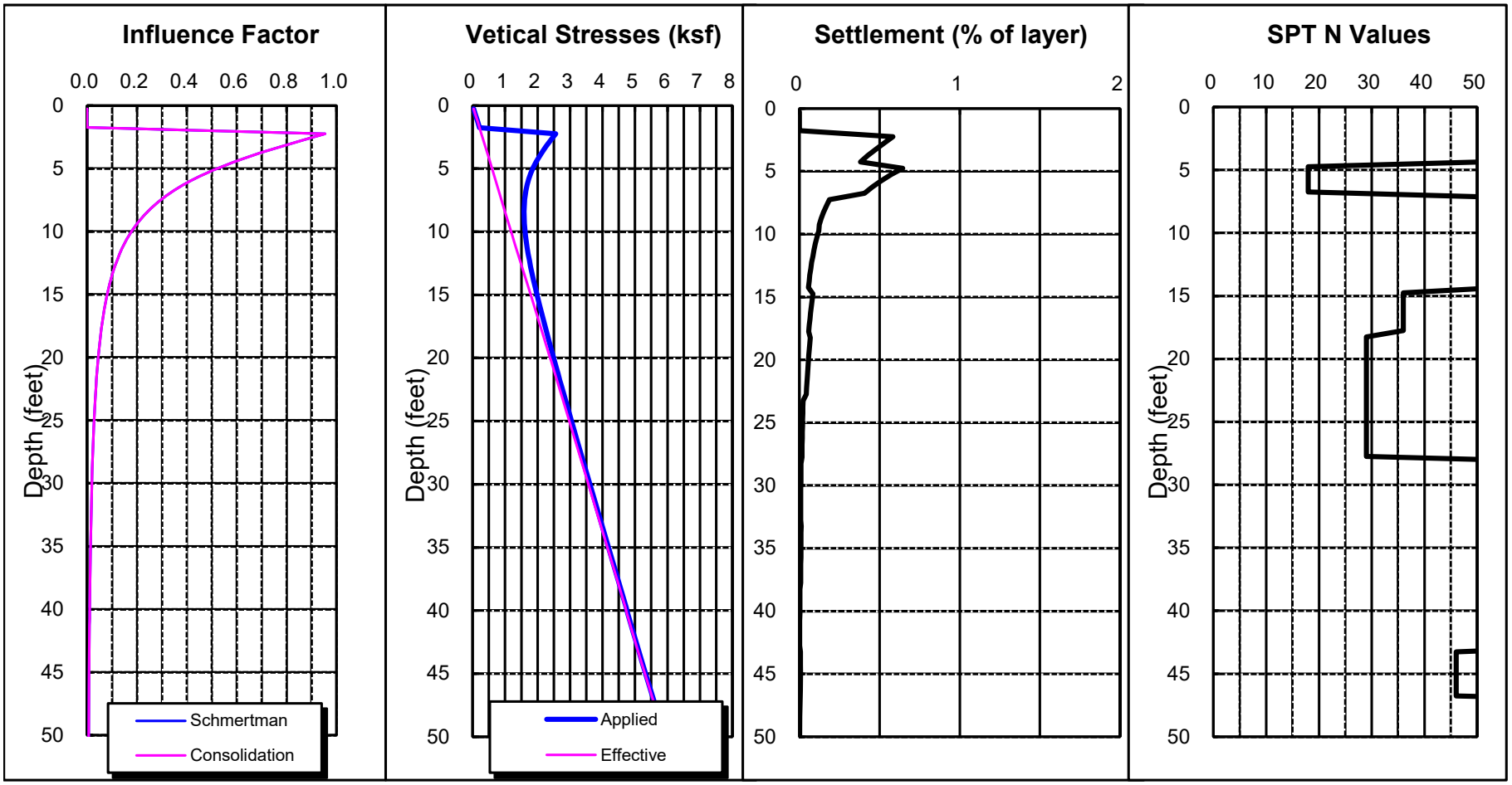

Load, Q: 4 kpf Embedment, feet: 1.5 **Boring: B-1**

**EARTH SYSTEMS - SETTLEMENT ANALYSES**

Proposed Tractor Supply Company

306687-002

Width, ft: 7.0 Length, ft: 7.0 Net pressure, ksf: 2.40 **Settlement, inches: 0.5**




Load, Q: 118 kips Embedment, feet: 2.0 **Boring: B-1**

**APPENDIX B**

Laboratory Test Results

File No.: 306687-002 July 26, 2024

Lab No.: 24-283

**UNIT DENSITIES AND MOISTURE CONTENT** ASTM D2937 & D2216

Job Name: Proposed Tractor Supply Company

| Sample Location | Depth (feet) | Unit Dry Density (pcf) | Moisture Content (%) | USCS Group Symbol |
|---|---|---|---|---|
| B-1 | 2.5 | 116 | 3 | SM |
| B-1 | 5 | 113 | 3 | SP |
| B-1 | 7.5 | 112 | 6 | ML |
| B-1 | 10 | 114 | 5 | ML |
| B-1 | 12.5 | 114 | 6 | SM |
| B-1 | 20 | 115 | 3 | SM |
| B-1 | 30 | 111 | 6 | ML |
| B-1 | 40 | 111 | 2 | SP |
| B-1 | 45 | --- | 8 | SM |
| B-1 | 47.5 | 110 | 5 | ML |
| B-2 | 2.5 | 96 | 4 | SM |
| B-2 | 5 | 105 | 6 | SM |
| B-2 | 7.5 | 119 | 2 | SM |
| B-2 | 10 | 99 | 5 | ML |
| B-3 | 2.5 | 118 | 4 | SM |
| B-3 | 5 | 122 | 3 | SM |
| B-3 | 7.5 | 124 | 2 | SM |
| B-3 | 10 | 118 | 4 | SM |
| B-4 | 2.5 | 112 | 5 | ML |
| B-4 | 5 | 116 | 4 | ML |
| B-5 | 2.5 | 112 | 2 | SP |
| B-5 | 5 | 116 | 3 | SM |

File No.: 306687-002 July 26, 2024

Lab No.: 24-283

**UNIT DENSITIES AND MOISTURE CONTENT** ASTM D2937 & D2216

Job Name: Proposed Tractor Supply Company

| Sample Location | Depth (feet) | Unit Dry Density (pcf) | Moisture Content (%) | USCS Group Symbol |
|---|---|---|---|---|
| B-6 | 2.5 | 115 | 3 | SM |
| B-6 | 5 | 109 | 7 | ML |
| B-6 | 7.5 | 111 | 6 | ML |
| B-6 | 10 | 112 | 6 | ML |
| B-7 | 2.5 | 100 | 6 | ML |
| B-7 | 5 | 100 | 4 | ML |
| B-8 | 2.5 | 105 | 3 | SM |
| B-8 | 5 | 113 | 3 | SM |
| B-8 | 7.5 | 111 | 2 | SP |
| B-8 | 10 | 105 | 7 | ML |
| B-9 | 2.5 | 100 | 5 | SM |
| B-9 | 5 | 83 | 6 | ML |
| B-9 | 7.5 | 111 | 2 | SP |
| B-9 | 10 | 108 | 2 | ML |

File No.: 306687-002 7/26/2024

Lab No.: 24-283

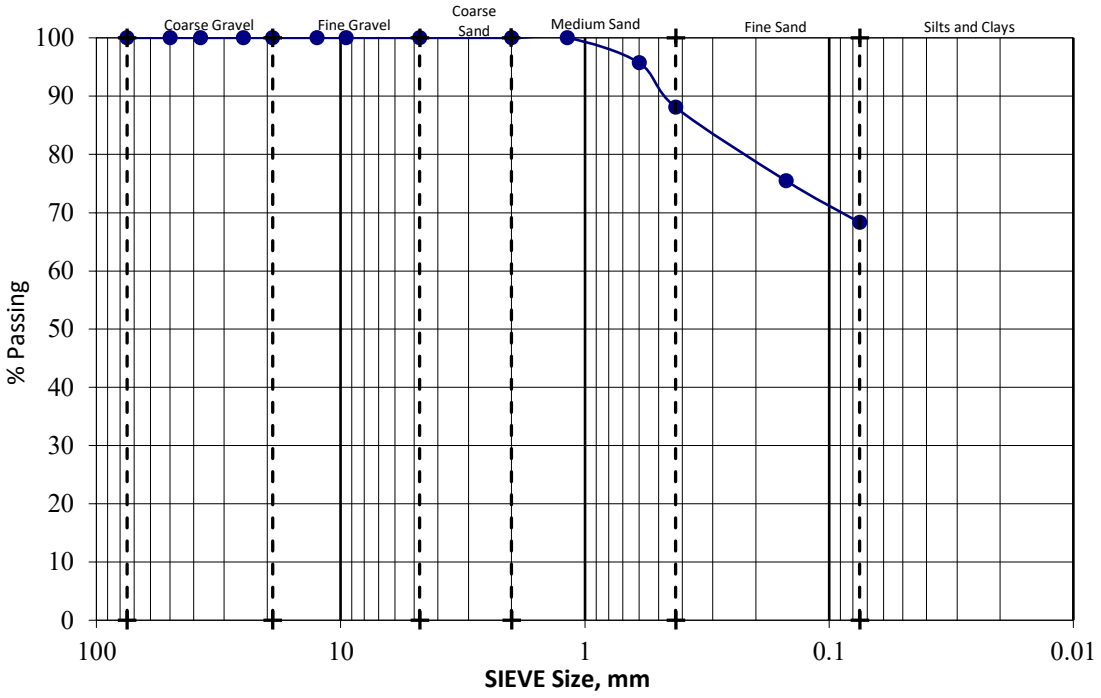
**SIEVE ANALYSIS** ASTM D6913

Job Name: Proposed Tractor Supply Company

Sample ID: B-7 @ 0-5'

Description: Sandy Silt (ML)

| 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 | 100 |
| #16 | 100 |
| #30 | 96 |
| #40 | 88 |
| #100 | 75 |
| #200 | 68.3 |




**FM= 0.47**

| | | | | | | |
|---|---|---|---|---|---|---|
| % Coarse Gravel: | 0 | % Coarse Sand: | 0 | | | |
| % Fine Gravel: | 0 | % Medium Sand: | 12 | Cu: | NA | |
| | | % Fine Sand: | 20 | Cc: | NA | Gradation |
| % Total Gravel | 0 | % Total Sand | 32 | % Fines: | 68.3 | NA |

File No.: 306687-002 July 26, 2024

Job Name: Proposed Tractor Supply Company

Lab Number: 24-283

**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-4 | 5 | 61.5 | ML | 10 |

File No.: 306687-002
Lab No.: 24-283

July 26, 2024

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

Proposed Tractor Supply Company
B-1 @ 5'

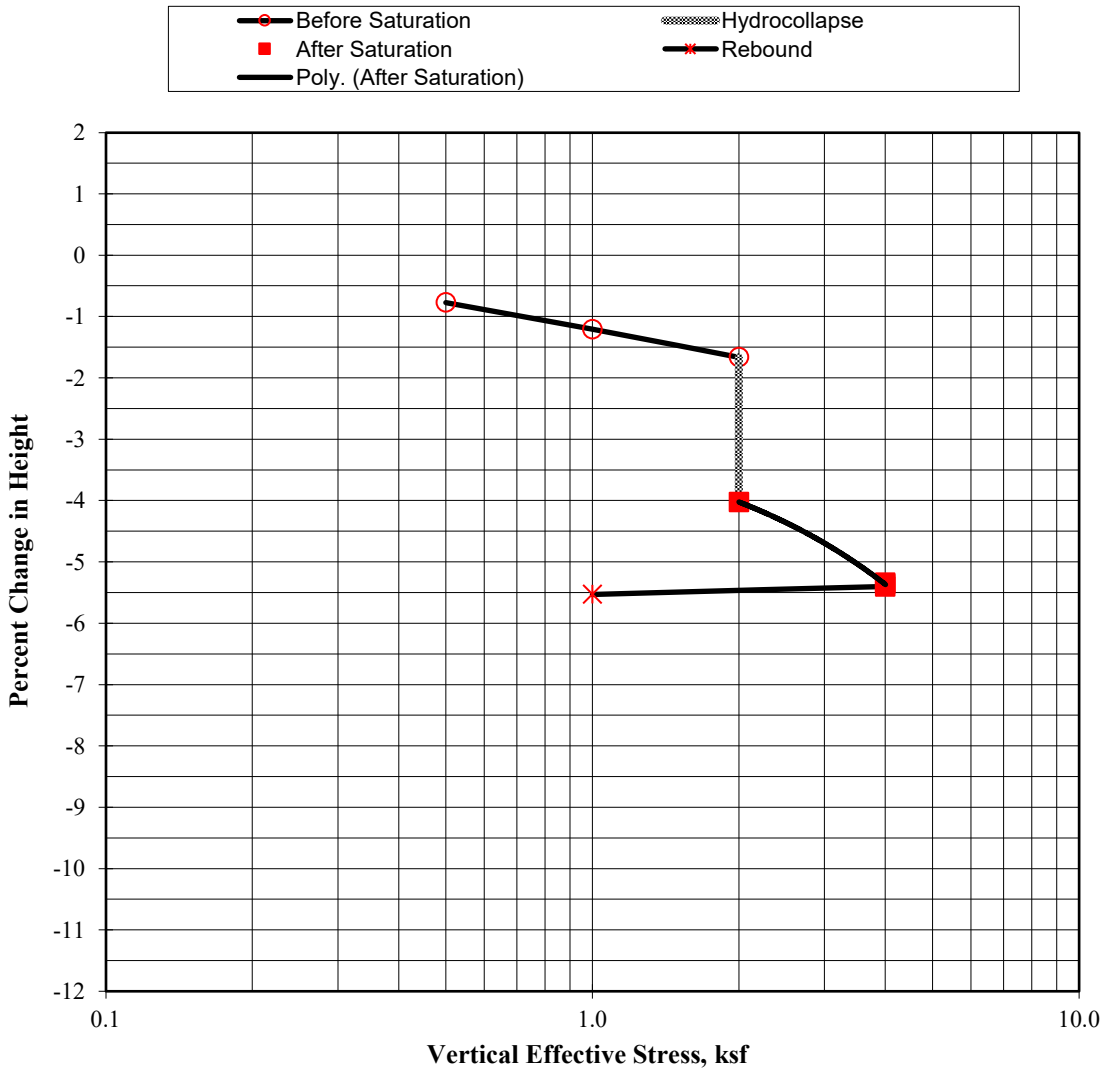
Sand (SP)

Ring Sample

Initial Dry Density: 112.0 pcf
Initial Moisture: 3.4%
Specific Gravity: 2.67
Initial Void Ratio: 0.489

Hydrocollapse: 2.4% @ 2.0 ksf

**% Change in Height vs Normal Pressure Diagram**



File No.: 306687-002
Lab No.: 24-283

July 26, 2024

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

Proposed Tractor Supply Company
B-1 @ 7.5'

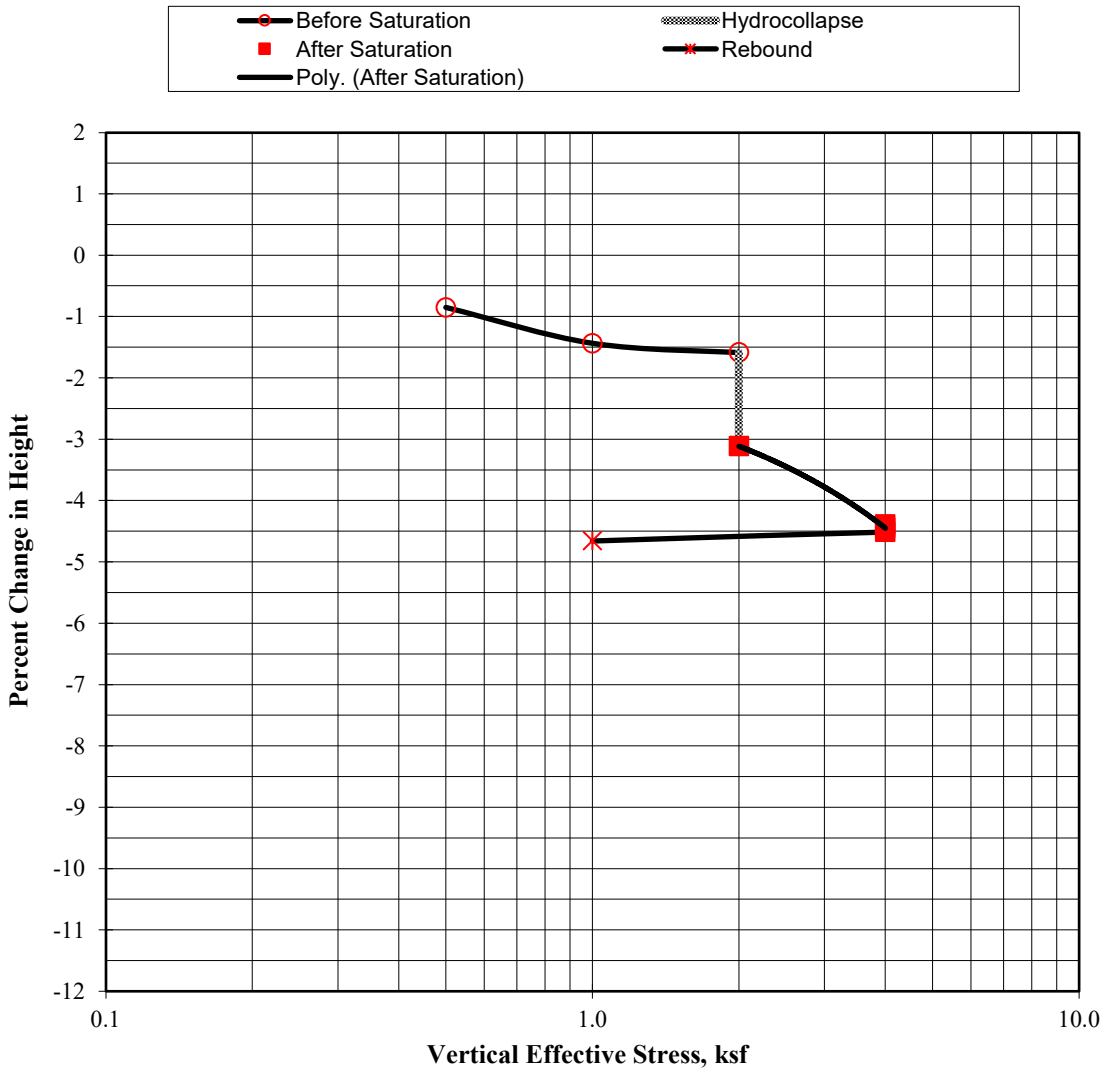
Sandy Silt (ML)

Ring Sample

Initial Dry Density: 115.2 pcf
Initial Moisture: 3.1%
Specific Gravity: 2.67
Initial Void Ratio: 0.447

Hydrocollapse: 1.5% @ 2.0 ksf

**% Change in Height vs Normal Pressure Diagram**



File No.: 306687-002
Lab No.: 24-283

July 26, 2024

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

Proposed Tractor Supply Company
B-1 @ 10'

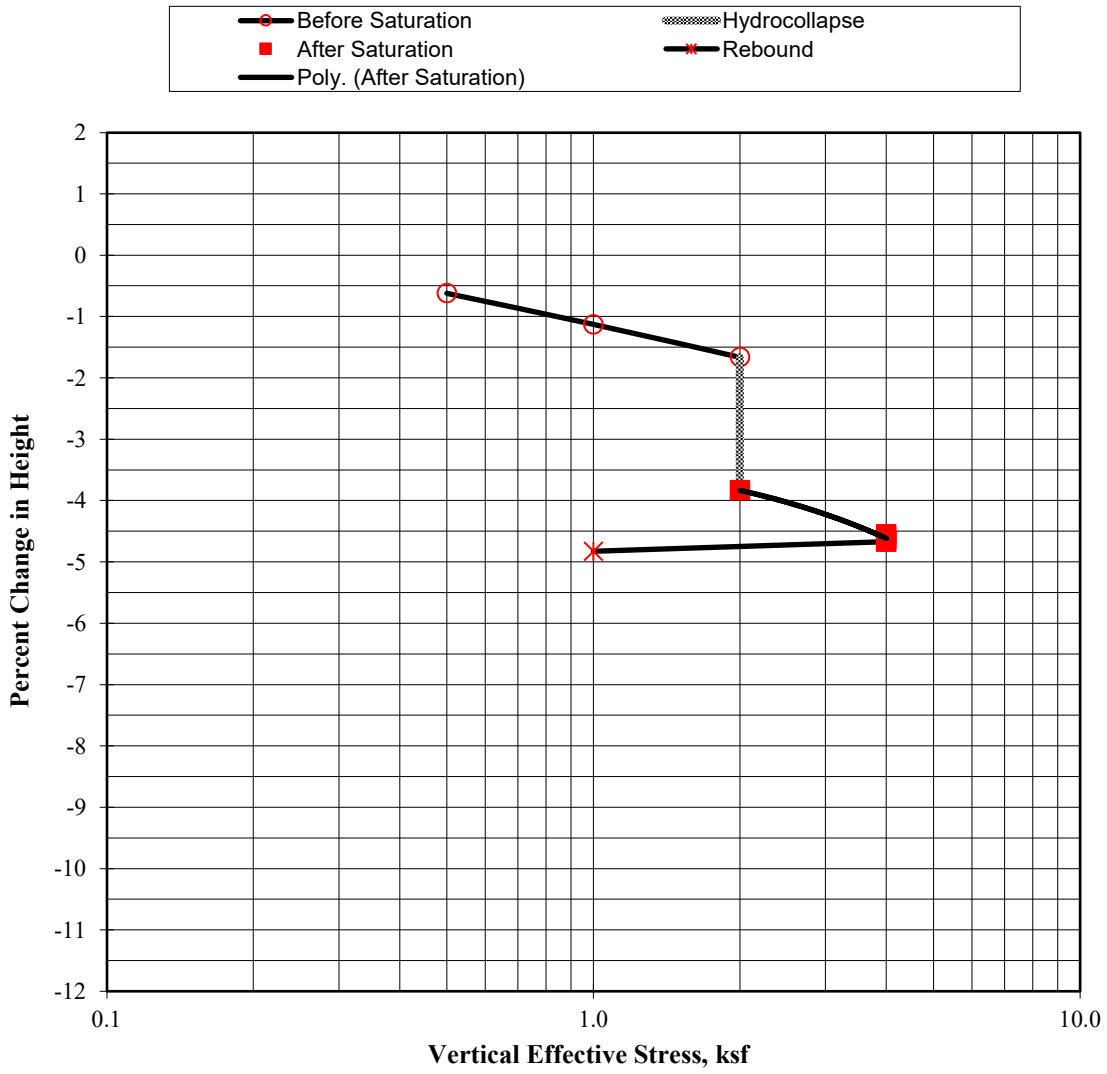
Sandy Silt (ML)

Ring Sample

Initial Dry Density: 113.6 pcf
Initial Moisture: 6.2%
Specific Gravity: 2.67
Initial Void Ratio: 0.468

Hydrocollapse: 2.2% @ 2.0 ksf

**% Change in Height vs Normal Pressure Diagram**



File No.: 306687-002
Lab No.: 24-283

July 26, 2024

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

Proposed Tractor Supply Company
B-1 @ 12.5'

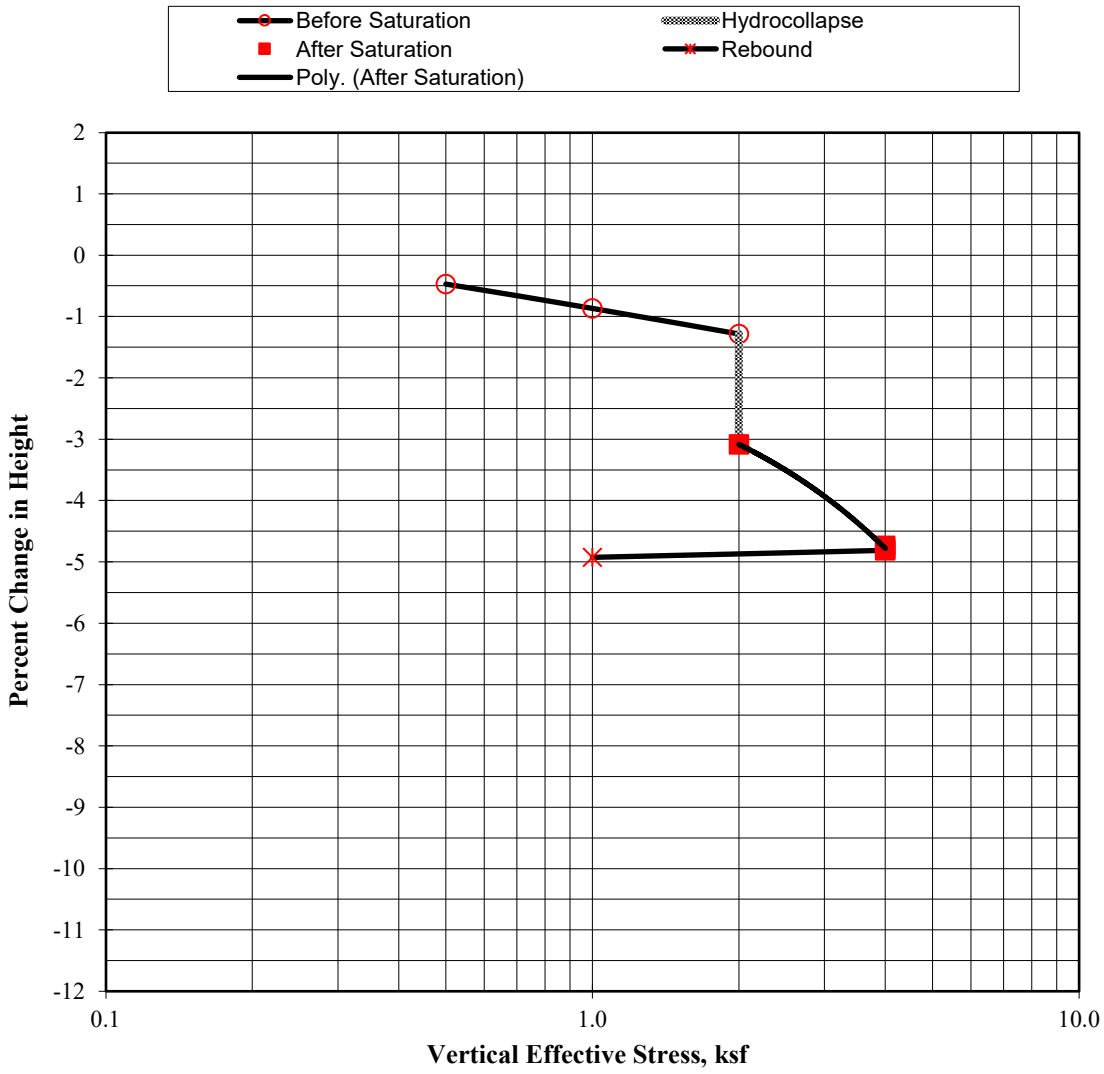
Silty Sand (SM)

Ring Sample

Initial Dry Density: 115.8 pcf
Initial Moisture: 4.8%
Specific Gravity: 2.67
Initial Void Ratio: 0.440

Hydrocollapse: 1.8% @ 2.0 ksf

**% Change in Height vs Normal Pressure Diagram**



File No.: 306687-002
Lab No.: 24-283

July 26, 2024

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

Proposed Tractor Supply Company
B-2 @ 5'

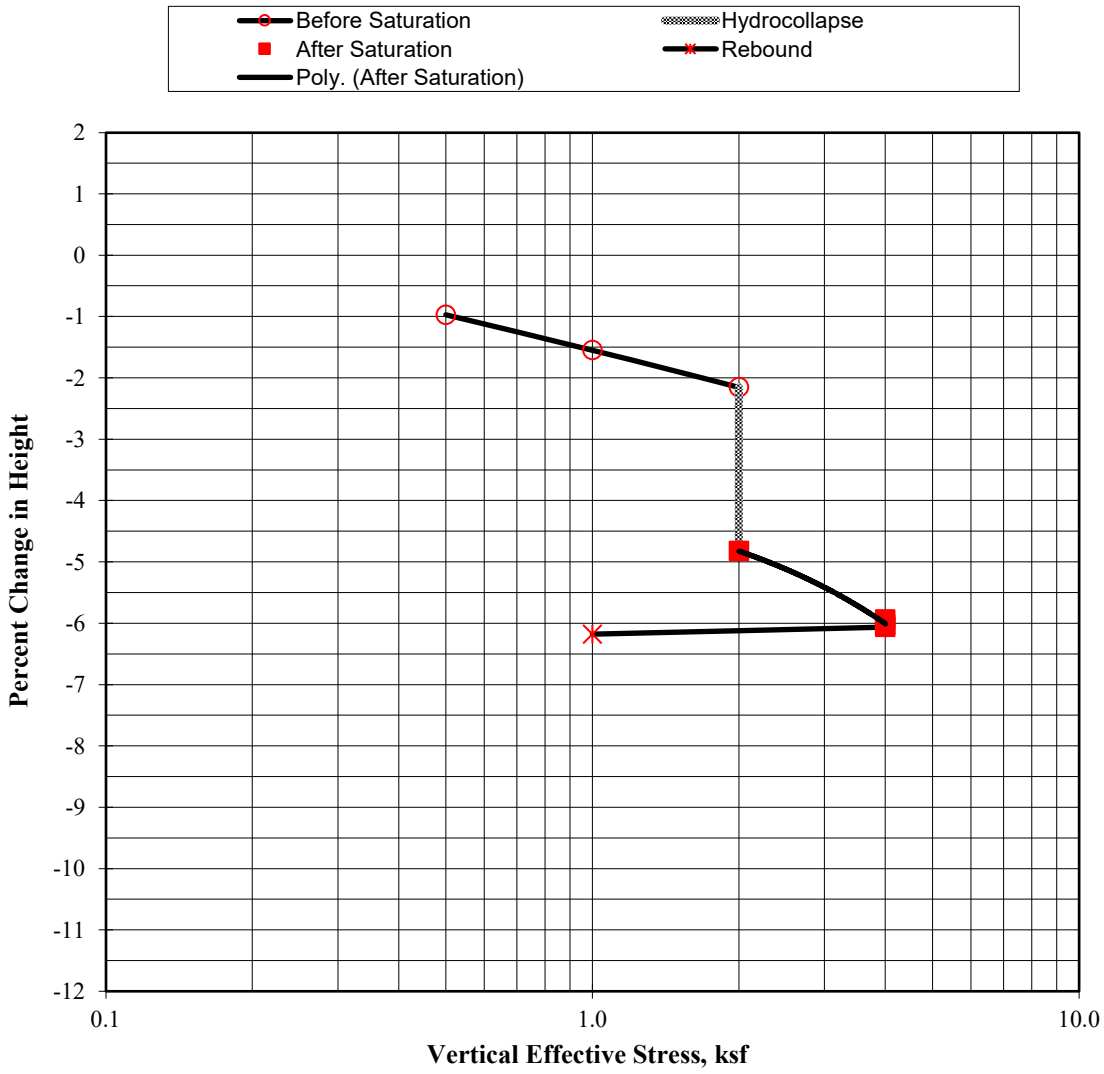
Silty Sand (SM)

Ring Sample

Initial Dry Density: 108.3 pcf
Initial Moisture: 2.9%
Specific Gravity: 2.67
Initial Void Ratio: 0.539

Hydrocollapse: 2.7% @ 2.0 ksf

**% Change in Height vs Normal Pressure Diagram**



File No.: 306687-002
Lab No.: 24-283

July 26, 2024

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

Proposed Tractor Supply Company
B-4 @ 2.5'

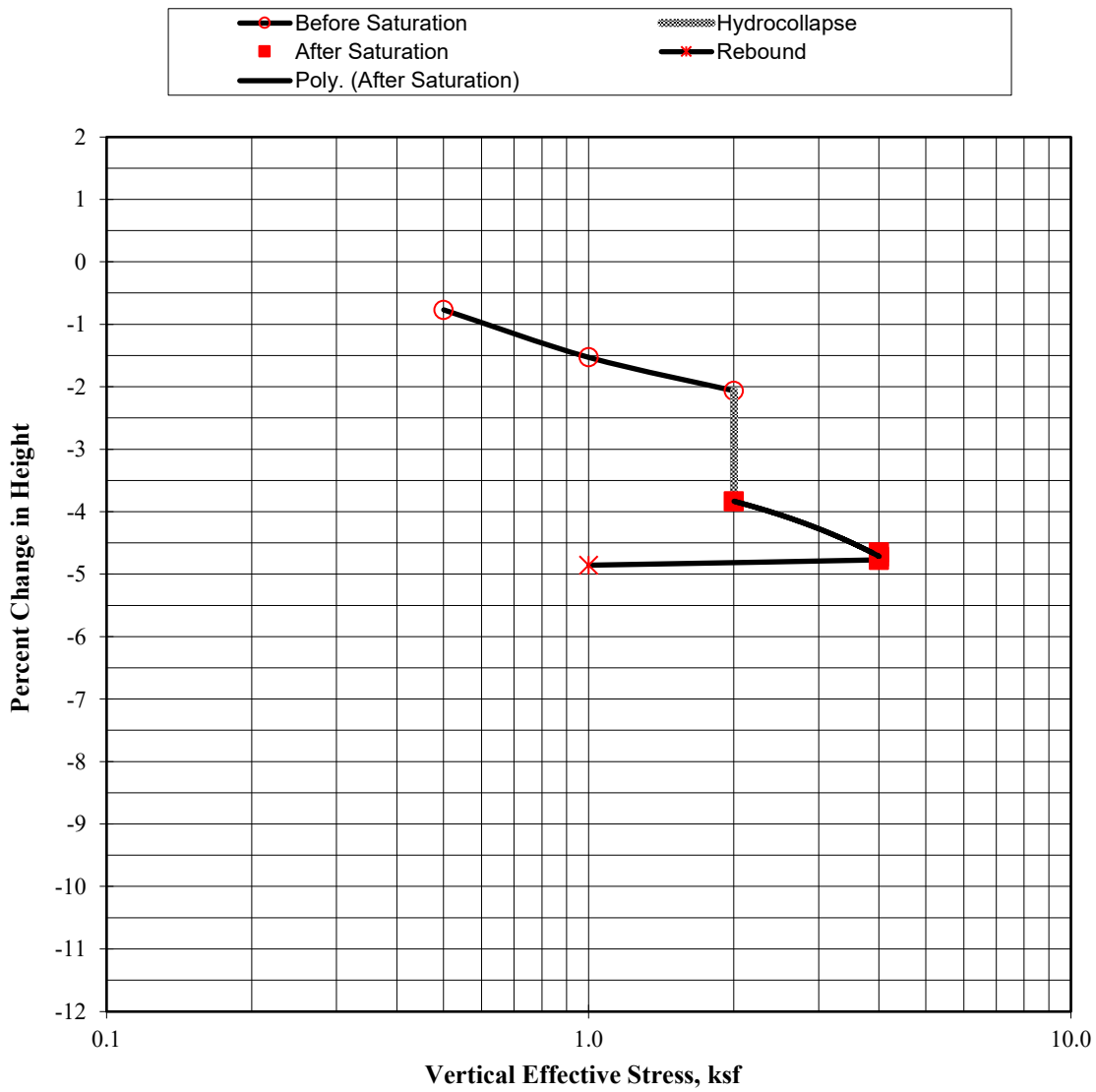
Sandy Silt (ML)

Ring Sample

Initial Dry Density: 113.7 pcf
Initial Moisture: 4.4%
Specific Gravity: 2.67
Initial Void Ratio: 0.466

Hydrocollapse: 1.8% @ 2.0 ksf

## % Change in Height vs Normal Pressure Diagram



File No.: 306687-002
Lab No.: 24-283

July 26, 2024

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

Proposed Tractor Supply Company
B-5 @ 5'

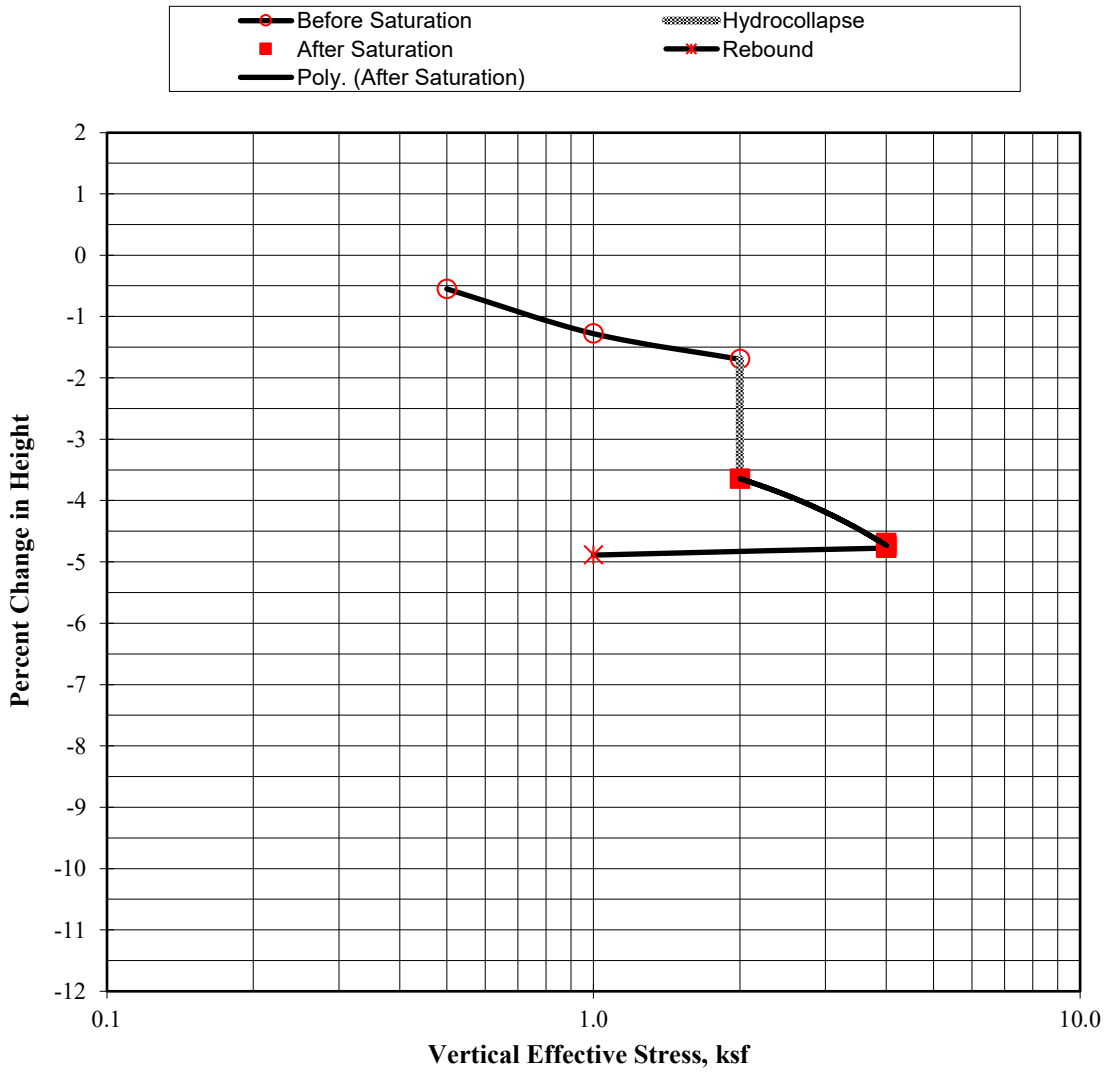
Silty Sand (SM)

Ring Sample

Initial Dry Density: 115.9 pcf
Initial Moisture: 4.4%
Specific Gravity: 2.67
Initial Void Ratio: 0.439

Hydrocollapse: 2.0% @ 2.0 ksf

**% Change in Height vs Normal Pressure Diagram**



File No.: 306687-002
Lab No.: 24-283

July 26, 2024

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

Proposed Tractor Supply Company
B-6 @ 2.5'

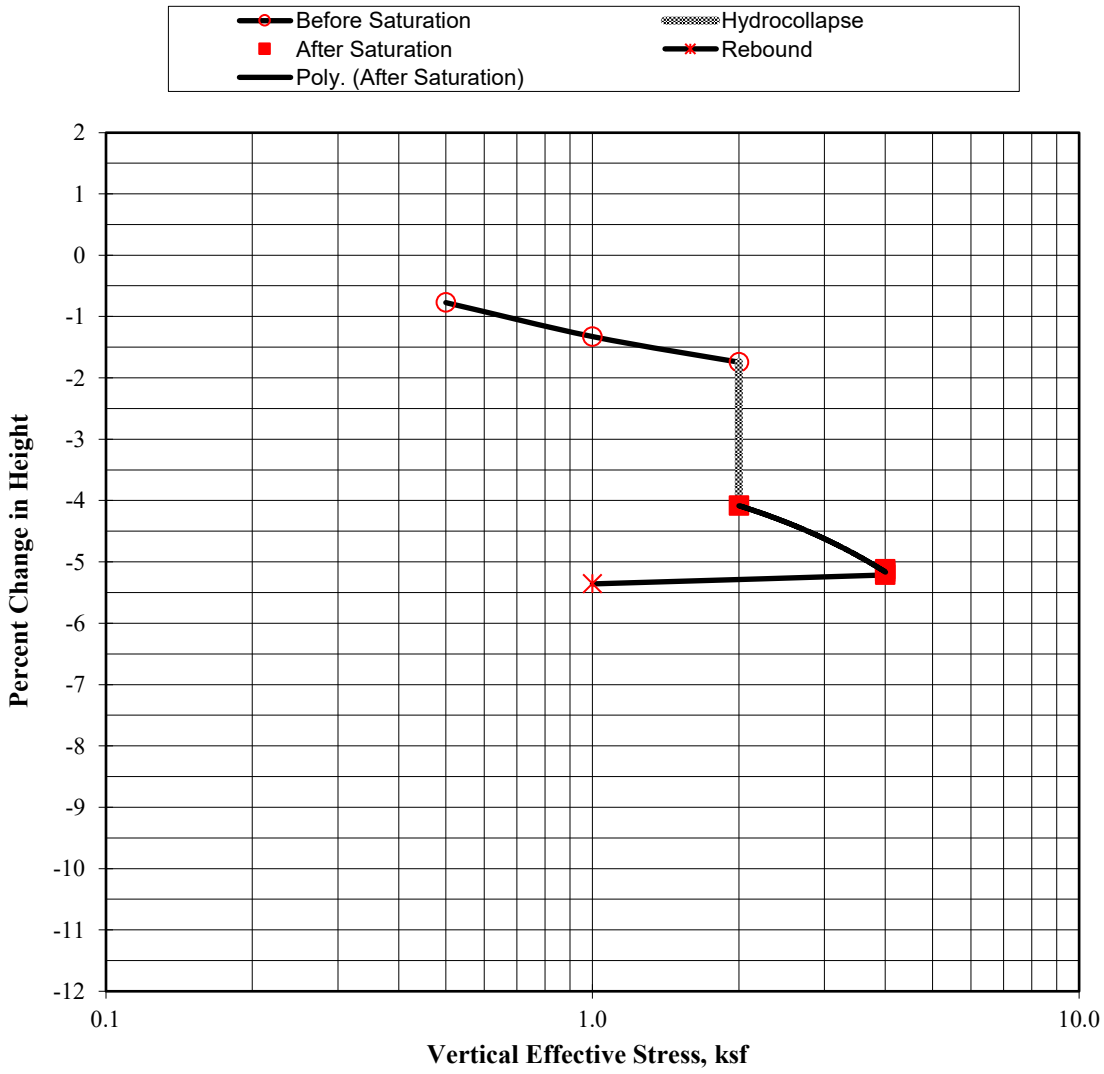
Silty Sand (SM)

Ring Sample

Initial Dry Density: 114.7 pcf
Initial Moisture: 3.6%
Specific Gravity: 2.67
Initial Void Ratio: 0.453

Hydrocollapse: 2.3% @ 2.0 ksf

**% Change in Height vs Normal Pressure Diagram**



File No.: 306687-002
Lab No.: 24-283

July 26, 2024

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

Proposed Tractor Supply Company
B-6 @ 10'

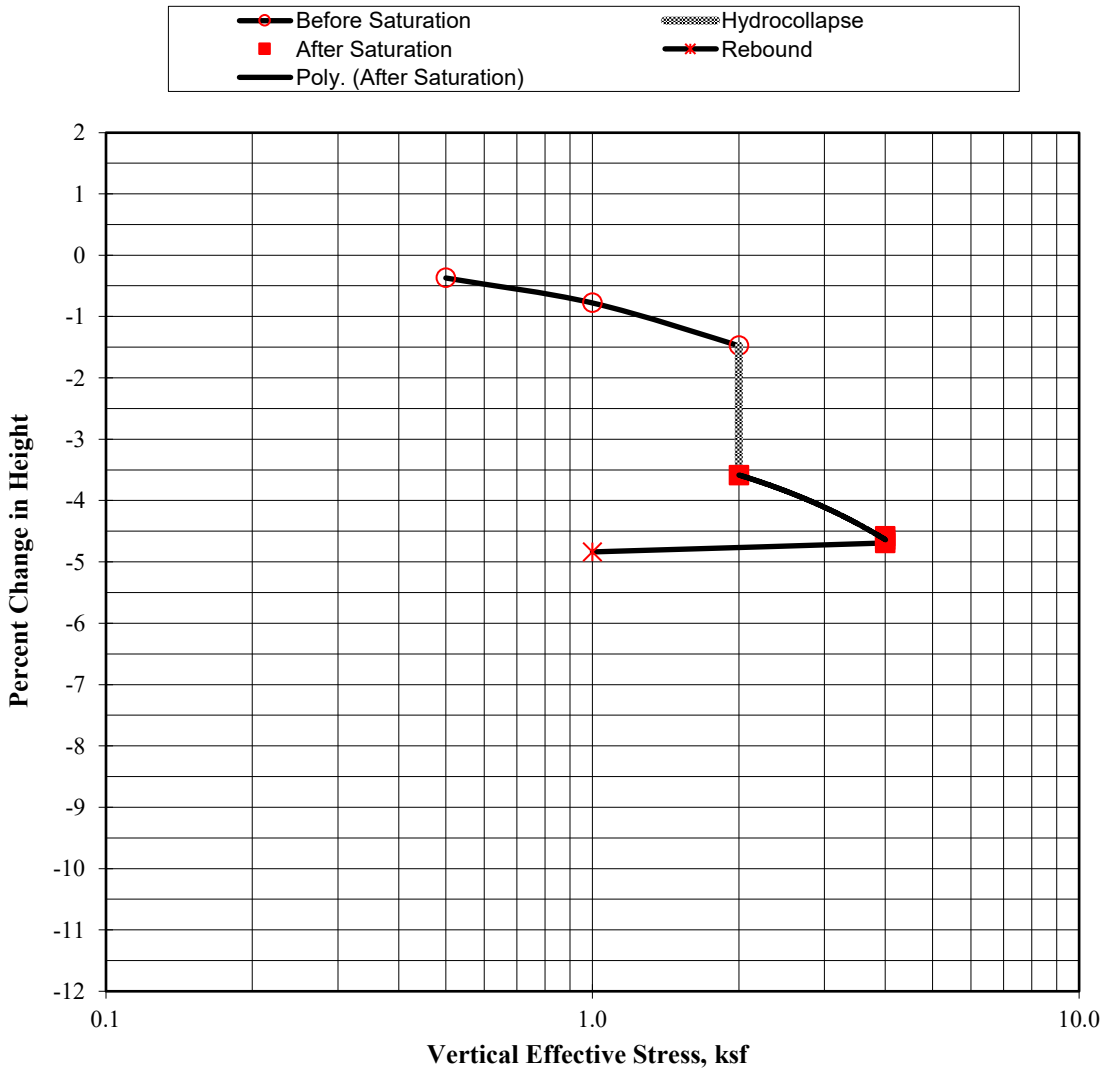
Sandy Silt (ML)

Ring Sample

Initial Dry Density: 116.9 pcf
Initial Moisture: 3.8%
Specific Gravity: 2.67
Initial Void Ratio: 0.425

Hydrocollapse: 2.1% @ 2.0 ksf

**% Change in Height vs Normal Pressure Diagram**



File No.: 306687-002
Lab No.: 24-283

July 26, 2024

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

Proposed Tractor Supply Company
B-9 @ 7.5'

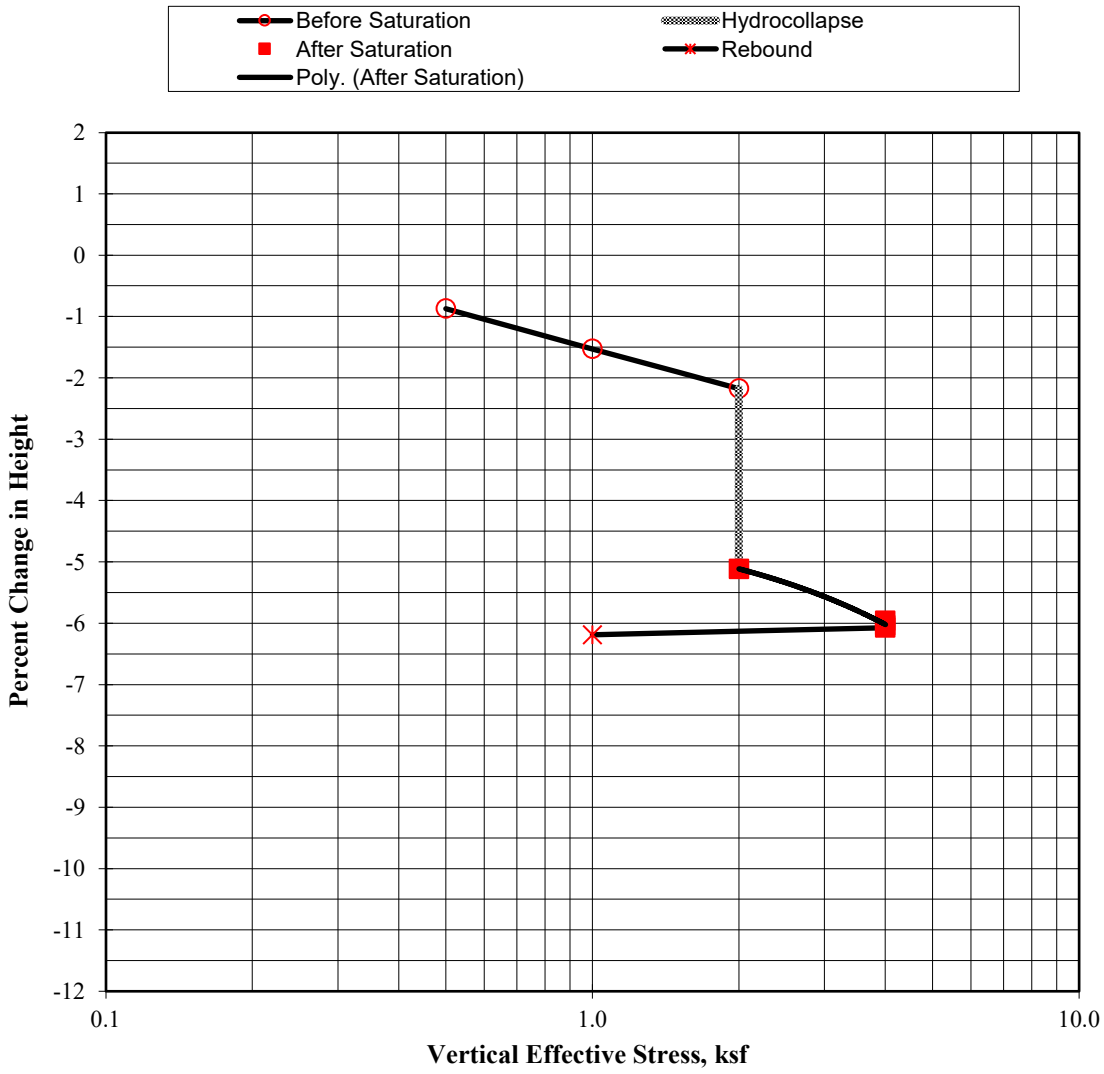
Sand (SP)

Ring Sample

Initial Dry Density: 109.7 pcf
Initial Moisture: 3.2%
Specific Gravity: 2.67
Initial Void Ratio: 0.520

Hydrocollapse: 2.9% @ 2.0 ksf

**% Change in Height vs Normal Pressure Diagram**



File No.: 306687-002 July 26, 2024
Lab No.: 24-283

**EXPANSION INDEX** **ASTM D-4829**

Job Name: Proposed Tractor Supply Company
Sample ID: B-1 @ 0-4'
Soil Description: Silty Sand (SM)

| | | |
|---|---|---|
| Initial Moisture, %: | 11.1 | |
| Initial Compacted Dry Density, pcf: | 103.4 | |
| Initial Saturation, %: | 48 | |
| Final Moisture, %: | 24.8 | |
| Volumetric Swell, %: | 0.4 | |
| **Expansion Index, EI:** | **3** | **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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July 26, 2024

**MAXIMUM DRY DENSITY / OPTIMUM MOISTURE**

ASTM D 1557 (Modified)

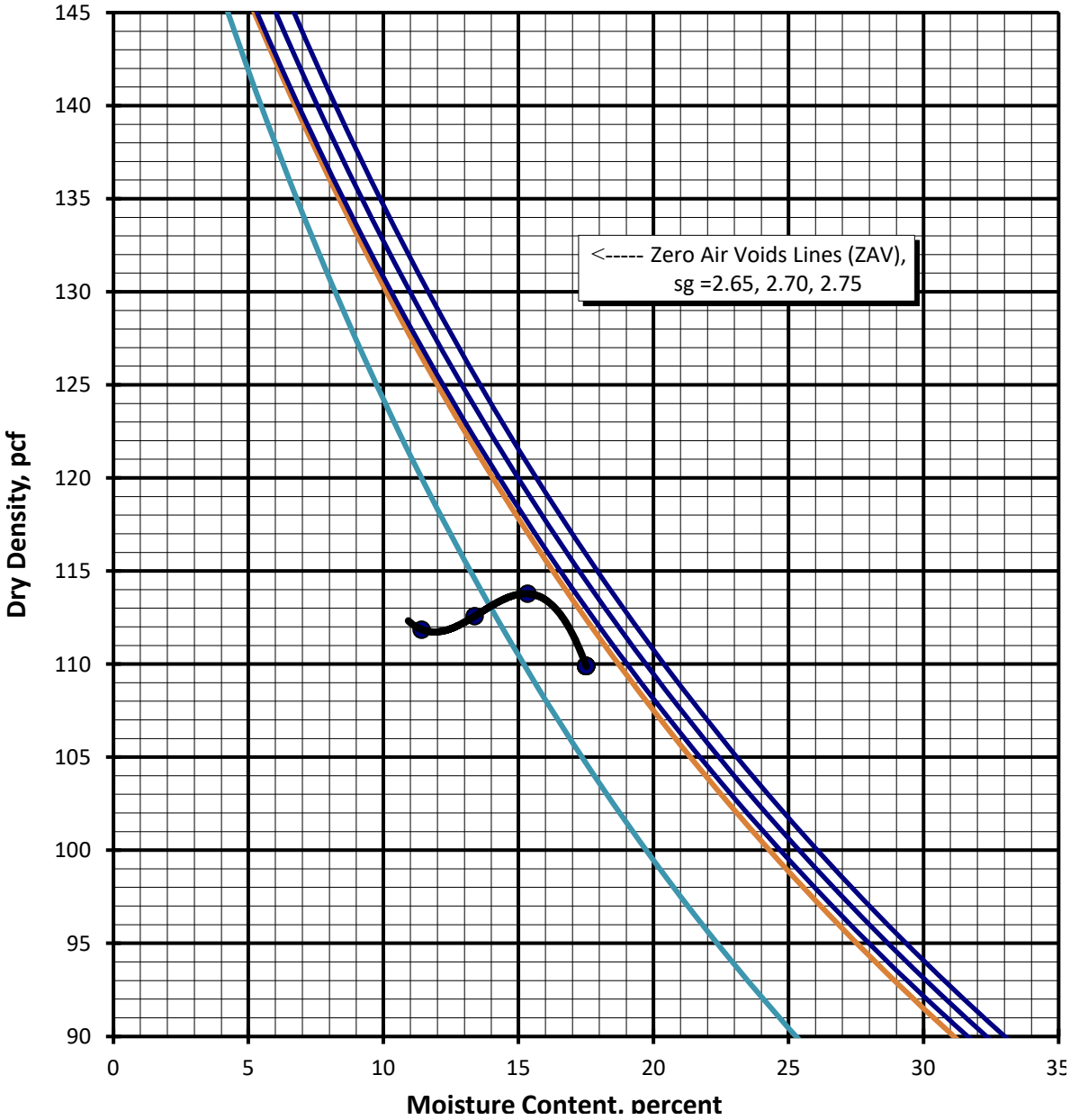
Job Name: Proposed Tractor Supply Company
Sample ID: 0
Location: B-1 @ 0-4'

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

Description: Silty Sand (SM)

**Maximum Dry Density: 113.8 pcf**
**Optimum Moisture: 15.3%**

| Sieve Size | % Retained (Cumulative) |
|---|---|
| 3/4" | 0.0 |
| 3/8" | 0.3 |
| #4 | 0.7 |



File No.: 306687-002 July 26, 2024
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**MAXIMUM DRY DENSITY / OPTIMUM MOISTURE** ASTM D 1557 (Modified)

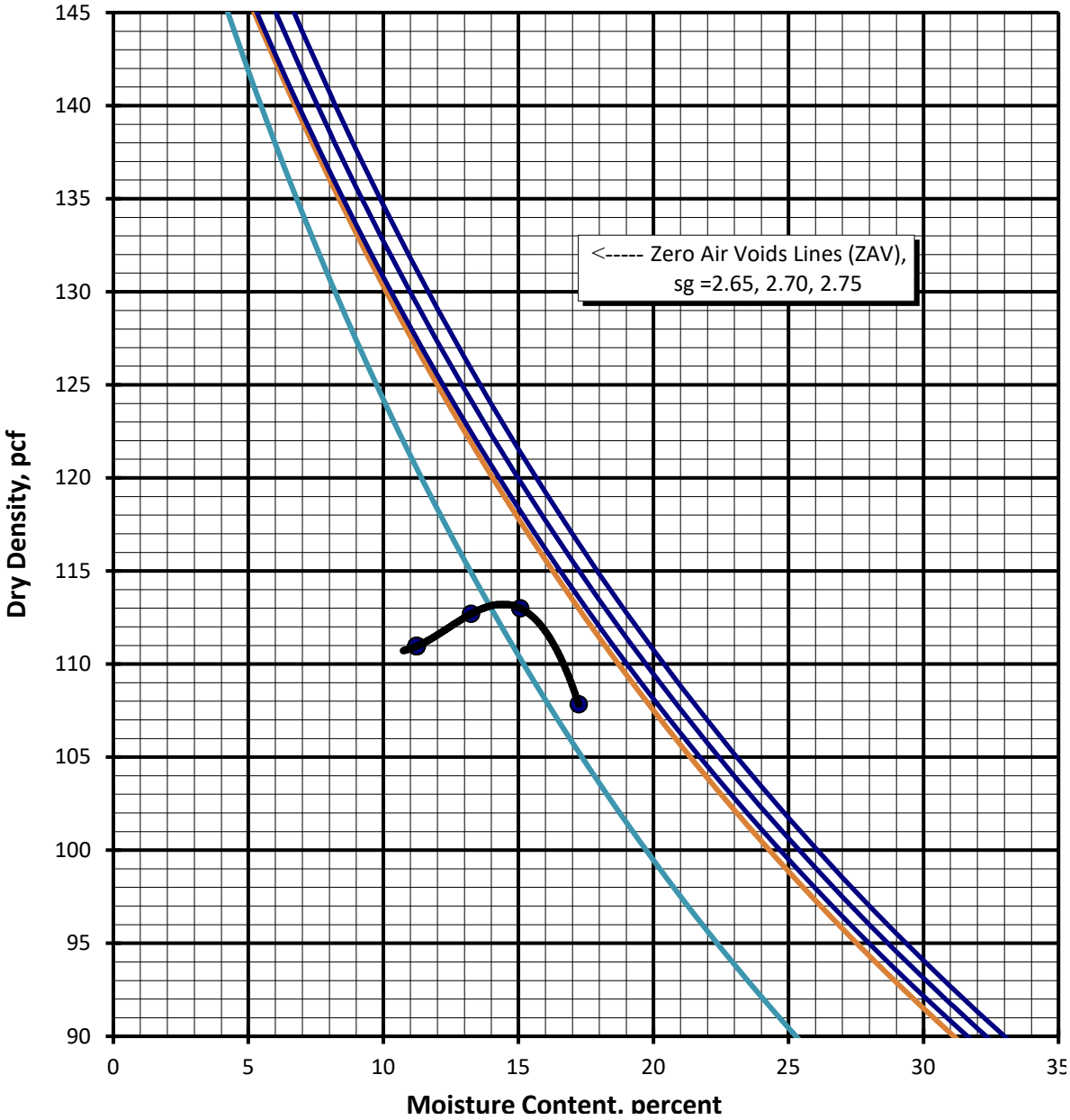
Job Name: Proposed Tractor Supply Company
Sample ID: 0
Location: B-7 @ 0-5'

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

Description: Sandy Silt (ML)

**Maximum Dry Density: 113.2 pcf**
**Optimum Moisture: 14.4%**

| Sieve Size | % Retained (Cumulative) |
|---|---|
| 3/4" | 0.0 |
| 3/8" | 0.2 |
| #4 | 2.4 |



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## SOIL CHEMICAL ANALYSES

| | |
|---|---|
| Job Name: | Proposed Tractor Supply Company |
| Job No.: | 306687-002 |
| Sample ID: | |
| Sample Location: | B-1 @ 0-4' |
| **Resistivity (Units)** | |
| as-received (ohm-cm) | >4,400,000 |
| saturated (ohm-cm) | 3,680 |
| **pH** | 8.4 |
| **Electrical Conductivity** (mS/cm) | 0.14 |
| **Chemical Analyses** | |
| **Cations** | |
| calcium $Ca^{2+}$ (mg/kg) | 122 |
| magnesium $Mg^{2+}$ (mg/kg) | 5 |
| sodium $Na^{1+}$ (mg/kg) | 46 |
| potassium $K^{1+}$ (mg/kg) | 4 |
| ammonium NH41+ (mg/kg) | ND |
| **Anions** | |
| carbonate $CO_3^{2-}$ (mg/kg) | 14 |
| bicarbonate $HCO_3^{1-}$(mg/kg) | 226 |
| fluoride $F^{1-}$ (mg/kg) | 13 |
| chloride $Cl^{1-}$ (mg/kg) | 5 |
| sulfate $SO_4^{2-}$ (mg/kg) | 96 |
| nitrate NO31- (mg/kg) | 3 |
| phosphate $PO_4^{3-}$ (mg/kg) | ND |
| **Other Tests** | |
| % moisture $H_2O$ (%) | na |
| total acidity $H^{1+}$ (mml/kg) | na |
| sulfide $S^{2-}$ (qual) | na |
| Redox (mV) | na |

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

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

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

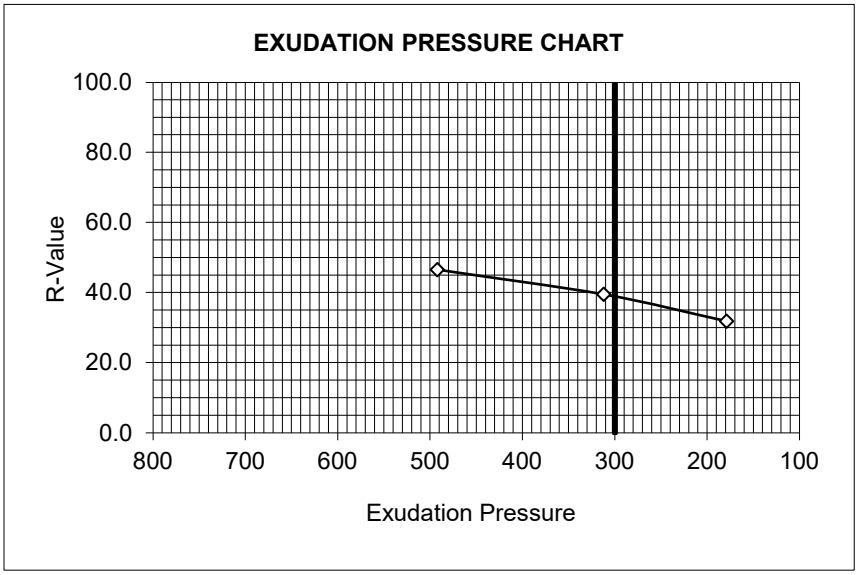
**General Guidelines for Soil Corrosivity**

| Chemical Agent | Amount in Soil | Degree of Corrosivity |
|---|---|---|
| Soluble 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.**

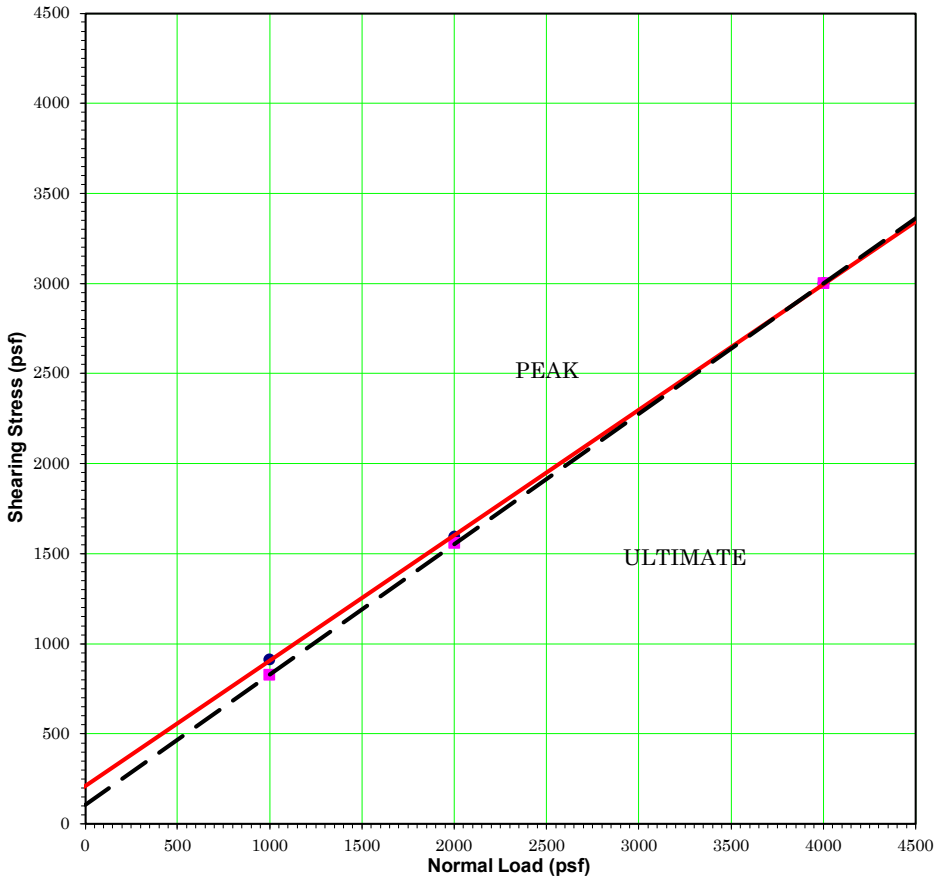


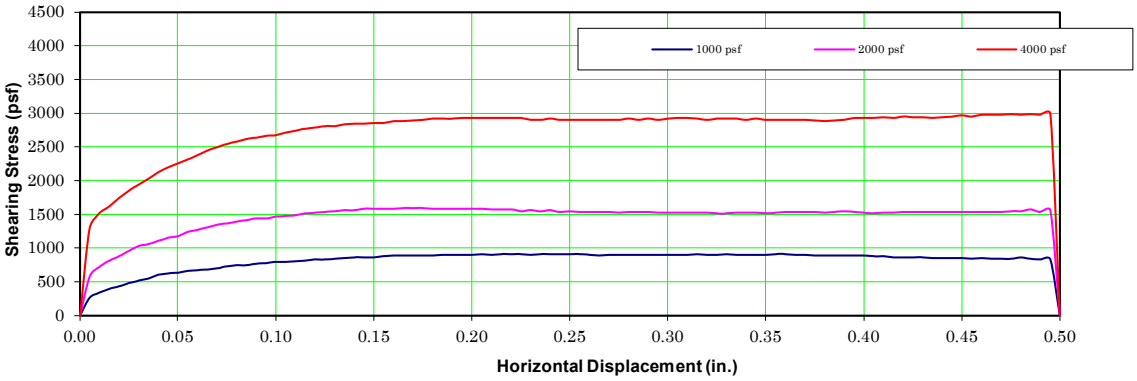

**JOB NAME:** Proposed Tractor Supply Company
**SAMPLE I. D.:** B-7 @ 0-5'
**SOIL DESCRIPTION:** Sandy Silt (ML)

| **SPECIMEN NUMBER** | D | E | F |
|---|---|---|---|
| **EXUDATION PRESSURE** | 492 | 312 | 179 |
| **RESISTANCE VALUE** | 46.5 | 39.5 | 31.8 |
| **EXPANSION DIAL(0.0001")** | 0 | 0 | 0 |
| **EXPANSION PRESSURE (PSF)** | 0.0 | 0.0 | 0.0 |
| **% MOISTURE AT TEST** | 12.3 | 13.0 | 13.7 |
| **DRY DENSITY AT TEST** | 119.0 | 118.5 | 117.4 |

**R-VALUE @ 300 PSI EXUDATION** **39**

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




## DIRECT SHEAR DATA*

Sample Location: B-1 @ 0-4'
Material: Silty Sand (SM)
Dry Density (pcf): 113.8

| | Initial | Final |
|---|---|---|
| Moisture Content (%): | 15.3 | 19.6 |
| Saturation (%): | 88 | 100 |
| | Peak | Ultimate |
| ϕ Angle of Friction (degrees): | 35 | 36 |
| c Cohesive Strength (psf): | 210 | 100 |

Test Type: Peak and Ulitimate
Shear Rate (in/min): 0.005

* Test Method: ASTM D-3080

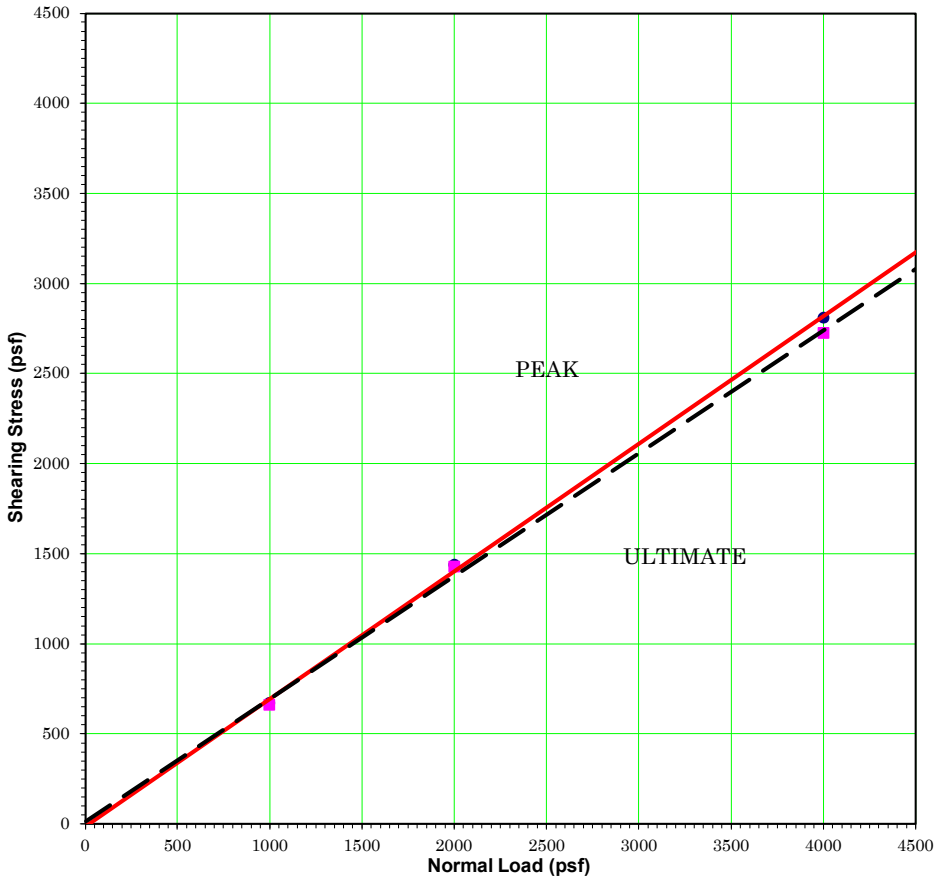
DIRECT SHEAR TEST

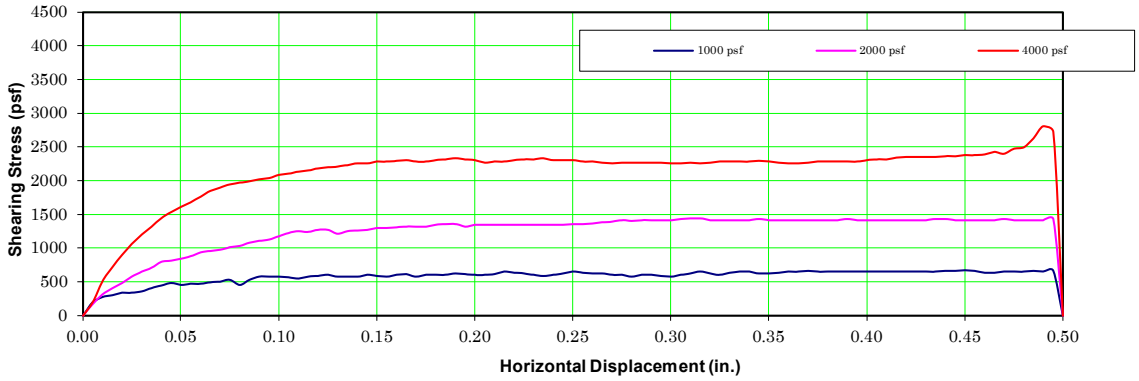
**Proposed Tractor Supply Company**

**Victorville, CA**

**Earth Systems Pacific**

| 7/26/2024 | 306687-002 |
|---|---|




## DIRECT SHEAR DATA*

Sample Location: B-8 @ 10'
Material: Sandy Silt (ML)
Dry Density (pcf): 105.0

| | Initial | Final |
|---|---|---|
| Moisture Content (%): | 7 | 18.3 |
| Saturation (%): | 32 | 100 |
| | Peak | Ultimate |
| ϕ Angle of Friction (degrees): | 35 | 34 |
| c Cohesive Strength (psf): | 0 | 10 |

Test Type: Peak and Ulitimate
Shear Rate (in/min): 0.005
* Test Method: ASTM D-3080

DIRECT SHEAR TEST

**Proposed Tractor Supply Company**

**Victorville, CA**

**Earth Systems Pacific**

| 7/26/2024 | 306687-002 |
|---|---|