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Geotechnical Engineering Report
Proposed Horseshoe Lake Park Improvements
Southwest Corner of Lakeview Avenue & Studio Place
Jurupa Valley, Riverside County, California

October 31, 2018

File No.: 302538-001
Doc. No.: 18-10-722

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Palm Desert, California 92260

Attention: Mr. Taku Shiozaki

Subject: **Geotechnical Engineering Report**

Project: **Proposed Horseshoe Lake Park Improvements**
Southwest Corner of Lakeview Avenue & Studio Place
Jurupa Valley, Riverside County, California

Earth Systems Pacific [Earth Systems] is pleased to submit this geotechnical engineering report for the referenced project located at Horseshoe Lake Park at the southwest corner of Lakeview Avenue and Studio Place, Jurupa Valley, Riverside County, California. The intent of this report is to provide geotechnical information for the proposed project. We understand the project improvements will include minor structures (such as covered play area, picnic shelter, and game tables), bridge, basketball court, 5 foot wide decomposed granite walking trail, 12-foot wide horse trail, horseshoe pits, 8 foot wide concrete walkway, exercise station, and minor grading of the park area.

This report completes our geotechnical scope of services in accordance with our agreement (PER-18-3-003) with an authorization date of September 5, 2018. Other geotechnical related services that may be required, such as plan reviews, responses to agency inquiries, and grading observation and testing are additional services and will be billed according to the Fee Schedule in effect at the time services are provided. Unless requested in writing, the Client is responsible to distribute the report to the appropriate governing agency and other members of the design team. Please review the Limitations (Section 6) of this report as it is vital to the understanding of this report.

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

Respectfully submitted,
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Geotechnical Engineering Report
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Southwest Corner of Lakeview Avenue & Studio Place
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Section 1
INTRODUCTION

1.1 Project Description

This geotechnical engineering report has been prepared for the proposed Horseshoe Lake Park Improvements located at the southwest corner of Lakeview Avenue and Studio Place in the city of Jurupa Valley, Riverside County, California, see Plate 1 (Site Vicinity Map). We understand the proposed park improvements will include minor structures (such as covered play area, picnic shelter, and game tables), bridge, basketball court, 5 foot wide decomposed granite walking trail, 12 foot wide horse trail, horseshoe pits, 8 foot wide concrete walkway, exercise station, and minor grading of the park area. Appurtenant site work is anticipated to include access walkways and underground utilities.

For structures, we have assumed one story masonry, wood-framed or metal construction founded on shallow permanent foundations, and there will be no below grade basement levels. Column loads are anticipated not to exceed approximately 32 kips for spread footings and 2 kip/LF for continuous footings loads. As the basis for the foundation recommendations, all loading is assumed to be dead plus actual live load. No preliminary design loading was provided by the structural engineer. If actual structural loading exceeds these assumed values, we will need to re-evaluate the given recommendations.

1.2 Site Description

The project site is located at Horseshoe Lake Park at the southwest corner of Lakeview Avenue and Studio Place in the city of Jurupa Valley, Riverside County, California. The site has an approximate latitude and longitude of 33.9703°N/117.4763°W. The legal description of the land is identified as Assessor Parcel Number [APN] 163-240-001 encompassing approximately 14 acres. The park is bounded by Lakeview Avenue on the northeast, Studio Place on the southeast, Kennedy Street on the south, and Kelsey Place on the west. Topographically, the site is located between topographical contours 700 feet Mean-Sea-Level (MSL) and 720 feet MSL (USGS, Riverside West, 1980). From google imagery, the elevation at the project site is approximately 716 feet above mean sea level.

1.3 Site Reconnaissance

Earth Systems personnel visited the site on multiple days: September 18, 2018; September 27, 2018; and October 2, 2018. Earth Systems personnel also reviewed select historic aerial photographs of the project site. The proposed site layout along with our exploration locations is presented in Plate 2.

Historical aerial photographs revealed items of interest. The images indicated below were viewed from latest (2018) to earliest (1948):

1. November 2013 to February 2018: Site has remained relatively unchanged.
2. June 2012 to November 2013: Monument sign was placed on southwest corner of Lakeview Avenue and Studio Place.
3. March 2011 to June 2012: Horseshoe shaped walkway was added throughout the park.
4. November 2009 to March 2011: Horse ring was constructed on southwest side of park.
5. 1967 to November 2009: Horseshoe shaped lake appears dry.
6. 1948 to 1967: Horseshoe shaped lake is present throughout the park.

1.4 Purpose and Scope of Services

The purpose for our services was to evaluate the site soil and geologic conditions at our exploration locations and to provide professional opinions and recommendations, from a geologic and geotechnical point of view, regarding the proposed development of the site. We understand that these proposed site improvements will be developed under the regulation of the current California Building Code (2016).

The conclusions and recommendations included in this report are based upon the data collected for this commission. The scope of services included:

<u>Task 1 - Literature and Photograph Reviews</u>

We began our services by reviewing select geologic and geotechnical literature pertaining to the project. This included a review of various hazard, fault, and geologic maps prepared by the California Geological Survey, the U.S. Geological Survey, the County of Riverside 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.

<u>Task 2 – Utility Clearance, USA Dig Alert</u>

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 consumer grade Global Positioning System (GPS) accurate to ± 15 feet in conjunction with pacing based upon the control provided or sighting from landmarks identified on the project topographic map.

<u>Task 3 – Field Exploration</u>

We evaluated the general subsurface conditions at the site by drilling seven small diameter borings, from approximately 5½ feet to 46½ feet in depth. The field exploration also included a site reconnaissance of the project area and immediate surroundings. Plate 2 shows the approximate location of each boring and the infiltration test locations.

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). Earth Systems does not practice corrosion engineering; however, these test results may be used by a qualified engineer in designing an appropriate corrosion plan for the project.

Task 5 – Infiltration Testing

Two test pits were excavated within the proposed storm infiltration locations for infiltration testing. These test pits were excavated on October 2, 2018 with a backhoe. Plate 2 shows the approximate location of each test.

Task 6 – Analysis and Report

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

- A description of the proposed project including a site plan showing the approximate boring locations;
- A description of the surface and subsurface site conditions including groundwater conditions, as encountered in our field exploration;
- A description of the site geologic setting and possible associated geology-related hazards, including liquefaction, subsidence, and seismic settlement analysis;
- A discussion of regional geology and site seismicity;
- A description of local and regional active faults, their distances from the site, their potential for future earthquakes;
- A discussion of other geologic hazards such as ground shaking, landslides, flooding, and tsunamis;
- A discussion of site conditions, including the geotechnical suitability of the site for the general type of construction proposed;
- A seismic analysis including recommendations for geotechnical seismic design coefficients and soil profile type in accordance with the 2016 California Building Code;
- Recommendations for imported fill for use in compacted fills;
- Recommendations for foundation design including parameters for shallow foundations and subgrade preparation;

- Anticipated total and differential settlements for the recommended foundation system;
- Recommendations for lateral load resistance (earth pressures and drainage);
- Recommendations for site preparation, earthwork, and fill compaction specifications;
- Discussion of anticipated excavation conditions;
- Recommendations for underground utility trench backfill;
- Recommendations for stability of temporary trench excavations;
- Recommendations for location-specific infiltration rates;
- Recommendations for slabs-on-grade, including recommendations for reducing the potential for moisture transmission through interior slabs;
- Recommendations for collapsible or expansive soils (if applicable);
- Recommendations for asphalt concrete and Portland cement concrete parking and drives;
- A discussion of the corrosion potential of the near-surface soils encountered during our field exploration;
- An appendix, which includes a summary of the field exploration (computer generated boring logs) and laboratory testing program (computer generated plots).

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

- ➢ An environmental Phase 1 assessment.
- ➢ 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.

Section 2
METHODS OF EXPLORATION AND TESTING

2.1 Field Exploration

<u>Exploratory Borings</u>

The subsurface exploratory program included advancing seven exploratory borings. The borings were drilled to depths ranging from approximately 5½ to 46½ feet below existing grades using a Mobile B-61 truck-mounted drill rig equipped with 8-inch hollow-stem augers provided by Cal-Pac drilling of Calimesa, 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. The locations shown are approximate, established by consumer grade Global Positioning System (GPS) accurate to ± 15 feet in conjunction with pacing based upon the control provided.

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

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

Design parameters provided by Earth Systems in this report have considered an estimated 70% hammer efficiency. The number of blows necessary to drive either a SPT sampler or a MC type ring sampler within the borings was recorded. Since the MC sampler was used in our field exploration to collect ring samples, the N-values using the California sampler can be roughly correlated to SPT N-values using a conversion factor that may vary from about 0.5 to 0.7. In general, a conversion factor of approximately 0.63 from the recent study at the Port of Los Angeles (Zueger and McNeilan, 1998 per SP 117A) is considered satisfactory. A value of 0.63 was applied in our calculations for this project.

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

<u>Infiltration Testing</u>

Two test pits were excavated within the proposed storm water infiltration locations for infiltration testing. Test pits were excavated on October 2, 2018 using a backhoe with a 24 inch bucket. Test pits reached depths of approximately 4 and 2.5 feet below the existing ground surface. Infiltration testing was performed in general accordance with the Standard Test Method for Infiltration Rate of Soils in Field Using Double-Ring Infiltrometer Test (ASTM D3385). The double-ring infiltrometer method consists of driving two open cylinders, one inside the other, into the ground, partially filling the rings with water and then maintaining the liquid at a constant level. The volume of liquid added to the inner ring, to maintain the liquid level constant is the measure of the volume of liquid that infiltrates the soil. The volume infiltrated during timed intervals is converted to an incremental infiltration, expressed in inches per hour. The infiltration locations are shown the Boring Location Map, Plate 2, in Appendix A. The locations shown are approximate, established by pacing and line-of-sight bearings from adjacent landmarks and consumer grade GPS coordinates (+/- 15 feet).

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.

Section 3
DISCUSSION

3.1 Soil Conditions

The field exploration indicates that site soils generally consist of interbedded silty sand, sandy silt, clayey gravel, and clayey sand (Unified Soils Classification System symbols of SM, ML, GC, and SC) to the maximum depth of exploration of 46½ feet below the ground surface. In general, the site is covered with shallow fill overlying naturally deposited soils. Fills and disturbed soils are typically within past use areas with surficial disturbance and were generally on the order of 4 feet thick. Native soils consist of older alluvial (water transported) deposits (Qc). Fill soils appear comprised of the native soils. In general, the observed sandy soils were medium dense to very dense to the depth explored. Fine grained soils were hard to the depth explored. Site soil moisture observations varied between dry to very moist with lab moistures ranging between 3 and 21 percent. A detailed description of the observed soils is provided on the boring logs in Appendix A.

3.2 Groundwater

No free groundwater water was encountered during our field exploration (maximum depth 46½). Perched moisture conditions were encountered in Boring B-1 from 30 to 46½ feet below the ground surface in the form of those soils near saturation (based on % calculation), but no free water was observed. Free water is defined as visible excess water on or in the sample or sample collection devices. The site is located within the Upper Santa Ana watershed that includes the Chino basin and Santa Ana River. The historic high depth to groundwater in the area is believed to be about 11-16 feet based on information from the Western Municipal Water District Cooperative Well Measuring Program (2009).

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

- State Well No. 02S06W27A001S is located approximately 0.8 miles northwest of the project site. The surface elevation of this well is approximately 660.55 feet and the groundwater readings as measured from 1994 to 2008 varied from 645.37 to 649.65 feet above mean sea level.
- State Well No. 02S06W24Q(SW) is located approximately 1 mile northeast of the project site. The surface elevation of this well is approximately 789.5 feet and the groundwater readings as measured from 2005 to 2008 varied from 771.86 to 773.55 feet above mean sea level.

Based on the above data, groundwater is not anticipated to be encountered during construction. The historic groundwater depth is estimated to be approximately 13 feet deep at the site (approximal water surface elevation of 697 feet MSL). Fluctuations of the groundwater level and localized zones of increased soil moisture content should be anticipated during and following the rainy season, from irrigation, or from filling of the onsite, non-lined, lake.

3.3 Collapse and Consolidation Potential

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

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

Table 1
Collapse Potential Values

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

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 to evaluate the potential hazard by collapsible soil at a specific site. A hazard ranking system associated with collapsible soil as developed by Hunt (1984) is presented in Table 2, Collapsible Soil Hazard Ranking System.

Table 2
Collapsible Soil Hazard Ranking System

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

The results of collapse potential tests performed on five selected samples from depths ranging from 5 to 20 feet below the ground surface indicated a collapse potential on the order of 1.7 to 5.7 percent. The goal of the collapse testing was to identify soils and densities where the potential for collapse decreased to accepted levels. This accepted level is defined as where on-site soils had collapse potential less than 1% to 2% or the estimated relative compaction is greater or equal to 80 to 85%, which is the typical standard of care based on the above Table 1 (1%) or where soil collapse becomes a concern for structural soils (2%) (County of Los Angeles, 2013).

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

1. Soils are generally granular in nature and no significant cementation was observed. Older alluvial soils with high blow counts predominate at the site: however low blow count, and lower density layers exist.
2. Pinhole voids were not observed.
3. High dry densities (DD > 114 pcf) of the soils determined during the laboratory testing generally had lower potential for collapse (less than 2%).
4. Collapsible soils were generally classified as Silty Sand (SM).
5. Soil collapse at the site appears to be directly related to in-place density (relative compaction).

For some deposits without cementation, studies suggest some sites with densities above 103 pounds per cubic foot (pcf) are “not likely to collapse” and N_{60} Values > 10 do not fit into the category of “Likely Collapsible” (Lommler, C. J. and Bandini). In addition, soils with greater than 85 percent relative compaction are compact, and it is accepted that they are not likely to settle, especially after initial inundation.

Based on the above criteria and our field and laboratory findings, we estimate there is a “Trouble” collapse potential from soil layers between 5 and 20 ft below the ground surface (bgs). Without collapse mitigation efforts, the collapse potential is approximately 3 inches. Assuming the recommended grading is accomplished according to Section 5.1 of this report, we estimate the collapse potential differential settlement is on the order of approximately 0.5 inches.

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” as defined by ASTM D 4829 and the 2016 California Building Code.

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

3.5 Corrosion Potential

One sample of the near-surface soils within the site was tested for potential corrosion of concrete and ferrous metals. Soils in the upper 0 to 5 feet were tested as a blended (composite) 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 7.4, chloride content of 23 ppm, sulfate content of 51 ppm and minimum resistivity of 1,720 Ohm-cm. Although Earth Systems does not practice corrosion engineering, the corrosion values from the soil tested are normally considered as being "severely" corrosive to buried metals and as possessing a "negligible" exposure to sulfate attack for concrete as defined in American Concrete Institute [ACI] 318, Section 4.3. The results of all 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 Storm Water Infiltration Testing

As indicated in Section 2.1 of this report, two test pits were excavated using a backhoe with a 24 inch bucket. Test pits were excavated on October 2, 2018 and reached depths of approximately 4 and 2.5 feet below the existing ground surface. These test locations represent the soils at the assumed bottom of the proposed infiltration systems. The infiltration test locations are shown on the Boring Location Map (Plate 2), in Appendix A.

Infiltration tests were performed on October 2, 2018. Test procedures followed the procedures for the double-ring infiltrometer test according to the Riverside County Flood Control and Water Conservation District Design Handbook for Low Impact Development Best Management Practices, September 2011. The soils encountered at each test location and the infiltration rates are presented in Table 5. A factor of safety should be applied to the tested infiltration rate in order to determine the design infiltration rate in accordance with Riverside County guidelines.

Table 5
Infiltration Rate Results

Test	Soil Condition	USCS Soil Description in Test Zone	Test Zone Below Existing Surface (feet)	Tested Infiltration Rate (in/hr)
P-1	(Native)	Silty Sand (SM)	4	1.9
P-2	(Native)	Silty Sand (SM)	2.5	0.2

3.7 Geologic Setting

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

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

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

Local Geology: The site is located north of the Santa Ana River on an elevated older alluvial fan situated southwest of the Pedley Hills. Lithologic materials within and adjacent to the Pedley Hills and Santa Ana River include intermixed intrusive igneous rocks of the southern California batholith overlain by Quaternary sediments, including Pleistocene older alluvium, and Holocene wash deposits. Older alluvial fan deposits composed of silty sand and sandy silt underlie the site. A Local Geologic Map is presented as Plate 4 in Appendix A.

No major faults have been mapped within the project limits. The site is not located within a currently designated Alquist-Priolo Earthquake Fault Zone or County fault zone. The nearest mapped active or potentially active fault is the Elsinore (Chino) fault zone located approximately twelve miles southwest of the site. The Fontana seismic zone is located about five miles from the site.

3.8 Geologic Hazards

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

3.8.1 Seismic Hazards

Seismic Sources: Several active faults or seismic zones lie within 36 miles of the project site as shown on Table A-1 in Appendix A. The primary seismic hazard to the site is strong ground shaking from earthquakes along the Elsinore, San Jacinto, Cucamonga, and San Andreas fault zones.

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

Historic Seismicity: The project area is in the seismically active southern California where approximately 30 earthquakes of magnitude 5.5 or greater have occurred within 60 miles of the park, usually originating on or near the San Andreas, San Jacinto, or Elsinore faults. These include the 1812 Wrightwood, 1894 Lytle Creek, 1899 San Jacinto, 1910 Elsinore (Glen Ivy Hot Springs), 1918 San Jacinto, and 1923 North San Jacinto earthquakes.

Of significance are the multiple earthquake events along the San Jacinto fault at the turn of the century in 1890, 1892, 1899, and 1923. Additional earthquakes in the region along this fault zone occurred in 1937 and 1954 suggesting that the San Jacinto fault is a significant source of large to major earthquakes. Of interest, the only significant historic earthquake along the local Elsinore fault was in 1910.

Seismic Risk: The primary seismic risk at the site is a potential earthquake along the active Elsinore, San Jacinto, Cucamonga, and San Andreas fault. While accurate earthquake predictions are not possible, various agencies have conducted statistical risk analyses. In 2002 and 2008, the California Geological Survey [CGS] and the United States Geological Survey [USGS] completed probabilistic seismic hazard maps. We have used these maps in our evaluation of the seismic risk at the site. The recent Working Group of California Earthquake Probabilities (WGCEP, 2007) estimated a 59 percent conditional probability that a magnitude 6.7 or greater earthquake may occur between 2008 and 2038 along the southern segment of the San Andreas fault, 11 percent for the Elsinore fault, and 31 percent along the San Jacinto fault. Recent estimates suggest a nearly 98% probability of a nearby magnitude 5 earthquake in the next 50 years.

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

In general, for the effects of liquefaction to be manifested at the surface, groundwater levels must be within 50 feet of the ground surface and the soils within the saturated zone must also be susceptible to liquefaction. We consider the potential for liquefaction to occur at this site as moderate to high because historic groundwater is generally less than 50 feet below the ground surface. The site is within a “high” liquefaction hazard zone as defined by Riverside County (Geographic Information Services, 2018). Liquefaction output considering historic groundwater levels are presented in Appendix A. Results indicate a liquefaction potential at depths greater than 15 feet with estimated liquefaction induced settlement of 1 inch in B-1. The potential for lateral spreading to the nearby channel (0.5 km) is considered low under a screening evaluation due to the blowcount >15 N_{160} for the liquefiable layer (Youd & Bartlett, 2002). Site ground screening displacement is estimated to be less than 3 feet (1 meter) and thus a “low potential”. Due to the density of overlying soils, the potential for sand boils is considered 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 (PGA_M = 0.50) and an earthquake magnitude of 8.2 to evaluate dry seismic settlement potential. The design peak ground acceleration values were obtained from the USGS online application (http://earthquake.usgs.gov/designmaps/us/application.php).

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 (50 feet) was calculated in our deep 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. Due to the general uniformity of the soils encountered, seismic settlement is expected to occur on an areal basis and as such per Special Publication 117A (CGS, 2008), the differential settlement is estimated to be approximately ½ of the total estimated dry seismic settlement (0.05 inches) considering soil remediation as recommended in Section 5.1.

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

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

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

3.8.2 Other Hazards

Landslides and Slope Instability: The site is relatively flat and slopes are anticipated to be less than 5 feet high. Therefore, potential hazards from slope instability, landslides, or debris flows are considered very low.

Flooding: The project site lies in an area designated as Zone X: “Areas of 0.2% annual chance floodplain; areas of 1% annual chance flood with average depths of less than 1 foot or with drainage areas less than 1 square mile; and areas protected by levees from 1% annual chance flood.” This project area and Zone X (orange colored and natural colored areas) are identified on FEMA Map No.: 06065C0705G, Panel 705 of 3805, Map Revised August 28, 2008 (Figure 1). Appropriate project design by the project civil engineer, construction, and maintenance can minimize the site sheet flooding potential.

Seiches: Seiching is defined as a periodic oscillation of liquid within a container or reservoir. Its period is determined by the resonant characteristics of the container, as controlled by its physical dimensions. Swimming pools are located approximately 200 feet to the west and south of the proposed site. However, it is likely any flooding associated with pool seiches related flooding would follow existing local drainage/roadway improvements, such that the impact to the site would be negligible. If the onsite lake is one day full, seiching is likely.

Section 4
CONCLUSIONS

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

General:

Based on our field exploration, laboratory testing, and geotechnical analyses conducted for this study, it is our professional opinion that the site is suitable, from a geotechnical standpoint, for construction 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.

<u>Geotechnical Constraints and Mitigation</u>:

- The primary geologic hazard is moderate to severe ground shaking from earthquakes originating on regional southern California faults. A major earthquake originating on the nearby segments of the Elsinore, San Jacinto, Cucamonga, and San Andreas fault zones and other associated faults would be the critical seismic events that may affect the site within the design life. Engineered design and earthquake-resistant construction increase safety and allow development within seismic areas.
- We consider another geotechnical constraint for development of this site, as identified by our study, to be hydro-consolidation and liquefaction induced ground settlement. It is our opinion that to construct the proposed structures, site soil improvement techniques will be required to reduce the potential distress to the proposed structures should hydro-consolidation or liquefaction occur. The recommendations presented are intended to reduce the magnitude and severity of potential distress to the proposed structures, such that the estimated ground settlement presented within can be accommodated in structural design.

  In order to prepare this site, the geotechnical design intent is based upon reducing the differential settlement component of total settlement, which is manifested at the surface, to tolerable levels such that the potential for structure distress is reduced and the structure can be designed using typical foundations and methodologies in a practical and economical manner. We are recommending a geogrid reinforced soil mat (densification) system and stiffened foundations as measures to increase the soil bridging (membrane effect) such that localized point differential settlement which may occur at depth due to hydro-consolidation is further distributed and attenuated within the foundation and slab area.

  We have combined two accepted methods of reducing localized differential settlement (reinforced foundations and soil densification) which are recommended in SP117A (2008, page 57).
- The recommendations presented within do not address performance in regard to flatwork, site perimeter walls, utilities, etc. It is our opinion that it is not practically feasible to mitigate or

reduce the potential for the occurrence of hydro-consolidation or liquefaction across the whole site. The manifestation and effect may generally affect the flatwork, pavement, site perimeter walls, basins, utilities, etc. through differential settlement of the site soils. These effects may cause localized distress to the portions of the site where hydro-consolidation occurs. It is our opinion that it may not be economically feasible or cost effective to implement engineering measures to mitigate the potential effects of hydro-consolidation. It is our opinion that the effects of hydro-consolidation and liquefaction and related distress would most likely require repair in the form of re-leveling should it occur and manifest to the surface. Selective design utilizing less sensitive fencing (chain link), earth paths, flexible pipe etc. can also reduce the impact of hydro-consolidation.

- The underlying geologic condition for seismic design is Site Class D. The site is about 12 miles from a Type A seismic source and 5.4 miles from a Type B 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 2016 edition of the California Building Code.
- The upper soils were found to be relatively non-uniform silty sands which are unsuitable in their present condition to support structures, fill, and hardscape. The soils within the building and structural areas will require moisture conditioning, over-excavation, and recompaction to improve bearing capacity and reduce the potential for differential settlement. Soils can be readily cut by normal grading equipment.
- The site is not within the County of Riverside designated fault zone, nor is the site within a currently designated Alquist-Priolo Earthquake Fault Zone. The site is within a Riverside County designated liquefaction zone.
- The potential for liquefaction settlement hazards are considered moderate to high for this project. The site is not within an area of significant documented areal subsidence.
- Other geologic hazards, including flooding, and landslides, are considered low potential on this site.
- Based on current conditions, groundwater is not anticipated to be encountered during construction, however wet soils will likely be encountered.
- Much of the existing on-site fill and alluvial soils are very low in Expansion Index and suitable for location under structures or hardscape after remedial grading. Building structure recommendations provided within are based upon using a very low in expansion potential fill material for the building pad.
- Laboratory testing of one sample showed potentially “severe” corrosivity to buried metallic elements and “negligible” for sulfate exposure to concrete. See Section 3.5 for further information. Site soils should be reviewed by an engineer competent in corrosion evaluation.
- In our professional opinion, structure foundations can be supported on shallow foundations bearing on a zone of properly prepared and compacted soils placed as recommended in Section 5.1. The recommendations that follow are based on “very low” expansion category soils.

Section 5
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 2016 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 South Coast Air Quality Management District [SCAQMD].

Observation of fill placement by the Geotechnical Engineer of Record should be in conformance with Section 17 of the 2016 California Building Code. California Building Code requires full time observation by the geotechnical consultant during site grading (fill placement). Therefore, we recommend that Earth Systems be retained during the construction of the proposed improvements to provide testing and observe compliance with the design concepts and geotechnical recommendations, and to allow design changes in the event that subsurface conditions or methods of construction differ from those assumed while completing this study. Additionally, the California Building Code requires the testing agency to be employed by the project owner or representative (i.e. architect) to avoid a conflict of interest if employed by the contractor. Unless noted otherwise, grading should be performed in general accordance with Appendix J of the 2016 CBC.

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

Undocumented fill, and 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 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 (Shelters, Bathroom Pads, Structure Pads, etc.): Because of the relatively non-uniform and under-compacted nature of the site soils and hydro-consolidation potential, we recommend recompaction of soils in the building area (Structures with foundations, play structures exempted) and inclusion of tri-axial or bi-axial geo-grid (Tensar TX140, Terrafix TBX3000, or equivalent as approved by the geotechnical engineer) within the building pad remedial grading.

The existing surface soils within the building pad and foundation areas should be over-excavated to a minimum depth of 4 feet below existing grade, finished grade, or a minimum of 2 feet below the footing level (whichever is lower). A minimum of 85% relative compaction should be confirmed in the undisturbed excavation bottom. The over-excavation should extend for 5 feet beyond the outer edge of exterior footings and include any covered walkway areas, where possible. The approved bottom of the sub-excavation should then be scarified, moisture-conditioned, and recompacted to at least 90% relative compaction (ASTM D 1557) for an additional depth of one foot. Then, a layer of geogrid should be placed and wrapped up the side walls a minimum of 2 feet, followed by another 12 inches of fill placed and compacted to at least 90% relative compaction (ASTM D 1557). Then, another layer of geogrid. Then 12 inches of fill should be placed and compacted to at least 90% relative compaction (ASTM D 1557). Then a final layer of geogrid should be placed and fill compacted to 90% relative compaction (ASTM D 1557) should then be placed to finished grade.

For clarification, the first geogrid layer should be placed at 4 feet below existing grades with subsequent layers spaced at 12 inches apart (3 feet and 2 feet below existing grade). Grid placement should be as prescribed by the grid manufacturer. Of importance is that the grid needs to be restrained (pinned) during placement to prevent sagging and looseness. Utility placement should be planned to minimize disruption or cutting of the grids. Typically, when grid is used, utilities are placed in well defined utility corridors. Geogrid which is cut should be repaired per the manufacturer's recommendations.

The design of the geogrid reinforced soil mat is based, in part, on the finding of a case study entitled *Embankment Reinforcement by Geogrid to Reduce Its Settlement During Earthquakes*, Yasushi Sasaki, Seiji Kano, and Tomoharu Tsuji, 13th World Conference on Earthquake Engineering, Vancouver, B.C., Canada August 1-6, 2004, Paper No. 642.

Auxiliary Structures Subgrade Preparation: Auxiliary structures, such as fence or retaining walls (with foundations), trash enclosures, etc., should have the foundation subgrade prepared similar to the building pad recommendations given above but limited to a 3 foot overexcavation below existing or finished grade, or two feet below footings, whichever is deeper and excludes the geogrid layer. Seismic performance can be improved with the inclusion of the deeper geogrid; however this option may be cost prohibitive (see the related discussion in the conclusions). The lateral extent of the over-excavation needs to extend only 2 feet beyond the face of the footing. Perimeter or fence walls should be constructed of lightweight material, such as chain-link, wood, or wrought iron/aluminum/steel to reduce the potential for damage during a seismic event if the geogrid is not incorporated.

Subgrade Preparation: In areas to receive fill not supporting structures or hardscape the subgrade should be scarified; moisture conditioned and compacted to at least 90% relative compaction (ASTM D 1557) for a depth of 1 foot below existing grade, or finished subgrade, whichever is deeper. Decomposed granite walkways and trails should be compacted to at least 95% relative compaction (Trails:upper 12 inches). Fill compacted to at least 90% compaction should be placed to finished subgrade. Compaction should be verified by testing.

Pavement and Hardscape Area Preparation: In street, drive, permanent parking areas, and hard scape/flat work areas the subgrade should be over-excavated, scarified, moisture conditioned, and compacted to at least 90% relative compaction (ASTM D 1557) for a depth of at least 24 inches below existing grade or finish grade (whichever is deeper), with the upper 1 foot compacted to at least 95% relative compaction. Compacted fill should be placed to finish subgrade elevation. Compaction should be verified by testing.

Retention Basin and Infiltrator Bottom Preparation: Compaction effort should be kept to a minimum at retention basin bottom areas and bottom areas used for any infiltrators (except under foundations or slabs). The subgrade below the bottom of basins and infiltrator bottoms should be compacted to approximately 85% relative compaction. Side slopes and any other fill or foundation or slab subgrade should be compacted to at least 90% relative compaction. Slope construction should be per this report. Loose rock, such as pea gravel or open graded rock placed in the basin bottoms does not require compaction testing, but should be placed in lifts no greater than 2 feet and consolidated by thoroughly wetting and consolidating by passes with heavy equipment (such as a loader with full bucket or full water truck) until firm such that none to minimal deformation (less than 1 inch) occurs under the weight of passing equipment.

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

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

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

Engineered fill (and any import) should be placed in maximum 8-inch lifts (loose) and compacted to at least 90 percent relative compaction (ASTM D 1557) near its optimum moisture content prior to placement of a subsequent loose lift. Within pavement areas, the upper 12 inches of subgrade should be compacted to at least 95 percent relative compaction (ASTM D 1557). Compaction should be verified by testing. Rocks larger than 6 inches in greatest dimension should be removed from fill or backfill material, with the exception of playfield areas, where local school district criteria necessitating a smaller oversize allowance may apply. Typically, in play field areas, the maximum oversize allowed is 1 inch.

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

A program of compaction testing, including frequency and method of test, should be developed by the project geotechnical engineer at the time of grading. Acceptable methods of testing may include Nuclear methods such as those outlined in ASTM D 6938 (Standard Test Methods for In-Place Density and Water Content of Soil and Soil-Aggregate by Nuclear Methods). Additionally, a minimum of 5% of the in-place density tests should be performed using an alternative method for quality assurance of compaction levels. Alternative methods may include methods outlined in ASTM D 1556 (Standard Test Method for Density and Unit Weight of Soil in Place by the Sand-Cone Method) or correlation probing with a hand probe.

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

<u>Shrinkage and Bulking</u>: The shrinkage factor for earthwork for the alluvial materials is expected to range from 4 to 31 percent for the upper excavated or scarified *site* soils based upon evaluation of 16 in-place densities (one standard deviation = 8, 95% Confidence Interval). This estimate is based on compactive effort to achieve an weighted average relative compaction of about 93 percent.

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

<u>Dust Control</u>: The proposed site lies within an area of moderate 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 to contain loose soil particles. Palliatives must be reapplied periodically as they weather and degrade. Further guidance for dust palliatives can be found in reviewing the United States Department of Agriculture Publication *Dust Palliative Selection and Application Guide*, Document No. 9977-1207-SDTDC. The recommended soil input parameters are Plasticity Index =3 to 8, and fines content 10 to 20 percent.

5.2 Excavations and Shoring

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

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

The borings were advanced with minimal to 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. Where overexcavation will be performed adjacent to existing structures, ABC slot cutting techniques may be used as pre-approved by the project geotechnical engineer.

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

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

Table 6
Temporary Cantilevered and Braced Shoring System Parameters

Equivalent Fluid Pressure pounds per cubic foot (pcf)	
Cantilevered	**Braced**
44	66

The values provided above assume a level ground surface adjacent to the top of the shoring and do not include a factor of safety. Fifty percent of an areal surcharge placed adjacent to the shoring may be assumed to act as an additional uniform horizontal pressure against the shoring. Special cases such as combinations of slopes and shoring or other surcharge loads may require an increase in the design values recommended above. These conditions should be evaluated by the project geotechnical or shoring engineer on a case-by-case basis. Shoring 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. Shoring walls with closer traffic or heavier traffic loads should be designed for a 400 psf surcharge load.

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 350 pcf. Additionally, we recommend a factor of safety of at least 1.2 be applied to the calculated embedment depth and that passive pressure be limited to 2,000 psf.

The contractor should be responsible for the structural design and safety of all temporary shoring systems. The contractor should carefully review the boring logs in this report, and perform their own assessment of potential construction difficulties, and methods should be selected accordingly. 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. We recommend that existing structures be monitored for both vertical and horizontal movement.

5.3 Utility Trenches

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

Due to the need to test pipeline subgrade, where excavations require trench entry, the contractor should make allowance and provide proper shoring or stabilize the excavation in accordance with CalOSHA for safe entry by the testing lab and workers.

<u>Trench Width and Vertical Loads on Pipelines</u>: Vertical loads to the pipeline are highly dependent upon the geometry of the trench. In general, the narrower the trench is at the top of the pipe/conduit with respect to the diameter of the conduit, the less vertical load is applied to the conduit. This is because as the trench backfill and bedding compress or consolidate over time, the weight of the soil mass is partially offset by the frictional resistance along the trench

sidewalls. In addition, the type of bedding supporting the pipeline affects the bearing strength of the conduit. This is accounted by a load factor that is multiplied to the design strength of the conduit. The pipe manufacturer recommendations for trench installation and maximum width should be followed to reduce the potential for overloading the pipe due to excess backfill load.

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

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 rights-of-way should be placed in conformance with the requirements of the governing agency (water district, public works department, etc.).

Pipe zone backfill material (the pipe area from the bedding to 12 inches above the top of pipe) may consist of native soils screened to a ¾" maximum particle size or import sand (as described above for bedding) as dictated by the pipe designer or manufacturer. The pipe zone backfill material should be placed in maximum 8-inch lifts (loose) and compacted near its optimum moisture content. 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 prior to the placement of subsequent lifts. Compaction should be assured in the pipe haunches.

The native soil may be suitable for use as trench zone and street zone (and manholes) backfill (from the top of pipe zone up to finished grade), as approved suitable by the pipe manufacture provided it is free of significant organic or deleterious matter and oversize materials. Native soil may not be suitable to support pipe haunches, there by requiring a sandy import, as above for bedding. This backfill should contain no particles larger than 3 inches in greatest dimension. The final backfill material should be placed in maximum 8-inch lifts (loose) and compacted to at least 90% relative compaction (ASTM D 1557) near its optimum moisture content for the trench zone and 95% for the street zone (upper 12 inches) where below pavement. Compaction should be verified by testing.

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

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

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.

Utilities connections which tie into the structures should be flexible. <u>Placement of underground utilities should take the geogrid location into consideration, such that damage or cutting/penetration of the grid is absolutely minimized during installation. Utility corridors, i.e. most utilities enter or exit a building at the same location, should be utilized to minimize penetration locations in the geogrid. Where utilities must penetrate geogrid or filter fabric, the specific geogrid/fabric repair method should be evaluated by the geotechnical engineer on a case by case basis prior to final design. Geogrid which is cut should be repaired per the manufacturer's recommendations</u>.

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

STRUCTURES

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

5.4 Foundations

Footing design of widths, depths, and reinforcing are the responsibility of the Structural Engineer, considering the structural loading and the geotechnical parameters given in this report. A minimum footing depth of 12 inches (below lowest adjacent grade) should be maintained and considers a "very low" Expansion Index soil. 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. <u>*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</u>.

Minimum Slope Setback for Foundations: Earth Systems recommends a minimum setback distance of 5 feet. The 2016 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.

Conventional Spread Foundations: The minimum footing depths presented below should be maintained below the lowest adjacent grade (lowest adjacent grade = lowest grade within 2 feet laterally). Allowable soil bearing pressures are given below for foundations bearing on recompacted soils as described in Section 5.1. Allowable bearing pressures are net (weight of footing and soil surcharge may be neglected). We utilized a factor-of-safety of 3.0 on ultimate bearing values for determining allowable bearing values.

- Reinforced foundations, 12-inch minimum width and 12-inch minimum depth below grade:

  1,800 psf for dead plus design live loads.

- Isolated pad foundations, 24 x 24-inch minimum in plan, 18-inch minimum embedment:

  2,000 psf for dead plus design live loads.

All pad foundations and isolated foundations should be tied to the main foundations system utilizing grade beams.

Where possible, it is recommended to keep foundations as shallow as possible to maintain continuity of geogrid layers throughout the structure with minimum vertical elevation deviation (i.e. to minimize potentially having to dip the geogrid layers below deeper footings). Structure foundations should be underlain by the 3 layers of geogrid system presented in Section 5.1.

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

If the anticipated loads exceed the estimated values stated in Section 1.1 (32 kips for isolated footings and 2 kip/linear-ft for continuous footings), the geotechnical engineer must reevaluate the allowable bearing values as the allowable bearing was controlled by the allowable total settlement from dry seismic, collapse, and static loads not exceeding 1.5 inches calculated. Underground utilities should be designed for an anticipated settlement within the building areas.

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

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

The table below is based upon the above presented allowable, short term, and ultimate bearing pressures. Values may be increased by the provisions given above. Short Term allowable bearing may use the values presented below (based on Allowable Stress Design) or be based on Code mandated structural reductions, whichever is less. Ultimate bearing capacities consider a factor of safety of 3 (ASD design) to control settlement (5,400 to 6000 psf ultimate) and a safety factor of 2.25 on transient loads (2,400 to 2,667 psf). Ultimate bearing to soil failure depends on foundation size and could be much greater than 6,000 psf. The restrictions of Section 1605A.1.1 apply to the cited bearing values for Allowable Stress Design (ASD).

Table 7

	Allowable Bearing Capacity (psf) (FS = 3)	**Short Term (Wind/Seismic) (FS = 2.25)**	**Ultimate Bearing Capacity (FS = 1)**
Continuous Foundations	1,800	2,400	5,400
Isolated Pad Foundations	2,000	2,667	6,000

FS = Factor of Safety

Footings should be designed and reinforced by the structural engineer for the specific loading, or settlement conditions. A minimum of four, #4 reinforcing bars should be placed. Two near the top of the footing and two near the bottom (3 inches above and below). This reinforcing is not intended to supersede any structural requirements provided by the structural engineer. Stepped foundations should be designed in accordance with the 2016 CBC.

CBC 2016 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 ‘E’. Due to the negligible sulfates in the site soils, normal cements may be and should be proportioned in accordance with ACI recommendations considering the time of year for placement. Hot weather proportions should be used during high ambient heat days during placement and curing.

Expected Settlement: Estimated total static settlement should be approximately 1 inch, based on footings founded on firm geogrid improved soils as recommended. Differential static settlement between exterior and interior bearing members should be less than 1 inch. As such, considering static, seismic, and collapse settlement applied over a typical foundation distance of 30 feet, we recommend the structural engineer design for an angular distortion of 1:360 (1 inch in 30 feet). Settlement will not result in the complete loss of soil support, but will be manifested as a tilting of the structure over the applied distance. Differential settlement from seismic activity is expected to be attenuated through the use of the geogrid.

Settlement calculations are presented in Appendix A and collapse results are provided in Section 3.4. 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.

Deep Foundations For Bridge Support: As an alternate to spread footing foundations for the bridge support, cast-in-place drilled piers may be used to support foundation loads. We recommend continuous observation by the geotechnical engineer or his representative during drilled pier construction. Drilled piers should have a minimum diameter of 24 inches. The piers should be constructed in accordance with ACI 336.IR-98.

Drilled piers may require temporary casing during installation because of caving dry sand. The contractor should be prepared to accommodate drilling in such conditions. Before placing concrete, a clean-out bucket should be used to remove loose soil from the bottom of the drilled pier excavation. Alternatively, drilling fluid (mud) may be used to stabilize the drill holes. The concrete should be tremie to the bottom of the excavated hole. The tremie can be withdrawn as the concrete fills the hole but should be kept a minimum of 5 feet below the top of concrete (embedded into fresh concrete).

Recommended loading for drilled piers are presented on figures given below. The drilled piers should be designed as skin friction only.

Pedestrian Bridge Allowable Load Graphs for Drilled Shafts: Based on the proposed maximum loading of the Pedestrian Bridge (assumed maximum 35 to 75 kips). Earth Systems provides drilled shaft recommendations for the bridge. The following graphs represent boring B-1 soil profile data and geotechnical recommendations per Section 5.1 of this report.

Figure 1 and 2 values are based on side friction only with Factors of Safety (FOS) of 2 for downward loading and 3 for upward loading applied to ultimate limit state.

Settlements: Total settlements of less than 1 inch and differential settlements of less than ½ inch between similarly loaded piles are anticipated for single piles designed according to the preceding recommendations. If pile spacing is at least 3.0 pile diameters center-to-center, no reduction in axial load capacity is considered necessary for a group effect. Once a drilled shaft type is selected and grouped, Earth Systems would be pleased to conduct additional analysis of group force capacity as additional services, if needed.

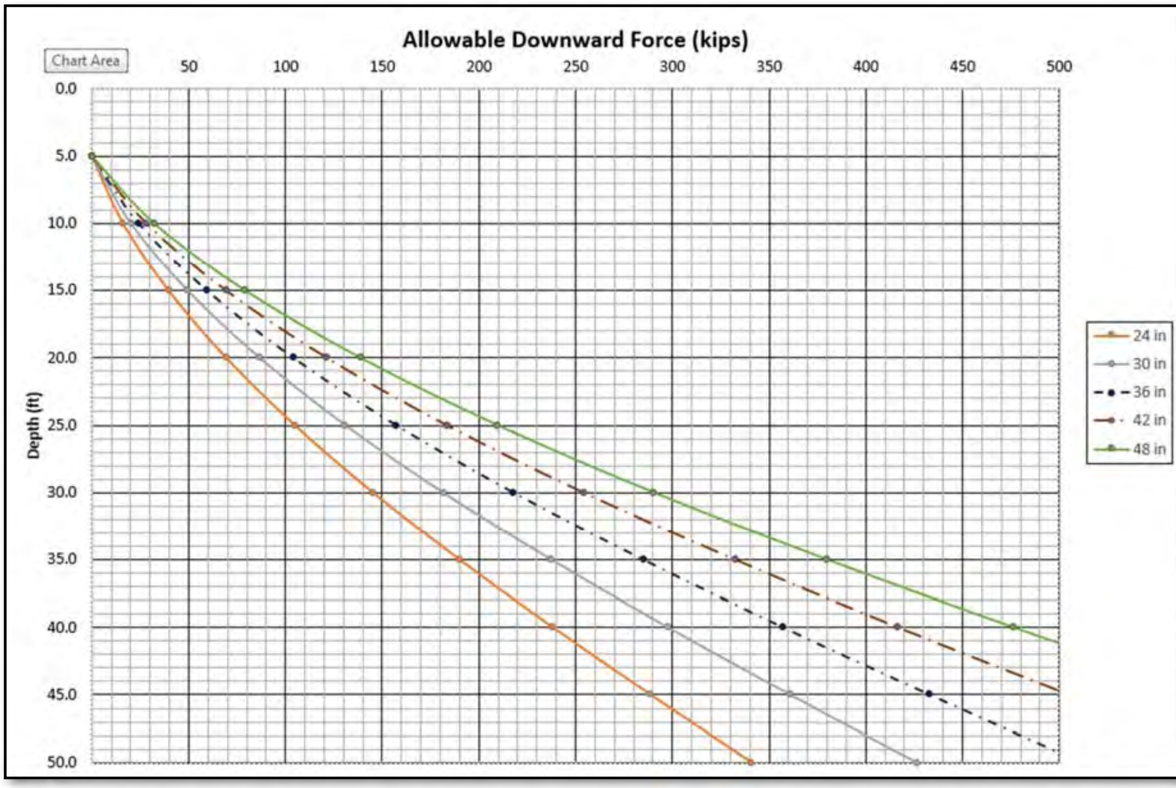


Figure 1 Drilled Shafts Downward Force (FOS = 2)

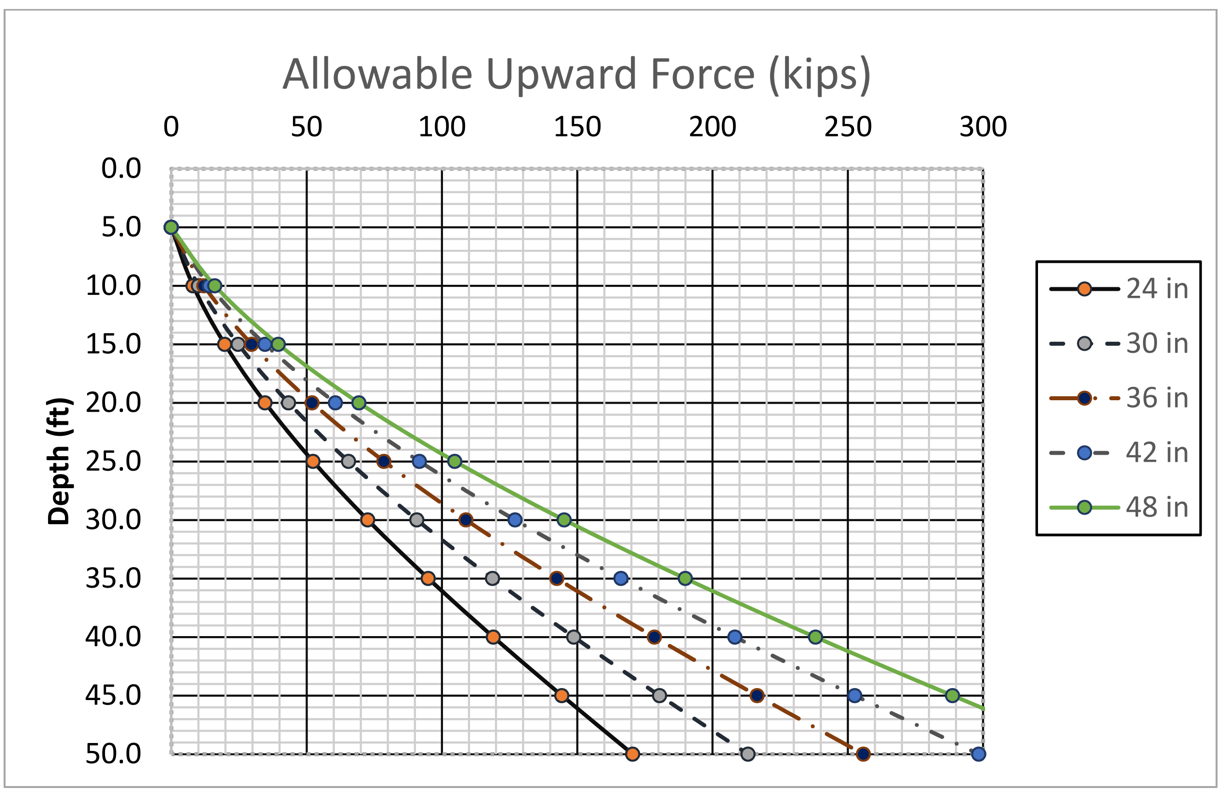


Figure 2 Drilled Shafts Upward Force (FOS = 3)

Downdrag Loading: Downdrag loading occurs during seismic events, subsidence due to groundwater removal, collapse, or fills placed on soft soils overlain by firmer soils. Only approximately ⅛ to ⅜ inch of relative movement is required to mobilize the full side friction resistance, so down drag can even occur in moderately stiff soil if the toe of the pile is bottomed very dense soils. Very dense soils were found at depths of 20 feet and more bgs. Movement of the soil along the pile shaft is estimated to occur between 5 feet to 20 feet bgs and thus these depths were evaluated for down drag forces.

To avoid detrimental buildup of stress within the drilled shaft due to drag forces, Earth Systems recommends shaft lengths be limited in depth of soil layers having relative densities less than “very dense” (below the neutral plane, see Figure 3 below). Therefore, drilled shafts should be limited to a depth of 20 feet to avoid drag down forces. However, if loading requires shafts to terminate on very dense soils, computations of the shaft adequacy should be performed by the structural engineer.

Observing Figure 1 above and noting the smallest project load is 35 kips, this loading can be accommodated on drilled shafts having diameters of 48 inches and a shaft length of just over 10 feet or a drilled shaft diameter of 24 inches with a shaft length of just over 15 feet. Based on the estimated relative movements of native soils occurring between 5 and 20 feet and these depths not contain layering designated “very dense” soils or “rock”, Earth Systems believes down drag forces should not be an issue.

Assuming a single drilled shaft with a project loading of 35 kips, we observe the static loading in Figure 1 above indicates shaft diameters of 24 inches or larger provide load resistance greater than 35 kips for shaft lengths between 11 and 15 feet, respectively. At these depths, soils are not denser than “very dense”, so down drag forces have a low potential for existence.

Assuming a single drilled shaft with a project loading of 75 kips, we observe the static loading in Figure 1 above indicates shaft diameters between 24 and 48 inches provide load resistance greater or equal than 75 kips for shaft lengths between 21 and 15 feet, respectively. At a depth of 20 feet or more, soils become more dense than “very dense”, so down drag forces have been estimated for shafts requiring a depth of 20 feet or more, which is the 24-inch diameter shaft only.

As shown on Table 8, the shaft diameters of 24 to 48 inches are found to provide enough resistance for the project loads of 35 kips without entering into a zone of “very dense” soils that can cause drag down forces. However, Table 9 shows limited analysis for 75-kip loads. The 24-inch diameter shafts require to be embedded in soils with limited settlement, thus drag down force potential is high and as shown the drag down forces exceed the allowable. Additional shaft diameters are provided for lengths between 15 and 19 feet bgs, which shows adequate allowable force and no drag down forces.

Additional shaft analysis for different shaft dimensions can be provided, please contact Earth Systems for additional drilled shaft analysis, if needed.

Table 8
35 Kip and Some 75 kip Project Static Loading with Additional Down-drag Force

Shaft Diameter (inches)	Shaft Length (feet)	Allowable Static Load (kip)	Total Load from Downdrag & Project Load (kip)
24	15	35	35
30	15	50	50
36	15	60	60
42	15	70	70
48	15	80	80

Table 9
75 Kip Project Static Loading with Additional Down-drag Force

Shaft Diameter (inches)	Shaft Length (feet)	Allowable Static Load (kip)	Total Load from Downdrag & Project Load (kip)
24	21	75	225
30	19	75	75
36	17	75	75
42	16	75	75
48	15	75	75

Additional analysis and understanding of the down drag force and design can be gained from review of Figure 3 below. The neutral plane was determined at a depth of 20 feet (for the boring B-1). This plane defines the boundary providing sufficient settlement from dry seismic and collapse loading to mobilize additional drag down forces and those are located above the neutral plane. Below the neutral plane, resistance forces are induced because of very little settlement caused by rock or “very dense” soils. Group piles may provide a greater allowable downward force with lesser shaft length. For additional analysis, please contact Earth Systems.

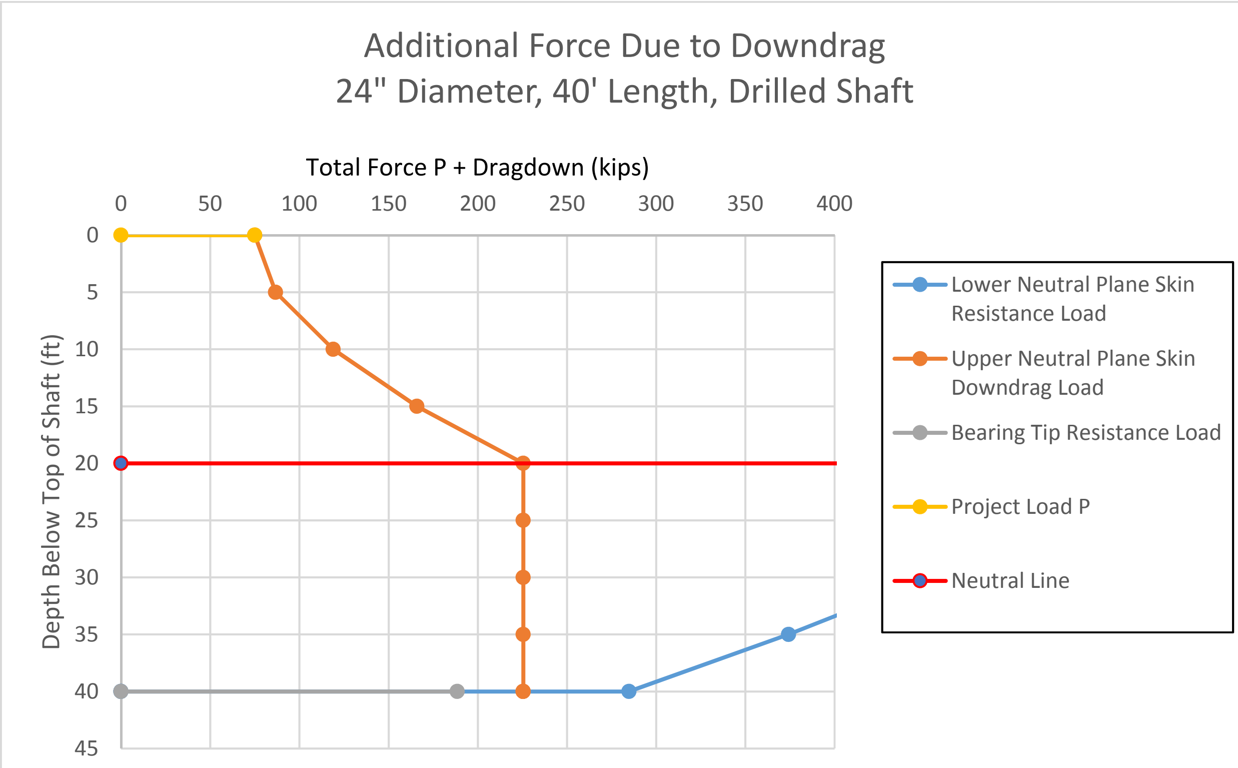


Figure 3 Example Down Drag Force Graph

<u>Lateral Load Capacity:</u> Lateral loading should be computed using software, such as *LPILE (Ensoft, 2013)*, that uses the "p-y Method." Once a pile or pier type is selected, Earth Systems would be pleased to conduct this analysis of lateral load capacity as additional services.

Earth Systems exploration and laboratory testing can be used to estimate the LPILE input parameters. These parameters may be used for LPILE input to estimate the pile deflection.

Table 10

LPILE Parameters

Static Loading, Groundwater 15 feet Below Ground Surface

Location (Boring)	Depth Below Shaft Top (ft)	Effective Unit Weight (pcf)	Soil Friction (ϕ)	Relative Density	Subgrade Modulus Static (k) (pci)*	Subgrade Modulus Static (k) (pci)**
B-1	0 to 15	115	32	Medium	90	
B-1	15 to 22	110 to 72	32	Medium		60
B-1	22 to 50	77	39	Dense		125

*--Reese Sand Above the Water Table / **-- Reese Sand Below the Water Table (Cyclic)

Table 11

LPILE Parameters

Static Loading, Lake is Full, Groundwater 0 feet Below Ground Surface ***

Location (Boring)	Depth Below Shaft Top (ft)	Effective Unit Weight (pcf)	Soil Friction (ϕ)	Relative Density	Subgrade Modulus Static (k) (pci)*	Subgrade Modulus Static (k) (pci)**
B-1	0 to 15	68	32	Medium		60
B-1	15 to 22	67 to 72	32	Medium		60
B-1	22 to 50	77	39	Dense		125

*--Reese Sand Above the Water Table / **-- Reese Sand Below the Water Table/*** -- Most Conservative Static Scenario Considering Adjacent Lake is Full”.

Table 12

LPILE Parameters

Liquefaction Loading, Groundwater 15 feet Below Ground Surface

Location (Boring)	Depth Below Shaft Top (ft)	Effective Unit Weight (pcf)	Soil Friction (ϕ)	Relative Density	Subgrade Modulus Static (k) (pci)*	Subgrade Modulus Static (k) (pci)**
B-1	0 to 15	115	32	Medium	90	
B-1	15 to 22	110 to 72	***	Medium		***
B-1	22 to 50	77	39	Dense		125

*--Reese Sand Above the Water Table / **-- Reese Sand Below the Water Table (Cyclic) / ***--Liquefaction Use Rollins per LPILE

Table 13

LPILE Parameters

Liquefaction Loading, Lake is Full, Groundwater 0 feet Below Ground Surface ****

Location (Boring)	Depth Below Shaft Top (ft)	Effective Unit Weight (pcf)	Soil Friction (ϕ)	Relative Density	Subgrade Modulus Static (k) (pci)*	Subgrade Modulus Static (k) (pci)** And ****
B-1	0 to 15	68	32	Medium		60
B-1	15 to 22	67 to 72	***	Medium		***
B-1	22 to 50	77	39	Dense		125

*--Reese Sand Above the Water Table / **-- Reese Sand Below the Water Table (Cyclic) / ***--Liquefaction Use Rollins per LPILE / **** -- Most Conservative Seismic Scenario Considering Adjacent Lake is Full".

5.5 Slope Construction

Slopes are not generally proposed for this project; however, minor slopes (less than 5 feet in height) may be constructed. Site soils are moderately susceptible to erosion. Compacted fill slopes protected against erosion (per approved methods such as significant planting, facing, or erosion blankets, etc.) should 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 occurs 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 or protected with heavy 12" minimum rip-Rap at 2:1 inclination.

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

Surficial Slope Failures: Site soils are highly susceptible to erosion from wind and water sources. 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 2016 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 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 or greater 24 hours prior to and immediately prior to placing concrete for a depth 12 inches. If the moisture condition is less than indicated, it shall be brought up to or above the indicated moisture content.

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

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

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

Table 14
(Capillary Break)
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 is caused by the difference in drying shrinkage between the top and bottom of the slab. Curling 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: Slabs should be a minimum of 4 inches in actual thickness and be reinforced with #3 bars at 18 inches on center both ways. 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.

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

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

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

Construction (cold) joints should consist of thickened butt joints with ¾-inch dowels at 18 inches on center (12 inches at doorways) embedded per ACI or a thickened keyed-joint to resist vertical deflection at the joint. All control joints in exterior flatwork should be sealed to reduce the potential of moisture or foreign material intrusion. These procedures will reduce the potential for randomly oriented cracks but may not prevent them from occurring.

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

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

5.7 Retaining Walls and Lateral Earth Pressures

Walls which are restrained at the top such as retaining wall returns, below-grade walls and walls tied to floor slabs should be designed with “at rest” earth pressures. Retaining walls, free to tilt at the top, may 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 soils are not suitable for wall backfill as they are not free draining. Silty sand material may be harvested and used for backfill or free draining material imported should be used as wall backfill. For native or import free-draining material, active and restrained walls equivalent fluid pressures are as follows:

- Conventional cantilever retaining walls may be backfilled with compacted on-site soils verified to be "very low" in expansion potential. If testing is not performed by the contractor, we recommend that proposed retaining walls and below grade walls be backfilled with non-expansive, or "very low" expansive import soil. Provided the wall is backfilled at a 1:1 projection upward from the heels of the wall footings with non-expansive sand, an active pressure of 44 pcf of equivalent fluid weight for well-drained, level backfill may be used. Similarly, an active pressure of 54 pcf of equivalent fluid weight may be used for well-drained backfill (i.e., silty sand) sloping at 2H:1V (horizontal to vertical). For the restrained level backfill condition, a pressure of 66 pcf of equivalent fluid weight should be used. An 18-inch thick cap of compacted native sandy soils should be placed above the sand. Filter fabric should be placed between the sand and native soils and/or backfill over the top.

- In addition to the active or at rest soil pressure, the proposed wall structures should be designed to include forces from dynamic (seismic) earth pressure. Dynamic pressures are additive to active and at-rest earth pressure and should be considered as 7 pcf for flexible walls, and 21 pcf for rigid walls. Seismic pressures are based on PGA_M of 0.50 g, Friction Soil Angle of 30^o, and a maximum density of 134 pcf.

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

- A back-drain or an equivalent system of backfill drainage should be incorporated into the wall design, whereby the collected water is conveyed to an approved point of discharge. Design should be in accordance with the 2016 California Building Code. Drain rock should be wrapped in filter fabric such as Mirafi 140N as a minimum and should have a volume of 1 cubic foot per foot of length. Backfill immediately behind the retaining structure should be a free-draining granular material. 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 divert water away from retaining walls.

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

- The above recommended values do not include compaction or truck-induced wall pressures. Care must be taken during the compaction operation not to overstress the wall. Heavy construction equipment should be maintained a distance of at least 3 feet away from the walls while the backfill soils are placed. Upward sloping backfill or surcharge loads from nearby footings can create larger lateral pressures. Should any walls be considered for retaining sloped backfill or placed next to foundations, our office should be contacted for recommended design parameters. Surcharge loads should be considered if they exist within a zone between the face of the wall and a plane projected 45 degrees upward from the base of the wall. The increase in lateral earth pressure should be taken as 50% of the surcharge load within this zone. Retaining walls subjected to traffic loads should include a minimum uniform surcharge load equivalent of 250 psf for auto and 400 psf for truck traffic kept back at least 3 feet from the wall back edge. Closer loads will impart higher pressures on the wall. Retaining walls should be designed with a minimum factor of safety of 1.5.

Frictional and Lateral Coefficients:

- Resistance to lateral loads (including those due to wind or seismic forces) may be provided by frictional resistance between the bottom of concrete foundations and the underlying soil, and by passive soil pressure against the foundations. An allowable coefficient of friction of 0.3 may be used between cast-in-place concrete foundations and slabs and the underlying soil. An allowable coefficient of friction of 0.25 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 350 pounds per cubic foot (pcf). Vertical uplift resistance may consider a soil unit weight of 105 pounds per cubic foot. The upper 1 foot of soil should not be considered when calculating passive pressure unless confined by overlying asphalt concrete pavement or Portland cement concrete slab. 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 350 pcf. The maximum passive resistance should not exceed 2,000 psf.
- Construction employing poles or posts (i.e. lamp posts) may utilize design methods presented in Section 1807.3 of the CBC for Silty Sand soils (SM) material class.
- 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 Elsinore, San Jacinto, Cucamonga, and San Andreas fault zones. Engineered design and earthquake-resistant construction increase safety and allow development of seismic areas. The minimum seismic design should comply with the 2016 edition of the California Building Code and ASCE 7-10 using the seismic coefficients given in the table below. The site is not within Alquist-Priolo or other hazard zone. The site is liquefiable and Site Class F. For structures with a fundamental period less than 0.5 seconds ASCE7-10 allows the site to be classified as D for parameter calculation and design, if not subject to bearing failure. The site is not subject to bearing failure. General Procedure seismic parameters are presented below considering a Site Class D (results in Appendix A).

2016 CBC (ASCE 7-10) Seismic Parameters

Site Coordinates: 33.9703°N and 117.4763°W	
Site Class:	F (D with exceptions)
Maximum Considered Earthquake [MCE] Ground Motion	
Short Period Spectral Response S_S:	1.50 g
1 second Spectral Response, S_1:	0.60 g
Code Design Earthquake Ground Motion	
Short Period Spectral Response, S_{DS}	1.00 g
1 second Spectral Response, S_{D1}	0.60 g
Peak Ground Acceleration (PGA_M)	0.50 g

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

Estimated peak horizontal site accelerations are based upon a probabilistic analysis (2 percent probability of occurrence in 50 years) is approximately 0.9 g for a stiff soil site. Actual accelerations may be more or less than estimated. Vertical accelerations are typically ⅓ to ⅔ of the horizontal accelerations, but can equal or exceed the horizontal accelerations, depending upon the local site effects and amplification.

5.9 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).

Automobile Traffic and Parking Areas: Pavement sections presented in the following table for automobile type traffic areas and are based on a tested R-value and current Caltrans design procedures. Traffic Indices (TI) of 5 and 7 were used to facilitate the design of asphalt concrete pavements for parking and main drives, including fire lanes. The fire lane calculation assumed a conservative traffic flow of one fire truck per day entering and exiting the site on the same path (20 year life cycle), and a maximum loading of an 80,000 lb Tandem Axle apparatus (approximate 20,000 lb front axle load and two 30,000 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 15
Preliminary Flexible Pavement Section Recommendations
On-site/Interior Automobile Drive Areas

R-Value of Subgrade Soils - 25 (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	6.0
7	Main Drive Areas	4	10.5

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

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

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

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

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

Table 16
Preliminary Portland Cement Concrete Pavement Sections

Area	Minimum Pavement PCC Thickness (inches)	Minimum 28 Day Flexural Strength (psi)	Concrete Compressive Strength (psi)
Truck/Bus Access or Loading/Unloading Areas (Traffic Category C, ADTT =100)	7	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:

- Pavement should be placed upon compacted fill processed as described in Section 5.1. The upper 12 inches of subgrade soils beneath the asphalt concrete and conventional PCC pavement section should be compacted to a minimum of 95% relative compaction (ASTM D 1557).
- Subsequent to utility installation, the entire pavement (including PCC) final subgrade should be scarified 12 inches, moisture conditioned to near optimum moisture content, and compacted to a minimum 95% relative compaction immediately prior (within a few days) to the placement and compaction of aggregate base to re-establish proper moisture content and compaction in site soils.
- Subgrade soils and aggregate base should be in a stable, non-pumping condition at the time of placement and compaction. Exposed subgrades should be proof-rolled to verify the absence of soft or unstable zones.
- Aggregate base materials should be compacted at near optimum moisture content to at least 95 percent relative compaction (ASTM D 1557) and should conform to Caltrans Class II criteria. Standard Specifications for Public Works Construction "Greenbook" standards (Crushed Aggregate Base class) may be used in lieu of Caltrans. Compaction efforts should include rubber tire proof-rolling of the aggregate base with heavy compaction-specific equipment (i.e. fully loaded water trucks).
- All concrete curbs separating pavement from landscaped areas should extend at least 6 inches into the subgrade soils to reduce the potential for movement of moisture into the aggregate base layer (this reduces the risk of pavement failures due to subsurface water originating from landscaped areas).
- Asphaltic concrete should be ½-in. or ¾-in. grading and compacted to a minimum of 95% of the 75-blow Marshall density (ASTM D 1559) or equivalent.
- Portland cement concrete pavements should be constructed with transverse joints at maximum spacing of 12 feet. A thickened edge should be used where possible and, as a minimum, where concrete pavements abut asphalt pavements. The thickened edge should

be 1.2 times the thickness of the pavement (8.4 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 suitable curing compound or keep continuously moist for a minimum of seven days;
  4. Keep all traffic off concrete until PCC compressive strength exceeds 2,000 pounds per square inch (truck traffic should be limited until the concrete meets the design strength (3,250 psi); and
  5. Consideration should be given to having PCC construction joints keyed or using slip dowels on 24-inch centers to strengthen control and construction joints. Dowels placed within dowel baskets should be incorporated into the concrete at each saw-cut control joint (i.e. dowel baskets and dowels are set in place prior to placement of concrete).

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

- Within the structural pavement section areas, positive drainage (both surface and subsurface) should be provided. In no instance should water be allowed to pond on the pavement. Roadway performance depends greatly on how well runoff water drains from the site. This drainage should be maintained both during construction and over the entire life of the project.

- Proper methods, such as hot-sealing or caulking, should be employed to limit water infiltration into the pavement base course and/or subgrade at construction/expansion joints and/or between existing and reconstructed asphalt concrete sections (if any). Water infiltration could lead to premature pavement failure.

- To reduce the potential for detrimental settlement, excess soil material, and/or fill material removed during any footing or utility trench excavation, should not be spread or placed over compacted finished grade soils unless subsequently compacted to at least 90% of the maximum dry unit weight, as evaluated by ASTM D 1557 test procedure, at near optimum moisture content, or 95% if placed under areas designated for pavement.

- Where new roadways will be installed against existing roadways, the repaired asphalt concrete pavement section should be designed and constructed to have at least the pavement and aggregate base section as the original pavement section thickness (for both

AC and base) or upon the newly calculated pavement sections presented within, whichever is greater.

- Pavement designs assume that heavy construction traffic will not be allowed on base cap or finished pavement sections.

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, 'V' ditches, or other suitable containment and redirection devices.

- Site drainage should be devised such that runoff should be directed away from the tops of all graded slopes. Water should not freely flow over constructed slopes. Diversion and conveyance structures which can accommodate water and eroded soil should be constructed at the tops and toes of all slopes. Lined swales or berms at the top and bottom of slopes are recommended.

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

- To reduce the potential for bearing loss and soil expansion, 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.

- The drainage pattern should be established at the time of final grading and maintained throughout the life of the project. Additionally, drainage structures should be maintained (including the de-clogging of piping) throughout their design life. Structural performance is dependent on many drainage-related factors such as landscaping, irrigation, lateral drainage patterns and other improvements. Cleanout should be provided in drainage piping.

- Maintenance of drainage systems and infiltration structures can be the most critical element in determining the success of a design. They must be protected and maintained from sediment-laden water both during and after construction to prevent clogging of the surficial soils any filter medium. The potential for clogging can be reduced by pre-treating structure inflow through the installation of maintainable forebays, biofilters, or sedimentation chambers. In addition, sediment, leaves, and debris must be removed from inlets and traps on a regular basis. Since these and other factors (such as varying soil conditions) may affect the rate of water infiltration, it is imperative to apply a conservative factor of safety [FOS] to the unfactored Basic Percolation/Infiltration Rates presented within to provide a reliable basis for design. In order to account not only for the unknown factors above but also for changes of conditions during the use of the structures such as potential clogging effects due to washing in of soil fines, a FOS between 3 and 10 should be applied to lower design infiltration rates.
- The factor of safety should be selected by the project drainage engineer and may be dependent on agency guidelines and the presence of filters and sedimentation structures. If these measures are provided, the factor of safety can be reduced.
- 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.

Section 6
LIMITATIONS AND ADDITIONAL SERVICES

6.1 Uniformity of Conditions and Limitations

Our findings and recommendations in this report are based on selected points of field exploration, laboratory testing, and our understanding of the proposed project. Furthermore, our findings and recommendations assume 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 assumes that an adequate program of client consultation, construction monitoring, and testing will be performed during the final design and construction phases to check compliance with these recommendations. Maintaining Earth Systems as the geotechnical consultant from beginning to end of the project will provide continuity of services.

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

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

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

REFERENCES

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

American Concrete Institute [ACI], 2004, ACI Manual of Concrete Practice, Parts 1 through 5.

American Concrete Institute (2014) "Building Code Requirements for Structural Concrete (ACI 318-11) and Commentary (ACI 318R-14) and ACI 330R."

American Society of Civil Engineers [ASCE], 2010, Minimum Design Loads for Buildings and Other Structures, ASCE 7-10.

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

Boulanger, R.W. and Idriss, I.M., 2006, Liquefaction Susceptibility Criteria for Soils and Clays, ASCE 132 (II) p. 1423-1426.

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

California Department of Water Resources, 2018, Water Data Library Website, http://www.water.ca.gov/waterdatalibrary/.

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

California Geologic Survey, 2018, Ground Motion Interpolator, http://www.quake.ca.gov/gmaps/PSHA/psha.

California Geological Survey, 2010, Fault Activity Map of California, CGS Data Map Series, Map No. 6.

California Geologic Survey, 2018, Earthquake Faults Zones, California Geological Survey Special Publication 42.

County of Los Angeles, Department of Public Works, 2013, Manual for the Preparation of Geotechnical Report, July 1, 2013.

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

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

Earth Consultants International, 2000, Seismic Hazards - County of Riverside, Natural Hazards Mapping, Analysis, and Mitigation: a Technical Background Report in Support of the Safety Element of the New Riverside County 2000 General Plan.

Ellsworth, W.L., 1990, "Earthquake History, 1769-1989" in: The San Andreas Fault System, California: U.S. Geological Survey Professional Paper 1515, 283 p.

Envicom Corporation and the County of Riverside Planning Department, 1976, Seismic Safety and Safety General Plan Elements Technical Report, County of Riverside.

FEMA, 2018, Map Service Center website http*:// msc.fema.gov/*

Fire Apparatus Manufacturers' Association (FAMA), 2011, Emergency Vehicle Size and Weight Regulation Guideline,https://fama.org/wp-content/uploads/2015/09/1441593313_55 ecf7e17d32d.pdf, November 22, 2011.

Hunt, Roy E., 1984, Geotechnical Engineering Practice, McGraw-Hill Book Company.

International Code Council [ICC], 2016, California Building Code, 2016 Edition.

Lommler, C. J. and Bandini, P. 2015, Characterization of Collapsible Soils, International Foundations Congress and Equipment Expo (IFCEE) 2015 ASCE 2015.

Morton, Douglas M., et al, 2001, Geologic Map of the Riverside West 7.5' Quadrangle, Riverside County, California, Version 1.0, USGS Open-File Report 01-451.

Occupational Safety and Health Standards – Excavations, Final Pub. 1989.

Riverside County Transportation and Land Management Agency, 2018, Riverside County Geographic Information Service, https://gis.rivcoit.org.

Riverside County Planning Department, 2003, Geotechnical Element of the Riverside County General Plan.

Rogers, T.H., 1966, Geologic Map of California - Santa Ana Sheet, California Division of Mines and Geology Regional Map Series, scale 1:250,000.

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

Southern California Earthquake Center (SCEC), 2018, Earthquake Data Center: http://www.data.scec.org/significant/chron-index.html.

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

United States Geological Survey, 2008, Documentation for the 2008 Update of the United States National Seismic Hazard Maps: U.S. Geological Survey Open-File Report 2008–1128, 61 p.

United States Geologic Survey, 2018, U.S. Seismic Design Maps Website, http://earthquake.usgs.gov/design maps/us/.

United States Geological Survey, 2018, Unified Hazard Tool, Earthquake Hazards Program, https://earthquake.usgs.gov/hazards/interactive/

United States Geologic Survey and California Geologic Survey, 2006, Quaternary fault and fold database for the United States, assessed March 2015, from USGS web site: http//earthquake.usgs.gov/hazards/qfaults/.

United States Geologic Survey, 2018, ShakeMap Archives Website, http://earthquake.usgs.gov/earthquakes/shake.

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

Working Group on California Earthquake Probabilities, 2008, The Uniform California Earthquake Rupture Forecast, Version 2 [UCERF 2]: U.S. Geological Survey Open-File Report 2007-1437 and California Geological Survey Special Report 203, 104 p.

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

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

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

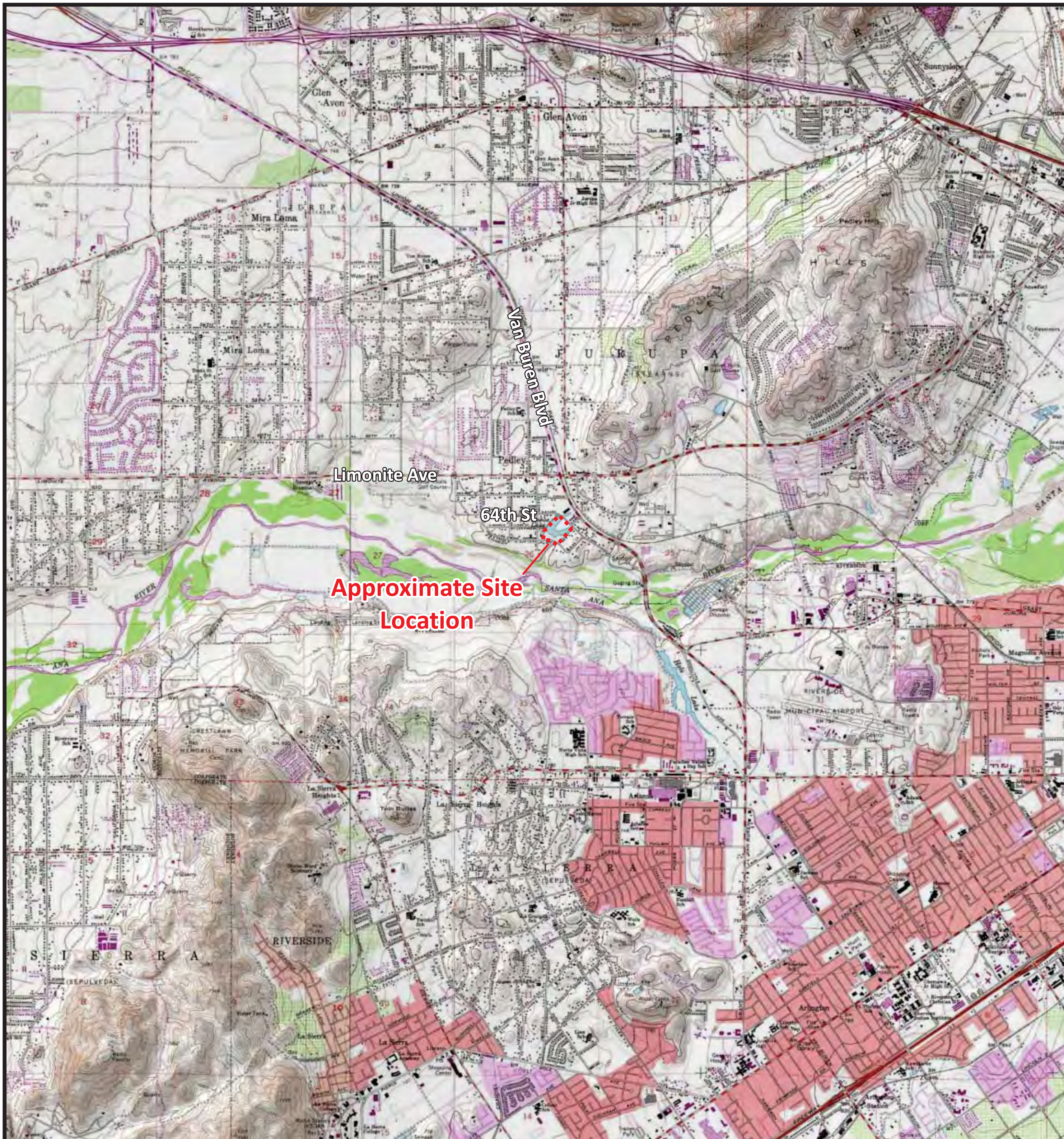
Aerial Photographs:

Google earth: 1994-2018

Historic Aerials: 1948-2014

APPENDIX A

Plate 1 – Site Vicinity Map
Plate 2 – Boring Location Map
Plate 3 – Regional Geologic Map
Plate 4 – Local Geologic Map
Table A-1 Fault Parameters
Terms and Symbols Used on Boring Logs
Soil Classification System
Log of Borings
Site Class Estimator Boring B-1
Seismic Settlement Graphs B-1
Spread Footing Static Load Settlement (1 page)
Continuous Footing Static Load Settlement (1 page)



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

LEGEND

Approximate Site Boundary

Scale: 1" = 1 Mile

0 1 Mile 2 Mile

N

Plate 1
Site Vicinity Map

Proposed Horseshoe Lake Park Improvements
SWC of Lakeview Avenue & Studio Place
Jurupa Valley, Riverside County, California

Earth Systems

10/31/2018	File No.: 302538-001

Source: Preliminary Master Plan, Horseshoe Lake Park, Jurupa Area Recreation & Park District.

LEGEND

- B-7 Approximate Boring Locations
- P-2 Approximate Percolation Test Locations

N

Approximate Scale: 1" = 125'

0 125' 250'

Plate 2
Boring Location Map

Proposed Horseshoe Lake Park Improvements
SWC of Lakeview Avenue & Studio Place
Jurupa Valley, Riverside County, California

Earth Systems

10/31/2018	File No.: 302538-001

LEGEND

Qs	Dune sand	Φc	Oligocene nonmarine
Qal	Alluvium	Ec	Eocene nonmarine
Ql	Lake deposits	E	Eocene marine
Qg	Glacial deposits	Ep	Paleocene marine
Qt	River terrace deposits	Tc	Tertiary nonmarine
Qm	Pleistocene marine and marine terrace deposits	Tm	Tertiary marine
Qpv	Pleistocene volcanic rocks Qpv^r-rhyolite Qpv^a-andesite Qpv^b-basalt Qpv^p-pyroclastic rocks	Ti	Tertiary intrusive (hypabyssal) rocks Ti^r-rhyolite Ti^a-andesite Ti^b-basalt
Qc	Pleistocene nonmarine	Tl	Tertiary lake deposits
QP	Plio-Pleistocene nonmarine	Tv	Tertiary volcanic rocks Tv^r-rhyolite Tv^a-andesite Tv^b-basalt 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 Pv^r-rhyolite Pv^a-andesite Pv^b-basalt Pv^p-pyroclastic rocks	JTv	Jurassic-Triassic metavolcanic rocks
Mc	Undivided Miocene nonmarine	m ls	Pre-Cretaceous metamorphic rocks (ls=limestone)
Muc	Upper Miocene nonmarine	ms	Pre-Cretaceous metasedimentary rocks
Mu	Upper Miocene marine	mv	Pre-Cretaceous metavolcanic rocks
Mm	Middle Miocene marine	gr-m	Pre-Cenozoic granitic and metamorphic rocks
Ml	Lower Miocene marine	pCc	Precambrian igneous and metamorphic rock complex
Mv	Miocene volcanic rocks Mv^r-rhyolite Mv^a-andesite Mv^b-basalt Mv^p-pyroclastic rocks	pC	Undivided Precambrian metamorphic rocks pCg=gneiss pCs=schist pCls=limestone and/or dolomite
		pCgr	Undivided Precambrian granitic rocks

– Fault: Dashed where approximate, dotted where concealed

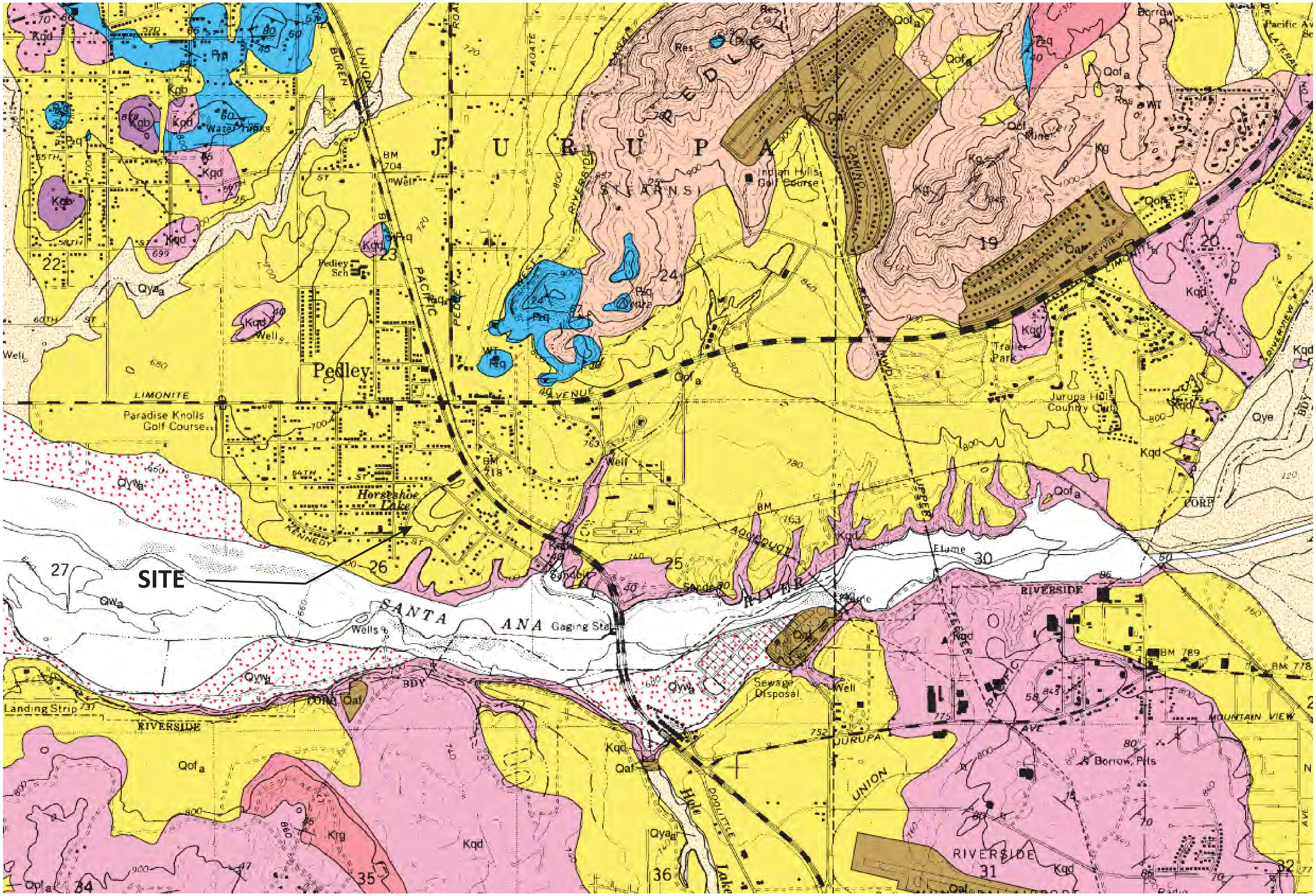
Source: USGS Geologic Map of California, Santa Ana & San Bernardino Sheets.

Plate 3
Regional Geologic Map

Proposed Horseshoe Lake Park Improvements
SWC of Lakeview Avenue & Studio Place
Jurupa Valley, Riverside County, California

Earth Systems

10/31/2018 | File No.: 302538-001



Source: Morton, 2001, USGS Open-File Report 01-451

LEGEND

Qaf: Artificial Fill
Qw: Holocene very young wash deposits of Santa Ana River
Qyw: Holocene to Pleistocene wash deposits of Santa Ana River
Qyf: Holocene young alluvial fan deposits
Qya: Holocene young channel deposits.
Qye: Holocene eolian deposits
Qof: Pleistocene old alluvial fan deposits
Krg: Cretaceous granite of Riverside area.
Kqd: Cretaceous quartz diorite
Pzq: Paleozoic quartzite

Plate 4
Local Geologic Map

Proposed Horseshoe Lake Park Improvements
SWC of Lakeview Avenue & Studio Place
Jurupa Valley, Riveride County, California

Earth Systems

10/31/2018	File No.: 302538-001

Table A-1
Fault Parameters

Fault Section Name	**Distance** (miles)	(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)
Fontana (Seismicity)	5.4	8.7	80	313	na	24	B'	**6.7**		
Chino, alt 2	10.5	16.9	65	234	150	29	B	**6.7**		1
Chino, alt 1	10.6	17.1	50	236	150	24	B	**6.6**		1
Elsinore (Glen Ivy) rev	11.6	18.7	90	218	180	26	A	**7.0**	222	5
Whittier, alt 1	11.7	18.9	70	24	150	46	A	**7.1**	530	2.5
Whittier, alt 2	11.7	18.9	75	24	150	46	A	**7.1**	530	2.5
San Jacinto (San Bernardino)	12.5	20.1	90	225	180	45	A	**7.4**	205	6
Cucamonga	13.6	21.9	45	347	90	28	B	**6.6**		5
San Jacinto (San Jacinto Valley) rev	14.1	22.6	90	223	180	18	A	**7.4**	199	18
Yorba Linda	14.2	22.8	90	153	na	18	B'	**6.5**		
San Jose	15.8	25.3	74	334	30	20	B	**6.6**		0.5
Peralta Hills	17.7	28.5	50	3	na	14	B'	**6.5**		
San Andreas (San Bernardino N)	18.2	29.2	90	212	180	35	A	**7.5**	103	22
San Andreas, (North Branch, Mill Creek)	18.2	29.3	76	204	180	106	A	**7.5**	110	17
Sierra Madre	18.4	29.6	53	19	90	57	B	**7.2**		2
San Andreas (San Bernardino S)	19.1	30.8	90	210	180	43	A	**7.6**	150	16
Richfield	19.8	31.8	28	353	na	6	B'	**6.2**		
Elsinore (Glen Ivy stepover)	20.6	33.1	90	216	180	11	A	**7.1**	322	2.5
Elsinore (Stepovers Combined)	20.6	33.1	90	224	180	12	B'	**6.3**		
Elsinore (Temecula stepover)	21.5	34.6	90	212	180	12	A	**7.6**	725	2.5
San Gabriel (Extension)	21.7	34.9	61	6	180	62	B'	**7.2**		
San Jacinto (Anza, stepover)	21.8	35.1	90	224	180	25	A	**7.6**	151	9
Cleghorn	22.4	36.0	90	187	0	25	B	**6.7**		3
Puente Hills	22.6	36.4	25	20	90	44	B	**7.1**		0.7
San Jacinto (San Jacinto Valley, stepover)	22.6	36.4	90	224	180	24	A	**7.4**	199	9
San Jacinto (Stepovers Combined)	22.6	36.4	90	229	180	25	B'	**6.7**		
Puente Hills (Coyote Hills)	23.0	36.9	26	358	90	17	B	**6.8**		0.7
San Gorgonio Pass	23.5	37.8	60	11	na	29	B'	**6.9**		
San Andreas (Mojave S)	24.2	39.0	90	206	180	98	A	**7.7**	102	29
North Frontal (West)	26.7	43.0	49	171	90	50	B	**7.2**		1
San Joaquin Hills	26.8	43.2	23	204	90	27	B	**7.0**		0.5
Elsinore (Temecula) rev	27.2	43.7	90	230	180	40	A	**7.4**	431	5
Clamshell-Sawpit	28.2	45.3	50	334	90	16	B	**6.6**		0.5
Elysian Park (Lower, CFM)	28.3	45.5	22	33	na	41	B'	**6.8**		
Anaheim	30.0	48.3	71	45	na	16	B'	**6.3**		
Puente Hills (Santa Fe Springs)	31.1	50.1	29	347	90	11	B	**6.6**		0.7
Raymond	32.3	51.9	79	348	60	22	B	**6.7**		1.5
Mission Creek	33.2	53.5	65	5	180	31	B'	**6.9**		
Newport-Inglewood, alt 2	35.9	57.9	90	49	180	66	B	**7.2**		1
Newport-Inglewood, alt 1	36.0	57.9	88	49	180	65	B	**7.2**		1

Reference: USGS OFR 2007-1437 (CGS SP 203)

Based on Site Coordinates of 33.9703 Latitude, -117.4763 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 2007-1437). Mean magntude is average of Ellworths-B and Hanks & Bakun moment area relationship.

DESCRIPTIVE SOIL CLASSIFICATION

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

SOIL GRAIN SIZE

U.S. STANDARD SIEVE

	12"		3"	3/4"	4	10	40	200		
BOULDERS		COBBLES	GRAVEL		SAND			SILT		CLAY
			COARSE	FINE	COARSE	MEDIUM	FINE			
	305		76.2	19.1	4.76	2.00	0.42	0.074	0.002	

SOIL GRAIN SIZE IN MILLIMETERS

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

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

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

CONSISTENCY OF COHESIVE SOILS (CLAY OR CLAYEY SOILS)

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

MOISTURE DENSITY

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

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

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

MOISTURE CONDITION

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

RELATIVE PROPORTIONS

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

PLASTICITY

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

LOG KEY SYMBOLS

Bulk, Bag or Grab Sample

Standard Penetration Split Spoon Sampler (2" outside diameter)

Modified California Sampler (3" outside diameter)

No Recovery

GROUNDWATER LEVEL

Water Level (measured or after drilling)

Water Level (during drilling)

Terms and Symbols Used on Boring Logs

Earth Systems

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

Soil Classification System

Earth Systems

Earth Systems

1680 Illinois Avenue, Suite 20, Perris, CA 92585
Phone (951) 928-9799

Boring No. B-1	Drilling Date: September 27, 2018
Project Name: Horseshoe Lake Park Improvements	Drilling Method: Mobile B-61 w/autohammer
Project Number 302538-001	Drill Type: 8" HSA
Boring Location: See Plate 2	Logged By: S. Clanton

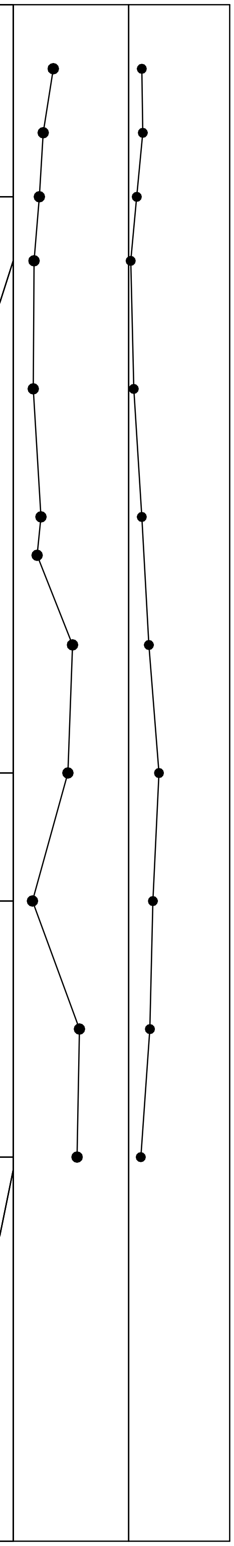
Description of Units

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

Depth (Ft.)	Sample Type (Bulk / SPT / MOD Calif.)	Penetration Resistance (Blows/6")	Symbol	USCS	Dry Density (pcf)	Moisture Content (%)	Description of Units
0	Bulk (0–5)			SM			SILTY SAND: reddish brown, loose, dry, fine to medium grained sand with trace gravel, some clay
	MOD Calif.	15,26,26			113	7	slightly moist, dense
5	MOD Calif.	14,18,21			114	6	
	MOD Calif.	14,18,16		SM	108	7	SILTY SAND WITH GRAVEL: reddish brown, medium dense, slightly moist, fine to medium grained sand, fine gravel
10	MOD Calif.	7,11,16		SM	102	5	SILTY SAND: dark brown, medium dense, slightly moist, fine to coarse grained sand
15	MOD Calif.	9,14,12			105	5	increasing silt content
20	MOD Calif.	11,15,21			113	6	with trace fine gravel
	SPT	12,15,16					
25	MOD Calif.	27,50/3"			120	10	increasing silt content, very dense, trace coarse sand
30	MOD Calif.	21,50/6"		SC	130	11	CLAYEY SAND: dark brown, very dense, moist, fine to coarse grained sand, possible perched water
35	MOD Calif.	25,50/4"		ML	124	14	SANDY SILT: dark brown, hard, moist, fine to medium grained sand
40	MOD Calif.	15,36,50/2"			121	18	with clay
45	MOD Calif.	14,33,50/4"		ML	112	21	SANDY SILT: mottled brown, very dense, moist, fine to medium grained sand
50							
55							
60							

Refusal at 46-1/2 feet due to hard drilling
Backfilled with cuttings
No groundwater encountered

Graphic Trend
Blow Count Dry Density



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Phone (951) 928-9799

Boring No. B-2
Project Name: Horseshoe Lake Park Improvements
Project Number 302538-001
Boring Location: See Plate 2

Drilling Date: September 27, 2018
Drilling Method: Mobile B-61 w/autohammer
Drill Type: 8" HSA
Logged By: S. Clanton

Description of Units

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

Depth (Ft.)	Sample Type (Bulk / SPT / MOD Calif.)	Penetration Resistance (Blows/6")	USCS	Dry Density (pcf)	Moisture Content (%)	Description of Units
0			SM			SILTY SAND: reddish brown, loose, slightly moist, fine to medium grained sand
2.5	MOD Calif.	8,8,41	SP-SM	109	7	SAND WITH SILT: reddish brown, medium dense, slightly moist, fine to coarse grained sand, some clay
5	MOD Calif.	28,50/5"		116	6	

Graphic Trend: Blow Count, Dry Density

Depth scale: 0, 5, 10, 15, 20, 25, 30, 35, 40, 45, 50, 55, 60

Rufusal at 6 feet
Backfilled with cuttings
No groundwater encountered

Earth Systems
1680 Illinois Avenue, Suite 20, Perris, CA 92585
Phone (951) 928-9799

Boring No. B-3	Drilling Date: September 27, 2018
Project Name: Horseshoe Lake Park Improvements	Drilling Method: Mobile B-61 w/autohammer
Project Number 302538-001	Drill Type: 8" HSA
Boring Location: See Plate 2	Logged By: S. Clanton

Description of Units

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

Depth (Ft.)	Sample Type: Bulk	Sample Type: SPT	Sample Type: MOD Calif.	Penetration Resistance (Blows/6")	Symbol	USCS	Dry Density (pcf)	Moisture Content (%)	Description of Units	Graphic Trend: Blow Count	Graphic Trend: Dry Density
0						SM			SILTY SAND: reddish brown, very dense, moist, fine to medium grained sand		
			■	10,20,50/4"			118	10			
5			■	50/6"				11			
10											
15											
20											
25											
30											
35											
40											
45											
50											
55									Boring completed at 5-1/2 feet Backfilled with cuttings No groundwater encountered		
60											

Earth Systems

1680 Illinois Avenue, Suite 20, Perris, CA 92585
Phone (951) 928-9799

Boring No. B-4

Project Name: Horseshoe Lake Park Improvements
Project Number 302538-001
Boring Location: See Plate 2

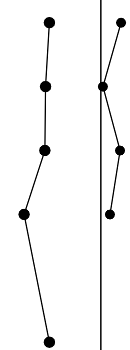
Drilling Date: September 27, 2018
Drilling Method: Mobile B-61 w/autohammer
Drill Type: 8" HSA
Logged By: S. Clanton

Description of Units

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

Depth (Ft.)	Sample Type (Bulk / SPT / MOD Calif.)	Penetration Resistance (Blows/6")	USCS	Dry Density (pcf)	Moisture Content (%)	Description of Units
0	Bulk (0–4.5)		SM			SILTY SAND: reddish brown, medium dense, slightly moist, fine to medium grained sand
	MOD Calif.	23,33,50		120	6	dense
5	MOD Calif.	28,50		102	9	fine to coarse grained sand
	MOD Calif.	27,50/5"		119	11	increasing silt content, moist, very dense
10	MOD Calif.	50/6"	ML	109	13	SANDY SILT: reddish brown, hard, moist, fine to medium grained sand
15	MOD Calif.	29,33,50/5"	SM		16	SILTY SAND: reddish brown, very dense, moist, fine to coarse grained sand

Graphic Trend
Blow Count Dry Density



Boring completed at 16-1/2 feet
Backfilled with cuttings
No groundwater encountered

Earth Systems

1680 Illinois Avenue, Suite 20, Perris, CA 92585
Phone (951) 928-9799

Boring No. B-5

Project Name: Horseshoe Lake Park Improvements
Project Number 302538-001
Boring Location: See Plate 2

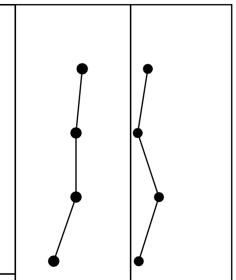
Drilling Date: September 27, 2018
Drilling Method: Mobile B-61 w/autohammer
Drill Type: 8" HSA
Logged By: S. Clanton

Description of Units

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

Depth (Ft.)	Sample Type: Bulk	Sample Type: SPT	Sample Type: MOD Calif.	Penetration Resistance (Blows/6")	Symbol	USCS	Dry Density (pcf)	Moisture Content (%)	Description of Units
0						SM			SILTY SAND: reddish brown, loose, dry, fine to medium grained sand
			■	10,37,50/5"			117	10	dense, moist
5			■	29,50/5"			107	8	very dense, trace fine gravel
			■	29,50/3"			128	10	increasing silt content
10			■	50/6"			108	9	
15									
20									
25									
30									
35									
40									
45									
50									
55									Boring completed at 10-1/2 feet Backfilled with cuttings No groundwater encountered
60									

Graphic Trend
Blow Count Dry Density



Earth Systems

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Boring No. B-6

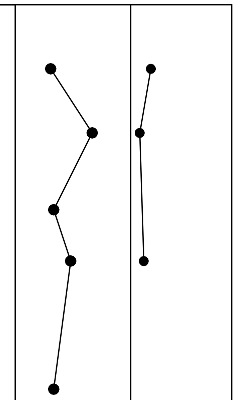
Project Name: Horseshoe Lake Park Improvements
Project Number 302538-001
Boring Location: See Plate 2

Drilling Date: September 27, 2018
Drilling Method: Mobile B-61 w/autohammer
Drill Type: 8" HSA
Logged By: S. Clanton

Depth (Ft.)	Sample Type: Bulk	Sample Type: SPT	Sample Type: MOD Calif.	Penetration Resistance (Blows/6")	Symbol	USCS	Dry Density (pcf)	Moisture Content (%)	Description of Units
0	■					SM			SILTY SAND: reddish brown, loose, dry, fine to medium grained sand
	■		■	19,20,26			120	6	dense, slightly moist
5			■	23,50/5"			109	11	moist
			■	50/6"				8	increasing silt content
10			■	22,50/6"			113	14	
15			■	50/5"					very dense, fine to medium grained sand

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

Graphic Trend
Blow Count Dry Density



Boring completed at 15-1/2 feet
Backfilled with cuttings
No groundwater encountered

Earth Systems
1680 Illinois Avenue, Suite 20, Perris, CA 92585
Phone (951) 928-9799

Boring No. B-7

Project Name: Horseshoe Lake Park Improvements
Project Number 302538-001
Boring Location: See Plate 2

Drilling Date: September 27, 2018
Drilling Method: Mobile B-61 w/autohammer
Drill Type: 8" HSA
Logged By: S. Clanton

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

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: brown, loose, dry, fine to medium grained sand
	MOD Calif.	5,6,9			113	3	damp
5	MOD Calif.	5,3,4			95	6	slightly moist
10							
15							
20							
25							
30							
35							
40							
45							
50							
55							Boring completed at 6-1/2 feet Backfilled with cuttings No groundwater encountered
60							

Boring No.	B-1	Project and Number	Horseshoe Lake Park	302538-001
		ESSW Field Staff	SC	
		Drilling Company	CalPac Drilling	
		Drilling Method	8" HSA	HSA Inner Diameter 3"
		Site Latitude (North)	Decimal Degrees 33.9703	
		Site Longitude (West)	Decimal Degrees -117.4763	

Input	Value
Date Drilled	9/27/2018
Hammer Weight (lbs)	140
Hammer Drop (inches)	30
Hammer Efficiency (E_M)	68
Borehole Correction (Cb)*	1

*inside diameter of Hollow Stem Auger

Calculation Results
Ave. SPT N-value (blows/ft)
37
(Based on Upper 46.5 feet)
Ave. Shear Wave Velocity (ft/sec)
966
(Based on Upper 46.5 feet)
Soil Profile Type (Site Class)
D
(Based on Upper 46.5 feet)
Ave. Friction Angle (degrees)
36
(Based on Upper 46.5 feet)
Estimated Shear Wave Velocity ** Based on Depth Less than 100' (ft/sec)
969 (Upper 100 feet)
Soil Profile Type (Site Class)**
D

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)

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).

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

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

1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
Bottom of Layer Depth (ft)	Blow Count***	Type of Sampler	d_i (feet)	N_{60} (blows/ft)	N70 (blows/ft)	N_{60HE} (blows/ft)	V_{si}** (m/sec)	V_{si} (ft/sec)	Φ_i (degrees)	d_i/N_{60i}	d_i/V_{si}	d_i/Φ_i	Consistency if Coarse Grained (Based on ASTM and Corrected for N60)	Consistency if Fine Grained (Based on ASTM and Corrected for N60)
5.0	52	c	5.0	29.61	25.38	39.49	291.83	957.21	35.85	0.12663	0.00522	0.139457	Medium Dense	Very Stiff
7.5	39	c	2.5	22.21	19.04	29.61	268.47	880.59	34.25	0.08442	0.00284	0.072997	Medium Dense	Very Stiff
9.5	34	c	2.0	19.36	16.60	25.82	258.00	846.24	33.53	0.07747	0.00236	0.059653	Medium Dense	Very Stiff
15.0	27	c	5.5	17.43	14.94	20.50	241.32	791.52	32.38	0.26827	0.00695	0.169872	Medium Dense	Very Stiff
20.0	26	c	5.0	18.76	16.08	19.74	238.69	782.90	32.20	0.25326	0.00639	0.155298	Medium Dense	Very Stiff
21.5	36	c	1.5	25.97	22.26	27.34	262.31	860.39	33.82	0.05487	0.00174	0.044347	Medium Dense	Very Stiff
25.0	31	s	3.5	40.05	34.33	35.13	282.11	925.33	35.19	0.09962	0.00378	0.099472	Dense	Hard
30.0	100	c	5.0	75.93	65.09	75.93	352.77	1157.09	40.03	0.06585	0.00432	0.124906	Very Dense	Hard
35.0	100	c	5.0	75.93	65.09	75.93	352.77	1157.09	40.03	0.06585	0.00432	0.124906	Very Dense	Hard
40.0	100	c	5.0	75.93	65.09	75.93	352.77	1157.09	40.03	0.06585	0.00432	0.124906	Very Dense	Hard
45.0	86	c	5.0	65.30	55.97	65.30	337.67	1107.57	39.00	0.07657	0.00451	0.128216	Very Dense	Hard
46.5	83	c	1.5	63.02	54.02	63.02	334.21	1096.22	38.76	0.02380	0.00137	0.0387	Very Dense	Hard
		Total:	46.5	Feet				Total:		1.26244	0.04813	1.282729		

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

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

Consistency classification based upon ASCE 1996

Spreadsheet Version 2.4, 2016: Prepared by Kevin L. Paul, PE, GE

LIQUEFY-v 2.3.XLS - A SPREADSHEET FOR EMPIRICAL ANALYSIS OF LIQUEFACTION POTENTIAL AND INDUCED GROUND SUBSIDENCE

Project: Horseshoe Lake Park
Job No: 302538-001
Date: 10/17/2018
Boring: B-1 Data Set: 1

Methods: Liquefaction Analysis using 1996 & 1998 NCEER workshop method (Youd & Idriss, editors)
Journal of Geotechnical and Enviromental Engineering (JGEE), October 2001, Vol 127, No. 10, ASCE
Settlement Analysis from Tokimatsu and Seed (1987), JGEE,Vol 113, No.8, ASCE
Modified by Pradel, JGEE, Vol 124, No. 4, ASCE

EARTHQUAKE INFORMATION:
Magnitude: 8.2 7.5
PGA, g: 0.50 0.63
MSF: 0.80
GWT: 13.0 feet
Calc GWT: 13.0 feet
Remediate to: 4.0 feet

SPT N VALUE CORRECTIONS:
Energy Correction to N60 (C_E): 1.20
Drive Rod Corr. (C_R): 1 Default
Rod Length above ground (feet): 3.0
Borehole Dia. Corr. (C_B): 1.00
Sampler Liner Correction for SPT?: 1 Yes
Cal Mod/ SPT Ratio: 0.63

Threshold Acceler., g: 0.39

Total (ft) Liquefied Thickness: 5

Total (in.) Induced Subsidence: 1.0
upper 50 ft

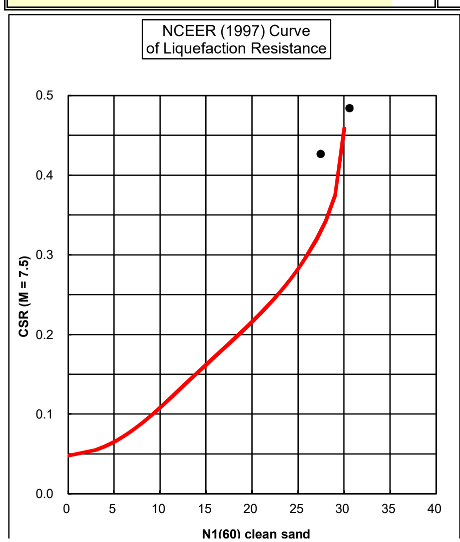
Required SF: 1.50
Minimum Calculated SF: 0.77

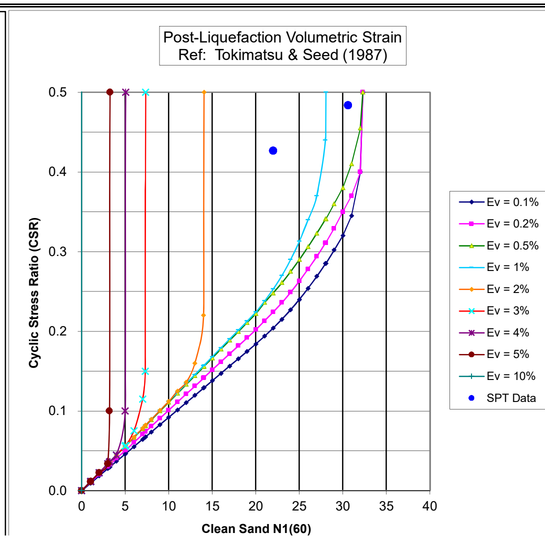
SETTLEMENT (SUBSIDENCE) OF DRY SANDS
Nc = 22.5

Base Depth (feet)	Cal Mod N	SPT N	Liquef. Suscept. (0 or 1)	Total Unit Wt. (pcf)	Fines Content (%)	Depth of SPT (feet)	Rod Length (feet)
5.0	52	33	1	121	31	2.5	5.5
7.5	39	25	1	121	31	5.0	8.0
9.5	34	21	1	115	31	7.5	10.5
15.0	27	17	1	107	14	10.0	13.0
20.0	26	16	1	110	31	15.0	18.0
21.5	36	23	1	119	14	20.0	23.0
25.0		31	1	119	14	21.5	24.5
30.0	100	63	1	132	31	25.0	28.0
35.0	100	63	1	144	35	30.0	33.0
40.0	100	63	1	141	50	35.0	38.0
45.0	86	54	1	142	50	40.0	43.0
46.5	83	52	1	136	50	45.0	48.0

Base Depth (feet)	Tot.Stress at SPT po (tsf)	Eff.Stress at SPT p'o (tsf)	rd	C_N	C_R	C_S	$N_{1(60)}$	Rel. Dens. Dr (%)	Trigger FC Adj. $\Delta N_{1(60)}$	Equiv. Sand $N_{1(60)CS}$	Kσ	M = 7.5 Available CRR	M = 7.5 Induced CSR*	Liquefac. Safety Factor	Post FC Adj. $\Delta N_{1(60)}$	$N_{1(60)CS}$	Volumetric Strain (%)	Induced Subsidence (in.)
	0.000																	
5.0	0.151	0.151	1.00	1.70	0.75	1.00	50.1	85	10.0	60.1	1.00	1.200	0.407	Non-Liq.	10.0	60.1	0.01	0.01
7.5	0.303	0.303	0.99	1.70	0.75	1.00	37.6	73	10.0	47.6	1.00	1.200	0.405	Non-Liq.	10.0	47.6	0.02	0.01
9.5	0.454	0.454	0.98	1.53	0.75	1.00	29.4	65	9.6	39.0	1.00	1.200	0.402	Non-Liq.	9.6	39.0	0.04	0.01
15.0	0.596	0.596	0.98	1.33	0.76	1.00	20.6	54	3.1	23.7	1.00	0.263	0.400	Non-Liq.	3.1	23.7	0.00	0.00
20.0	0.863	0.801	0.97	1.15	0.86	1.00	19.5	53	7.9	27.5	1.00	0.330	0.427	0.77	2.5	22.0	1.42	0.85
21.5	1.138	0.920	0.96	1.07	0.93	1.00	27.3	62	3.4	30.6	1.00	1.200	0.484	2.48	3.4	30.6	0.69	0.12
25.0	1.227	0.962	0.95	1.05	0.95	1.30	48.2	83	4.2	52.4	1.00	1.200	0.497	2.42	4.2	52.4	0.00	0.00
30.0	1.436	1.061	0.94	1.00	0.98	1.00	74.2	100	10.0	84.2	1.00	1.200	0.521	2.30	10.0	84.2	0.00	0.00
35.0	1.766	1.235	0.92	0.93	1.00	1.00	70.0	100	10.0	80.0	0.94	1.200	0.572	2.10	10.0	80.0	0.00	0.00
40.0	2.126	1.439	0.89	0.86	1.00	1.00	64.8	96	10.0	74.8	0.88	1.200	0.608	1.97	10.0	74.8	0.00	0.00
45.0	2.478	1.636	0.85	0.80	1.00	1.00	52.3	86	10.0	62.3	0.84	1.200	0.627	1.91	10.0	62.3	0.00	0.00
46.5	2.833	1.835	0.80	0.76	1.00	1.00	47.7	83	10.0	57.7	0.80	1.200	0.632	1.90	10.0	57.7	0.00	0.00

Base Depth (feet)	p (tsf)	G_{max} (tsf)	τ_{av} (tsf)	Shear Strain γ	Strain E_{15}	Strain Enc	Dry Sand Subsidence (in.)
5.0	0.101	557	0.049	1.7E-04	4.5E-05	5.4E-05	0.01
7.5	0.203	729	0.097	2.6E-04	9.3E-05	1.1E-04	0.01
9.5	0.304	836	0.145	3.5E-04	1.6E-04	1.9E-04	0.01
15.0	0.399	811	0.190	5.9E-04			
20.0	0.578	1,026	0.272	5.9E-04			
21.5	0.762	1,221	0.354	5.9E-04			
25.0	0.822	1,517	0.380	4.2E-04			
30.0	0.962	1,922	0.439	3.4E-04			
35.0	1.183	2,095	0.528	3.7E-04			
40.0	1.424	2,248	0.615	4.0E-04			
45.0	1.660	2,283	0.685	4.5E-04			
46.5	1.898	2,379	0.740	4.6E-04			





$N_{1(60)} = C_N{*}C_E{*}C_B{*}C_R{*}C_S{*}N$

C_R = 0.75 for Rod lengths < 3m, 1.0 for > 10m
$= \min(1,\max(0.75,1.4666-2.556/(z(ft))^{0.5}))$

$C_N = (1\ atm/p'o)^{0.5}$, max 1.7

$C_S = \max(1.1,\min(1.3,1+N_{1(60)}/100))$ for SPT without liners

$MSF = 10^{2.24}/M^{2.56}$

z = Depth (m)

pa = 1 atm = 101 KPa = 1.058 tsf

$p = 0.67{*}po$

$\tau_{av} = 0.65{*}PGA{*}po{*}rd$

$G_{max} = 447{*}N_{1(60)CS}{}^{(1/3)}{*}p^{0.5}$

$a = 0.0389{*}(p/1)+0.124$

$b = 6400{*}(p/1)^{(-0.6)}$

$\gamma = [1+a{*}EXP(b{*}\tau_{av}/G_{max})]/[(1+a){*}\tau_{av}/G_{max}]$

$E_{15} = \gamma{*}(N_{1(60)CS}/20)^{-1.2}$

$E_{nc} = (Nc/15)^{0.45}{*}E15$

$Nc = (MAG-4)^{2.17}$

$S = 2{*}H{*}E_{nc}$

rd = (1-0.4113*z^0.5+0.04052*z+0.001753*z^1.5)/(1-0.4177*z^0.5+0.05729*z-0.006205*z^1.5+0.00121*z^2))

$\Delta N_{1(60)}$ = min(10,IF(FC<35,exp(1.76-(190/FC^2)),5)+IF(FC<=5,1,IF(FC<35,0.99+(FC^1.5/1000),1.2))*N1(60) - N1(60)

$N_{1(60)CS} = N_{1(60)CS} + \Delta N_{1(60)}$

$K\sigma$ = min of 1.0 or $(p'o/1.058)^{(IF(Dr>0.7,0.6,IF(Dr<0.5,0.8,0.7))-1)}$

$Dr = (N_{1(60)}/70)^{0.5}$

$CSReq = 0.65{*}PGA{*}(po/p'o){*}rd$

$CSR^{*} = CSReq/MSF/K\sigma$

$CRR_{7.5}$ = (0.048-0.004721*N+0.0006136*N^2-0.00001673*N^3)/(1-0.1248*N+0.009578*N^2-0.0003285*N^3+0.000003714*N^4))

$N = N_{1(60)CS}$

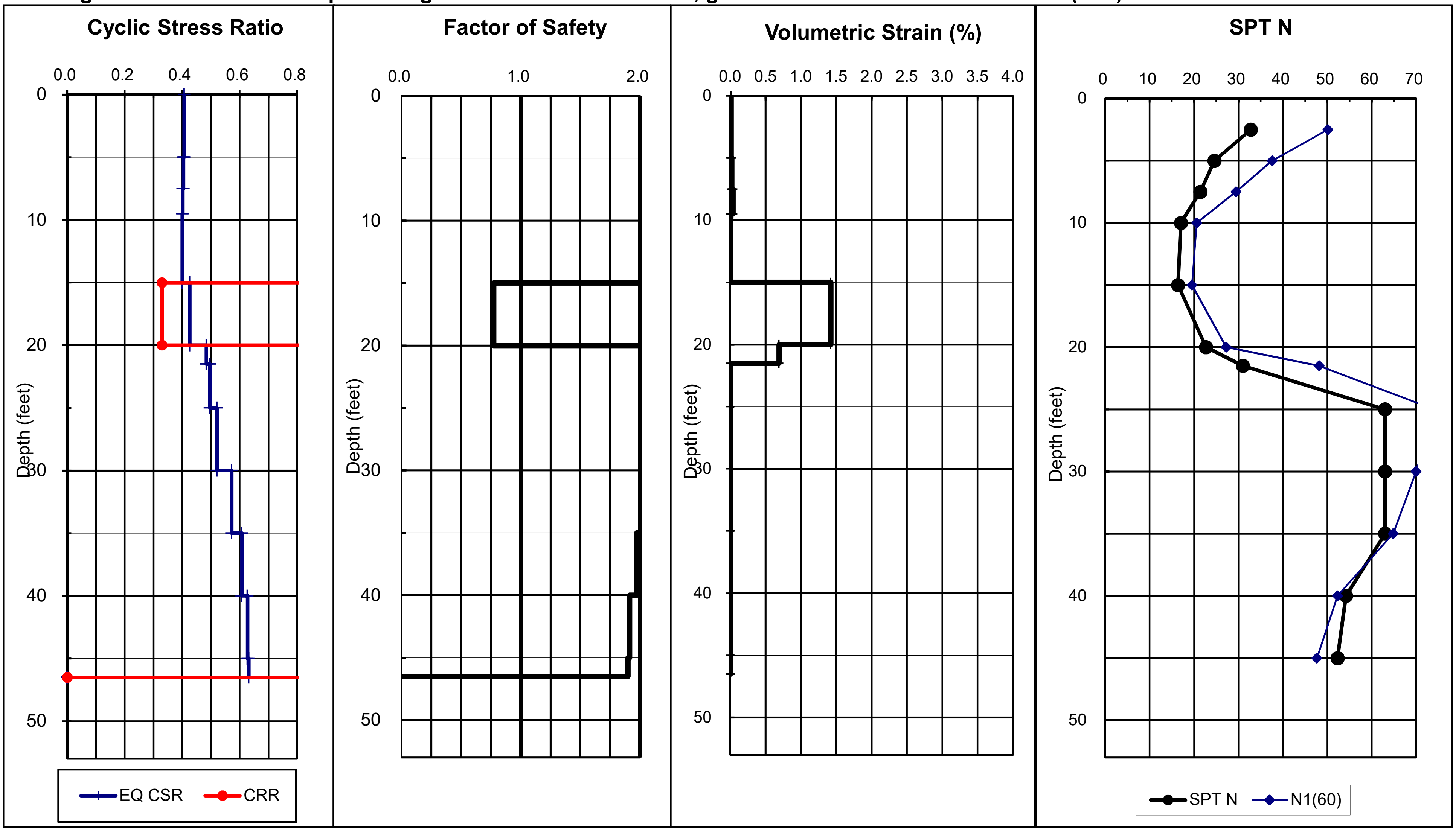
$SF = CRR_{7.5,1atm}/CSR^{*}$

EARTH SYSTEMS - EVALUATION OF LIQUEFACTION POTENTIAL AND INDUCED SUBSIDENCE

Horseshoe Lake Park **Project No: 302538-001**

1996/1998 NCEER Method
Ground Compaction Remediated to 4 foot depth

Boring: B-1 **Earthquake Magnitude: 8.2** **PGA, g: 0.50** **Calc GWT (feet): 13**



Total Thickness of Liquefiable Layers: 5.0 feet

Estimated Total Ground Subsidence: 1.0 inches

LIQUEFY-v 2.3.XLS - A SPREADSHEET FOR EMPIRICAL ANALYSIS OF LIQUEFACTION POTENTIAL AND INDUCED GROUND SUBSIDENCE

Coryright & Developed 2007 by Shelton L. Stringer, PE, GE, PG , EG - Earth Systems Southwest

Project: Horseshoe Lake Park
Job No: 302538-001
Date: 10/17/2018
Boring: B-1 Data Set: 1

Methods: Liquefaction Analysis using 1996 & 1998 NCEER workshop method (Youd & Idriss, editors)
Journal of Geotechnical and Enviromental Engineering (JGEE), October 2001, Vol 127, No. 10, ASCE
Settlement Analysis from Tokimatsu and Seed (1987), JGEE,Vol 113, No.8, ASCE
Modified by Pradel, JGEE, Vol 124, No. 4, ASCE

EARTHQUAKE INFORMATION:

Magnitude: 8.2 7.5
PGA, g: 0.33 0.42
MSF: 0.80
GWT: 50.0 feet
Calc GWT: 50.0 feet
Remediate to: 4.0 **feet**

SPT N VALUE CORRECTIONS:

Energy Correction to N60 (C_E): 1.20
Drive Rod Corr. (C_R): 1 Default
Rod Length above ground (feet): 3.0
Borehole Dia. Corr. (C_B): 1.00
Sampler Liner Correction for SPT?: 1 Yes
Cal Mod/ SPT Ratio: 0.63

Threshold Acceler., g: **#N/A**

Total (ft) Liquefied Thickness	Total (in.) Induced Subsidence
0	0.1
	upper 50 ft

Required SF: **1.50**
Minimum Calculated SF: **#N/A**

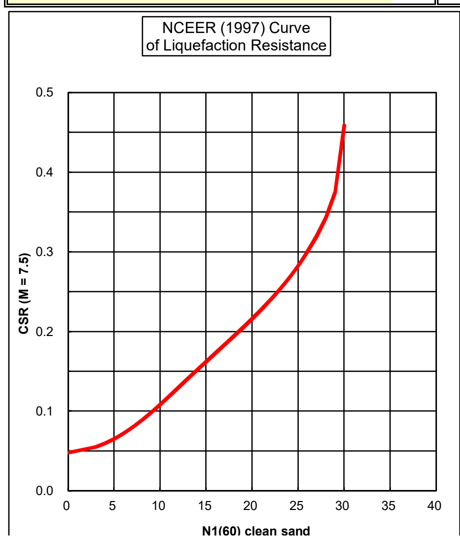
SETTLEMENT (SUBSIDENCE) OF DRY SANDS

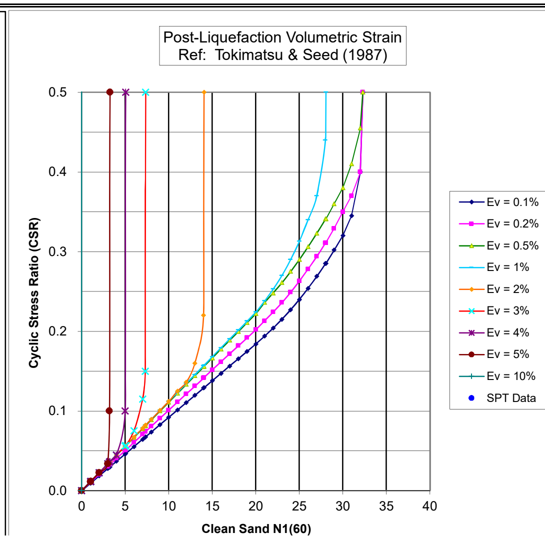
Nc = 22.5

Base Depth (feet)	Cal Mod N	SPT N	Liquef. Suscept. (0 or 1)	Total Unit Wt. (pcf)	Fines Content (%)	Depth of SPT (feet)	Rod Length (feet)
5.0	52	**33**	1	121	31	2.5	5.5
7.5	39	**25**	1	121	31	5.0	8.0
9.5	34	**21**	1	115	31	7.5	10.5
15.0	27	**17**	1	107	14	10.0	13.0
20.0	26	**16**	1	110	31	15.0	18.0
21.5	36	**23**	1	119	14	20.0	23.0
25.0		**31**	1	119	14	21.5	24.5
30.0	100	**63**	1	132	31	25.0	28.0
35.0	100	**63**	1	144	35	30.0	33.0
40.0	100	**63**	1	141	50	35.0	38.0
45.0	86	**54**	1	142	50	40.0	43.0
46.5	83	**52**	1	136	50	45.0	48.0

Base Depth (feet)	Tot.Stress at SPT po (tsf)	Eff.Stress at SPT p'o (tsf)	rd	C_N	C_R	C_S	$N_{1(60)}$	Rel. Dens. Dr (%)	*Trigger FC Adj.* $\Delta N_{1(60)}$	*Equiv. Sand* $N_{1(60)CS}$	$K\sigma$	M = 7.5 Available CRR	M =7.5 Induced CSR*	**Liquefac. Safety Factor**	*Post FC Adj.* $\Delta N_{1(60)}$	$N_{1(60)CS}$	Volumetric *Strain (%)*	**Induced Subsidence (in.)**
	0.000																	
5.0	0.151	0.151	1.00	1.70	0.75	1.00	50.1	85	10.0	60.1	1.00	1.200	0.271	**Non-Liq.**	10.0	60.1	0.01	**0.00**
7.5	0.303	0.303	0.99	1.70	0.75	1.00	37.6	73	10.0	47.6	1.00	1.200	0.269	**Non-Liq.**	10.0	47.6	0.01	**0.00**
9.5	0.454	0.454	0.98	1.53	0.75	1.00	29.4	65	9.6	39.0	1.00	1.200	0.268	**Non-Liq.**	9.6	39.0	0.02	**0.00**
15.0	0.596	0.596	0.98	1.33	0.76	1.00	20.6	54	3.1	23.7	1.00	0.263	0.266	**Non-Liq.**	3.1	23.7	0.05	**0.03**
20.0	0.863	0.863	0.97	1.11	0.86	1.00	18.8	52	7.8	26.6	1.00	0.312	0.264	**Non-Liq.**	7.8	26.6	0.05	**0.03**
21.5	1.138	1.138	0.96	0.96	0.93	1.00	24.5	59	3.2	27.7	0.98	0.336	0.266	**Non-Liq.**	3.2	27.7	0.05	**0.01**
25.0	1.227	1.227	0.95	0.93	0.95	1.30	42.7	78	4.0	46.7	0.94	1.200	0.275	**Non-Liq.**	4.0	46.7	0.02	**0.01**
30.0	1.436	1.436	0.94	0.86	0.98	1.00	63.8	95	10.0	73.8	0.89	1.200	0.290	**Non-Liq.**	10.0	73.8	0.01	**0.01**
35.0	1.766	1.766	0.92	0.77	1.00	1.00	58.5	91	10.0	68.5	0.81	1.200	0.307	**Non-Liq.**	10.0	68.5	0.01	**0.01**
40.0	2.126	2.126	0.89	0.71	1.00	1.00	53.3	87	10.0	63.3	0.76	1.200	0.320	**Non-Liq.**	10.0	63.3	0.01	**0.01**
45.0	2.478	2.478	0.85	0.65	1.00	1.00	42.5	78	10.0	52.5	0.71	1.200	0.325	**Non-Liq.**	10.0	52.5	0.02	**0.01**
46.5	2.833	2.833	0.80	0.61	1.00	1.00	38.3	74	10.0	48.3	0.67	1.200	0.324	**Non-Liq.**	10.0	48.3	0.02	**0.00**

Base Depth (feet)	p (tsf)	G_{max} (tsf)	τ_{av} (tsf)	Shear Strain γ	Strain E_{15}	Strain Enc	**Dry Sand Subsidence (in.)**
5.0	0.101	557	0.033	8.1E-05	2.2E-05	2.6E-05	**0.00**
7.5	0.203	729	0.065	1.2E-04	4.4E-05	5.3E-05	**0.00**
9.5	0.304	836	0.097	1.6E-04	7.4E-05	8.9E-05	**0.00**
15.0	0.399	811	0.126	2.4E-04	2.0E-04	2.4E-04	**0.03**
20.0	0.578	1,015	0.181	2.7E-04	1.9E-04	2.3E-04	**0.03**
21.5	0.762	1,182	0.236	2.9E-04	2.0E-04	2.4E-04	**0.01**
25.0	0.822	1,459	0.253	2.3E-04	8.4E-05	1.0E-04	**0.01**
30.0	0.962	1,839	0.293	2.0E-04	4.2E-05	5.0E-05	**0.01**
35.0	1.183	1,989	0.352	2.2E-04	5.1E-05	6.1E-05	**0.01**
40.0	1.424	2,126	0.410	2.4E-04	6.1E-05	7.3E-05	**0.01**
45.0	1.660	2,156	0.456	2.7E-04	8.5E-05	1.0E-04	**0.01**
46.5	1.898	2,243	0.493	2.8E-04	9.6E-05	1.2E-04	**0.00**





$N_{1(60)}$ = C_N*C_E*C_B*C_R*C_S*N

C_R = 0.75 for Rod lengths < 3m, 1.0 for > 10m
= min(1,max(0.75,1.4666-2.556/$(z(ft))^{0.5}$))

C_N = $(1\ atm/p'o)^{0.5}$, max 1.7

C_S = max(1.1,min(1.3,1+$N_{1(60)}$/100)) for SPT without liners

MSF = $10^{2.24}/M^{2.56}$

z = Depth (m)

pa = 1 atm = 101 KPa = 1.058 tsf

p = 0.67*po

Nc = $(MAG-4)^{2.17}$

τ_{av} = 0.65*PGA*po*rd

G_{max} = 447*$N_{1(60)CS}^{(1/3)}$*$p^{0.5}$

a = 0.0389*(p/1)+0.124

b = 6400*$(p/1)^{(-0.6)}$

γ = [1+a*EXP(b*τ_{av}/G_{max})]/[(1+a)*τ_{av}/G_{max}]

E_{15} = γ*$(N_{1(60)CS}/20)^{-1.2}$

E_{nc} = $(Nc/15)^{0.45}$*E15

S = 2*H*E_{nc}

rd = (1-0.4113*z^0.5+0.04052*z+0.001753*z^1.5)/(1-0.4177*z^0.5+0.05729*z-0.006205*z^1.5+0.00121*z^2))

$\Delta N_{1(60)}$ = min(10,IF(FC<35,exp(1.76-(190/FC^2)),5)+IF(FC<=5,1,IF(FC<35,0.99+(FC^1.5/1000),1.2))*N1(60) - N1(60)

$N_{1(60)CS}$ = $N_{1(60)CS}$ + $\Delta N_{1(60)}$

$K\sigma$ = min of 1.0 or $(p'o/1.058)^{(IF(Dr>0.7,0.6,IF(Dr<0.5,0.8,0.7))-1)}$

Dr = $(N_{1(60)}/70)^{0.5}$

CSReq = 0.65*PGA*(po/p'o)*rd

CSR* = CSReq/MSF/$K\sigma$

$CRR_{7.5}$ = (0.048-0.004721*N+0.0006136*N^2-0.00001673*N^3)/(1-0.1248*N+0.009578*N^2-0.0003285*N^3+0.000003714*N^4))

N = $N_{1(60)CS}$

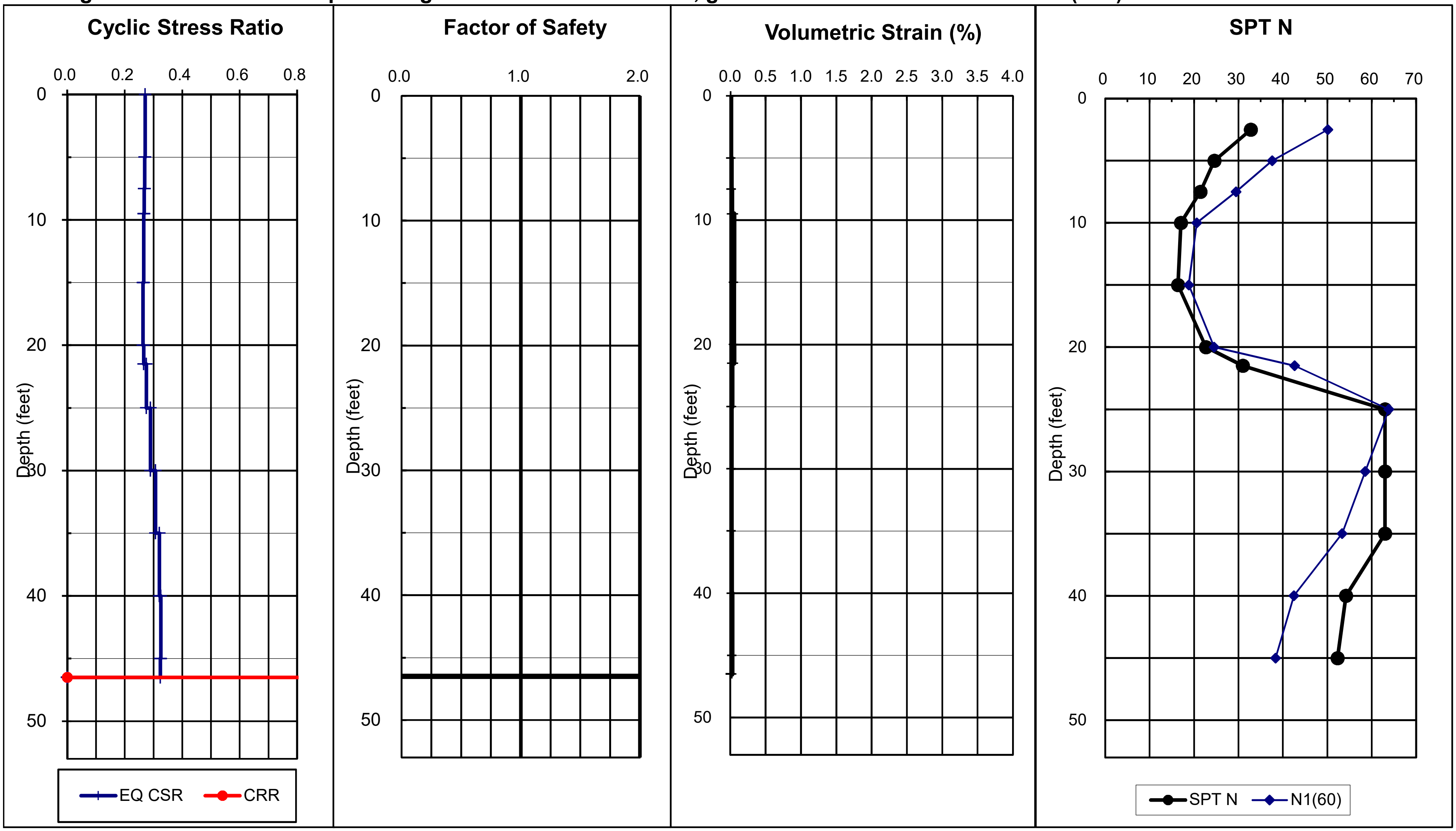
SF = $CRR_{7.5,1atm}$/CSR*

EARTH SYSTEMS - EVALUATION OF LIQUEFACTION POTENTIAL AND INDUCED SUBSIDENCE

Horseshoe Lake Park **Project No: 302538-001** 1996/1998 NCEER Method

Ground Compaction Remediated to 4 foot depth

Boring: B-1 **Earthquake Magnitude: 8.2** **PGA, g: 0.33** **Calc GWT (feet): 50**



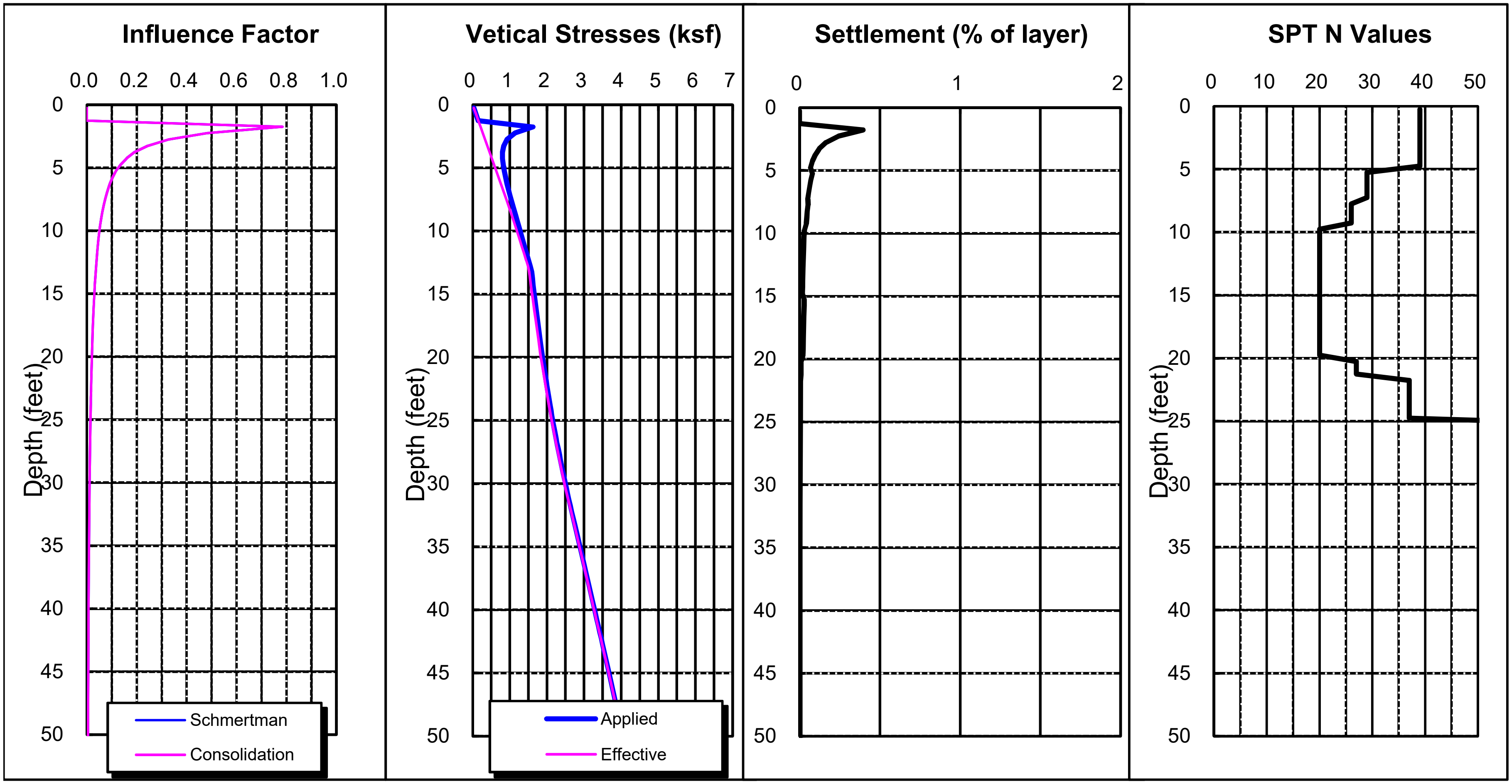
Total Thickness of Liquefiable Layers: 0.0 feet **Estimated Total Ground Subsidence: 0.1 inches**

EARTH SYSTEMS SOUTHWEST - SETTLEMENT ANALYSES

Horseshoe Lake Park

302538-001

Width, ft: 1.0 Length, ft: 40.0 Net pressure, ksf: 1.80 **Settlement, inches: 0.1**



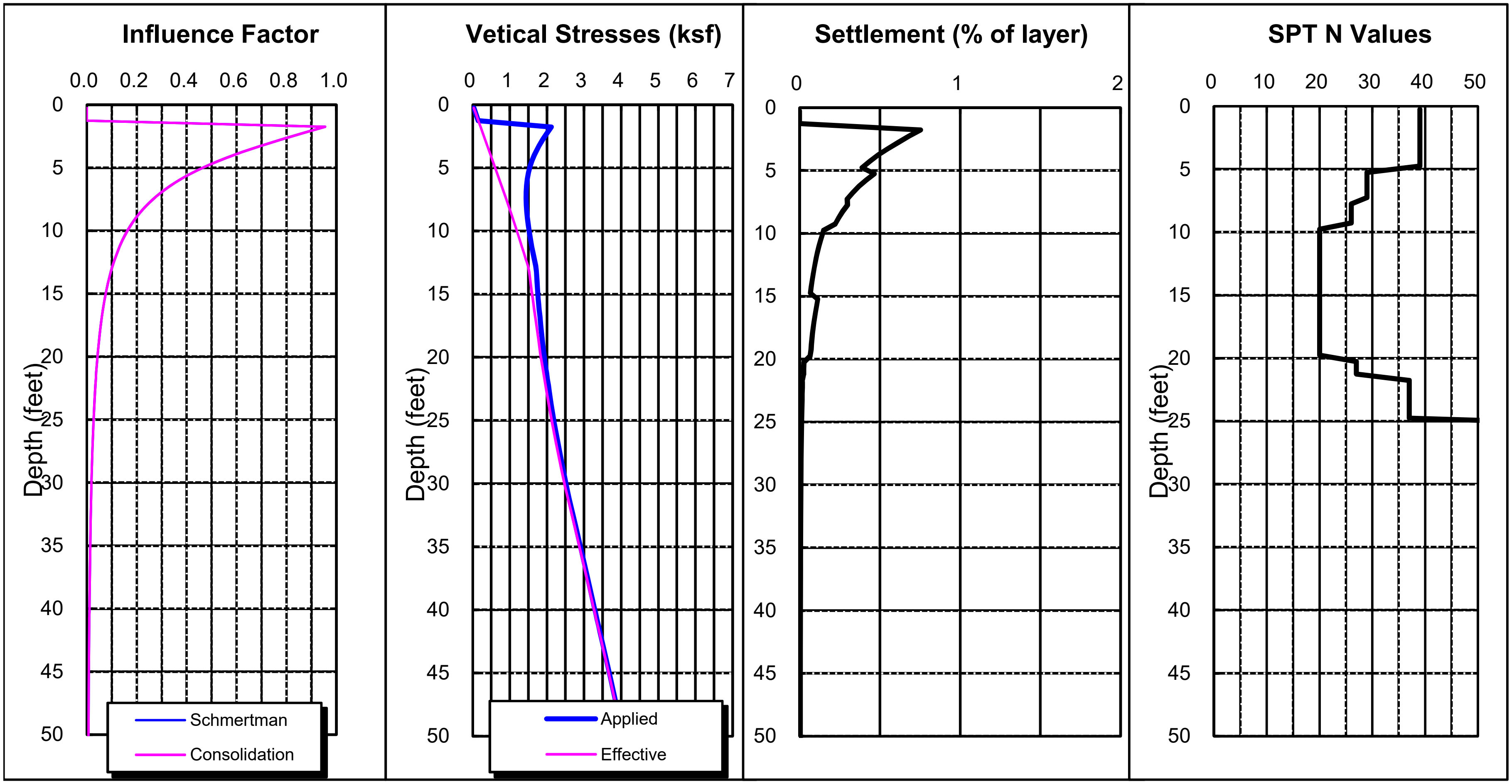
Load, Q: 2 kpf Embedment, feet: 1.5 **Boring: B-1**

EARTH SYSTEMS SOUTHWEST - SETTLEMENT ANALYSES

Horseshoe Lake Park

302538-001

Width, ft: 7.0 Length, ft: 7.0 Net pressure, ksf: 2.00 **Settlement, inches: 0.6**



Load, Q: 98 kips Embedment, feet: 1.5 **Boring: B-1**

APPENDIX B

Laboratory Test Results

UNIT DENSITIES AND MOISTURE CONTENT ASTM D2937 & D2216

Job Name: Horseshoe Lake Park Improvements

Sample Location	Depth (feet)	Unit Dry Density (pcf)	Moisture Content (%)	USCS Group Symbol
B1	2.5	113	7	SM
B1	5	114	6	SM
B1	7.5	108	7	SM
B1	10	102	5	SM
B1	15	105	5	SM
B1	20	113	6	SM
B1	25	120	10	SM
B1	30	130	11	SC
B1	35	124	14	ML
B1	40	121	18	ML
B1	45	112	21	ML
B2	2.5	109	7	SM
B2	5	116	6	SM
B3	2.5	118	10	SM
B3	5	--	11	SM
B4	2.5	120	6	SM
B4	5	102	9	SM
B4	7.5	119	11	SM
B4	10	109	13	ML
B4	15	--	16	SM
B5	2.5	117	10	SM
B5	5	107	8	SM
B5	7.5	128	10	SM
B5	10	108	9	SM

UNIT DENSITIES AND MOISTURE CONTENT ASTM D2937 & D2216

Job Name: Horseshoe Lake Park Improvements

Sample Location	Depth (feet)	Unit Dry Density (pcf)	Moisture Content (%)	USCS Group Symbol
B6	2.5	120	6	SM
B6	5	109	11	SM
B6	7.5	--	8	SM
B6	10	113	14	SM
B7	2.5	113	3	SM
B7	5	95	6	SM

File No.: 302538-001 10/31/2018

Lab No.: 18-142

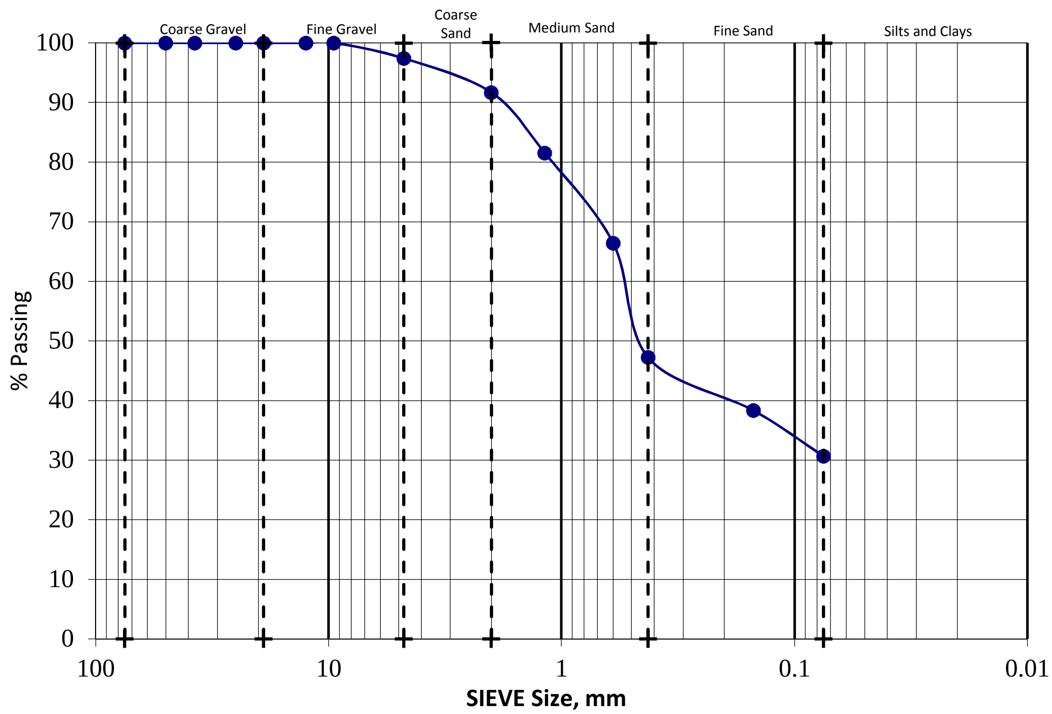
SIEVE ANALYSIS ASTM D6913

Job Name: Horseshoe Lake Park Improvements

Sample ID: B1 @ 2 1/2 feet

Description: Silty Sand (SM)

Sieve Size	% Passing	
3"	100	-
2"	100	-
1-1/2"	100	-
1"	100	-
3/4"	100	-
1/2"	100	-
3/8"	100	-
#4	97	-
#10	92	-
#16	82	-
#30	66	-
#40	47	-
#100	38	-
#200	30.7	-



% Coarse Gravel:	0	% Coarse Sand:	6				
% Fine Gravel:	3	% Medium Sand:	44	Cu:	NA		
		% Fine Sand:	17	Cc:	NA	Gradation	
% Total Gravel	3	% Total Sand	67	% Fines:	30.7	NA	

File No.: 302538-001 10/31/2018

Lab No.: 18-142

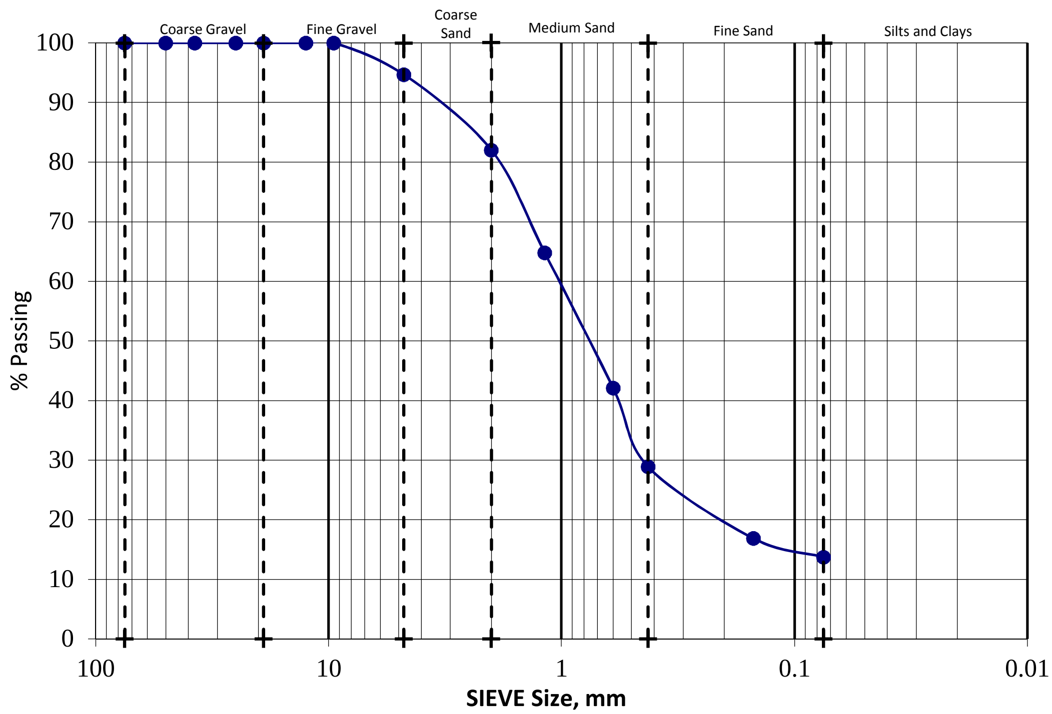
SIEVE ANALYSIS ASTM D6913

Job Name: Horseshoe Lake Park Improvements

Sample ID: B1 @ 10 feet

Description: Silty Sand (SM)

Sieve Size	% Passing	
3"	100	-
2"	100	-
1-1/2"	100	-
1"	100	-
3/4"	100	-
1/2"	100	-
3/8"	100	-
#4	95	-
#10	82	-
#16	65	-
#30	42	-
#40	29	-
#100	17	-
#200	13.7	-



% Coarse Gravel:	0	% Coarse Sand:	13				
% Fine Gravel:	5	% Medium Sand:	53	Cu:	NA		
		% Fine Sand:	15	Cc:	NA	Gradation	
% Total Gravel	5	% Total Sand	81	% Fines:	13.7	NA	

File No.: 302538-001 10/31/2018

Lab No.: 18-142

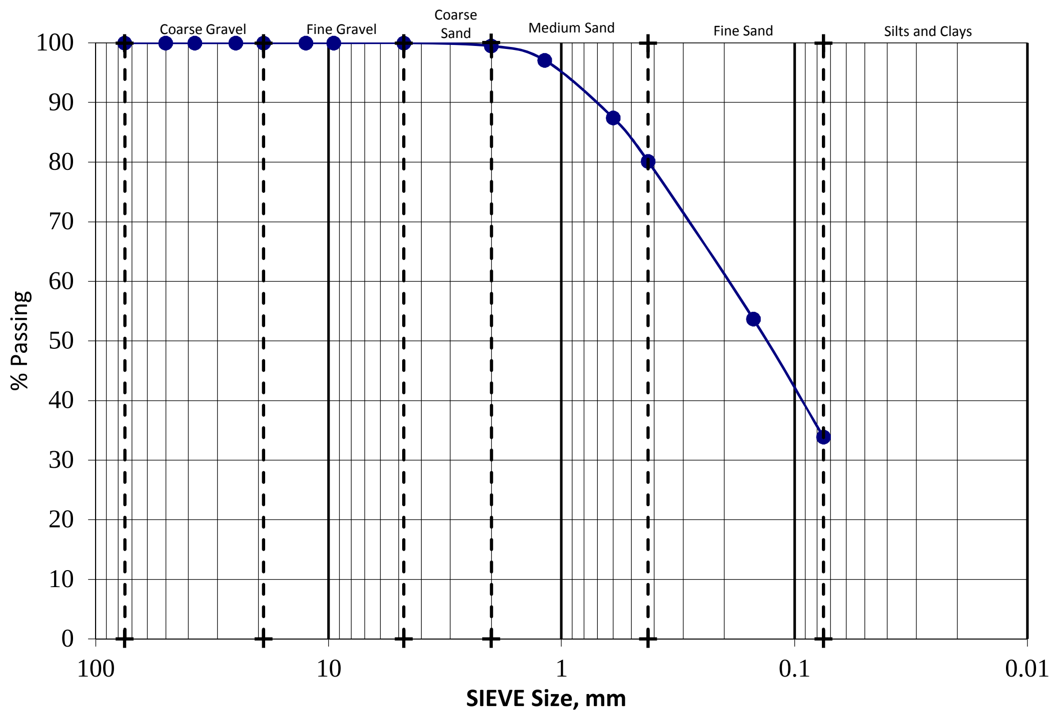
SIEVE ANALYSIS ASTM D6913

Job Name: Horseshoe Lake Park Improvements

Sample ID: P1 @ 4 feet

Description: Silty Sand (SM)

Sieve Size	% Passing	
3"	100	-
2"	100	-
1-1/2"	100	-
1"	100	-
3/4"	100	-
1/2"	100	-
3/8"	100	-
#4	100	-
#10	99	-
#16	97	-
#30	87	-
#40	80	-
#100	54	-
#200	33.9	-



% Coarse Gravel:	0	% Coarse Sand:	1			
% Fine Gravel:	0	% Medium Sand:	19	Cu:	NA	
		% Fine Sand:	46	Cc:	NA	Gradation
% Total Gravel	0	% Total Sand	66	% Fines:	33.9	NA

File No.: 302538-001 10/31/2018

Lab No.: 18-142

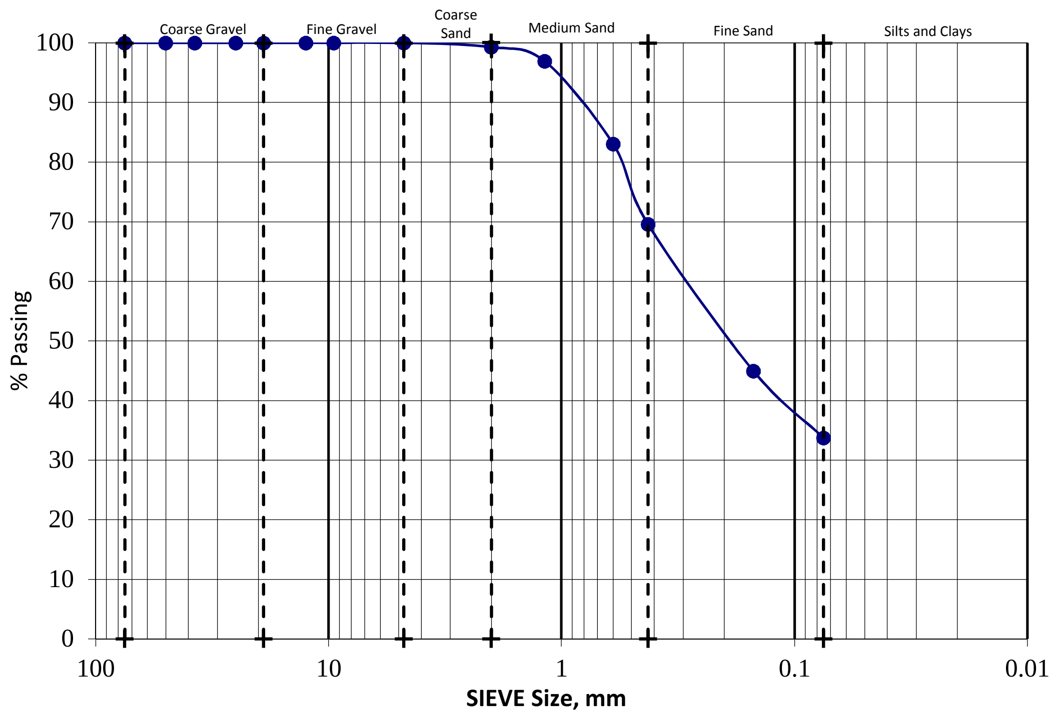
SIEVE ANALYSIS ASTM D6913

Job Name: Horseshoe Lake Park Improvements

Sample ID: P2 @ 2 1/2 feet

Description: Silty Sand (SM)

Sieve Size	% Passing	
3"	100	-
2"	100	-
1-1/2"	100	-
1"	100	-
3/4"	100	-
1/2"	100	-
3/8"	100	-
#4	100	-
#10	99	-
#16	97	-
#30	83	-
#40	70	-
#100	45	-
#200	33.7	-



% Coarse Gravel:	0	% Coarse Sand:	1				
% Fine Gravel:	0	% Medium Sand:	30	Cu:	NA		
		% Fine Sand:	36	Cc:	NA	Gradation	
% Total Gravel	0	% Total Sand	66	% Fines:	33.7	NA	

File No.: 302538-001 October 31, 2018
Job Name: Horseshoe Lake Park Improvements
Lab Number: 18-142

AMOUNT PASSING NO. 200 SIEVE

ASTM D 1140

Sample Location	Depth (feet)	Fines Content (%)	USCS Group Symbol
B2	2.5	8.2	SP-SM

File No.: 302538-001
Lab No.: 18-142

October 31, 2018

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

Horseshoe Lake Park Improvements
B1 @ 5 feet

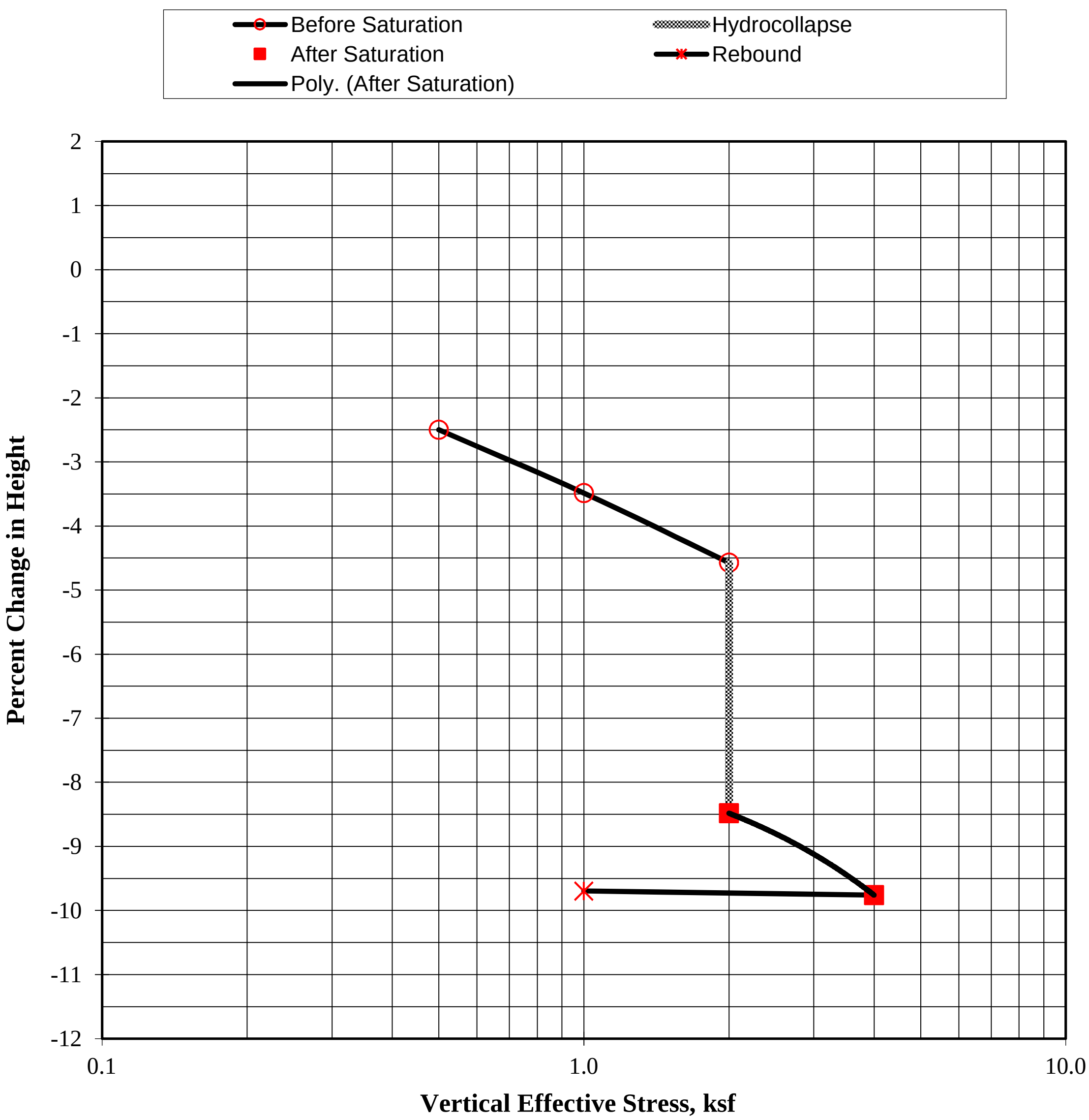
Silty Sand w/Trace Gravel (SM)

Ring Sample

Initial Dry Density: 106.6 pcf
Initial Moisture: 5.4%
Specific Gravity: 2.67
Initial Void Ratio: 0.564

Hydrocollapse: 3.9% @ 2.0 ksf

% Change in Height vs Normal Pressure Diagram



File No.: 302538-001
Lab No.: 18-142

October 31, 2018

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

Horseshoe Lake Park Improvements
B1 @ 10 feet

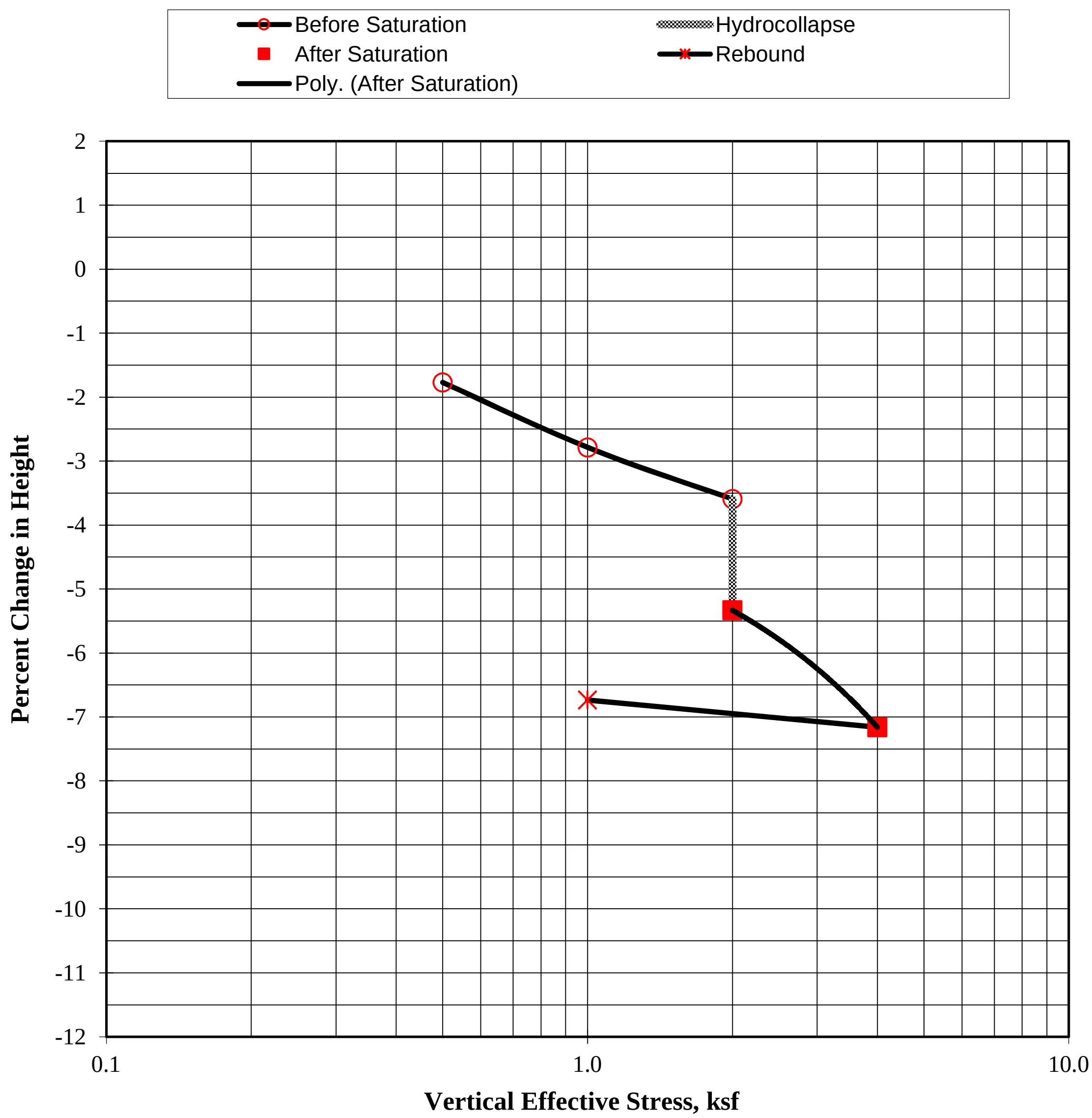
Silty Sand (SM)

Ring Sample

Initial Dry Density: 100.5 pcf
Initial Moisture: 9.1%
Specific Gravity: 2.67
Initial Void Ratio: 0.658

Hydrocollapse: 1.7% @ 2.0 ksf

% Change in Height vs Normal Pressure Diagram



File No.: 302538-001
Lab No.: 18-142

October 31, 2018

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

Horseshoe Lake Park Improvements
B1 @ 15 feet

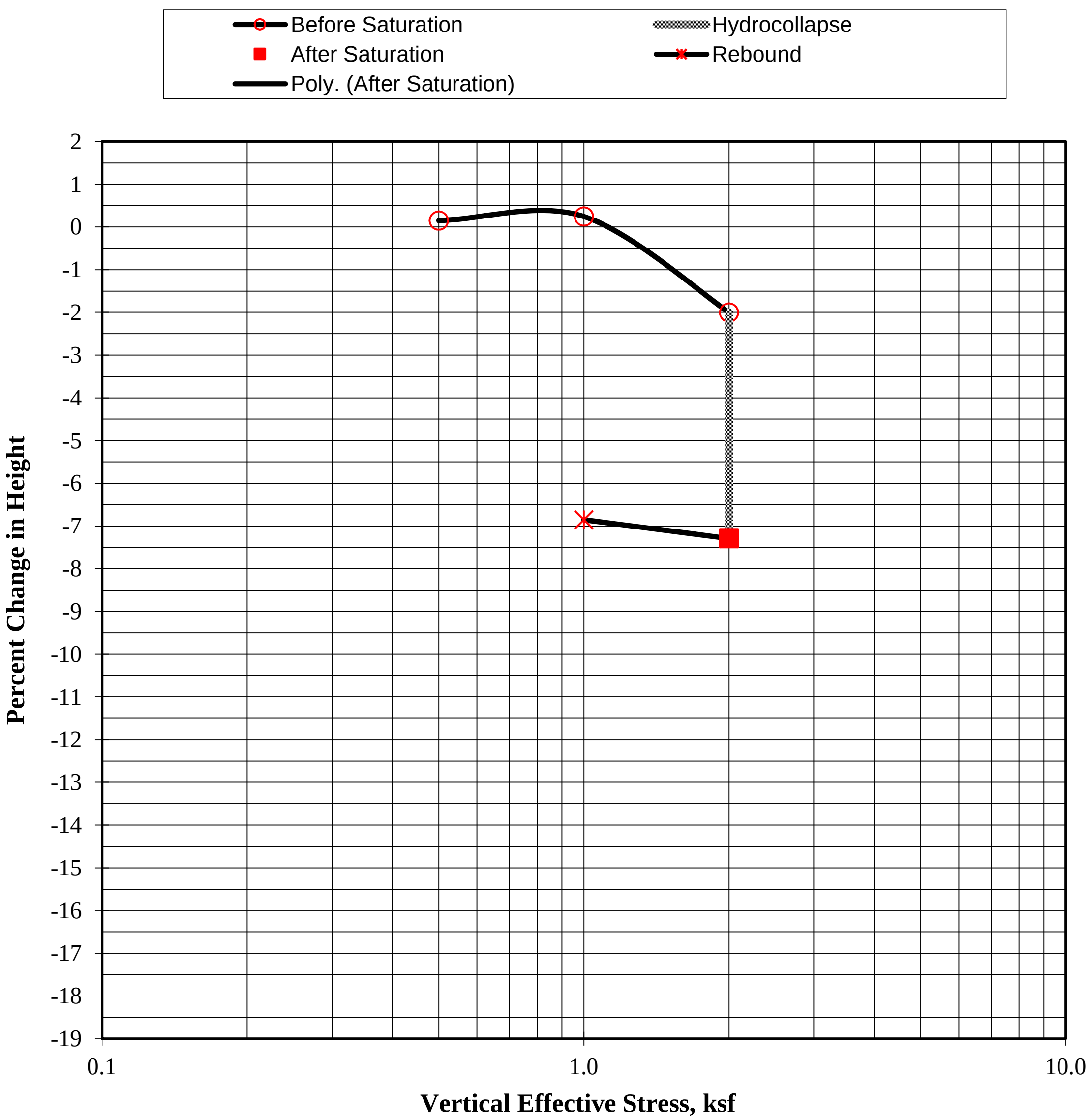
Silty Sand (SM)

Ring Sample

Initial Dry Density: 99.5 pcf
Initial Moisture: 5.5%
Specific Gravity: 2.67
Initial Void Ratio: 0.676

Hydrocollapse: 5.3% @ 2.0 ksf

% Change in Height vs Normal Pressure Diagram



File No.: 302538-001 October 31, 2018
Lab No.: 18-142

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

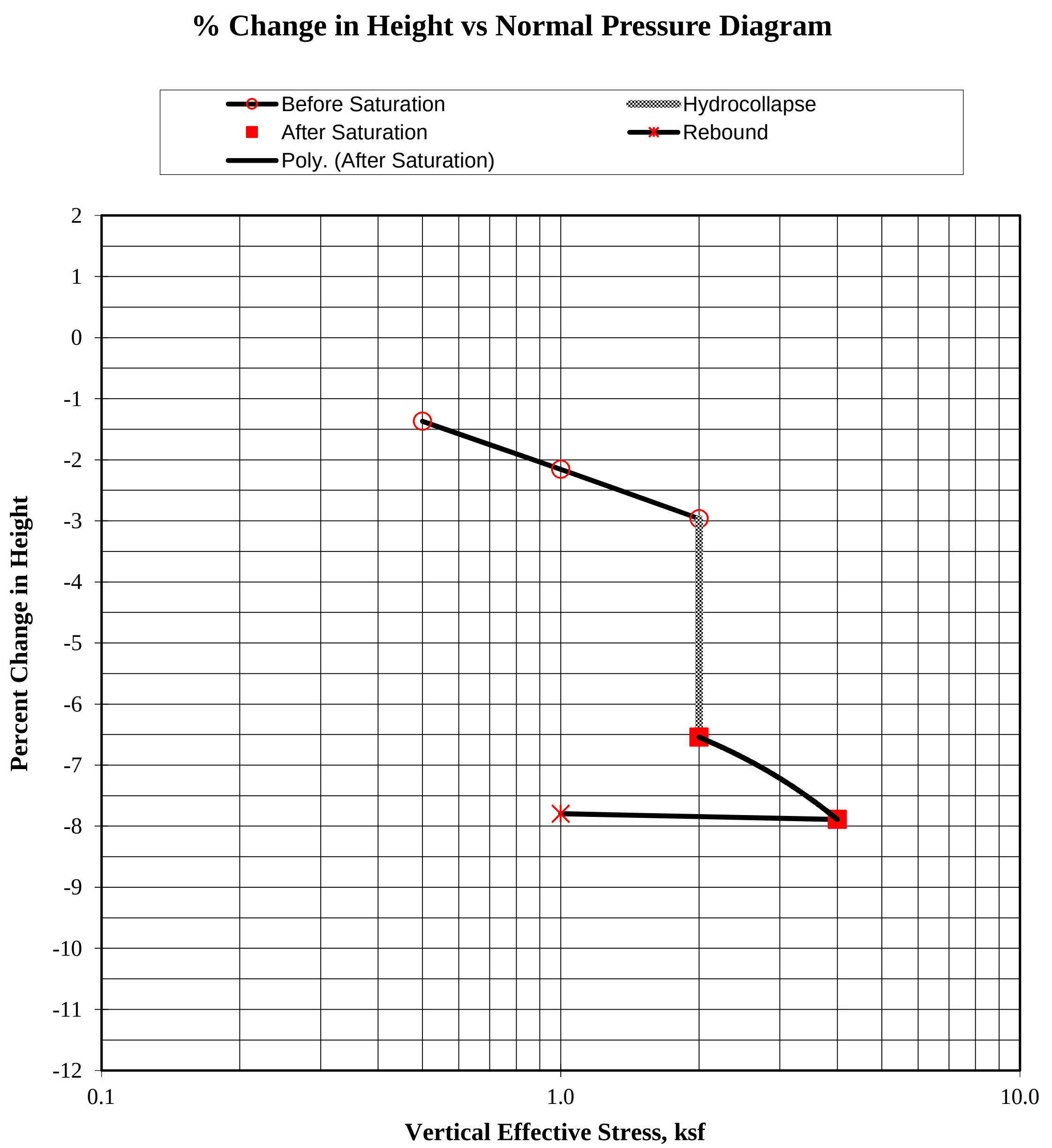
Horseshoe Lake Park Improvements
B1 @ 20 feet

Silty Sand (SM)

Ring Sample

Initial Dry Density: 107.8 pcf
Initial Moisture: 5.8%
Specific Gravity: 2.67
Initial Void Ratio: 0.452

Hydrocollapse: 3.6% @ 2.0 ksf



File No.: 302538-001
Lab No.: 18-142

October 31, 2018

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

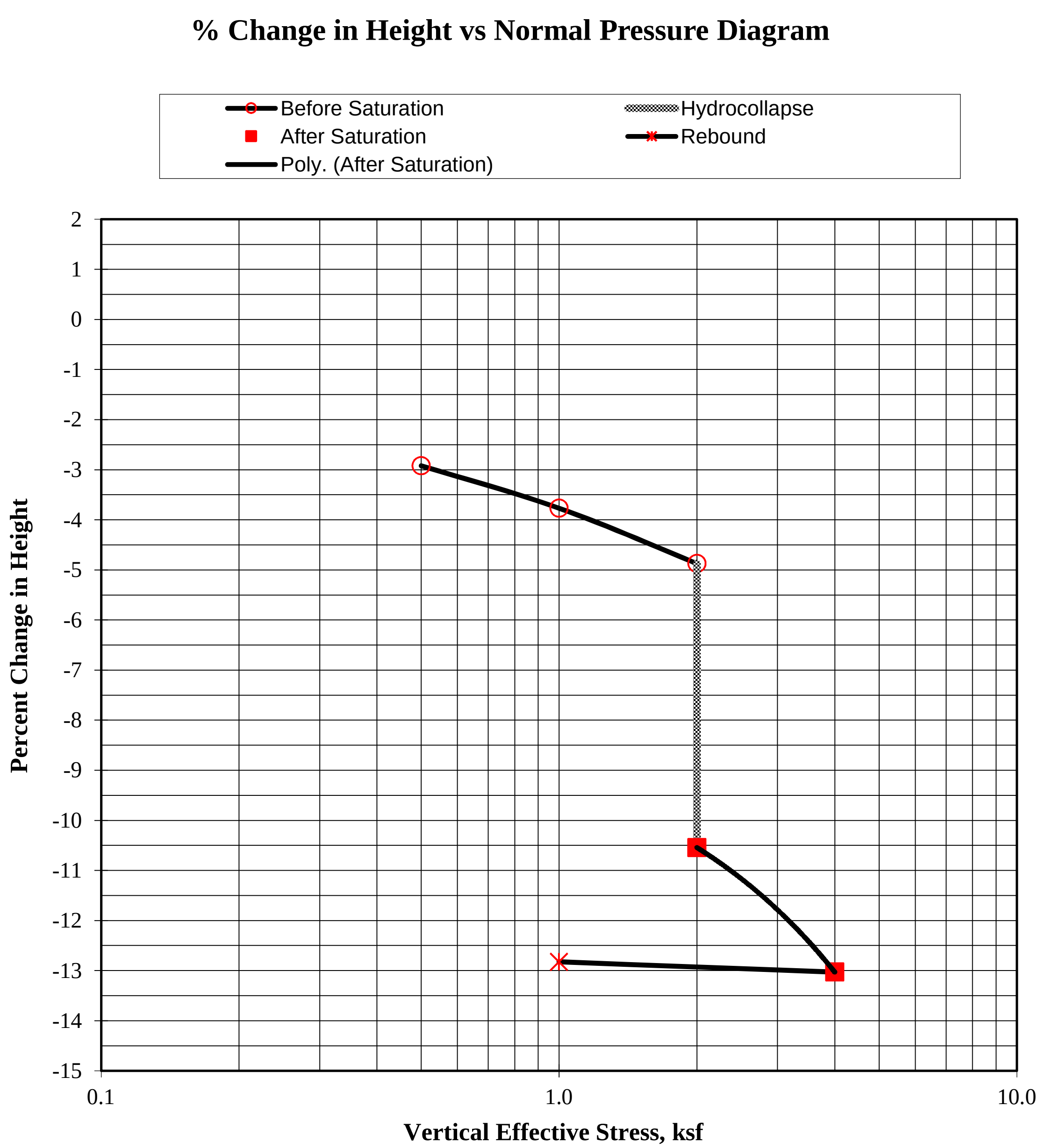
Horseshoe Lake Park Improvements
B7 @ 5 feet

Silty Sand (SM)

Ring Sample

Initial Dry Density: 93.9 pcf
Initial Moisture: 5.2%
Specific Gravity: 2.67
Initial Void Ratio: 0.775

Hydrocollapse: 5.7% @ 2.0 ksf



File No.: 302538-001 October 31, 2018
Lab No.: 18-142

EXPANSION INDEX **ASTM D-4829**

Job Name: Horseshoe Lake Park Improvements
Sample ID: B1 @ 0-5 feet
Soil Description: Silty Sand (SM)

Initial Moisture, %:	9.5	
Initial Compacted Dry Density, pcf:	112.1	
Initial Saturation, %:	51	
Final Moisture, %:	16.5	
Volumetric Swell, %:	0.8	
Expansion Index, EI:	**9**	**Very Low**

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

File No.: 302538-001 October 31, 2018

Lab No.: 18-142

MAXIMUM DRY DENSITY / OPTIMUM MOISTURE

ASTM D 1557 (Modified)

Job Name: Horseshoe Lake Park Improvements

Sample ID: 1

Location: B4 @ 0-5 feet

Description: Silty Sand (SM)

Procedure Used: C

Preparation Method: Moist

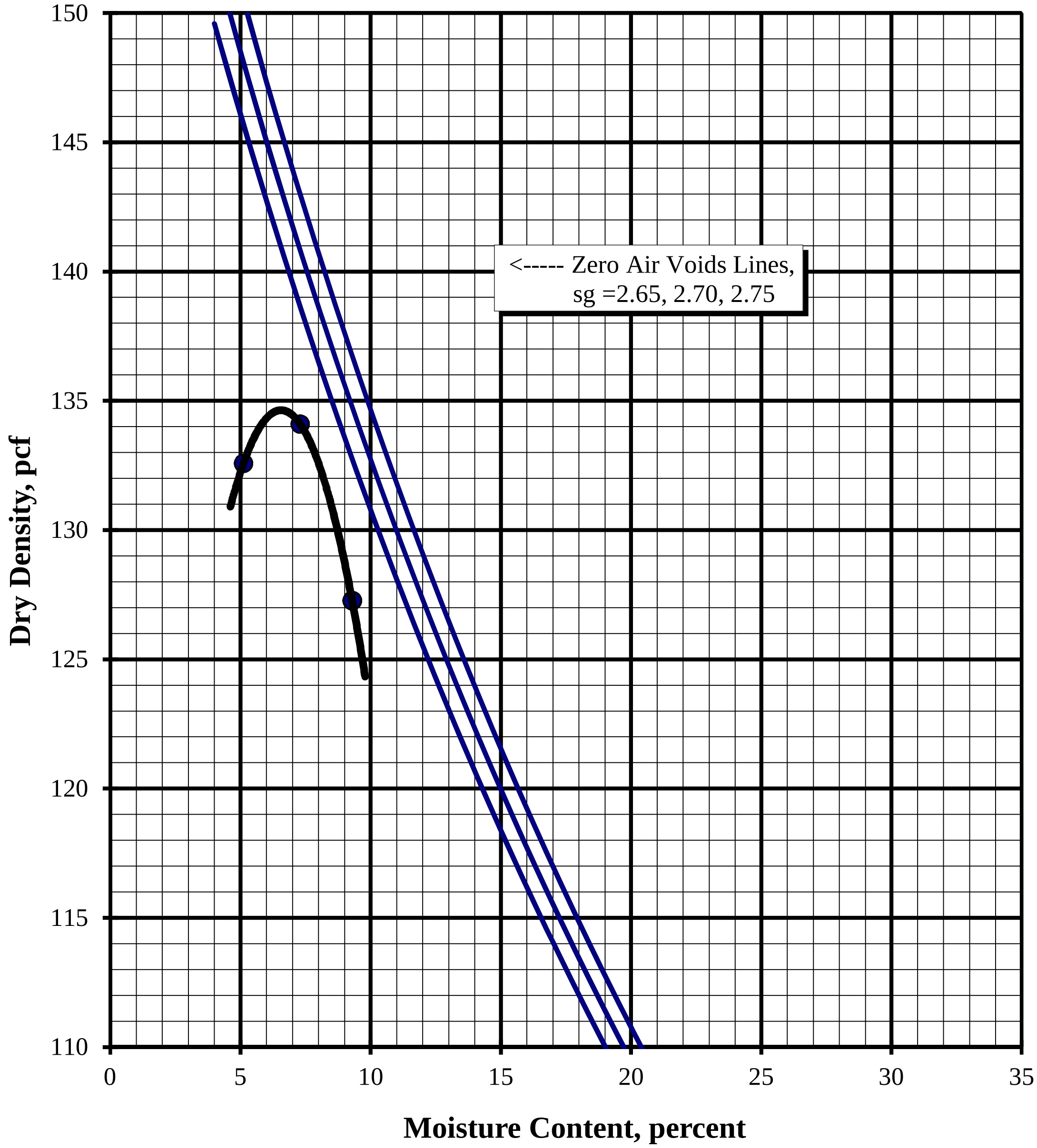
Rammer Type: Mechanical

Lab Number: 18-142

Maximum Dry Density: **134.4 pcf**

Optimum Moisture: **6.7%**

Sieve Size	% Retained (Cumulative)
3/4"	2.9
3/8"	11.2
#4	20.4



File No.: 302538-001 10/31/2018

Lab No.: 18-142

SOIL CHEMICAL ANALYSES

Job Name:	Horseshoe Lake Park Improvements
Job No.:	302538-001
Sample ID:	B1
Sample Location:	0-5 feet
Resistivity (Units)	
as-received (ohm-cm)	60,000
saturated (ohm-cm)	1,720
pH	7.4
Electrical Conductivity (mS/cm)	0.11
Chemical Analyses	
Cations	
calcium Ca^{2+} (mg/kg)	43
magnesium Mg^{2+} (mg/kg)	14
sodium Na^{1+} (mg/kg)	82
potassium K^{1+} (mg/kg)	5.4
Anions	
carbonate CO_3^{2-} (mg/kg)	12
bicarbonate HCO_3^{1-}(mg/kg)	159
fluoride F^{1-} (mg/kg)	4.9
chloride Cl^{1-} (mg/kg)	23
sulfate SO_4^{2-} (mg/kg)	51
phosphate PO_4^{3-} (mg/kg)	ND
Other Tests	
ammonium NH_4^{1+} (mg/kg)	ND
nitrate NO_3^{1-} (mg/kg)	10
sulfide S^{2-} (qual)	na
Redox (mV)	na

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

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

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

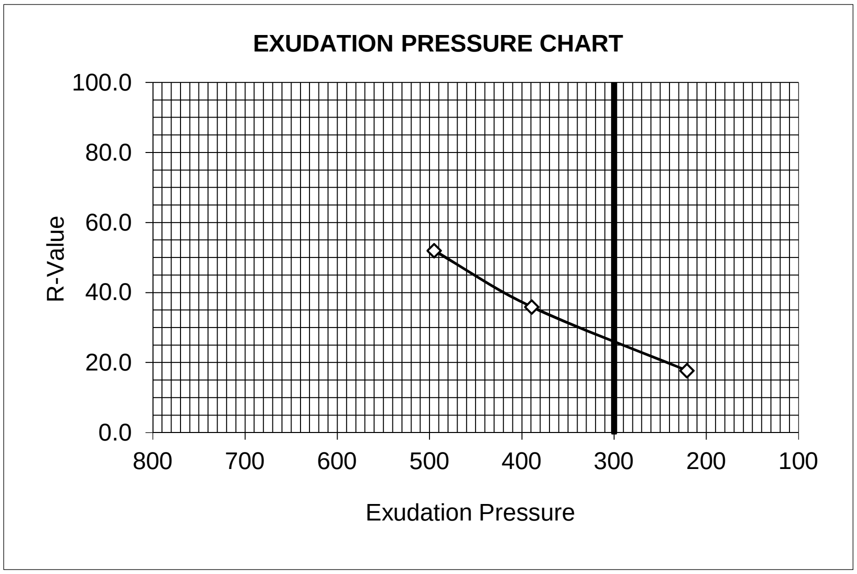
General Guidelines for Soil Corrosivity

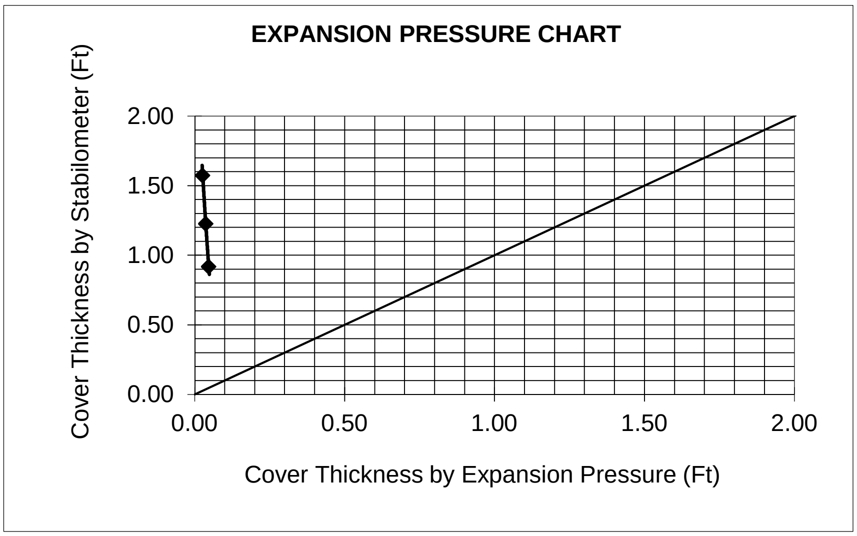
Chemical Agent	Amount in Soil	Degree of Corrosivity
Soluble	0 -1,000 mg/Kg (ppm) [0-.1%]	Low
Sulfates[1]	1,000 - 2,000 mg/Kg (ppm) [0.1-0.2%]	Moderate
	2,000 - 20,000 mg/Kg (ppm) [0.2-2.0%]	Severe
	> 20,000 mg/Kg (ppm) [>2.0%]	Very Severe
Resistivity[2]	0- 900 ohm-cm	Very Severely Corrosive
(Saturated)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

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

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

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





JOB NAME: Horseshoe Lake Park Improvements
SAMPLE I. D.: B1 @ 0-5 feet
SOIL DESCRIPTION: Silty Sand (SM) w/ Some Clay

SPECIMEN NUMBER	D	E	F
EXUDATION PRESSURE	495	389	221
RESISTANCE VALUE	52.0	35.9	17.7
EXPANSION DIAL(0.0001")	14	11	8
EXPANSION PRESSURE (PSF)	60.6	47.6	34.6
% MOISTURE AT TEST	11.0	11.8	12.5
DRY DENSITY AT TEST	125.8	123.9	125.9

R-VALUE @ 300 PSI EXUDATION	**25**
R-VALUE by Expansion Pressure*	**N/A**

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