APPENDIX D

GEOTECHNICAL REPORT

Palm Springs Unified School District
Facility Planning and Development
150 District Center Drive
Palm Springs, California 92264

Geotechnical Engineering and Geohazards Report
Proposed Desert Hot Springs High School CTE Building
65850 Pierson Boulevard
Desert Hot Springs, Riverside County, California

September 6, 2018

File No.: 302396-001
Doc. No.: 18-09-701

Earth Systems
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Palm Springs Unified School District
Facilities Planning and Development
150 District Center Drive
Palm Springs, California 92264

Attention: Mr. Kent Hems

Subject: **Geotechnical Engineering and Geohazards Report**

Projects: **Proposed Desert Hot Springs High School CTE Building**
65850 Pierson Boulevard
Desert Hot Springs, Riverside County, California

Earth Systems Pacific (Earth Systems) is pleased to present this Geotechnical Engineering and Geohazards Report for the proposed CTE Building at the existing Desert Hot Springs High School located at 65850 Pierson Boulevard in the city of Desert Hot Springs in Riverside County, California. This report presents our findings and recommendations for site grading and foundation design, incorporating the information provided to our office. The site is suitable for the proposed development, provided the recommendations in this report are followed in design and construction. This report should stand as a whole and no part of the report should be excerpted or used to the exclusion of any other part.

This report completes our scope of services in accordance with our proposal BER-18-6-005, dated June 26, 2018 and the Palm Springs Unified School District Contract Number C-0003407, dated July 27, 2018. Unless requested in writing, the client is responsible for distributing this report to the appropriate governing agency or other members of the design team.

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

Respectfully submitted,
EARTH SYSTEMS PACIFIC

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Proposed Desert Hot Springs High School CTE Building
65850 Pierson Boulevard
Desert Hot Springs, Riverside County, California

Section 1
INTRODUCTION

1.1 Project Description

This Geotechnical Engineering and Geohazards Report has been prepared for the proposed Desert Hot Springs High School CTE Building and associated site improvements located at the west side of the campus located at 65850 Pierson Boulevard in the city of Desert Hot Springs, Riverside County, California, see Plate 1 (Site Vicinity Map). We understand that the proposed CTE Building has, in general, three subareas consisting of the Renewable Energy Academy of Learning (REAL), Public Safety Academy (PSA), and outdoor spaces. Specifically, the additional site improvements will consist of an approximately 5,010 square foot REAL area, an approximately 5,010 square foot PSA area, and approximately 2,000 square foot of outdoor spaces, see Plate 2. In addition, appurtenant site work is assumed to include underground utilities, landscaping, reconfiguration and expansion of flatwork, access drives, and temporary parking areas. This report will address proposed site improvements as shown on Plate 2, which is a copy of the Site Plan prepared by PBK (architect).

We have assumed one-to two-story masonry, wood-framed or steel construction founded on shallow permanent foundations, and that there will be no below grade basement levels. Based on information provided by the structural engineer, we have assumed the anticipated loads will be less than 36 kips for isolated spread footings and 2.5 kip/lineal foot for continuous footings. As the basis for the foundation recommendations, all loading is assumed to be dead plus actual live load. If actual structural loading exceeds these assumed values, we will need to re-evaluate the given recommendations.

1.2 Site Description and Aerial Photo Review

The legal description of the land is identified as Accessor Parcel number (APN) 664-190-040-6 encompassing approximately 50.7 acres. The proposed CTE Building is located on the west side of the campus on a vacant triangular lot where Golden Eagle Way fronts the project area in the city of Desert Hot Springs, Riverside County, California. The site is covered with irrigated grass turf. The coordinates used are latitude 33.96457°N and longitude 116.51847°W near the northeastern portion of the proposed CTE building, which is the closest building area to the active faulting to the northeast. Plate 1 in Appendix A presents the approximate site location. The site is bounded on the west by Golden Eagle Way and on the north, east, and south by existing school buildings. Topographically, the site has been graded generally flat with elevation ranging from approximately 1,100 feet Mean-Sea-Level (MSL) at the southwest corner to 1,108 feet MSL near the east corner.

It appears the campus was constructed between 1996 and 2002. Construction of the campus appears to have been completed between February 2003 and September 2004. Since September 2004 the campus has remained relatively unchanged.

1.3 Purpose and Scope of Services

The purpose for our services was to evaluate the site soil and geologic conditions at our exploration locations and to provide professional opinions and recommendations, from a geologic and geotechnical point of view, regarding the proposed development of the site. We understand that these proposed additional school site improvements will be developed under the regulation of the Division of the State Architect (DSA), the current California Building Code (2016), and California Geological Survey (CGS) Note 48 requirements (2013) for geotechnical reports submitted to DSA.

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

Task 1 - Literature and Photograph Reviews

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

Earth Systems has performed several geologic and geotechnical engineering studies within the near school site area and these reports were reviewed and are listed in the attached reference list.

Task 2 – Utility Clearance, USA Dig Alert

Each of our proposed field exploration locations was located and marked in the field, and cleared with known utility lines as identified by Underground Service Alert (USA), "Dig Alert". Our exploration locations were located in the field by 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.

Task 3 – Field Exploration

We evaluated the general subsurface conditions at the site by drilling 6 small diameter borings, from 5 feet to 12 feet in depth. Refusal was encountered in each of the borings, therefore test pits were used to evaluate the general subsurface conditions. Six test pits were excavated from 5 ½ feet to 10 feet in depth. The borings and test pits were located in general accordance with CGS Note 48 requirements for school facilities which requires a minimum of one exploration point within the near-vicinity of the building footprint for every 5,000 square feet of first floor area, with a minimum of two borings per structure. Should the proposed structures be moved from the presently proposed and previously discussed locations, the current boring locations may not satisfy the required criteria and additional borings may be required. The field exploration also included a site reconnaissance of the project area and immediate surroundings. In addition, three borings were previously excavated within the surrounding areas for other projects and the data utilized for this commission. Plate 3 shows the surface geology and the approximate location of each boring and pit.

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, R-value, and direct shear characteristics. 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 – 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 and test pit 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 site specific geologic map and geologic cross sections;
- A discussion of regional geology and site seismicity, including a regional geology map;
- 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 "General Procedure" and Site Specific Procedure 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, deep 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 slabs-on-grade, including recommendations for reducing the potential for moisture transmission through interior slabs;
- Recommendations for collapsible or expansive soils (if applicable);
- Recommendations for asphalt concrete and Portland cement concrete parking and drives;
- A discussion of the corrosion potential of the near-surface soils encountered during our field exploration;
- An appendix, which includes a summary of the field exploration (computer generated boring logs) and laboratory testing program (computer generated plots).

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

- An environmental Phase 1 assessment.
- 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

Exploratory Borings

The subsurface exploration program included advancing six exploratory borings. The borings were drilled to depths ranging from approximately 5 to 12 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. Refusal was encountered in all of the borings due to hard drilling on dense soils with cobbles and/or boulders. The borings were advanced to observe soil profiles and obtain samples for laboratory testing. The approximate boring locations are shown on Plate 3, 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.

A geologist from Earth Systems maintained a log of the subsurface conditions encountered and obtained samples for visual observation, classification and laboratory testing. Soils were logged in general accordance with the Unified Soil Classification System. Our typical sampling interval within the borings was approximately every 2½ to 5 feet to the full depth explored; however, sampling intervals were adjusted depending on the materials encountered on-site. Samples were obtained within the test borings using a Modified California [MC] ring sampler (ASTM D 3550). The MC sampler has an approximate 3-inch outside diameter and a 2.4-inch inside diameter.

The sampler was attached to screwed connection drill rod and driven using a 140-pound automatic hammer falling for a height of 30 inches. Design parameters provided by Earth Systems in this report have considered an estimated 70% hammer efficiency based on data provided by the drilling sub contractor. Since the MC sampler was used in our field exploration to collect ring samples, the N-values (blow count) using the California sampler can be correlated to SPT N-values using a conversion factor that may vary from about 0.5 to 0.7. In general, a conversion factor of approximately 0.63 from a study at the Port of Los Angeles (Zueger and McNeilan, 1998 per SP 117A) is considered satisfactory. A value of 0.63 was applied in our calculations for this project.

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

The final logs of the borings represent our interpretation of the contents of the field logs and the results of laboratory testing performed. The final exploration logs are included in Appendix A of this report. The stratification lines represent the approximate boundaries between soil types, although the transitions may be gradational.

Test Pits

Six exploratory test pits (TP-1 through TP-6) were excavated using a rubber tire backhoe. Test pit depths ranged from approximately 5½ feet to 10 feet below existing grades. The test pits were excavated for classification, more specifically to evaluate plus 6-inch oversize material and soil matrix density. Soil dry density was measured using nuclear techniques via ASTM D6938 with moisture correction measured in the lab via ASTM D2216. The test pit locations are shown on Plate 3 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.

In reviewing the boring logs and test pit logs and legend, the reader should recognize that the information is intended as a guideline only, and there are a number of conditions that may influence the soil characteristics as observed during excavation. These include, but are not limited to, the presence of cobbles or boulders, cementation, variations in soil moisture, presence of groundwater, and other factors. The logs present field blow counts per 6 inches of driven embedment (or portion thereof) for a total driven depth attempted of 18 inches. The blow counts are uncorrected (i.e. not corrected for overburden, sampler type, etc.). Consequently, the user must correct the blow counts per standard methodology if they are to be used for design and exercise judgment in interpreting soil characteristics, possibly resulting in soil descriptions that vary somewhat from the legend.

2.2 Laboratory Testing

Samples were reviewed along with field logs to select those that would be analyzed further. Those selected for laboratory testing include soils that would be exposed and used during grading and those deemed to be within the influence of the proposed structure. 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 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).

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).
- Direct Shear to evaluate the relative frictional strength of the surficial slope soils. Specimens were in a saturated condition prior to and during testing and were sheared under normal loads ranging from 1.0 to 4.0 kips per square foot (ASTM D 3080).

- Expansion Index tests to evaluate the expansive nature of the soil. The samples were surcharged under 144 pounds per square foot at moisture contents of near 50% saturation. Samples were then submerged in water for 24 hours and the amount of expansion was recorded with a dial indicator (ASTM D 4829).
- Chemical Analyses (Soluble Sulfates and Chlorides (ASTM D 4327), pH (ASTM D 1293), and Electrical Resistivity/Conductivity (ASTM D 1125) to evaluate the potential for adverse effects of the soil on concrete and steel.
- R-Value for pavement section analysis (CTM 301).

Section 3
DISCUSSION

3.1 Soil Conditions

The field exploration and literature review indicates that site geologic materials consist generally of artificial fill (af) overlying Quaternary alluvium. Per Dibblee (#DF-121, 2004) the mapped native materials are identified as Quaternary (Holocene) "alluvial sand and gravel of valley areas".

The artificial fill is loose to very dense, fine to coarse grained sand and sand with silt (SP and SP-SM per the Unified Soil Classification System, [USCS]) that appears to be locally derived. No compaction report for these fills have been provided and as such are considered to be undocumented. The fill is estimated to be 2½ feet to 6 feet thick. The observed native alluvial soils consist of find to coarse grained sands and gravelly sands with varying amounts of cobbles and boulders (SP-SW per the Unified Soil Classification System, [USCS]). The cobble and boulder content appear to increase with depth, based upon the frequent refusal of the drilling at various depths below the ground surface. Estimated cobble and boulder content is discussed in Section 5.1. The boring test pit logs provided in Appendix A include more detailed descriptions of the soils encountered.

3.2 Groundwater

No groundwater or perched water was encountered during our field exploration (maximum depth 12 feet) despite ongoing site irrigation to maintain grass turf. Groundwater depths are influenced by the fault systems surrounding the site; however, the site is not within a mounded groundwater area. Desert Hot Springs High School is located within the Mission Creek subbasin groundwater storage unit which is bounded on the north by the Mission Creek fault, on the south by the Banning fault, on the west by the San Bernardino Mountains, and on the east by the Indio Hills and the Mission Creek fault. Per the Mission Creek and Garnet Hill Water Management Plan, historic groundwater in the site vicinity was reported to be approximately 800 feet Mean Sea Level (MSL) in 1936 (Psomas, 2013). Significant dewatering in portions of the Mission Creek subbasin has resulted in groundwater level declines of approximately 100 feet between 1936 and 2003 (Psomas, 2013).

Based on calculation of percent saturation of soil samples collected, considering moisture content and density, saturated conditions were not observed or indicated in our testing. As soils are generally granular, the potential for saturated perched water conditions to develop is considered low.

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

- Well No. 02S05E30Q001S located approximately 1 mile to the east of the site is at an elevation of approximately 1,105.5 feet. The groundwater varied from 1,047.4 to 1,052.7 feet MSL from 2011 to 2018.

- Well No. 02S05E31H001S located approximately 1.6 miles southeast of the site is located just south of the intersection of Verbena Drive and Granada Avenue. The surface elevation of this well is approximately 1,033 feet and the groundwater elevation as measured from 2011 to 2018 varied from 1,020.8 feet to 1,024.45 feet MSL, a mounded condition due to faulting.
- Well No. 02S04E23N001S located approximately 1.7 miles northwest of the site is located just north of the intersection of Mission Lakes Boulevard and North Indian Canyon Drive. The surface elevation of this well is approximately 1,283.5 feet and the groundwater elevation as measured from 2011 to 2018 varied from 729.9 to 761.4 feet MSL.

Based on the above data, groundwater is not anticipated to be encountered during construction and is expected to be deeper than 50 feet at the site such that liquefaction is not a concern. Fluctuations of the groundwater level, localized zones of increased soil moisture content may be anticipated during and following the rainy season or from irrigation.

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 increase in external loads. Soils susceptible to collapse include loose or 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 derived residual soils. Soils such as these exist on-site.

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 (Department of Navy, 1986) Design Manual 7.1, the severity of collapse potential is commonly evaluated by the following Table, Collapse Potential Values.

Table 1
Collapse Potential Values

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

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

Table 2
Collapsible Soil Hazard Ranking System

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

The results of collapse potential tests performed on 4 selected samples of soil matrix from depths ranging from 2 to 10 feet below the ground surface indicated a collapse potential on the order of 0.8 to 1.3 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). Plotting and analysis of the results of the 4 tests indicates that collapse potential is generally less than 2% when the dry density is greater than 88 pcf (relative to ASTM D 1557), and generally less than 1% when the dry density is greater than 103 pcf (relative to ASTM D 1557).

For some deposits without cementation, studies suggest some sites with densities above 103 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. Earth Systems data for this project supports this in that $N_{60} < 8$ in conjunction with densities less than 103 pcf could have increased potential for collapse.

Based on the above criteria and our field and laboratory findings, we estimate there is a “Moderate” collapse potential from soil layers between 2 and 10 ft bgs. Without collapse mitigation efforts, the collapse potential is approximately 1.3 inches. Assuming the recommended grading is accomplished according to Section 5.1 of this report, we estimate the collapse potential is on the order of approximately 0.3 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 Index 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 Corrosivity

One sample of the near-surface soils within the site were tested for potential corrosion of concrete and ferrous metals. Soils in the upper 0 to 3 feet were tested, both as blended (composite) samples. The tests were conducted in general accordance with the ASTM Standard Test Methods to evaluate pH, resistivity, and water-soluble sulfate and chloride content. The test results are presented in Appendix B. These tests should be considered as only an indicator of corrosivity for the samples tested. Other earth materials found on site may be more, less, or of a similar corrosive nature.

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

Table 3

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

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

Table 4

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

Test results show a pH value of 8.6, chloride content of "not detected", sulfate content of 54 ppm and minimum resistivity of 8,000 Ohm-cm. Although Earth Systems does not practice corrosion engineering, the corrosion values from the soils tested are normally considered as being mildly corrosive to buried metals and as possessing a "negligible" exposure to sulfate attack for concrete as defined in American Concrete Institute (ACI, 2011) 318, Section 4.3. The results of 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 Geologic Setting

Regional Geology: The site lies within the Coachella Valley, a part of the Colorado Desert geomorphic province. A significant feature within the Colorado Desert geomorphic province is the Salton Trough, a large northwest-trending structural depression that extends approximately 180 miles from the San Gorgonio Pass to the Gulf of California (see Plate 4). Much of this depression in the area of the Salton Sea is below sea level.

The Coachella Valley forms the northerly part of the Salton Trough and contains a thick sequence of Miocene to Holocene sedimentary deposits. Mountains surrounding the upper Coachella Valley include the Little San Bernardino Mountains on the northeast, foothills of the San Bernardino Mountains on the west and northwest, and the San Jacinto and Santa Rosa Mountains on the southwest. These mountains expose primarily Precambrian metamorphic and Mesozoic granitic rocks. The San Andreas fault zone within the Coachella Valley, traversing along the northeast margin of the valley, consists of the Garnet Hill, Banning, and the Mission Creek faults.

Locally, the site is situated at the northwestern or upper portion of the Coachella Valley (see Plate 5). The Mission Creek alluvial fan complex, a broad area of coalesced alluvial fans emanating from the conjunction of the San Bernardino and Little San Bernardino Mountains, extends southeastward towards Desert Hot Springs and North Palm Springs. Quaternary sediments compose the fan complex with igneous and metamorphic rock the predominant rock types in the surrounding mountains.

Local Geology: Desert Hot Springs High School is located in the northwestern portion of the Coachella Valley approximately five miles east of the eastern terminus of the San Bernardino Mountains and approximately 1.5 miles southwest of the Little San Bernardino Mountains. The project site is located upon the broad Mission Creek alluvial fan complex. Predominantly

Holocene alluvial sediments compose the fan complex, although older (Pleistocene) fans at Devers Hill and Whitewater Hill are elevated above current drainage base lines.

Per Proctor (1968) the site is underlain by Holocene alluvial deposits. Approximately 2½ to 6 feet of engineered artificial fill, resulting from mass grading of the overall school campus underlie the project site.

Local faults associated with the San Andreas fault system in the upper Coachella Valley include the Mission Creek fault (North branch of the San Andreas fault) located along the southwestern front of the Little San Bernardino Mountains about 0.4 miles northeast of the project, and the Banning fault (South branch) near Whitewater Hill about 3.4 miles southwest of the campus. A northeast trending fault near Devers Hill is located approximately 2.6 miles southwest of the campus.

Regional Faulting: No active faults have been mapped within the project limits based upon local and regional select published geologic maps by the California Geological Survey (2010) or United States Geological Survey fault database (2006). The site is not located within a currently designated Alquist-Priolo Earthquake fault zone (Plate 6) or Riverside County fault zone. The nearest mapped fault is the San Andreas fault zone (Type A) located approximately 0.4 mile northeast of the site (see Plates 6 and 7).

Other nearby active regional faults within approximately 30 miles of the site include the Pinto Mountain, Burnt Mountain, Landers, and San Jacinto faults. Table A-1, in Appendix A lists local and regional faults located within approximately 52 miles of the site.

In addition, there are abundant active or potentially active faults located in southern California that are capable of generating earthquakes that could affect the Desert Hot Springs area. These include the Mojave and San Bernardino segments of the San Andreas fault, the many faults within the Mojave Desert located north of the Little San Bernardino Mountains and numerous faults located in the vicinity of the Los Angeles basin and coastal southern California, see Plate 8.

3.7 Geologic Hazards

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

3.7.1 Seismic Hazards

Seismic Sources: Several active faults or seismic zones lie within 52 miles of the project site as shown on Table A-1 in Appendix A. The primary seismic hazard to the site is strong ground shaking from earthquakes along regional faults including the San Andreas faults.

Surface Fault Rupture: The project site does not lie within a currently delineated State of California, *Alquist-Priolo* Earthquake Fault Zone (CGS 2018). Well-delineated fault lines cross through this region as shown on California Geological Survey [CGS] maps (Jennings, 2010) or

United States Geologic Survey fault database (2006); however, no active faults are mapped in the immediate vicinity of the site.

Mapping by Smith (1979) for the CDMG Fault Evaluation Report 86 and by Kahle, et al (1987) for Fault Evaluation Report 185 do not indicate any active faulting in the immediate vicinity of the site, especially relating to the 1986 North Palm Springs earthquake, where minor fault rupture and cracking was observed along the nearby Mission Creek and Banning fault zones.

The closest active fault is the Mission Creek (north branch) segment of the San Andreas fault zone, located approximately 0.4 miles northeast of the project. Therefore, active fault rupture is unlikely to occur at the site. While fault rupture would most likely occur along previously established fault traces, future fault rupture could occur at other locations.

Historic Seismicity: The site is located within an active seismic area in southern California where large numbers of earthquakes are recorded each year. Our research of regional faulting indicates that at least 40 active faults or seismic zones lie within 50 miles of the project site as shown on Table A-1 in Appendix A. The primary seismic hazard to the site is strong ground shaking from earthquakes along the San Andreas and San Jacinto faults. Plate 9, Earthquake Epicenter Map, depicts epicenters of significant seismic events greater than magnitude 5.5 that have occurred in southern California between 1769 and 1999. Magnitudes that are above 6 and prior to accurate instrumental measurements (after 1933) are based on moment magnitudes (M_W). Magnitudes that are below 6 or earthquakes prior to 1933 are based on local magnitudes (M_L).

Many of the major historic earthquakes felt in the vicinity of Desert Hot Springs have originated from faults located outside the area. These include the 1857 Fort Tejon, 1933 Long Beach, 1952 Arvin-Tehachapi, 1971 San Fernando, 1987 Whittier Narrows, 1992 Landers, 1994 Northridge, and 1999 Hector Mine earthquakes.

Six historic seismic events (5.9 M or greater) have significantly affected the Coachella Valley this century. They are as follows:

- *Desert Hot Springs Earthquake* - On December 4, 1948, a magnitude 6.5 M_L (6.0M_W) earthquake occurred east of Desert Hot Springs. This event was strongly felt in the Desert Hot Springs area.
- *North Palm Springs Earthquake* - A magnitude 5.9 M_L (6.2M_W) earthquake occurred on July 8, 1986 in the Painted Hills causing minor surface creep of the Banning and Mission Creek segments of the San Andreas fault. This event was strongly felt in the Desert Hot Springs area and caused structural damage, as well as injuries. The epicenter of this earthquake was approximately six miles west of the project site.
- *Joshua Tree Earthquake* - On April 22, 1992, a magnitude 6.1 M_L (6.1M_W) earthquake occurred in the mountains 9 miles east of Desert Hot Springs. Structural damage and minor injuries occurred in the Indio area as a result of this earthquake.
- *Landers & Big Bear Earthquakes* - Early on June 28, 1992, a magnitude 7.5 M_S (7.3M_W) earthquake occurred near Landers, the largest seismic event in Southern California for 40 years. Surface rupture occurred just south of the town of Yucca Valley and extended some

43 miles toward Barstow. About three hours later, a magnitude 6.6 M_S (6.4M_W) earthquake occurred near Big Bear Lake.

- *Hector Mine Earthquake* - On October 16, 1999, a magnitude 7.1 M_W earthquake occurred on the Lavic Lake and Bullion Mountain faults north of 29 Palms. This event while widely felt, no significant structural damage has been reported in the Coachella Valley.

The most significant recent earthquakes with respect to proximity to the project site include the 1948 6.0 magnitude Desert Hot Springs earthquake and 1986 6.2 North Palm Springs earthquake with epicenters 8 and 6 miles respectively from the site. While these earthquakes were generated by minor fault rupture along the local segments of the San Andreas fault system, they were well below the maximum magnitude earthquakes of approximately 8.2 anticipated for a multi-segment rupture along the San Andreas fault. The last major fault rupture along the local segments of the San Andreas fault is thought to be in 1690.

Other earthquakes of significance include earthquakes along the San Jacinto fault in 1899 and 1918, with epicenters approximately 30 miles from the site, the 1992 Joshua Tree earthquake, and the 1992 Landers/Big Bear earthquake events. Table 5 lists select significant recorded earthquakes felt in the Desert Hot Springs area and the estimated intensity of ground shaking near the site based on the Modified Mercalli Scale. A description of damage based on the Modified Mercalli Scale is included as Table 6 of this report.

Table 5
Significant Historical Earthquakes

Earthquake	~ Distance to Epicenter Miles (km)	Earthquake Magnitude*	Estimated Intensity**	Date
N. Palm Springs	6 (10)	6.0	VII-VIII	1986
Desert Hot Springs	8 (13)	6.0	VI	1948
Joshua Tree	12 (19)	6.2	VI	1992
Landers	17 (27)	7.3	VII	1992
Big Bear	23 (37)	6.5	VI	1992
San Jacinto	30 (48)	6.7	VI	1899
San Jacinto	31 (50)	6.8	VI-VII	1918
South Anza	41 (66)	6.8	VI	1890
San Bernardino	42 68)	6.2	V-VI	1923
Hector Mine	46 (74)	7.1	VI	1999
Arroyo Salida	51 (82)	6.3	VI	1954

* Moment Magnitude after 1933 or above 6, or Local Magnitude prior to 1933 or below 6 (S.C.E.C.)

** Modified Mercalli Scale

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

Table 6

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

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

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

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

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

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

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

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

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

Table 7 summarizes select significant regional faults that represent potential earthquake sources for this site.

Table 7 - Significant Regional Faults

Fault	Maximum Moment Magnitude	Approximate Distance to Site*
San Andreas – Mission Creek	7.2	0.4 (0.7)
San Andreas – Banning Branch	7.2	3 (5)
Pinto Mountain	7.2	9 (13)
Burnt Mountain	6.7	7 (12)
S. Emerson-Copper Mountain	7.0	23 (37)
San Jacinto-Anza	7.6	26 (42)
San Jacinto-San Jacinto Valley	7.4	26 (42)
Helendale – S. Lockhardt	7.4	30 (48)
San Jacinto-Clark	7.6	33 (53)
Elsinore - Temecula	7.4	48 (77)
Cucamonga	7.0	56 (90)
San Andreas - Mojave	7.8	60 (97)

* Approximate closest distance to fault in miles (kilometers).
Note: Fault parameters are presented in Appendix A.
Note: Multi-segment fault rupture on San Andreas fault could result in magnitude on the order of 8.2

Seismic Risk: 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 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.

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

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.

The soils encountered in the borings generally consist of sands. In general, for the effects of liquefaction to be manifested, groundwater levels must be within 50 feet of the ground surface and the soils within the saturated zone must also be susceptible to liquefaction. The project lies in a zone designated by Riverside County Safety Element and online Land Information System to have a "moderate" liquefaction potential due to relatively deep or unknown groundwater, but susceptible sediments. We consider the potential for liquefaction to occur at this site is low because a groundwater research indicates water is generally more than 50 feet below the ground surface historically and perched water conditions were not encountered despite ongoing site irrigation. The potential for lateral spreading is considered low due to non-liquefiable geologic materials and lack of descending slopes in the proximity of the campus.

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 = 0.923) and an earthquake magnitude of 8.2 to evaluate dry seismic settlement potential. The 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 (10 to 12 feet) was calculated in our three deepest borings at the site and estimated to be 0.7 inches (Boring B-1), 0.1 inches (Boring B-3), and 0 inches (Boring B-4). Seismic settlement is based on post grading recommendations stated in Section 5.1 and considered the observed increasing density with depth demonstrating the settlement is confined to the upper soils. Due to the general uniformity of the soils encountered and the relative minor difference in settlement potential between the borings evaluated, 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 (1/3 inch) considering soil remediation as recommended in Section 5.1.

Due to the shallow site exploration due to refusal on dense soil, boulders and cobbles, seismically induced dry settlement was also evaluated at two nearby sites previously explored by Earth Systems. The fist site is located just south of Desert Hot Springs High School at the southwest corner of the intersection of Pierson Boulevard and Cholla Drive. Dry seismic settlement for the soil column height of 13.5 to 15.5 feet and was calculated to be from 0.1 inches to 0.3 inches. The second site is located east of Desert Springs High School at 66-135 4th Street. Dry seismic settlement for the soil column height of 16.5 to 31.5 feet was calculated to be from 0.1 inches to 0.4 inches. Therefore, calculated seismically induced dry settlement are similar in the vicinity of Desert Hot Springs High School due to the general uniformity of the soils encountered, and support our previously stated comments regarding uniformity and magnitude of settlement estimated.

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

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

Seismic Hazard Zones: This portion of Riverside County has not been mapped for the California Seismic Hazard Mapping Act (Ca. PRC 2690 to 2699).

Site Acceleration and Seismic Coefficients: In developing site specific seismic design criteria, the characteristics of the earth units underlying the site are an important input to evaluate the site response at a given site. Based on the results of our evaluation at the site, the project site is underlain by artificial fill overlying Quaternary alluvial deposits. Based on our estimation of Shear Wave Velocity for the upper 100 feet of site soils (see Appendix A for output), the site classification for site response is Site Class D according to Table 20.3-1 of ASCE7-10. The D characterization is defined as a soil profile consisting of stiff soil with shear wave velocities between 600 to 1,200 fps.

Probabilistic Analysis and General Procedure: The Seismic Design Category for this site is E. The Code seismic parameter S_1 is 0.898 g (greater than 0.75 g). The site is not within a designated Alquist-Priolo Earthquake Fault Zone, or County Fault Zone. Therefore, per CGS Note 48, Section 16, a probabilistic analysis is required and presented in Section 5.8 due to the S_1 Spectral Acceleration and Seismic Design category E.

2016 CBC Seismic Coefficients: The California Building Code [CBC] seismic design parameters criteria are based on a Design Earthquake that has an earthquake ground motion $^2/_3$ of the lesser of 2 percent probability of occurrence in 50 years or maximum 84th percentile of the mean deterministic maximum considered earthquake. The seismic and site coefficients given in Chapter 16 of the 2016 California Building Code are provided in Section 5.8 of this report.

3.7.2 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 are identified on FEMA Map No.: 06065C0885G, Panel 885 of 3805, Map Revised August 28, 2008. Appropriate project design, construction, and maintenance can minimize the site sheet flooding potential.

Seiches: Seiching is defined as a periodic oscillation of liquid within a container or reservoir. Its period is determined by the resonant characteristics of the container, as controlled by its physical dimensions. A swimming pool is located approximately 400 feet to the west of the proposed additions. It is likely any flooding associated with pool seiches would follow existing on-site drainage improvements, such that the impact to the site would be negligible.

Section 4
CONCLUSIONS

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

General:

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

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

Geotechnical Constraints and Mitigation:

- The primary geologic hazard is moderate to severe ground shaking from earthquakes originating on regional faults. A major earthquake originating on the nearby segments of the San Andreas fault zone, and other associated faults would be the critical seismic event that may affect the site within the design life of the campus. Engineered design and earthquake-resistant construction increase safety and allow development within seismic areas.
- The underlying geologic condition for seismic design is Site Class D. The site is about 0.4 miles from a Type A seismic source as defined by the California Geological Survey. A qualified professional should design any permanent structure constructed on the site. The ***minimum*** seismic design should comply with the 2016 edition of the California Building Code.
- Due to the spectral acceleration S_1 (0.898 g) being greater than 0.75, a site specific response spectra is included. The Seismic Design Category is E.
- 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. Therefore, the potential for surface fault rupture at the site is considered very low.
- The potential for ground subsidence and liquefaction settlement hazards are considered very low to low for this project. The site is not within an area of documented areal subsidence.
- The soils are susceptible to wind and water erosion.
- 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.
- 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 granular fill material, very low in expansion potential, for the building pad such that standard foundations and reinforcing can be used.

- The upper site soils have been previously graded and placed as fill. The fill has some non-uniformity but is generally dense. We understand a grading and compaction report for these fills is not available. As such, they are considered undocumented and should be remedial graded as recommended within. Due to the ongoing irrigation in the site area, soil moistures were generally near optimum.
- Site soils have areas with higher percentages of boulders and cobbles, which are "oversize" and must be handled during grading.
- Laboratory testing of one sample showed potentially mild corrosivity to buried metallic elements and "negligible" for sulfate exposure to concrete. See Section 3.5 for further information. Site soils should be reviewed by an engineer competent in corrosion evaluation.
- In our professional opinion, 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 rootball), large roots, overly wet and/or soft soil, undocumented fill, pavements, foundations, construction debris, septic tanks, leach fields, deleterious material, trash, and abandoned underground utilities should be removed from the proposed building areas. Organic growth should be stripped off the surface and removed from the construction area. Areas disturbed during demolition and clearing should be properly backfilled and compacted as described below.

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: Building pads are shown on Plate 2 in the Appendix A. Subsequent to clearing and grubbing operations, the entire building pad and other structural areas (such as areas of fill (including all fill slopes), shade structures, canopies, overhead awnings, or any areas with foundations, etc.), should be over-excavated to remove existing undocumented fill and the upper near surface compressible portion of the alluvial soils. The existing soils within the building pad and foundation areas should be over-excavated a minimum of 5 feet below existing grade or 3 feet below the bottom of the foundation, whichever is lower. The exposed undisturbed subgrade bottom should be observed and tested by the geotechnical engineer or their representative to verify an in-place density of the subgrade is at or greater than 85% relative compaction per ASTM D 1557 or soils are firm (as determined by the geotechnical engineer or his representative). Deeper over-excavation may be recommended if the required in-place density is not achieved, or soils are not firm. The over-excavation should extend horizontally for at least 5 feet or the depth of the over-excavation, whichever is greater, beyond the outer edge of the building pad where possible and include all exterior footings or slabs and include any overhead canopy/or covered walkway and patio areas.

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

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

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. Compaction should be verified by testing.

Pavement and Hardscape Area Preparation: In street, drive, permanent parking, and hardscape areas the subgrade should be over-excavated a minimum depth of two feet below existing grade or finish grade (whichever is deeper). The excavation bottom should be scarified 12 inches, moisture conditioned to near or over optimum moisture content and be recompacted to at least 90% relative compaction. Engineered fill should then be moisture conditioned, placed in suitable lifts, and compacted to a minimum of 90% relative compaction to finish grade, with the upper 1 foot compacted to at least 95% relative compaction in parking and drive areas. Compacted fill should be placed to finish subgrade elevation. Compaction should be verified by testing.

Retention Basin and Infiltrator Bottom Preparation: Compaction effort should be kept to a minimum at retention basin bottom areas and bottom areas used for any infiltrators (except under foundations). The subgrade below the bottom of basins and infiltrator bottoms should be compacted to approximately 85% relative compaction. Side slopes and any other fill or foundation subgrade should be compacted to at least 90% relative compaction. Slope construction should be per this report. Loose rock, such as pea gravel or open graded rock placed in the basin bottoms does not require compaction testing, but should be placed in lifts no greater than 2 feet and consolidated by thoroughly wetting and consolidating by passes with heavy equipment (such as a loader with full bucket or full water truck) until firm such that none to minimal deformation (less than 1 inch) occurs under the weight of passing of equipment. Basins are recommended to have hydrocollapsible soils removed to competent soil. Infiltrator bottoms are recommended to be at least 6 feet deep below existing grades and have hydrocollapsible soils removed to competent soil. Competent soil is defined as soil meeting the compaction or density criteria as described for *Building Pads*.

<u>Slope Construction</u>: Please see Section 5.5 for detailed slope preparation recommendations.

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

<u>Engineered Fill Soils</u>: The 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 50), vegetation, trash, large roots, overly wet and/or soft soil, clods larger than 3 inches, construction debris, oversized rock (greater than 6 inches) and other deleterious material as determined by the geotechnical engineer or his representative. From research of USDA and during drill logging, oversize rock (cobbles and boulders) exist onsite. The USDA designation of Carsitas Fine Sand indicates 0-15 percent of the 3 inch or larger rock may exist (see also next few paragraphs with discussion and table regarding percent oversize estimation in our exploration). Fill should remain substantially soil (at least 70% passing a ¾" sieve and similar to the gradation below for import soils, although an actual specification is not proposed). Unprocessed materials should be hauled offsite.

Screening and crushing, or rock picking will likely be required to properly remove oversize material and obtain a soil fill. Rock fills are not recommended due to the shallow fill and overexcavation depths. Import fill soils (if needed) should be non-expansive, granular soils meeting the USCS classifications of SM, SP-SM, or SW-SM with a maximum rock size of 3 inches and have at least 70% passing a ¾ inch sieve and at least 40% passing the No. 16 sieve and 5 to 35% passing the No. 200 sieve (or as 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 soils will not be prequalified by Earth Systems.

Within areas to receive foundations and slabs-on-grade the fill should be "very low" in Expansion Index. Expansive soils which are identified should be removed and replaced with low permeability soils which are "very low" in expansion potential. Soils which are found to have an

Expansive Index greater than "very low" will require differing compaction and moisture conditioning requirements which should be provided on a case by case basis for each specific building location.

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. 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. In general, oversize rocks larger than 6-inches in greatest dimension should be removed from fill. Oversize material may be hauled offsite, used for landscaping, or crushed for use in engineering fill. Crushed rock should conform to the specification for import fill.

A program of compaction testing, including frequency and method of test, should be developed by the project geotechnical engineer at the time of grading. Acceptable methods of test may include Nuclear methods such as those outlined in ASTM D 6938 (Standard Test Methods for In-Place Density and Water Content of Soil and Soil-Aggregate by Nuclear Methods). 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. Fill must remain less than 30% retained on a ¾" sieve and meet the requirements for oversize.

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 Oversize Loss</u>: The shrinkage factor for earthwork for the alluvial soil materials is expected to range from -15 to 10 percent for the upper excavated or scarified *site* soils based upon evaluation of 16 in-place densities (one standard deviation = 6, 95% Confidence Interval). This estimate is based on compactive effort to achieve a weighted average relative compaction of about 93 percent.

Greater soil loss from shrinkage will result from the removal of oversize material (rocks larger than 6 inches). We have performed a rock volume estimate based upon a rock count performed at each test pit in relation to the volume of the test pits excavated and have considered rock volumes in various zones in upper approximate 5 ½ to 10 feet. Oversize rock quantity estimates are presented in the table below. Losses from oversize rock quantity are in addition to losses from soil shrinkage.

Table 8
Estimated Losses Due to Oversize Rock

Test pit Location	Depth (feet)	Estimated Oversize Rock Percentage by Volume
TP-1	0 – 4.5	1
TP-1	4.5 – 6	5
TP-2	0 – 4	5 – 10
TP-2	4 – 7	30
TP-3	0 – 2.5	5
TP-3	2.5 – 5.5	30
TP-4	0 – 3	<5
TP-4	3 – 8	5
TP-5	0 – 8	<5
TP-5	8 – 10	5 – 10
TP-6	0 – 5.5	<3
TP-6	5.5 – 9	5 – 10

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

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

5.2 Excavations and Shoring

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

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

Excavations 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. 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 9
Temporary Cantilevered and Braced Shoring System Parameters

Equivalent Fluid Pressure pounds per cubic foot (pcf)	
Cantilevered	**Braced**
35	55

The values provided above assume a level ground surface adjacent to the top of the shoring and do not include a factor of safety. Fifty percent of an areal surcharge placed adjacent to the shoring may be assumed to act as an additional uniform horizontal pressure against the shoring. Special cases such as combinations of slopes and shoring or other surcharge loads may require an increase in the design values recommended above. These conditions should be evaluated by the project geotechnical or shoring engineer on a case-by-case basis. Retaining walls subjected to traffic loads should include a uniform surcharge load equivalent to at least 250 psf for auto or delivery truck (2 axle) traffic kept at least 3 feet from the back of the wall. Retaining walls with closer traffic or heavier traffic loads should be designed for a 400 psf surcharge load. Retaining walls should be designed with a minimum factor of safety of 1.5.

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

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

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

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

5.3 Utility Trenches

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

Trench Width and Vertical Loads on Pipelines: 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 shall consist of sand 100 percent passing a No. 4 sieve and less than 5 percent fines (passing a No. 200 sieve), and a sand equivalent of 30 or more or as approved by the project inspector and geotechnical engineer. The unprocessed native soils are not typical of that used for bedding and import will be required if needed.

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

Pipe zone backfill material (the pipe area from the bedding to 12 inches above the top of pipe) 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. Compaction should be assured in the pipe haunches.

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

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

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

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

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

STRUCTURES

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

5.4 Foundations

Footing design of widths, depths, and reinforcing are the responsibility of the Structural Engineer, considering the structural loading and the geotechnical parameters given in this report. A minimum footing depth of 12 to 24 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. *All footing excavations should be probed for uniformity. Soft or loose zones should be excavated and recompacted to finish foundation bottom subgrade. The bottom of all foundations should be tested to confirm compaction effort and moisture contents as stated in Section 5.1 of this report are met.* The moisture contents should be at least the indicated moisture content 24 hours prior to and immediately prior to placing concrete for a depth of at least 12 inches below the foundation subgrade. If the moisture condition is less than indicated, it shall be brought up to or above the indicated moisture content.

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

- minimum depth below grade:

  1,800 psf for dead plus design live loads

- Pad foundations, 2 x 2-foot minimum and 4 x 4-foot maximum in plan and 24 inches below grade:

  2,500 psf for dead plus design live loads

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

If the anticipated loads exceed the estimated values stated in Section 1.1 (36 kips for Isolated Footings and 2.5 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.

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

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

An average modulus of subgrade reaction, k, of 200 pounds per cubic inch (pci) can be used to design lightly loaded footings and slabs founded upon compacted fill. Other foundations such as mat slabs, will require the use of differing modulus of subgrade reaction values than used for lightly loaded slabs. 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 7,500 psf ultimate) and a safety factor of 2.25 on transient loads (2,400 to 3,333 psf). Ultimate bearing to soil failure depends on foundation size and could be much greater than 7,500 psf. The restrictions of Section 1605A.1.1 apply to the cited bearing values for Allowable Stress Design (ASD).

Table 10

	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	2,500	3,333	7,500

FS = Factor of Safety

Footings should be designed and reinforced by the structural engineer for the specific loading, settlement, or expansive soil conditions. A minimum of two, #4 reinforcing bars should be placed. One near the top of the footing and one 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, seismic, and collapse settlement should be less than 1.5 inches, based on footings founded on firm soils as recommended. Differential static settlement between exterior and interior bearing members should be less than 3/4 inch. As such, considering static, seismic, and collapse settlement applied over a typical foundation distance of 40 feet, we recommend the structural engineer design for an angular distortion of 1:480 (1 inch in 40 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.

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. Due to the generally granular nature of the site soils, a substantial portion of the total static settlement is expected to occur during construction.

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

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

The on-site soils are expected to be very difficult to excavate with conventional drilling. Drilled piers should have a minimum 3 inches of clearance between the embedded post and the soil side wall to allow for adequate placement and flow of concrete.

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

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

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

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

Pier capacity is greatly dependent on the soil conditions at the location of the pier and upon contractor means and methods of placement. It is recommended that drilling operations and concrete placement be performed in the continuous presence of the geotechnical consultant or his representative to confirm that suitable materials for pier support are penetrated, that the dimensions of the installed piers meet the design dimensions, and that the installation has been performed as specified by the 2016 California Building Code. <u>Observation during drilling is required by the 2016 California Building Code on a full-time basis by the geotechnical engineer or his representative</u>. If subsurface conditions noted during drilled pier installation are significantly different than those encountered in our borings, it may be necessary to adjust the overall length of the pier.

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

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

5.5 Slope Construction

Slopes are not generally proposed for this project; however, minor slopes (less than 5 feet in height) may be constructed. Site soils are highly 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 which 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.

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

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

5.6 Slabs-on-Grade

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

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

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

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

Table 11
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 200 pounds per cubic inch can be used in concrete lightly loaded (not mat) slab design for the expected compacted subgrade. Mat slab design will require differing modulus values. 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.

<u>Slab-On-Grade Control Joints</u>: 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 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.

<u>Curing and Quality Control</u>: 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. Native sand material may be used for backfill or free draining material imported 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 by the contractor to be "very low" in expansion potential. 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 35 pcf of equivalent fluid weight for well-drained, level backfill may be used. Similarly, an active pressure of 44 pcf of equivalent fluid weight may be used for well-drained backfill sloping at 2H:1V (horizontal to vertical). For the restrained level backfill condition, a pressure of 55 pcf of equivalent fluid weight should be used.
- 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 61 pcf for flexible walls, and 77 pcf for rigid walls. Seismic pressures are based on PGA_M of 0.92g, Friction Soil Angle of 34^o, and a maximum dry density of 125 pcf.
- Retaining wall foundations should be placed upon compacted fill described in Section 5.1.
- A backdrain or an equivalent system of backfill drainage should be incorporated into the wall design, whereby the collected water is conveyed to an approved point of discharge. 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. 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 0.35 may be used between cast-in-place concrete foundations and slabs and the underlying soil. An allowable coefficient of friction of 0.30 may be used between pre-cast or formed concrete foundations and slabs and the underlying soil
- Allowable passive pressure may be taken as equivalent to the pressure exerted by a fluid weighing 300 pounds per cubic foot (pcf). The upper 1 foot of soil should not be considered when calculating passive pressure unless confined by overlying asphalt concrete pavement or Portland cement concrete slab. The soils pressures presented have considered onsite fill soils. Testing or observation should be performed during grading by the soils engineer or his representative to confirm or revise the presented values.
- Passive resistance for thrust blocks bearing against firm natural soil or properly compacted backfill can be calculated using an equivalent fluid pressure of 300 pcf. The maximum passive resistance should not exceed 2,000 psf.
- Construction employing poles or posts (i.e. lamp posts) may utilize design methods presented in Section 1807.3 of the CBC for Sandy soils (SP) 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 San Andreas fault zone. 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; however the spectral acceleration S1 is greater than 0.75 g. Therefore, a site specific response spectra is required per CGS guidelines. General Procedure and site specific seismic parameters are presented below considering a Site Class D (results in Appendix A). Our site specific analysis considered the San Andreas fault and the results for each analysis are listed below and see Tables 3 and 4 in Appendix A. The structural design engineer should use the most conservative results based on the specific building design and spectral response.

2016 CBC (ASCE 7-10) Seismic Parameters

Seismic Design Category:	E
Site Class:	D
Maximum Considered Earthquake [MCE] Ground Motion	
Short Period Spectral Response S_S:	2.514 g
1 second Spectral Response, S_1:	0.898 g
Code Design Earthquake Ground Motion	
Short Period Spectral Response, S_{DS}	1.676 g
1 second Spectral Response, S_{D1}	0.899 g
Peak Ground Acceleration (PGA_M)	0.924 g

Required Site Specific Design Earthquake Ground Motions*
(Appendix A)

*Short Period Spectral Response, S_{DS}	1.383 g (San Andreas)
*1 second Spectral Response, S_{D1}	1.428 g (San Andreas)

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 12
Preliminary Flexible Pavement Section Recommendations
On-site/Interior Automobile Drive Areas

R-Value of Subgrade Soils - 65 (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	4
7	Main Drive Areas	4	4

*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 13
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 B, ADTT =25)	5.5	550	3,650

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).
- <u>Subsequent to utility installation, the entire pavement (including PCC) final subgrade should be scarified 12 inches, moisture conditioned to near optimum moisture content, and compacted to a minimum 95% relative compaction immediately prior (within a few days) to the placement and compaction of aggregate base to re-establish proper moisture content and compaction in site soils</u>.
- Subgrade soils and aggregate base should be in a stable, <u>non-pumping</u> 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. maximum-medium 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 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 (3,650 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.
- It is highly recommended that landscape irrigation or other sources of water be collected and conducted to an approved drainage device. Site drainage should be devised such that runoff should be directed away from the tops of all graded slopes. Water should not freely flow over 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 at the top and bottom of slopes are recommended.
- In no instance should water be allowed to flow or pond against structures, slabs or foundations or flow over unprotected slope faces. Adequate provisions should be employed to control and limit moisture changes in the subgrade beneath foundations or structures to reduce the potential for soil saturation. Landscape borders should not act as traps for water within landscape areas. Potential sources of water such as piping, drains, over-spray broken sprinklers, etc, should be frequently examined. Any such leakage, over-spray, or plugging should be immediately repaired.
- Maintenance of drainage systems and infiltration structures can be the most critical element in determining the success of a design. They must be protected and maintained from sediment-laden water both during and after construction to prevent clogging of the surficial soils any filter medium. The potential for clogging can be reduced by pre-treating

structure inflow through the installation of maintainable forebays, biofilters, or sedimentation chambers. In addition, sediment, leaves, and debris must be removed from inlets and traps on a regular basis. Since these and other factors (such as varying soil conditions) may affect the rate of water infiltration, it is imperative to apply a conservative factor of safety [FOS] to unfactored Basic Percolation/Infiltration Rates to provide a reliable basis for design. In order to account not only for the unknown factors above but also for changes of conditions during the use of the structures such as potential clogging effects due to washing in of soil fines, a FOS between 3 and 10 should be applied to lower infiltration rates.

- The factor of safety should be selected by the project drainage engineer and may be dependent on agency guidelines and the presence of testing, filters, and sedimentation structures. If these measures are provided, the factor of safety can be reduced.
- The drainage pattern should be established at the time of final grading and maintained throughout the life of the project. Additionally, drainage structures should be maintained (including the de-clogging of piping, basin bottom scarification, soil crush 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 are based on the assumption that soil conditions do not vary significantly from those found at specific exploratory locations. Variations in soil or groundwater conditions could exist between and beyond the exploration points. The nature and extent of these variations may not become evident until construction. Variations in soil or groundwater may require additional studies, consultation, and possible revisions to our recommendations.

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

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

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

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

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

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

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

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

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

6.2 Additional Services

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

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

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

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

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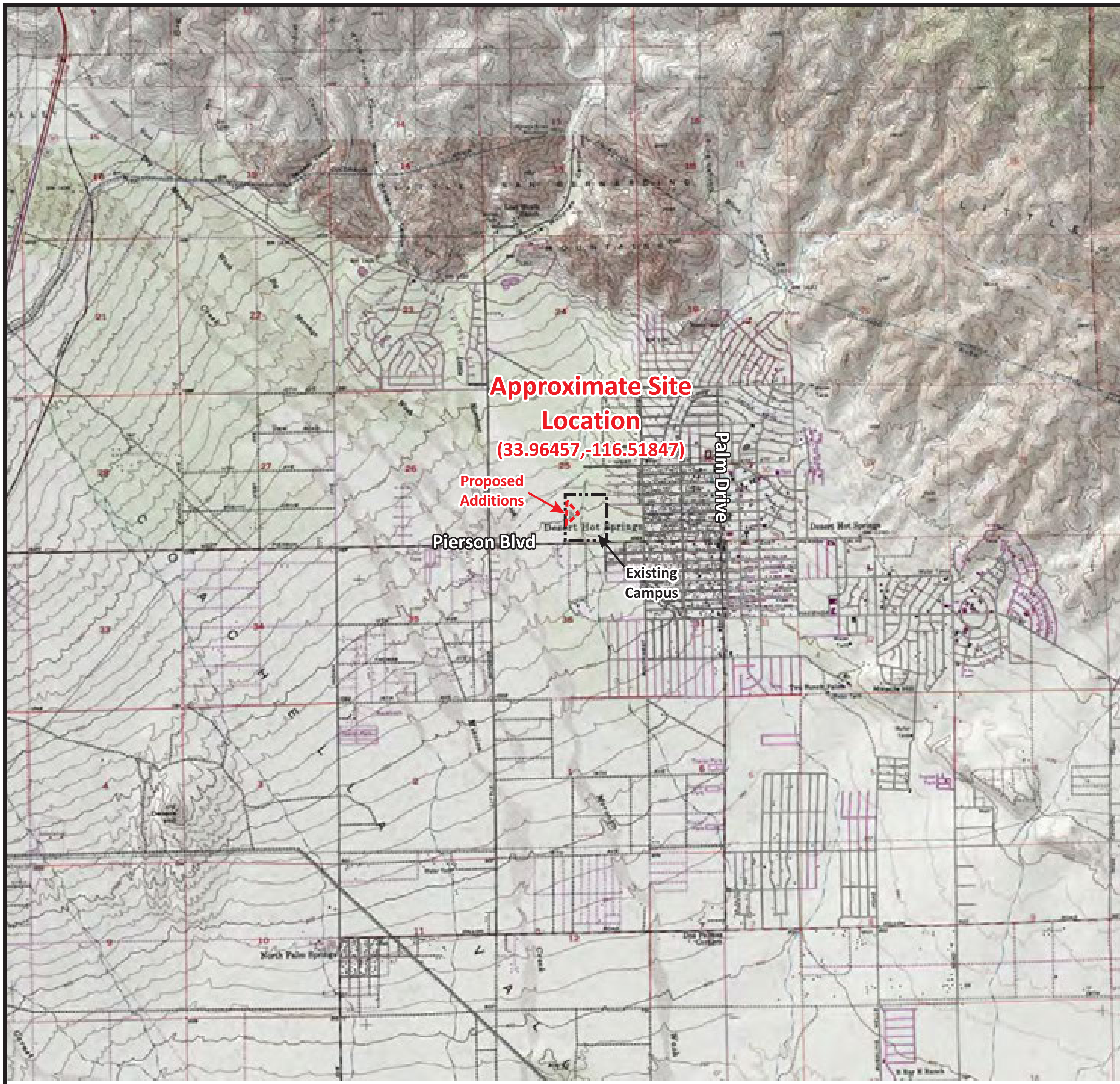
Aerial Photographs:

Google Earth Pro: 1995-2018

Historical Aerials, website https://www.historicaerials.com/viewer: Aerial Photographs, 1972 to 2014; Historical Topographic Maps, 1944 to 2015.

APPENDIX A



Reference: Google Earth satellite image with USGS topographic map overlay image date February 19, 2018.

LEGEND

Approximate Site Boundary

Scale: 1" = 1 Mile

0 1 Mile 2 Miles

N

Plate 1
Site Vicinity Map

Desert Hot Springs High School - CTE Building
65850 Pierson Boulevard
Desert Hot Springs, Riverside County, California

Earth Systems

9/6/2018

File No.: 302396-001

Reference: Google Earth satellite image dated 2/19/18 with proposed PBK CTE Building Plan overlay.

Approximate Scale: 1" = 50’

0 50’ 100’

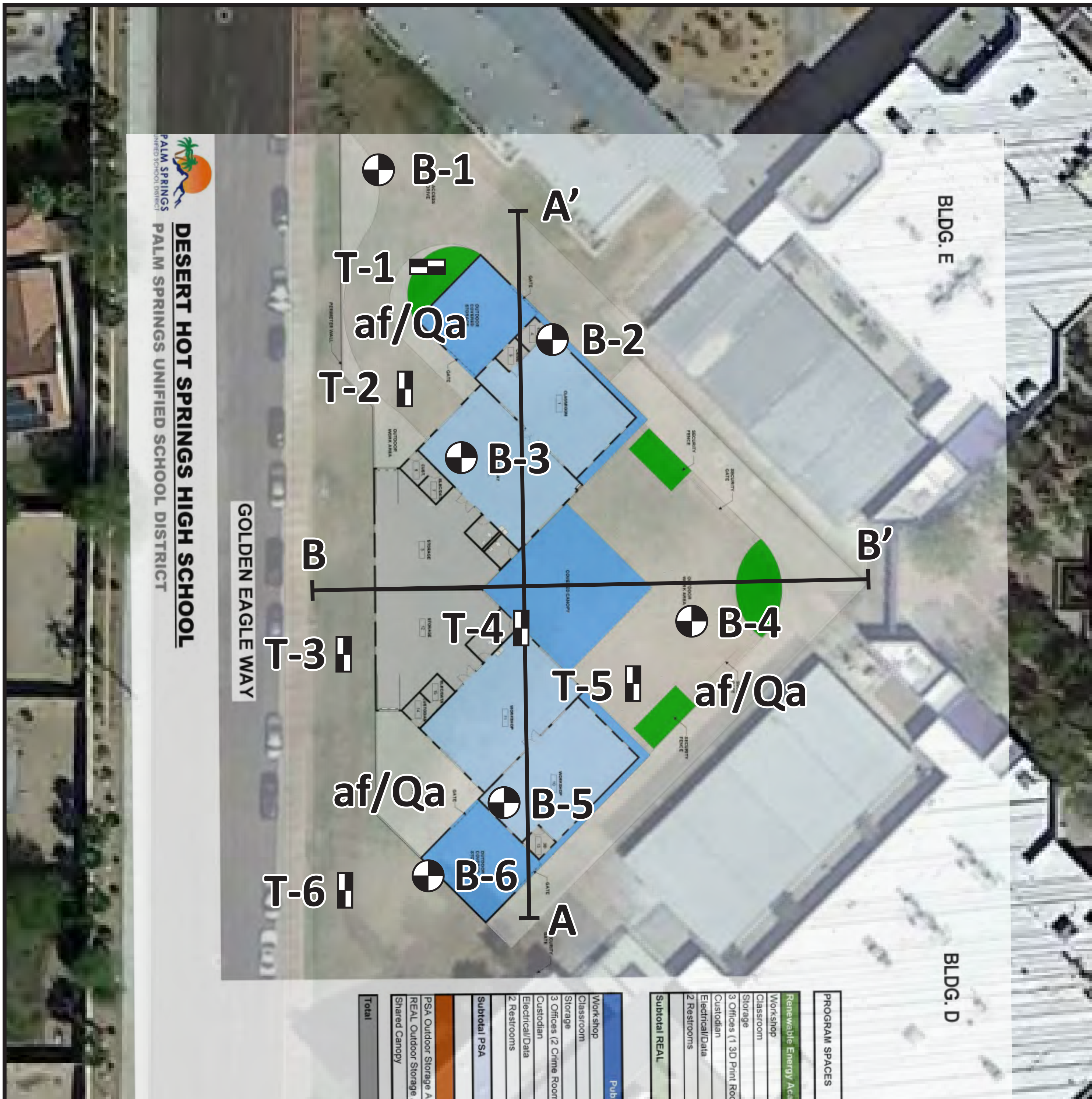
N

Plate 2
Proposed Site Improvements

Desert Hot Springs High School - CTE Building
65850 Pierson Boulevard
Desert Hot Springs, Riverside County, California

Earth Systems

9/6/2018	File No.: 302396-001



Reference: Google Earth satellite image dated 2/19/18 with proposed PBK CTE Building Plan overlay.

Approximate Scale: 1" = 50'

0 50' 100'

LEGEND

B-6 Approximate Boring Locations

T-6 Approximate Test Pit Locations

B B' Approximate Cross Section Locations

af/Qa Artificial Fill over Quaternary Alluvium

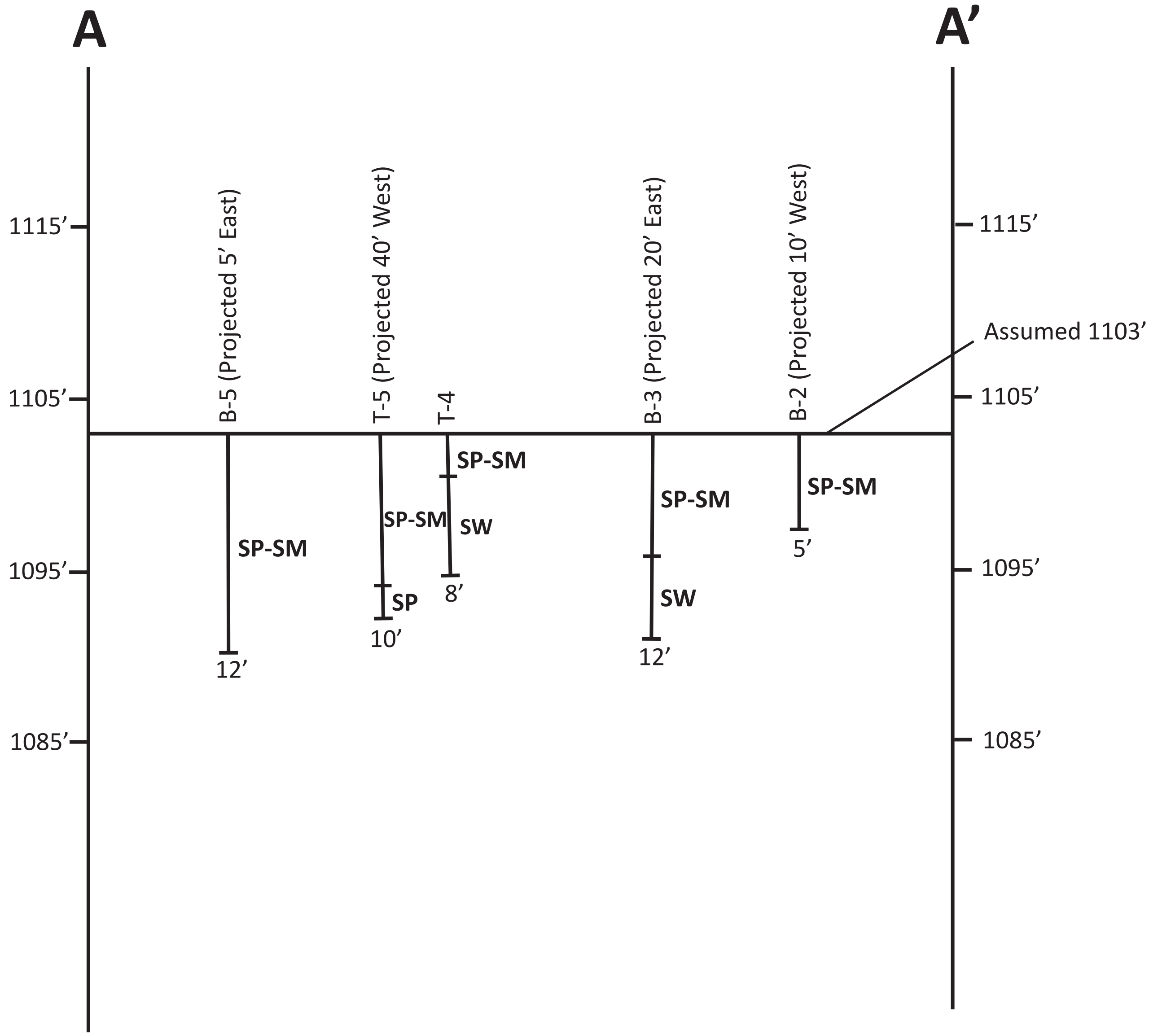
N

Plate 3
Boring & Test Pit Location Map & Site Geology

Desert Hot Springs High School - CTE Building
65850 Pierson Boulevard
Desert Hot Springs, Riverside County, California

Earth Systems

9/6/2018	File No.: 302396-001



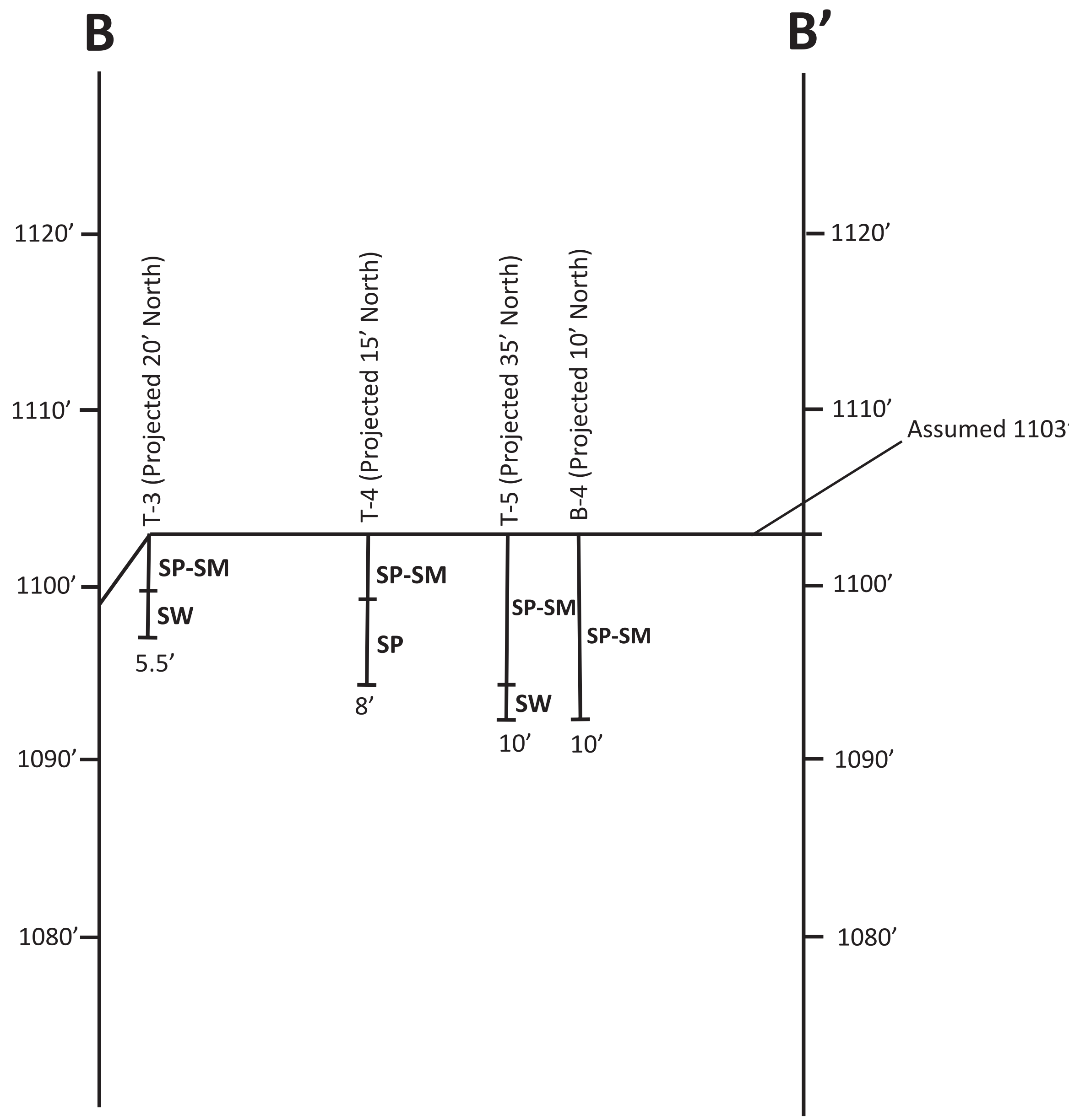
Approximate Horizontal Scale: 1" = 50'
Approximate Vertical Scale: 1" = 10'

Plate 3a
Geologic Cross Section A-A'

Desert Hot Springs High School - CTE Building
65850 Pierson Boulevard
Desert Hot Springs, Riverside County, California

Earth Systems

9/6/2018	File No.: 302396-001



Approximate Horizontal Scale: 1" = 50'
Approximate Vertical Scale: 1" = 10'

Plate 3b
Geologic Cross Section B-B'

Desert Hot Springs High School - CTE Building
65850 Pierson Boulevard
Desert Hot Springs, Riverside County, California

Earth Systems

9/6/2018	File No.: 302396-001

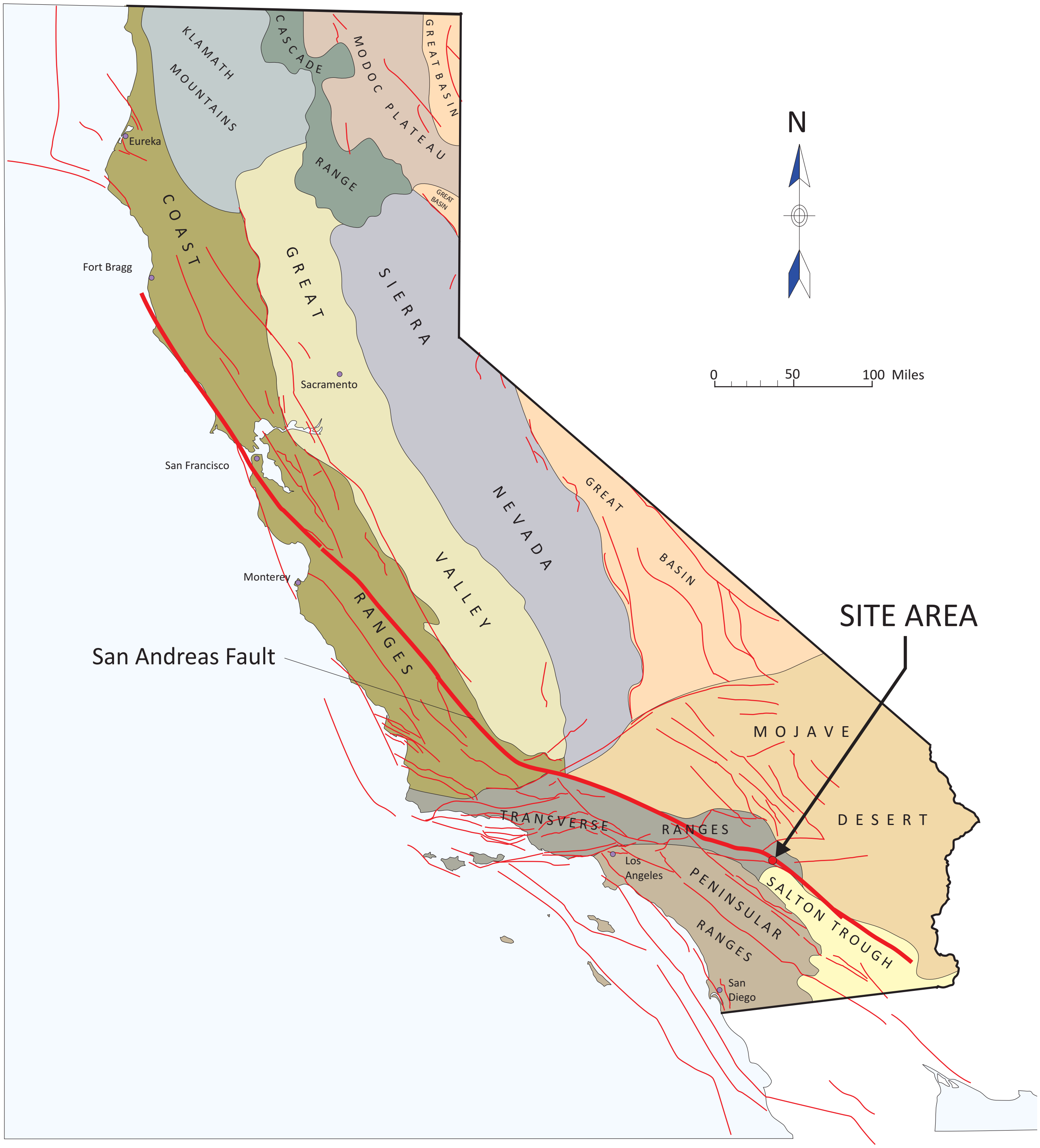


Figure 3

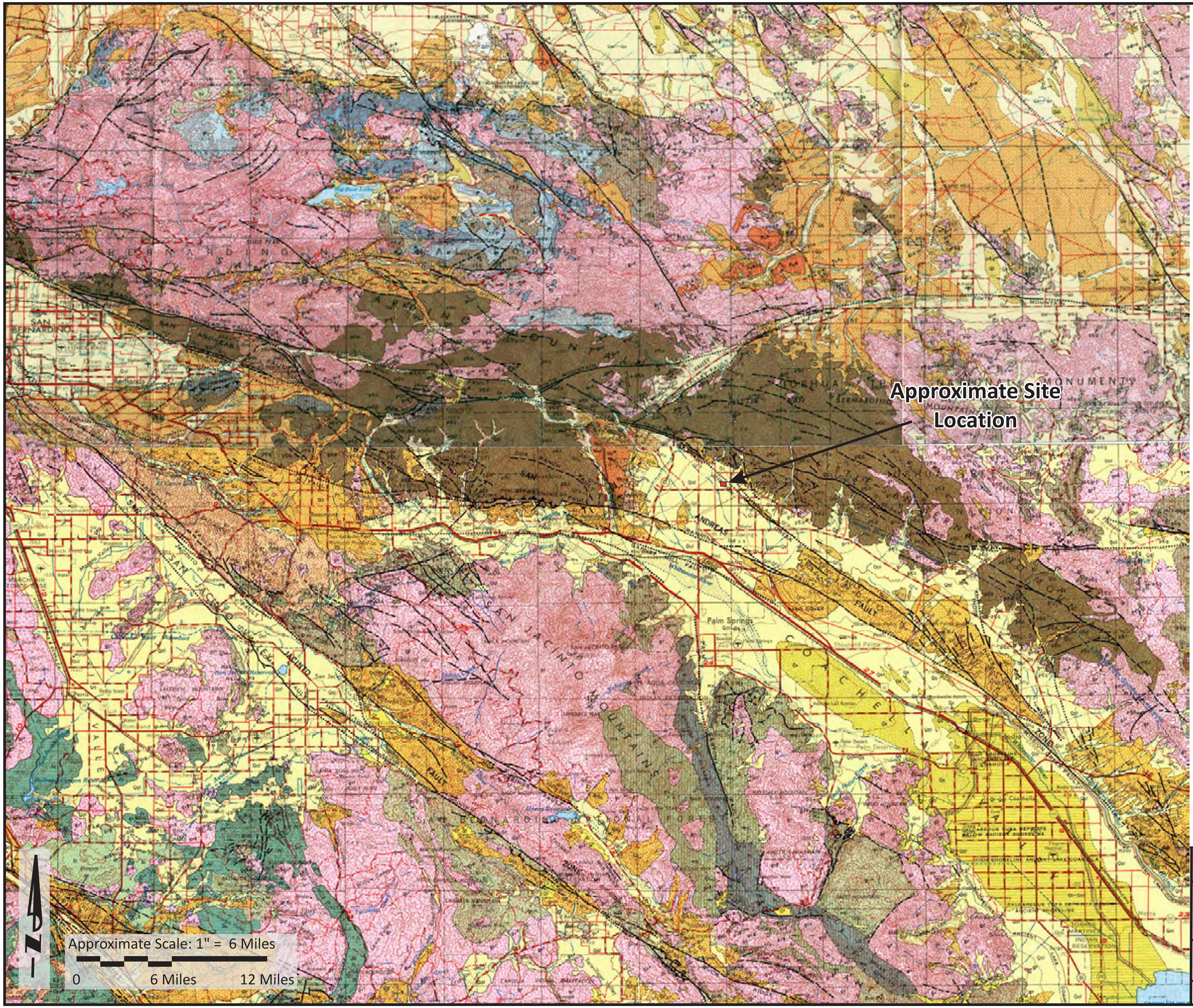
Map showing geomorphic provinces of California and major active and potentially active faults. Fault locations are based on Jennings (1994) and Blake (2000).

Plate 4
State of California Geomorphic Map

Desert Hot Springs High School - CTE Building
65850 Pierson Boulevard
Desert Hot Springs, Riverside County, California

Earth Systems

9/6/2018 | File No.: 302396-001



LEGEND

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

– Fault: Dashed where approximate, dotted where concealed

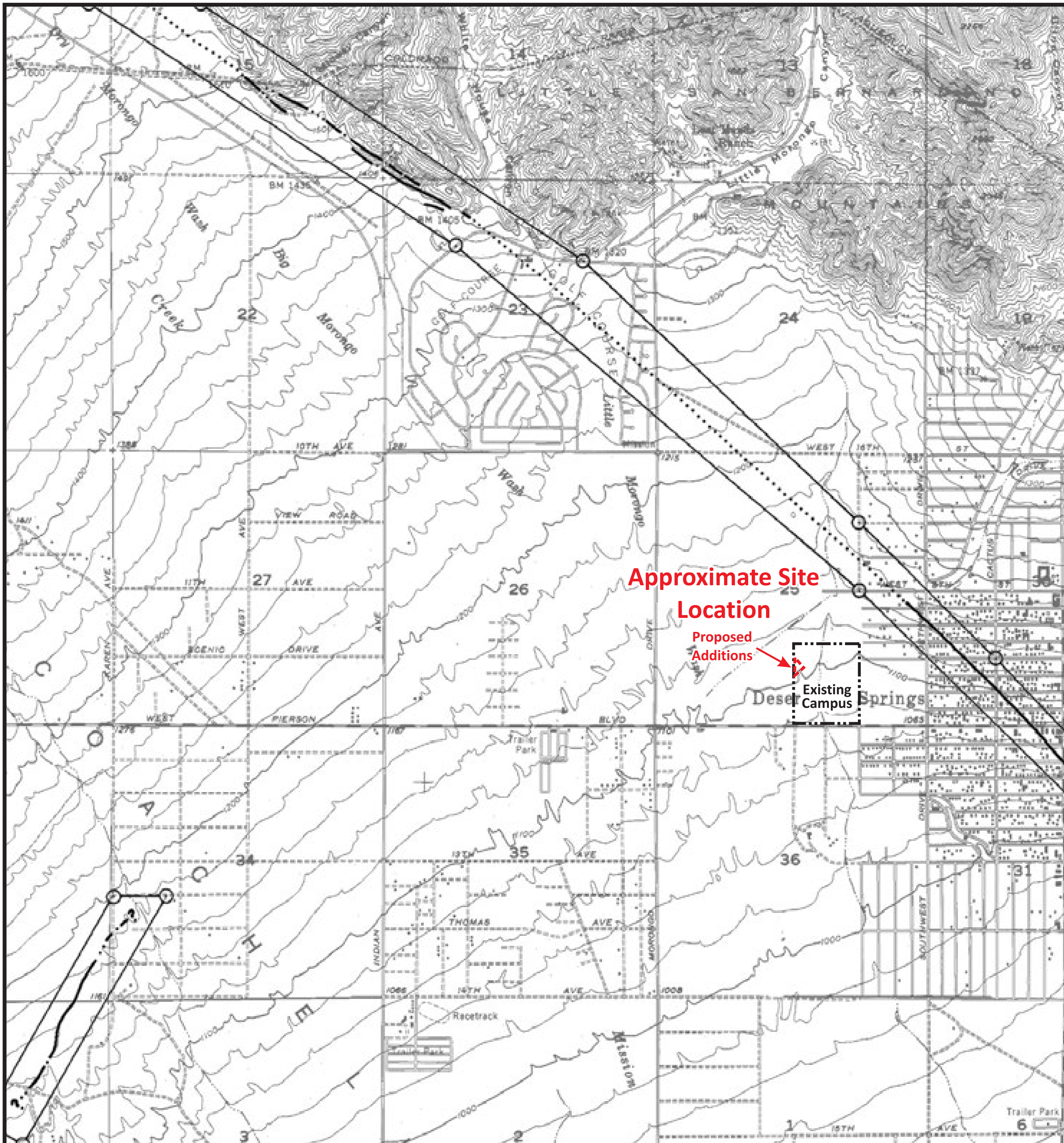
Source: USGS Geologic Map of California, Santa Ana sheet, dated 1965

Plate 5
Regional Geologic Map

Desert Hot Springs High School - CTE Building
65850 Pierson Boulevard
Desert Hot Springs, Riverside County, California

Earth Systems

9/6/2018 | File No.: 302396-001



Source: State of California Special Studies Zones, Desert Hot Springs Quadrangle, January 1, 1980.

LEGEND

Approximate Site Location

Scale: 1" = ½ Mile

0 ½ Mile 1 Mile

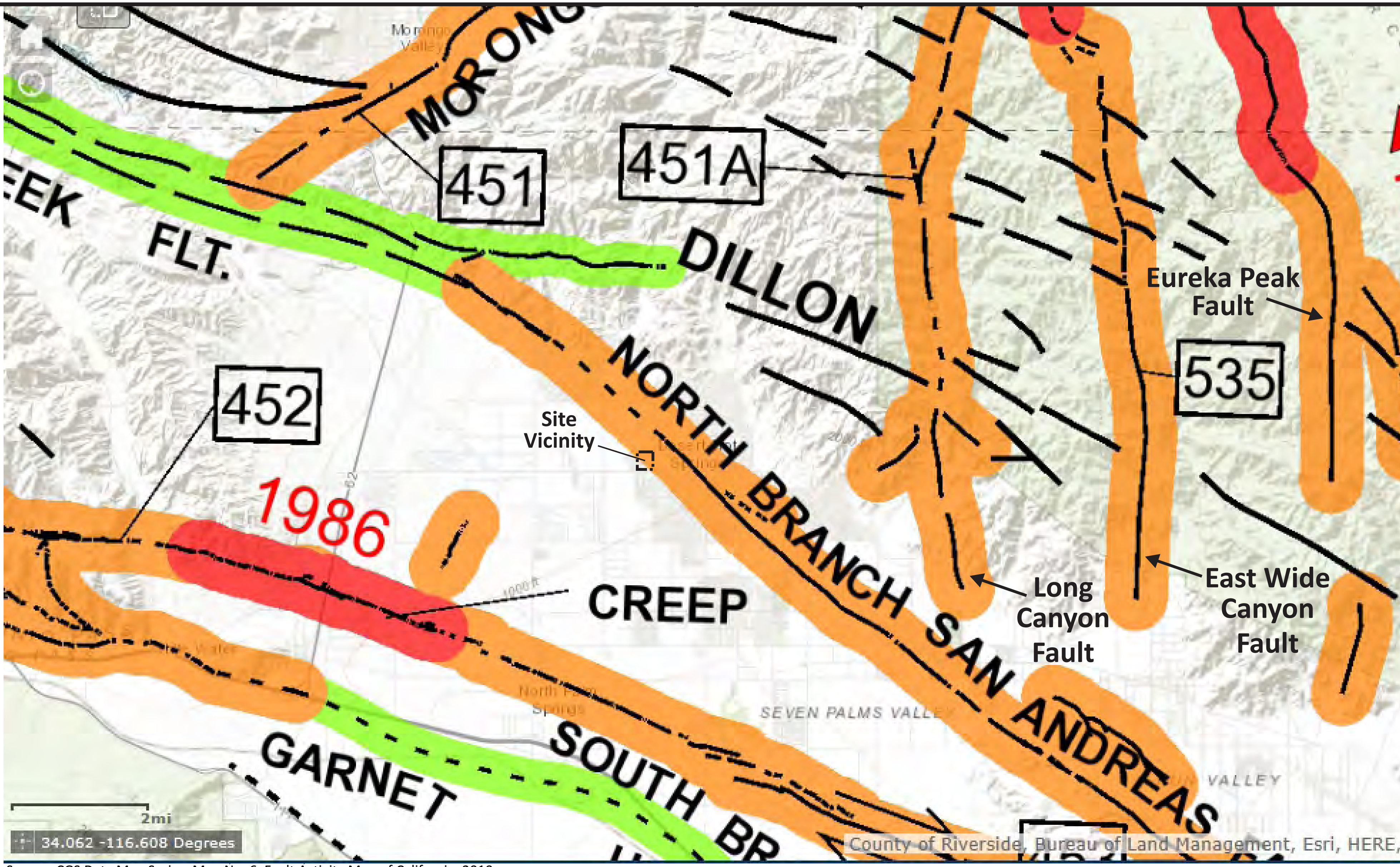
N

Plate 6
Alquist-Priolo Earthquake Fault Zones

Desert Hot Springs High School - CTE Building
65850 Pierson Boulevard
Desert Hot Springs, Riverside County, California

Earth Systems

9/6/2018	File No.: 302396-001



Source: CGS Data Map Series, Map No. 6, Fault Activity Map of California, 2010.

LEGEND

- Fault along which historic (last 200 years) displacement has occurred.
- Fault along which Holocene (last 11,700 years) displacement, has occurred.
- Fault along which Late Quaternary (past 700,000 years) displacement has occurred.
- Fault along which Quaternary (past 1.6 million years) displacement has occurred.

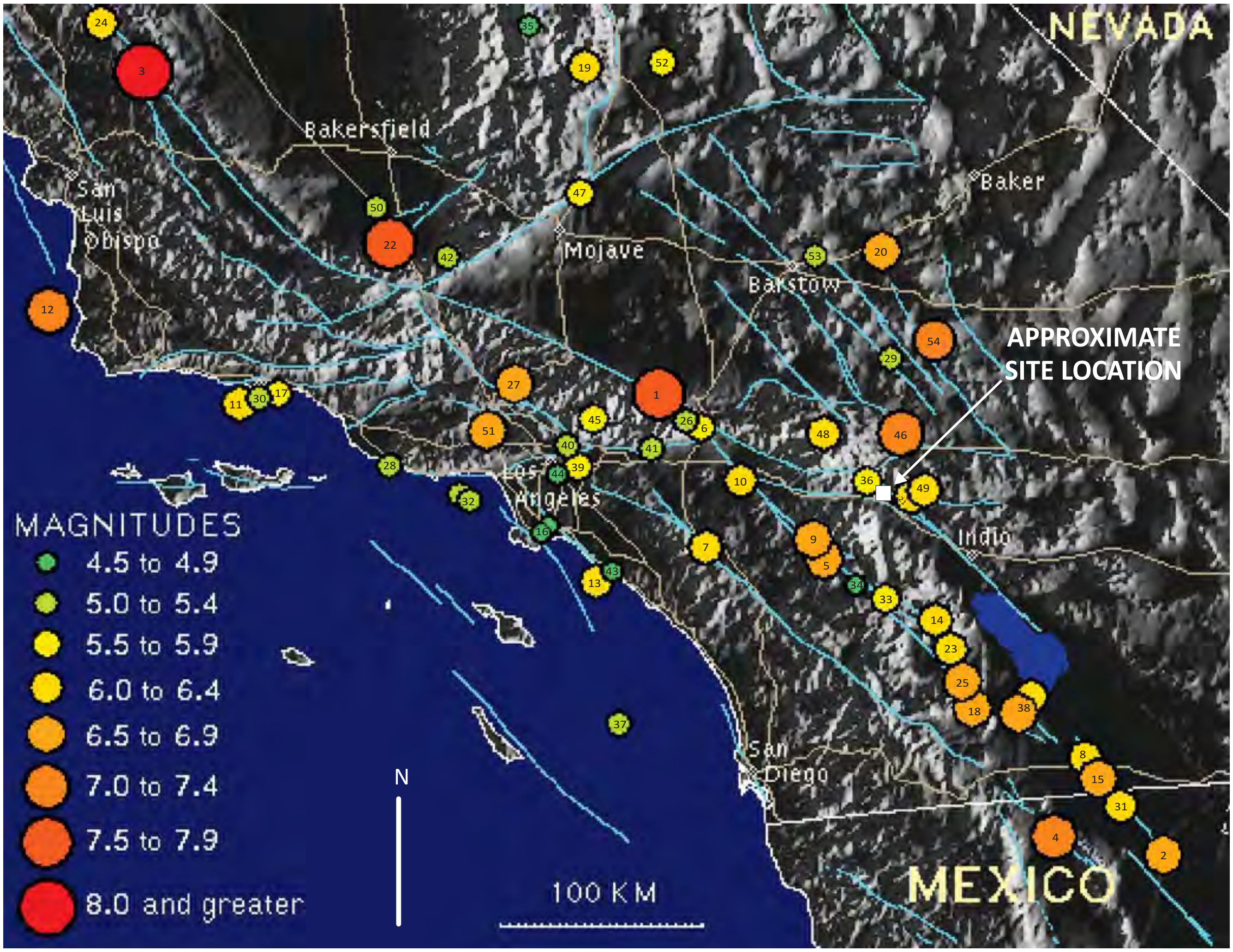
N

Plate 7
Regional Fault Map

Desert Hot Springs High School - CTE Building
65850 Pierson Boulevard
Desert Hot Springs, Riverside County, California

Earth Systems

9/6/2018	File No.: 302396-001



HISTORIC EARTHQUAKES AND EPICENTERS

1. 1812, WRIGHTWOOD
2. 1852, VOLCANO LAKE
3. 1857, FORT TEJON
4. 1892, LAGUNA SALADA
5. 1899, SAN JACINTO
6. 1899, CAJON PASS
7. 1910, ELSINORE
8. 1915, IMPERIAL VALLEY
9. 1918, SAN JACINTO
10. 1923, NORTH SAN JACINTO
11. 1925, SANTA BARBARA
12. 1927, LOMPOC
13. 1933, LONG BEACH
14. 1937, SAN JACINTO
15. 1940, IMPERIAL VALLEY
16. 1941, TORRANCE-GARDENA
17. 1941, SANTA BARBARA
18. 1942, FISH CREEK MOUNTAINS
19. 1946, WALKER PASS
20. 1947, MANIX
21. 1948, DESERT HOT SPRINGS
22. 1952, KERN COUNTY
23. 1954, SAN JACINTO
24. 1966, PARKFIELD
25. 1968, BORREGO MOUNTAINS
26. 1970, LYTLE CREEK
27. 1971, SAN FERNANDO
28. 1973, POINT MAGU
29. 1975, GALWAY LAKE
30. 1978, SANTA BARBARA
31. 1979, IMPERIAL VALLEY
32. 1979, MALIBU
33. 1980, WHITE WASH
34. 1982, ANZA GAP
35. 1983, DURRWOOD MEADOWS SWARM
36. 1986, NORTH PALM SPRINGS
37. 1986, OCEANSIDE
38. 1987, ELMORE RANCH & SUPERSTITION HILLS
39. 1987, WHITTIER NARROWS
40. 1988, PASADENA
41. 1988, UPLAND
42. 1988, TEJON RANCH
43. 1989, NEWPORT BEACH
44. 1989, MONTEBELLO
45. 1991, SIERRA MADRE
46. 1992, LANDERS
47. 1992, MOJAVE
48. 1992, BIG BEAR
49. 1992, JOSHUA TREE
50. 1993, WHEELER RIDGE
51. 1994, NORTHRIDGE
52. 1995, RIDGECREST
53. 1997, CALICO
54. 1999, HECTOR MINE

MAP SHOWING LOCATIONS OF SIGNIFICANT HISTORICAL EARTHQUAKES IN SOUTHERN CALIFORNIA FROM 1812 TO 2000

SOURCE: SOUTHERN CALIFORNIA EARTHQUAKE CENTER, WEB PAGE, 2000

Plate 8
Earthquake Epicenter Map

Desert Hot Springs High School - CTE Building
65850 Pierson Boulevard
Desert Hot Springs, Riverside County, California

Earth Systems

9/6/2018	File No.: 302396-001

Table A-1
Fault Parameters

Fault Section Name	Distance (miles)	Distance (km)	Avg Dip Angle (deg.)	Avg Dip Direction (deg.)	Avg Rake (deg.)	Trace Length (km)	Fault Type	**Mean Mag**	Mean Return Interval (years)	Slip Rate (mm/yr)
San Andreas, (North Branch, Mill Creek)	0.4	0.7	76	204	180	106	A	**7.5**	110	17
Mission Creek	4.1	6.6	65	5	180	31	B'	**6.9**		
San Andreas (San Gorgonio Pass-Garnet HIll)	5.1	8.3	58	20	180	56	A	**7.6**	219	10
Burnt Mtn	7.2	11.5	67	265	180	21	B	**6.7**		0.6
Pinto Mtn	8.5	13.7	90	175	0	74	B	**7.2**		2.5
Eureka Peak	10.1	16.2	90	75	180	19	B	**6.6**		0.6
Joshua Tree (Seismicity)	11.5	18.5	90	271	na	17	B'	**6.5**		
Blue Cut	15.0	24.2	90	177	na	79	B'	**7.1**		
San Gorgonio Pass	15.1	24.3	60	11	na	29	B'	**6.9**		
Landers	15.1	24.3	90	60	180	95	B	**7.4**		0.6
San Andreas (San Bernardino S)	17.3	27.8	90	210	180	43	A	**7.6**	150	16
San Andreas (Coachella) rev	19.8	31.8	90	224	180	69	A	**7.2**	69	20
So Emerson-Copper Mtn	22.9	36.9	90	51	180	54	B	**7.0**		0.6
North Frontal (East)	23.9	38.4	41	187	90	27	B	**6.9**		0.5
Johnson Valley (No)	24.3	39.1	90	51	180	35	B	**6.8**		0.6
Calico-Hidalgo	25.5	41.0	90	52	180	117	B	**7.4**		1.8
San Jacinto (Anza) rev	26.3	42.3	90	216	180	46	A	**7.6**	151	18
San Jacinto (San Jacinto Valley, stepover)	26.3	42.4	90	224	180	24	A	**7.4**	199	9
Lenwood-Lockhart-Old Woman Springs	27.5	44.3	90	43	180	145	B	**7.5**		0.9
San Jacinto (Anza, stepover)	27.6	44.5	90	224	180	25	A	**7.6**	151	9
San Jacinto (Stepovers Combined)	27.6	44.5	90	229	180	25	B'	**6.7**		
Helendale-So Lockhart	30.1	48.4	90	51	180	114	B	**7.4**		0.6
Pisgah-Bullion Mtn-Mesquite Lk	30.9	49.7	90	60	180	88	B	**7.3**		0.8
San Jacinto (Clark) rev	32.8	52.8	90	214	180	47	A	**7.6**	211	14
San Jacinto (San Jacinto Valley) rev	33.0	53.1	90	223	180	18	A	**7.4**	199	18
North Frontal (West)	33.8	54.5	49	171	90	50	B	**7.2**		1
San Jacinto (Coyote Creek)	34.3	55.1	90	223	180	43	A	**7.3**	259	4
Hector Mine	39.3	63.2	90	246	na	28	B'	**6.7**		
San Jacinto (San Bernardino)	41.3	66.5	90	225	180	45	A	**7.4**	205	6
San Andreas (San Bernardino N)	42.2	68.0	90	212	180	35	A	**7.5**	103	22
Cleghorn	44.9	72.3	90	187	0	25	B	**6.7**		3
Earthquake Valley (No Extension)	45.0	72.4	90	221	180	33	B'	**6.9**		
Elsinore (Temecula) rev	48.1	77.4	90	230	180	40	A	**7.4**	431	5
Elsinore (Glen Ivy stepover)	49.3	79.3	90	216	180	11	A	**7.1**	322	2.5
Elsinore (Stepovers Combined)	50.1	80.6	90	224	180	12	B'	**6.3**		
Elsinore (Temecula stepover)	50.1	80.6	90	212	180	12	A	**7.6**	725	2.5
Ludlow	50.2	80.7	90	239	na	70	B'	**7.0**		
Elsinore (Julian)	50.3	80.9	84	36	180	75	A	**7.6**	725	3
Fontana (Seismicity)	52.4	84.4	80	313	na	24	B'	**6.7**		
Elsinore (Glen Ivy) rev	52.7	84.7	90	218	180	26	A	**7.0**	222	5

Reference: USGS OFR 2007-1437 (CGS SP 203) Based on Site Coordinates of 33.96457 Latitude, -116.51847 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.

Site Coordinates: 33.965 N 116.518 W

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

Day	*Year*	*Epicenter* *Latitude* (Degrees)	*Longitude*	*Distance from Site (mi)*	*Magnitude* M_W
7/8	1986	34.00	116.61	5.8	**6.0**
4/23	1992	33.96	116.32	11.4	**6.2**
6/29	1992	34.10	116.40	11.5	**5.7**
6/28	1992	34.13	116.41	13.0	**5.8**
2/7	1889	34.10	116.70	14.0	**5.6**
6/28	1992	34.12	116.32	15.6	**5.7**
12/4	1948	34.00	116.23	16.7	**6.0**
6/28	1992	34.20	116.44	16.9	**7.3**
6/28	1992	34.16	116.85	23.3	**5.5**
6/28	1992	34.20	116.83	24.1	**6.5**
3/15	1979	34.33	116.44	25.6	**5.5**
1/16	1930	34.20	116.90	27.2	**5.5**
6/6	1918	33.60	116.70	27.2	**5.5**
10/2	1928	33.60	116.70	27.2	**5.5**
11/22	1880	34.00	117.00	27.7	**5.5**
12/19	1880	34.00	117.00	27.7	**5.9**
4/3	1926	34.00	116.00	29.8	**5.5**
12/25	*1899	33.80	117.00	29.8	**6.7**
4/21	*1918	33.75	117.00	31.3	**6.8**
4/11	1910	33.50	116.50	32.1	**5.8**
3/25	1937	33.46	116.44	35.1	**5.6**
10/16	1999	34.24	117.04	35.4	**5.6**
9/20	*1907	34.20	117.10	37.0	**5.8**
2/9	*1890	33.40	116.30	41.0	**6.8**
7/23	*1923	34.00	117.25	42.0	**6.2**
10/16	1999	34.59	116.27	45.5	**7.1**
5/2	1949	33.99	115.67	48.6	**5.7**
10/16	1999	34.68	116.29	51.1	**5.8**
12/16	1858	34.20	117.40	53.0	**6.0**
7/22	1899	34.20	117.40	53.0	**5.9**
3/19	1954	33.29	116.07	53.2	**6.4**
5/15	1910	33.70	117.40	53.8	**6.0**
5/28	*1892	33.20	116.20	55.9	**6.5**
9/30	1916	33.20	116.10	58.0	**5.7**
6/14	1892	34.20	117.50	58.4	**5.5**
4/9	1968	33.17	116.09	60.1	**6.6**
9/21	1856	33.10	116.70	60.6	**5.5**
7/22	1899	34.30	117.50	60.7	**6.4**
7/30	1894	34.30	117.60	66.0	**6.2**
2/28	1990	34.14	117.70	68.7	**5.7**

From full earthquake catalog in USGS OFR 2007-1437h. For events with an asterisk, alternate solutions are given in the OFR.

Table A-3 - Spectral Response Values

Probabilistic and Deterministic Response Spectra for MCE compared to Code Spectra

for 5% Viscous Damping Ratio

Natural Period T (seconds)	GeoMean Probab. 2% in 50 yr MCE Spectrum (1) 2475-yr	Max Rotated Probab. 2% in 50 yr MCEr (2) 2475-yr	Max 84th Percentile Determ. MCE Spectrum (3)	Determ. Lower Limit MCE Spectrum (4)	Determ. MCE Spectrum (5) max(3,4)	**Site Specific MCE Spectrum** (6) min(2.5)	**2016CBC MCE Spectrum** (7)	Site Specific Design Spectrum (8) 2/3*(6)*	2016 CBC Design Spectrum (9) 2/3*(7)
0.00	0.893	0.964	1.070	0.600	1.070	**0.964**	**1.006**	0.643	0.670
0.05	1.209	1.304	1.260	0.975	1.260	**1.260**	**1.709**	0.912	1.140
0.10	1.524	1.644	1.658	1.350	1.658	**1.644**	**2.413**	1.287	1.609
0.15	1.723	1.860	1.964	1.500	1.964	**1.860**	**2.514**	1.341	1.676
0.20	1.923	2.075	2.115	1.500	2.115	**2.075**	**2.514**	1.383	1.676
0.30	1.985	2.134	2.316	1.500	2.316	**2.134**	**2.514**	1.423	1.676
0.40	1.914	2.143	2.378	1.500	2.378	**2.143**	**2.514**	1.429	1.676
0.50	1.843	2.144	2.477	1.500	2.477	**2.144**	**2.514**	1.430	1.676
0.75	1.616	1.940	2.559	1.200	2.559	**1.940**	**1.796**	1.293	1.197
1.00	1.390	1.718	2.366	0.900	2.366	**1.718**	**1.347**	1.145	0.898
1.50	1.128	1.394	2.116	0.600	2.116	**1.394**	**0.898**	0.930	0.599
2.00	0.866	1.071	1.798	0.450	1.798	**1.071**	**0.674**	0.714	0.449

Crs: 0.981 * > 80% of (9)

Cr1: 0.951

Probabilistic Spectrum from 2008 USGS Ground Motion Mapping Program adjusted for site conditions and maximum rotated component of ground motion using NGA, Column 2 has risk coefficients Cr applied.

Reference: ASCE 7-10, Chapters 21.2, 21.3, 21.4 and 11.4

Mapped MCE Acceleration Values			Site Coefficients		**Site-Specific Design Acceleration Values**		
PGA	0.924	g	F_{PGA}	1.00	**PGA_M**	**0.924**	**g**
Ss	2.514	g	F_a	1.00	**S_{DS}**	**1.383**	**g**
S_1	0.898	g	F_v	1.50	**S_{D1}**	**1.428**	**g**

Spectral Amplification Factor for different viscous damping, D (%):

0.5%	2%	10%	20%
1.50	1.23	0.83	0.67

1 g = 980.6 cm/sec^2 =32.2 ft/sec^2

PSV (ft/sec) = 32.2(Sa)T/(2π)

Key: Probab. = Probabilistic, Determ. = Deterministic, MCE = Maximum Considered Earthquake

Table A-4 - Spectral Response Values
Deterministic NGA Response Spectra for Largest Median Earthquake Ground Motion

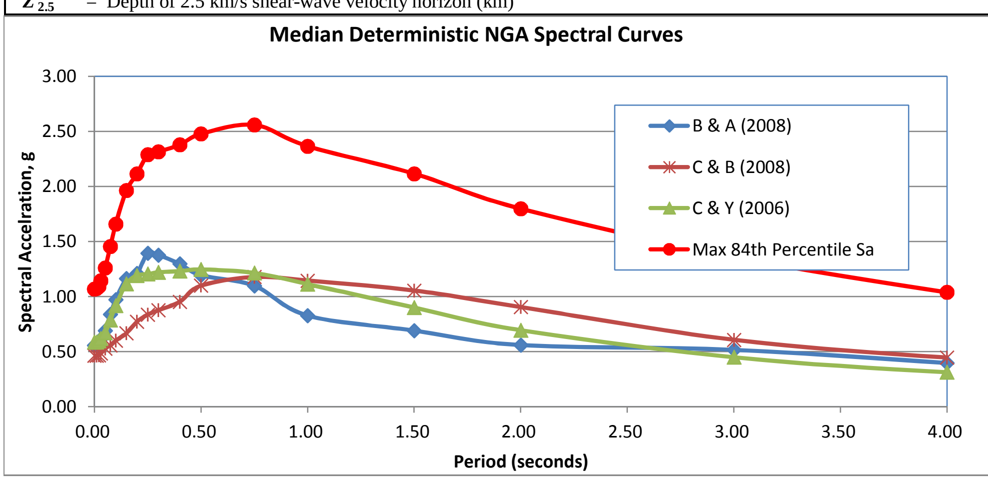
Average of NGA: Boore & Atkinson (2008), Campbell & Borzognia (2008), and Chiou and Youngs (2006)

Mean Spectra Response from Attentuation Relationships

	B & A (2008)		C & B (2008)		C & Y (2006)		Average of B&A, C&B, C&Y		
Input Variables	Period (sec)	Mean Sa (g)	Period (sec)	Mean Sa (g)	Period (sec)	Mean Sa (g)	**Period (sec)**	Mean Sa (g)	**Max 84th Percentile Sa (g)**
	Use:	1	Use:	1	Use:	1			
M	0.00	0.56	0.00	0.46	0.00	0.58	**0.00**	0.53	**1.07**
8.20	0.01	0.56	0.01	0.46	0.01	0.58	**0.01**	0.54	**1.07**
R_{RUP}	0.02	0.57	0.02	0.47	0.02	0.60	**0.02**	0.55	**1.09**
0.71	0.03	0.62	0.03	0.48	0.03	0.62	**0.03**	0.57	**1.14**
R_{JB}	0.05	0.69	0.05	0.53	0.05	0.67	**0.05**	0.63	**1.26**
0.71	0.075	0.84	0.075	0.56	0.075	0.79	**0.075**	0.73	**1.45**
V_{S30}	0.10	0.97	0.10	0.60	0.10	0.92	**0.10**	0.83	**1.66**
278	0.15	1.16	0.15	0.67	0.15	1.11	**0.15**	0.98	**1.96**
F_{RV}	0.20	1.21	0.20	0.77	0.20	1.19	**0.20**	1.06	**2.12**
0	0.25	1.39	0.25	0.84	0.25	1.20	**0.25**	1.14	**2.29**
F_{NM}	0.30	1.38	0.30	0.88	0.30	1.22	**0.30**	1.16	**2.32**
0	0.40	1.30	0.40	0.95	0.40	1.23	**0.40**	1.16	**2.38**
W	0.50	1.19	0.50	1.10	0.50	1.25	**0.50**	1.18	**2.48**
15.00	0.75	1.10	0.75	1.18	0.75	1.21	**0.75**	1.16	**2.56**
Z_{TOR}	1.00	0.83	1.00	1.15	1.00	1.11	**1.00**	1.03	**2.37**
0.00	1.50	0.69	1.50	1.05	1.50	0.90	**1.50**	0.88	**2.12**
$Z_{2.5}$	2.00	0.56	2.00	0.90	2.00	0.69	**2.00**	0.72	**1.80**
2.00	3.00	0.52	3.00	0.61	3.00	0.45	**3.00**	0.52	**1.36**
d	4.00	0.40	4.00	0.45	4.00	0.31	**4.00**	0.38	**1.04**
76	5.00	0.28	5.00	0.38	5.00	0.23	**5.00**	0.29	**0.79**
	7.50	0.21	7.50	0.29	7.50	0.11	**7.50**	0.20	**0.55**
	10.00	0.09	10.00	0.24	10.00	0.06	**10.00**	0.13	**0.35**

Definition of Parameters

- **M** = Moment magnitude
- R_{RUP} = Closest distance to coseismic rupture (km)
- R_{JB} = Closest distance to surface projection of coseismic rupture (km)
- F_{RV} = Reverse-faulting factor: 0 for strike slip, normal, normal-oblique; 1 for reverse, reverse-oblique and thrust
- F_{NM} = Normal-faulting factor: 0 for strike slip, reverse, reverse-oblique and thrust; 1 for normal and normal-oblique
- Z_{TOR} = Depth to top of coseismic rupture (km)
- **d** = Average dip of rupture plane (degrees)
- V_{S30} = Average shear-wave velocity in top 30m of site profile
- **W** = Rupture Width (km)
- $Z_{2.5}$ = Depth of 2.5 km/s shear-wave velocity horizon (km)



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
79-811B Country Club Drive, Bermuda Dunes, CA 92201

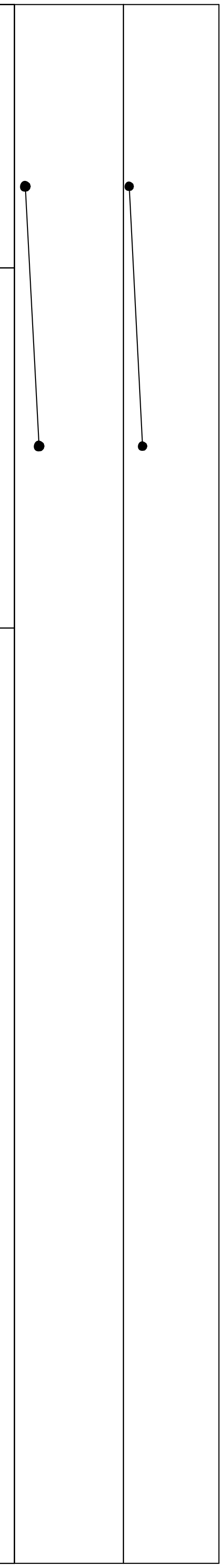
Boring No. B-1	Drilling Date: August 3, 2018
Project Name: Desert Hot Springs High School CTE Building	Drilling Method: Mobile B-61 w/autohammer
Project Number 302396-001	Drill Type: 8" HSA
Boring Location: See Plate 3	Logged By: J. Geisiner

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				SP			SAND: dark brown, loose, slightly moist, fine to coarse grained sand, with gravel to 1" and some small cobbles, fill, grass turf on surface
							medium dense
5	MOD Calif.	23,8,7		SP	106	4	SAND: dark brown, loose, slightly moist, fine to coarse grained sand, with gravel to 1" and some small cobbles, native
10	MOD Calif.	8,12,22			120	4	medium to coarse grained sand, with gravel, cobbles and boulders, slight caving
15							
20							
25							Boring terminated at 12 feet, refusal on boulders Backfilled with cuttings No groundwater encountered
30							

Graphic Trend
Blow Count Dry Density



Boring No. B-2

Project Name: Desert Hot Springs High School CTE Building
Project Number 302396-001
Boring Location: See Plate 3

Drilling Date: August 3, 2018
Drilling Method: Mobile B-61 w/autohammer
Drill Type: 8" HSA
Logged By: J. Geisiner

Depth (Ft.)	Sample Type: Bulk	SPT	MOD Calif.	Penetration Resistance (Blows/6")	Symbol	USCS	Dry Density (pcf)	Moisture Content (%)	Description of Units	Graphic Trend: Blow Count	Dry Density
0						SP-SM			SAND WITH SILT: dark brown, dense, moist, fine to coarse grained sand, fill, grass turf on surface		
			■	13,18,50			120	7			
5											
10											
15											
20											
25									Boring terminated at 5 feet, refusal on boulders Backfilled with cuttings No groundwater encountered		
30											

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

Boring No. B-3

Project Name: Desert Hot Springs High School CTE Building
Project Number 302396-001
Boring Location: See Plate 3

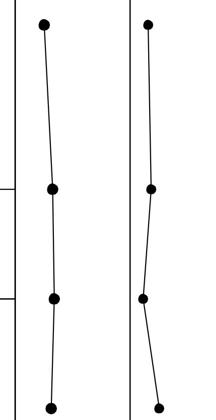
Drilling Date: August 3, 2018
Drilling Method: Mobile B-61 w/autohammer
Drill Type: 8" HSA
Logged By: J. Geisiner

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			SP-SM			SAND WITH SILT: dark brown, medium dense, moist, fine to coarse grained sand with trace gravel and cobbles, fill, grass turf on surface
2	MOD Calif.	11,14,24		118	8	
5	MOD Calif.	10,15,34	SP-SM	121	8	SAND WITH SILT: brown, dense, moist, fine to coarse grained sand, with gravel and cobbles, native
7	MOD Calif.	18,27,24	SW	113	3	GRAVELLY SAND: brown, dense, damp, medium to coarse grained sand, with cobbles and boulders, slight caving
9	MOD Calif.	12,19,28		129	3	

Graphic Trend
Blow Count Dry Density



Boring terminated at 12 feet, refusal on boulders

Backfilled with cuttings

No groundwater encountered

Depth (Ft.): 0, 5, 10, 15, 20, 25, 30

Boring No. B-4

Project Name: Desert Hot Springs High School CTE Building
Project Number 302396-001
Boring Location: See Plate 3

Drilling Date: August 3, 2018
Drilling Method: Mobile B-61 w/autohammer
Drill Type: 8" HSA
Logged By: J. Geisiner

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						SP-SM			SAND WITH SILT: dark brown, very dense, fine to coarse grained sand, with gravel to 3", traces of cobbles, fill, grass turf at surface		
2.5			■	21,50/5"						●	
5			■	25,39,36						●	
6						SP-SM			SAND WITH SILT: dark brown, very dense, fine to coarse grained sand, with gravel to 3", traces of cobbles, native traces of boulders		
7.5			■	14,50/5"						●	
10											
15											
20											
25									Boring terminated at 10 feet, refusal on boulders Backfilled with cuttings No groundwater encountered		
30											

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

Boring No. B-5

Project Name: Desert Hot Springs High School CTE Building
Project Number 302396-001
Boring Location: See Plate 3

Drilling Date: August 3, 2018
Drilling Method: Mobile B-61 w/autohammer
Drill Type: 8" HSA
Logged By: J. Geisiner

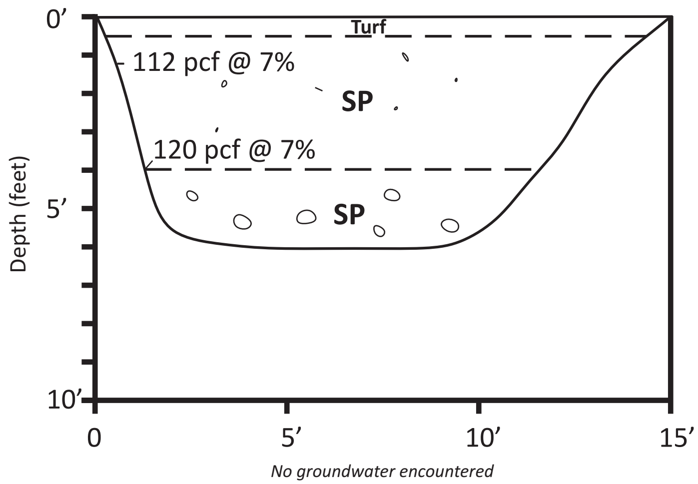
Depth (Ft.)	Sample Type: Bulk	Sample Type: SPT	Sample Type: MOD Calif.	Penetration Resistance (Blows/6")	Symbol	USCS	Dry Density (pcf)	Moisture Content (%)	Description of Units Note: The stratification lines shown represent the approximate boundary between soil and/or rock types and the transition may be gradational.	Graphic Trend: Blow Count	Graphic Trend: Dry Density
0						SP-SM			SAND WITH SILT: dark brown, fine to coarse grained sand, with gravel to 3", fill, grass turf at surface		
5						SP-SM			SAND WITH SILT: dark brown, fine to coarse grained sand, with gravel to 3", cobbles and boulders, slight caving, native		
10											
15											
20											
25									Boring terminated at 12 feet, refusal on boulders Backfilled with cuttings No groundwater encountered		
30											

Boring No. B-6

Project Name: Desert Hot Springs High School CTE Building
Project Number 302396-001
Boring Location: See Plate 3

Drilling Date: August 3, 2018
Drilling Method: Mobile B-61 w/autohammer
Drill Type: 8" HSA
Logged By: J. Geisiner

Depth (Ft.)	Sample Type: Bulk	Sample Type: SPT	Sample Type: MOD Calif.	Penetration Resistance (Blows/6")	Symbol	USCS	Dry Density (pcf)	Moisture Content (%)	Description of Units (Note: The stratification lines shown represent the approximate boundary between soil and/or rock types and the transition may be gradational.)	Graphic Trend: Blow Count	Graphic Trend: Dry Density
0						SP			SAND: dark brown, damp, dense, fine to coarse grained sand, with traces of gravel and cobbles, fill, grass turf at surface		
5									possible native soil at 6' due to cobble and boulder hard drilling		
10											
15											
20											
25									Boring terminated at 7 feet, refusal on cobbles and boulders Backfilled with cuttings No groundwater encountered		
30											



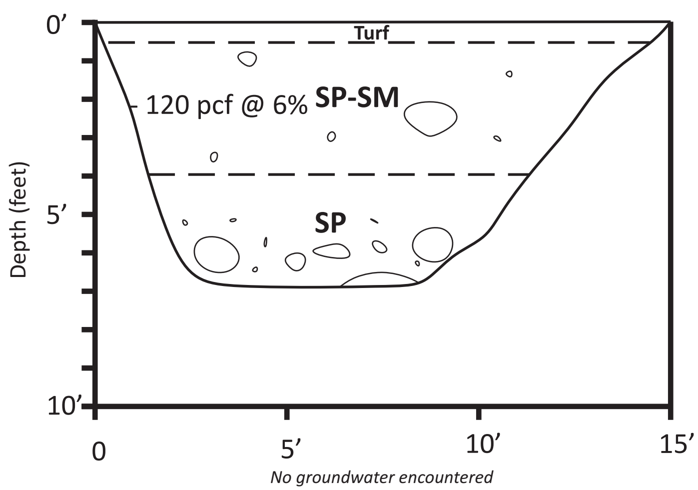
T-1

0 - 0.5': Turf

0.5' - 4.5': SP: Sand, Dark Brown, Dense, Moist, Fine to Coarse Grained Sand, with Gravel to ½" (Fill)

4.5' - 6': SP: Sand, Brown, Dense, Moist, Fine to Coarse Grained Sand, with Gravel to 1", Trace Cobbles to 8"(Native)

Slight caving
No Bedrock, Groundwater, or Refusal



T-2

0 - 0.5': Turf

0.5' - 4': SP-SM: Sand with Silt, Dark Brown, Dense, Moist, Fine to Coarse Grained Sand, with Gravel to ¼", Trace Cobbles to 6" (Fill)

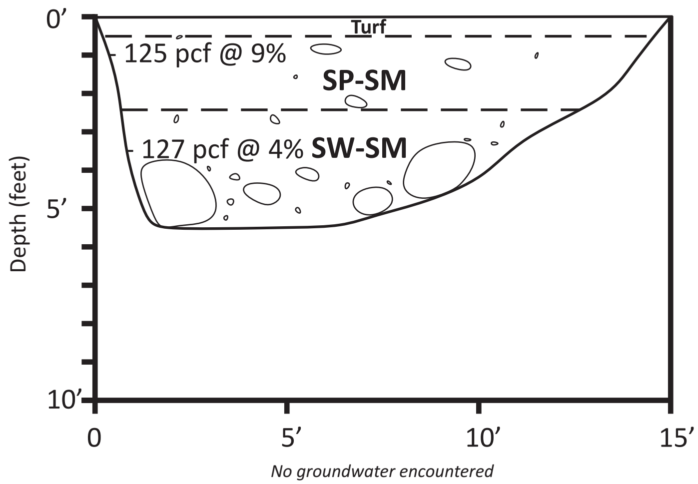
4 '- 7': SP: Sand, Dark Brown, Medium Dense, Moist, Fine to Coarse Grained Sand, with Gravel to 1", Cobbles, and Traces of Boulders (Native)

Slight Caving 4' to 7'
No Bedrock, Groundwater, or Refusal

Horizontal and Vertical
Scale: 1" = 5'

0 5' 10'

Test Pit Logs	
Desert Hot Springs High School - CTE Building 65850 Pierson Boulevard Desert Hot Springs, Riverside County, California	
Earth Systems	
9/6/2018	File No.: 302396-001



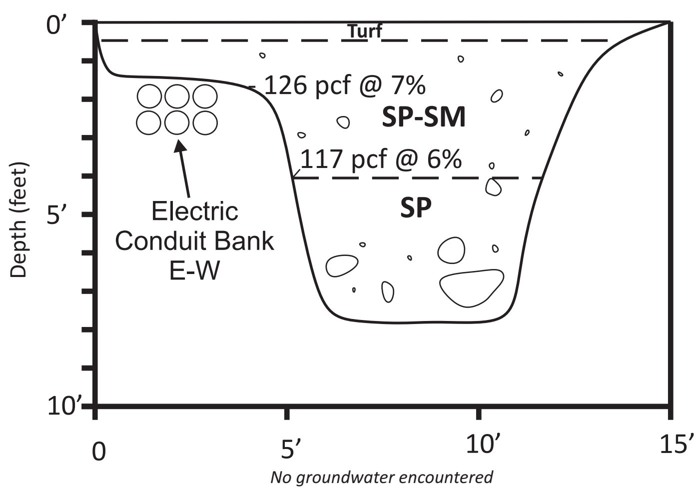
T-3

0 - 0.5': Turf

0.5' - 2.5': SP-SM: Sand with Silt, Dark Brown, Moist, Fine to Coarse Grained Sand with Trace Gravel and Cobbles to 6" (Fill)

2.5'- 5.5': SW-SM: Sand with Silt, Brown, Moist, Fine to Coarse Grained Sand, with Cobbles and Boulders to 2', Moderate Caving (Native)

Moderate Caving 2' - 5.5'
No Bedrock, Groundwater, or Refusal



T-4

0 - 0.5': Turf

0.5' - 4': SP-SM: Sand with Silt, Dark Brown, Moist, Fine to Coarse Grained Sand, with Gravel and Trace Cobbles to 6" (Fill)

4' - 8': SP: Sand, Dark Brown, Medium Dense, Moist, Fine to Coarse Grained Sand, with Gravel, Cobbles, and Trace Boulders, Slight Caving (Native)

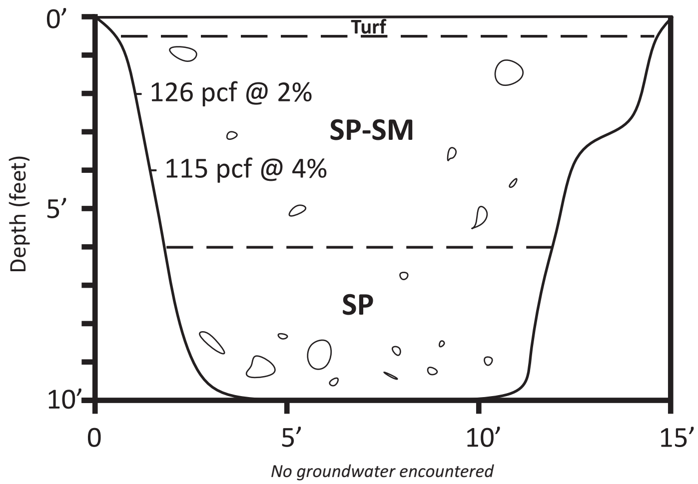
Slight Caving 5'-8'
No Bedrock, Groundwater, or Refusal

Horizontal and Vertical
Scale: 1" = 5'

0 5' 10'

Test Pit Logs	
Desert Hot Springs High School - CTE Building 65850 Pierson Boulevard Desert Hot Springs, Riverside County, California	
Earth Systems	
9/6/2018	File No.: 302396-001

T-5



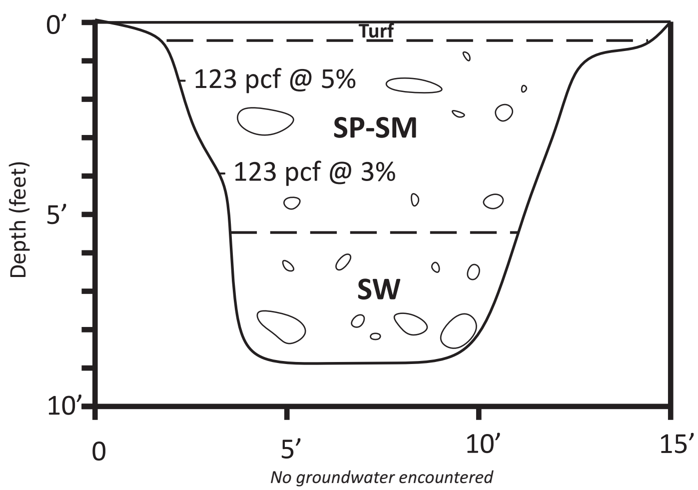
0 - 0.5': Turf

0.5' - 6': SP-SM: Sand with Silt, Dark Brown, Damp to Moist, Fine to Coarse Grained Sand, with Traces of Gravel 1" (Fill)

6' - 10': SP: Sand, Brown, Damp, Fine to Coarse Grained Sand, with Gravel to 3" and Trace of Cobbles to 6" (Native)

No Bedrock, Groundwater, Refusal, or Caving

T-6



0 - 0.5': Turf

0.5' - 5.5': SP-SM: Sand with Silt, Dark Brown, Dense, Damp to Moist, Fine to Coarse Grained Sand, Trace Cobbles to 6" (Fill)

5.5' - 9': SW: Sand, Brown, Medium Dense, Moist, Fine to Coarse Grained Sand, with Trace of Cobbles, Slight Caving (Native)

Slight Caving 5'-9'
No Bedrock, Groundwater, or Refusal

Horizontal and Vertical
Scale: 1" = 5'

Test Pit Logs	
Desert Hot Springs High School - CTE Building 65850 Pierson Boulevard Desert Hot Springs, Riverside County, California	
Earth Systems	
9/6/2018	File No.: 302396-001

Boring No.	B-3	Project and Number	Desert Hot Springs Hig	302396-001
		ESSW Field Staff	JG	
		Drilling Company	Pac Drill	
		Drilling Method	8" HSA	HSA Inner Diameter 3"
		Site Latitude (North)	Decimal Degrees 33.9647	
		Site Longitude (West)	Decimal Degrees -116.5184	

Input	Calculation Results
Date Drilled: 8/3/2018	Ave. SPT N-value (blows/ft): 33 (Based on Upper 12 feet)
Hammer Weight (lbs): 140	Ave. Shear Wave Velocity (ft/sec): 913 (Based on Upper 12 feet)
Hammer Drop (inches): 30	Soil Profile Type (Site Class): D (Based on Upper 12 feet)
Hammer Efficiency (E_M): 68	Ave. Friction Angle (degrees): 35 (Based on Upper 12 feet)
Borehole Correction (Cb)*: 1	Estimated Shear Wave Velocity ** Based on Depth Less than 100' (ft/sec): 917 (Upper 100 feet)
*inside diameter of Hollow Stem Auger	Soil Profile Type (Site Class)**: D

Sampler Liner Correction (Cs)
1.2 Applied if SPT Sampler Used
1.0 Applied if Cal Sampler Used

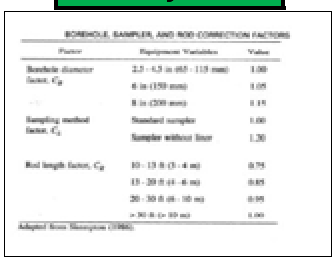
Rod Length Above Ground (ft)
3

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)

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

→ 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	38	c	5.0	21.64	18.55	28.85	266.46	873.98	34.11	0.17328	0.00572	0.146587	Medium Dense	Very Stiff
7.0	49	c	2.0	27.91	23.92	37.21	286.85	940.85	35.51	0.05375	0.00213	0.056321	Medium Dense	Very Stiff
10.0	51	c	3.0	29.04	24.90	38.73	290.19	951.83	35.74	0.07747	0.00315	0.083937	Medium Dense	Very Stiff
12.0	47	c	2.0	30.34	26.00	35.69	283.40	929.55	35.27	0.05604	0.00215	0.056699	Dense	Hard
		Total:	12.0	Feet				Total:		0.36054	0.01315	0.343544		

**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

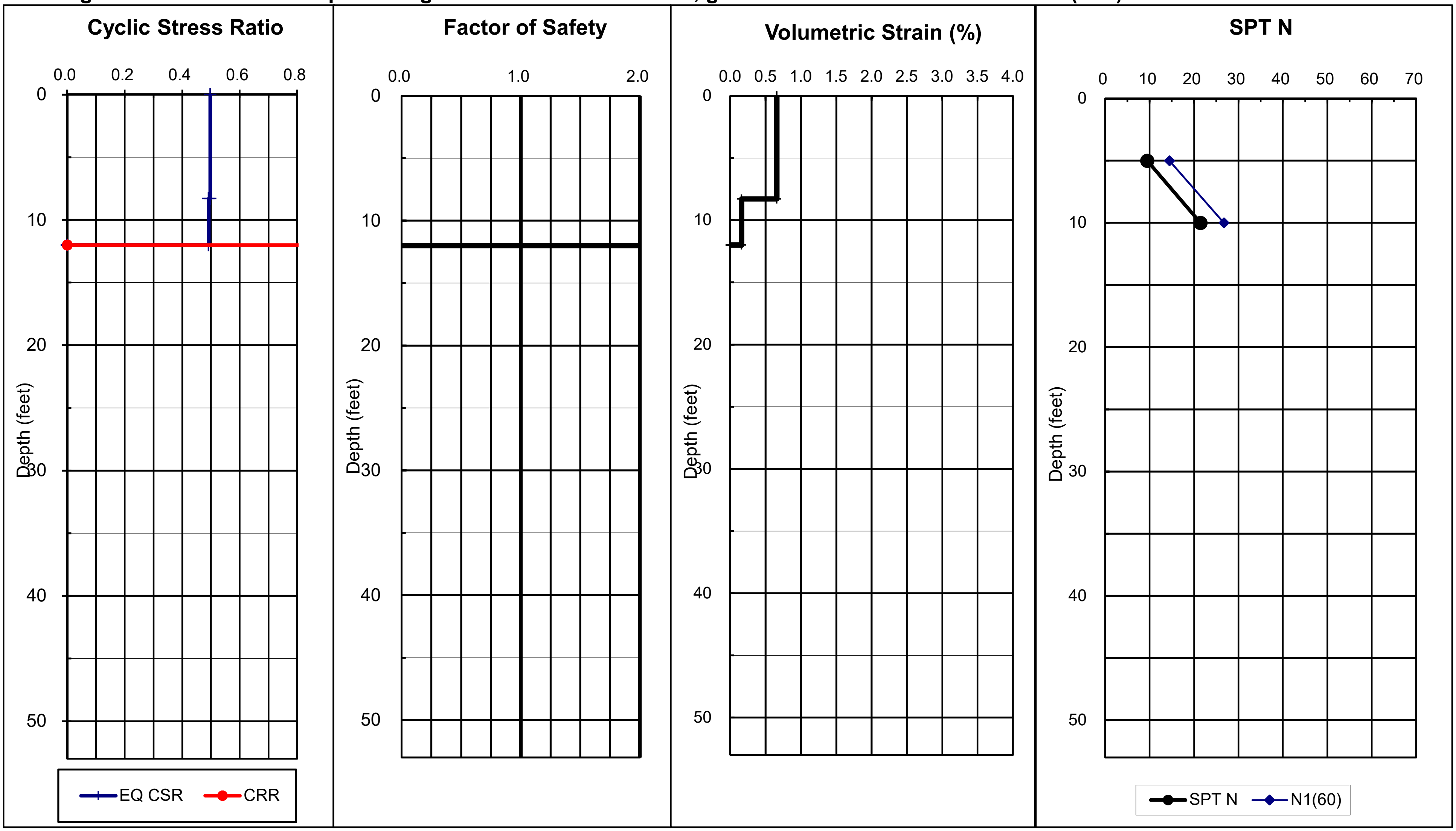
Spreadsheet Version 2.4, 2016: Prepared by Kevin L. Paul, PE, GE

EARTH SYSTEMS - EVALUATION OF LIQUEFACTION POTENTIAL AND INDUCED SUBSIDENCE

Desert Hot Springs High School CTE Building **Project No: 302396-001** 1996/1998 NCEER Method

Ground Compaction Remediated to 5 foot depth

Boring: B-1 **Earthquake Magnitude: 8.2** **PGA, g: 0.62** **Calc GWT (feet): 50**



Total Thickness of Liquefiable Layers: 0.0 feet **Estimated Total Ground Subsidence: 0.7 inches**

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

Project: Desert Hot Springs High School CTE Building
Job No: 302396-001
Date: 9/10/2018
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: 0.62 0.77
MSF: 0.80
GWT: 50.0 feet
Calc GWT: 50.0 feet
Remediate to: 5.0 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

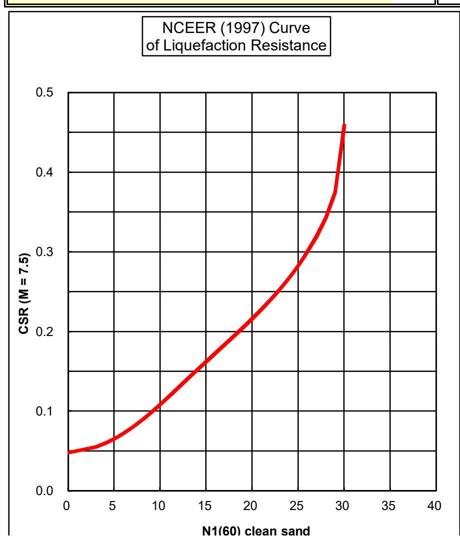
Total (ft) Liquefied Thickness: 0

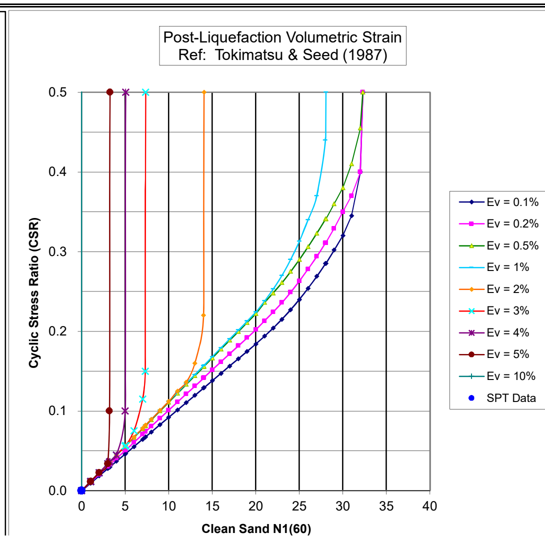
Total (in.) Induced Subsidence: 0.7
upper 50 ft

Required SF: 1.50
Threshold Acceler., g: #N/A **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)	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.)**	p (tsf)	G_{max} (tsf)	τ_{av} (tsf)	Shear Strain γ	Strain E_{15}	Strain Enc	**Dry Sand Subsidence (in.)**
								0.000																								
8.3	15	**9**	1	110	3	5.0	8.0	0.275	0.275	0.99	1.70	0.75	1.00	14.5	45	0.0	14.5	1.00	0.156	0.498	**Non-Liq.**	0.0	14.5	0.66	**0.65**	0.184	467	0.109	1.9E-03	2.7E-03	3.3E-03	**0.65**
12.0	34	**21**	1	124	3	10.0	13.0	0.562	0.562	0.98	1.37	0.76	1.00	26.7	62	0.0	26.7	1.00	0.314	0.492	**Non-Liq.**	0.0	26.7	0.16	**0.07**	0.376	820	0.220	9.5E-04	6.7E-04	8.1E-04	**0.07**
		50					3.0			1.00		0.75	1.10	0.0										######								





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

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

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

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

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

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

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)

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

Kσ = 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/Kσ

$CRR_{7.5}$ = (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))

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

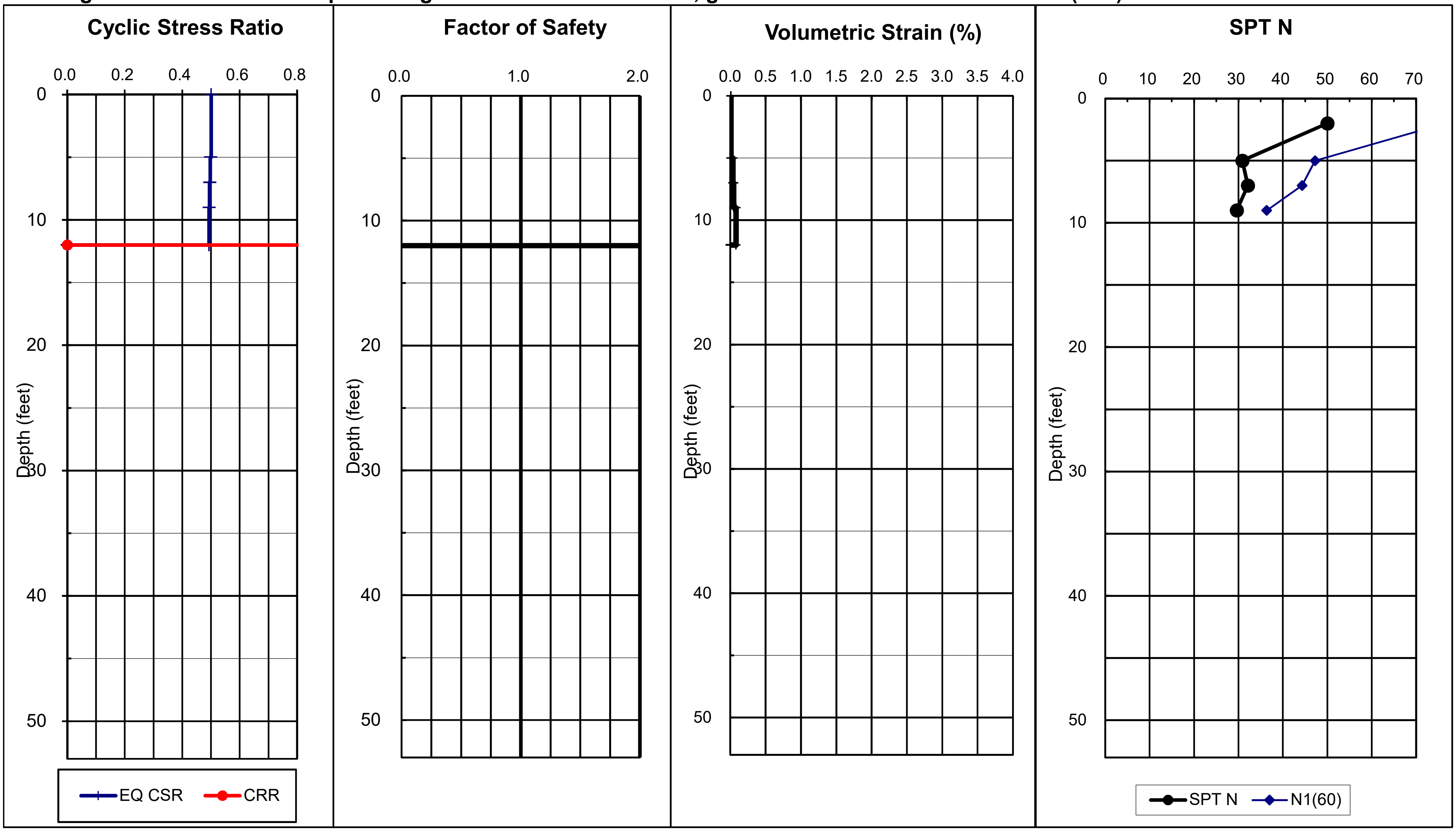
$SF = CRR_{7.5,1atm}/CSR^*$

EARTH SYSTEMS - EVALUATION OF LIQUEFACTION POTENTIAL AND INDUCED SUBSIDENCE

Desert Hot Springs High School CTE Building **Project No: 302396-001** 1996/1998 NCEER Method

Ground Compaction Remediated to 5 foot depth

Boring: B-3 **Earthquake Magnitude: 8.2** **PGA, g: 0.62** **Calc GWT (feet): 50**



Total Thickness of Liquefiable Layers: 0.0 feet **Estimated Total Ground Subsidence: 0.1 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: Desert Hot Springs High School CTE Building
Job No: 302396-001
Date: 9/10/2018
Boring: B-3 Data Set: 2

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: 0.62 0.77
MSF: 0.80
GWT: 50.0 feet
Calc GWT: 50.0 feet
Remediate to: 5.0 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: 0

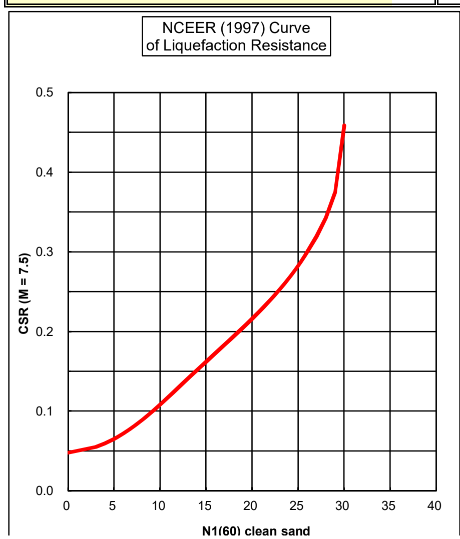
Total (in.) Induced Subsidence: 0.1
upper 50 ft

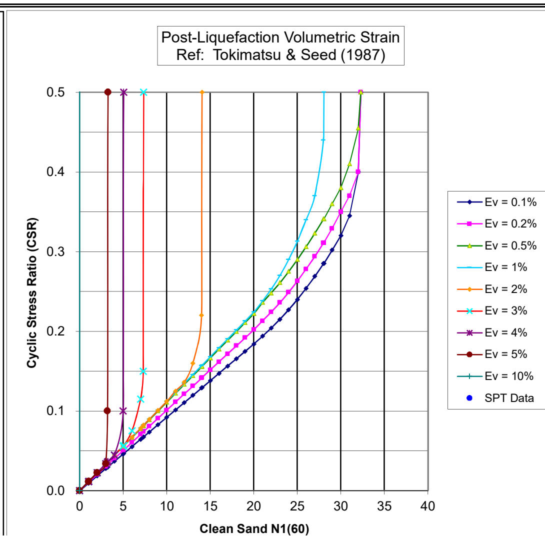
Required SF: 1.50
Minimum Calculated SF: #N/A

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)	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. $\Delta N_{1(60)}$	Post $N_{1(60)CS}$	Volumetric Strain (%)	Induced Subsidence (in.)	p (tsf)	G_{max} (tsf)	τ_{av} (tsf)	Shear Strain γ	Strain E_{15}	Strain Enc	Dry Sand Subsidence (in.)
								0.000																								
5.0	38	50	1	128	7	2.0	5.0	0.128	0.128	1.00	1.70	0.75	1.00	76.5	100	0.8	77.3	1.00	1.200	0.501	Non-Liq.	0.8	77.3	0.01	0.01	0.086	558	0.051	2.1E-04	4.2E-05	5.1E-05	0.01
7.0	49	31	1	131	7	5.0	8.0	0.320	0.320	0.99	1.70	0.75	1.00	47.2	82	0.5	47.8	1.00	1.200	0.498	Non-Liq.	0.5	47.8	0.04	0.01	0.214	751	0.127	4.5E-04	1.6E-04	1.9E-04	0.01
9.0	51	32	1	117	3	7.0	10.0	0.451	0.451	0.99	1.53	0.75	1.00	44.3	80	0.0	44.3	1.00	1.200	0.495	Non-Liq.	0.0	44.3	0.05	0.01	0.302	869	0.178	5.4E-04	2.1E-04	2.5E-04	0.01
12.0	47	30	1	132	3	9.0	12.0	0.568	0.568	0.98	1.36	0.75	1.00	36.4	72	0.0	36.4	1.00	1.200	0.493	Non-Liq.	0.0	36.4	0.08	0.03	0.381	914	0.223	7.0E-04	3.4E-04	4.1E-04	0.03





$N_{1(60)}$ = 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

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

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)

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

Kσ = 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/Kσ

$CRR_{7.5}$ = (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 = $N_{1(60)CS}$

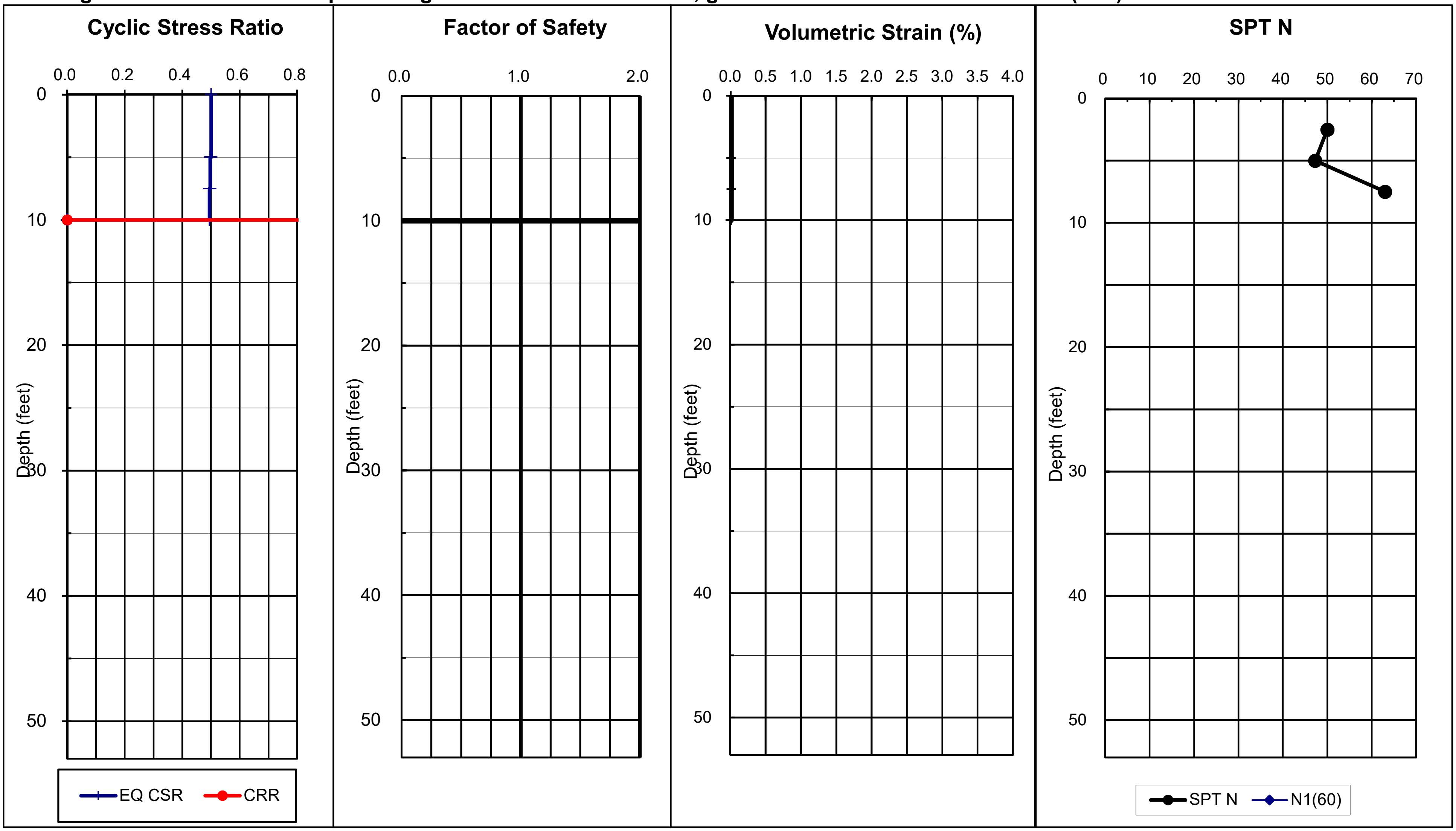
SF = $CRR_{7.5,1atm}$/CSR*

EARTH SYSTEMS - EVALUATION OF LIQUEFACTION POTENTIAL AND INDUCED SUBSIDENCE

Desert Hot Springs High School CTE Building **Project No: 302396-001** 1996/1998 NCEER Method

Ground Compaction Remediated to 5 foot depth

Boring: B-4 **Earthquake Magnitude: 8.2** **PGA, g: 0.62** **Calc GWT (feet): 50**



Total Thickness of Liquefiable Layers: 0.0 feet **Estimated Total Ground Subsidence: 0.0 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: Desert Hot Springs High School CTE Building
Job No: 302396-001
Date: 9/10/2018
Boring: B-4 Data Set: 3

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: 0.62 0.77
MSF: 0.80
GWT: 50.0 feet
Calc GWT: 50.0 feet
Remediate to: 5.0 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

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

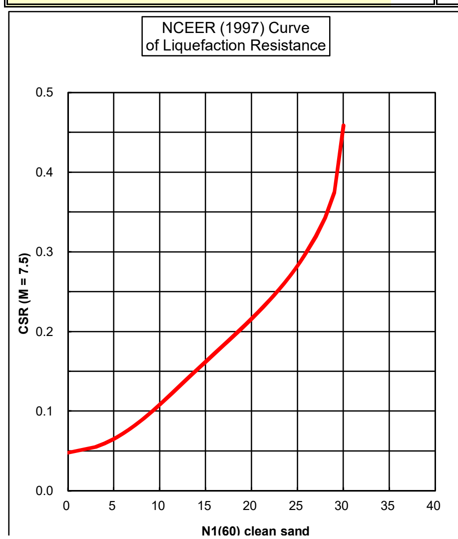
Total (ft) Liquefied Thickness	**Total (in.) Induced Subsidence**
0	**0.0**

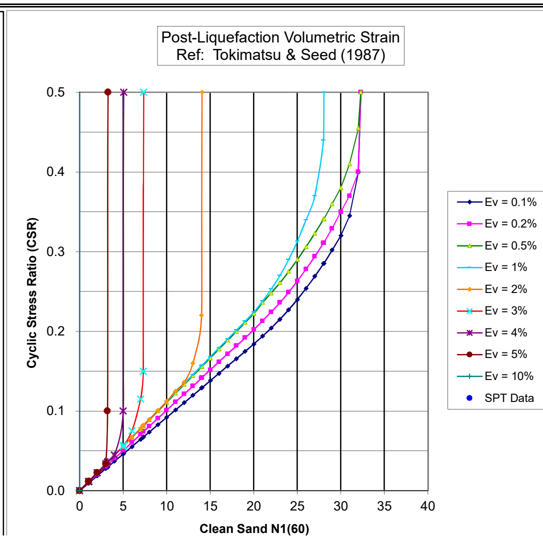
upper 50 ft

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)	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.)**	p (tsf)	G_{max} (tsf)	τ_{av} (tsf)	Shear Strain γ	Strain E_{15}	Strain Enc	**Dry Sand Subsidence (in.)**
								0.000																								
5.0	100	50	1	119	7	2.5	5.5	0.149	0.149	1.00	1.70	0.75	1.00	76.5	100	0.8	77.3	1.00	1.200	0.501	**Non-Liq.**	0.8	77.3	0.01	**0.01**	0.100	601	0.059	2.3E-04	4.5E-05	5.4E-05	**0.01**
7.5	75	47	1	125	7	5.0	8.0	0.298	0.298	0.99	1.70	0.75	1.00	72.3	100	0.7	73.0	1.00	1.200	0.498	**Non-Liq.**	0.7	73.0	0.02	**0.00**	0.199	834	0.118	3.0E-04	6.4E-05	7.7E-05	**0.00**
10.0	100	63	1	125	7	7.5	10.5	0.454	0.454	0.98	1.53	0.75	1.00	86.6	100	0.9	87.4	1.00	1.200	0.495	**Non-Liq.**	0.9	87.4	0.01	**0.00**	0.304	1,094	0.179	3.1E-04	5.3E-05	6.3E-05	**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))$^{0.5}$))

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

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pa = 1 atm = 101 KPa = 1.058 tsf

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τ_{av} = 0.65*PGA*po*rd

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

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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

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)

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

Kσ = 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/Kσ

$CRR_{7.5}$ = (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))

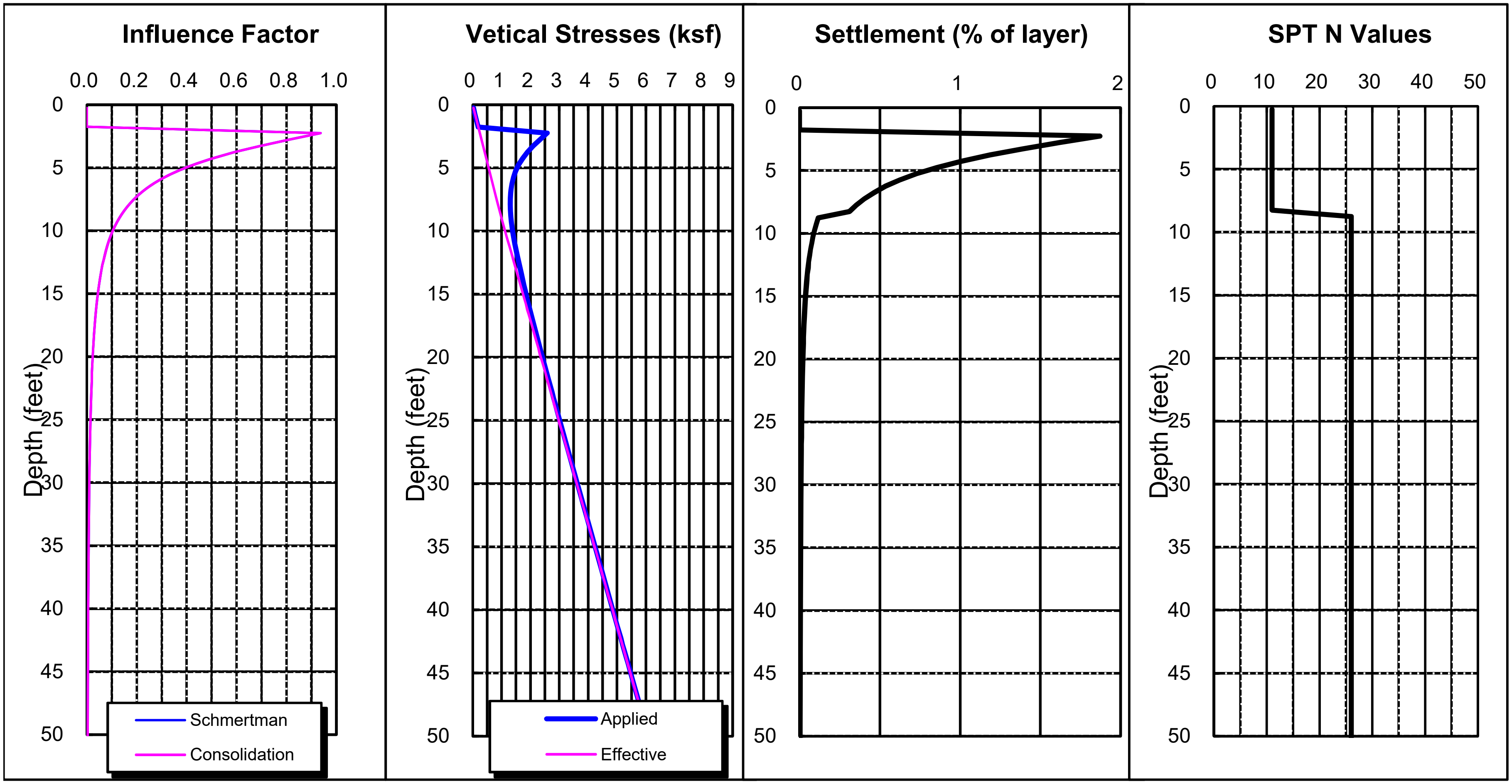
N = $N_{1(60)CS}$

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

EARTH SYSTEMS SOUTHWEST - SETTLEMENT ANALYSES

Desert Hot Springs High School CTE Building 302396-001

Width, ft: 5.0 Length, ft: 5.0 Net pressure, ksf: 2.50 **Settlement, inches: 0.8**

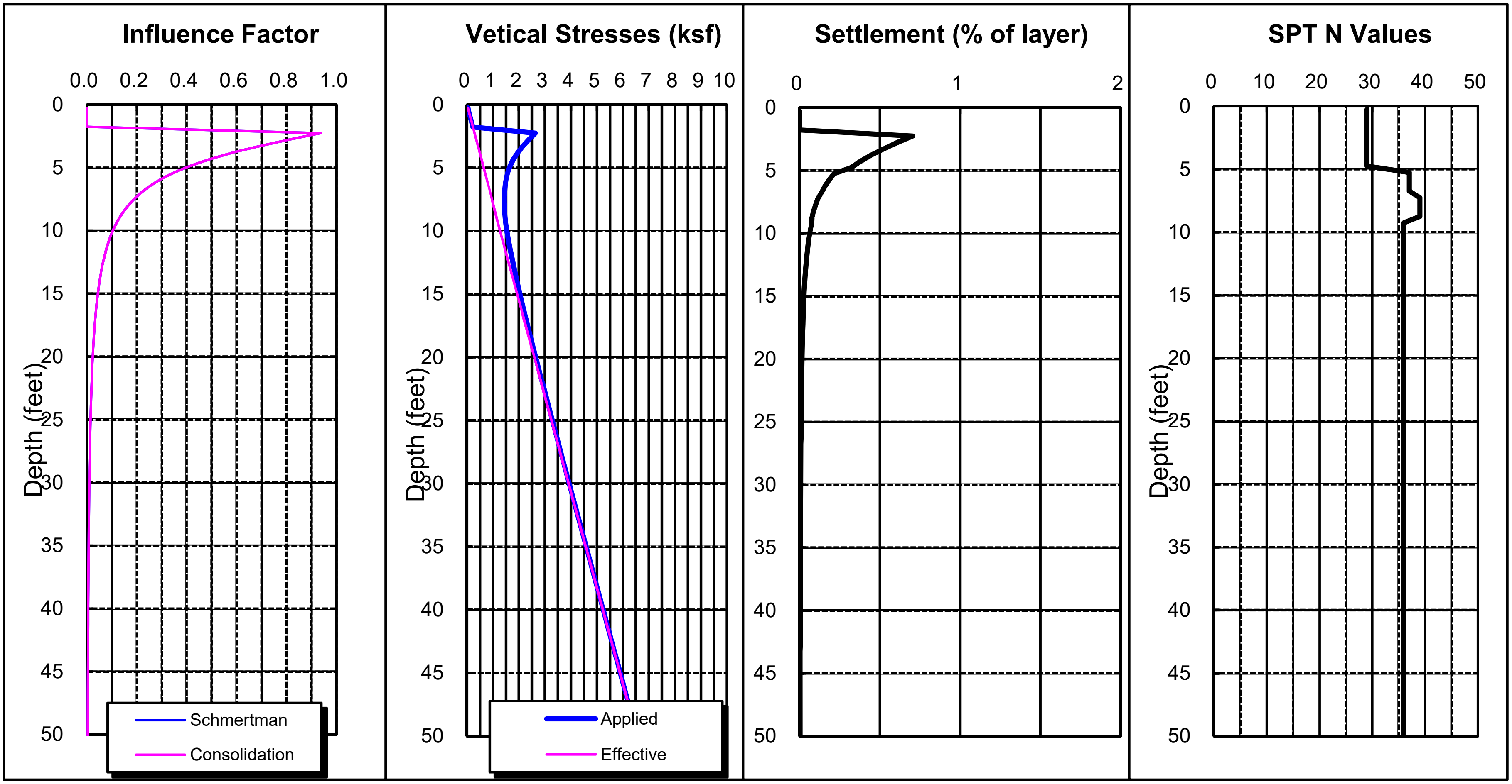


Load, Q: 63 kips Embedment, feet: 2.0 **Boring: B-1**

EARTH SYSTEMS SOUTHWEST - SETTLEMENT ANALYSES

Desert Hot Springs High School CTE Building 302396-001

Width, ft: 5.0 Length, ft: 5.0 Net pressure, ksf: 2.50 **Settlement, inches: 0.3**

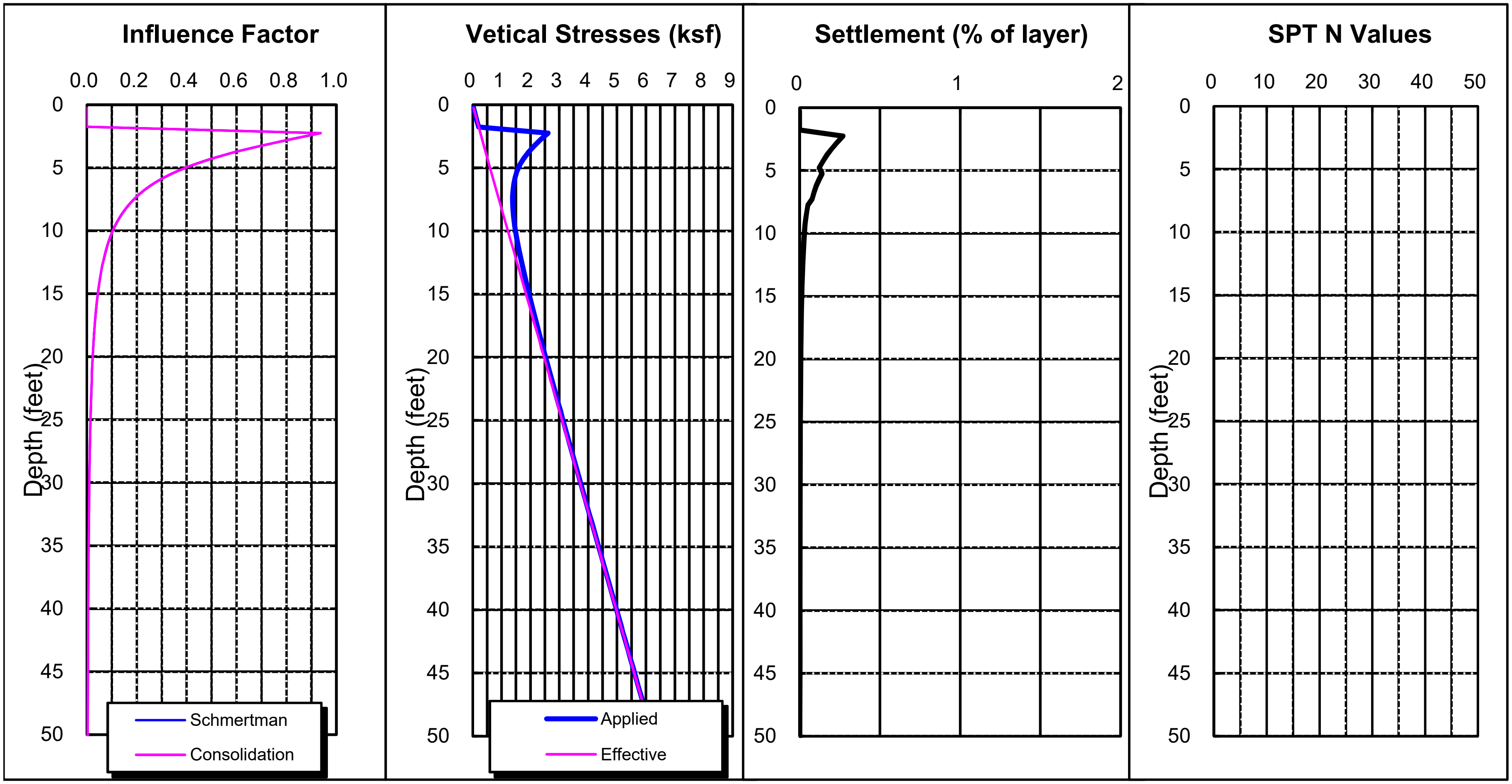


Load, Q: 63 kips Embedment, feet: 2.0 **Boring: B-3**

EARTH SYSTEMS SOUTHWEST - SETTLEMENT ANALYSES

Desert Hot Springs High School CTE Building 302396-001

Width, ft: 5.0 Length, ft: 5.0 Net pressure, ksf: 2.50 **Settlement, inches: 0.1**

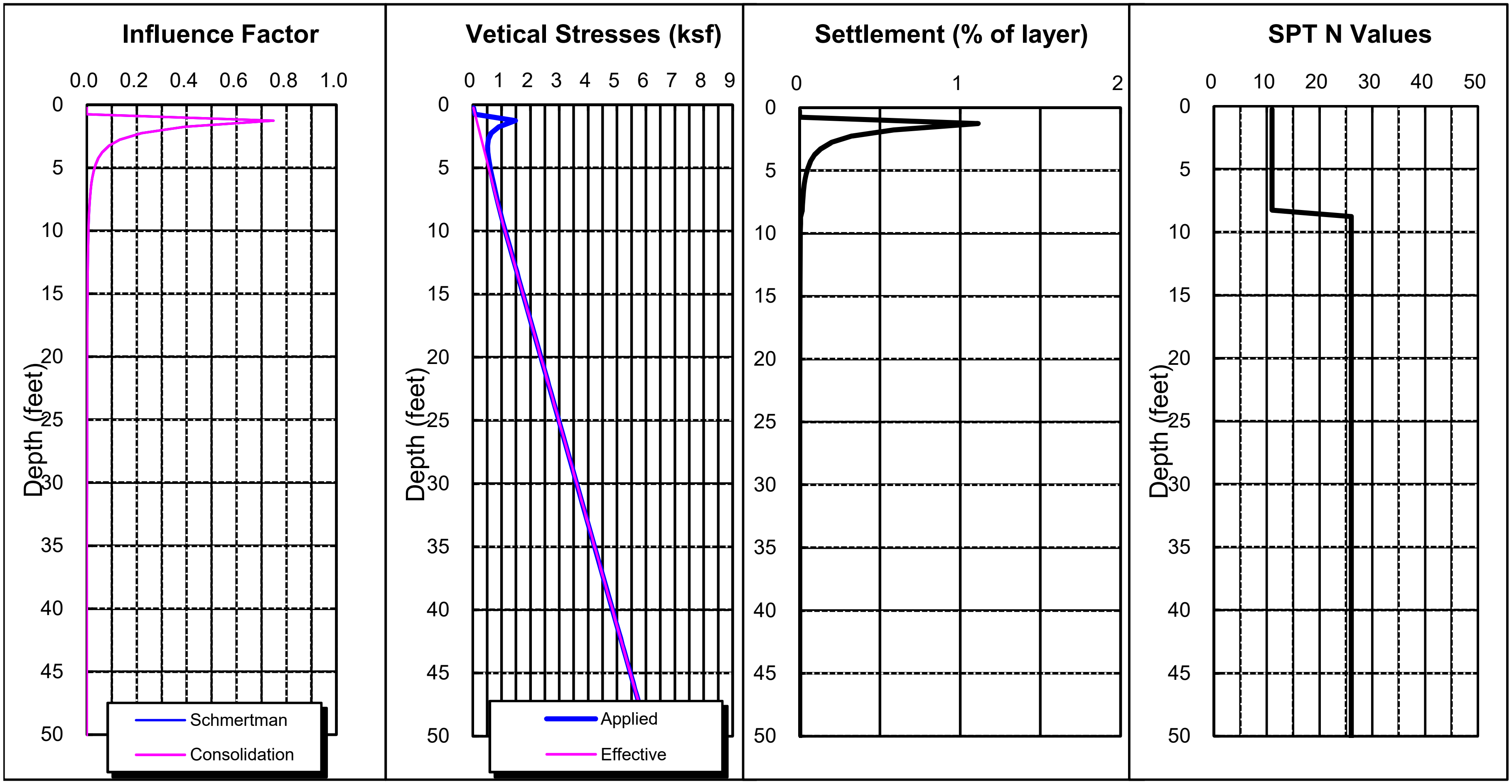


Load, Q: 63 kips Embedment, feet: 2.0 **Boring: B-4**

EARTH SYSTEMS SOUTHWEST - SETTLEMENT ANALYSES

Desert Hot Springs High School CTE Building 302396-001

Width, ft: 1.7 Length, ft: 1.0 Net pressure, ksf: 1.80 **Settlement, inches: 0.2**



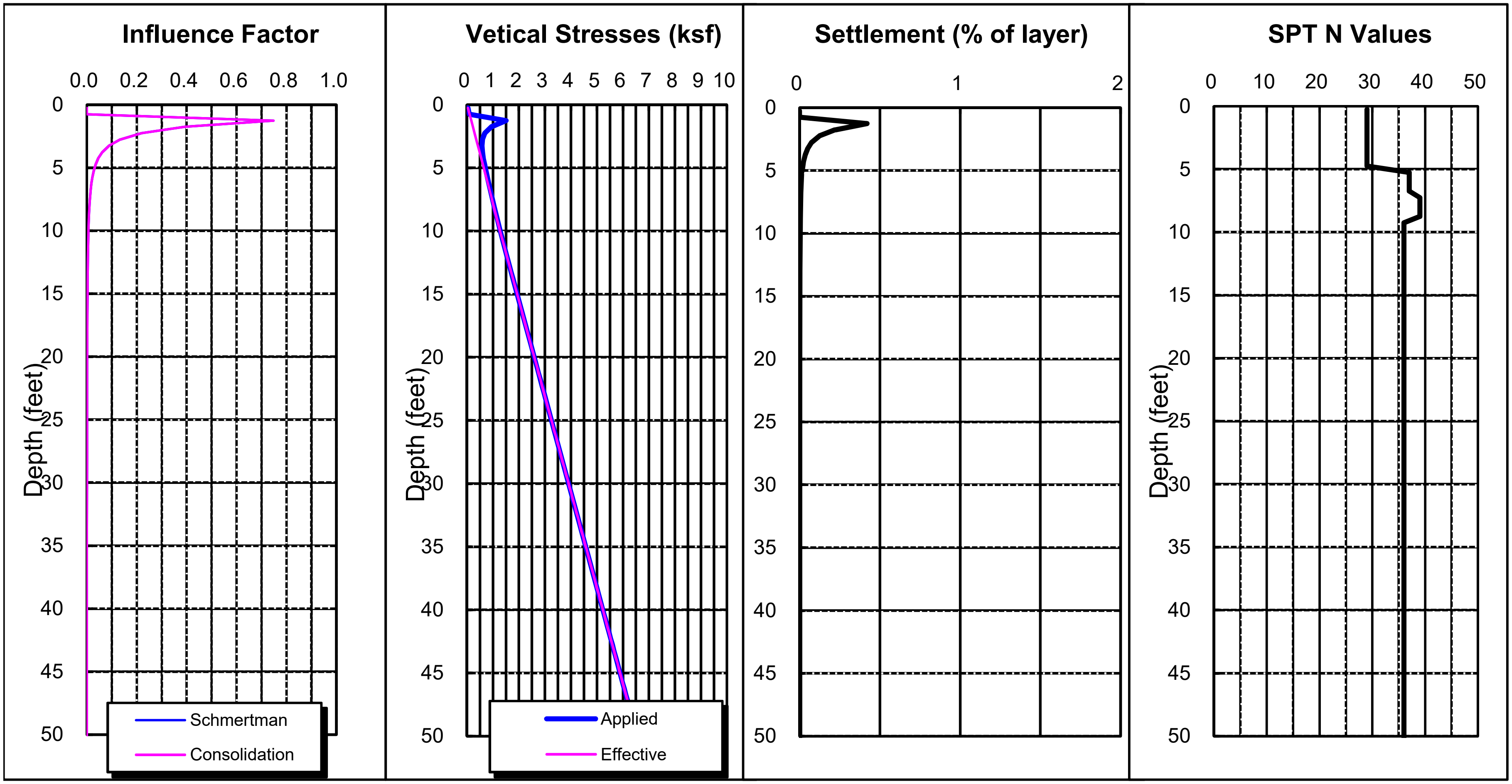
Load, Q: 3 kpf Embedment, feet: 1.0 **Boring: B-1**

EARTH SYSTEMS SOUTHWEST - SETTLEMENT ANALYSES

Desert Hot Springs High School CTE Building

302396-001

Width, ft: 1.7 | Length, ft: 1.0 | Net pressure, ksf: 1.80 | **Settlement, inches: 0.1**

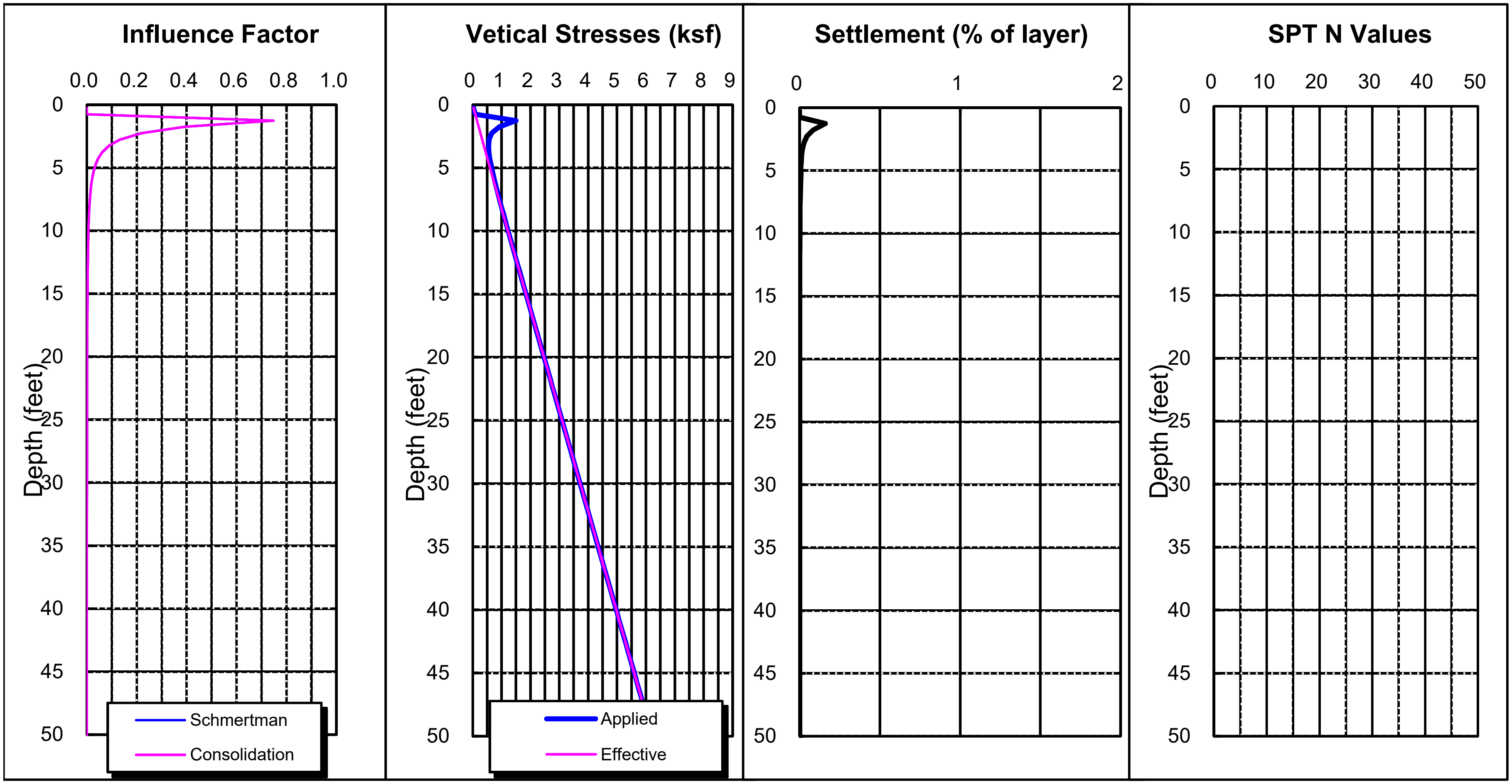


Load, Q: 3 kpf | Embedment, feet: 1.0 | **Boring: B-3**

EARTH SYSTEMS SOUTHWEST - SETTLEMENT ANALYSES

Desert Hot Springs High School CTE Building 302396-001

Width, ft: 1.7 Length, ft: 1.0 Net pressure, ksf: 1.80 **Settlement, inches: 0.0**



Load, Q: 3 kpf Embedment, feet: 1.0 **Boring: B-4**

APPENDIX B

Laboratory Test Results

UNIT DENSITIES AND MOISTURE CONTENT ASTM D2937 & D2216

Job Name: Desert Hot Springs High School CTE Building

Sample Location	Depth (feet)	Unit Dry Density (pcf)	Moisture Content (%)	USCS Group Symbol
B1	5	106	4	SP
B1	10	120	4	SP
B2	1.5	120	7	SP-SM
B3	2	118	8	SP-SM
B3	5	121	8	SP-SM
B3	7	113	3	SW
B3	9	129	3	SW

UNIT DENSITIES AND MOISTURE CONTENT ASTM D6938 & D2216

Job Name: Desert Hot Springs High School CTE Building

Sample Location	Depth (feet)	Unit Dry Density (pcf)	Moisture Content (%)	USCS Group Symbol
T-1	1	112	7	SP
T-1	4	120	7	SP
T-2	2	120	6	SP-SM
T-3	1	125	9	SP-SM
T-3	3.5	127	4	SW
T-4	1	126	7	SP-SM
T-4	4	117	6	SP
T-5	2	126	2	SP-SM
T-5	4	115	4	SP-SM
T-6	2	123	5	SP-SM
T-6	4	123	3	SP-SM

File No.: 302396-001 9/10/2018

Lab No.: 18-114

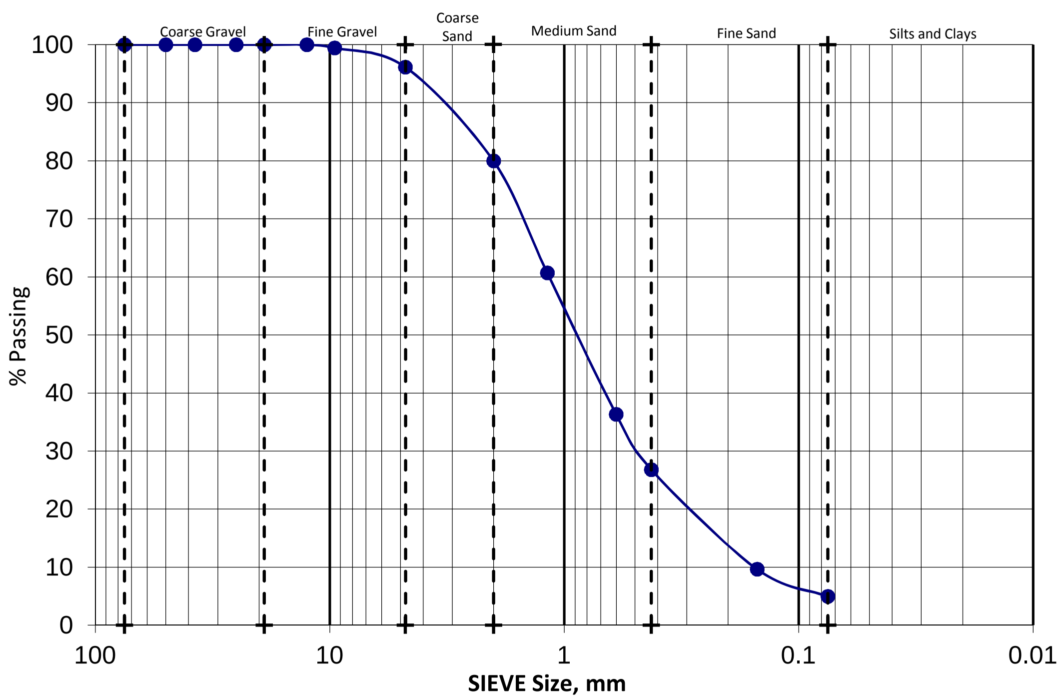
SIEVE ANALYSIS ASTM D6913

Job Name: Desert Hot Springs High School CTE Building

Sample ID: T-6 @ 6-8 feet

Description: Sand (SW)

Sieve Size	% Passing
3"	100
2"	100
1-1/2"	100
1"	100
3/4"	100
1/2"	100
3/8"	99
#4	96
#10	80
#16	61
#30	36
#40	27
#100	10
#200	5



% Coarse Gravel:	0	% Coarse Sand:	16			
% Fine Gravel:	4	% Medium Sand:	53	Cu:	7.54	
		% Fine Sand:	22	Cc:	1.28	Gradation
% Total Gravel	4	% Total Sand	91	% Fines:	4.9	Well Graded

File No.: 302396-001

Job Name: Desert Hot Springs High School CTE Building

Lab Number: 18-114

September 6, 2018

AMOUNT PASSING NO. 200 SIEVE

ASTM D 1140

Sample Location	Depth (feet)	Fines Content (%)	USCS Group Symbol
T-3	1	6.6	SP-SM
T-3	3.5	9.7	SW-SM
B1	5	3.3	SP

File No.: 302396-001 September 6, 2018
Lab No.: 18-114

CONSOLIDATION TEST ASTM D 2435 & D 5333

Desert Hot Springs High School CTE Building
B1 @ 5 feet

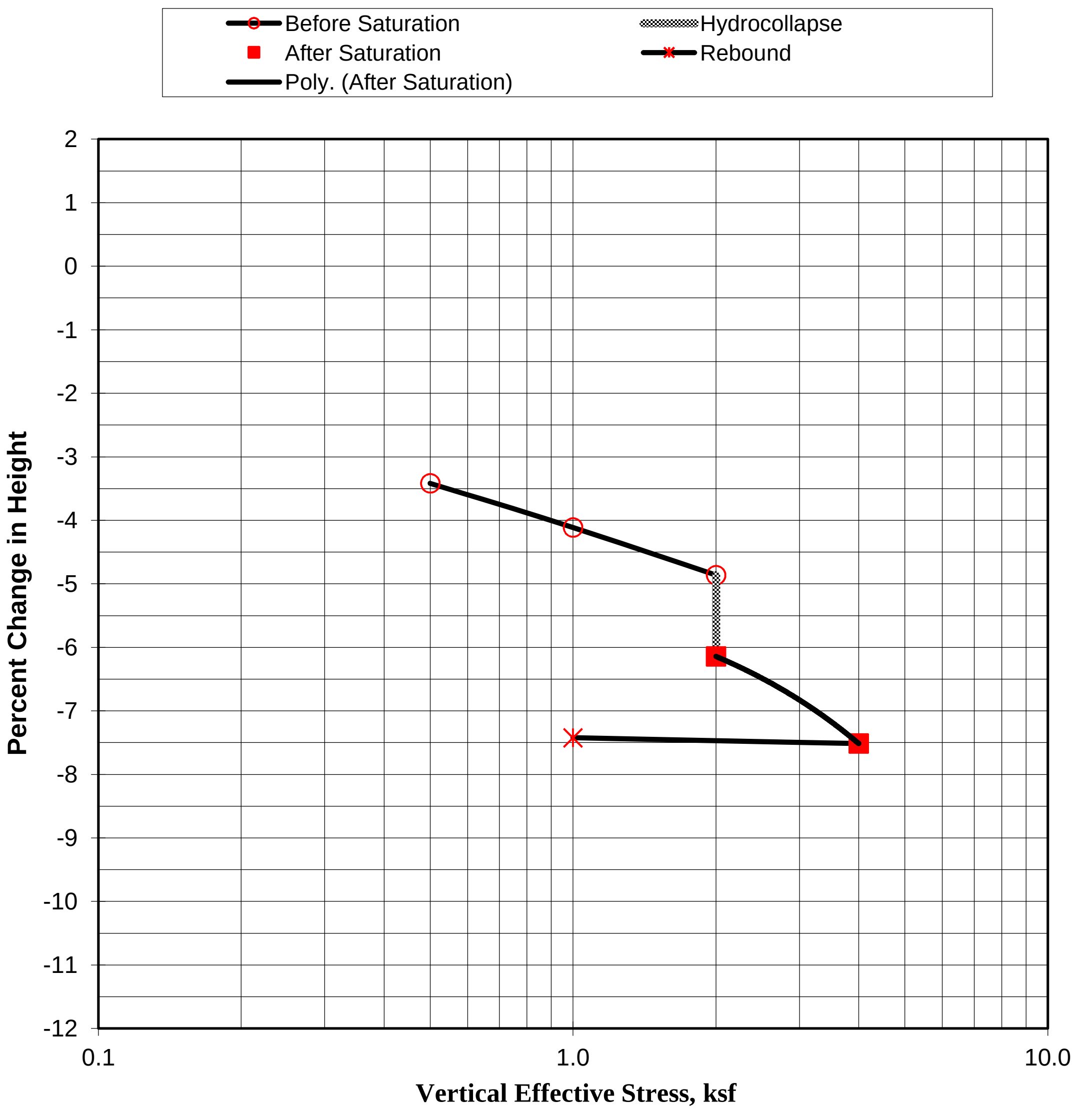
Sand (SP)

Ring Sample

Initial Dry Density: 99.1 pcf
Initial Moisture: 5.3%
Specific Gravity: 2.67
Initial Void Ratio: 0.681

Hydrocollapse: 1.3% @ 2.0 ksf

% Change in Height vs Normal Presssure Diagram



File No.: 302396-001
Lab No.: 18-114

September 6, 2018

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

Desert Hot Springs High School CTE Building
B1 @ 10 feet

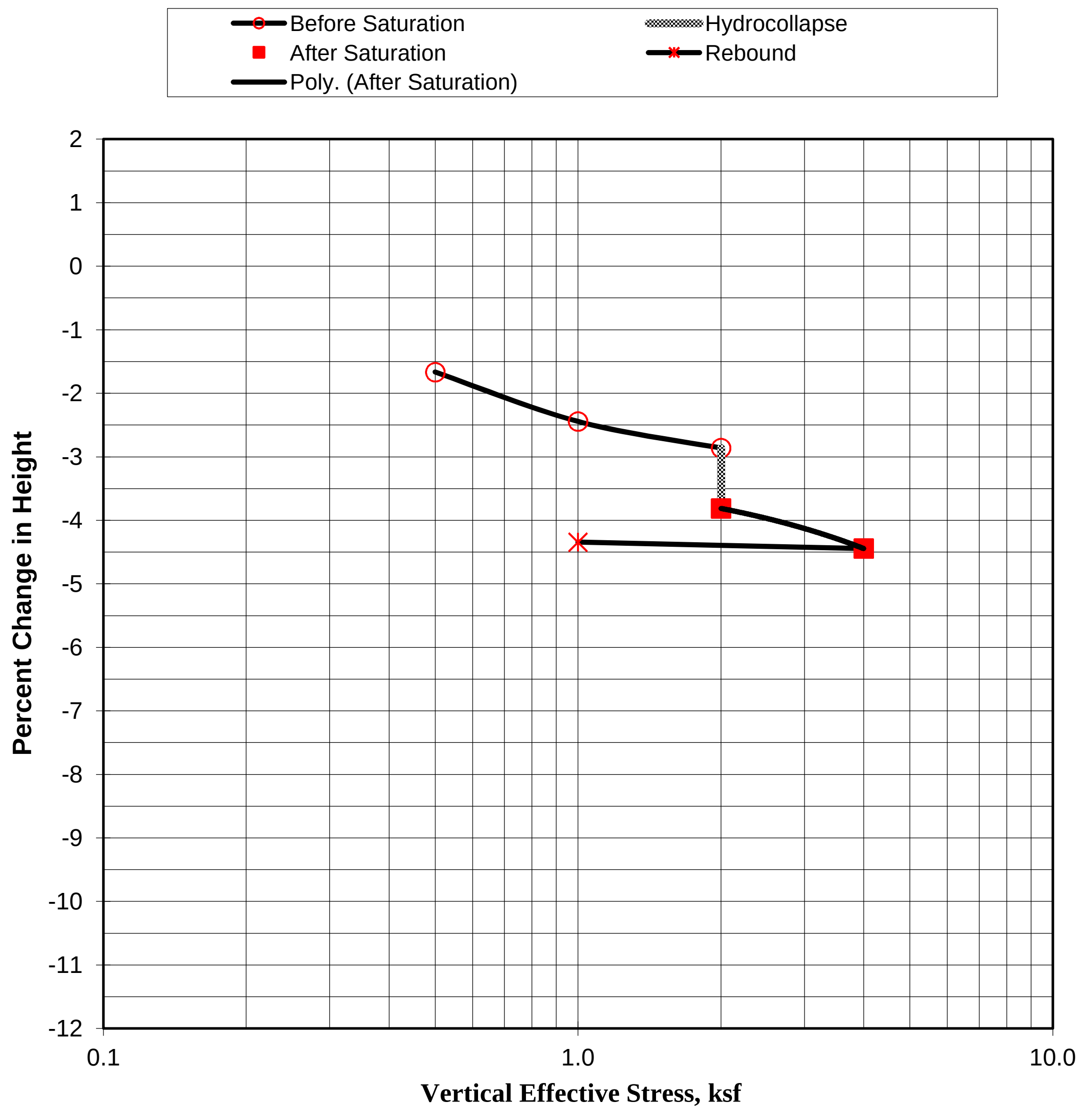
Sand (SP)

Ring Sample

Initial Dry Density: 100.6 pcf
Initial Moisture: 4.4%
Specific Gravity: 2.67
Initial Void Ratio: 0.657

Hydrocollapse: 0.9% @ 2.0 ksf

% Change in Height vs Normal Presssure Diagram



File No.: 302396-001
Lab No.: 18-114

September 6, 2018

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

Desert Hot Springs High School CTE Building
B3 @ 2 feet

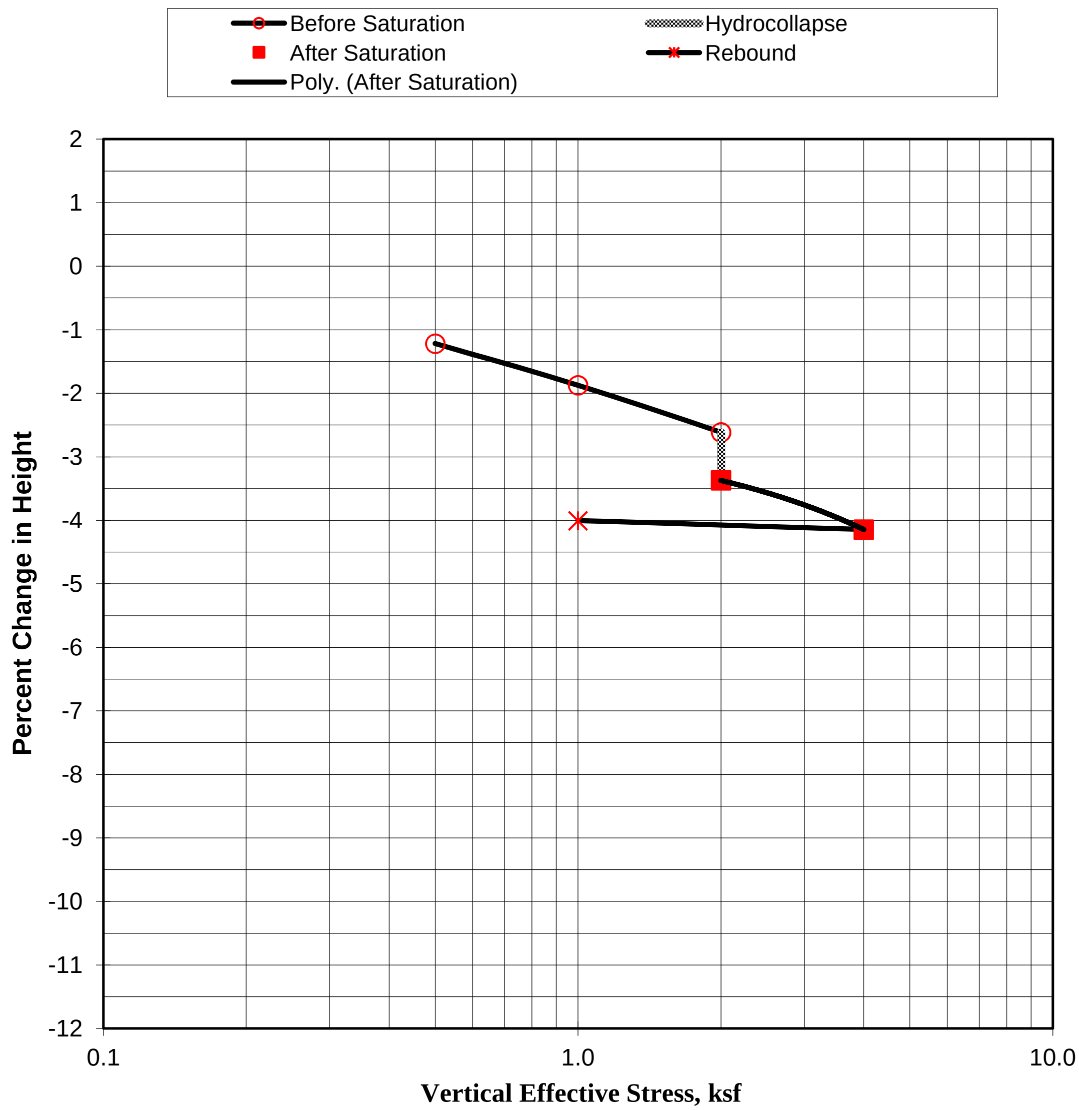
Sand with Silt (SP-SM)

Ring Sample

Initial Dry Density: 107.9 pcf
Initial Moisture: 9.3%
Specific Gravity: 2.67
Initial Void Ratio: 0.545

Hydrocollapse: 0.8% @ 2.0 ksf

% Change in Height vs Normal Presssure Diagram



File No.: 302396-001
Lab No.: 18-114

September 6, 2018

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

Desert Hot Springs High School CTE Building
B3 @ 5 feet

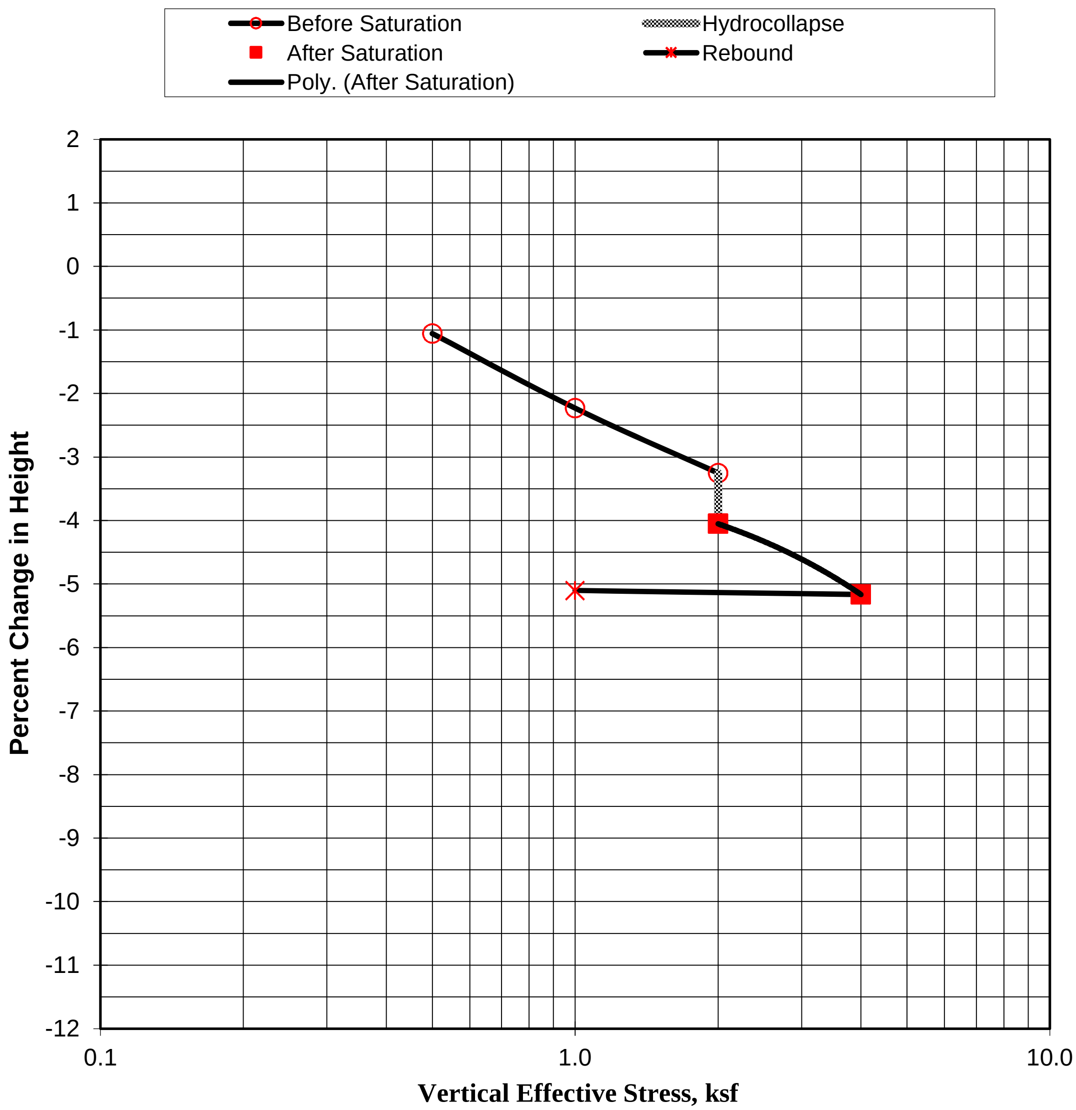
Sand with Silt (SP-SM)

Ring Sample

Initial Dry Density: 106.3 pcf
Initial Moisture: 9.7%
Specific Gravity: 2.67
Initial Void Ratio: 0.568

Hydrocollapse: 0.8% @ 2.0 ksf

% Change in Height vs Normal Presssure Diagram



EXPANSION INDEX **ASTM D-4829**

Job Name: Desert Hot Springs High School CTE Building
Sample ID: T-1 @ 1-3 feet
Soil Description: Sand (SP)

Initial Moisture, %:	10.1
Initial Compacted Dry Density, pcf:	109.6
Initial Saturation, %:	51
Final Moisture, %:	13.6
Volumetric Swell, %:	-0.7

Expansion Index, EI: **0** **Very Low**

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

MAXIMUM DRY DENSITY / OPTIMUM MOISTURE

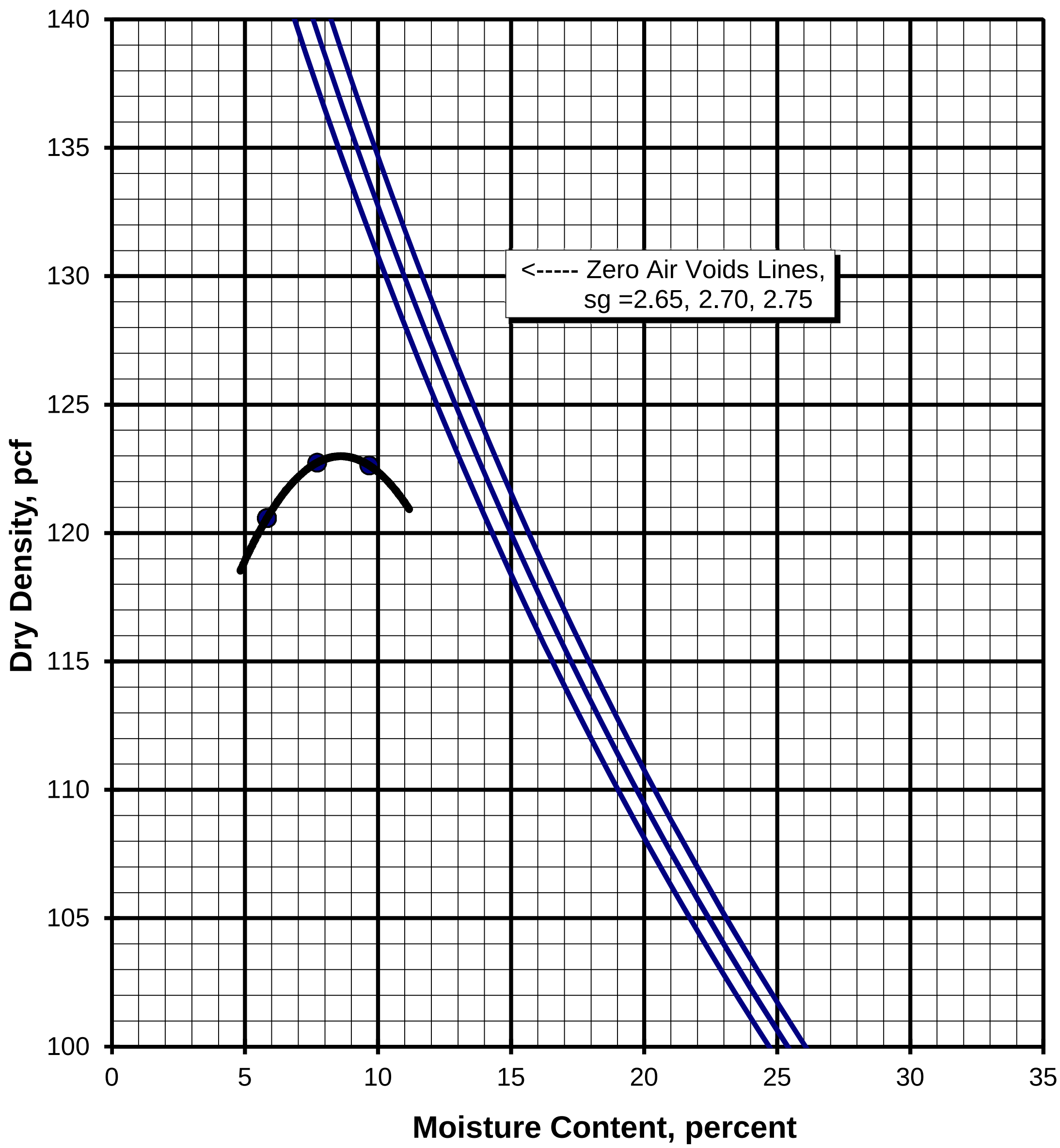
ASTM D 1557 (Modified)

Job Name: Desert Hot Springs High School CTE Building
Sample ID: 1
Location: T1 @ 1-3 feet
Description: Dark Brown Fine to Coarse Sand (SP)

Procedure Used: A
Preparation Method: Moist
Rammer Type: Mechanical
Lab Number: 18-114

Maximum Dry Density: **123.1 pcf**
Optimum Moisture: **8.7%**
Corrected for Oversize (ASTM D4718)

Sieve Size	% Retained (Cumulative)
3/4"	2.4
3/8"	3.8
#4	6.3



File No.: 302396-001 September 6, 2018

Lab No.: 18-114

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

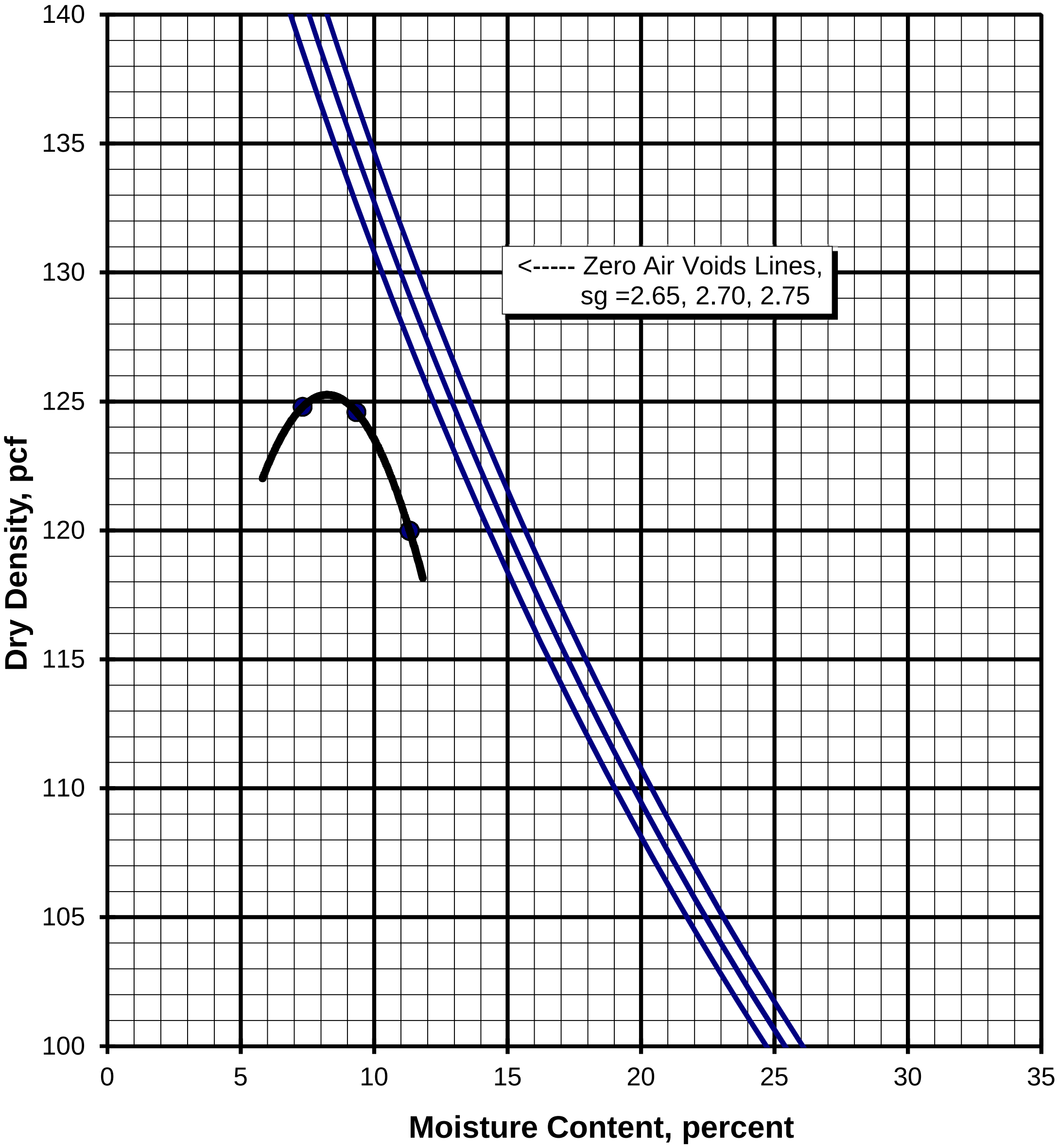
Job Name:	Desert Hot Springs High School CTE Building	Procedure Used:	A
Sample ID:	2	Preparation Method:	Moist
Location:	T-5 @ 1-5 feet	Rammer Type:	Mechanical
Description:	Dark Brown Fine to Coarse Sand with Silt (SP-SM)	Lab Number:	18-114

Maximum Dry Density: **125.1 pcf**

Optimum Moisture: **8.3%**

Corrected for Oversize (ASTM D4718)

Sieve Size	% Retained (Cumulative)
3/4"	2.3
3/8"	3.2
#4	5.0



File No.: 302396-001 September 6, 2018

Lab No.: 18-114

MAXIMUM DRY DENSITY / OPTIMUM MOISTURE

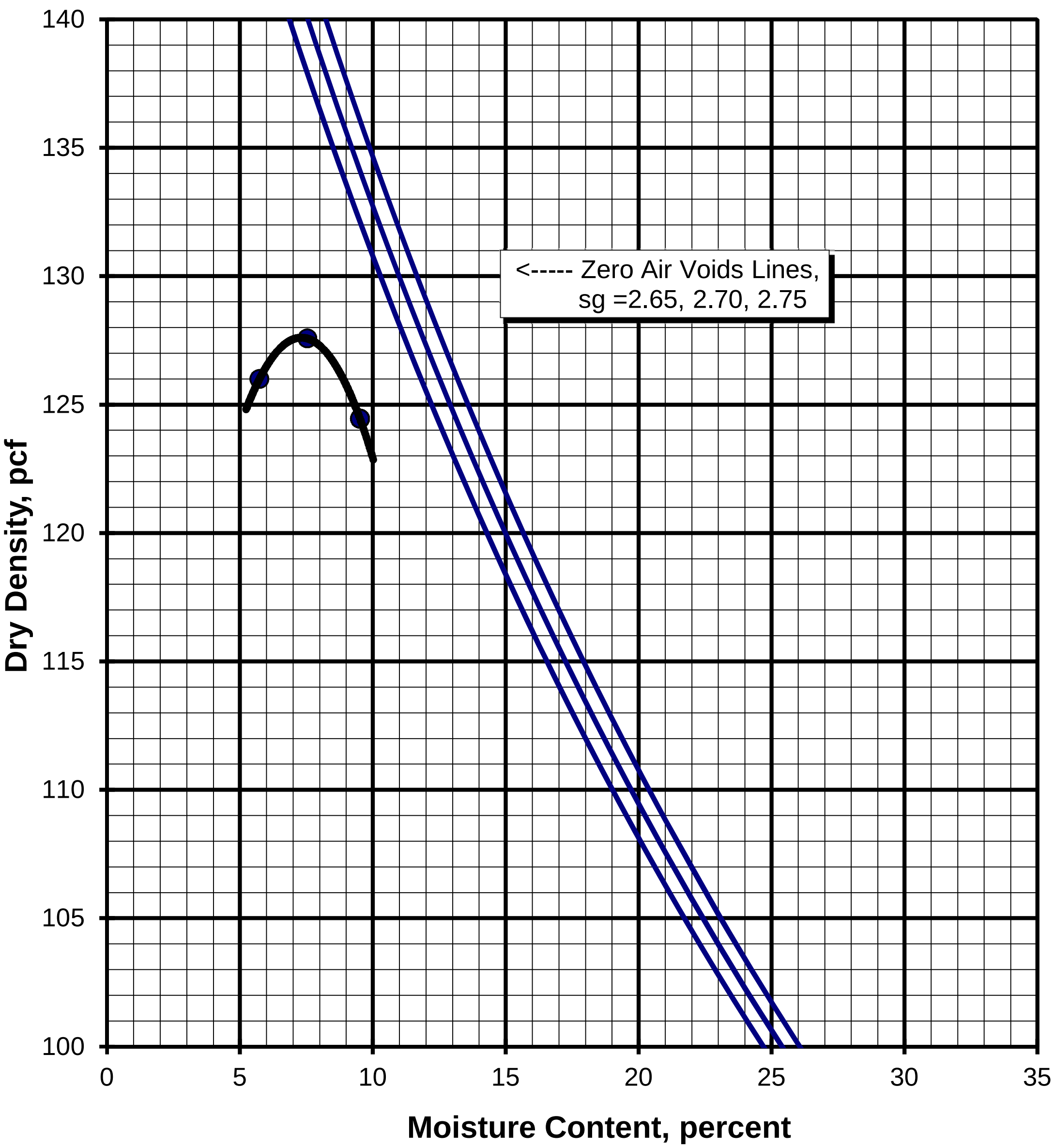
ASTM D 1557 (Modified)

Job Name: Desert Hot Springs High School CTE Building
Sample ID: 3
Location: T-6 @ 2-4 feet
Description: Dark Borwn Fine to Coarse Sand with Silt (SP-SM)

Procedure Used: A
Preparation Method: Moist
Rammer Type: Mechanical
Lab Number: 18-114

Maximum Dry Density: **127.6 pcf**
Optimum Moisture: **7.5%**
Corrected for Oversize (ASTM D4718)

Sieve Size	% Retained (Cumulative)
3/4"	4.1
3/8"	5.7
#4	7.7



File No.: 302396-001 9/6/2018
Lab No.: 18-114

SOIL CHEMICAL ANALYSES

Job Name: Desert Hot Springs High School CTE Building
Job No.: 302396-001
Sample ID: T-1
Sample Location: 1-3 feet

Resistivity (Units)	
as-received (ohm-cm)	1,680,000
saturated (ohm-cm)	8,000
pH	8.6
Electrical Conductivity (mS/cm)	0.08
Chemical Analyses	
Cations	
calcium Ca^{2+} (mg/kg)	28
magnesium Mg^{2+} (mg/kg)	6
sodium Na^{1+} (mg/kg)	49
potassium K^{1+} (mg/kg)	22
Anions	
carbonate CO_3^{2-} (mg/kg)	6
bicarbonate HCO_3^{1-}(mg/kg)	85
fluoride F^{1-} (mg/kg)	11
chloride Cl^{1-} (mg/kg)	ND
sulfate SO_4^{2-} (mg/kg)	54
phosphate PO_4^{3-} (mg/kg)	ND
Other Tests	
ammonium NH_4^{1+} (mg/kg)	ND
nitrate NO_3^{1-} (mg/kg)	4.5
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 Sulfates[1]	0 -1,000 mg/Kg (ppm) [0-.1%]	Low
	1,000 - 2,000 mg/Kg (ppm) [0.1-0.2%]	Moderate
	2,000 - 20,000 mg/Kg (ppm) [0.2-2.0%]	Severe
	> 20,000 mg/Kg (ppm) [>2.0%]	Very Severe
Resistivity[2] (Saturated)	0- 900 ohm-cm	Very Severely Corrosive
	900 to 2,300 ohm-cm	Severely Corrosive
	2,300 to 5,000 ohm-cm	Moderately Corrosive
	5,000-10,000 ohm-cm	Mildly Corrosive
	10,000+ ohm-cm	Progressively Less Corrosive

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

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

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

File No.: 302396-001 September 6, 2018

Lab No.: 18-114

MAXIMUM DRY DENSITY / OPTIMUM MOISTURE

ASTM D 1557 (Modified)

Job Name: Desert Hot Springs High School CTE Building

Sample ID: 3

Location: T-6 @ 2-4 feet

Description: Dark Brown Fine to Coarse Sand with Silt (SP-SM)

Procedure Used: A

Preparation Method: Moist

Rammer Type: Mechanical

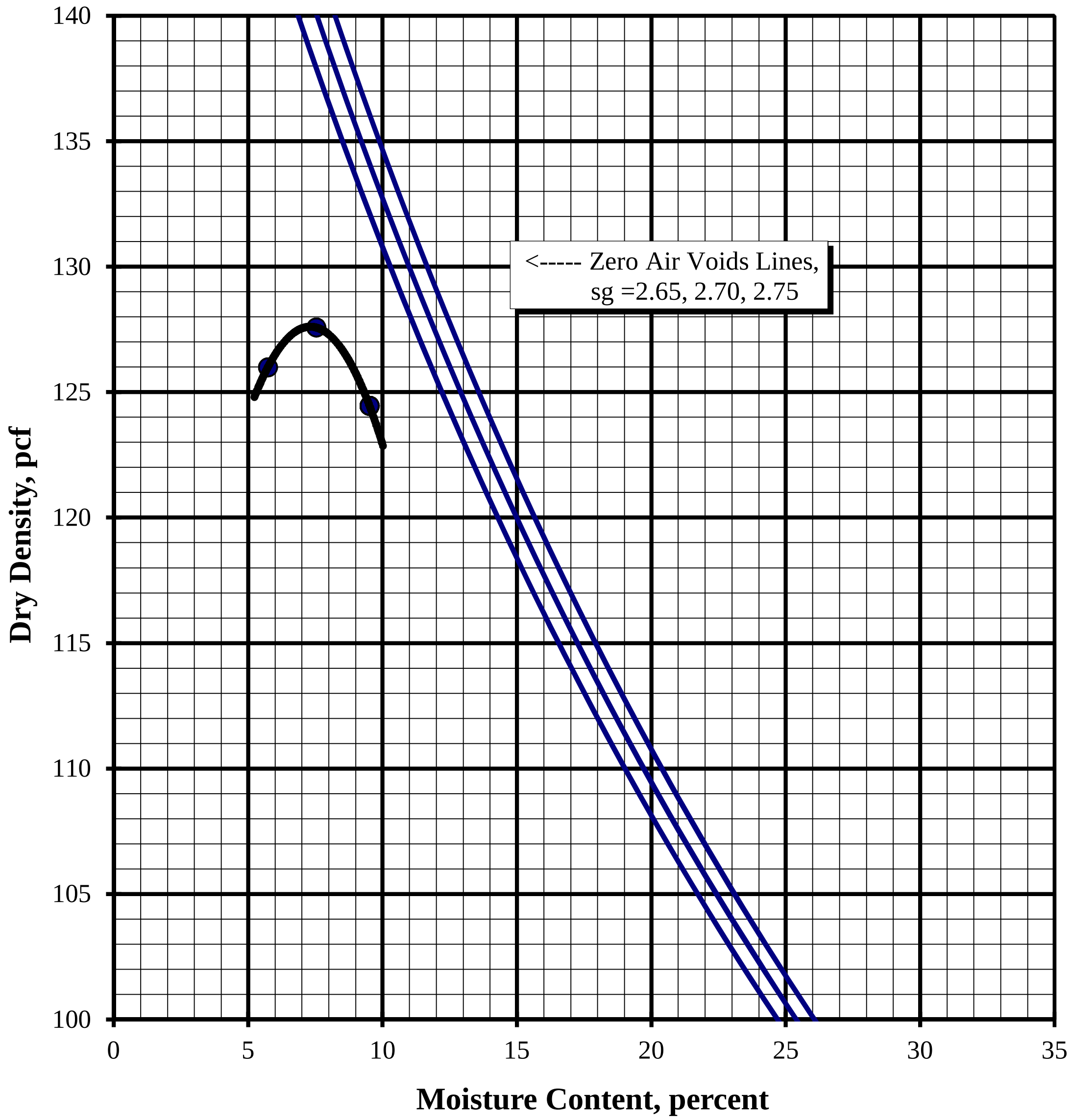
Lab Number: 18-114

Maximum Dry Density: 127.6 pcf

Optimum Moisture: 7.5%

Corrected for Oversize (ASTM D4718)

Sieve Size	% Retained (Cumulative)
3/4"	4.1
3/8"	5.7
#4	7.7



Desert Hot Springs School
CTE Building

302396-001

RESISTANCE '*R* ' VALUE AND EXPANSION PRESSURE

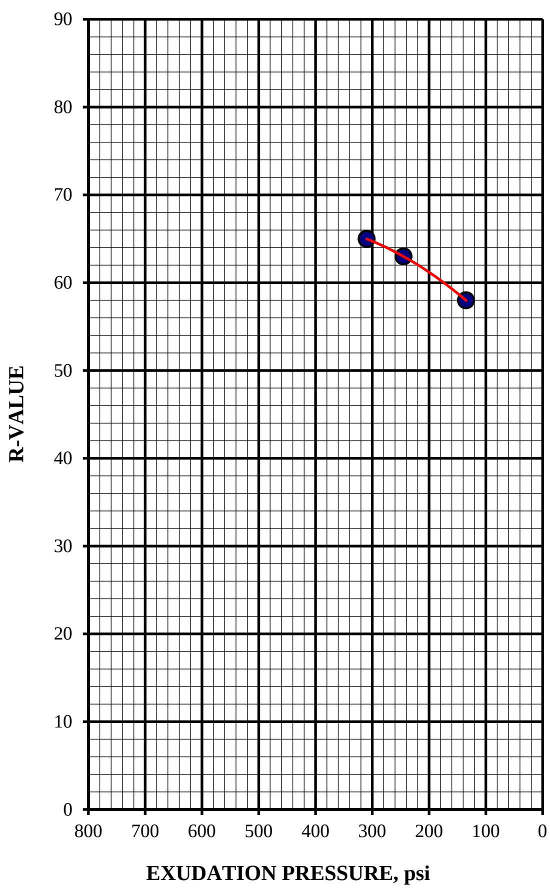
ASTM D 2844/D2844M-13

September 6, 2018

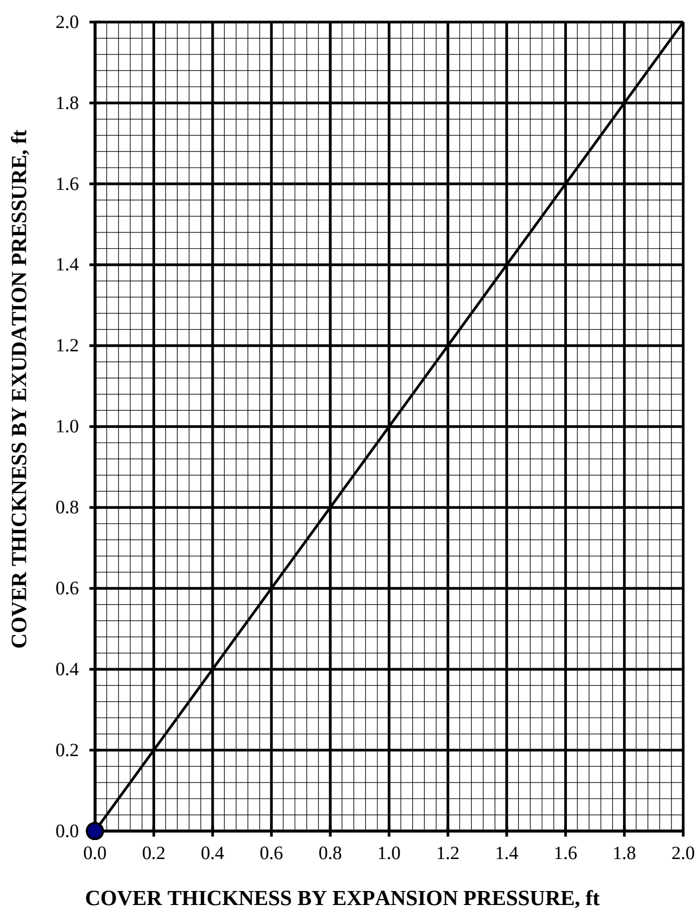
T-5 @ 1.0 - 5.0'
Brown Poorly Graded Sand with Silt (SP-SM)

Dry Density @ 300 psi Exudation Pressure: 126.0-pcf
%Moisture @ 300 psi Exudation Pressure: 12.6%
R-Value - Exudation Pressure: 65
R-Value - Expansion Pressure: N/A
R-Value @ Equilibrium: 65

EXUDATION PRESSURE CHART



EXPANSION PRESSURE CHART



APPENDIX C

Dry Seismic Settlement For Nearby Sites

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

Project: SWC Pierson Blvd & Cholla Drive, Desert Hot S
Job No: 10710-01
Date: 9/7/2018
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: 0.62 0.77
MSF: 0.80
GWT: 50.0 feet
Calc GWT: 50.0 feet
Remediate to: 5.0 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: 0

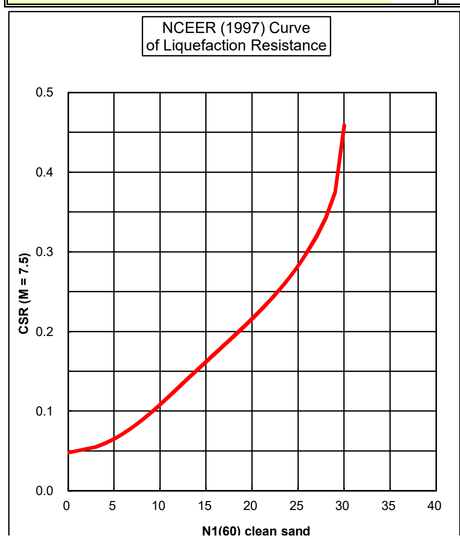
Total (in.) Induced Subsidence: 0.3 upper 50 ft

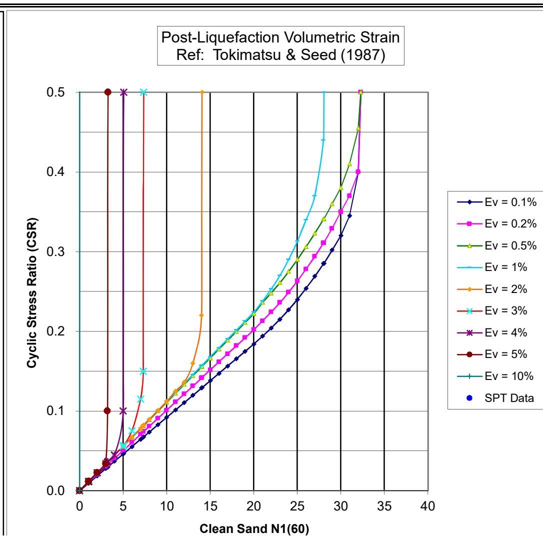
Required SF: 1.50
Minimum Calculated SF: #N/A

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)	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.)	p (tsf)	G_{max} (tsf)	τ_{av} (tsf)	Shear Strain γ	Strain E_{15}	Strain Enc	Dry Sand Subsidence (in.)
								0.000																								
5.0		50	1	125	4	2.5	5.5	0.156	0.156	1.00	1.70	0.75	1.30	99.5	100	0.0	99.5	1.00	1.200	0.501	Non-Liq.	0.0	99.5	0.01	0.00	0.105	670	0.062	1.9E-04	2.7E-05	3.3E-05	0.00
8.3		10	1	125	7	5.0	8.0	0.313	0.313	0.99	1.70	0.75	1.18	18.1	51	0.3	18.4	1.00	0.198	0.498	Non-Liq.	0.3	18.4	0.36	0.14	0.209	540	0.124	1.3E-03	1.5E-03	1.8E-03	0.14
15.5		18	1	125	7	10.0	13.0	0.625	0.625	0.98	1.30	0.76	1.26	26.7	62	0.3	27.1	1.00	0.321	0.492	Non-Liq.	0.3	27.1	0.16	0.14	0.419	869	0.245	9.7E-04	6.7E-04	8.1E-04	0.14





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

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

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/Kσ

$CRR_{7.5}$ = (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))

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

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

p = 0.67*po

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

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

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

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

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

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

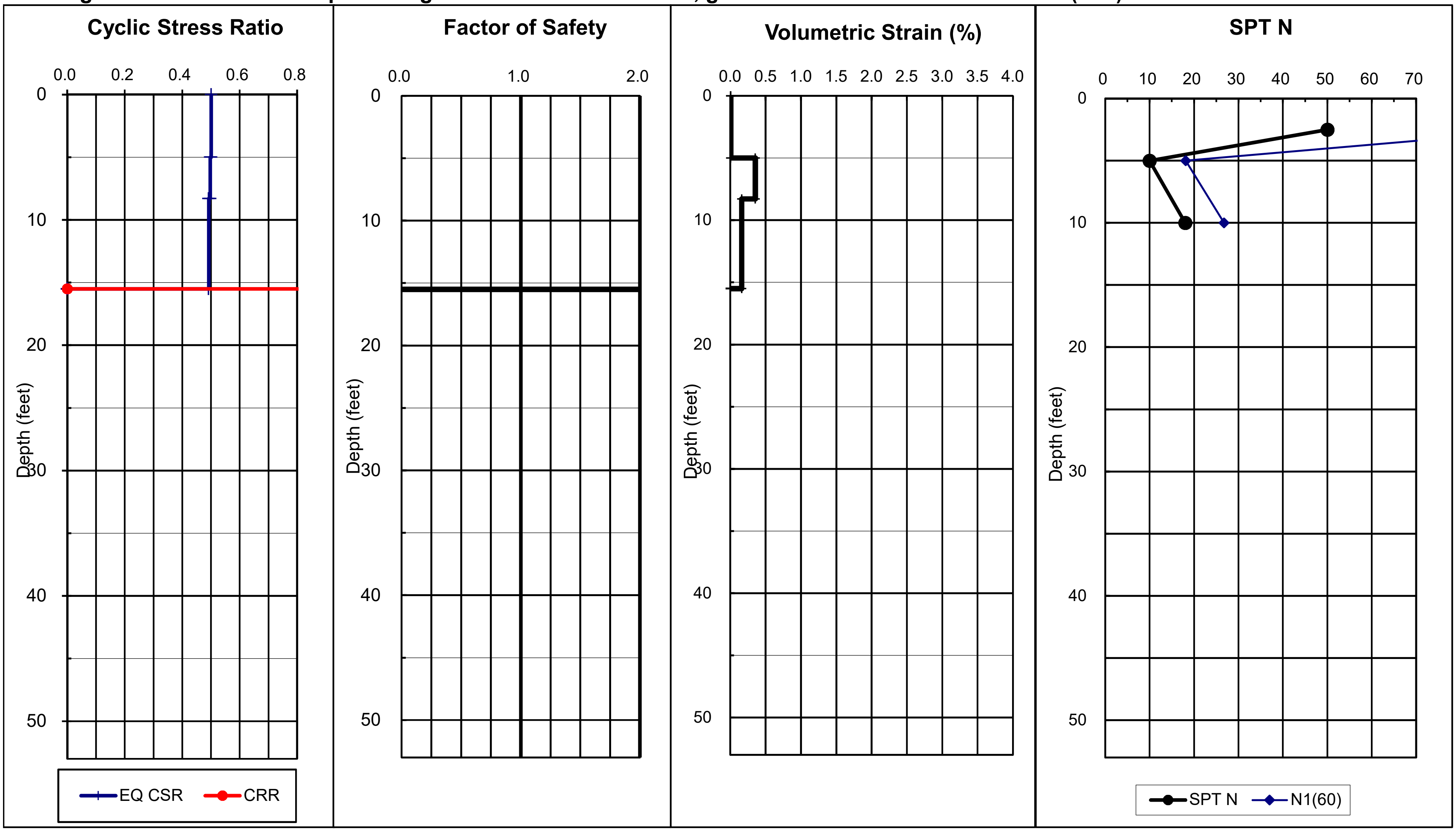
$S = 2{*}H{*}E_{nc}$

EARTH SYSTEMS - EVALUATION OF LIQUEFACTION POTENTIAL AND INDUCED SUBSIDENCE

SWC Pierson Blvd & Cholla Drive, Desert Hot Springs **Project No: 10710-01** 1996/1998 NCEER Method

Ground Compaction Remediated to 5 foot depth

Boring: B-1 **Earthquake Magnitude: 8.2** **PGA, g: 0.62** **Calc GWT (feet): 50**



Total Thickness of Liquefiable Layers: 0.0 feet **Estimated Total Ground Subsidence: 0.3 inches**

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

Project: SWC Pierson Blvd & Cholla Drive, Desert Hot S
Job No: 10710-01
Date: 9/7/2018
Boring: B-2 Data Set: 2

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: 0.62 0.77
MSF: 0.80
GWT: 50.0 feet
Calc GWT: 50.0 feet
Remediate to: 5.0 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

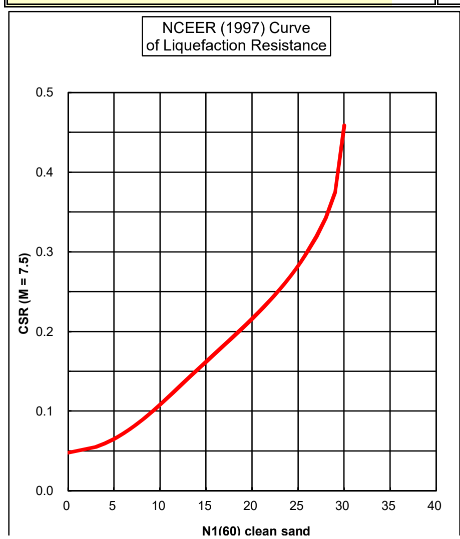
Required SF: 1.50
Minimum Calculated SF: #N/A

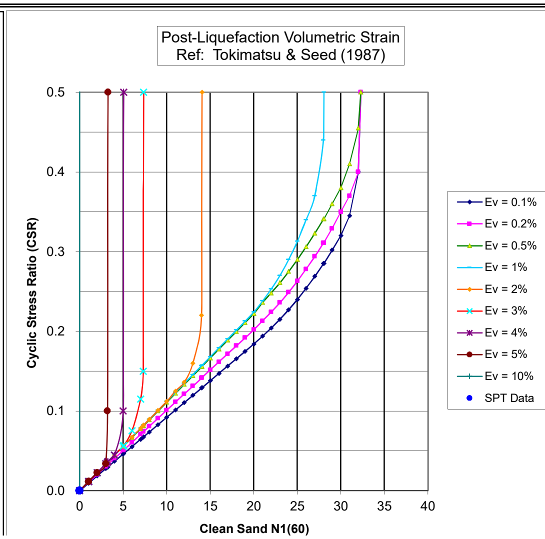
SETTLEMENT (SUBSIDENCE) OF DRY SANDS

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)	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
								0.000													
8.3		27	1	125	7	5.0	8.0	0.313	0.313	0.99	1.70	0.75	1.30	53.7	88	0.6	54.3	1.00	1.200	0.498	Non-Liq.
15.0		36	1	125	7	10.0	13.0	0.625	0.625	0.98	1.30	0.76	1.30	55.4	89	0.6	55.9	1.00	1.200	0.492	Non-Liq.
		50					3.0			1.00		0.75	1.10	0.0							

Base Depth (feet)	Post FC Adj. $\Delta N_{1(60)}$	Post $N_{1(60)CS}$	Volumetric Strain (%)	Induced Subsidence (in.)	p (tsf)	G_{max} (tsf)	τ_{av} (tsf)	Shear Strain γ	Strain E_{15}	Strain Enc	Dry Sand Subsidence (in.)
8.3	0.6	54.3	0.03	0.03	0.209	774	0.124	4.0E-04	1.2E-04	1.4E-04	0.03
15.0	0.6	55.9	0.03	0.03	0.419	1,106	0.245	4.9E-04	1.4E-04	1.7E-04	0.03
			######								





$N_{1(60)}$ = 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)
$N_{1(60)CS}$ = $N_{1(60)CS}$ + $\Delta N_{1(60)}$
Kσ = 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/Kσ
$CRR_{7.5}$ = (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 = $N_{1(60)CS}$

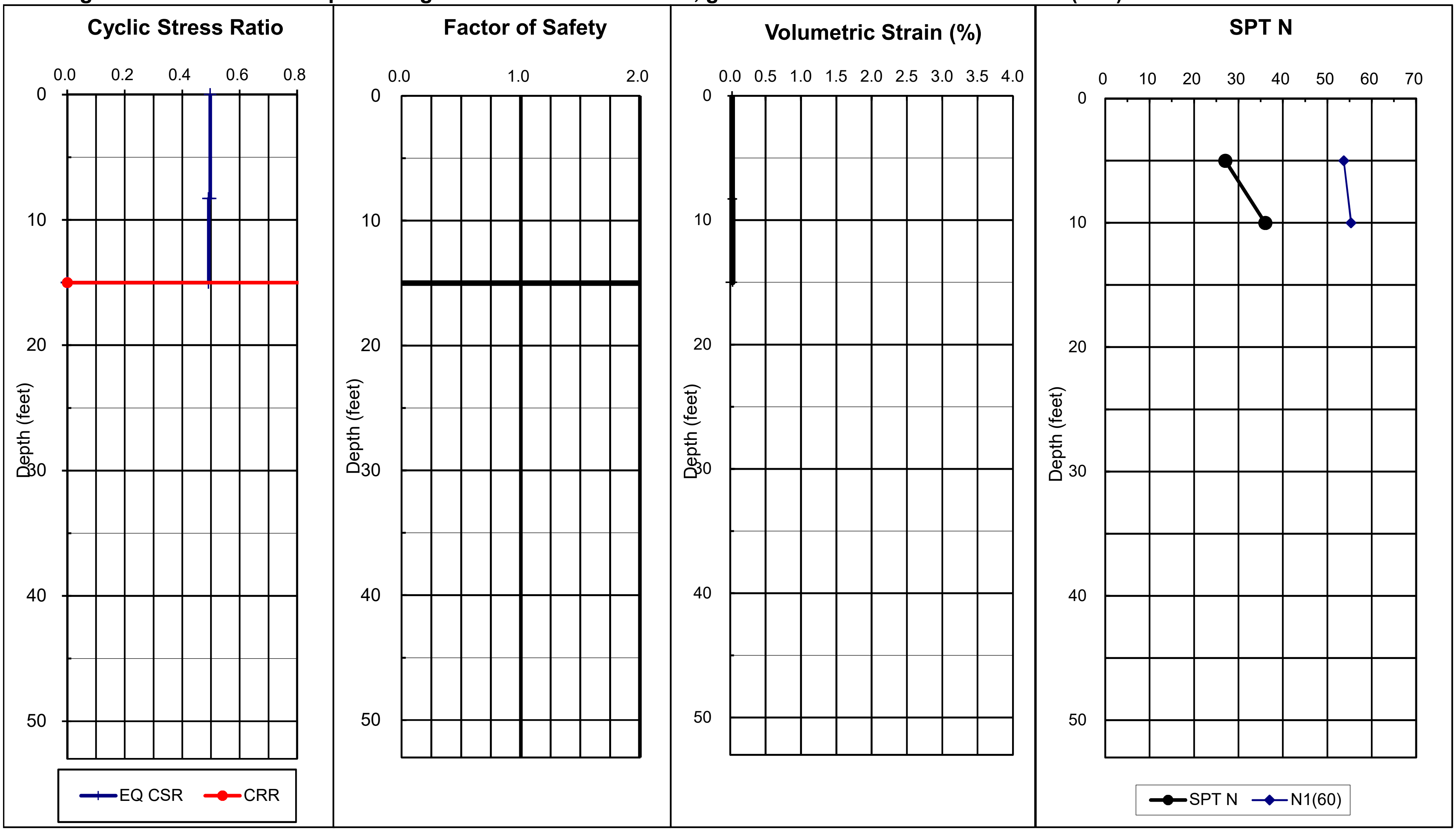
SF = $CRR_{7.5,1atm}$/CSR*

EARTH SYSTEMS - EVALUATION OF LIQUEFACTION POTENTIAL AND INDUCED SUBSIDENCE

SWC Pierson Blvd & Cholla Drive, Desert Hot Springs **Project No: 10710-01** 1996/1998 NCEER Method

Ground Compaction Remediated to 5 foot depth

Boring: B-2 **Earthquake Magnitude: 8.2** **PGA, g: 0.62** **Calc GWT (feet): 50**



Total Thickness of Liquefiable Layers: 0.0 feet

Estimated Total Ground Subsidence: 0.1 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: SWC Pierson Blvd & Cholla Drive, Desert Hot S
Job No: 10710-01
Date: 9/7/2018
Boring: B-3 Data Set: 3

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: 0.62 0.77
MSF: 0.80
GWT: 50.0 feet
Calc GWT: 50.0 feet
Remediate to: 5.0 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: 0

Total (in.) Induced Subsidence: 0.2
upper 50 ft

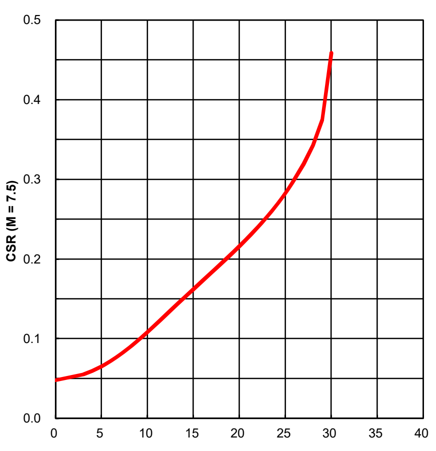
Required SF: 1.50
Minimum Calculated SF: #N/A

SETTLEMENT (SUBSIDENCE) OF DRY SANDS

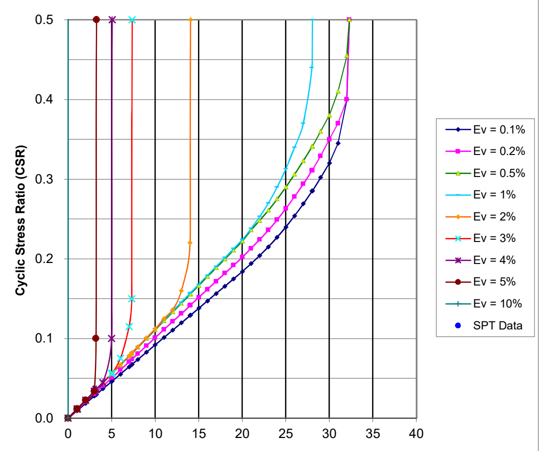
Nc = 22.5

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)	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.)	p (tsf)	G_{max} (tsf)	τ_{av} (tsf)	Shear Strain γ	Strain E_{15}	Strain Enc	Dry Sand Subsidence (in.)
								0.000																								
5.8		23	1	125	4	2.5	5.5	0.156	0.156	1.00	1.70	0.75	1.30	45.7	81	0.0	45.7	1.00	1.200	0.501	Non-Liq.	0.0	45.7	0.03	0.02	0.105	517	0.062	3.8E-04	1.4E-04	1.7E-04	0.02
10.8		13	1	125	4	7.5	10.5	0.469	0.469	0.98	1.50	0.75	1.21	21.3	55	0.0	21.3	1.00	0.231	0.495	Non-Liq.	0.0	21.3	0.27	0.16	0.314	694	0.184	1.2E-03	1.1E-03	1.3E-03	0.16
13.5		81	1	125	4	12.5	15.5	0.781	0.781	0.97	1.16	0.82	1.30	####	100	0.0	120.2	1.00	1.200	0.489	Non-Liq.	0.0	120.2	0.01	0.00	0.523	1,596	0.304	3.1E-04	3.6E-05	4.4E-05	0.00

NCEER (1997) Curve of Liquefaction Resistance



Post-Liquefaction Volumetric Strain
Ref: Tokimatsu & Seed (1987)



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

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

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

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

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

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

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

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)

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

Kσ = 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/Kσ

$CRR_{7.5}$ = (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))

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

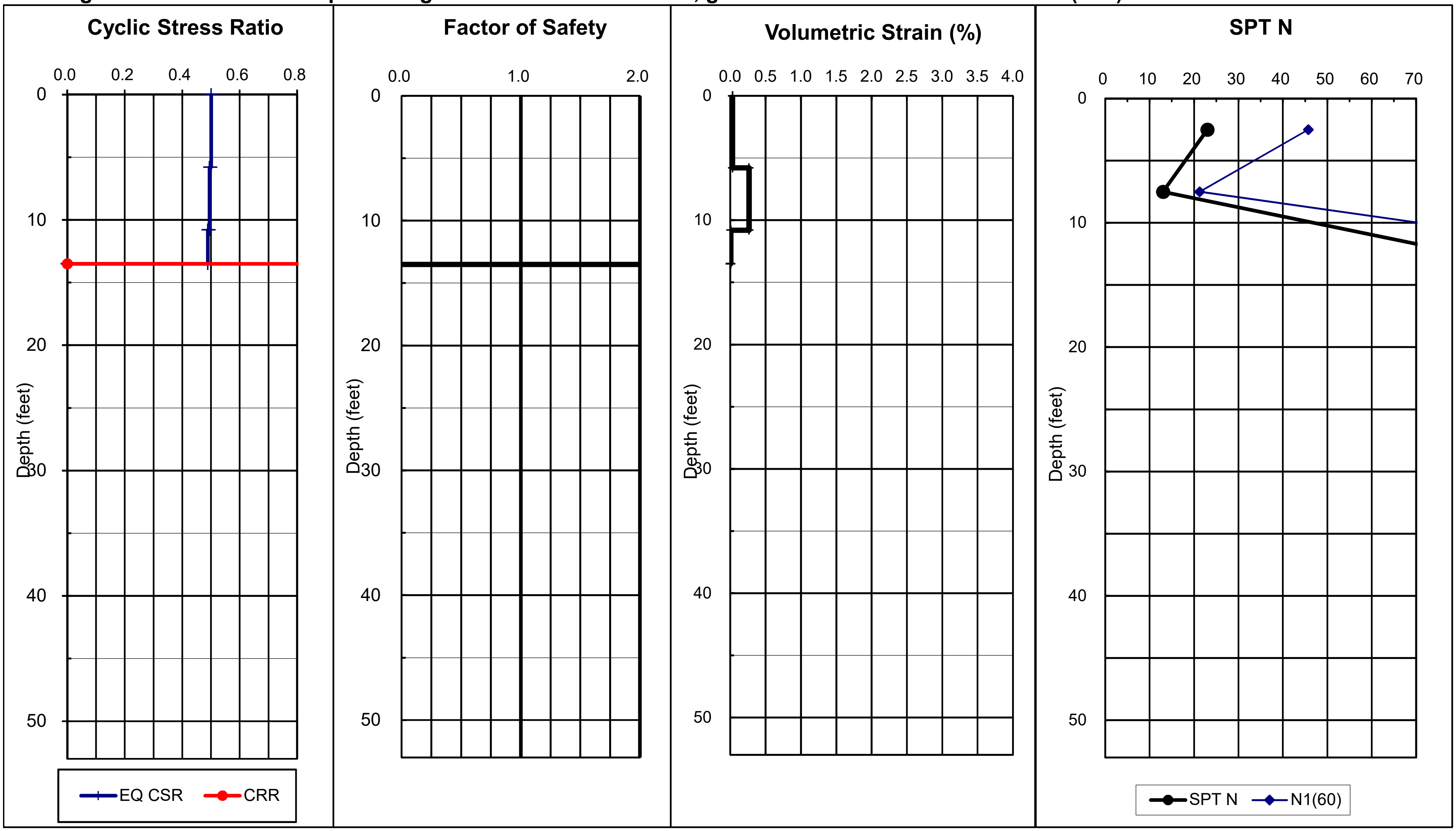
$SF = CRR_{7.5,1atm}/CSR^*$

EARTH SYSTEMS - EVALUATION OF LIQUEFACTION POTENTIAL AND INDUCED SUBSIDENCE

SWC Pierson Blvd & Cholla Drive, Desert Hot Springs **Project No: 10710-01** 1996/1998 NCEER Method

Ground Compaction Remediated to 5 foot depth

Boring: B-3 **Earthquake Magnitude: 8.2** **PGA, g: 0.62** **Calc GWT (feet): 50**



Total Thickness of Liquefiable Layers: 0.0 feet **Estimated Total Ground Subsidence: 0.2 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: 66-135 4th Street, Desert Hot Springs
Job No: 11280-01
Date: 9/7/2018
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: 0.62 0.77
MSF: 0.80
GWT: 50.0 feet
Calc GWT: 50.0 feet
Remediate to: 5.0 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: 0

Total (in.) Induced Subsidence: 0.2 upper 50 ft

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

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)	Tot.Stress at SPT po (tsf)	Eff.Stress at SPT p'o (tsf)
								0.000	
5.0	21	**50**	1	116	7	2.5	5.5	0.145	0.145
7.5	22	**14**	1	107	7	5.0	8.0	0.290	0.290
9.5	45	**28**	1	123	7	7.5	10.5	0.424	0.424
15.0	42	**26**	1	123	7	10.0	13.0	0.578	0.578
20.0	53	**33**	1	123	7	15.0	18.0	0.885	0.885
21.5		**26**	1	123	7	20.0	23.0	1.193	1.193

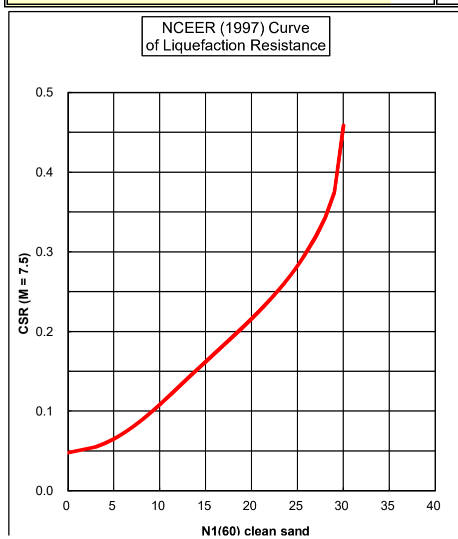
Base Depth (feet)	rd	C_N	C_R	C_S	$N_{1(60)}$	Rel. Dens. Dr (%)	*Trigger FC Adj.* $\Delta N_{1(60)}$	*Equiv. Sand* $N_{1(60)CS}$	Kσ	M = 7.5 Available CRR	M =7.5 Induced CSR*	**Liquefac. Safety Factor**
5.0	1.00	1.70	0.75	1.00	76.5	100	0.8	77.3	1.00	1.200	0.501	**Non-Liq.**
7.5	0.99	1.70	0.75	1.00	21.2	55	0.3	21.5	1.00	0.234	0.498	**Non-Liq.**
9.5	0.98	1.58	0.75	1.00	40.3	76	0.5	40.8	1.00	1.200	0.495	**Non-Liq.**
15.0	0.98	1.35	0.76	1.00	32.6	68	0.4	33.0	1.00	1.200	0.492	**Non-Liq.**
20.0	0.97	1.09	0.86	1.00	37.9	74	0.4	38.3	1.00	1.200	0.487	**Non-Liq.**
21.5	0.96	0.94	0.93	1.30	35.7	71	0.4	36.1	0.95	1.200	0.504	**Non-Liq.**

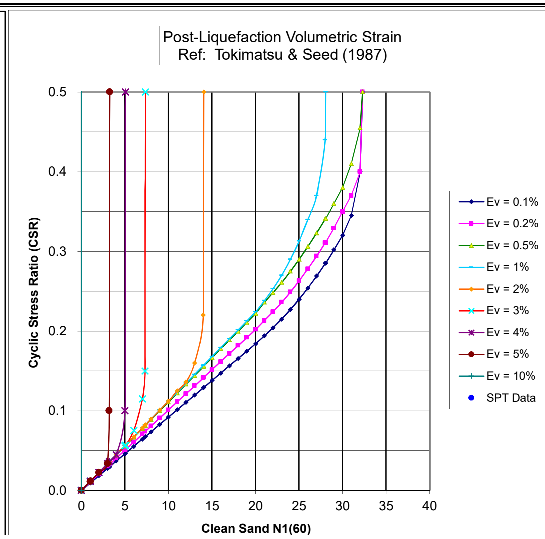
Base Depth (feet)	*Post FC Adj.* $\Delta N_{1(60)}$	$N_{1(60)CS}$	Volumetric *Strain (%)*	**Induced Subsidence (in.)**
5.0	0.8	77.3	0.01	**0.01**
7.5	0.3	21.5	0.23	**0.07**
9.5	0.5	40.8	0.06	**0.01**
15.0	0.4	33.0	0.10	**0.07**
20.0	0.4	38.3	0.08	**0.05**
21.5	0.4	36.1	0.10	**0.02**

SETTLEMENT (SUBSIDENCE) OF DRY SANDS

Nc = 22.5

p (tsf)	G_{max} (tsf)	τ_{av} (tsf)	Shear Strain γ	Strain E_{15}	Strain Enc	**Dry Sand Subsidence (in.)**
0.097	593	0.058	2.2E-04	4.4E-05	5.3E-05	**0.01**
0.194	548	0.115	1.1E-03	9.7E-04	1.2E-03	**0.07**
0.284	820	0.167	5.7E-04	2.4E-04	2.9E-04	**0.01**
0.387	891	0.226	7.7E-04	4.2E-04	5.1E-04	**0.07**
0.593	1,160	0.343	7.6E-04	3.5E-04	4.2E-04	**0.05**
0.799	1,320	0.456	8.8E-04	4.3E-04	5.2E-04	**0.02**





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

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

$C_N = (1\ \text{atm}/p'o)^{0.5}$, 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 = $6400{*}(p/1)^{(-0.6)}$

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

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

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

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

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)

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

Kσ = 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/Kσ

$CRR_{7.5}$ = (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))

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

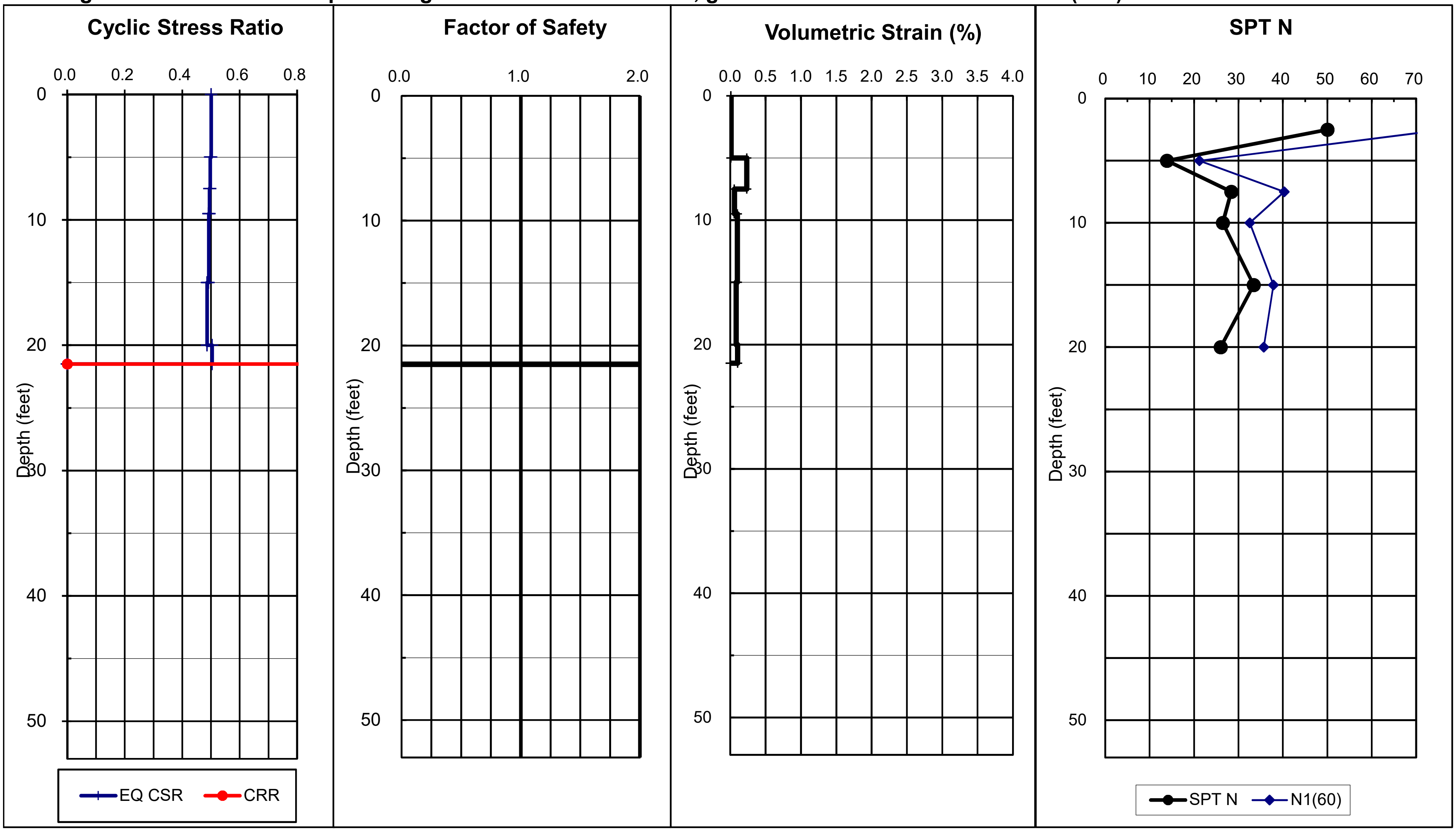
$SF = CRR_{7.5,1atm}/CSR^*$

EARTH SYSTEMS - EVALUATION OF LIQUEFACTION POTENTIAL AND INDUCED SUBSIDENCE

66-135 4th Street, Desert Hot Springs **Project No: 11280-01** 1996/1998 NCEER Method

Ground Compaction Remediated to 5 foot depth

Boring: B-1 **Earthquake Magnitude: 8.2** **PGA, g: 0.62** **Calc GWT (feet): 50**



Total Thickness of Liquefiable Layers: 0.0 feet **Estimated Total Ground Subsidence: 0.2 inches**

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

Project: 66-135 4th Street, Desert Hot Springs
Job No: 11280-01
Date: 9/7/2018
Boring: B-2 Data Set: 2

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: 0.62 0.77
MSF: 0.80
GWT: 50.0 feet
Calc GWT: 50.0 feet
Remediate to: 5.0 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	0.4

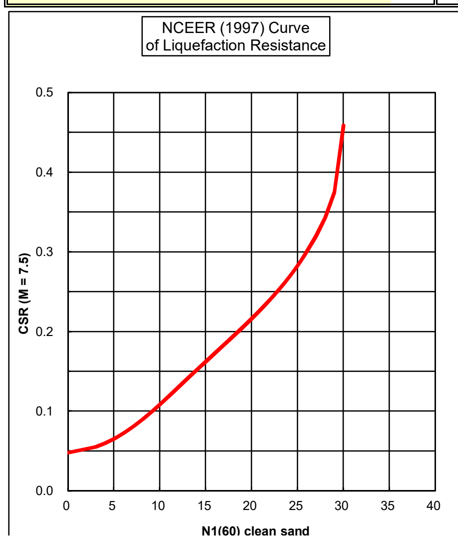
upper 50 ft

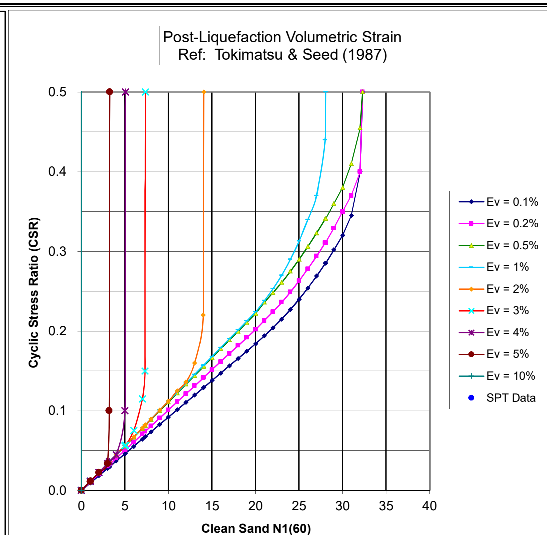
Required SF: 1.50
Minimum Calculated SF: #N/A

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)	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.)	p (tsf)	G_{max} (tsf)	τ_{av} (tsf)	Shear Strain γ	Strain E_{15}	Strain Enc	Dry Sand Subsidence (in.)
								0.000																								
3.0	24	50	1	116	7	1.0	4.0	0.058	0.058	1.00	1.70	0.75	1.00	76.5	100	0.8	77.3	1.00	1.200	0.502	Non-Liq.	0.8	77.3	0.01	0.00	0.039	375	0.023	1.7E-04	3.3E-05	3.9E-05	0.00
5.0	24	50	1	116	7	3.0	6.0	0.174	0.174	0.99	1.70	0.75	1.00	76.5	100	0.8	77.3	1.00	1.200	0.500	Non-Liq.	0.8	77.3	0.01	0.00	0.117	650	0.069	2.4E-04	4.7E-05	5.6E-05	0.00
7.0	19	12	1	107	7	5.0	8.0	0.290	0.290	0.99	1.70	0.75	1.00	18.3	51	0.3	18.6	1.00	0.201	0.498	Non-Liq.	0.3	18.6	0.34	0.08	0.194	522	0.115	1.3E-03	1.4E-03	1.7E-03	0.08
9.3	24	15	1	123	7	7.0	10.0	0.397	0.397	0.99	1.63	0.75	1.00	22.2	56	0.3	22.5	1.00	0.247	0.495	Non-Liq.	0.3	22.5	0.22	0.06	0.266	651	0.156	1.1E-03	9.3E-04	1.1E-03	0.06
15.0	35	22	1	123	7	10.0	13.0	0.582	0.582	0.98	1.35	0.76	1.00	27.0	62	0.4	27.4	1.00	0.328	0.492	Non-Liq.	0.4	27.4	0.15	0.11	0.390	841	0.228	9.4E-04	6.4E-04	7.7E-04	0.11
20.0	45	28	1	123	7	15.0	18.0	0.889	0.889	0.97	1.09	0.86	1.00	32.1	68	0.4	32.5	1.00	1.200	0.487	Non-Liq.	0.4	32.5	0.12	0.07	0.596	1,100	0.344	8.9E-04	5.0E-04	6.0E-04	0.07
25.0		26	1	123	7	20.0	23.0	1.197	1.197	0.96	0.94	0.93	1.30	35.6	71	0.4	36.0	0.95	1.200	0.505	Non-Liq.	0.4	36.0	0.10	0.06	0.802	1,322	0.458	8.8E-04	4.4E-04	5.2E-04	0.06
30.0		49	1	123	7	25.0	28.0	1.504	1.504	0.94	0.84	0.98	1.30	63.1	95	0.7	63.7	0.87	1.200	0.545	Non-Liq.	0.7	63.7	0.04	0.02	1.008	1,792	0.566	6.0E-04	1.5E-04	1.8E-04	0.02
31.5		35	1	123	7	30.0	33.0	1.812	1.812	0.92	0.76	1.00	1.30	41.7	77	0.5	42.2	0.81	1.200	0.574	Non-Liq.	0.5	42.2	0.08	0.02	1.214	1,715	0.667	8.5E-04	3.5E-04	4.2E-04	0.02





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

C_R = 0.75 for Rod lengths < 3m, 1.0 for > 10m

= min(1,max(0.75,1.4666-2.556/(z(ft))$^{0.5}$))

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

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

Kσ = 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/Kσ

$CRR_{7.5}$ = (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))

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

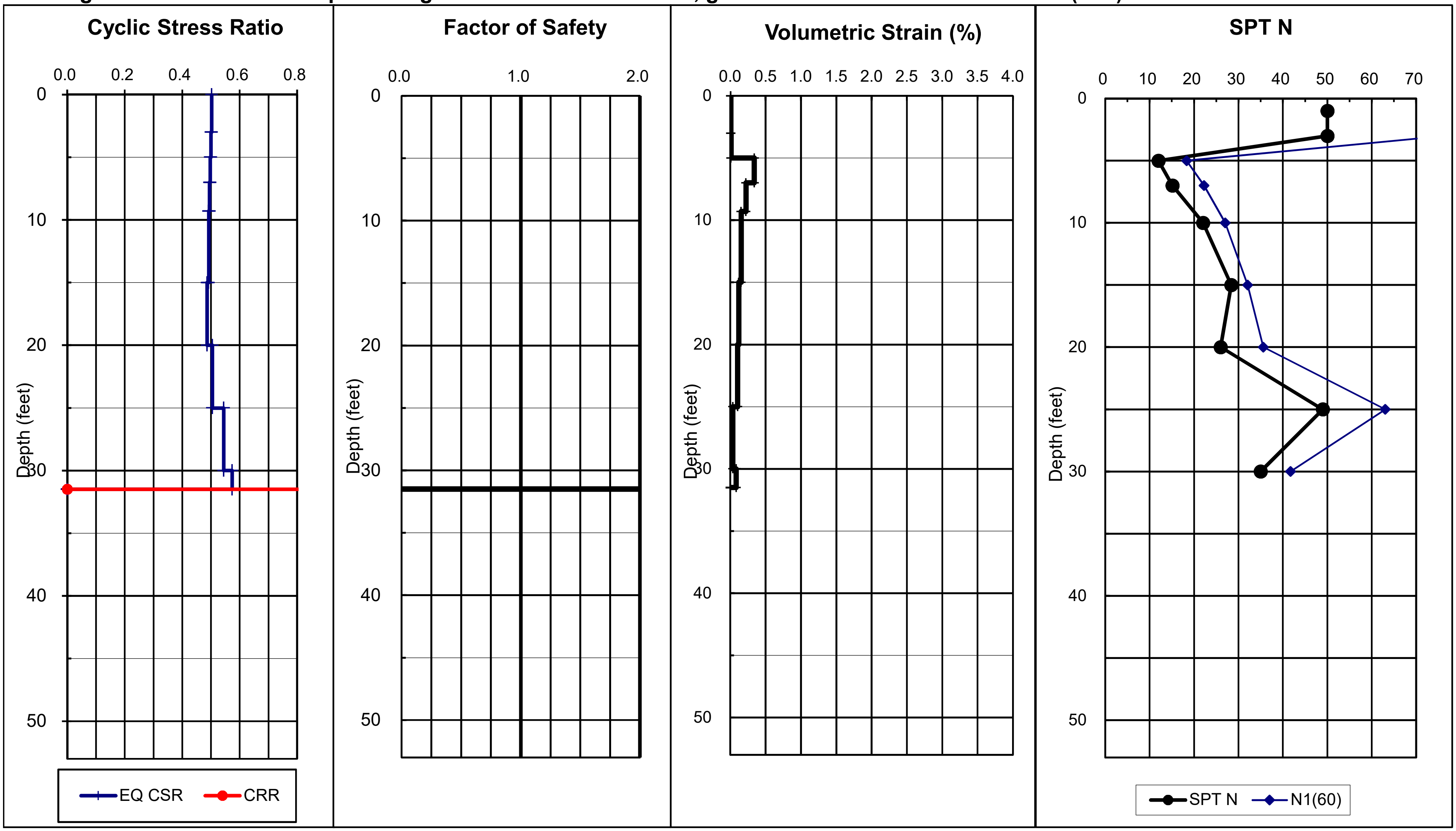
$SF = CRR_{7.5,1atm}/CSR^*$

EARTH SYSTEMS - EVALUATION OF LIQUEFACTION POTENTIAL AND INDUCED SUBSIDENCE

66-135 4th Street, Desert Hot Springs **Project No: 11280-01** 1996/1998 NCEER Method

Ground Compaction Remediated to 5 foot depth

Boring: B-2 **Earthquake Magnitude: 8.2** **PGA, g: 0.62** **Calc GWT (feet): 50**



Total Thickness of Liquefiable Layers: 0.0 feet **Estimated Total Ground Subsidence: 0.4 inches**

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

Project: 66-135 4th Street, Desert Hot Springs
Job No: 11280-01
Date: 9/7/2018
Boring: B-3 Data Set: 3

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: 0.62 0.77
MSF: 0.80
GWT: 50.0 feet
Calc GWT: 50.0 feet
Remediate to: 5.0 **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: 0

Total (in.) Induced Subsidence: 0.1
upper 50 ft

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

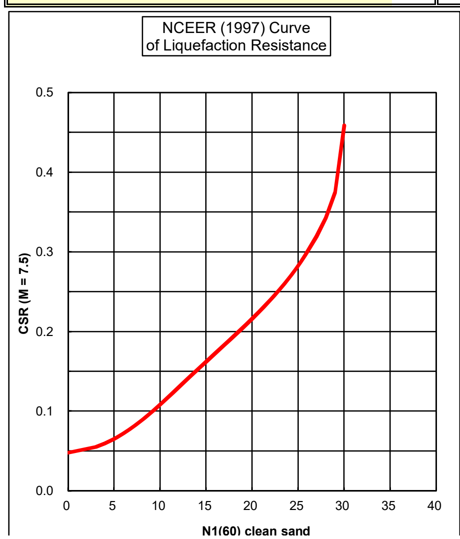
SETTLEMENT (SUBSIDENCE) OF DRY SANDS

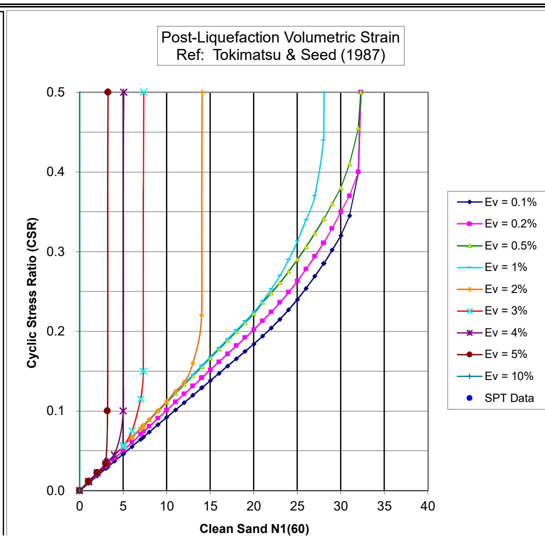
Nc = 22.5

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)
3.0	37	**50**	1	121	7	1.0	4.0
5.0	25	**50**	1	121	7	3.0	6.0
8.3	34	**21**	1	118	7	5.0	8.0
15.0	66	**42**	1	118	7	10.0	13.0
16.5	38	**24**	1	118	7	15.0	18.0

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																	
3.0	0.061	0.061	1.00	1.70	0.75	1.00	76.5	100	0.8	77.3	1.00	1.200	0.502	**Non-Liq.**	0.8	77.3	0.01	**0.00**
5.0	0.182	0.182	0.99	1.70	0.75	1.00	76.5	100	0.8	77.3	1.00	1.200	0.500	**Non-Liq.**	0.8	77.3	0.01	**0.00**
8.3	0.303	0.303	0.99	1.70	0.75	1.00	32.8	68	0.4	33.2	1.00	1.200	0.498	**Non-Liq.**	0.4	33.2	0.08	**0.03**
15.0	0.598	0.598	0.98	1.33	0.76	1.00	50.3	85	0.5	50.8	1.00	1.200	0.492	**Non-Liq.**	0.5	50.8	0.04	**0.03**
16.5	0.893	0.893	0.97	1.09	0.86	1.00	27.0	62	0.4	27.4	1.00	0.328	0.487	**Non-Liq.**	0.4	27.4	0.17	**0.03**

Base Depth (feet)	p (tsf)	G_{max} (tsf)	τ_{av} (tsf)	Shear Strain γ	Strain E_{15}	Strain Enc	**Dry Sand Subsidence (in.)**
3.0	0.041	383	0.024	1.7E-04	3.3E-05	4.0E-05	**0.00**
5.0	0.122	664	0.072	2.4E-04	4.8E-05	5.7E-05	**0.00**
8.3	0.203	647	0.120	6.4E-04	3.5E-04	4.2E-04	**0.03**
15.0	0.400	1,048	0.234	5.2E-04	1.7E-04	2.0E-04	**0.03**
16.5	0.598	1,042	0.346	1.1E-03	7.2E-04	8.7E-04	**0.03**





$N_{1(60)}$ = 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
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

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)
$N_{1(60)CS}$ = $N_{1(60)CS}$ + $\Delta N_{1(60)}$
Kσ = 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/Kσ
$CRR_{7.5}$ = (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 = $N_{1(60)CS}$

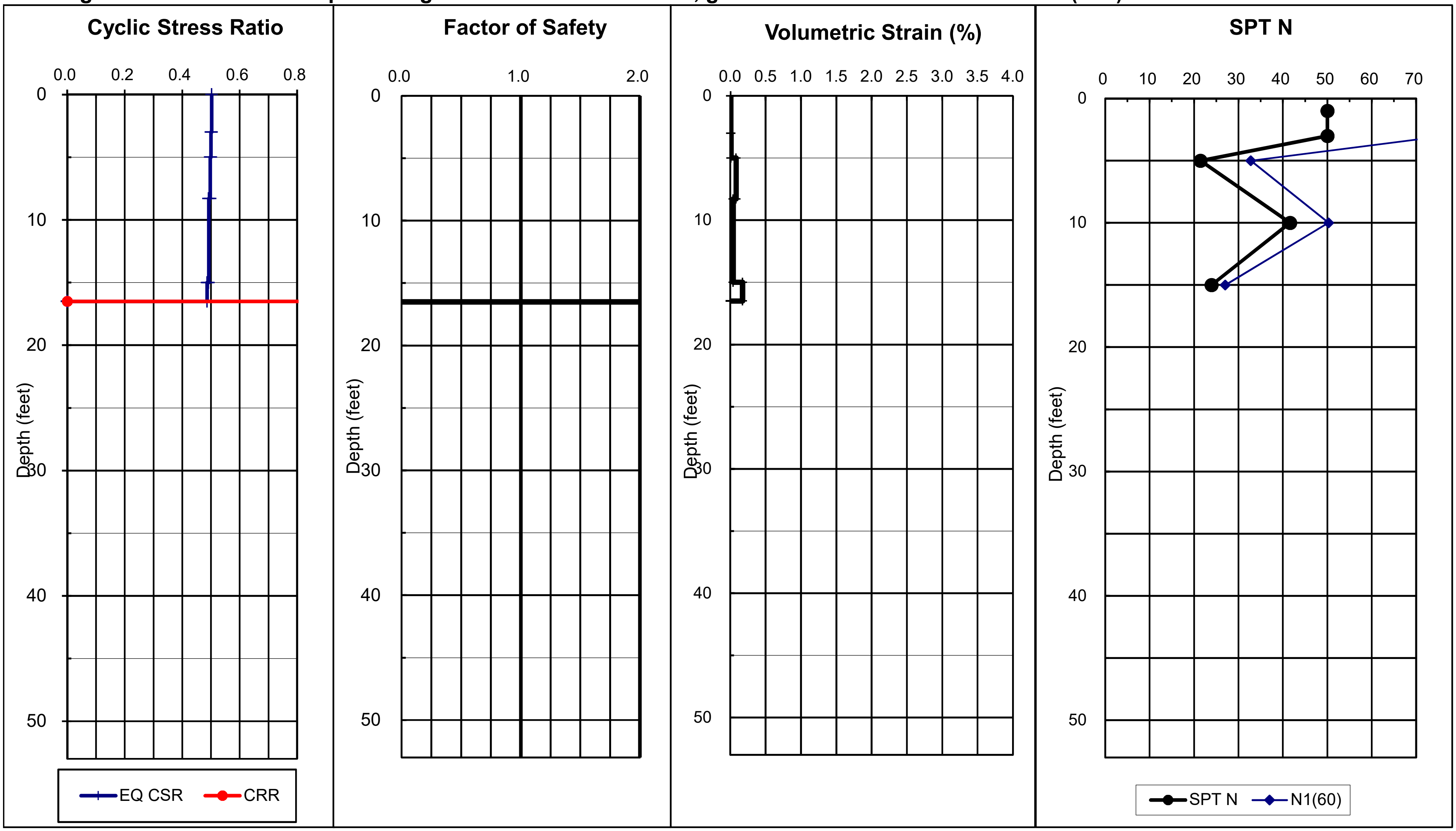
SF = $CRR_{7.5,1atm}$/CSR*

EARTH SYSTEMS - EVALUATION OF LIQUEFACTION POTENTIAL AND INDUCED SUBSIDENCE

66-135 4th Street, Desert Hot Springs **Project No: 11280-01** 1996/1998 NCEER Method

Ground Compaction Remediated to 5 foot depth

Boring: B-3 **Earthquake Magnitude: 8.2** **PGA, g: 0.62** **Calc GWT (feet): 50**



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