Appendix E

Geotechnical Engineering and Percolation Testing Report

Mr. Ahmad Zaki
45 Cinch Road
Bell Canyon, California 91307

Geotechnical Engineering and
Percolation Testing Report
Proposed Bamiyan Marketplace
15749 Grand Avenue
Lake Elsinore, Riverside County, California

January 17, 2019

File No.: 302169-002
Doc. No.: 19-01-707

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Mr. Ahmad Zaki
45 Cinch Road
Bell Canyon, California 91307

Attention: Mr. Ahmad Zaki

Subject: **Geotechnical Engineering and PercolationTesting Report**

Project: **Proposed Bamiyan Marketplace**
15749 Grand Avenue
Lake Elsinore, Riverside County, California

Earth Systems Pacific (Earth Systems) is pleased to submit this geotechnical engineering and percolation report for the referenced project located on the northwest corner of Grand Avenue and Ortega Highway in the city of Lake Elsinore, 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 (PER-18-3-007AR) with an authorization date of May 28, 2018. Other geotechnical related services that may be required, such as plan reviews, responses to agency inquiries, and grading observation and testing are additional services and will be billed according to the Fee Schedule in effect at the time services are provided. Unless requested in writing, the Client is responsible to distribute the report to the appropriate governing agency and other members of the design team. Please review the Limitations (Section 6) of this report as they are vital to the understanding of this report.

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

Respectfully submitted,
Earth Systems Pacific

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15749 Grand Avenue
Lake Elsinore, Riverside County, California

Section 1
INTRODUCTION

1.1 Project Description

This geotechnical engineering and percolation testing report has been prepared for the proposed Bamiyan Marketplace development located at the northwest corner of Grand Avenue and Ortega Highway (Highway 74) in the city of Lake Elsinore, Riverside County, California, see Plate 1 (Site Vicinity Map). We understand the property is proposed to be developed as mixed-use with residential, retail, and commercial purposes. Based upon the Preliminary Site Plan provided, seven commercial structures (including a gas station) and associated improvements will encompass the southern two-thirds of the site. Approximately 20 residential units are proposed for the northern one-third of the site. Appurtenant site work is anticipated to include underground utilities, Water Quality Management Plan (WQMP) improvements, hardscape, parking drive improvements, a 6 foot high retaining wall along the western property line boundary slope, and landscaping. We have assumed site grades will be similar in elevation to the surrounding street grades (+-5 feet). The proposed site layout along with our exploration locations is presented in Plate 2.

We have assumed masonry, wood-framed or metal construction founded on shallow permanent foundations, and there will be no below grade basement levels. Column loads are anticipated not to exceed approximately 90 kips for spread footings and 5 kip/LF for continuous footing loads. As the basis for the foundation recommendations, all loading is assumed to be dead plus actual live load.

No preliminary design loading was provided by the structural engineer. If actual structural loading exceeds these assumed values, we will need to re-evaluate the given recommendations.

1.2 Site Description

The project is located on the northwest corner of Grand Avenue and Ortega Highway in the city of Lake Elsinore. The site has an approximate latitude and longitude of 33.6591°N/117.3782°W. The project contains two legal lots (APN 381-320-020 and 381-320-023) and they are currently vacant. The area of the parcels is approximately 7.91 acres and 4.64 acres for APN 381-320-020 and 381-320-023, respectively. The site is bounded by Grand Avenue to the northeast, Ortega Highway to the southeast, Macy Street to the northwest, and residential developments atop an approximately 15-foot slope to the southwest. The 15-foot ascending slope is within the property boundary. It is also our understanding that there is a utility easement through the property for an existing underground storm drain (see Plate 2). We estimate depths on the order of 10 feet deep for the storm channel/drain system. From google imagery, the elevation at the project site varies from approximately 1,300 to 1,320 feet above Mean-Sea-Level (MSL). Drainage appears to be by sheet flow to the northeast. The site is approximately 1400 feet from the current shoreline of Lake Elsinore which is at approximate elevation 1,245 feet.

1.3 Site Reconnaissance

Earth Systems personnel visited the site on various days from June to December 2018. Earth Systems personnel also reviewed select historic aerial photographs of the project site. Historical aerial photographs (Google Images, "Historic Aerials" between 1938 and 2018, and stereo photographs on file with the County of Riverside Flood Control District) revealed items of interest. Based on our review of these historical photographs, it is our opinion that agricultural activities began at the site prior to 1962. The site underwent significant grading between 1984 and 1990 resulting in variable fill and cut thickness across the site. The site has remained relatively unchanged from 1990 to 2018 based on our review of aerial photos.

1.4 Purpose and Scope of Services

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

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

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

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

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

Each of our proposed field exploration locations was located and marked in the field and cleared with known utility lines as identified by Underground Service Alert (USA), "Dig Alert". Our exploration locations were located in the field by consumer grade Global Positioning System (GPS) accurate to ± 15 feet in conjunction with pacing based upon the control provided or sighting from landmarks identified on the project topographic map.

<u>Task 3 – Field Exploration</u>

We evaluated the general subsurface conditions at the site by drilling fourteen small diameter borings, from approximately 11½ feet to 50½ feet in depth, excavating four test pits and two fault trenches. The field exploration also included a visual site reconnaissance of the project area and immediate surroundings. Plate 2 shows the approximate location of each boring, test pit, fault trench and the percolation test locations. The fault trench locations were surveyed by Inland Empire Survey & Engineering, Inc.

Task 4 – Laboratory Testing

Laboratory tests were performed on selected samples to evaluate the physical characteristics of the materials encountered during our field exploration. Laboratory testing included moisture content, dry unit weight, maximum dry density/optimum moisture content, sieve analysis, consolidation/collapse potential, Expansion Index, and R-value. The testing was performed in general accordance with American Society for Testing and Materials (ASTM) or appropriate test procedures. Selected samples were also tested for a preliminary screening level of corrosion potential (pH, electrical resistivity, water-soluble sulfates and water-soluble chlorides). Earth Systems does not practice corrosion engineering; however, these test results may be used by a qualified engineer in designing an appropriate corrosion plan for the project.

Task 5 – Percolation Testing

Five borings were drilled within the proposed stormwater infiltration locations, as designated by Inland Empire Survey & Engineering, Inc. for percolation testing. These holes were drilled on December 10, 2018 with the same drill rig as the exploration borings. Plate 2 shows the approximate location of each test.

Task 6 – Analysis and Report

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

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

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

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 14 exploratory borings. The borings were drilled to depths ranging from approximately 11½ to 50½ feet below existing grades using a Mobile B-61 truck-mounted drill rig equipped with 6-inch hollow-stem augers provided by Cal-Pac Drilling of Calimesa, California. The borings were advanced to observe soil profiles and obtain samples for laboratory testing. The approximate boring locations are shown on Plate 2, in Appendix A. The locations shown are approximate, established by consumer grade Global Positioning System (GPS) accurate to ± 15 feet in conjunction with pacing based upon the control provided.

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

Design parameters provided by Earth Systems in this report have considered an estimated 70% hammer efficiency based on data provided by the drilling subcontractor and limits per SP117A. 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 roughly correlated to SPT N-values using a conversion factor that may vary from about 0.5 to 0.7. In general, a conversion factor of approximately 0.63 from a study at the Port of Los Angeles (Zueger and McNeilan, 1998 per SP 117A) is considered satisfactory. A value of 0.63 was applied in our calculations for this project.

Bulk samples of the soil materials were obtained from the drill auger cuttings, representing a mixture of soils encountered at the depths noted. The depth to groundwater, if any, was measured in the boreholes. Following drilling, sampling, and logging, the borings were backfilled with the cuttings and tamped upon completion. Where water was encountered, borings were sealed with bentonite. 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 on the samples obtained during the subsurface exploration. The final logs are included in Appendix A of this report. The stratification lines represent the approximate boundaries between soil types, although the transitions may be gradual. In reviewing the logs and legend, the reader should recognize the legend is intended as a guideline only, and there are a number of conditions that may influence the soil characteristics observed during drilling. These include, but are not limited to cementation, variations in soil moisture, presence of groundwater, and other factors.

The boring logs present field blow counts per 6 inches of driven embedment (or portion thereof) for a total driven depth attempted of 18 inches. The blow counts on the logs are uncorrected (i.e. not corrected for overburden, sampling, etc.). Consequently, the user must correct the blow counts per standard methodology if they are to be used for design and exercise judgment in interpreting soil characteristics, possibly resulting in soil descriptions that vary somewhat from the legend.

Test Pit Excavations and Compaction Tests

Four test pits were excavated using a rubber-tire backhoe to approximate depths of 15 to 18 feet below the existing ground surface. The test pits were advanced to observe soil profiles for estimating soil ages and anticipated depths for the fault hazard exploration trenching. The approximate test pit locations are shown on Plate 2, in Appendix A. In addition, compaction tests (ASTM D 6938) were taken on the western slope face to obtain density data. A demarcation line between higher and lower densities was seen about mid-slope height with higher densities seen in the suspected fill over the native cut. Densities for each zone were averaged and are shown on the slope stability output.

Fault Trench Excavations

The Willard fault has been mapped to cross or come very close to the subject site. The Willard fault is not currently considered to be an active fault (movement within the last 11,000 years). However, the County of Riverside has designated that most of the site is within a special study zone for faulting. The City of Lake Elsinore has subcontracted a private geotechnical firm (NV5) to perform geologic reviews for this project. Prior to performance of fault hazard exploration by trenching, Earth Systems collaborated with NV5's geologist to develop an exploration program. NV5 concurred that an exploration program by geologic trenching is necessary to determine if active faults exist within the site. Prior to excavating the exploration trench, the location for the proposed trench was presented to NV5 for their review and concurrence.

This main fault trench is identified as T-1 and a small secondary fault trench (T-2) was excavated adjacent to T-1. The main exploration trench extended in a northeast to southwest direction, generally perpendicular to the regional fault trends and extended across most of the site, excluding the ascending graded slope and the Grand Avenue easement. The trench was excavated with a large excavator and was benched for OSHA compliance. The depth was approximately 10 feet. Trench walls were scraped to remove loose soil and expose the geologic strata. The trench walls were allowed to weather to allow for more subtle features to be revealed. Graphic logs of the exposed materials were prepared by our certified engineering geologists and are included in Appendix A.

The T-2 fault trench was performed for clarifying some of the geologic features observed in T-1. The T-1 fault trench was excavated to an approximate depth of 10 feet, where-as T-2 was excavated to a depth of approximately 5 feet. An engineering geologist from NV5 performed a site visit (September 9, 2018) after completion of the trenching and geologic logging to review the exposed geologic units and discuss our conclusion that no faulting was observed. The units exposed consisted of minor fill, alluvium and older alluvium. The older alluvium exposed in both trenches is in our opinion older than 11,000 years based on the development of paleo B soil horizons and within the older alluvium. The NV5 geologist verbally indicated that in general he agreed with our opinion that the older alluvium exposed in the fault trenches is older than 11,000 years and no evidence of faulting was observed.

Percolation Test Holes

Five shallow borings were drilled within the proposed stormwater infiltration locations for percolation testing. These holes were drilled on December 10, 2018 with the same drill rig as the exploration borings. Test holes reached depths of approximately 5 feet bgs. Percolation testing was performed in general accordance with the Riverside County Design Handbook for Low Impact Development Best Management Practices (Riverside County Flood Control and Water Conservation District, 2011). We installed 3-inch diameter perforated pipe along the entire length of the test holes. Then we backfilled the holes between the pipe and borehole sidewalls with clean gravel. After the gravel placement, we inundated the borehole with clean potable water. The percolation locations are shown on the Boring Location Map (Plan View), Plate 2, in Appendix A. The locations shown are approximate, established by pacing and line-of-sight bearings from adjacent landmarks and consumer grade GPS coordinates (+/- 15 feet). Refusal and groundwater were not encountered at the test hole locations.

Staff from Earth Systems maintained a log of the subsurface profile encountered in the test zone and performed visual observation of the soils. Subsurface conditions encountered were categorized and logged in general accordance with the Unified Soil Classification System [USCS] and ASTM D 2487 and 2488 (current edition).

2.2 Laboratory Testing

Samples were reviewed along with field logs to select those that would be analyzed further. Those selected for laboratory testing include, but were not limited to, soils that would be exposed and those deemed to be within the influence of the proposed structures. Test results are presented in graphic and tabular form in Appendix B of this report. Testing was performed in general accordance with American Society for Testing and Materials (ASTM) or other appropriate test procedure. Selected samples were also tested for a screening level of corrosion potential (pH, electrical resistivity, water-soluble sulfates, and water-soluble chlorides). Earth Systems does not practice corrosion engineering; however, these test results may be used by a qualified corrosion engineer in designing an appropriate corrosion control plan for the project.

Our testing program consisted of the following:

- Density and Moisture Content of select samples of the site soils (ASTM D 2937 & 2216).
- Maximum Dry Density/Optimum Moisture Content tests to evaluate the moisture-density relationship of typical soils encountered (ASTM D 1557).
- Particle Size Analysis to classify and evaluate soil composition. The gradation characteristics of selected samples were made by sieve analysis procedures (ASTM D 6913).
- Plasticity Index in accordance with ASTM D 4318.
- 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).
- Screening Level Chemical Analyses (Soluble Sulfates and Chlorides (ASTM D 4327), pH (APHA 2320-B), and Electrical Resistivity/Conductivity (ASTM G 187) to evaluate the potential for adverse effects of the soil on concrete and steel.
- R-Value for pavement section analysis (CTM 301).

Section 3
DISCUSSION

3.1 Soil Conditions

The field exploration indicates that site soils consist predominantly of alluvial type soils of silty sand with lesser poorly graded sand and clayey sand (Unified Soils Classification System symbols of SM, SP, SP-SM, and SC) to the maximum depth of exploration of 50½ feet below the ground surface. Fill, which appears to be locally derived and undifferentiated from the alluvium, overlies the alluvium and is variable in thickness up to approximately 5 feet. The boring logs provided in Appendix A includes more detailed descriptions of the soils encountered. Site soils are classified as Type C in accordance with Cal OSHA.

3.2 Groundwater

Free groundwater was encountered during our field exploration at approximately 28 and 47.5 feet bgs (maximum drill depth 50½ feet). Significant perched moisture conditions were encountered in various areas within site soils in the form of those soils at or near saturation (based on % calculation). Free water is defined as visible excess water on or in the sample of sample collection devices. Perched moisture was variable in depth.

Based on calculation of percent saturation of soil samples tested considering moisture content and density, isolated zones of increased moisture were observed. The perched water appears only to be impeding the downward migration of water, but does not appear to be mounding it. This is due to the non-observation of saturated, free water above high moisture content zones, and the observation of soils with significantly less moisture and percent saturation above these high moisture zones. The perched water also does not appear to be laterally continuous as seen by the variability of moisture content in our borings in the area despite ongoing irrigation of adjacent properties for at least 50 years. The perched conditions are likely a result of farming and irrigation throughout the years. The boring logs in Appendix A present locations of calculated near saturated or saturated conditions, shown as "very moist" or "wet".

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

- Well No. 06S05W02A001S is located approximately 1.8 miles northeast of the project site. The surface elevation of this well is approximately 1,277 feet and the groundwater readings as measured from 2011 to 2018 varied from 1,000 to 1,076 feet above mean sea level.
- Well No. 06S04W19F001S is located approximately 2.3 miles southeast of the project site. The surface elevation of this well is approximately 1,288.5 feet and the groundwater readings as measured from 2012 to 2018 varied from 1,249.5 to 1,267.5 feet above mean sea level.

Based on the above data, groundwater is not anticipated to be encountered during construction. Based on the fault trench study, mottled soil conditions suggestive of past shallow groundwater were observed as shallow as 5 feet deep, however conditions were variable. The historic groundwater depth is estimated to be approximately 5 feet deep at the site based on the fault trench study. Fluctuations of the groundwater level and localized zones of increased soil moisture content should be anticipated during and following the rainy season or from irrigation.

3.3 Collapse Potential/Consolidation Potential

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

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

Table 1
Collapse Potential Values

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

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

Table 2
Collapsible Soil Hazard Ranking System

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

The results of collapse potential tests performed on nine selected samples from depths ranging from 5 to 20 feet below the ground surface indicated a collapse potential on the order of 0.4 to 2.4 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 of the results of the 9 tests indicates that collapse potential is generally less than 2% when the dry density is greater than 109 pcf (relative to ASTM D 1557), and generally less than 1% when the dry density is greater than 121 pcf (relative to ASTM D 1557).

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

1. Soils are generally granular in nature and no significant cementation was observed. Older alluvial soils with high blow counts predominate at the site: however low blow count, and lower density layers exist, with predominate voids in the upper 5 feet which are less significant with depth.
2. High dry densities (DD > 109 pcf) of the soils determined during the laboratory testing generally had lower potential for collapse (less than 2%).
3. Collapsible soils were generally classified as Silty Sand (SM).
4. Soil collapse at the site appears to be directly related to in-place density (relative compaction) which exists in site soils in the upper approximately 5 to 10 feet.

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

Based on the above criteria and our field and laboratory findings, we estimate there is a "Moderate" collapse potential from soil layers between 0 and 10 ft below the ground surface (bgs). Without collapse mitigation efforts, the collapse potential is variable in the borings and layers but up to approximately 0.9 inches. Assuming the recommended grading is accomplished according to Section 5.1 of this report, we estimate the collapse potential differential settlement is building structure areas 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 potential of the on-site soils tested is generally "very low" as defined by ASTM D 4829 and the 2016 California Building Code.

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

3.5 Corrosion Potential

Two samples of the near-surface soils were tested for potential corrosion of concrete and ferrous metals. Soils in the upper 0 to 5 feet were tested as a blended (composite) sample. 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 pH values ranging from 7.2 to 7.3, chloride contents of 2.9 to 6.4 ppm, sulfate contents of 2.7 to 16 ppm and minimum resistivity's of 13,200 to 17,200 Ohm-cm. Although Earth Systems does not practice corrosion engineering, the corrosion values from the soil tested are normally considered as being "very mildly" corrosive to buried metals and as possessing a "negligible" exposure to sulfate attack for concrete as defined in American Concrete Institute [ACI] 318, Section 4.3. 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 Stormwater Percolation Testing

As indicated in Section 2.1 of this report, five test holes were drilled using the same drill rig as the exploration borings. Test holes were excavated on December 10, 2018 and reached depths of approximately 5 feet below the ground surface. These test locations represent the soils at the assumed bottom of the proposed infiltration systems. The percolation boring locations are shown on the Boring Location Map (Plate 2), in Appendix A.

The presence of gravel and the PVC pipe (inserted in the boring) were accounted for in the percolation test results. The borings were pre-saturated with potable water at least 24 hours prior to testing and again immediately prior to testing. Test results were taken with a water surface at approximate depths between 3 and 5 feet below existing grade at the test location, respectively (see Table 5).

Test procedures followed the procedures for deep boring percolation testing according to the Riverside County Flood Control and Water Conservation District Design Handbook for Low Impact Development Best Management Practices, September 2011. The soils encountered at each test location and the percolation rates as well as empirically correlated infiltration rate are presented in Table 5. A factor of safety of 3 in accordance with the Riverside County Manual (2011) was applied to the tested empirical infiltration rate in order to determine the design infiltration rate.

Table 5
Infiltration Rate Results

Test	Soil Condition	USCS Soil Description in Test Zone	Test Zone Below Existing Surface (feet)	Percolation Rate (min/in)	Porchet Empirical Infiltration Rate (in/hr)	Design Infiltration Rate (in/hr) (FOS = 3)
P-1	(Native)	Silty Sand (SM)	3.5 to 5	12.5	0.33 in/hr	0.11 in/hr
P-2	(Native)	Silty Sand (SM)	3 to 5	8.1	0.54 in/hr	0.18 in/hr
P-3	(Native)	Silty Sand (SM)	3.5 to 5	14.9	0.32 in/hr	0.11 in/hr
P-4	(Native)	Silty Sand (SM)	3 to 5	18.5	0.23 in/hr	0.08 in/hr
P-5	(Native)	Silty Sand (SM)	3.5 to 5	10.1	0.47 in/hr	0.16 in/hr

3.7 Geologic Setting

Regional Geology: The site is located within the Elsinore Trough, which in turn is located within a larger structural block known as the Perris Block. The Perris Block, which is a part of the Peninsular Ranges Geomorphic Province, is bounded on the northeast by the San Jacinto fault, on the north by the Cucamonga fault, and on the southwest by the Santa Ana Mountains.

Local Geology: The Elsinore Trough has been filled with up to approximately 2,300 feet of alluvial materials of sand, silty sand, clayey, silt and clay. The site is underlain with younger and older alluvial materials which consist of slightly consolidated to weakly cemented silty sand, clayey sand, and poorly graded sand. Morton and Weber (2003), has identified these alluvial units as younger alluvial valley deposits which overlie the older alluvial fan deposits. The older alluvial fan deposits are late Pleistocene. The site is near the Santa Ana Mountains located west of Lake Elsinore which are generally comprised of granitic bedrock. In Borings B-1 and B-2 it appears that highly weathered granitic bedrock may have been encountered near the bottom of each boring. Due to the small samples obtained, the material could also be highly weathered cobble or boulder. The depth of the granitic rock was 25 feet and 45 feet, respectively. The "granitic bedrock" was only encountered in these two borings and it appears the contact between older alluvium and granitic bedrock could be highly variable, if it exists at these locations.

Within the exploration trench T-1, older alluvium was exposed. The presence of poorly to moderately developed paloesols is indicative of a pre-Holocene age, confirming the Pleistocene or pre-Holocene designation.

3.8 Geologic Hazards

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

3.8.1 Seismic Hazards

Seismic Sources: Several active faults or seismic zones lie within 40 miles of the project site as shown on Table A-1 in Appendix A. The primary seismic hazard to the site is strong ground shaking from earthquakes along the Elsinore, Chino, Whittier and San Jacinto fault zones.

Surface Fault Rupture: The project site does not lie within a currently delineated State of California, Alquist-Priolo Earthquake Fault Zone (CGS, 2018). Well-delineated fault lines cross through this region as shown on California Geological Survey [CGS] maps (Jennings, 2010), a copy of a portion of this map is attached in Appendix A). The Willard fault, a segment of the Elsinore fault zone is mapped through the edge of this site, close to or under Grand Avenue. The Willard fault has not been identified by the State of California as an Active fault. The main or primary Elsinore fault (Glen Ivy) is mapped approximately 0.2 miles northeast of the site. The closest Alquist-Priolo (A-P) Special Studies Zone is approximately 2.75 miles north of the site and the A-P Special Studies Zone for the Wildomar fault is located approximately 3.4 miles southeast of the site. However, the County of Riverside has identified almost the entire site as a special study zone for faulting, so the fault trenching performed is intended to address the Riverside County Special Study requirement, as well as the requirements by NV5.

Based on our lineament analysis and fault trench observations, it our professional opinion that "active" fault rupture has not occurred within the subject site. Previous fault trenching by Lewis S. Lohr & Associates (1978) on the property immediately northwest of the project, also did not encounter evidence of active faulting across a previously mapped trace of the Willard fault. While fault rupture generally occurs on previously known faults, there is no guarantee that future fault rupture will not occur at other locations. Fault trench logs are presented in Appendix A. NV5 was also onsite to observe the weathered trench and discuss, for concurrence of, our finding of no active fault rupture.

Lineament Analysis (Aerial Photograph Review): A lineament analysis was performed for this site by reviewing historical aerial photographs from Google Earth, Historical Aerials website and stereo photographs on file with the Riverside County Flood Control District. The exact photographs reviewed are listed in the References Section of this report. Based on our review of these historical photographs, it is our opinion that agricultural activities began at the site prior to 1962. The site underwent significant grading between 1984 and 1990 resulting in removal of soil in the southeast (south) corner, the cuts taper northward and westward. On the eastern side they taper from the maximum in the south corner to natural at about Grand Avenue. On the west side, along Macy Street the maximum cut is within the western corner and tapers to about natural grade about midway between the property line and Grand Avenue.

A storm drain was observed on an image that was reviewed at Riverside County Flood Control District. The storm drain coincides with 2 manholes observed on site. The grading on site may have been necessary to install this storm drain, identified as Ortega Channel (laterals A and A-1); or the site may have been used as a borrow site to achieve grades for the development to the southwest. Depths of the channel are estimated to be on the order of up to approximately 10 feet below existing grades.

No evidence of lineaments, suggestive of faulting was noted on the reviewed photographs.

Historic Seismicity: The site is located within an active seismic area in southern California where large numbers of earthquakes are recorded each year. Many of the major historic earthquakes felt in the vicinity of western Riverside County 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.

Over 11,000 recorded earthquakes (mostly small earthquakes) have occurred within 30 miles of the Lake Elsinore area since 1931 (Homefacts website, 2019). Approximately 40 historic earthquakes of magnitude 5.5 or greater have occurred within 65 miles of the site usually originating on or near the San Andreas, San Jacinto, or Elsinore faults. These include the 1812 Wrightwood, 1894 Lytle Creek, 1899 San Jacinto, 1910 Elsinore (Glen Ivy, Hot Springs), 1918 San Jacinto, and 1923 North San Jacinto earthquakes.

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

Historically, the San Andreas fault is responsible for two of the three great earthquakes experienced in the southern California area. These are the 1812 Wrightwood and the 1857 Fort Tejon earthquakes. The 1857 rupture extended along the San Andreas fault from Parkfield to Cajon Pass and was felt throughout most of California. While the epicenter of this earthquake is assumed to be located near Parkfield, California, approximately 180 miles northwest of Lake Elsinore, the fault rupture extended southeastward to the vicinity of Cajon Pass, just 44 miles northeast of the site. No significant earthquakes or fault movements have been attributed to this segment of the San Andreas fault since 1857. A great earthquake that occurred in 1812 near Wrightwood in the eastern San Gabriel Mountains also originated on the nearby San Andreas fault.

The 1899 San Jacinto earthquake, although not well located due to poor documentation at the turn of the century, was estimated to have had a local magnitude of approximately 6.5. Significant damage to structures in San Jacinto and Hemet occurred, especially to unreinforced brick or adobe buildings. This earthquake is thought to have originated from fault rupture along the San Jacinto fault.

In 1910, the large Glen Ivy Hot Springs (Elsinore) earthquake occurred near Lake Elsinore. Estimated to have had a local magnitude of approximately 6, this earthquake was preceded by two foreshocks and did damage to structures in Wildomar, Corona, and Temescal. The earthquake was felt in San Diego and Los Angeles. The causative fault is thought to be the Elsinore fault, a fault with no other documented historic earthquakes of magnitude 6 or greater.

The 1918 San Jacinto earthquake again shook the towns of San Jacinto and Hemet where most of the damage occurred. This local magnitude 6.8 earthquake caused significant cracking to roadways, canals, and the ground. Landslides were common. The San Jacinto fault was the causative fault.

In 1923, a magnitude 6.2 earthquake occurred along the northern portion of the San Jacinto fault zone. The towns of San Bernardino and Redlands were most affected. Most damage was minor, although the San Bernardino Hospital and Hall of Records were significantly damaged.

The 1933 Long Beach earthquake was the result of a 6.4 magnitude earthquake on the Newport-Inglewood fault zone near present day Huntington Beach. Most damage occurred to unreinforced masonry buildings including many school buildings.

The 1971 San Fernando earthquake resulted in extensive damage to structures in parts of San Fernando and the Santa Clarita Valley. The epicenter of the earthquake was located near Soledad Junction approximately 60 miles northwest of the site. Strong motion accelerographs recorded ground accelerations as high as 1.25g at Pacoima Dam near the epicenter of the earthquake. Some structures designed in accordance with the Building Code in affect at the time were extensively damaged.

The 1987 Whittier Narrows earthquake shook the Corona area for several seconds. The epicenter of this 5.9 magnitude earthquake, located near Monterey Park, was approximately 32 miles northwest of the site. This earthquake occurred on an unsuspected seismogenic feature known as a buried, or "blind", thrust fault underlying the Elysian Park-Montebello Hills area.

The major 1992 Landers/Big Bear earthquakes also shook the Corona area. Damage was minimal. This earthquake was generated by a system of strike-slip faults in the mountain and desert areas over 69 miles northeast of the site.

The 1994 Northridge earthquake and related aftershocks significantly shook the Corona area. Like the Whittier Narrows earthquake, this event was produced by a buried thrust fault that underlies portions of the San Fernando Valley and the Santa Susana Mountains. No actual fault rupture associated with the main thrust faulting occurred at the surface. Primary fault rupture terminated approximately 3 to 4.3 miles (5 to 7 km) below the ground surface.

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. Recent estimates suggest a nearly 98% probability of a nearby 5.0 in the next 50 years.

Soil Liquefaction and Lateral Spreading: Liquefaction is the loss of soil strength from sudden shock (usually earthquake shaking), causing the soil to become a fluid mass. Liquefaction describes a phenomenon in which saturated soil loses shear strength and deforms as a result of increased pore water pressure induced by strong ground shaking during an earthquake.

Dissipation of the excess pore pressures will produce volume changes within the liquefied soil layer, which can cause settlement. Shear strength reduction combined with inertial forces from the ground motion may also result in lateral migration (lateral spreading). Factors known to influence liquefaction include soil type, structure, grain size, relative density, confining pressure, depth to groundwater, and the intensity and duration of ground shaking. Soils most susceptible to liquefaction are saturated, loose sandy soils and low plasticity clay and silt.

In general, for the effects of liquefaction to be manifested at the surface, groundwater levels must be within 50 feet of the ground surface and the soils within the saturated zone must also be susceptible to liquefaction. We consider the potential for liquefaction to occur at this site as moderate to high because historic groundwater is generally less than 50 feet below the ground surface. The site is within a "moderate" liquefaction hazard zone as defined by Riverside County (Geographic Information Services, 2018). Liquefaction output considering historic high groundwater levels of 5 feet and soils above the groundwater are presented in Appendix A. Results indicate a worst case liquefaction potential at depths greater than 7.5 and 9.5 feet with estimated dry seismic and liquefaction induced settlement of 1 inch in B-2 and 1.9 inches in B-13. The potential for lateral spreading to the nearby lake is considered low under a screening evaluation due to the blowcount >15 N_{160} for the liquefiable layer (Youd & Bartlett, 2002). Due to the density of overlying soils, the potential for sand boils is considered low. Due to the depth of liquefaction and layer settlement in relation to the footing influence zone for the maximum footing sizes presented within, the potential for bearing failure is considered low.

Dry Seismic Settlement: The amount of dry seismic settlement is dependent on relative density of the soil, ground motion, and earthquake duration. In accordance with current CGS policy (Earth Systems discussion with Jennifer Thornburg, CGS May 2014), we used a site peak ground acceleration of ⅔ PGA_M (PGA_M = 0.91) and an earthquake magnitude of 7.7 to evaluate dry seismic settlement potential. The design peak ground acceleration values were obtained from the SEAOC online application (https://seismicmaps.org/).

Based upon methods presented by Tokimatsu and Seed (1987), the potential for seismically induced dry settlement of soils above the full dry groundwater table for the full soil column height (50 feet) was calculated and estimated to be 0.5 inches in Boring B-2 and 0.5 inches in Boring B-13. The remaining deeper borings onsite had similar potential. Seismic settlement is based on post grading recommendations stated in Section 5.1. Due to the general uniformity of the soils encountered, seismic settlement is expected to occur on an areal basis and as such per Special Publication 117A (CGS, 2008), the differential settlement is estimated to be approximately ½ of the total estimated dry seismic settlement (¼ inch) considering soil remediation as recommended in Section 5.1.

Fissuring and Ground Subsidence: The Riverside County Parcel report indicates that the site is within a "Susceptible" potential subsidence area. In areas of fairly uniform thickness of alluvium, fissures are thought to be the result of tensional stress near the ground surface and generally occur near the margins of the areas of maximum subsidence. Surface runoff and erosion of the incipient fissures augment the appearance and size of the fissures. Fissuring was not observed onsite or in aerial photo review.

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

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

3.8.2 Other Hazards

Landslides and Slope Instability: The site is relatively flat except the existing approximate 15 to 17-feet high ascending graded slope located along the southwest margin of the site. This graded 2:1 slope is likely a fill over cut slope graded for the subdivision located southwest of the project and appeared intact with no evidence of gross or surficial instability despite being in-place for nearly 30 years. Earth Systems performed static, seismic, and temporary construction slope stability analysis for a 2:1 slope having a slope height of 20 feet (20 feet due to inaccuracy in height measurement available). Two soils (compacted fill and native) were used in the study and given engineering soil parameters based on laboratory data, SPT blow counts, and classifications determinations. Soil property values varied depending on the analysis performed. Saturated Ultimate direct shear values were used for static analysis and saturated direct Peak values were used for seismic and temporary construction. A lightly loaded shear was run for native soil analysis, and surficial analysis. Surcharge loads were not included at the top of slope as significant structure (home) loads are setback at least 15 feet and the yards too small to allow heavy development right near the top of slopes (15 feet, 1:1 setback). Pools exist but unload soils. A 100 psf surcharge per the CBC was included for flatwork. Laboratory soil strength cohesion parameters were reduced by 30% in accordance with typical practice and SP117. Historic groundwater levels were considered. Pseudostatic "k" values of 0.3 horizontal and 0.1 vertical were utilized and considered guidance in the Riverside County Technical Guidelines for Review of Geotechnical and Geologic Reports (2000).

For the slope analysis, we used the Janbu and Bishop Simplified Methods in the Slide 8 (Roscience) software, which provided the results for static, seismic, and temporary construction modeling. Results included in Appendix A provide the engineering soil parameters and Factor of Safety for the static, seismic loading, and temporary construction conditions. Note, acceptable Factor of Safety for static loading conditions are 1.5, 1.1 for pseudo static conditions, and 1.2 for temporary construction. Results indicate a factor of safety above 1.5 for static conditions, 1.1 for seismic conditions, and 1.2 for postulated temporary construction conditions. Therefore, the potential for global static and pseudo static slope instability of the present conditions are considered to be low. Due to the "low" potential for lateral flow failure, slope stability under these conditions was not evaluated.

Surficial stability analysis for the 2:1 slope indicates a Factor of Safety of 1.03 (greater than 1) in an unprotected slope face. This is below the mandated Factor of Safety of 1.5. While a Factor of Safety of 1.03 indicates an inherent stability, as confirmed by the lack of evidence for surficial instability, the low factor of safety does suggest a potential hazard assuming full-depth saturation of the slope face (4'). Currently the slope is partially vegetated, including large trees that improve the overall stability of the slope. Erosion and minor sluffing of slopes could occur.

Flooding: Most of 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." A small portion of the project site located to the northwest lies in an area designated as Zone D: "Areas in which flood hazards are undetermined, but possible." This project area and Zone X and Zone D are identified on FEMA Map No.: 06065C2017G, Panel 2017 of 3805, Map Revised August 28, 2008. Appropriate project design by the project civil engineer, construction, and maintenance can minimize the site sheet flooding potential.

Seiches: Seiching is defined as a periodic oscillation of liquid within a container or reservoir. Its period is determined by the resonant characteristics of the container, as controlled by its physical dimensions. Swimming pools are located on the residential lots immediately southwest of the site. Any pool seiches related flooding could exit the back yards and flow over the southwest margin slope, resulting in erosion and minor flooding.

The site is elevated approximately 60 above the Lake Elsinore high water elevation and about 1460 feet laterally from the shoreline. Thus, the on-site hazards from seiching of Lake Elsinore is considered low.

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 and geologic standpoint, for construction as proposed, provided the recommendations presented in this report are incorporated into project design and construction.

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

Geotechnical Constraints and Mitigation:

- The primary geologic hazard is moderate to severe ground shaking from earthquakes originating on regional southern California faults. A major earthquake originating on the nearby segments of the Elsinore, San Jacinto, and San Andreas fault zones and other associated faults would be the critical seismic events that may affect the site within the design life. Engineered design and earthquake-resistant construction increase safety and allow development within seismic areas.
- The underlying geologic condition for seismic design is Site Class D. The site is about 0.2 miles from a Type A seismic source as mapped by the California Geological Survey. However, the site is approximately 2.75 miles from a Type A seismic source and an Alquist-Priolo Special Studies Zone. A qualified professional should design any permanent structure constructed on the site. The minimum seismic design should comply with the 2016 edition of the California Building Code.
- The site is within a County of Riverside designated fault zone, but is not within a currently designated Alquist-Priolo Earthquake Fault Zone. Evidence of faulting, including active faulting was not observed in the fault hazard exploration trenches excavated for this project. Nor were there any significant aerial photograph lineaments noted on the historic aerial photographs suggestive of active faulting. 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 moderate for this project. The site is not within an area of documented areal subsidence.
- Other geologic hazards, including flooding, and landslides, are considered low potential on this site. Surficial instability of the existing ascending 2:1 graded slope along the southwest margin of the property is considered a moderate hazard. However, assuming construction of the planned retaining wall along the toe of this slope and slope protection is implemented, the potential for slumps and soil creep is reduced, and a general maintenance issue.
- Based on current conditions, groundwater is not anticipated to be encountered during

construction.

- Much of the existing on-site fill and alluvial soils are very low in Expansion Index and suitable for location under structures or hardscape after remedial grading. Building structure recommendations provided within are based upon using a very low in expansion potential fill material.

- The upper site soils have variable blow counts, low in-place densities, and associated potential for hydrocollapse. In our opinion, the upper loose alluvial soils are considered compressible and will require over-excavation within the proposed building pads, storm drain channels, hardscape, parking, drives and other settlement sensitive areas. In-place density test results of 85% or higher (or firm soils) will need to be attained within the bottom of the structure over-excavations before an over-excavation is approved for fill placement.

- Laboratory testing of two samples showed potentially “very 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.

- Setbacks are provided for structures, including setback from the onsite storm channel easement.

- Specific retaining wall foundation design recommendations are provided to minimize disturbance and back cuts into existing slopes providing support for up-slope properties and homes.

Section 5
RECOMMENDATIONS

5.1 Site Development – Grading

A representative of Earth Systems should observe site clearing, grading, and the bottoms of excavations before placing fill. Local variations in soil conditions may warrant increasing 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: Due to the non-uniform and variable low-density of shallow 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 footings, whichever is lower. The exposed undisturbed subgrade bottom should be observed and tested by the geotechnical engineer or his representative to verify an in-place density of the subgrade is at or greater than 85% relative compaction per ASTM D 1557 or soils are firm (as determined by the geotechnical engineer or his representative). Deeper over-excavation may be recommended if the required in-place density is not achieved, soils are not firm, or undocumented fill exists.

The approved bottom of the 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 in suitable compacted lifts. 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 in suitable, compacted lifts. 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 or be located at least 20 feet from foundations. 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*.

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

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

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

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

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

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

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

Shrinkage and Oversize Loss: The shrinkage factor for earthwork for the alluvial soil materials is expected to range from -3 to 18 percent for the upper excavated or scarified *site* soils based upon evaluation of 23 in-place densities (one standard deviation = 5, 95% Confidence Interval). This estimate is based on compactive effort to achieve a weighted average relative compaction of about 93 percent.

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

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 20-30 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. 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 should be protected from water flow over the exposed surface and saturation.

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 6
Temporary Cantilevered and Braced Shoring System Parameters

Equivalent Fluid Pressure pounds per cubic foot (pcf)	
Cantilevered	**Braced**
42	64

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 240 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 450 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 350 pcf.

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

The contractor should be responsible for the structural design and safety of all temporary shoring systems. The contractor should carefully review the 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.

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

<u>Pipe Subgrade and Bedding:</u> 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.

<u>Pipe-Zone, Trench–Zone, Trench Backfill and Compaction:</u> 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 prior to the placement of subsequent lifts. 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. Gravel should be separated from backfill with a filter fabric such as Mirafi 140N or equivalent as approved by the soils engineer. 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 18 or 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. We recommend a structure setback of at least 20 feet from the onsite storm channel easement, or 30 feet from the edge of pipe or channel, whichever is greater.

Conventional Spread Foundations: Allowable soil bearing pressures are given below for foundations bearing on recompacted soils as described in Section 5.1 and considered historic water conditions. Allowable bearing pressures are net (weight of footing and soil surcharge may be neglected).

- Continuous wall foundations, 1 foot minimum and 2.5 foot maximum width and 18-inch minimum depth below grade:

  1,500 psf for dead plus design live loads

- Pad foundations, 2 x 2-foot minimum and 7 x 7-foot maximum in plan and 24 inches below grade:

  1,850 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.

Retaining wall foundations along the existing slope to the west should be designed as an eccentric foundation with the foundation located away from the slope to minimize disturbance and backcuts within the existing slope supporting homes and improvements. Subsurface tanks should be designed for the potential high groundwater conditions detailed within which may require "deadmen" or other means to resist buoyant forces.

If the anticipated loads exceed the estimated values stated in Section 1.1 (90 kips for Isolated Footings and 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 differential settlement from dry seismic, collapse, and static loads. 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 150 pounds per cubic inch (pci) can be used to design lightly loaded footings, beams, pavement, 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 and bearing failure considering high groundwater (4,500 to 5,550 psf ultimate) and a safety factor of 2.25 on transient loads (2,000 to 2,450 psf). Ultimate bearing to soil failure depends on foundation size and could be greater than 5,550 psf. The restrictions of Section 1605A.1.1 apply to the cited bearing values for Allowable Stress Design (ASD).

Table 7

	Allowable Bearing Capacity (psf) (FS = 3)	**Short Term (Wind/Seismic) (FS = 2.25)**	**Ultimate Bearing Capacity (FS = 1)**
Continuous Foundations	1,500	2,000	4,500
Isolated Pad Foundations	1,850	2,450	5,550

FS = Factor of Safety

Footings should be designed and reinforced by the structural engineer for the specific loading, settlement, or collapse soil conditions defined herein.

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, and collapse settlement should be approximately 1 inch, based on footings founded on firm soils as recommended. Differential static settlement between similar bearing members should be less than ½ inch. As such, considering static, and collapse differential settlement applied over a typical foundation distance of 40 feet, we recommend the structural engineer design for a standard angular distortion of 1:480. Considering the static, collapse, and seismic case, we recommend the structural engineer design for 1.7 inches in 40 feet or an angular distortion of 1:280. 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.

Seismic settlements are considered "small scale" as per SP117A and as such, structural solutions may be used to resist such hazards.

Settlement calculations are presented in Appendix A and collapse results are provided in Section 3.3. 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.

Earthquake Performance Statement: Depending upon the extent of structural and geotechnical design, some damage due to seismic events will occur. We recommend a standard statement for purchasers or end users of the property and within title reports that seismic induced damage may occur. Note that all of southern California in general is in earthquake country. Site developments in southern California are typically not designed to mitigate anticipated seismic events without some damage. In fact, the Building Code is intended to provide Life-Safety performance, not complete damage-free design. In other words, some damage from earthquakes in the form of structural damage, settlement, cracking, and disruption of utilities is expected and that repair after an earthquake event will likely be required. It is not the current standard of care for site developers to fully mitigate all anticipated earthquake induced hazards. It is incumbent on the developer to advise the end-users of the project of the anticipated hazards in the form of disclosure statements during the initial and subsequent purchase processes.

According to literature from Robert W. Day, doors and windows may stick at distortion angles between 1:240 and 1:175. In this situation, a human being could be put in a life-threatening situation. Therefore, Earth Systems recommends (for shallow foundation design) the maximum distortion angle using all the settlement conditions including seismic settlements be 1:240. The estimated angular distortions for this project are better than this threshold.

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 Silty Sand (SM) 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.

<u>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.</u> 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.

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. Observation during drilling is required by the 2016 California Building Code on a full-time basis by the geotechnical engineer or his representative. If subsurface conditions noted during drilled pier installation are significantly different than those encountered in our borings, it may be necessary to adjust the overall length of the pier.

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

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

5.5 Slope Construction

New slopes are not generally proposed for this project; however, minor slopes (less than 5 feet in height) may be constructed. Modification of the existing 2:1 slope may occur to accommodate a retaining wall. For remedial grading of the slope, new fills should be benched into firm existing soils. A backdrain behind the retaining wall is recommended.

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. Site soils are granular and generally free draining such that "rapid draw down" strength loss will not occur.

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

Surficial Slope Failures: Site soils are highly susceptible to erosion from wind and water sources. All slopes will be exposed to weathering, resulting in decomposition of surficial earth materials, thus potentially reducing shear strength properties of the surficial soils. In addition, these slopes become increasingly susceptible to rodent burrowing. As these slopes deteriorate, they can be expected to become susceptible to surficial instability such as soil slumps, erosion, soil creep, and debris flows. Development areas immediately adjacent to ascending or descending slopes should address future surficial sloughing of soil material and erosion. Such measures may include debris fences, slope facing, catchment areas or walls, diversion ditches or berms, soil planting, velocity reducers or other techniques to contain soil material away from developed areas and reduce erosion. Additionally, foundations should be set back at least 5 feet from the edge of slope or as per the 2016 CBC, whichever is greater.

Operation and maintenance inspections should be done after a significant rainfall event and on a time-based 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).

More detailed stability and value engineering analysis of the retaining wall/ascending slope is recommended once grading plan and retaining wall plans are progressing. Backcut configurations during construction of the retaining wall will need to be stable to prevent instability of the adjacent lots at the top of the slope.

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 8
Percent Passing Sieve Size

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

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

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

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

Slab Thickness and Reinforcement: Structure slabs should be a minimum of 4 inches in actual thickness and be reinforced with # 3 bars at 18 inches on center both ways. Slabs in contact with earth should use closer joints to control cracking or be thickened to allow adequate earth to rebar clearance. Reinforcing bars should extend at least 40 bar diameters into the footings and slabs. Concrete slabs-on-grade and flatwork should be supported by compacted and moisture conditioned soil placed in accordance with this report. If slabs are structural, they should be designed for the specific settlement conditions presented within.

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

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

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

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.

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

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

5.7 Retaining Walls and Lateral Earth Pressures

Walls which are restrained at the top such as retaining wall returns, below-grade walls and walls tied to floor slabs should be designed with "at rest" earth pressures. Retaining walls, free to tilt at the top, may be designed for "active" earth pressures.

The following list presents lateral earth pressures for use in wall design. The values are given as equivalent fluid pressures ***without*** surcharge loads or hydrostatic pressure. Clay soils are not suitable for wall backfill as they are not free draining. 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 onsite sand, an active pressure of 42 pcf of equivalent fluid weight for well-drained, level backfill may be used. Similarly, an active pressure of 52 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 64 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 (where not excepted) to include forces from dynamic (seismic) earth pressure. Dynamic pressures are additive to active and at-rest earth pressure and should be considered as 63 pcf for flexible walls, and 80 pcf for rigid walls. Seismic pressures are based on PGA_M of 0.91g, Friction Soil Angle of 31°, and a maximum dry density of 133 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 240 psf for auto and 450 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.

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

- Resistance to lateral loads (including those due to wind or seismic forces) may be provided by frictional resistance between the bottom of concrete foundations and the underlying soil, and by passive soil pressure against the foundations. An allowable coefficient of friction 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 350 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 350 pcf. The maximum passive resistance should not exceed 2,000 psf.
- Construction employing poles or posts (i.e. lamp posts) may utilize design methods presented in Section 1807.3 of the CBC for Sandy soils (SM) material class.
- The passive resistance of the subsurface soils will diminish or be non-existent if trench sidewalls slough, cave, or are over widened during or following excavations. If this condition is encountered, our firm should be notified to review the condition and provide remedial recommendations, if warranted.

5.8 Seismic Design Criteria

This site is subject to strong ground shaking due to potential fault movements along regional faults including the 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. General Procedure seismic parameters are presented below per ASCE7-10 exception, considering a Site Class D (based on Vs shear wave velocity) for structures not greater than 0.5 seconds in period. For foundations described within, site soils are not subject to bearing failure.

2016 CBC (ASCE 7-10) Seismic Parameters

Seismic Design Category:	E
Site Class:	D (F*)
Maximum Considered Earthquake [MCE] Ground Motion	
Short Period Spectral Response S_s:	2.288 g
1 second Spectral Response, S_1:	0.921 g
Code Design Earthquake Ground Motion	
Short Period Spectral Response, S_{DS}	1.525 g
1 second Spectral Response, S_{D1}	0.921 g
Peak Ground Acceleration (PGA_M)	0.91 g

*Site is potentially liquefiable and for structures greater than 0.5 seconds in period, Site Class is F applies and the above values do not apply. For Site Class F, site specific evaluation is required. Please contact Earth Systems should this case apply.

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 9
Preliminary Flexible Pavement Section Recommendations
On-site/Interior Automobile Drive Areas

R-Value of Subgrade Soils - 52 (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 10
Preliminary Portland Cement Concrete Pavement Sections

Area	Minimum Pavement PCC Thickness (inches)	Minimum 28 Day Flexural Strength (psi)	Concrete Compressive Strength (psi)
Truck/Bus Access or Loading/Unloading Areas (Traffic Category C, ADTT =100)	6.5	525	3,250

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

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

- Subsequent to utility installation, the entire pavement (including PCC) final subgrade should be scarified 12 inches, moisture conditioned to near optimum moisture content, and compacted to a minimum 95% relative compaction immediately prior (within a few days) to the placement and compaction of aggregate base to re-establish proper moisture content and compaction in site soils.

- Subgrade soils and aggregate base should be in a stable, non-pumping condition at the time of placement and compaction. Exposed subgrades should be proof-rolled to verify the absence of soft or unstable zones.

- Aggregate base materials should be compacted at near optimum moisture content to at least 95 percent relative compaction (ASTM D 1557) and should conform to Caltrans Class II criteria. Standard Specifications for Public Works Construction "Greenbook" standards (Crushed Aggregate Base class) may be used in lieu of Caltrans. Compaction efforts should include rubber tire proof-rolling of the aggregate base with heavy compaction-specific equipment (i.e. fully loaded water trucks).

- All concrete curbs separating pavement from landscaped areas should extend at least 6 inches into the subgrade soils to reduce the potential for movement of moisture into the aggregate base layer (this reduces the risk of pavement failures due to subsurface water originating from landscaped areas).

- Asphaltic concrete should be ½-in. or ¾-in. grading and compacted to a minimum of 95% of the 75-blow Marshall density (ASTM D 1559) or equivalent.

- Portland cement concrete pavements should be constructed with transverse joints at maximum spacing of 15 feet. A thickened edge should be used where possible and, as a minimum, where concrete pavements abut asphalt pavements. The thickened edge should be 1.2 times the thickness of the pavement (8 inches for a 6.5-inch pavement), and should taper back to the PCC thickness over a horizontal distance on the order of 3 feet.

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

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

  1. Slope pavement at least ½ percent to provide drainage;
  2. Provide rough surface texture for traction;
  3. Cure PCC concrete with curing compound or keep continuously moist for a minimum of seven days;

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

- Portland cement concrete placement and curing should, at a minimum, be in accordance with the American Concrete Institute [ACI] recommendations contained in ACI 211, 304, 305, 308, 309, and 318.
- Within the structural pavement section areas, positive drainage (both surface and subsurface) should be provided. In no instance should water be allowed to pond on the pavement. Roadway performance depends greatly on how well runoff water drains from the site. This drainage should be maintained both during construction and over the entire life of the project.
- Proper methods, such as hot-sealing or caulking, should be employed to limit water infiltration into the pavement base course and/or subgrade at construction/expansion joints and/or between existing and reconstructed asphalt concrete sections (if any). Water infiltration could lead to premature pavement failure.
- To reduce the potential for detrimental settlement, excess soil material, and/or fill material removed during any footing or utility trench excavation, should not be spread or placed over compacted finished grade soils unless subsequently compacted to at least 90% of the maximum dry unit weight, as evaluated by ASTM D 1557 test procedure, at near optimum moisture content, or 95% if placed under areas designated for pavement.
- Where new roadways will be installed against existing roadways, the repaired asphalt concrete pavement section should be designed and constructed to have at least the pavement and aggregate base section as the original pavement section thickness (for both AC and base) or upon the newly calculated pavement sections presented within, whichever is greater.
- Pavement designs assume that heavy construction traffic will not be allowed on 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 slopes or retaining wall faces. Diversion and conveyance structures which can accommodate water and eroded soil should be constructed at the tops and toes of all slopes. Lined swales at the top and bottom of slopes, and at the top of retaining walls are recommended.
- In no instance should water be allowed to flow or pond against structures, slabs or foundations or flow over unprotected slope faces. Adequate provisions should be employed to control and limit moisture changes in the subgrade beneath foundations or structures to reduce the potential for soil saturation. Landscape borders should not act as traps for water within landscape areas. Potential sources of water such as piping, drains, over-spray broken sprinklers, etc, should be frequently examined. Any such leakage, over-spray, or plugging should be immediately repaired.
- 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 crust removal, etc.) throughout their design life. Maintenance of these structures should be incorporated into the facility operation and maintenance manual. Structural performance is dependent on many drainage-related factors such as landscaping, irrigation, lateral drainage patterns and other improvements.

Section 6
LIMITATIONS AND ADDITIONAL SERVICES

6.1 Uniformity of Conditions and Limitations

Our findings and recommendations in this report are based on selected points of field exploration, laboratory testing, and our understanding of the proposed project. Furthermore, our findings and recommendations 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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Other Aerial Photographs:

Google Earth: 1994-2018

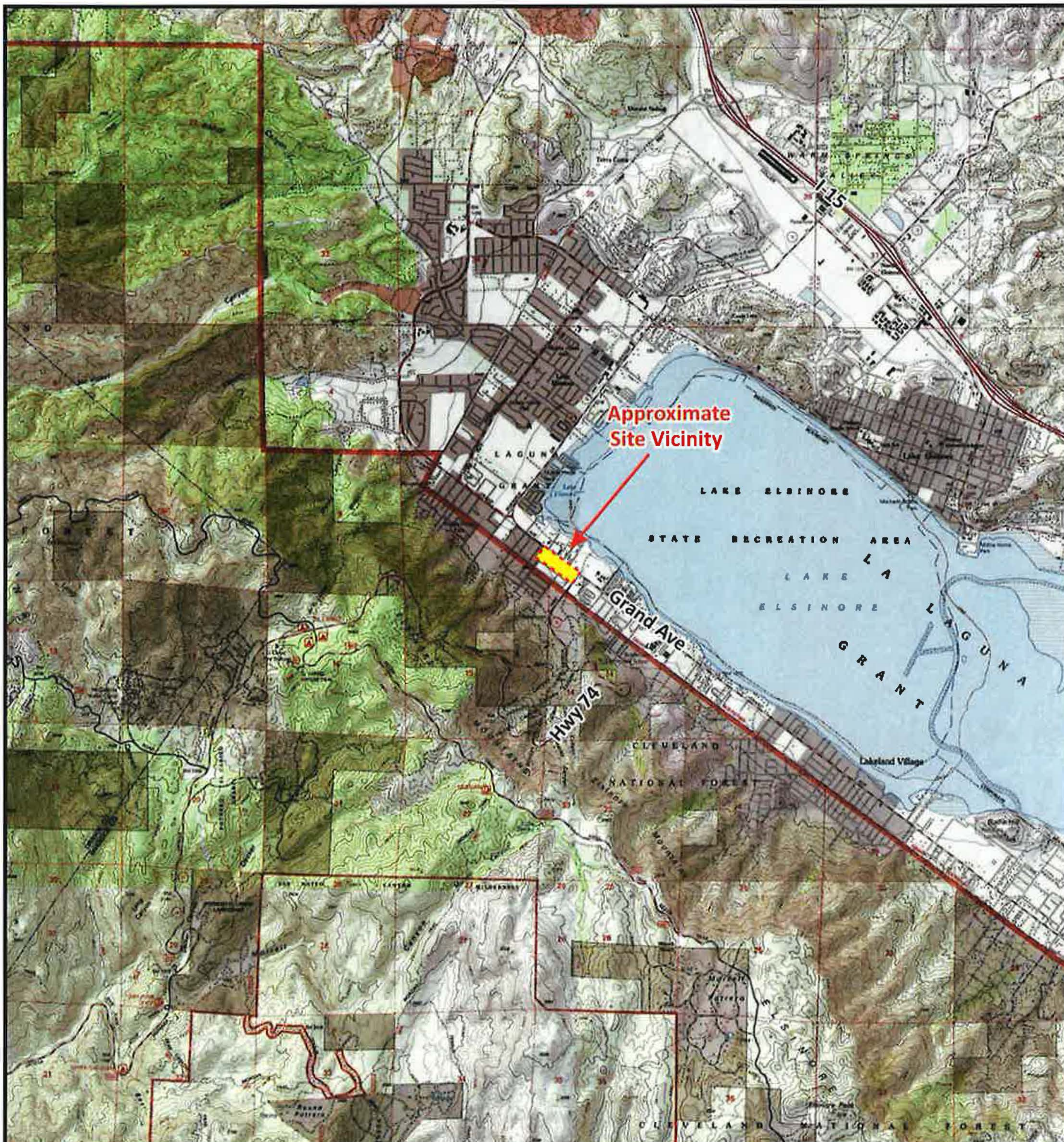
Historic Aerials: 1938-2014

APPENDIX A

Plate 1 – Site Vicinity Map
Plate 2 – Exploration Location Sketch
Plate 3 – Regional Geology Map
Plate 4 – Regional Fault Map
Table A-1 Fault Parameters
Terms and Symbols Used on Boring Logs
Soil Classification System
Logs of Borings (14 pages)
Test Pit Logs (4 pages)
Fault Trench Logs (5 pages)
Site Class Estimator (2 pages)
Seismic Settlement (6 pages)
Spread Footing Static Load Settlement (2 pages)
Continuous Footing Static Load Settlement (2 pages)
Slope Stability Output (5 pages)

APPENDIX B

Laboratory Test Results



LEGEND

Approximate Site Boundary

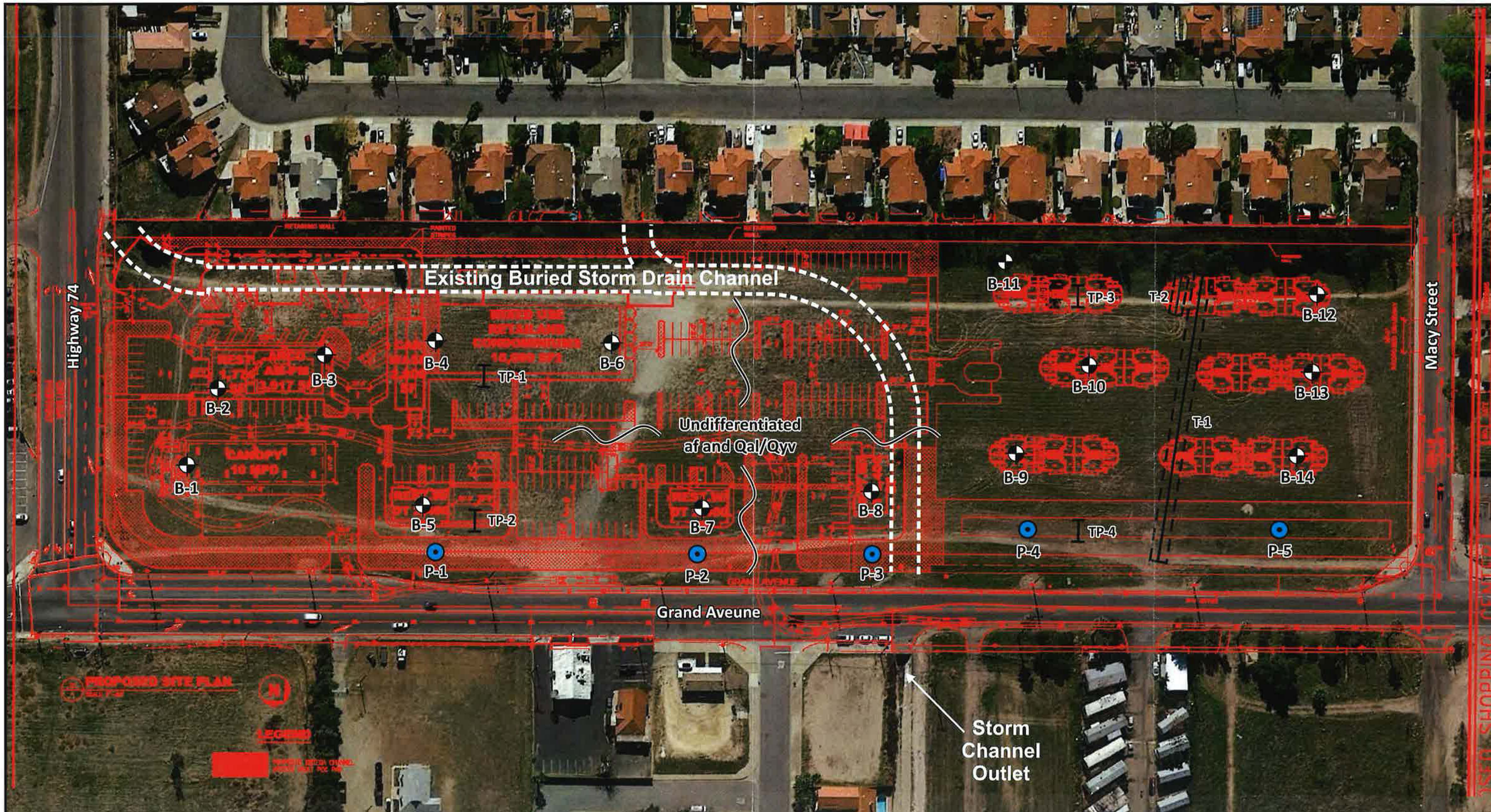
Approximate Scale: 1" = 1 Mile

0 1 Mile 2 Miles

N

Source: Google Earth satellite image with USGS Topographic Map Overlay.

Plate I Site Vicinity Map	
Proposed Bamiyan Marketplace 15749 Grand Avenue Lake Elsinore, Riverside County, California	
Earth Systems	
1/17/2019	File No.: 302169-002



LEGEND

B-14 Approximate Boring Locations

P-5 Approximate Percolation Test Locations

TP-4 Approximate Test Pit Locations

T-2 Approximate Fault Trench Locations

Approximate Scale: 1" = 100'

0 100' 200'

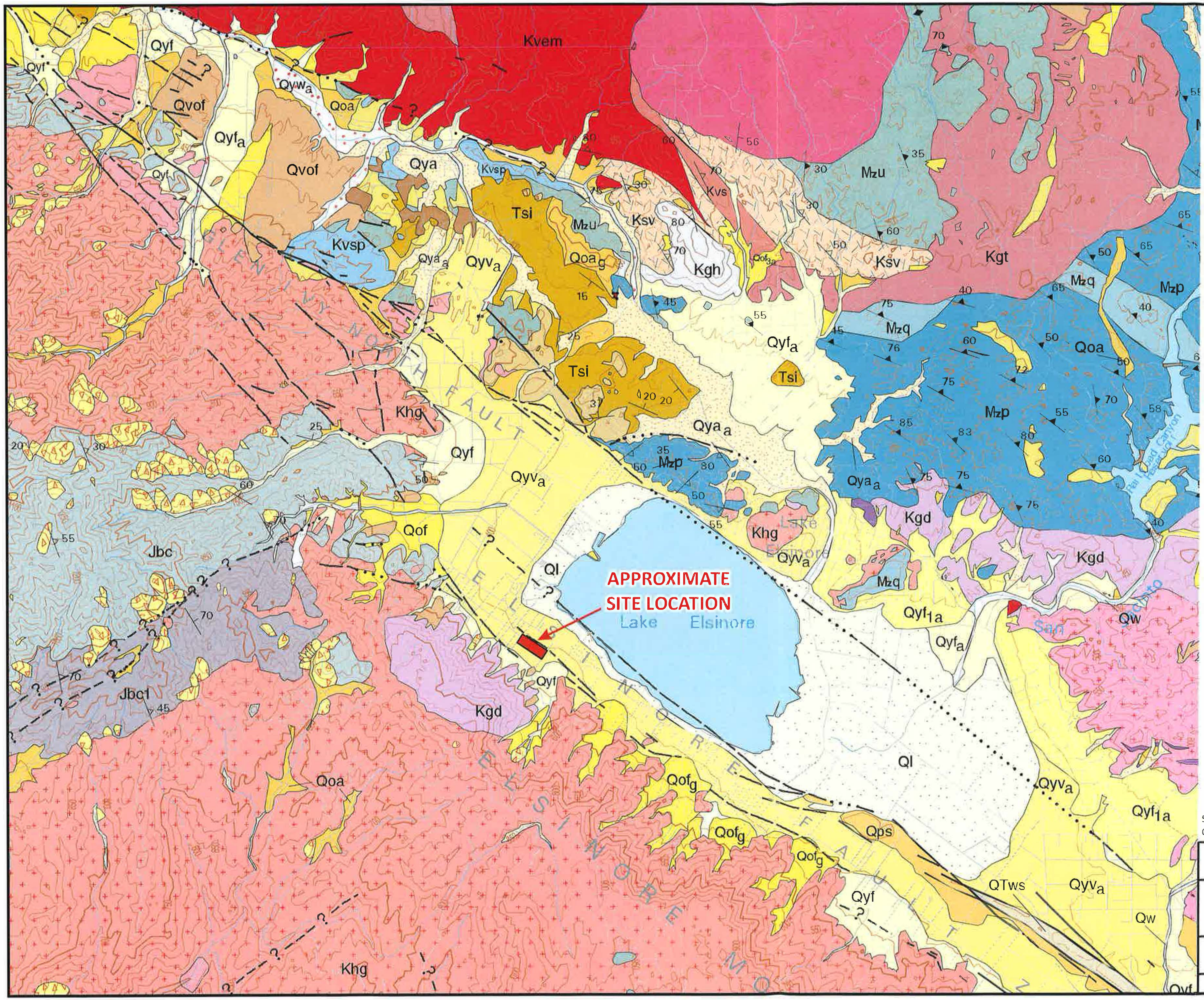
Source: Google Earth satellite image dated 2/19/2018 with AGC Design Concept, Inc. Proposed Site Plan, Sheet CUP-1

Plate 2
Exploration Location Sketch

Proposed Bamiyan Marketplace
15749 Grand Avenue
Lake Elsinore, Riverside County, California

Earth Systems

1/17/2019	File No.: 302169-002



LEGEND

Qyf	**Young alluvial fan deposits (Holocene and latest Pleistocene)**-Unconsolidated deposits of alluvial fans and headward drainages of fans. Consists predominately of gravel, sand, and silt. Trunk drainages and proximal parts of fans contain higher percentage of coarse-grained sediment than distal parts.
Qyv	**Young alluvial valley deposits (Holocene and late Pleistocene)**-Fluvial deposits along valley floors. Consists of unconsolidated sand, silt, and clay-bearing alluvium.
Qof	**Old alluvial fan deposits (late to middle Pleistocene)** Reddish brown, gravel and sand alluvial fan deposits; indurated, commonly slightly dissected. In places includes thin alluvial fan deposits of Holocene age.
Kqd	**Quartz diorite (Cretaceous)**-Medium- to coarse-grained biotite-hornblende quartz diorite. Most is slightly to well foliated with discoidal to pancake-shaped melanocratic inclusions in foliation plane. Grades into diorite and biotite-hornblende tonalite
Khg	**Heterogeneous granitic rocks (Cretaceous)**-A wide variety of heterogeneous granitic rocks occur in Santa Ana quadrangle. Some heterogeneous assemblages include large proportions of schist and gneiss. Rocks in Santa Ana Mountains include a mixture of monzogranite, granodiorite, tonalite, and gabbro. Tonalite composition rock is most abundant rock type. Tonalite from Hot Springs Canyon (Cañada Gobernadora 7.5' quadrangle), gave zircon age of 119.2 Ma_{id} and 116.5 Ma_{ip} (Pb/U ages, W.R Premo per. commun., 1999). Heterogeneous granitic rocks adjacent to east and south of Lakeview Mountains and Reinhardt Canyon pluton contain large amount of metamorphic rock. Except for southern part, granitic component consists of potassium feldspar-bearing tonalite and granodiorite

N

Approximate Scale 1" = 1 Mile

0 1 Mile 2 Miles

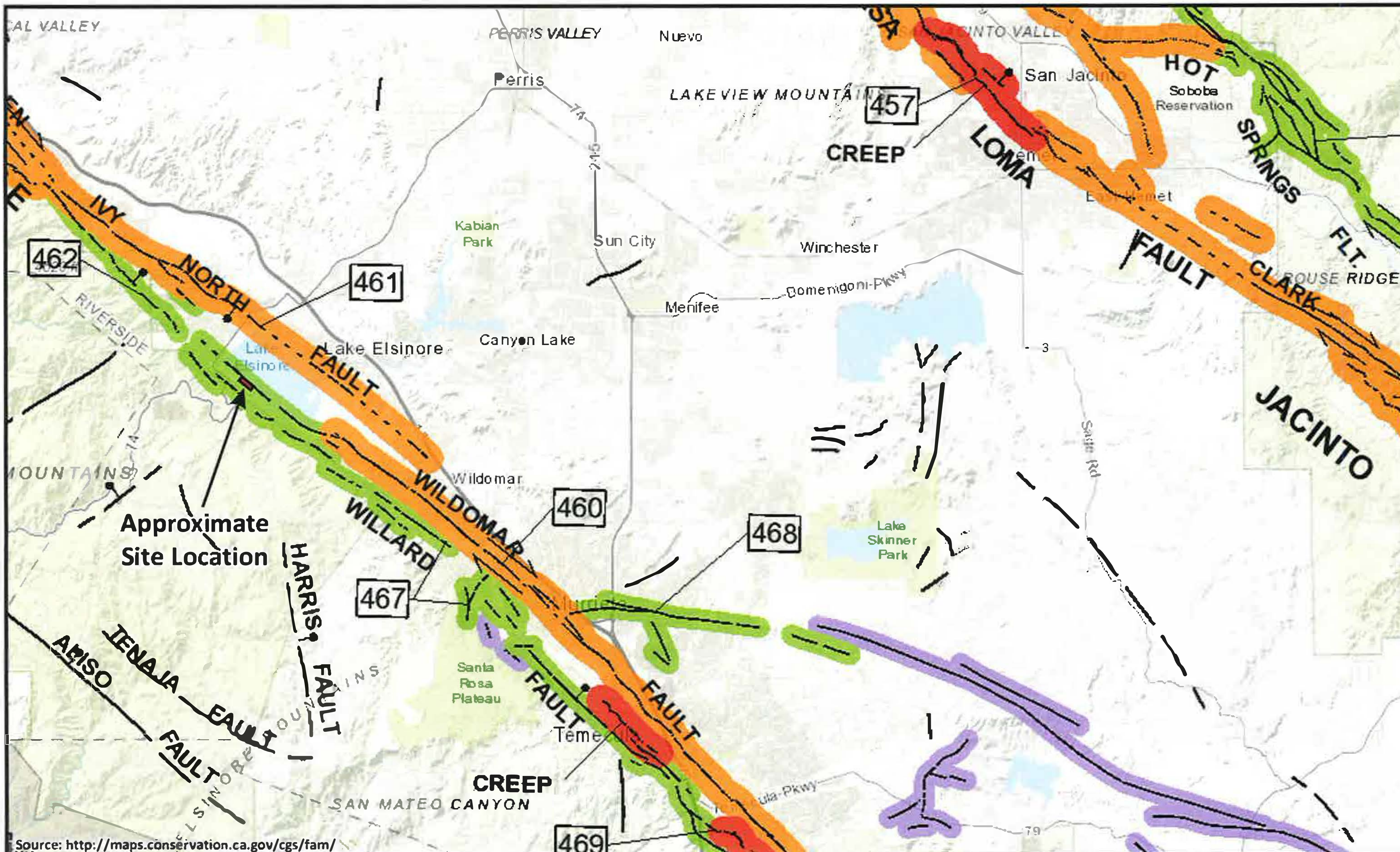
Source: CGS Preliminary Digital Geologic Map of the Santa Ana 30' X 60' Quadrangle, Southern California, Version 2.0, 2004.

Plate 3
Regional Geologic Map

Proposed Bamiyan Marketplace
15749 Grand Avenue
Lake Elsinore, Riverside County, California

Earth Systems

1/17/2019	File No.: 302169-002



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 4
Regional Fault Map

Proposed Bamiyan Marketplace
15749 Grand Avenue
Lake Elsinore, Riverside County, California

Earth Systems

1/17/2019	File No.: 302169-002

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)
Elsinore (Temecula stepover)	0.2	0.4	90	212	180	12	A	**7.6**	725	2.5
Elsinore (Stepovers Combined)	1.6	2.6	90	224	180	12	B'	**6.3**		
Elsinore (Glen Ivy stepover)	1.7	2.7	90	216	180	11	A	**7.1**	322	2.5
Elsinore (Glen Ivy) rev	1.8	3.0	90	218	180	26	A	**7.0**	222	5
Elsinore (Temecula) rev	6.5	10.5	90	230	180	40	A	**7.4**	431	5
Chino, alt 2	15.6	25.2	65	234	150	29	B	**6.7**		1
Whittier, alt 1	16.9	27.1	70	24	150	46	A	**7.1**	530	2.5
Whittier, alt 2	16.9	27.1	75	24	150	46	A	**7.1**	530	2.5
San Joaquin Hills	17.5	28.1	23	204	90	27	B	**7.0**		0.5
Chino, alt 1	18.3	29.4	50	236	150	24	B	**6.6**		1
San Jacinto (Anza, stepover)	22.3	35.9	90	224	180	25	A	**7.6**	151	9
San Jacinto (San Jacinto Valley, stepover)	23.7	38.2	90	224	180	24	A	**7.4**	199	9
San Jacinto (Stepovers Combined)	23.7	38.2	90	229	180	25	B'	**6.7**		
San Jacinto (San Jacinto Valley) rev	24.1	38.8	90	223	180	18	A	**7.4**	199	18
Peralta Hills	24.2	38.9	50	3	na	14	B'	**6.5**		
Newport-Inglewood (Offshore)	25.1	40.4	90	227	180	66	B	**6.9**		1.5
Fontana (Seismicity)	25.5	41.0	80	313	na	24	B'	**6.7**		
San Jacinto (San Bernardino)	26.0	41.9	90	225	180	45	A	**7.4**	205	6
Yorba Linda	27.1	43.6	90	153	na	18	B'	**6.5**		
San Jacinto (Anza) rev	27.2	43.8	90	216	180	46	A	**7.6**	151	18
Oceanside	27.4	44.0	23	69	na	120	B'	**7.5**		
Richfield	28.3	45.5	28	353	na	6	B'	**6.2**		
San Gorgonio Pass	28.8	46.3	60	11	na	29	B'	**6.9**		
Elsinore (Julian)	30.6	49.3	84	36	180	75	A	**7.6**	725	3
Newport-Inglewood, alt 2	31.6	50.8	90	49	180	66	B	**7.2**		1
Newport-Inglewood, alt 1	32.1	51.6	88	49	180	65	B	**7.2**		1
Puente Hills (Coyote Hills)	32.5	52.3	26	358	90	17	B	**6.8**		0.7
Anaheim	33.5	54.0	71	45	na	16	B'	**6.3**		
Puente Hills	33.6	54.0	25	20	90	44	B	**7.1**		0.7
San Andreas (San Bernardino S)	34.9	56.1	90	210	180	43	A	**7.6**	150	16
San Andreas (San Bernardino N)	35.1	56.4	90	212	180	35	A	**7.5**	103	22
San Andreas, (North Branch, Mill Creek)	35.1	56.4	76	204	180	106	A	**7.5**	110	17
Elysian Park (Lower, CFM)	35.4	56.9	22	33	na	41	B'	**6.8**		
Cucamonga	35.5	57.2	45	347	90	28	B	**6.6**		5
San Jose	35.6	57.3	74	334	30	20	B	**6.6**		0.5
Earthquake Valley (No Extension)	36.1	58.1	90	221	180	33	B'	**6.9**		
Rose Canyon	37.1	59.8	90	68	180	70	B	**6.8**		1.5
Mission Creek	37.4	60.2	65	5	180	31	B'	**6.9**		
Sierra Madre	38.1	61.4	53	19	90	57	B	**7.2**		2
San Andreas (San Gorgonio Pass-Garnet Hill)	38.9	62.6	58	20	180	56	A	**7.6**	219	10

Reference: USGS OFR 2007-1437 (CGS SP 203) Based on Site Coordinates of 33.659065 Latitude, -117.379207 Longitude

Mean Magnitude for Type A Faults based on 0.1 weight for unsegmented section, 0.9 weight for segmented model (weighted by probability of each scenario with section listed as given on Table 3 of Appendix G in OFR 2007-1437). Mean magntude is average of Ellworths-B and Hanks & Bakun moment area relationship.

DESCRIPTIVE SOIL CLASSIFICATION

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

SOIL GRAIN SIZE

U.S. STANDARD SIEVE

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

SOIL GRAIN SIZE IN MILLIMETERS

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

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

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

CONSISTENCY OF COHESIVE SOILS (CLAY OR CLAYEY SOILS)

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

MOISTURE DENSITY

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

MOISTURE CONDITION

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

PLASTICITY

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

GROUNDWATER LEVEL

Water Level (measured or after drilling)

Water Level (during drilling)

RELATIVE PROPORTIONS

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

LOG KEY SYMBOLS

Bulk, Bag or Grab Sample

Standard Penetration Split Spoon Sampler (2" outside diameter)

Modified California Sampler (3" outside diameter)

No Recovery

Terms and Symbols Used on Boring Logs

Earth Systems Southwest

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

Soil Classification System

Earth Systems Southwest

Earth Systems

1680 Illinois Ave., Suite 20, Perris, CA 92571
Phone (951) 928-9799

Boring No. B-1	Drilling Date: December 4, 2018
Project Name: Bamiyan Marketplace	Drilling Method: Mobile B-61 w/autohammer
Project Number 302169-002	Drill Type: 6" HSA
Boring Location: See Plate 2, Approximate Elevaton 1,313 feet (MSL)	Logged By: D. Hamelehle

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	Graphic Trend Blow Count / Dry Density
0				SM			SILTY SAND: brown, medium dense, slightly moist, fine to coarse grained sand, trace pinholes, Alluvium	
	MOD Calif.	7,8,8			111	5		
5	MOD Calif.	4,4,5			112	6	loose	
	MOD Calif.	5,7,10		SM	111	4	SILTY SAND: brown, medium dense, damp, fine to medium grained sand	
10	MOD Calif.	5,7,9			112	4		
	MOD Calif.	5,7,13			110	7	slightly moist	
15	MOD Calif.	7,7,11			114	7	light gray brown, with cobbles, older alluvium	
20	MOD Calif.	23,39,50/3"		SP	131	4	SAND: gray brown, very dense, damp, fine to very coarse grained sand, trace gravel, possible decomposed granitic bedrock or boulder	
25	MOD Calif.	50/2"					possible moderately weathered granitic rock or bulder	

Refusal at 26 feet due to hard drilling
Backfilled with cuttings
No groundwater encountered

Earth Systems

1680 Illinois Ave., Suite 20, Perris, CA 92571
Phone (951) 928-9799

Boring No. B-2

Project Name: Bamiyan Marketplace
Project Number 302169-002
Boring Location: See Plate 2, Approximate Elevation 1,311 feet (MSL)

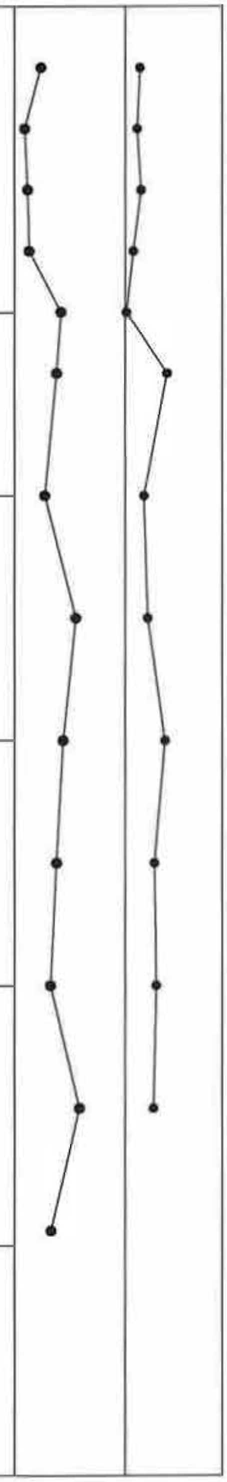
Drilling Date: December 4, 2018
Drilling Method: Mobile B-61 w/autohammer
Drill Type: 6" HSA
Logged By: D. Hamelehle

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				SM			SILTY SAND: brown, medium dense, dry, fine to medium grained sand, trace clay, Alluvium
	MOD Calif.	17,18,18			115	5	slightly moist
5	MOD Calif.	5,6,8			112	5	loose
	MOD Calif.	5,7,11			116	5	fine to coarse grained sand, medium dense
10	MOD Calif.	7,9,11			108	4	light brown, trace micaceous
	MOD Calif.	7,13,50/3"		SP		5	SAND: light brown, dense, damp to slightly moist, fine to coarse grained sand, trace micaceous, Older Alluvium, sample disturbed
15	MOD Calif.	31,23,34				3	highly to moderately weathered granitic cobble
20	MOD Calif.	18,20,21		SM	119	6	SILTY SAND: light gray brown, medium dense, slightly moist, fine to coarse grained sand
25	MOD Calif.	11,33,50/2.5"			123	14	highly weathered granitic cobble, moist
30	MOD Calif.	20,31,35		SM	141	4	SILTY SAND: gray brown, dense, damp, fine to coarse grained sand, trace clay, some cobbles
35	MOD Calif.	15,31,26			130	9	gray, medium dense, moist trace gravel
40	MOD Calif.	12,27,22		SM	132	9	SILTY SAND: brown, dense, moist, fine to coarse grained sand
45	MOD Calif.	34,38,50/4.5"			129	11	possible highly weathered granitic bedrock
50	MOD Calif.	50/6"					no recovery
55							
60							Boring completed at 50-1/2 feet Backfilled with cuttings and sealed with bentonite Groundwater encountered at 47-1/2 feet

Graphic Trend
Blow Count Dry Density



Earth Systems

1680 Illinois Ave., Suite 20, Perris, CA 92571
Phone (951) 928-9799

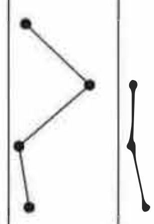
Boring No. B-3	Drilling Date: December 4, 2018
Project Name: Bamiyan Marketplace	Drilling Method: Mobile B-61 w/autohammer
Project Number 302169-002	Drill Type: 6" HSA
Boring Location: See Plate 2, Approximate Elevation 1,309 feet (MSL)	Logged By: D. Hamelehle

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				SM			SILTY SAND: brown, medium dense, dry, fine to coarse grained sand, trace clay, Alluvium
2.5	MOD Calif.	7,10,13					
5	MOD Calif.	4,7,9			115	4	damp
7.5	MOD Calif.	4,6,8			114	7	slightly moist
10	MOD Calif.	4,10,18			128	5	

Graphic Trend
Blow Count Dry Density



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

Earth Systems

1680 Illinois Ave., Suite 20, Perris, CA 92571
Phone (951) 928-9799

Boring No. B-4

Project Name: Bamiyan Marketplace
Project Number 302169-002
Boring Location: See Plate 2, Approximate Elevation 1,308 feet (MSL)

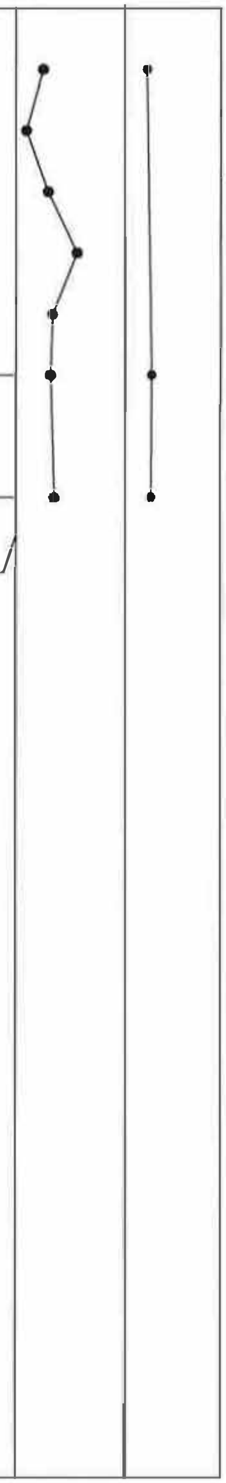
Drilling Date: December 4, 2018
Drilling Method: Mobile B-61 w/autohammer
Drill Type: 6" HSA
Logged By: D. Hamelehle

Description of Units

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

Depth (Ft.)	Sample Type (Bulk / SPT / MOD Calif.)	Penetration Resistance (Blows/6")	USCS	Dry Density (pcf)	Moisture Content (%)	Description of Units
0			SM			SILTY SAND: brown, medium dense, dry, fine to coarse grained sand, Alluvium
2.5	MOD Calif.	17,21,16		123	2	
5	MOD Calif.	5,5,9				red brown, moist, fine to medium grained sand
7.5	Bulk; MOD Calif.	21,25,19				no recovery
10	MOD Calif.	13,33,50/5"				no recovery
12.5	SPT	50/5.5"				no recovery
15	MOD Calif.	15,23,24	SP-SM	128	6	SAND WITH SILT: brown, dense, slightly moist, fine to coarse grained sand, Older Alluvium
20	MOD Calif.	12,20,32	SM	127	10	SILTY SAND: gray brown, dense, moist, fine to coarse grained sand with gravel

Graphic Trend
Blow Count Dry Density



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

Earth Systems

1680 Illinois Ave., Suite 20, Perris, CA 92571
Phone (951) 928-9799

Boring No. B-5

Project Name: Bamiyan Marketplace
Project Number 302169-002
Boring Location: See Plate 2, Approximate Elevation 1,303 feet (MSL)

Drilling Date: December 4, 2018
Drilling Method: Mobile B-61 w/autohammer
Drill Type: 6" HSA
Logged By: D. Hamelehle

Description of Units

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

Graphic Trend: Blow Count, Dry Density

Depth (Ft.)	Sample Type (Bulk / SPT / MOD Calif.)	Penetration Resistance (Blows/6")	Symbol	USCS	Dry Density (pcf)	Moisture Content (%)	Description of Units
0				SM			SILTY SAND: brown, medium dense, dry, fine to coarse grained sand, trace clay, Alluvium
	MOD Calif.	26,14,18			114	5	damp
5	MOD Calif.	6,7,7			104	11	moist
	MOD Calif.	4,5,7			116	8	
10	MOD Calif.	4,6,10			106	7	light brown, fine to medium grained sand
	Bulk; MOD Calif.	9,10,14		SM	112	5	SILTY SAND: red brown, medium dense, slightly moist, fine to coarse grained sand, Older Alluvium
15	MOD Calif.	11,27,33			134	6	gray brown, dense
20	MOD Calif.	14,25,38			127	10	brown
25	MOD Calif.	11,21,29			125	10	olive brown, moist, fine to medium grained sand
30	MOD Calif.	16,29,43		SP	130	9	SAND: olive gray, very dense, moist, fine to coarse grained sand
35							
40							
45							
50							
55							Boring completed at 31-1/2 feet Backfilled with cuttings No groundwater encountered
60							

Earth Systems

1680 Illinois Ave., Suite 20, Perris, CA 92571
Phone (951) 928-9799

Boring No. B-6

Project Name: Bamiyan Marketplace
Project Number 302169-002
Boring Location: See Plate 2, Approximate Elevation 1,306 feet (MSL)

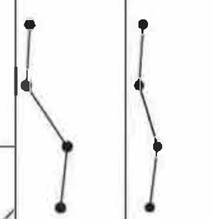
Drilling Date: December 4, 2018
Drilling Method: Mobile B-61 w/autohammer
Drill Type: 6" HSA
Logged By: D. Hamelehle

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				SM			SILTY SAND: brown, medium dense, moist, fine to coarse grained sand, trace fine gravel, Alluvium
	MOD Calif.	9,9,10			117	7	
5	MOD Calif.	5,5,9			113	10	
	MOD Calif.	13,26,44		SM	132	10	SILTY SAND: olive brown, very dense, very moist, fine to medium grained sand, Older Alluvium
10	MOD Calif.	15,26,35		SP-SM	124	8	SAND WITH SILT: olive brown, dense, moist, fine to coarse grained sand

Graphic Trend
Blow Count Dry Density



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

Earth Systems

1680 Illinois Ave., Suite 20, Perris, CA 92571
Phone (951) 928-9799

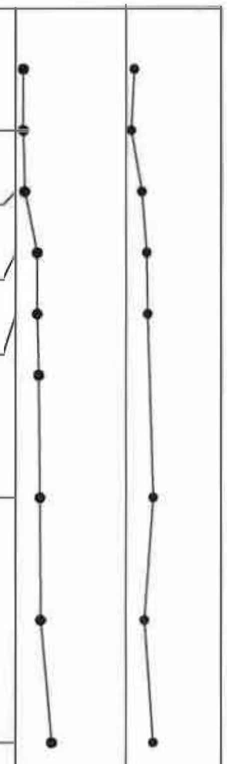
Boring No. B-7	Drilling Date: December 11, 2018
Project Name: Bamiyan Marketplace	Drilling Method: Mobile B-61 w/autohammer
Project Number 302169-002	Drill Type: 6" HSA
Boring Location: See Plate 2, Approximate Elevation 1,301 feet (MSL)	Logged By: D. Hamelchle

Description of Units

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

Depth (Ft.)	Sample Type (Bulk / SPT / MOD Calif.)	Penetration Resistance (Blows/6")	USCS	Dry Density (pcf)	Moisture Content (%)	Description of Units
0			SM			SILTY SAND: brown, loose, moist, fine to medium grained sand, trace clay, Fill
	MOD Calif.	4,5,5		108	7	
5	MOD Calif.	3,4,6	SM	105	6	SILTY SAND: light brown, loose, moist, fine to medium grained sand, Alluvium
	MOD Calif.	4,6,6	SP-SM	116	18	SAND WITH GRAVEL AND SILT: gray brown, loose, very moist, fine to medium grained sand, fine gravel
10	MOD Calif.	9,12,17	CL	121	13	SILTY SANDY CLAY: olive gray, very stiff, moist, fine to medium grained sand, Older Alluvium
	MOD Calif.	7,12,17	SC	122	13	CLAYEY SAND: brown, medium dense, very moist, fine to medium grained sand
15	MOD Calif.	8,19,21				
20	MOD Calif.	6,13,20	SM	128	11	SILTY SAND: red brown, medium dense, very moist, fine to coarse grained sand
25	MOD Calif.	6,13,21		119	15	red brown to gray brown, fine to medium grained sand
30	MOD Calif.	8,19,30	SP	128	9	SAND WITH GRAVEL: gray, dense, moist, fine to coarse grained sand, slightly micacous

Graphic Trend
Blow Count Dry Density



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

Earth Systems

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Boring No. B-8

Project Name: Bamiyan Marketplace
Project Number 302169-002
Boring Location: See Plate 2, Approximate Elevation 1,305 feet (MSL)

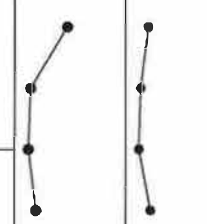
Drilling Date: December 4, 2018
Drilling Method: Mobile B-61 w/autohammer
Drill Type: 6" HSA
Logged By: D. Hamelehle

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				SM			SILTY SAND: light brown to brown, very dense, dry, fine to coarse grained sand, Alluvium
	MOD Calif.	21,50/6"			124	4	damp
5	MOD Calif.	8,10,11			116	5	brown, medium dense, slightly moist, fine to medium grained sand
	MOD Calif.	4,5,13		SM	114	9	SITY SAND: gray brown, medium dense, moist, fine to medium grained sand, Older Aluvium
10	MOD Calif.	9,12,16			126	12	very moist

Graphic Trend
Blow Count Dry Density



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

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

Earth Systems

1680 Illinois Ave., Suite 20, Perris, CA 92571
Phone (951) 928-9799

Boring No. B-9
Project Name: Bamiyan Marketplace
Project Number 302169-002
Boring Location: See Plate 2, Approximate Elevation 1,303 feet (MSL)

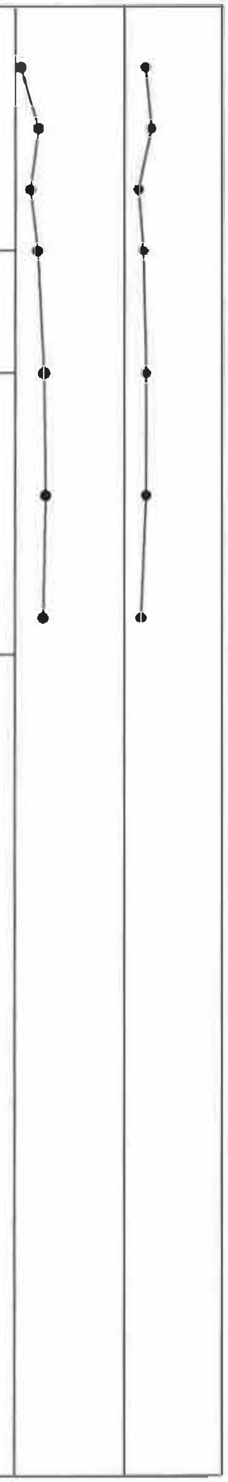
Drilling Date: December 10, 2018
Drilling Method: Mobile B-61 w/autohammer
Drill Type: 6" HSA
Logged By: S. Clanton

Description of Units

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

Depth (Ft.)	Sample Type (Bulk / SPT / MOD Calif.)	Penetration Resistance (Blows/6")	Symbol	USCS	Dry Density (pcf)	Moisture Content (%)	Description
0				SM			SILTY SAND: brown, loose, moist, fine to medium grained sand, Alluvium
2.5	MOD Calif.	4,4,6			121	10	
5	MOD Calif.	10,14,17			127	9	
7.5	MOD Calif.	5,9,11			114	12	
10	MOD Calif.	6,13,17		SC	119	14	CLAYEY SAND: gray brown, medium dense, very moist, fine to medium grained sand
15	MOD Calif.	8,15,24		SM	122	13	SILTY SAND: gray brown, medium dense, very moist, fine to coarse grained sand
20	MOD Calif.	6,16,25			122	12	
25	MOD Calif.	5,14,24			117	12	

Graphic Trend
Blow Count Dry Density



Boring completed at 26-1/2 feet
Backfilled with cuttings
No groundwater encountered

Earth Systems

1680 Illinois Ave., Suite 20, Perris, CA 92571
Phone (951) 928-9799

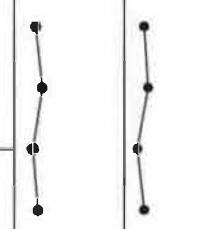
Boring No. B-10	Drilling Date: December 4, 2018
Project Name: Bamiyan Marketplace	Drilling Method: Mobile B-61 w/autohammer
Project Number 302169-002	Drill Type: 6" HSA
Boring Location: See Plate 2, Approximate Elevation 1,300 feet (MSL)	Logged By: D. Hamelehle

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				SM			SILTY SAND: gray brown, medium dense, dry, fine to medium grained sand, Alluvium
	MOD Calif.	10,13,16			120	7	slightly moist
5	MOD Calif.	8,15,23			124	9	light brown, dense, damp, fine to coarse grained sand
	MOD Calif.	8,9,16		SM	113	16	SILTY SAND: mottled orange, gray, brown, medium dense, moist, fine to medium grained sand, Older Alluvium,
10	MOD Calif.	8,12,20			120	14	very moist

Graphic Trend
Blow Count Dry Density



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

Earth Systems

1680 Illinois Ave., Suite 20, Perris, CA 92571
Phone (951) 928-9799

Boring No. B-11
Project Name: Bamiyan Marketplace
Project Number 302169-002
Boring Location: See Plate 2, Approximate Elevation 1,314 feet (MSL)

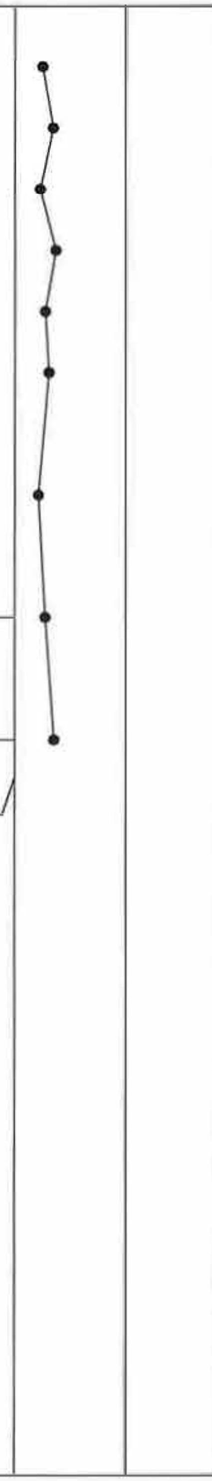
Drilling Date: December 11, 2018
Drilling Method: Mobile B-61 w/autohammer
Drill Type: 6" HSA
Logged By: D. Hamelehle

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				SM			SILTY SAND: brown, medium dense, dry, fine to coarse grained sand, Older Alluvium
2.5	MOD Calif.	8,16,21					
5	MOD Calif.	10,21,30			121	8	gray brown, dense moist
7.5	MOD Calif.	15,15,19			127	9	medium dense
10	MOD Calif.	16,21,34			129	10	red brown, dense, fine to medium grained sand, very moist
12.5	MOD Calif.	7,17,24					brown, medium dense, moist, slightly micaceous
15	MOD Calif.	11,21,25			128	10	
20	MOD Calif.	6,15,17			129	10	
25	MOD Calif.	11,17,24		SP	124	12	SAND: brown, medium dense, wet, fine to medium grained sand
28							groundwater
30	MOD Calif.	9,21,32		SM	118	14	SILTY SAND: brown, dense, wet, fine to coarse grained sand

Graphic Trend
Blow Count Dry Density



Boring completed at 31-1/2 feet
Backfilled with cuttings, sealed with bentonite
Groundwater encountered at 28 feet

Earth Systems
1680 Illinois Ave., Suite 20, Perris, CA 92571
Phone (951) 928-9799

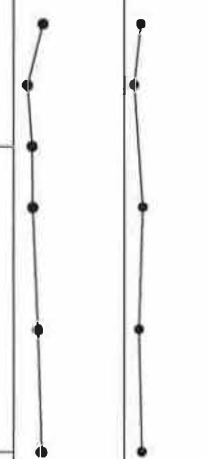
Boring No. B-12	Drilling Date: December 10, 2018
Project Name: Bamiyan Marketplace	Drilling Method: Mobile B-61 w/autohammer
Project Number 302169-002	Drill Type: 6" HSA
Boring Location: See Plate 2, Approximate Elevation 1,305 feet (MSL)	Logged By: S. Clanton

Description of Units

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

Depth (Ft.)	Sample Type (Bulk / SPT / MOD Calif.)	Penetration Resistance (Blows/6")	USCS	Dry Density (pcf)	Moisture Content (%)	Description of Units
0			SM			SILTY SAND: brown, dense, damp, fine to medium grained sand, trace clay, Alluvium
	MOD Calif.	29,23,17		117	7	slightly moist
5	MOD Calif.	7,8,11		110	6	
	MOD Calif.	8,10,15	SM			SILTY SAND: olive brown, medium dense, moist, fine to medium grained sand
10	MOD Calif.	8,10,16		119	13	trace clay
15	MOD Calif.	7,15,18		115	10	gray brown
20	MOD Calif.	12,14,24	SM	118	14	SILTY SAND WITH CLAY: reddish brown, medium dense, moist, fine to medium grained sand

Graphic Trend
Blow Count Dry Density



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

Earth Systems

1680 Illinois Ave., Suite 20, Perris, CA 92571
Phone (951) 928-9799

Boring No. B-13
Project Name: Bamiyan Marketplace
Project Number 302169-002
Boring Location: See Plate 2, Approximate Elevation 1,305 feet (MSL)

Drilling Date: December 10, 2018
Drilling Method: Mobile B-61 w/autohammer
Drill Type: 6" HSA
Logged By: S. Clanton

Description of Units

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

Depth (Ft.)	Sample Type (Bulk / SPT / MOD Calif.)	Penetration Resistance (Blows/6")	Symbol	USCS	Dry Density (pcf)	Moisture Content (%)	Description of Units
0				SM			SITLY SAND: brown, medium dense, damp, fine to medium grained sand, fill
2.5	MOD Calif.	11,11,11			114	3	
5	MOD Calif.	6,8,10		SM	107	4	SILTY SAND: light brown, medium dense, dry, fine to coarse grained sand, Alluvium
7.5	MOD Calif.	6,12,13			109	6	
10	MOD Calif.	10,11,13			104	9	
15	MOD Calif.	9,21,25			112	6	reddish brown, dense, fine to coarse grained sand, dense older alluvium
20	MOD Calif.	2,8,14			116	15	dark gray brown, medium dense, moist
25	MOD Calif.	11,19,34		SM	120	13	SILTY SAND: brown, wet, dense fine to coarse grained sand
28							groundwater
30	MOD Calif.	15,26,35			130	12	
35	MOD Calif.	21,35,50			111	11	very dense, with clay

Graphic Trend
Blow Count Dry Density

Boring refusal at 39 feet
Backfilled with cuttings, sealed with bentonite
Groundwater encountered at 28 feet

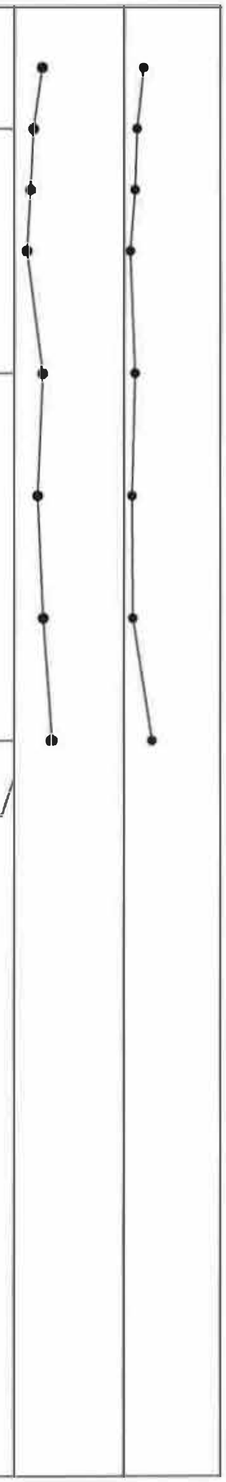
Earth Systems

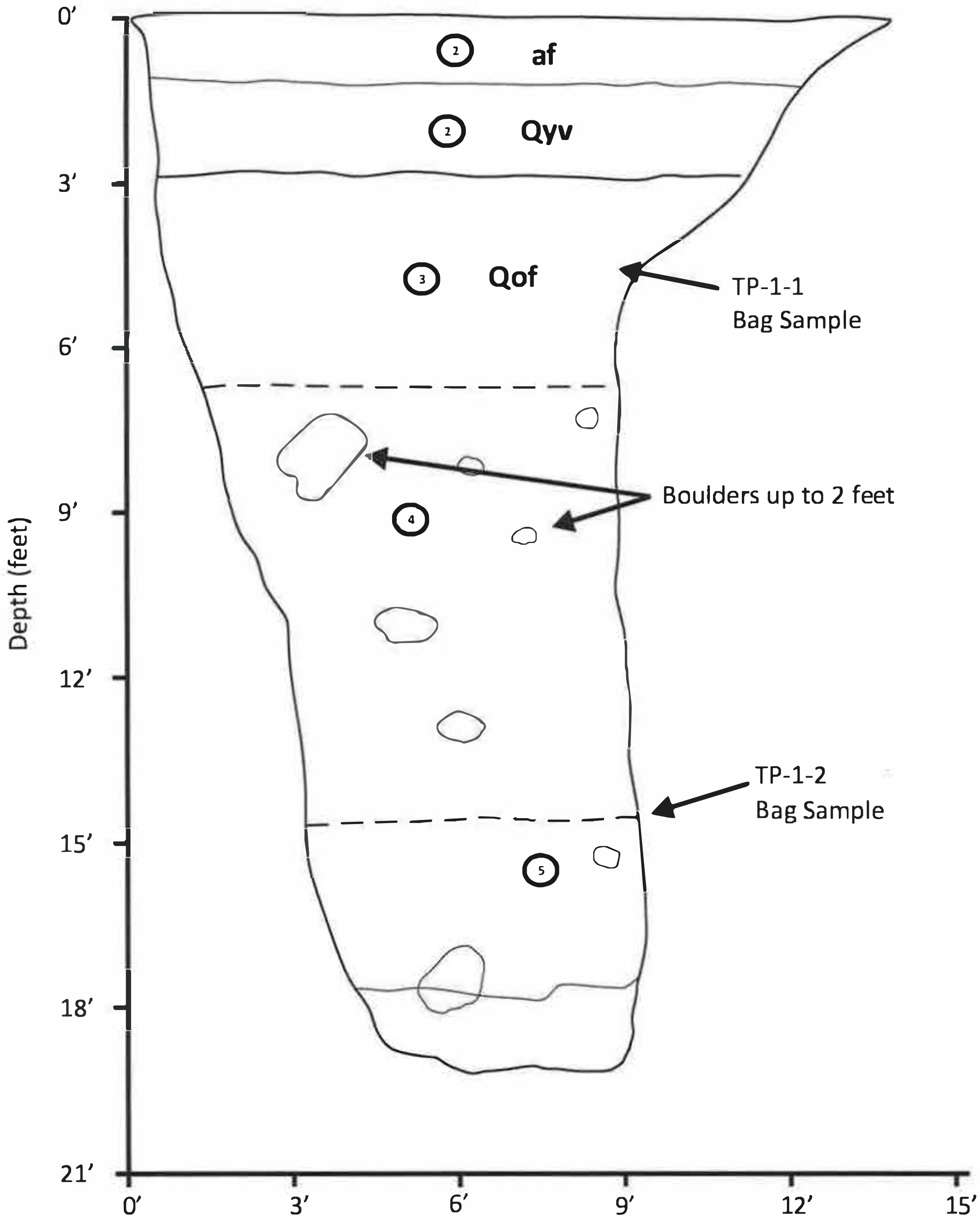
1680 Illinois Ave., Suite 20, Perris, CA 92571
Phone (951) 928-9799

Boring No. B-14	Drilling Date: December 10, 2018
Project Name: Bamiyan Marketplace	Drilling Method: Mobile B-61 w/autohammer
Project Number 302169-002	Drill Type: 6" HSA
Boring Location: See Plate 2, Approximate Elevation 1,302 feet (MSL)	Logged By: S. Clanton

Depth (Ft.)	Sample Type (Bulk / SPT / MOD Calif.)	Penetration Resistance (Blows/6")	Symbol	USCS	Dry Density (pcf)	Moisture Content (%)	Description of Units Note: The stratification lines shown represent the approximate boundary between soil and/or rock types and the transition may be gradational.
0				SM			SITLY SAND: reddish brown, medium dense, damp, fine to medium grained sand, fill
	MOD Calif.	7,16,22			120	5	
5	MOD Calif.	10,12,14		SM	113	5	SILTY SAND: brown, medium dense, slightly moist, fine to medium grained sand, Alluvium
	MOD Calif.	6,9,13			111	4	light brown, fine to coarse grained sand
10	MOD Calif.	8,5,12			106	4	
15	MOD Calif.	11,17,22		SM	111	8	SILTY SAND: reddish brown, medium dense, moist, fine to coarse grained sand, Older Alluvium
20	MOD Calif.	14,14,18			108	6	
25	MOD Calif.	12,17,23			109	10	some gravel
30	MOD Calif.	10,20,32		SM	129	12	SILTY SAND: brown, dense, moist, fine to coarse grained sand
35							
40							
45							
50							
55							Boring completed at 31-1/2 feet Backfilled with cuttings No groundwater encountered
60							

Graphic Trend
Blow Count Dry Density





Approximate Elevation 1,307 feet

TP-1

Descriptions

2 af, Fill, medium dense (SM)
f-c silty sand, trace gravel, dry,
dark yellowish brown (10yr 3/4).

2 Qyv, Young alluvial fan deposits (SM)
medium dense, silty f-c sand, trace
gravel, dry, dark reddish brown
(5yr 3/4). Abundant pinholes.

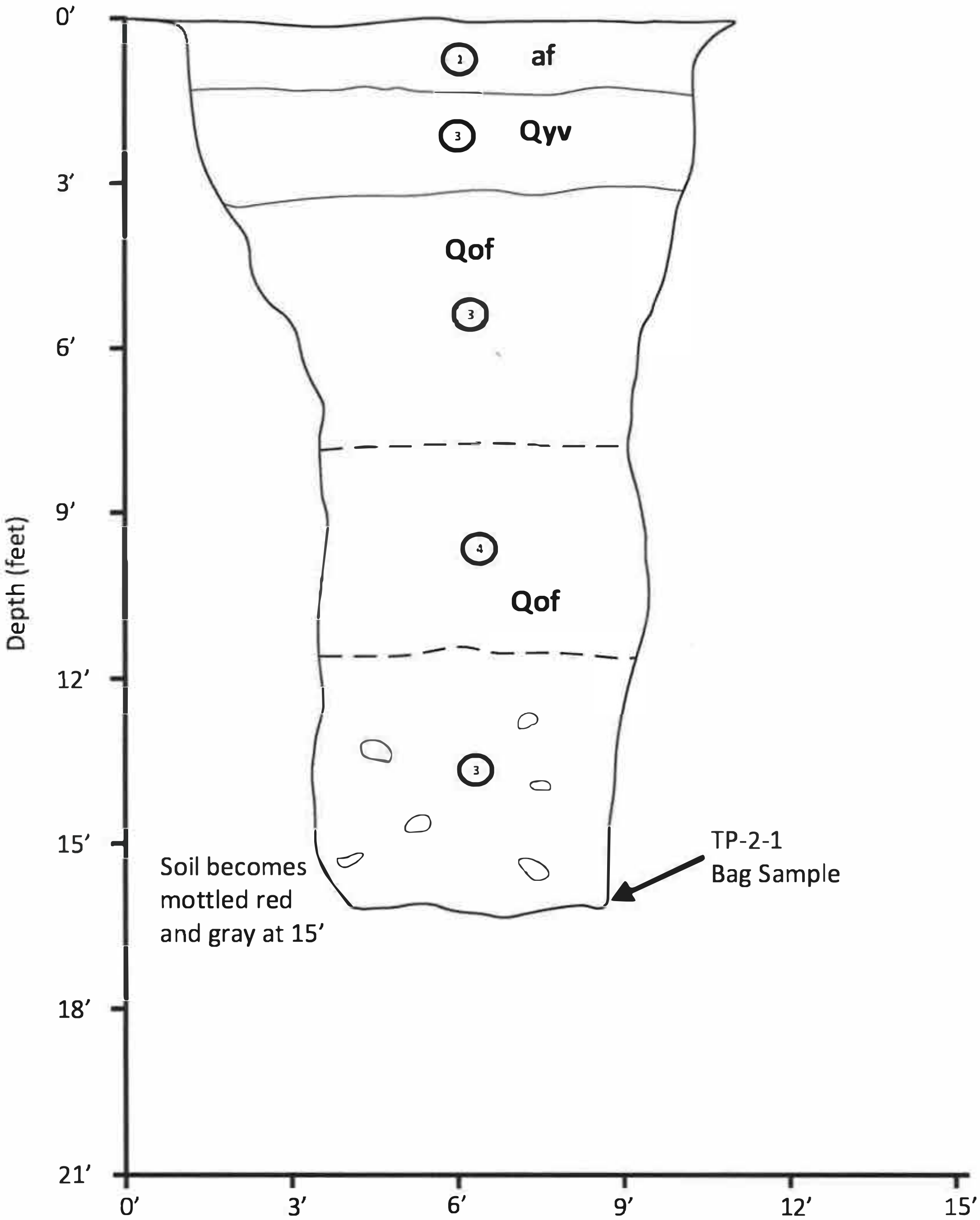
3 Qof, Old alluvial fan deposits (SM), Qa,
medium dense, f-c sand with silt,
moist, dark reddish brown,
(2.5yr 3/3). Some pinholes.

4 Qof, Old alluvial fan deposits (SM)
medium dense, silty f-c sand with
boulders subrounded up to 2 feet
long.

5 Qof, Old alluvial fan deposits (SC)
medium dense, f-c sand trace silt,
with boulders subrounded up to 18",
moist grayish brown (2.5y 5/2).

Test Pit Logs	
Proposed Bamiyan Marketplace 15749 Grand Avenue, Lake Elsinore Riverside County, CA	
Earth Systems	
1/17/2019	302169-001

TP-2



Approximate Elevation 1,308 feet

Descriptions

3 af, Fill, medium dense (SM) silty f-c sand, trace gravel, dry, brown (7.5yr 4/4). Some pinholes.

1 Qyv, Young alluvial fan deposits (SM) medium dense, silty f-c sand, some gravel, dry, reddish brown (5yr 4/4). Some pinholes.

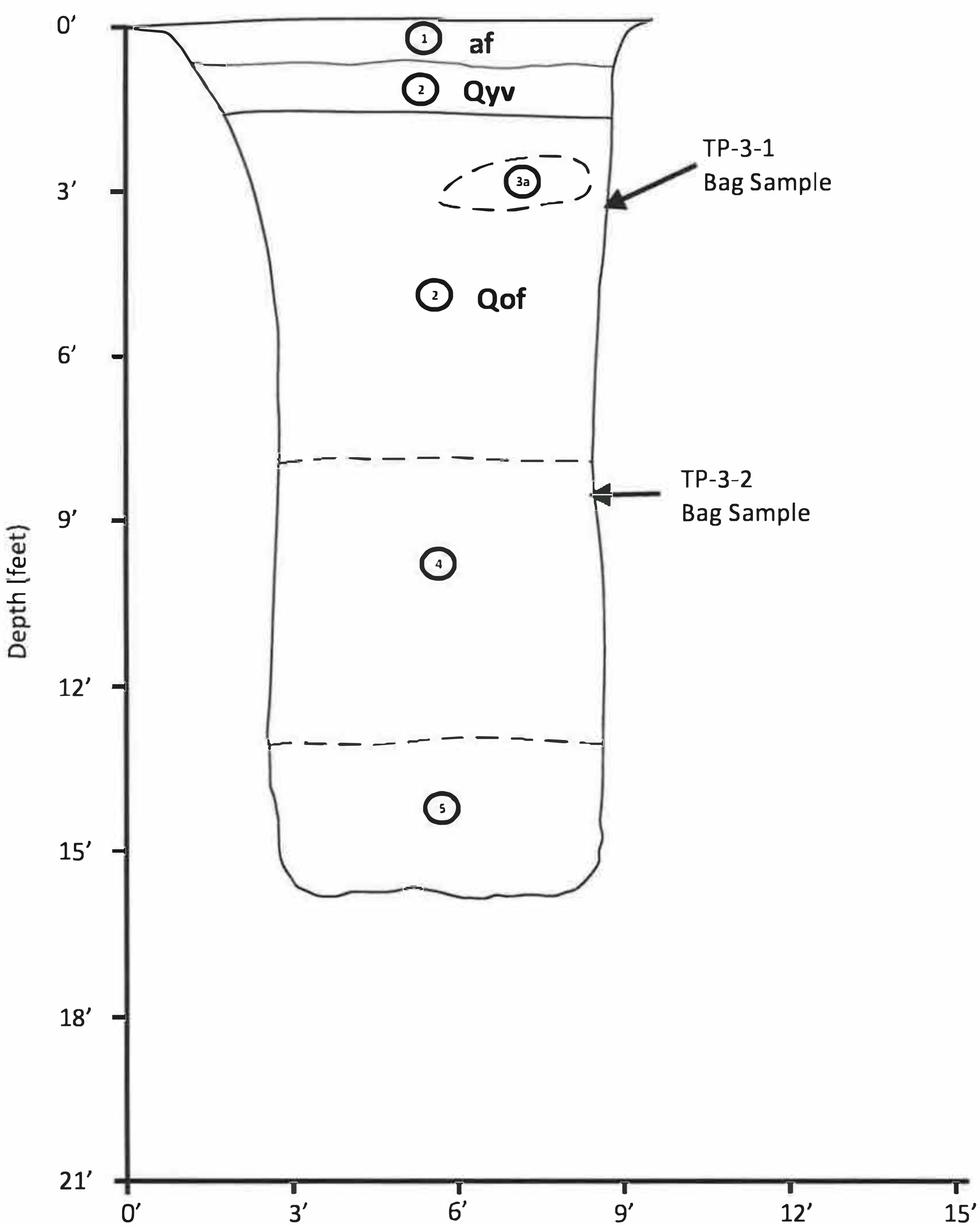
3 Qof, Old alluvial fan deposits (SM) medium dense, f-c sand with silt, trace gravel, moist, reddish brown, (5yr 4/3).

4 Qof, Old alluvial fan deposits, (SM) Qa, medium dense, silty f-m sand, moist, dark reddish brown (2.5yr 3/3). Few pinholes.

3 Qof, Old alluvial fan deposits (SC) medium dense, f-c sand trace silt, dry to moist, dusty red (2.5yr 3/2). Some subround cobbles up to 8".

Test Pit Logs	
Proposed Bamiyan Marketplace 15749 Grand Avenue, Lake Elsinore Riverside County, CA	
Earth Systems	
1/17/2019	302169-001

TP-3



Approximate Elevation 1,302 feet

Descriptions

1 af, Fill, medium dense (SM) silty f-c sand, some gravel, dry, brown (7.5yr 4/3).

2 Qyv, Young alluvial fan deposits (SM) medium dense, silty f-c sand, trace gravel, dry, brown (7.5yr 4/2).

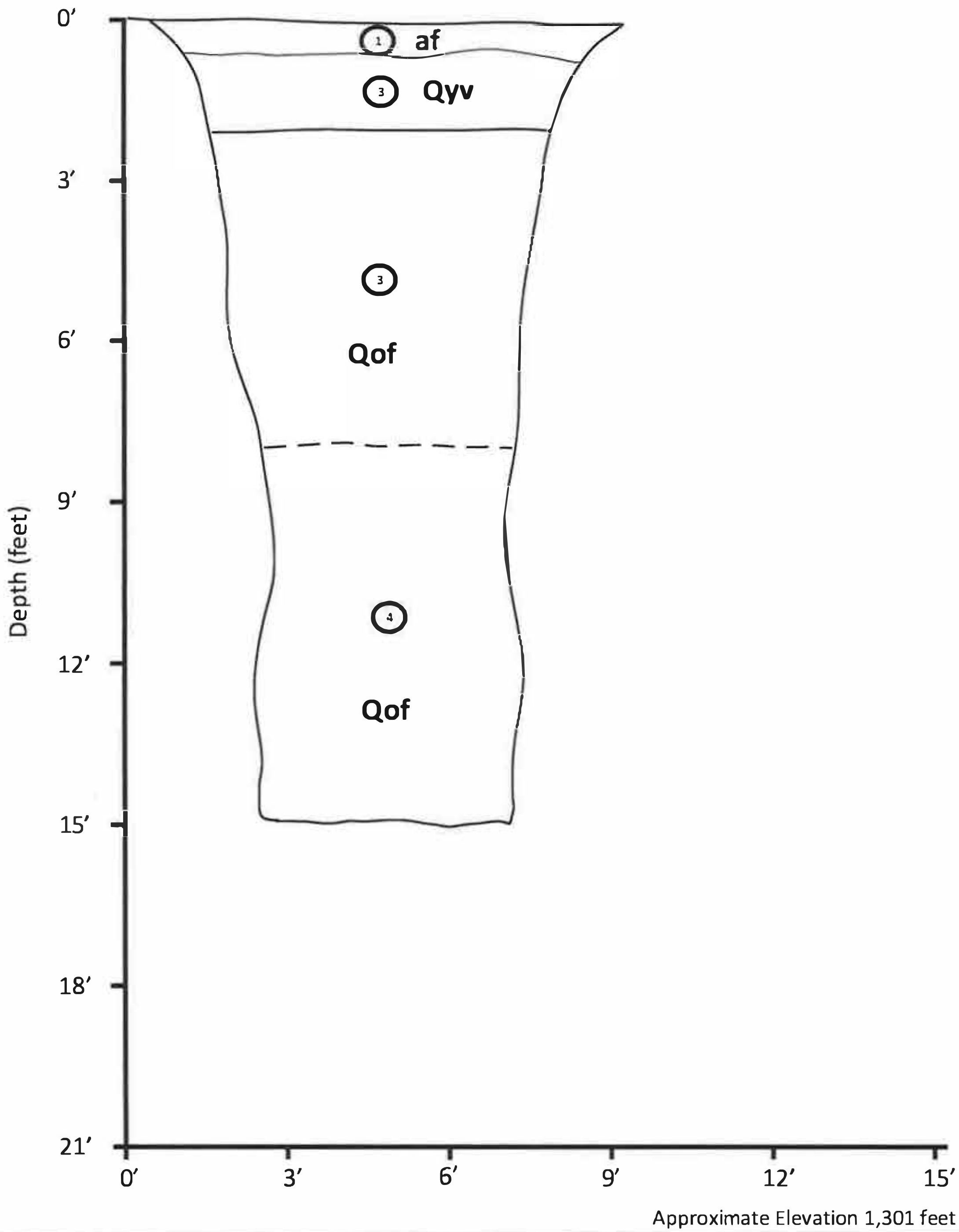
3 Qof, Old alluvial fan deposits (SC) medium dense, f-c sand, trace gravel, moist, mottled 5 to 8, yellowish red 5yr 4/6 to dark reddish brown (5yr 3/3).

3a Qof, Old alluvial fan deposits Dense f-c sand, moist, dark gray (2.5y 4/1)

4 Qof, Old alluvial fan deposits (CL to SC) medium dense to firm, sandy clay to Oayey f-c sand, very moist, mottled dark gray (5y 4/1) to olive yellow (2.5y 6/6).

5 Qof, Old alluvial fan deposits (SC) medium dense, f-c sand trace silt, very moist, gray (2.5y 5/1). Some mottled dark reddish brown (2.5yr 5/4).

Test Pit Logs	
Proposed Bamiyan Marketplace 15749 Grand Avenue, Lake Elsinore Riverside County, CA	
Earth Systems	
1/17/2019	302169-001



TP-4

Descriptions

1 af, Fill, medium dense (SM)
silty f-c sand, dry, brown (10y 4/3).
Some pinholes.

3 Qyv, Young alluvial fan deposits (SM)
medium dense, silty f-c sand, trace
gravel, dry, brown (7.5yr 4/4).

3 Qof, Old alluvial fan deposits (SM)
medium dense, f-c sand with silt,
damp, reddish brown,(5yr 4/4).

4 Qof, Old alluvial fan deposits (SC)
medium dense, silty f-c sand,
trace gravel, moist, mottled dark
grayish brown 2.5y 4/2 to olive yellow
2.5y 6/6. Few pinholes.

Test Pit Logs	
Proposed Bamiyan Marketplace 15749 Grand Avenue, Lake Elsinore Riverside County, CA	
Earth Systems	
1/17/2019	302169-001

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

Project: Bamiyan Marketplace
Job No: 302169-002
Date: 1/17/2019
Boring: B-2 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: 7.7 7.5
PGA, g: 0.91 0.97
MSF: 0.93
GWT: 5.0 feet
Calc GWT: 5.0 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: 0.31
Required SF: 1.50
Minimum Calculated SF: 0.34

Total (In.) Induced Subsidence
1.0

upper 50 ft

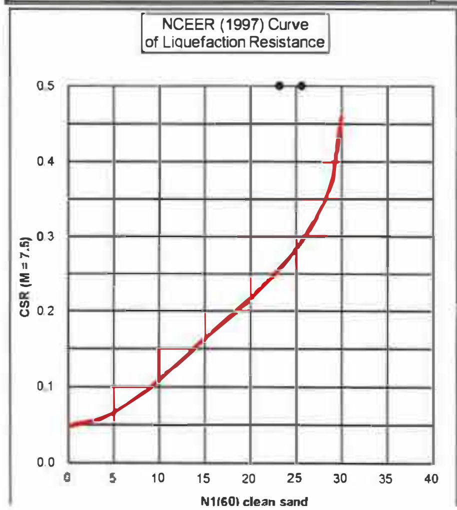
Base Depth (feet)	Cal Mod N	SPT N	Liquef. Suscept. (0 or 1)	Total Unit Wt (pcf)	Fines Content (%)	Depth of SPT (feet)	Rod Length (feet)
5.0	36	50	1	122	24	2.5	5.5
7.5	14	9	1	117	24	5.0	8.0
10.0	18	11	1	122	24	7.5	10.5
12.5	20	13	1	113	33	10.0	13.0
14.5	63	40	1	106	10	12.5	15.5
20.0	57	36	1	146	10	15.0	18.0
25.0	41	26	1	126	40	20.0	23.0
30.0	83	52	1	140	40	25.0	28.0
35.0	66	42	1	146	40	30.0	33.0
40.0	57	36	1	142	40	35.0	38.0
45.0	49	31	1	145	40	40.0	43.0
50.0	88	55	1	143	40	45.0	48.0
50.5	100	63	1	143	40	50.0	53.0

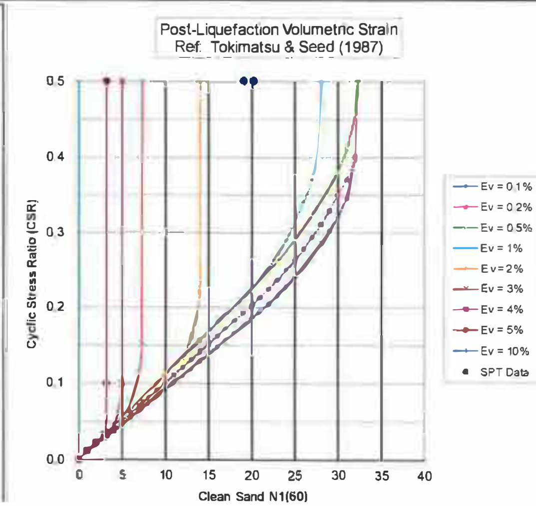
Base Depth (feet)	Tot Stress at SPT po (tsf)	Eff Stress at SPT p'o (tsf)	rd	C_N	C_R	C_S	$N_{1(60)}$	Rel Dens. Dr (%)	Trigger FC Adj. $\Delta N_{1(60)}$	Equiv. Sand $N_{1(60)cs}$	Kσ	M = 7.5 Available CRR	M = 7.5 Induced CSR*	Liquefac. Safety Factor	Post FC Adj $\Delta N_{1(60)}$	$N_{1(60)cs}$	Volumetric Strain (%)	Induced Subsidence (in.)
	0.000																	
5.0	0.153	0.153	1.00	1.70	0.75	1.00	76.5	###	10.0	86.5	1.00	1.200	0.631	**Non-Liq.**	10.0	86.5	0.03	**0.02**
7.5	0.305	0.305	0.99	1.70	0.75	1.00	13.5	44	5.6	19.1	1.00	0.206	0.627	**Non-Liq.**	5.6	19.1	0.00	**0.00**
10.0	0.451	0.373	0.98	1.68	0.75	1.00	17.2	50	6.0	23.2	1.00	0.256	0.753	**0.34**	1.9	19.1	1.65	**0.49**
12.5	0.604	0.448	0.98	1.54	0.76	1.00	17.6	50	8.0	25.7	1.00	0.294	0.836	**0.35**	2.6	20.2	1.57	**0.47**
14.5	0.745	0.511	0.97	1.44	0.82	1.00	56.0	89	2.1	58.1	1.00	1.200	0.899	**1.34**	2.1	58.1	0.00	**0.00**
20.0	0.888	0.576	0.97	1.36	0.86	1.00	50.5	85	2.0	52.4	1.00	1.200	0.945	**1.27**	2.0	52.4	0.00	**0.00**
25.0	1.253	0.785	0.96	1.16	0.93	1.00	33.6	69	10.0	43.6	1.00	1.200	0.967	**1.24**	10.0	43.6	0.00	**0.00**
30.0	1.568	0.944	0.94	1.06	0.98	1.00	65.3	97	10.0	75.3	1.00	1.200	0.990	**1.21**	10.0	75.3	0.00	**0.00**
35.0	1.918	1.138	0.92	0.96	1.00	1.00	48.1	83	10.0	58.1	0.97	1.200	1.011	**1.19**	10.0	58.1	0.00	**0.00**
40.0	2.283	1.347	0.89	0.89	1.00	1.00	38.2	74	10.0	48.2	0.91	1.200	1.052	**1.14**	10.0	48.2	0.00	**0.00**
45.0	2.638	1.546	0.85	0.83	1.00	1.00	30.6	66	10.0	40.6	0.89	1.200	1.030	**1.17**	10.0	40.6	0.00	**0.00**
50.0	3.000	1.752	0.80	0.78	1.00	1.00	51.7	86	10.0	61.7	0.82	1.200	1.066	**1.13**	10.0	61.7	0.00	**0.00**
50.5	3.358	1.954	0.75	0.74	1.00	1.00	55.6	89	10.0	65.6	0.78	1.200	1.046	**1.15**	10.0	65.6	0.00	**0.00**

SETTLEMENT (SUBSIDENCE) OF DRY SANDS

Nc = 17.1

Base Depth (feet)	p (tsf)	G_{max} (tsf)	τ_{av} (tsf)	Shear Strain γ	Strain E_{15}	Strain Enc	Dry Sand Subsidence (in.)
5.0	0.102	632	0.090	7.0E-04	1.2E-04	1.3E-04	**0.02**
7.5	0.204	540	0.179	9.6E-03			
10.0	0.302	701	0.263	6.5E-03			
12.5	0.405	838	0.350	5.4E-03			
14.5	0.499	1,223	0.429	1.6E-03			
20.0	0.595	1,290	0.508	1.9E-03			
25.0	0.839	1,441	0.709	2.6E-03			
30.0	1.050	1,935	0.873	1.4E-03			
35.0	1.285	1,963	1.044	1.9E-03			
40.0	1.529	2,012	1.202	2.3E-03			
45.0	1.767	2,043	1.328	2.6E-03			
50.0	2.010	2,504	1.426	1.5E-03			
50.5	2.250	2,704	1.495	1.3E-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))

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

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

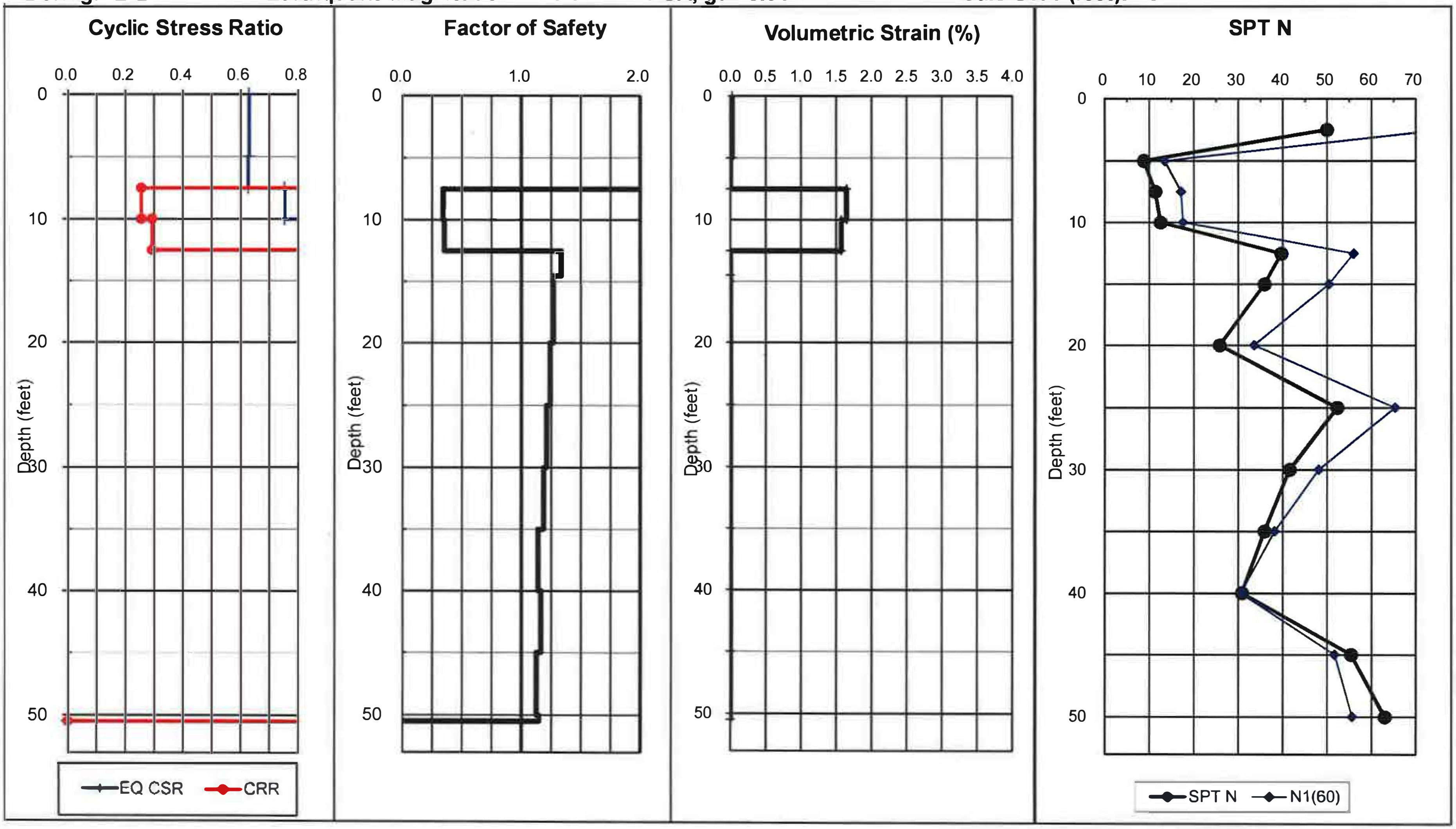
$SF = CRR_{7.5,1atm}/CSR^*$

EARTH SYSTEMS - EVALUATION OF LIQUEFACTION POTENTIAL AND INDUCED SUBSIDENCE

Bamiyan Marketplace **Project No: 302169-002** 1996/1998 NCEER Method

Ground Compaction Remediated to 5 foot depth

Boring: B-2 **Earthquake Magnitude: 7.7** **PGA, g: 0.91** **Calc GWT (feet): 5**



Total Thickness of Liquefiable Layers: 0.0 feet

Estimated Total Ground Subsidence: 1.0 inches

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

Project: Bamiyan Marketplace
Job No: 302169-002
Date: 1/17/2019
Boring: B-13 Data Set: 5

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: 7.7 7.5
PGA, g: 0.91 0.97
MSF: 0.93
GWT: 5.0 feet
Calc GWT: 5.0 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

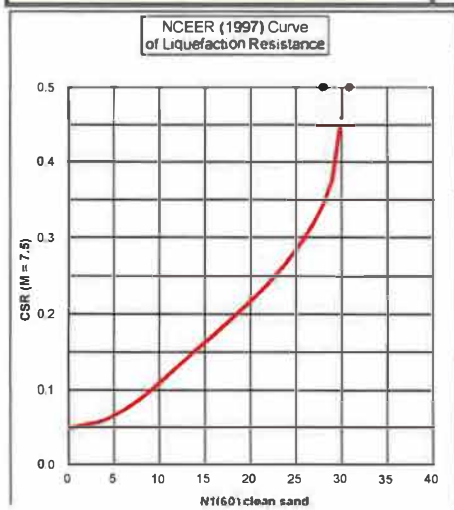
Threshold Acceler., g: 0.31
Required SF: 1.50
Minimum Calculated SF: 0.34

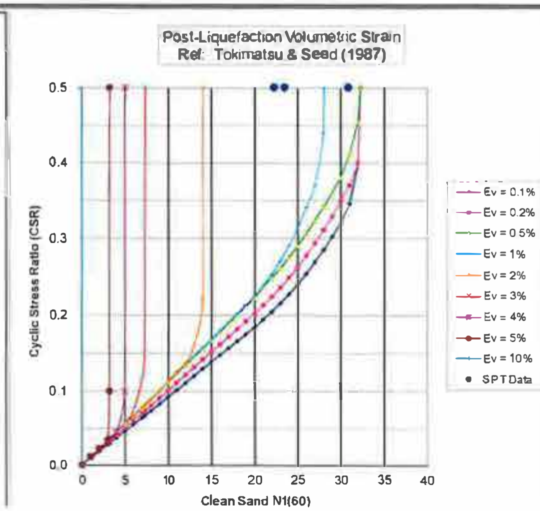
Total (in.) Induced Subsidence: 1.9 upper 50 ft

SETTLEMENT (SUBSIDENCE) OF DRY SANDS

Nc = 17.1

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	22	50	1	118	24	2.5	5.5	0.148	0.148	1.00	1.70	0.75	1.00	76.5	###	10.0	86.5	1.00	1.200	0.631	Non-Liq.	10.0	86.5	0.03	0.02	0.099	621	0.087	7.0E-04	1.2E-04	1.3E-04	0.02
7.5	18	11	1	117	24	5.0	8.0	0.295	0.295	0.99	1.70	0.75	1.00	17.4	50	6.0	23.4	1.00	0.259	0.627	Non-Liq.	6.0	23.4	0.00	0.00	0.198	568	0.173	6.3E-03			
9.5	25	16	1	116	24	7.5	10.5	0.441	0.363	0.98	1.70	0.75	1.00	24.1	59	6.8	30.9	1.00	1.200	0.757	1.59	6.8	30.9	0.68	0.16	0.296	762	0.257	3.9E-03			
15.0	24	15	1	113	24	10.0	13.0	0.586	0.430	0.98	1.57	0.76	1.00	21.6	56	6.5	28.1	1.00	0.345	0.845	0.41	1.9	23.5	1.34	0.88	0.392	851	0.339	4.6E-03			
20.0	46	29	1	119	20	15.0	18.0	0.868	0.556	0.97	1.38	0.86	1.00	41.4	77	6.9	48.4	1.00	1.200	0.957	1.25	6.9	48.4	0.00	0.00	0.582	1,242	0.497	2.1E-03			
25.0	22	14	1	134	40	20.0	23.0	1.166	0.698	0.96	1.23	0.93	1.00	19.1	52	8.8	27.9	1.00	0.342	1.012	0.34	3.2	22.3	1.43	0.86	0.781	1,199	0.660	4.9E-03			
30.0	53	33	1	136	40	25.0	28.0	1.501	0.877	0.94	1.10	0.98	1.00	43.3	79	10.0	53.3	1.00	1.200	1.021	1.18	10.0	53.3	0.00	0.00	1.005	1,687	0.836	2.1E-03			
35.0	61	38	1	146	40	30.0	33.0	1.841	1.061	0.92	1.00	1.00	1.00	46.1	81	10.0	56.1	1.00	1.200	1.012	1.19	10.0	56.1	0.00	0.00	1.233	1,900	1.002	2.0E-03			
39.0	85	54	1	123	40	35.0	38.0	2.206	1.270	0.89	0.91	1.00	1.00	58.7	92	10.0	68.7	0.93	1.200	1.053	1.14	10.0	68.7	0.00	0.00	1.478	2,225	1.162	1.6E-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+N1(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))

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

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+0.000613 6*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

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

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

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

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

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

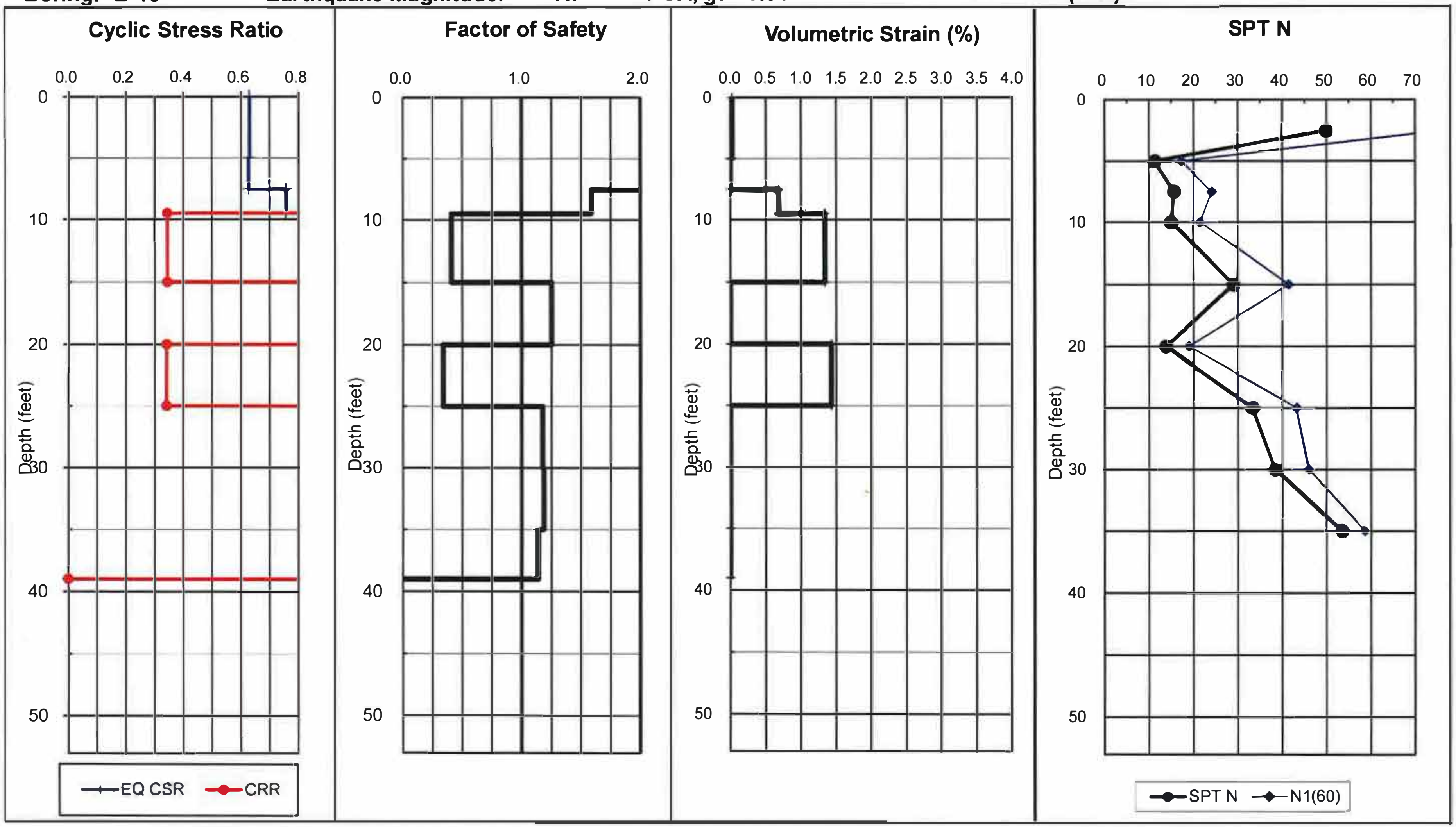
$S = 2{*}H{*}E_{nc}$

EARTH SYSTEMS - EVALUATION OF LIQUEFACTION POTENTIAL AND INDUCED SUBSIDENCE

Bamiyan Marketplace **Project No: 302169-002** 1996/1998 NCEER Method

Ground Compaction Remediated to 5 foot depth

Boring: B-13 **Earthquake Magnitude: 7.7** **PGA, g: 0.91** **Calc GWT (feet): 5**



Total Thickness of Liquefiable Layers: 0.0 feet

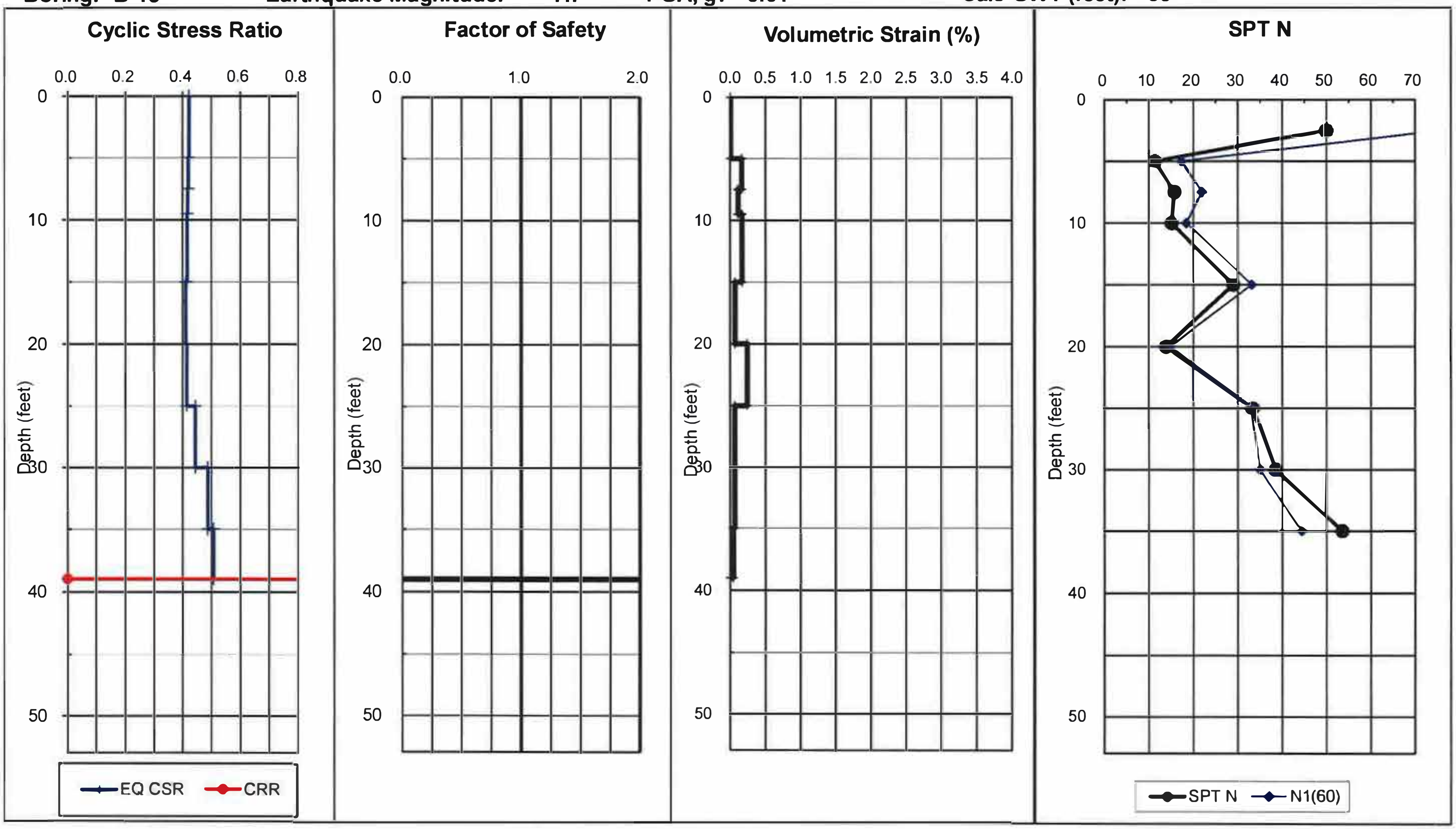
Estimated Total Ground Subsidence: 1.9 inches

EARTH SYSTEMS - EVALUATION OF LIQUEFACTION POTENTIAL AND INDUCED SUBSIDENCE

Bamiyan Marketplace **Project No: 302169-002** 1996/1998 NCEER Method

Ground Compaction Remediated to 5 foot depth

Boring: B-13 **Earthquake Magnitude: 7.7** **PGA, g: 0.61** **Calc GWT (feet): 50**



Total Thickness of Liquefiable Layers: 0.0 feet

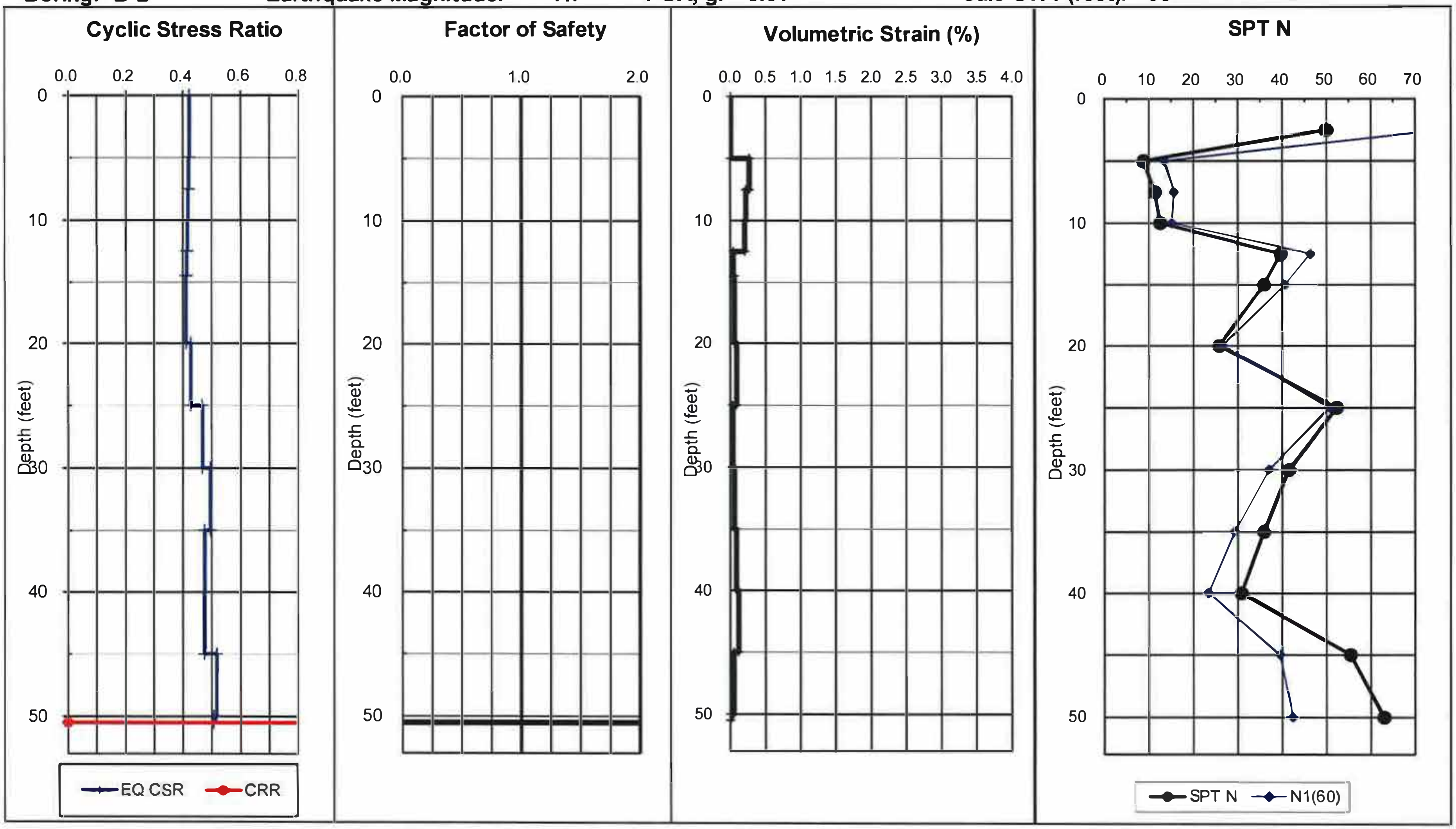
Estimated Total Ground Subsidence: 0.5 inches

EARTH SYSTEMS - EVALUATION OF LIQUEFACTION POTENTIAL AND INDUCED SUBSIDENCE

Bamiyan Marketplace **Project No: 302169-002** 1996/1998 NCEER Method

Ground Compaction Remediated to 5 foot depth

Boring: B-2 **Earthquake Magnitude: 7.7** **PGA, g: 0.61** **Calc GWT (feet): 50**



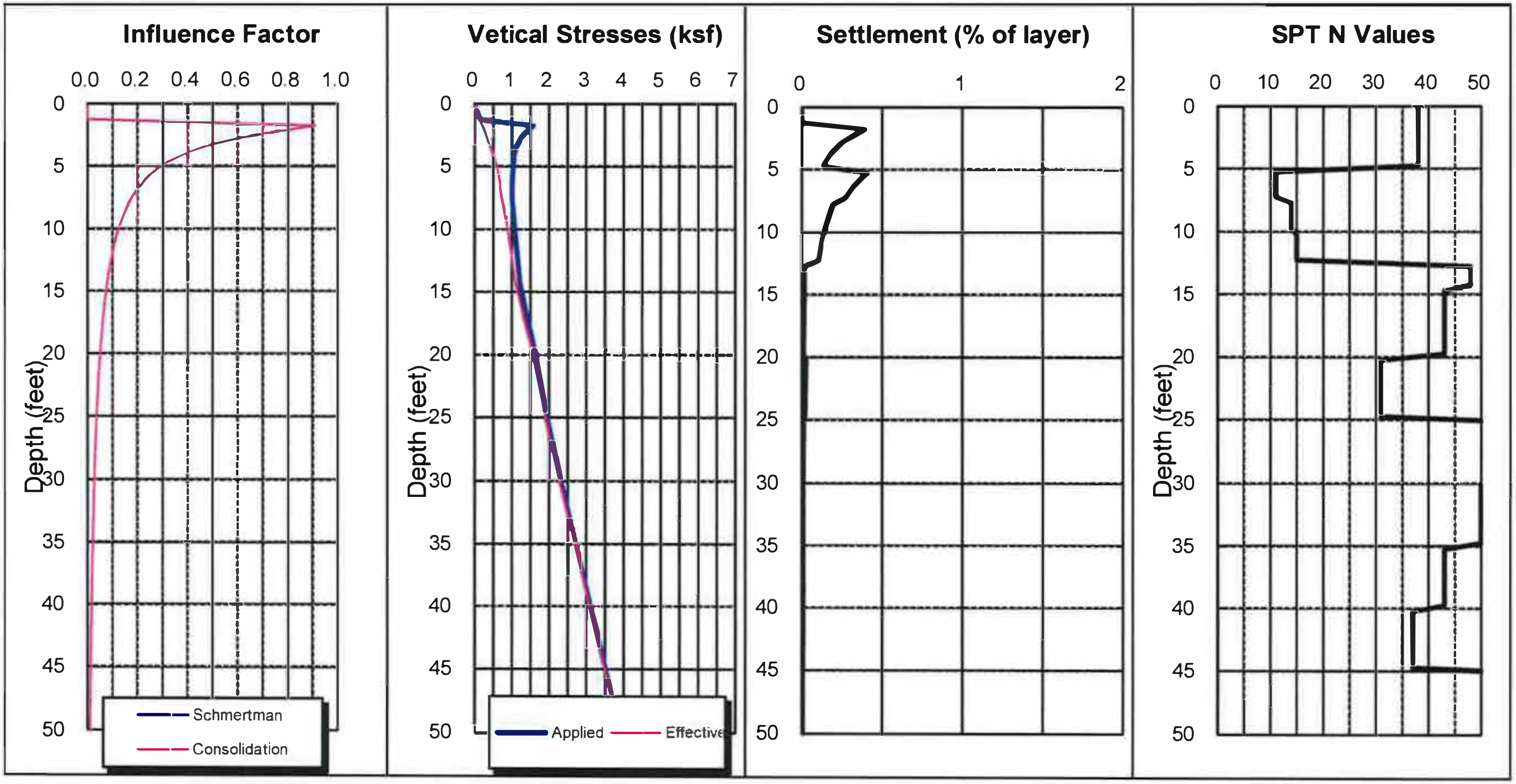
Total Thickness of Liquefiable Layers: 0.0 feet **Estimated Total Ground Subsidence: 0.5 inches**

EARTH SYSTEMS SOUTHWEST - SETTLEMENT ANALYSES

Bamiyan Marketplace

302169-002

Width, ft: 2.5 Length, ft: 40.0 Net pressure, ksf: 1.50 **Settlement, inches: 0.34**



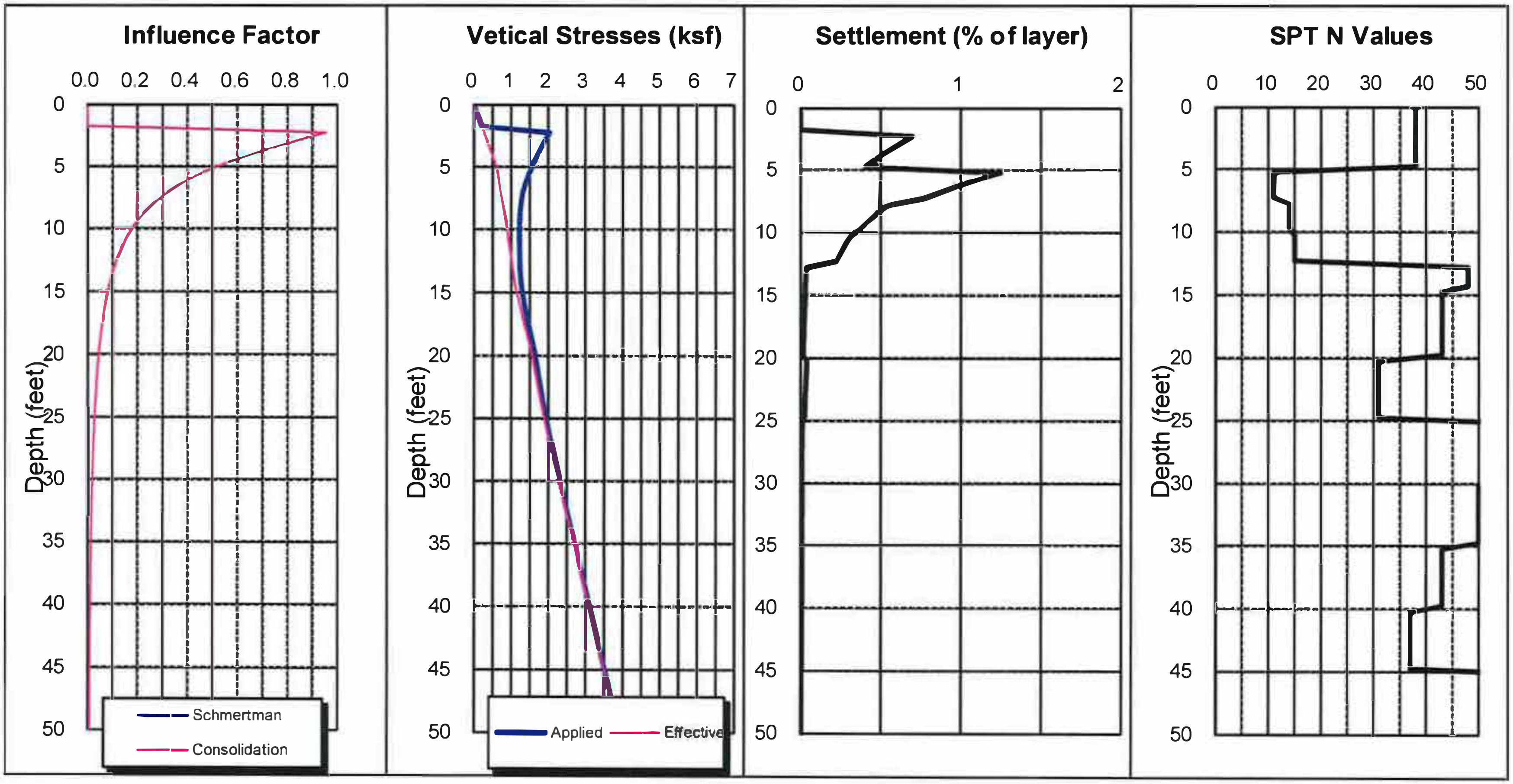
Load, Q: 4 kpf Embedment, feet: 1.5 **Boring: B-2**

EARTH SYSTEMS SOUTHWEST - SETTLEMENT ANALYSES

Bamiyan Marketplace

302169-002

Width, ft: 7.0 Length, ft: 7.0 Net pressure, ksf: 1.85 **Settlement, inches: 0.79**



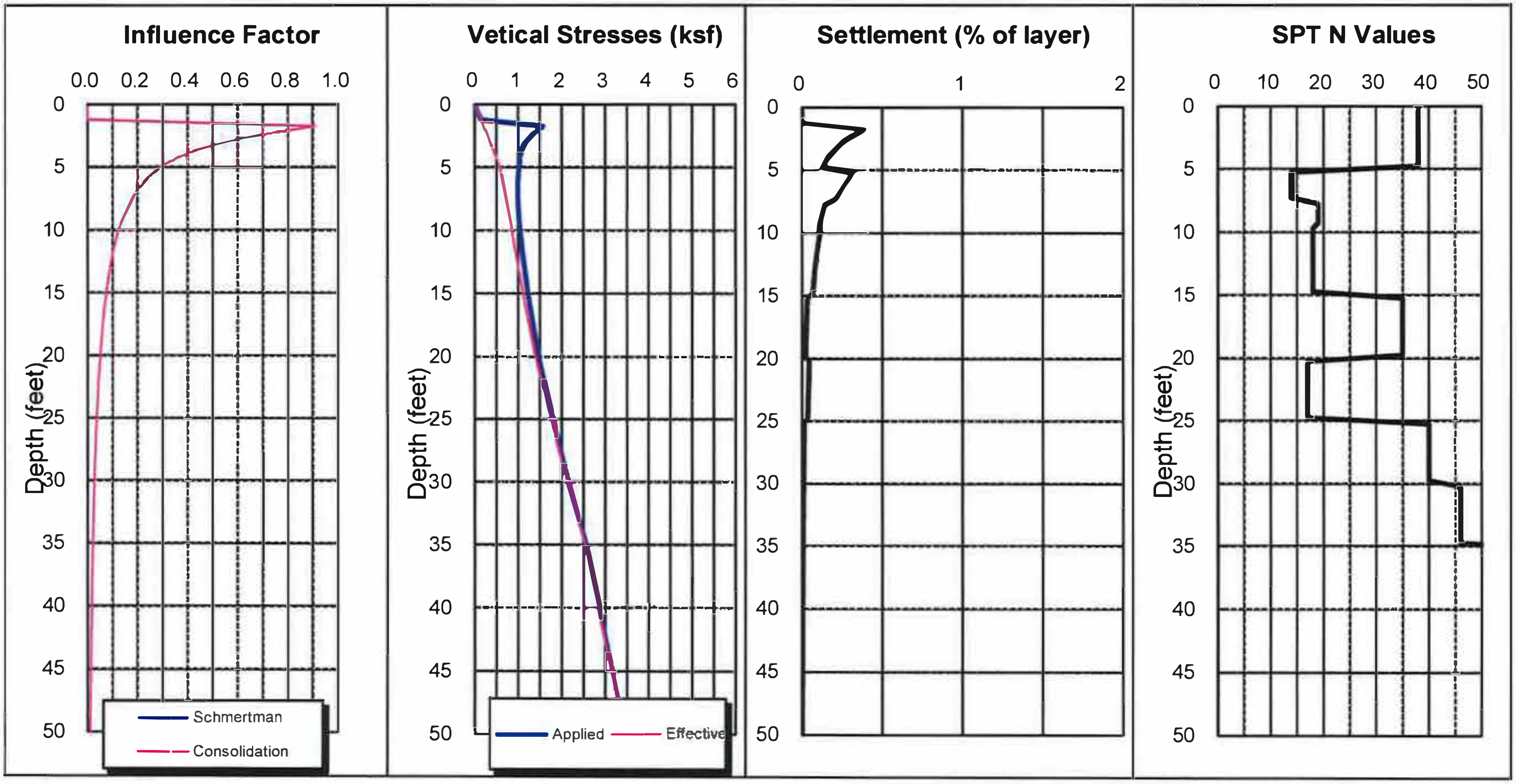
Load, Q: 91 kips Embedment, feet: 2.0 **Boring: B-2**

EARTH SYSTEMS SOUTHWEST - SETTLEMENT ANALYSES

Bamiyan Marketplace

302169-002

Width, ft: 2.5 | Length, ft: 40.0 | Net pressure, ksf: 1.50 | **Settlement, inches: 0.34**



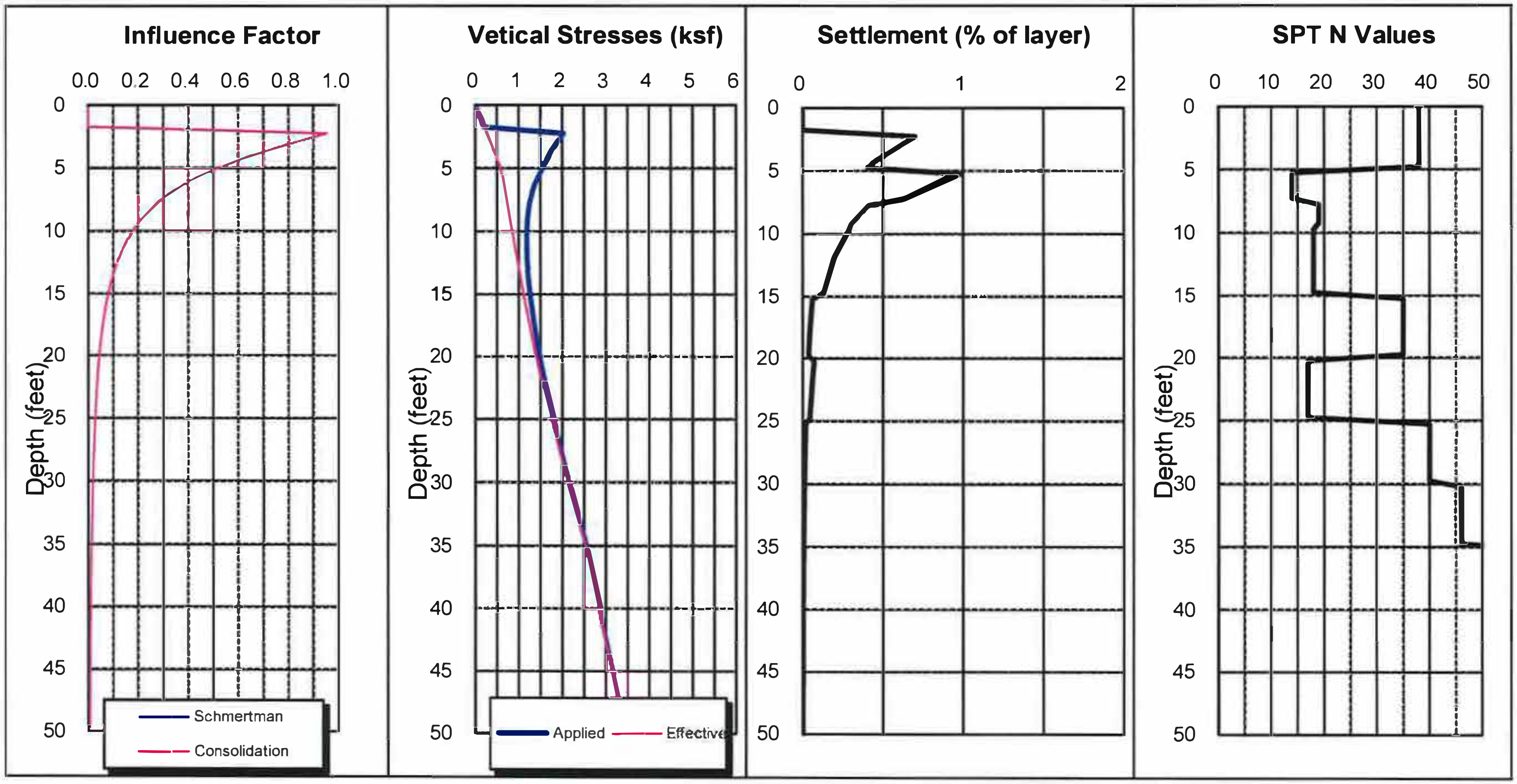
Load, Q: 4 kpf | Embedment, feet: 1.5 | **Boring: B-13**

EARTH SYSTEMS SOUTHWEST - SETTLEMENT ANALYSES

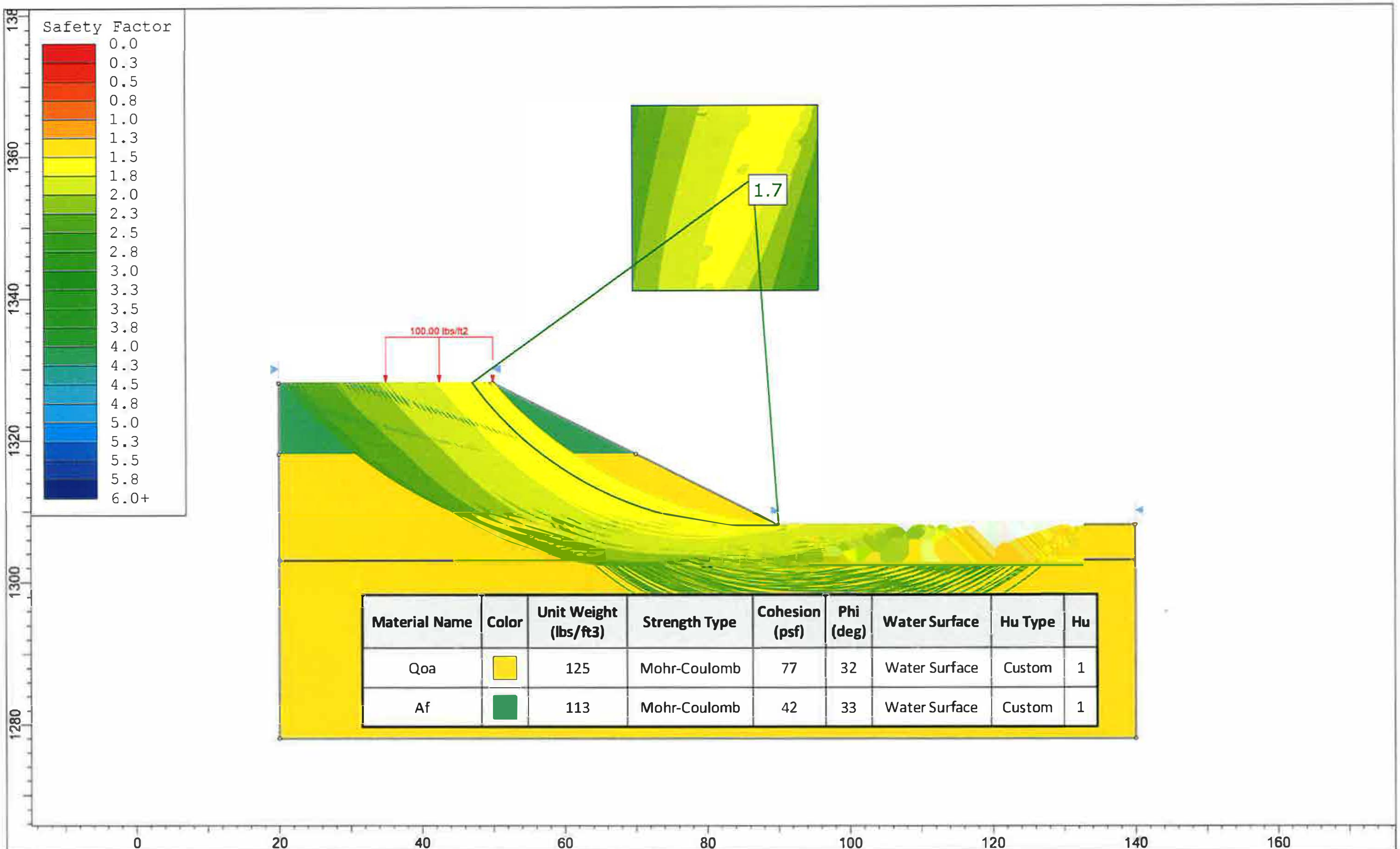
Bamiyan Marketplace

302169-002

Width, ft: 7.0 Length, ft: 7.0 Net pressure, ksf: 1.85 **Settlement, inches: 0.74**



Load, Q: 91 kips Embedment, feet: 2.0 **Boring: B-13**



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface	Hu Type	Hu
Qoa		125	Mohr-Coulomb	77	32	Water Surface	Custom	1
Af		113	Mohr-Coulomb	42	33	Water Surface	Custom	1

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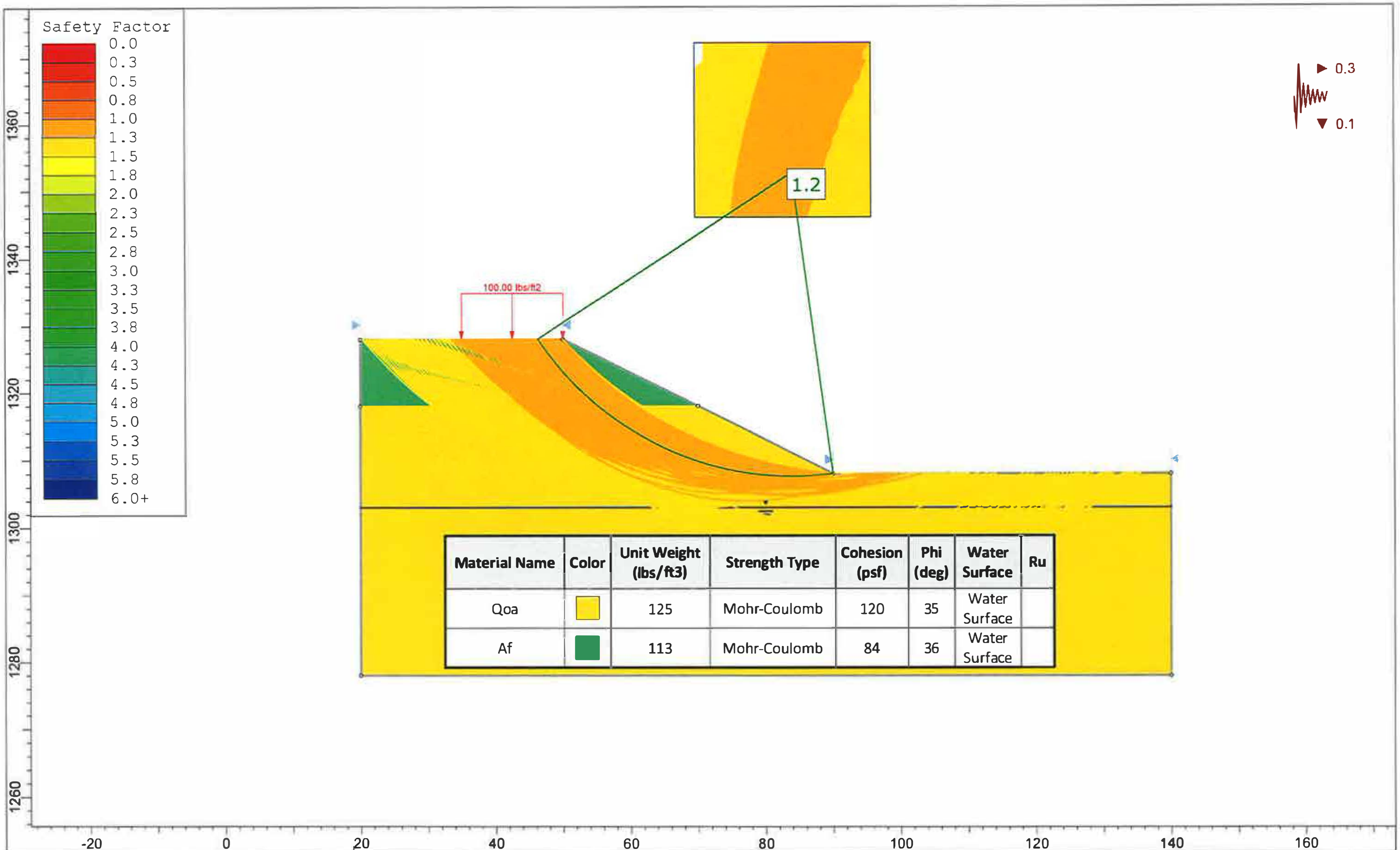
SLIDEINTERPRET 8.020

Project	Bamiyan Marketplace
Analysis Description	Current Condition - Static
Drawn By	
Scale	1:223
Company	Earth Systems
Date	1/9/2019, 12:22:48 PM
File Name	302169-002_Bamiyan Marketplace.slmd



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface	Ru
Qoa		125	Mohr-Coulomb	120	35	Water Surface	
Af		113	Mohr-Coulomb	84	36	Water Surface	

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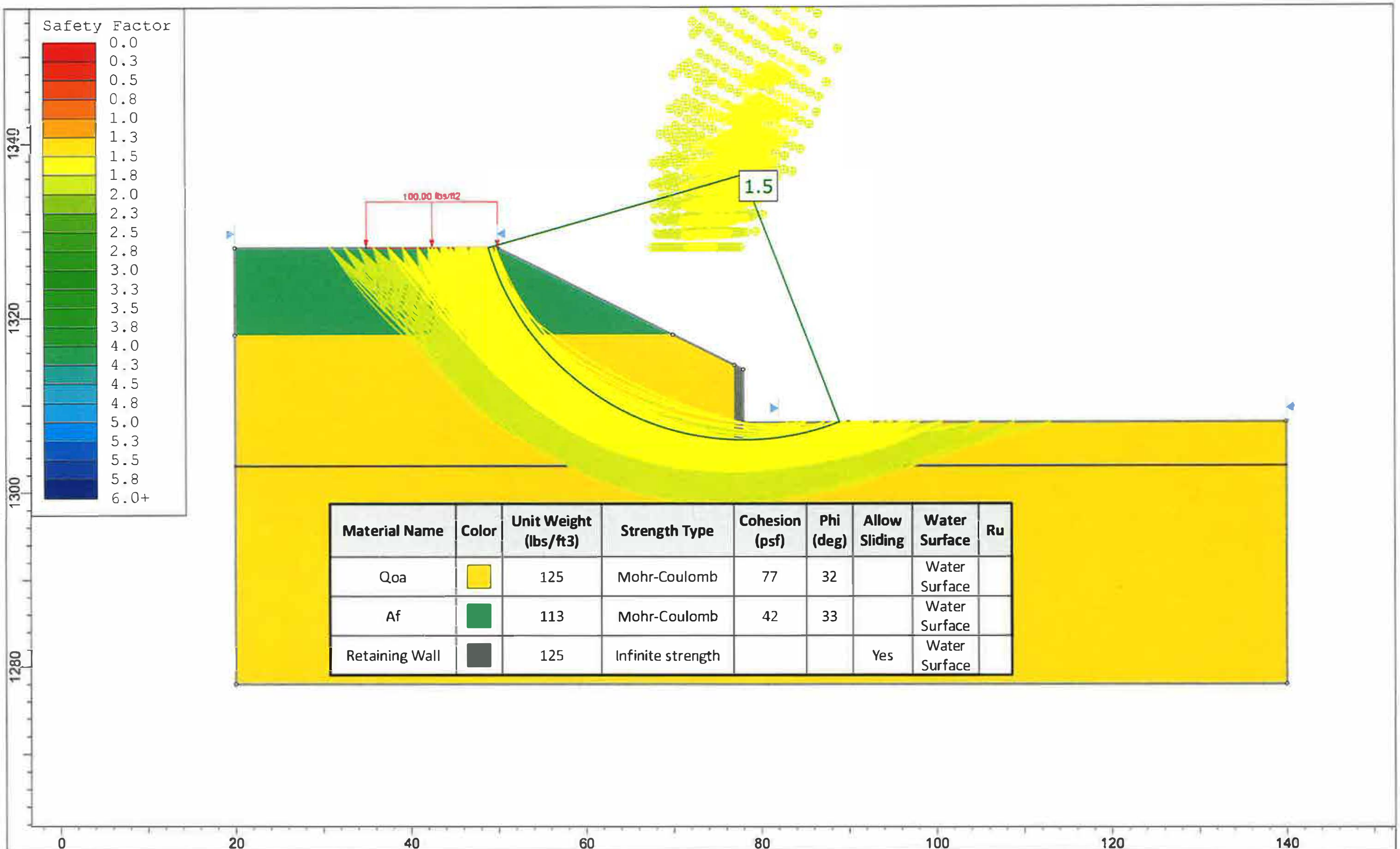
SLIDEINTERPRET 8.020

Project	Bamiyan Marketplace		
Analysis Description	Current Condition - Seismic		
Drawn By	*Scale* 1:235	*Company*	Earth Systems
Date	1/9/2019, 12:22:48 PM	*File Name*	302169-002_Bamiyan Marketplace.slmd



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Allow Sliding	Water Surface	Ru
Qoa		125	Mohr-Coulomb	77	32		Water Surface	
Af		113	Mohr-Coulomb	42	33		Water Surface	
Retaining Wall		125	Infinite strength			Yes	Water Surface	

rocscience

SLIDEINTERPRET 8.020

Project Bamiyan Marketplace

Analysis Description Proposed Retaining Wall

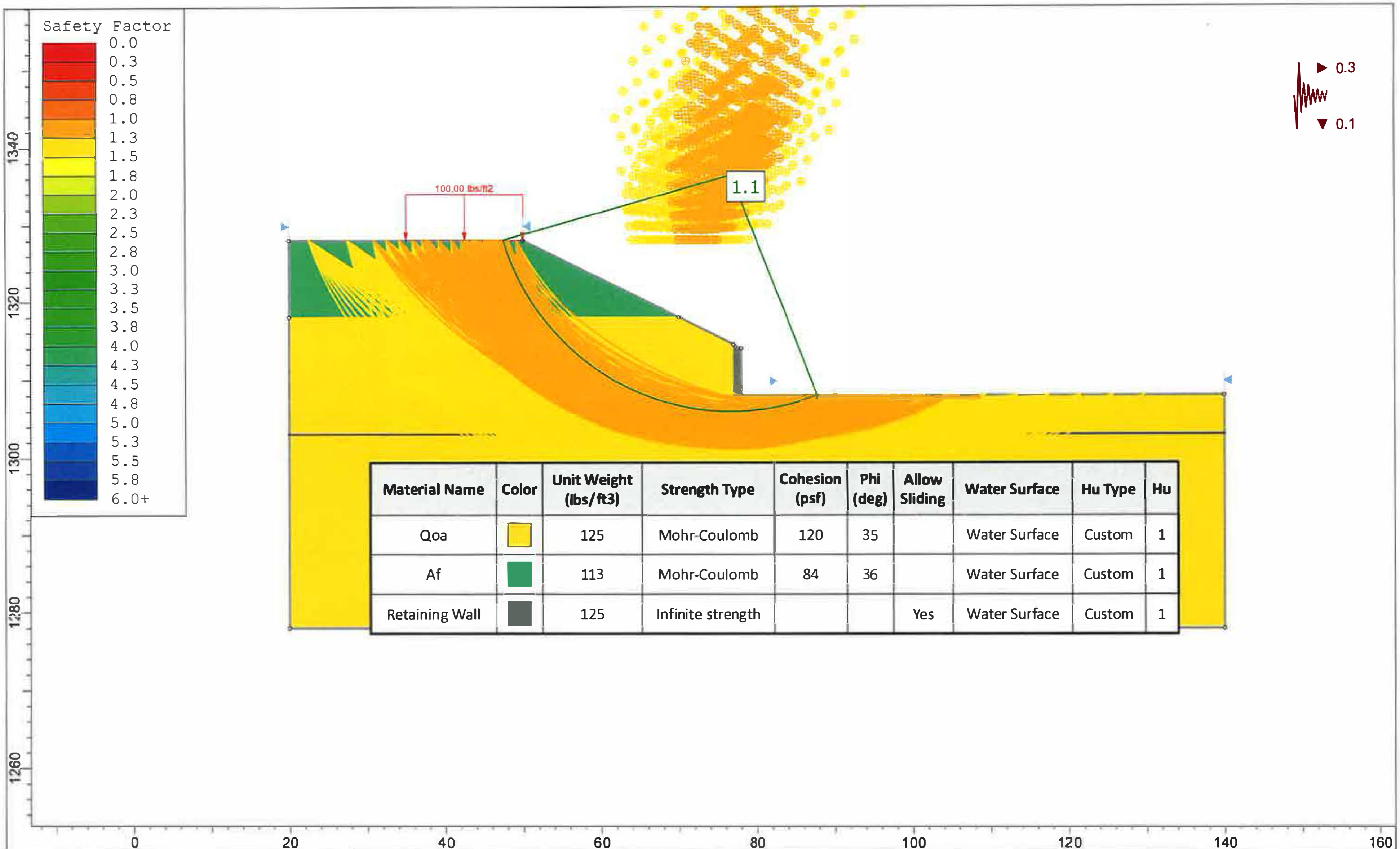
Drawn By

Scale 1:181

Company Earth Systems

Date 1/9/2019, 12:22:48 PM

File Name 302169-002_Bamiyan Marketplace.slmd



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Allow Sliding	Water Surface	Hu Type	Hu
Qoa		125	Mohr-Coulomb	120	35		Water Surface	Custom	1
Af		113	Mohr-Coulomb	84	36		Water Surface	Custom	1
Retaining Wall		125	Infinite strength			Yes	Water Surface	Custom	1

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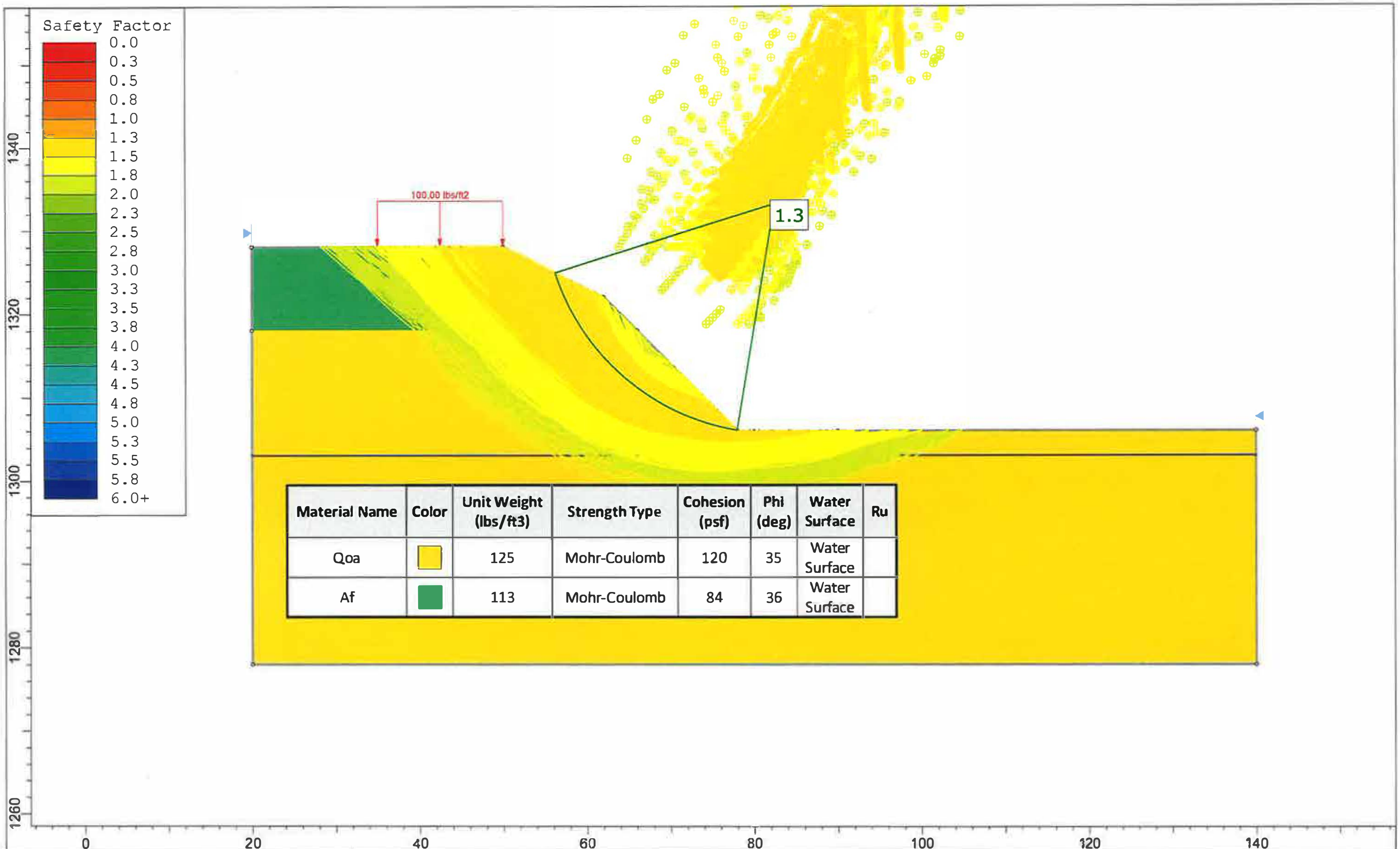
SLIDEINTERPRET 8.020

Project	Bamiyan Marketplace
Analysis Description	Proposed Retaining Wall - Seismic
Drawn By	
Scale	1:204
Company	Earth Systems
Date	1/9/2019, 12:22:48 PM
File Name	302169-002_Bamiyan Marketplace.slmd



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface	Ru
Qoa		125	Mohr-Coulomb	120	35	Water Surface	
Af		113	Mohr-Coulomb	84	36	Water Surface	

rocscience

SLIDEINTERPRET 8.020

Project	Bamiyan Marketplace		
Analysis Description	Temporary Construction		
Drawn By		*Scale* 1:190	*Company* Earth Systems
Date	1/9/2019, 12:22:48 PM	*File Name*	302169-002_Bamiyan Marketplace.slmd

APPENDIX B

Laboratory Test Results

File No.: 302169-002 January 17, 2019
Lab No.: 18-178

UNIT DENSITIES AND MOISTURE CONTENT ASTM D2937 & D2216

Job Name: Bamiyan Marketplace

Sample Location	Depth (feet)	Unit Dry Density (pcf)	Moisture Content (%)	USCS Group Symbol
B1	2.5	111	5	SM
B1	5	112	6	SM
B1	7.5	111	4	SM
B1	10	112	4	SM
B1	12.5	110	7	SM
B1	15	114	7	SM
B1	20	131	4	SP
B2	2.5	115	5	SM
B2	5	112	5	SM
B2	7.5	116	5	SM
B2	10	108	4	SM
B2	12.5	101	5	SP
B2	15	--	3	SP
B2	20	119	6	SM
B2	25	123	14	SM
B2	30	141	4	SM
B2	35	130	9	SM
B2	40	132	9	SM
B2	45	129	11	SM
B3	5	115	4	SM
B3	7.5	114	7	SM
B3	10	128	5	SM

File No.: 302169-002 January 17, 2019

Lab No.: 18-178

UNIT DENSITIES AND MOISTURE CONTENT ASTM D2937 & D2216

Job Name: Bamiyan Marketplace

Sample Location	Depth (feet)	Unit Dry Density (pcf)	Moisture Content (%)	USCS Group Symbol
B4	2.5	123	2	SM
B4	15	128	6	SP-SM
B4	20	127	10	SM
B5	2.5	114	5	SM
B5	5	104	11	SM
B5	7.5	116	8	SM
B5	10	106	7	SM
B5	12.5	112	5	SM
B5	15	134	6	SM
B5	20	127	10	SM
B5	25	125	10	SM
B5	30	130	9	SP
B6	2.5	117	7	SM
B6	5	113	10	SM
B6	7.5	132	10	SM
B6	10	124	8	SP-SM
B7	5	108	7	SM
B7	7.5	105	6	SP-SM
B7	10	116	18	CL
B7	12.5	121	13	SC
B7	15	122	13	SC
B7	20	128	11	SM
B7	25	119	15	SM
B7	30	128	9	SP

File No.: 302169-002 January 17, 2019

Lab No.: 18-178

UNIT DENSITIES AND MOISTURE CONTENT ASTM D2937 & D2216

Job Name: Bamiyan Marketplace

Sample Location	Depth (feet)	Unit Dry Density (pcf)	Moisture Content (%)	USCS Group Symbol
B8	2.5	124	4	SM
B8	5	116	5	SM
B8	7.5	114	9	SM
B8	10	126	12	SM
B9	2.5	121	10	SM
B9	5	127	9	SM
B9	7.5	114	12	SM
B9	10	119	14	SC
B9	15	122	13	SM
B9	20	122	12	SM
B9	25	117	12	SM
B10	2.5	120	7	SM
B10	5	124	9	SM
B10	7.5	113	16	SM
B10	10	120	14	SM
B11	5	121	8	SM
B11	7.5	127	9	SM
B11	10	129	10	SM
B11	15	128	10	SM
B11	20	129	10	SM
B11	25	124	12	SP
B11	30	118	14	SM

File No.: 302169-002 January 17, 2019
Lab No.: 18-178

UNIT DENSITIES AND MOISTURE CONTENT ASTM D2937 & D2216

Job Name: Bamiyan Marketplace

Sample Location	Depth (feet)	Unit Dry Density (pcf)	Moisture Content (%)	USCS Group Symbol
B12	2.5	117	7	SM
B12	5	110	6	SM
B12	10	119	13	SM
B12	15	115	10	SM
B12	20	118	14	SM
B13	2.5	114	3	SM
B13	5	107	4	SM
B13	7.5	109	6	SM
B13	10	104	9	SM
B13	15	112	6	SM
B13	20	116	15	SM
B13	25	120	13	SM
B13	30	130	12	SM
B13	35	111	11	SM
B14	2.5	120	5	SM
B14	5	113	5	SM
B14	7.5	111	4	SM
B14	10	106	4	SM
B14	15	111	8	SM
B14	20	108	6	SM
B14	25	109	10	SM
B14	30	129	12	SM

File No.: 302169-002 January 17, 2019
Lab No.: 18-178

PLASTICITY INDEX

ASTM D 4318

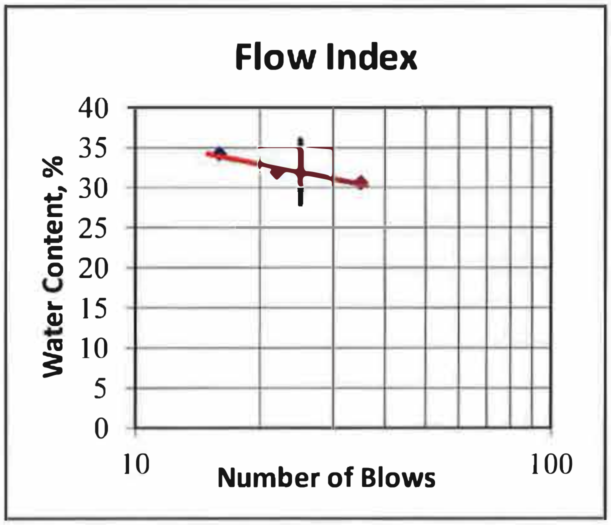
Job Name: Bamiyan Marketplace
Sample ID: B7 @ 10 feet
Soil Description: Silty Sandy Clay (CL)

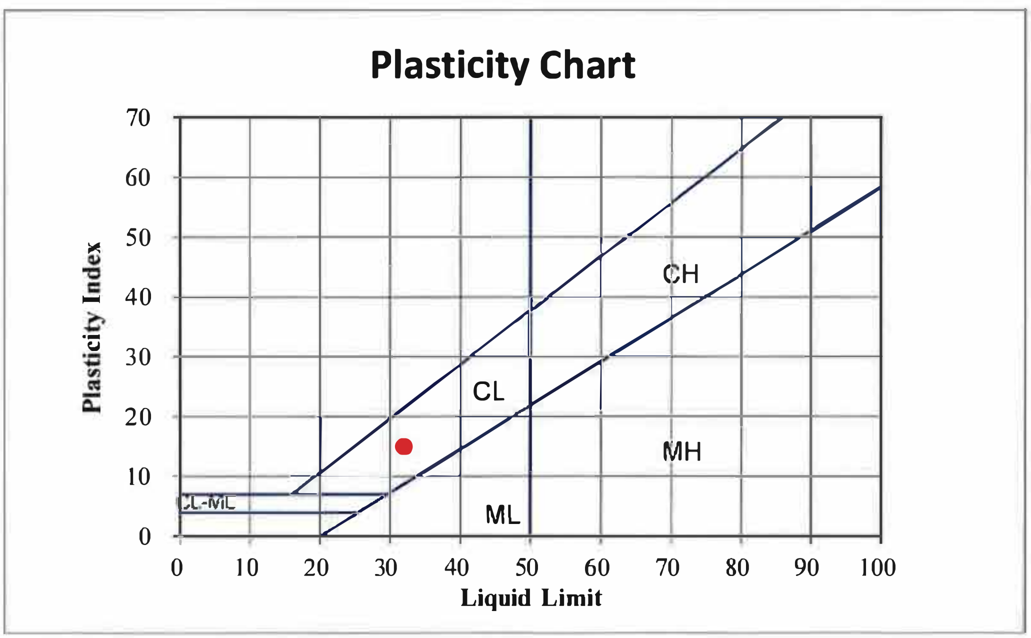
DATA SUMMARY

Number of Blows:	16	22	35
Water Content, %	34.2	31.9	30.6

TEST RESULTS

LIQUID LIMIT	**32**
PLASTIC LIMIT	**17**
PLASTICITY INDEX	**15**





File No.: 302169-002 1/17/2019

Lab No.: 18-178

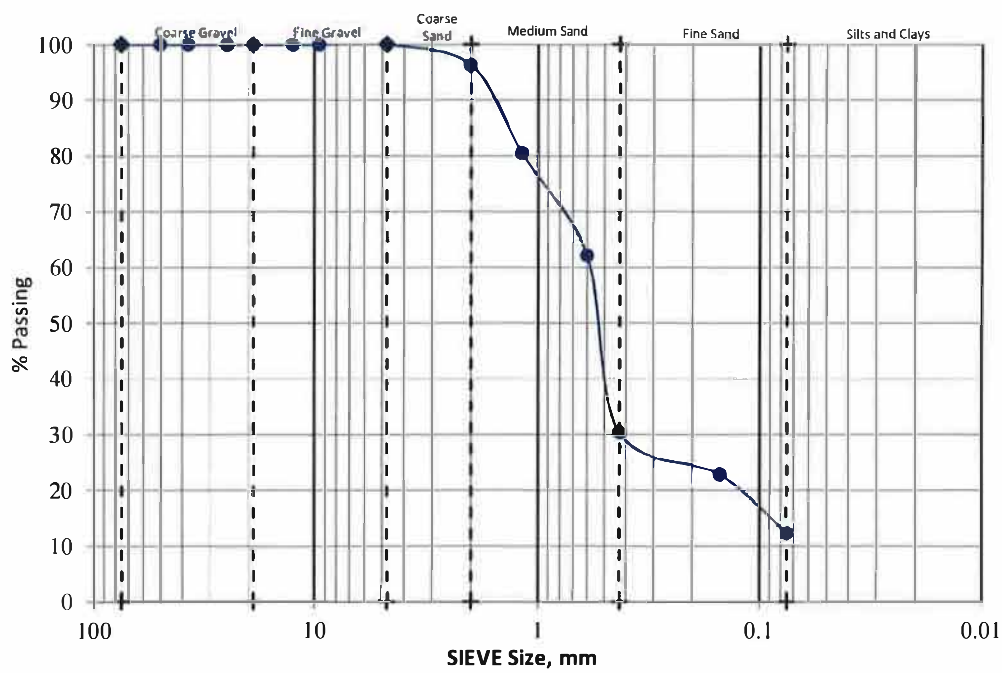
SIEVE ANALYSIS ASTM D6913

Job Name: Bamiyan Marketplace

Sample ID: B1 @ 7 1/2 feet

Description: Silty Sand (SM)

Sieve Size	% Passing	
3"	100	-
2"	100	-
1-1/2"	100	-
1"	100	-
3/4"	100	-
1/2"	100	-
3/8"	100	-
#4	100	-
#10	96	-
#16	81	-
#30	62	-
#40	30	-
#100	23	-
#200	12.3	-



% Coarse Gravel:	0	% Coarse Sand:	4				
% Fine Gravel:	0	% Medium Sand:	66	Cu:	NA		
		% Fine Sand:	18	Cc:	NA	Gradation	
% Total Gravel	0	% Total Sand	88	% Fines:	12.3	NA	

File No.: 302169-002 1/17/2019

Lab No.: 18-178

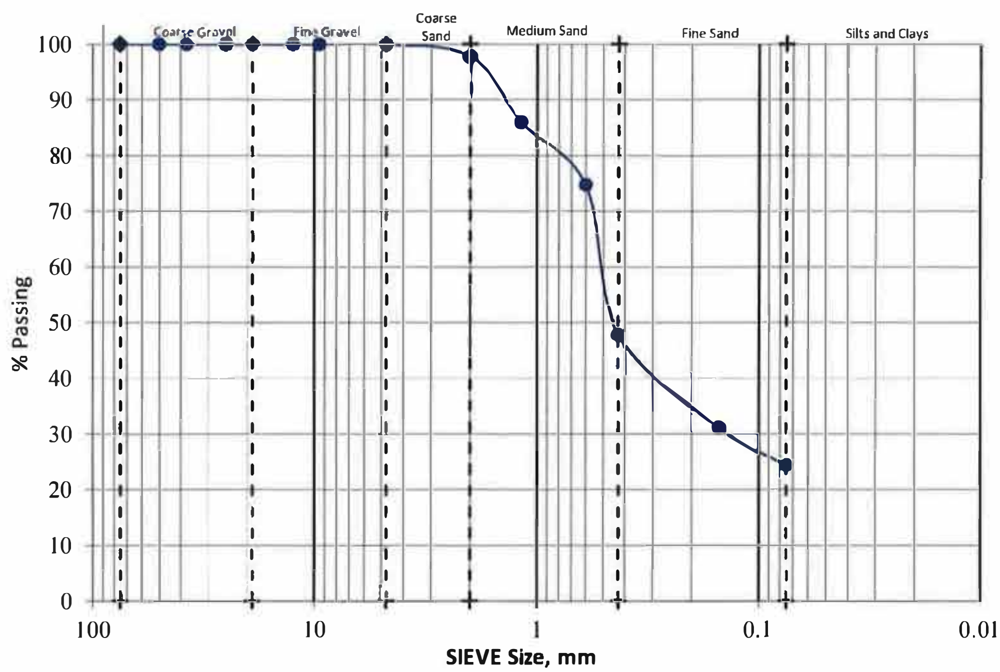
SIEVE ANALYSIS ASTM D6913

Job Name: Bamiyan Marketplace

Sample ID: B2 @5 feet

Description: Silty Sand (SM)

Sieve Size	% Passing	
3"	100	-
2"	100	-
1-1/2"	100	-
1"	100	-
3/4"	100	-
1/2"	100	-
3/8"	100	-
#4	100	-
#10	98	-
#16	86	-
#30	75	-
#40	48	-
#100	31	-
#200	24.4	-



% Coarse Gravel:	0	% Coarse Sand:	2				
% Fine Gravel:	0	% Medium Sand:	50	Cu:	NA		
		% Fine Sand:	23	Cc:	NA	Gradation	
% Total Gravel	0	% Total Sand	76	% Fines:	24.4	NA	

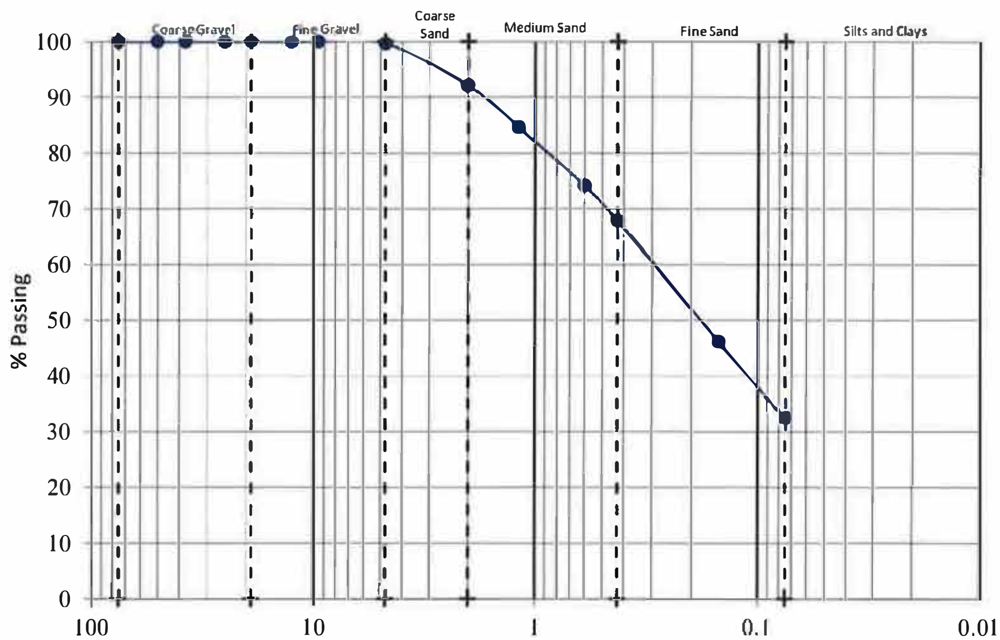
File No.: 302169-002 1/17/2019

Lab No.: 18-178

SIEVE ANALYSIS ASTM D6913

Job Name: Bamiyan Marketplace
Sample ID: B2 @ 10 feet
Description: Silty Sand (SM)

Sieve Size	% Passing	
3"	100	-
2"	100	-
1-1/2"	100	-
1"	100	-
3/4"	100	-
1/2"	100	-
3/8"	100	-
#4	100	-
#10	92	-
#16	85	-
#30	74	-
#40	68	-
#100	46	-
#200	32.5	-



% Coarse Gravel:	0	% Coarse Sand:	8				
% Fine Gravel:	0	% Medium Sand:	24	Cu:	NA		
		% Fine Sand:	35	Cc:	NA	Gradation	
% Total Gravel	0	% Total Sand	67	% Fines:	32.5	NA	

File No.: 302169-002 1/17/2019

Lab No.: 18-178

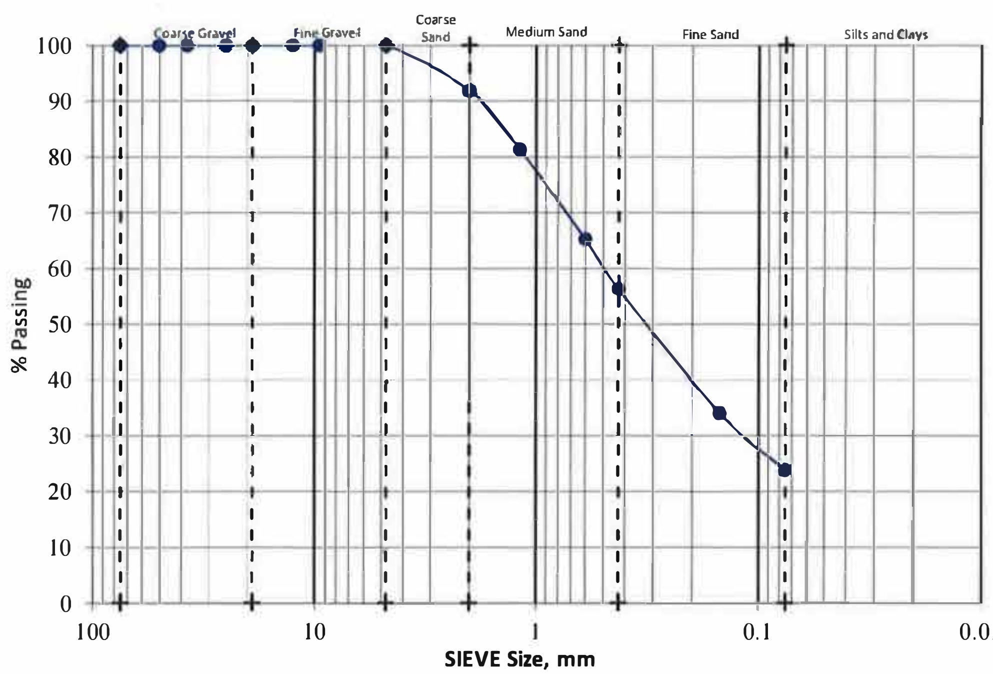
SIEVE ANALYSIS ASTM D6913

Job Name: Bamiyan Marketplace

Sample ID: B13 @ 5 feet

Description: Silty Sand (SM)

Sieve Size	% Passing
3"	100
2"	100
1-1/2"	100
1"	100
3/4"	100
1/2"	100
3/8"	100
#4	100
#10	92
#16	81
#30	65
#40	56
#100	34
#200	23.9



% Coarse Gravel:	0	% Coarse Sand:	8			
% Fine Gravel:	0	% Medium Sand:	36	Cu:	NA	
		% Fine Sand:	32	Cc:	NA	Gradation
% Total Gravel	0	% Total Sand	76	% Fines:	23.9	NA

File No.: 302169-002 1/17/2019

Lab No.: 18-178

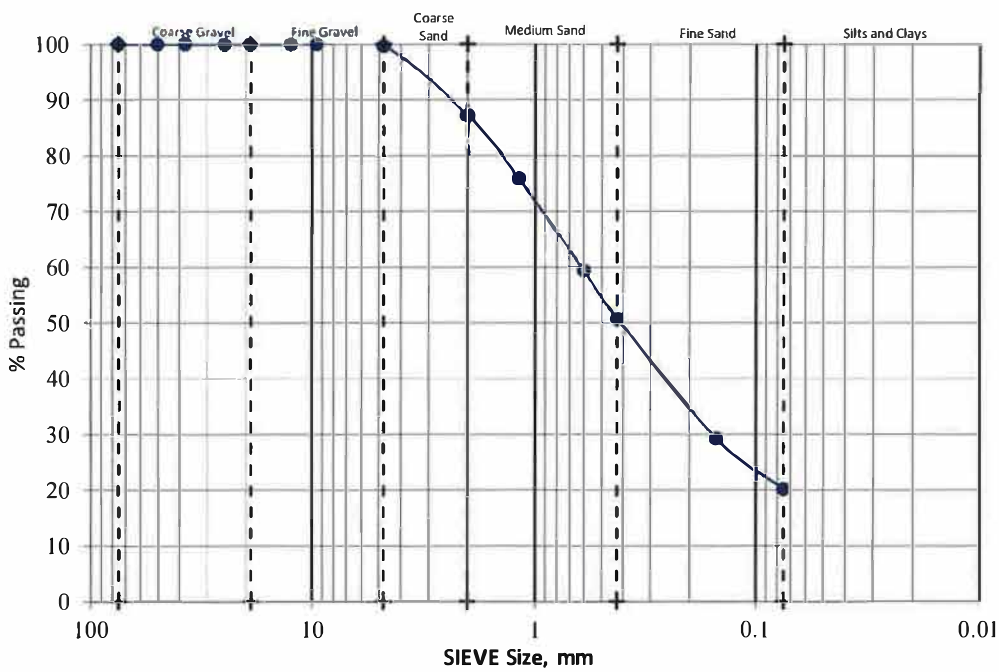
SIEVE ANALYSIS ASTM D6913

Job Name: Bamiyan Marketplace

Sample ID: B13 @ 15 feet

Description: Silty Sand (SM)

Sieve Size	% Passing	
3"	100	-
2"	100	-
1-1/2"	100	-
1"	100	-
3/4"	100	-
1/2"	100	-
3/8"	100	-
#4	100	-
#10	87	-
#16	76	-
#30	59	-
#40	51	-
#100	29	-
#200	20.2	-



% Coarse Gravel:	0	% Coarse Sand:	12				
% Fine Gravel:	0	% Medium Sand:	36	Cu:	NA		
		% Fine Sand:	31	Cc:	NA	Gradation	
% Total Gravel	0	% Total Sand	79	% Fines:	20.2	NA	

File No.: 302169-002 1/17/2019

Lab No.: 18-178

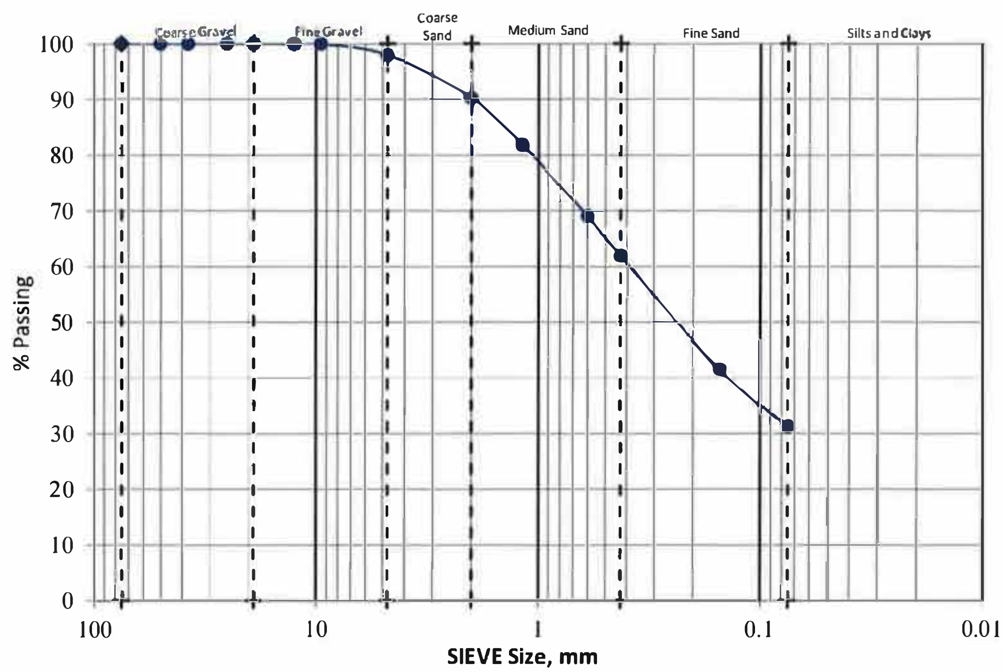
SIEVE ANALYSIS ASTM D6913

Job Name: Bamiyan Marketplace

Sample ID: P2 @ 4.5-5.0 feet

Description: Silty Sand (SM)

Sieve Size	% Passing	
3"	100	-
2"	100	-
1-1/2"	100	-
1"	100	-
3/4"	100	-
1/2"	100	-
3/8"	100	-
#4	98	-
#10	90	-
#16	82	-
#30	69	-
#40	62	-
#100	41	-
#200	31.3	-



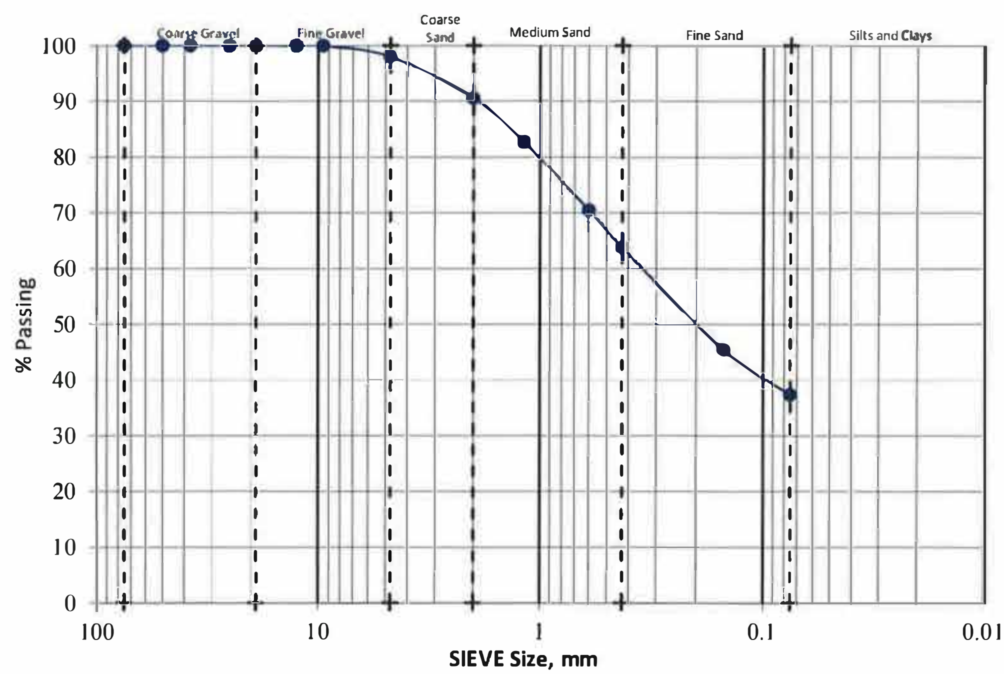
% Coarse Gravel:	0	% Coarse Sand:	8				
% Fine Gravel:	2	% Medium Sand:	28	Cu:	NA		
		% Fine Sand:	31	Cc:	NA	Gradation	
% Total Gravel	2	% Total Sand	67	% Fines:	31.3	NA	

File No.: 302169-002 1/17/2019
Lab No.: 18-178

SIEVE ANALYSIS ASTM D6913

Job Name: Bamiyan Marketplace
Sample ID: P4 @ 4.5-5.0 feet
Description: Silty Sand (SM)

Sieve Size	% Passing	
3"	100	-
2"	100	-
1-1/2"	100	-
1"	100	-
3/4"	100	-
1/2"	100	-
3/8"	100	-
#4	98	-
#10	91	-
#16	83	-
#30	71	-
#40	64	-
#100	45	-
#200	37.4	-



% Coarse Gravel:	0	% Coarse Sand:	7				
% Fine Gravel:	2	% Medium Sand:	27	Cu:	NA		
		% Fine Sand:	26	Cc:	NA	Gradation	
% Total Gravel	2	% Total Sand	61	% Fines:	37.4	NA	

File No.: 302169-002
Job Name: Bamiyan Marketplace
Lab Number: 18-178

January 17, 2019

AMOUNT PASSING NO. 200 SIEVE

ASTM D 1140

Sample Location	Depth (feet)	Fines Content (%)	USCS Group Symbol
B5	5	39.7	SM
B9	10	42.9	SC
B12	20	39.8	SM

File No.: 302169-002
Lab No.: 18-178

January 17, 2019

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

Bamiyan Marketplace
B2 @ 5 feet

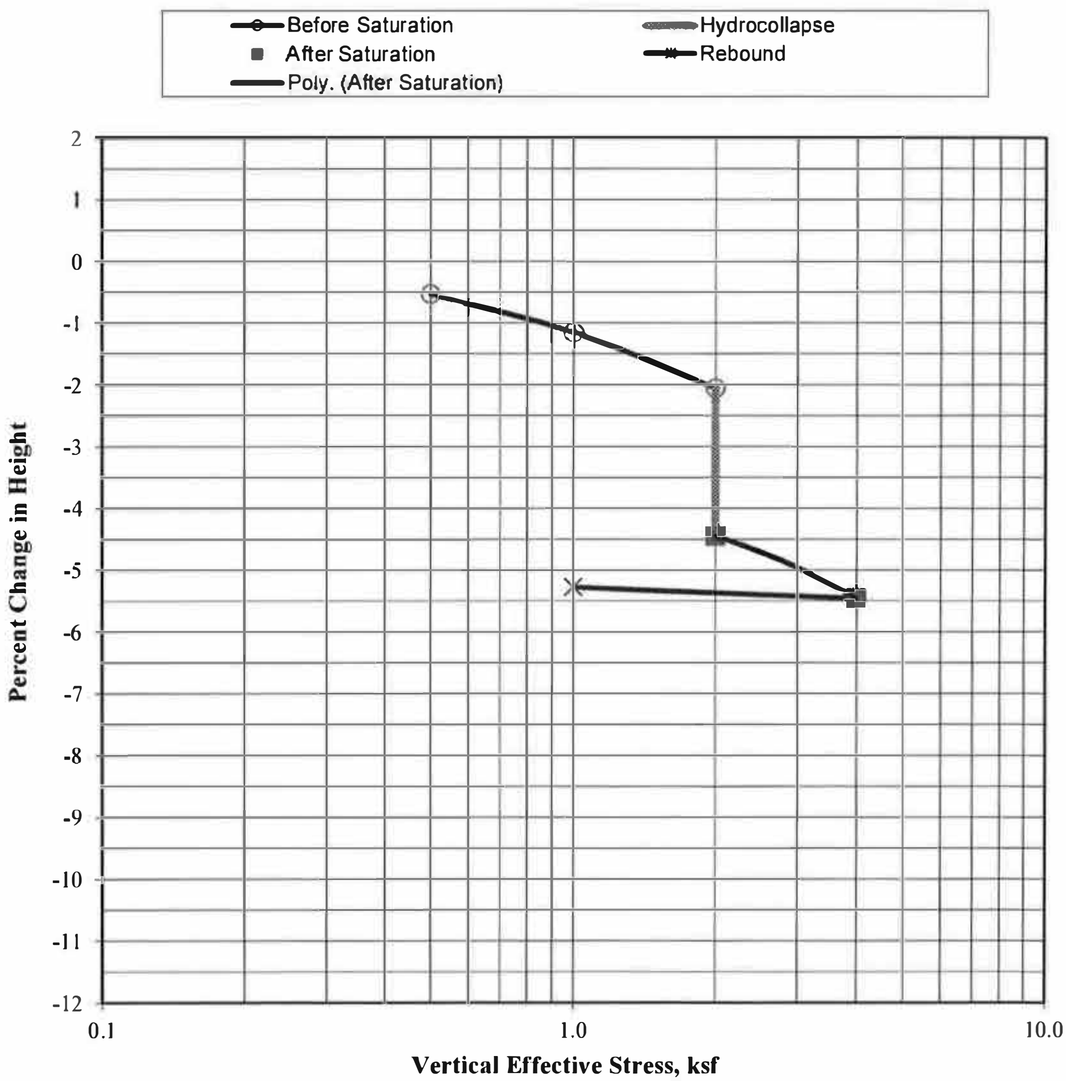
Silty Sand (SM)

Ring Sample

Initial Dry Density: 106.7 pcf
Initial Moisture: 5.3%
Specific Gravity: 2.67
Initial Void Ratio: 0.563

Hydrocollapse: 2.4% @ 2.0 ksf

% Change in Height vs Normal Pressure Diagram



File No.: 302169-002
Lab No.: 18-178

January 17, 2019

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

Bamiyan Marketplace
B2 @ 10 feet

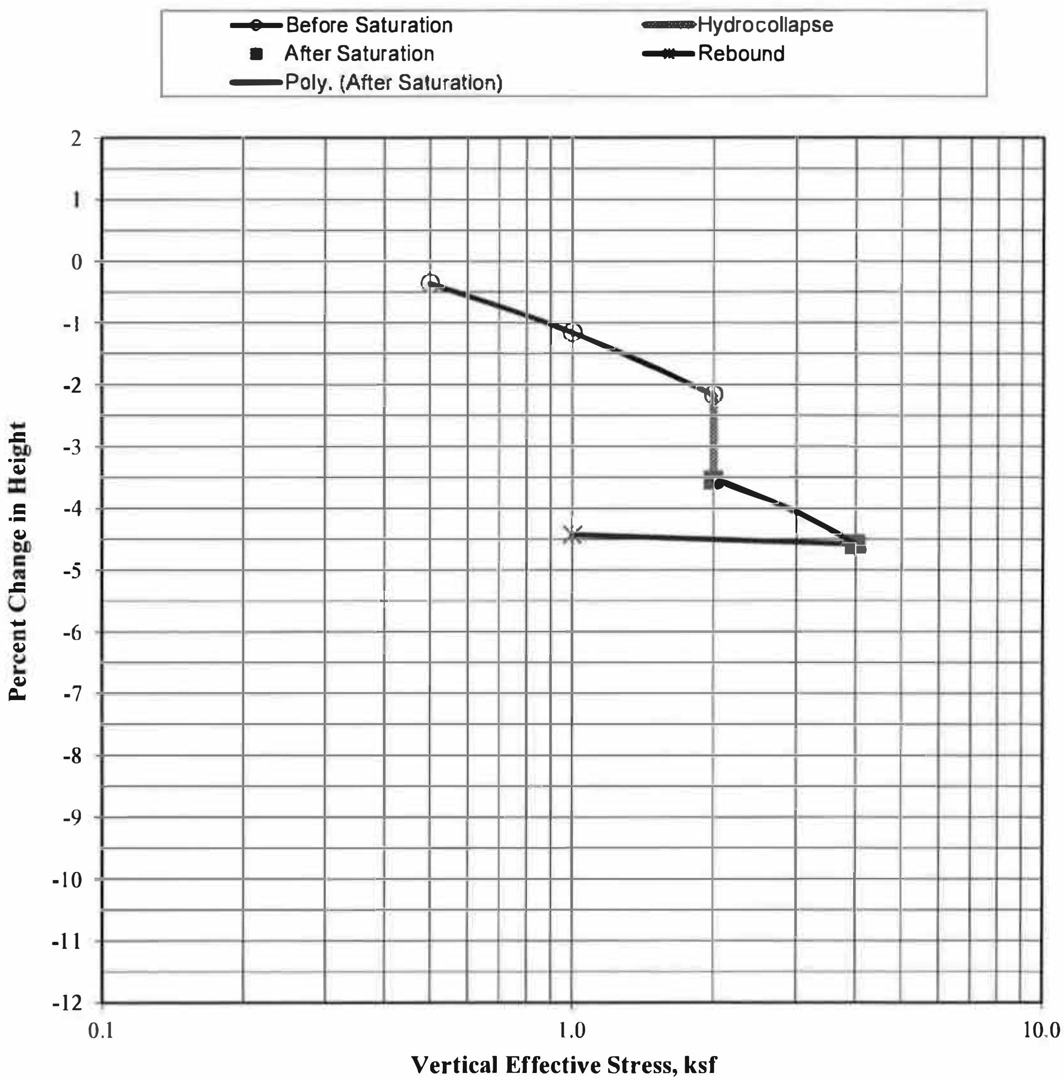
Silty Sand (SM)

Ring Sample

Initial Dry Density: 101.6 pcf
Initial Moisture: 5.1%
Specific Gravity: 2.67
Initial Void Ratio: 0.640

Hydrocollapse: 1.4% @ 2.0 ksf

% Change in Height vs Normal Pressure Diagram



File No.: 302169-002
Lab No.: 18-178

January 17, 2019

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

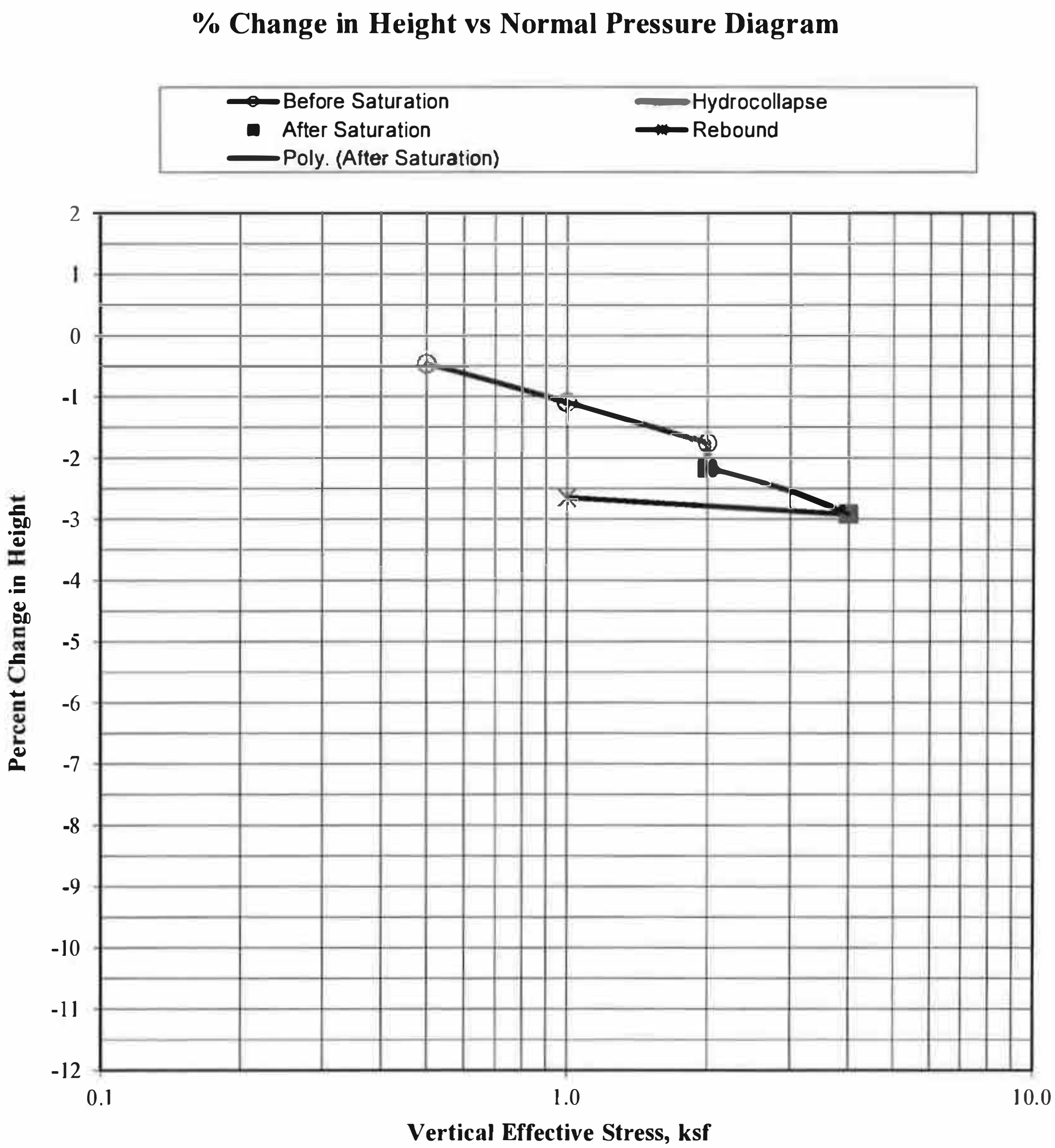
Bamiyan Marketplace
B2 @ 15 feet

Silty Sand w/Trace Clay (SM)

Ring Sample

Initial Dry Density: 124.9 pcf
Initial Moisture: 3.1%
Specific Gravity: 2.67
Initial Void Ratio: 0.151

Hydrocollapse: 0.4% @ 2.0 ksf



File No.: 302169-002
Lab No.: 18-178

January 17, 2019

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

Bamiyan Marketplace
B7 @ 10 feet

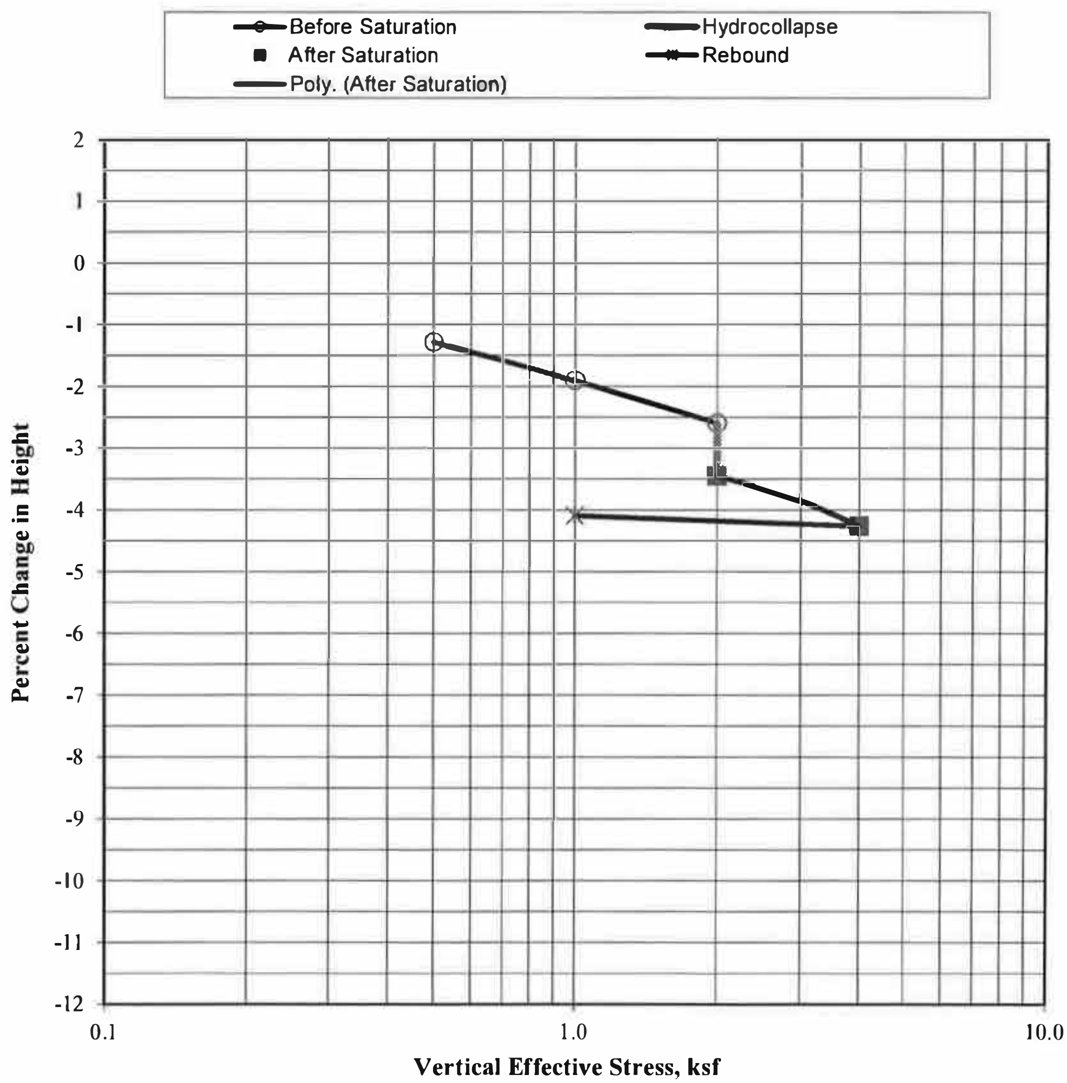
Silty Sandy Clay (CL)

Ring Sample

Initial Dry Density: 119.0 pcf
Initial Moisture: 9.0%
Specific Gravity: 2.67
Initial Void Ratio: 0.401

Hydrocollapse: 0.9% @ 2.0 ksf

% Change in Height vs Normal Pressure Diagram



File No.: 302169-002
Lab No.: 18-178

January 17, 2019

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

Bamiyan Marketplace
B13 @ 5 feet

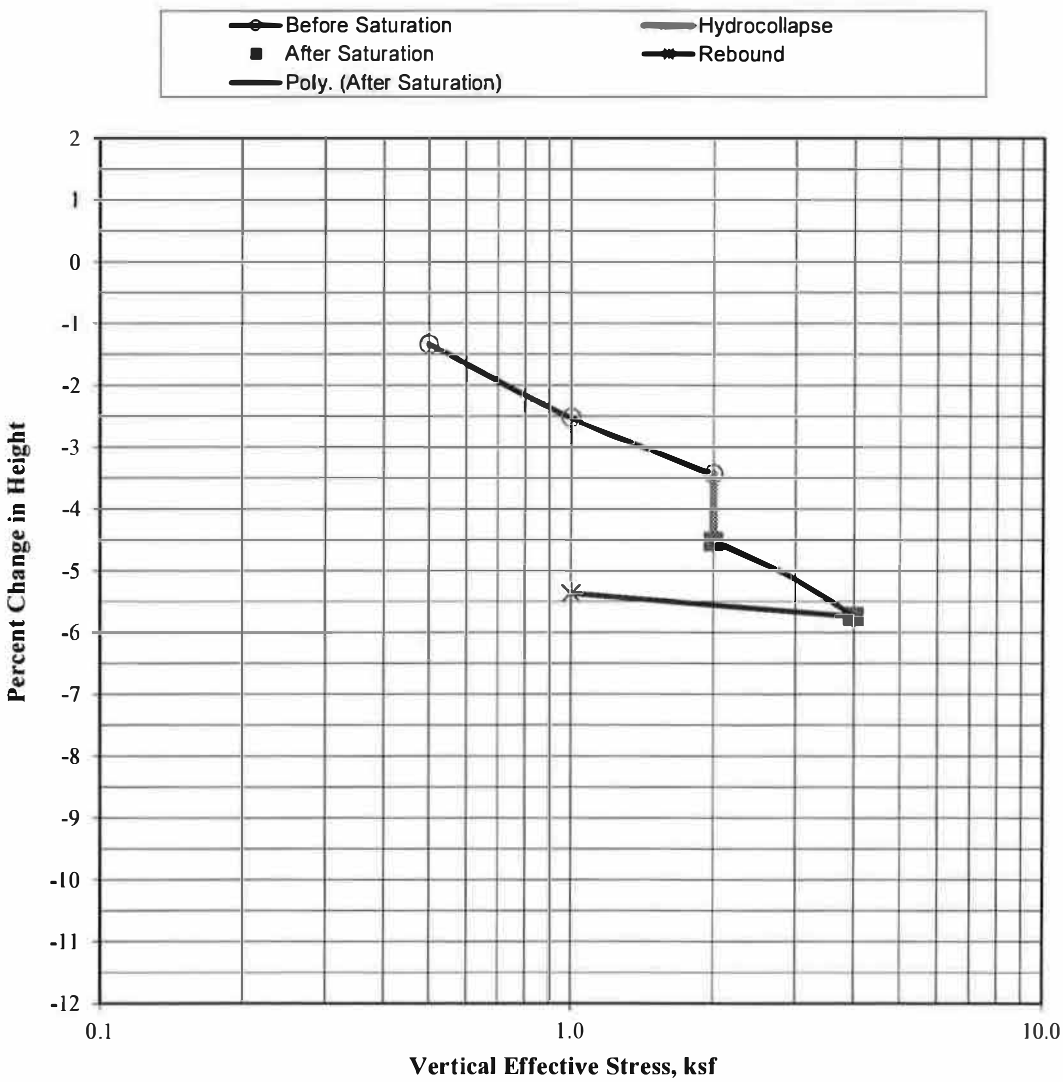
Silty Sand (SM)

Ring Sample

Initial Dry Density: 119.2 pcf
Initial Moisture: 12.6%
Specific Gravity: 2.67
Initial Void Ratio: 0.399

Hydrocollapse: 1.1% @ 2.0 ksf

% Change in Height vs Normal Pressure Diagram



File No.: 302169-002
Lab No.: 18-178

January 17, 2019

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

Bamiyan Marketplace
B13 @ 10 feet

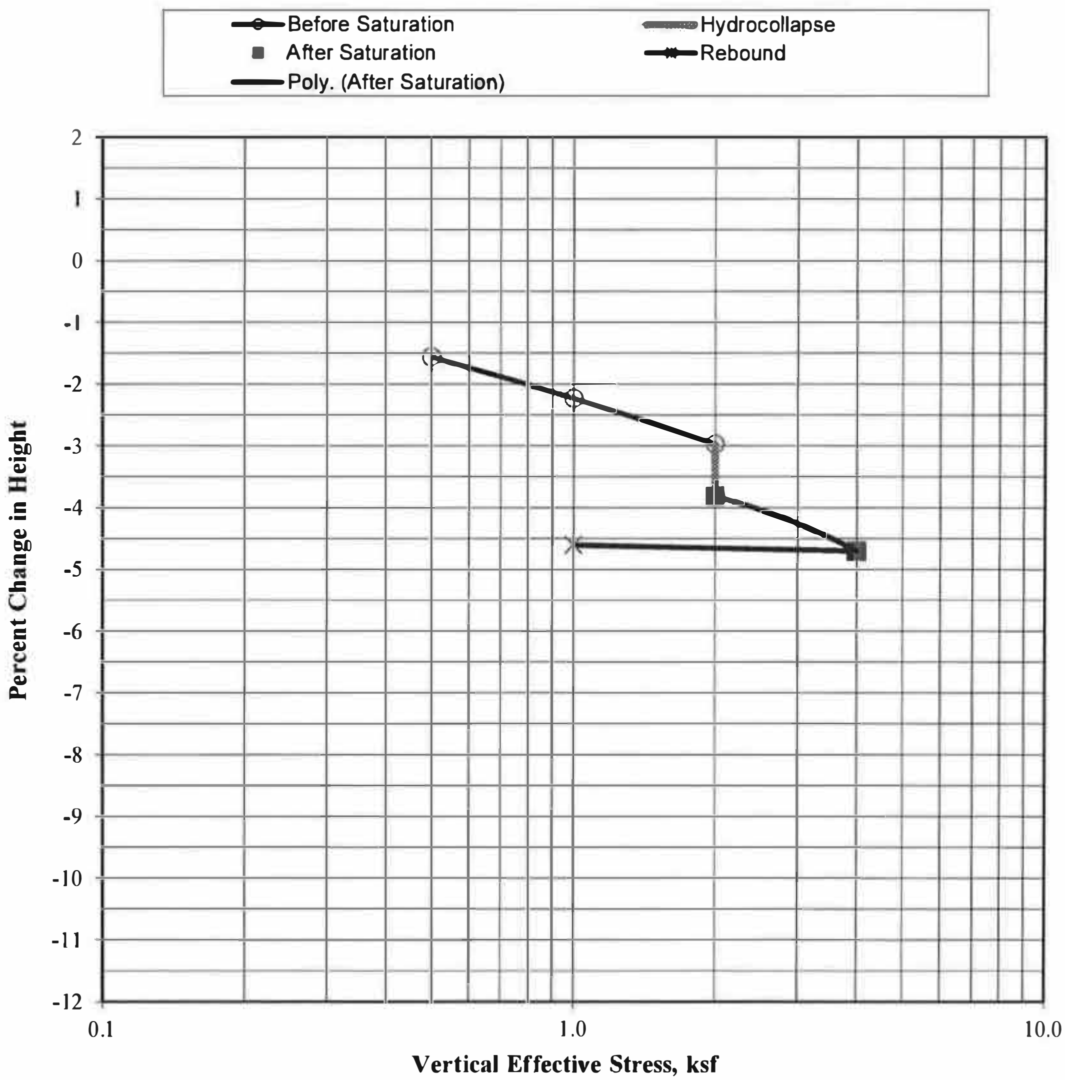
Silty Sand w/Gravel (SM)

Ring Sample

Initial Dry Density: 119.1 pcf
Initial Moisture: 9.7%
Specific Gravity: 2.67
Initial Void Ratio: 0.400

Hydrocollapse: 0.9% @ 2.0 ksf

% Change in Height vs Normal Pressure Diagram



File No.: 302169-002
Lab No.: 18-178

January 17, 2019

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

Bamiyan Marketplace
B13 @ 15 feet

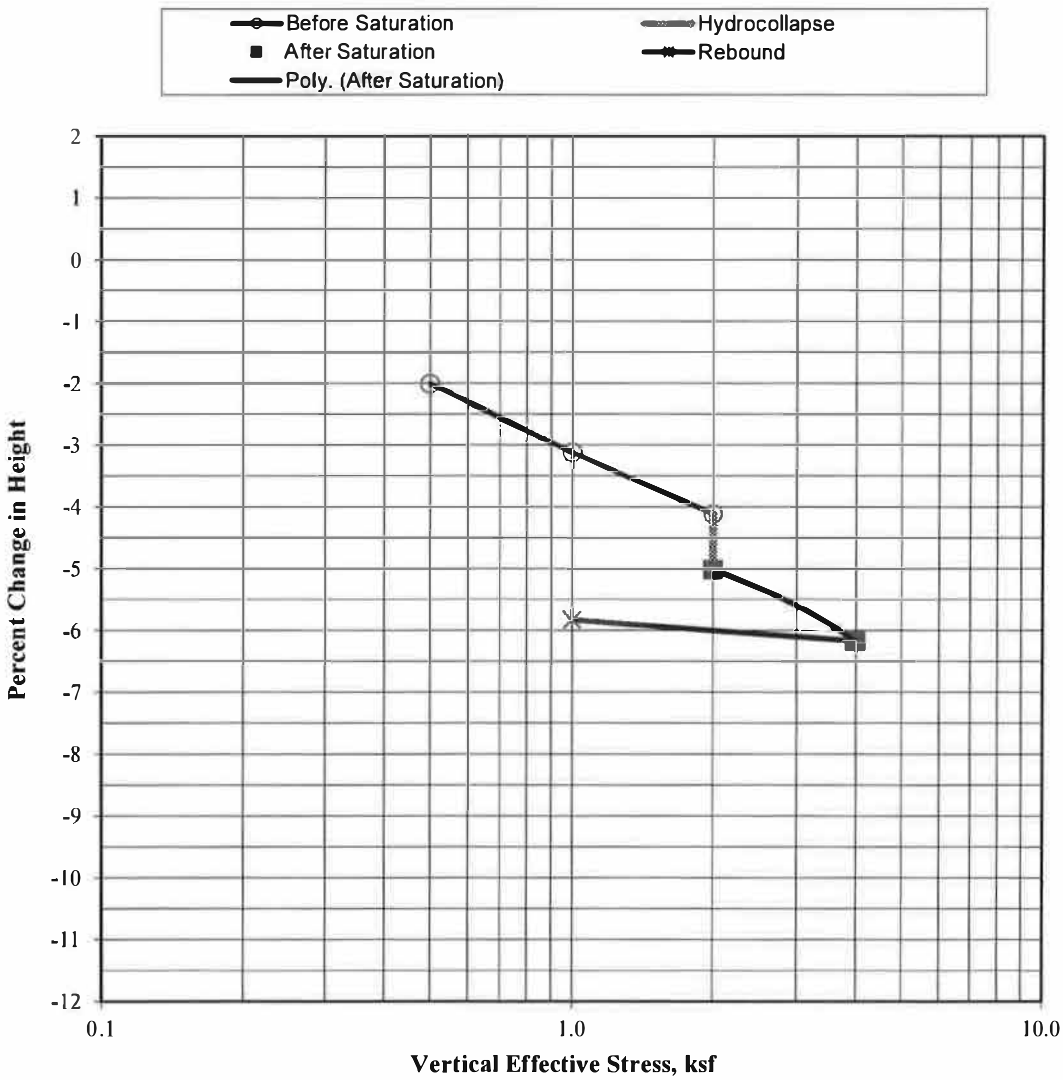
Silty Sand (SM)

Ring Sample

Initial Dry Density: 121.2 pcf
Initial Moisture: 9.7%
Specific Gravity: 2.67
Initial Void Ratio: 0.376

Hydrocollapse: 0.9% @ 2.0 ksf

% Change in Height vs Normal Pressure Diagram



File No.: 302169-002
Lab No.: 18-178

January 17, 2019

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

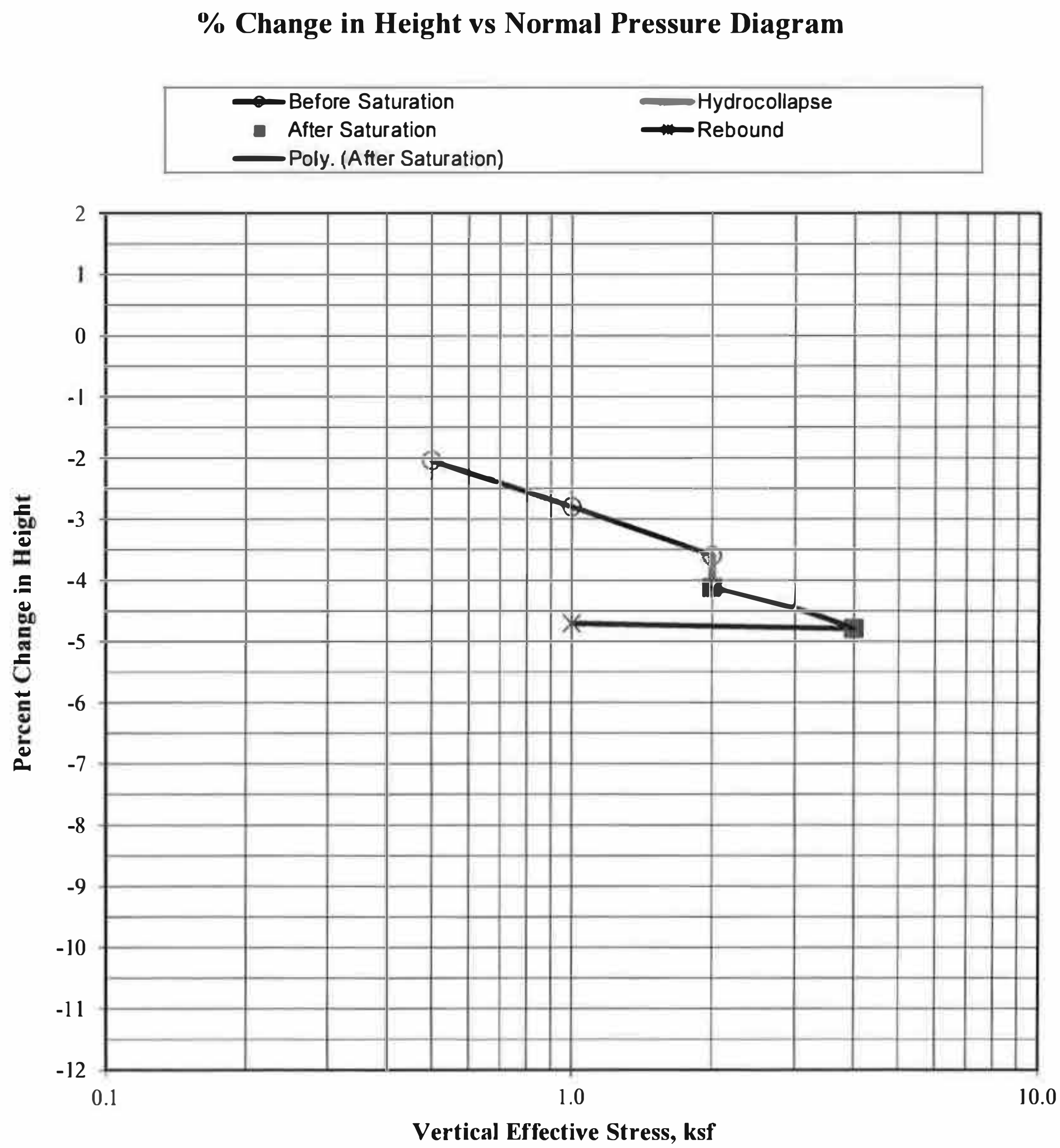
Bamiyan Marketplace
B13 @ 20 feet

Silty Sand (SM)

Ring Sample

Initial Dry Density: 124.8 pcf
Initial Moisture: 9.8%
Specific Gravity: 2.67
Initial Void Ratio: 0.336

Hydrocollapse: 0.5% @ 2.0 ksf



File No.: 302169-002
Lab No.: 18-178
EXPANSION INDEX

January 17, 2019
ASTM D-4829

Job Name: Bamiyan Marketplace
Sample ID: B2 @ 2.5 feet
Soil Description: Silty Sand w/Trace Clay (SM)

Initial Moisture, %:	8.4	
Initial Compacted Dry Density, pcf:	116.3	
Initial Saturation, %:	51	
Final Moisture, %:	17.1	
Volumetric Swell, %:	0.3	
Expansion Index, EI:	**3**	**Very Low**

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

File No.: 302169-002 January 17, 2019

Lab No.: 18-178

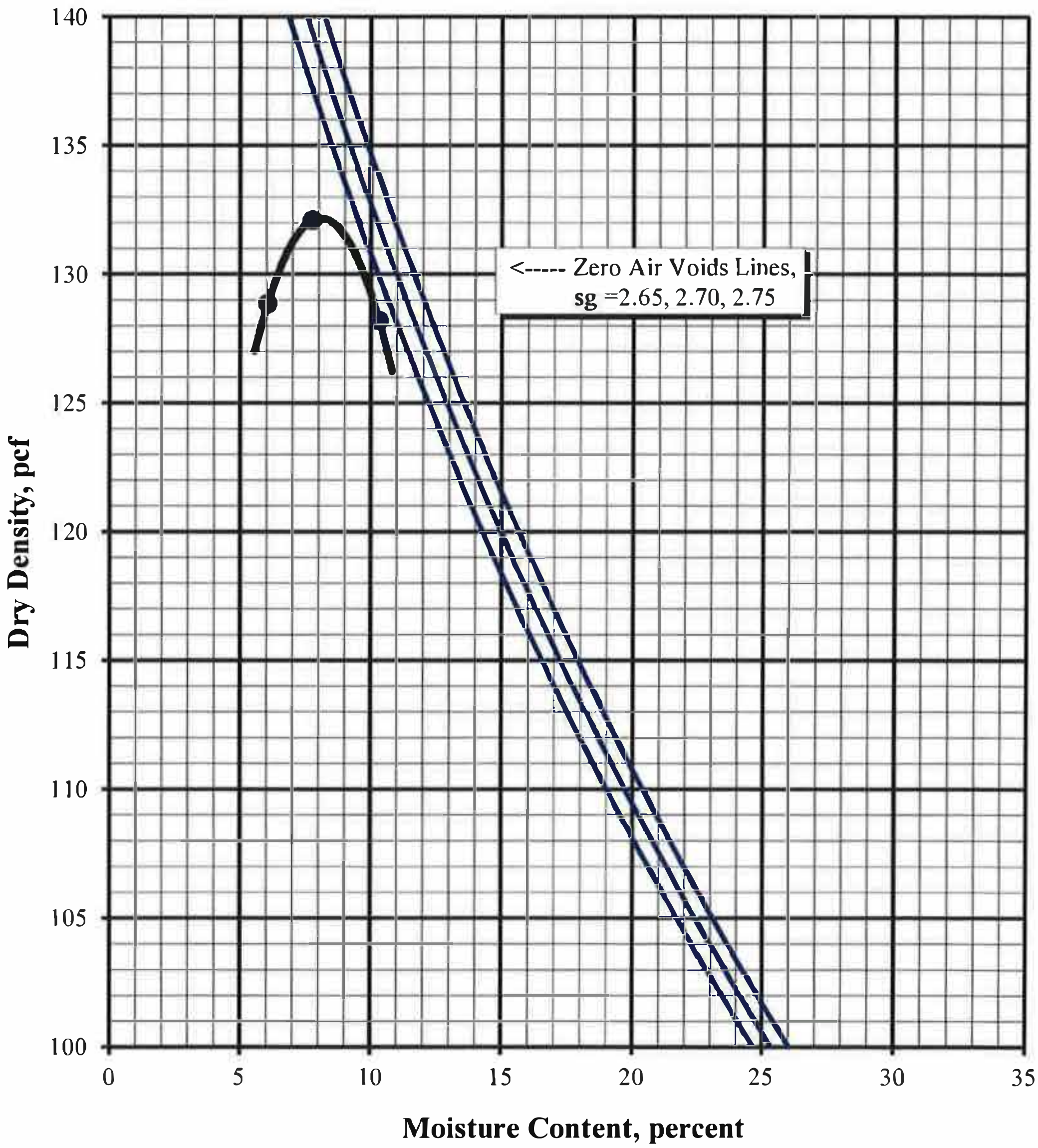
MAXIMUM DRY DENSITY / OPTIMUM MOISTURE ASTM D 1557 (Modified)

Job Name: Bamiyan Marketplace
Sample ID: 1
Location: B13 @ 0-5 feet
Description: Brown Silty F - M Sand (SM)

Procedure Used: A
Preparation Method: Moist
Rammer Type: Mechanical
Lab Number: 18-178

Maximum Dry Density: **132.3 pcf**
Optimum Moisture: **8.3%**

Sieve Size	% Retained (Cumulative)
3/4"	0.0
3/8"	0.3
#4	1.0



File No.: 302169-002 1/17/2019
Lab No.: 18-178

SOIL CHEMICAL ANALYSES

Job Name:	Bamiyan Marketplace	
Job No.:	302169-002	
Sample ID:	B3	B13
Sample Location:	2.5 feet	0-5 feet
Resistivity (Units)		
as-received (ohm-cm)	36,400	52,000
saturated (ohm-cm)	17,200	13,200
pH	7.3	7.2
Electrical Conductivity (mS/cm)	0.02	0.04
Chemical Analyses		
Cations		
calcium Ca^{2+} (mg/kg)	6	12
magnesium Mg^{2+} (mg/kg)	1.4	1.9
sodium Na^{1+} (mg/kg)	27	31
potassium K^{1+} (mg/kg)	1.7	22
Anions		
carbonate CO_3^{2-} (mg/kg)	ND	ND
bicarbonate HCO_3^{1-}(mg/kg)	43	76
fluoride F^{1-} (mg/kg)	0.7	ND
chloride Cl^{1-} (mg/kg)	2.9	6.4
sulfate SO_4^{2-} (mg/kg)	2.7	16
phosphate PO_4^{3-} (mg/kg)	2	17
Other Tests		
ammonium NH_4^{1+} (mg/kg)	ND	ND
nitrate NO_3^{1-} (mg/kg)	3.3	4.4
sulfide S^{2-} (qual)	na	na
Redox (mV)	na	na

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

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

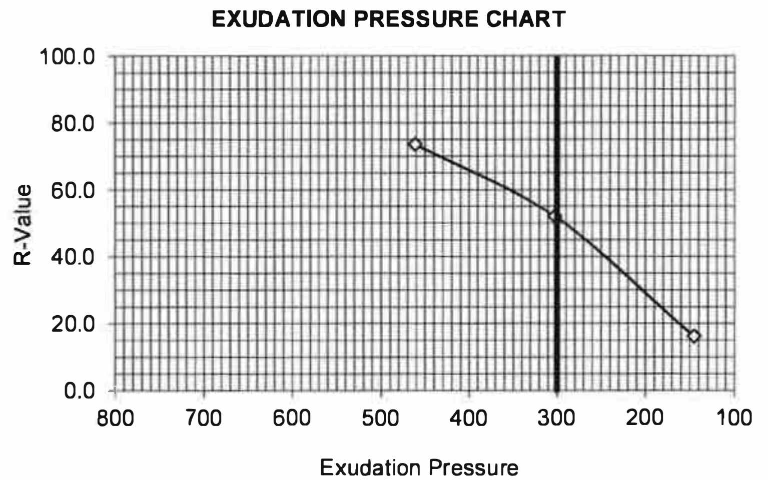
Resistivity per ASTM G187, Cations per ASTM D6919, Anions per ASTM D4327, and Alkalinity per APHA 2320-B. Electrical conductivity in millisiemens/cm and chemical analyses were made on a 1:5 soil-to-water extract.

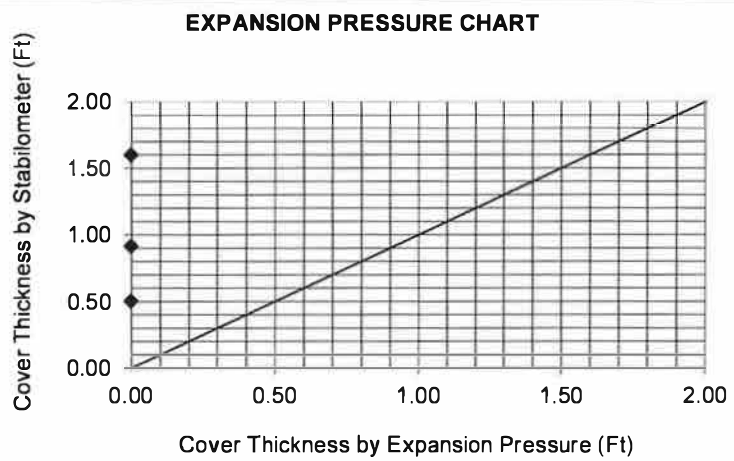
General Guidelines for Soil Corrosivity		
Chemical Agent	Amount in Soil	Degree of Corrosivity
Soluble	0 -1,000 mg/Kg (ppm) [0-.1%]	Low
Sulfates[1]	1,000 - 2,000 mg/Kg (ppm) [0.1-0.2%]	Moderate
	2,000 - 20,000 mg/Kg (ppm) [0.2-2.0%]	Severe
	> 20,000 mg/Kg (ppm) [>2.0%]	Very Severe
Resistivity[2]	0- 900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

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

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

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



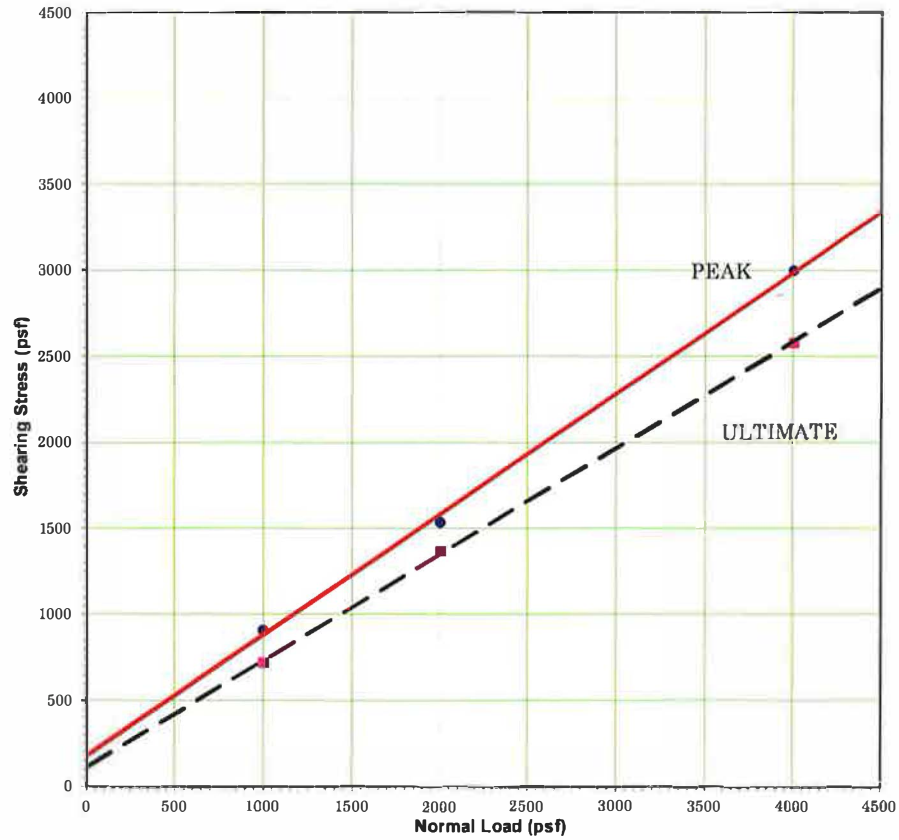


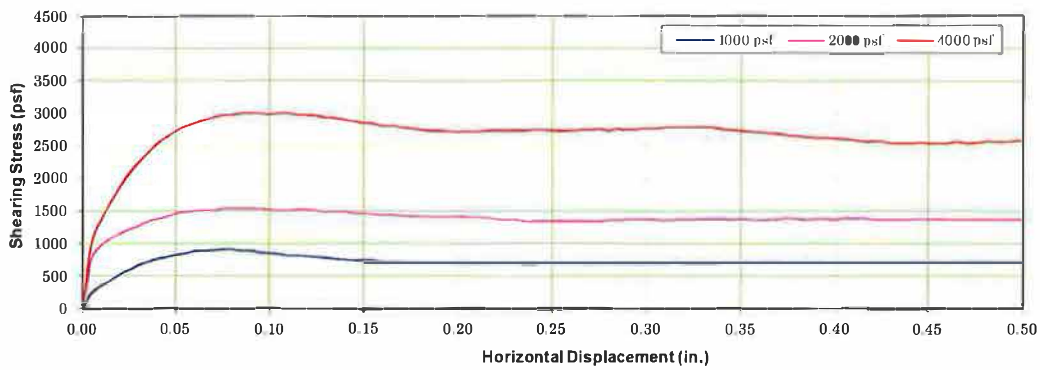
JOB NAME: Bamiyan Marketplace
SAMPLE I. D.: B-13@0-5'
SOIL DESCRIPTION: Silty Sand (MS)

SPECIMEN NUMBER	A	B	C
EXUDATION PRESSURE	461	302	145
RESISTANCE VALUE	73.6	52.0	16.2
EXPANSION DIAL(0.0001")	0	0	0
EXPANSION PRESSURE (PSF)	0.0	0.0	0.0
% MOISTURE AT TEST	8.7	9.8	11.0
DRY DENSITY AT TEST	126.6	126.9	125.2

R-VALUE @ 300 PSI EXUDATION	52
R-VALUE by Expansion Pressure*	N/A

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





DIRECT SHEAR DATA*

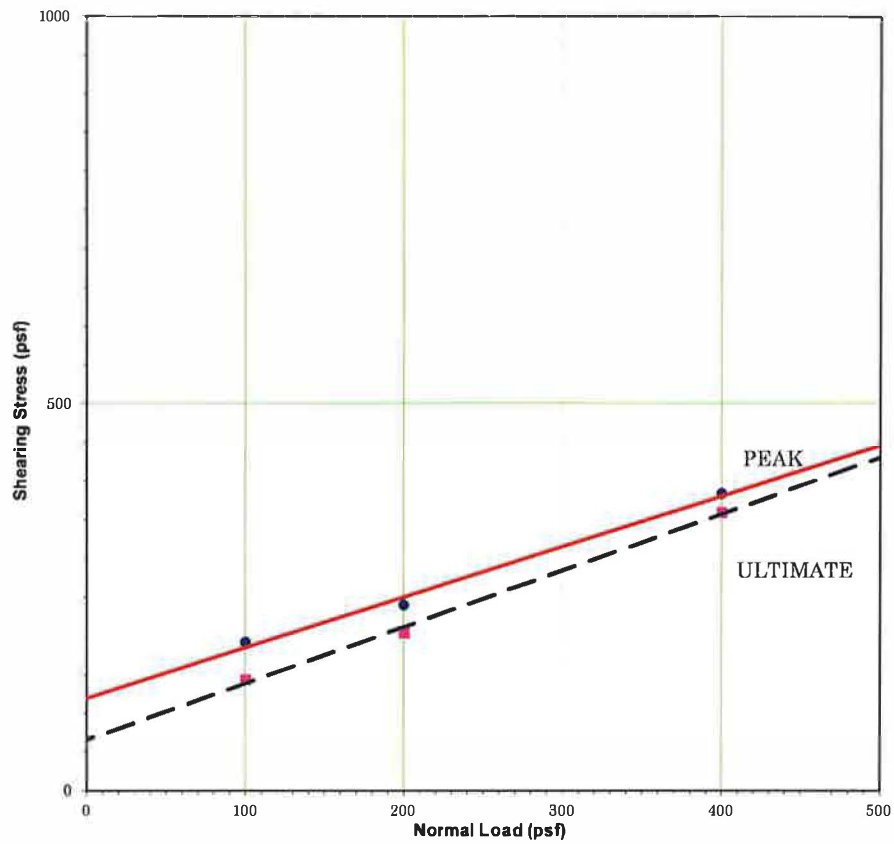
Sample Location:	B13 @ 0-5 feet	
Material:	Silty Sand (SM)	
Dry Density (pcf):	119 (90% remolded	
	Initial	Final
Moisture Content (%):	8.3	14.0
Saturation (%):	######	#VALUE!
	Peak	Ultimate
ϕ Angle of Friction (degrees):	35	32
c Cohesive Strength (psf):	180	110

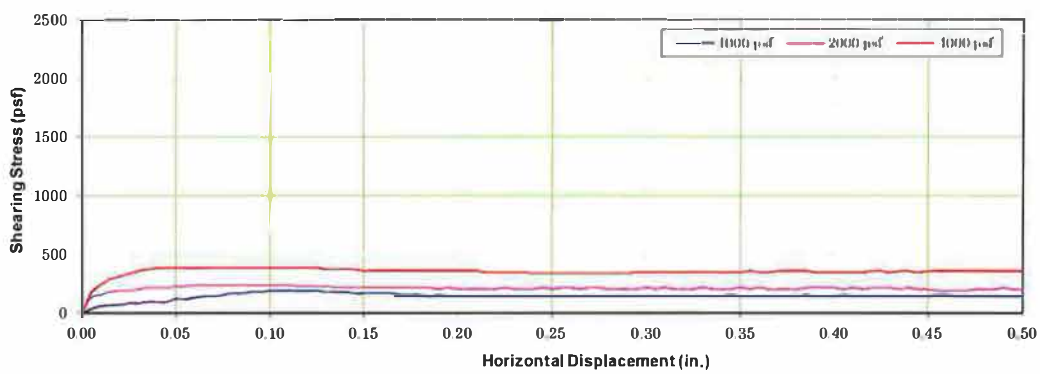
Test Type: Peak and Ulitimate

Shear Rate (in/min): 0.007

* Test Method: ASTM D-3080

DIRECT SHEAR TEST	
Bamiyan Marketplace **Lake Elsinore, California**	
Earth Systems	
1/17/2019	302169-002





DIRECT SHEAR DATA*

Sample Location: TL-1 @ 0-1 feet
Material: Silty Sand (SM)
Dry Density (pcf): 108.0

	Initial	Final
Moisture Content (%):	16	18.4
Saturation (%):	79	100
	Peak	**Ultimate**
ϕ Angle of Friction (degrees):	36	33
c Cohesive Strength (psf):	120	60

Test Type: Peak and Ulitimate
Shear Rate (in/min): 0.007

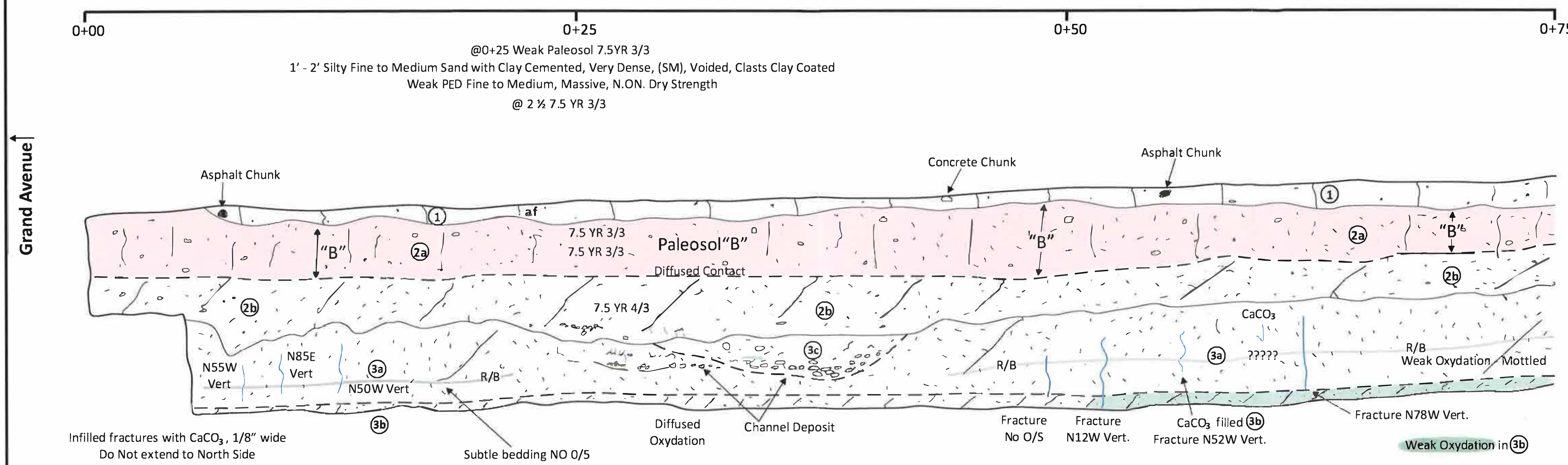
* Test Method: ASTM D-3080

DIRECT SHEAR TEST	
Bamiyan Marketplace **Lake Elsinore, California**	
Earth Systems	
1/17/2019	302169-002

Scale: 1" = 5'

T-1

Trench Orientation
N48E
South Side



(1) Fill, Fine to Coarse Sand with Clay (SC), Dry, Loose to Medium Dense, Some Gravel, Trace Cobble, Very Dark Grayish Brown (10 yr 3/2).

(2a) Alluvium, Fine to Coarse Sand with Clay (SC), Visible Voids, High Dry/Strength, Dry, Medium Dense, Trace Gravel, Trace Cobble, Dark Brown (7.5 yr 3/2), Weakly Cemented, Weak Blocky Structure, Weak B-Horizon, Massive.

(2b) Alluvium, Fine to Coarse Sand with Gravel, with Silt (SM), Visibly Voided, Dry, Medium Dense, Brown (7.5 yr 4/3), Gradational Change from (2a) to (2b), Massive.

(3a) Older Alluvium, Fine to Coarse Sand with Gravel, Trace Silt (SP - SM), Dry, Medium Dense, Mottled Brown (10 yr 5/3) to Reddish Yellow (7.5 yr 6/8), from Sta 2+25 West Becomes Silty Sand, Massive.

(3b) Older Alluvium, Silty Fine to Coarse Sand (SM), Moist, Medium Dense, Mottled Yellowish Brown (10 yr 5/6) to Reddish Yellow (7.5 yr 6/8), Massive.

(3c) Older Alluvium (Channel Deposit), It Appears During the Deposition of (3a) and Prior to (2b), Numerous Channels Cut Into (3a) and Deposited Coarser Grained Materials, These Channel Deposits Vary from Clean Fine to Coarse Sand (SP) to Gravelly Fine to Coarse Sand with Cobble and Small Lenses of Fine Sandy Silt, Dry, Loose to Medium Dense, Typically the Channel Deposits on the Upper Portion Will Grade Into (3a). One Channel Deposit Sta 0+35 to 0+40 is Overlain by (2b).

R/B - Subtle Relic Bedding

LEGEND

Bt - Horizon — Lithologic Contact

Weak B - Horizon — Geologic Contact

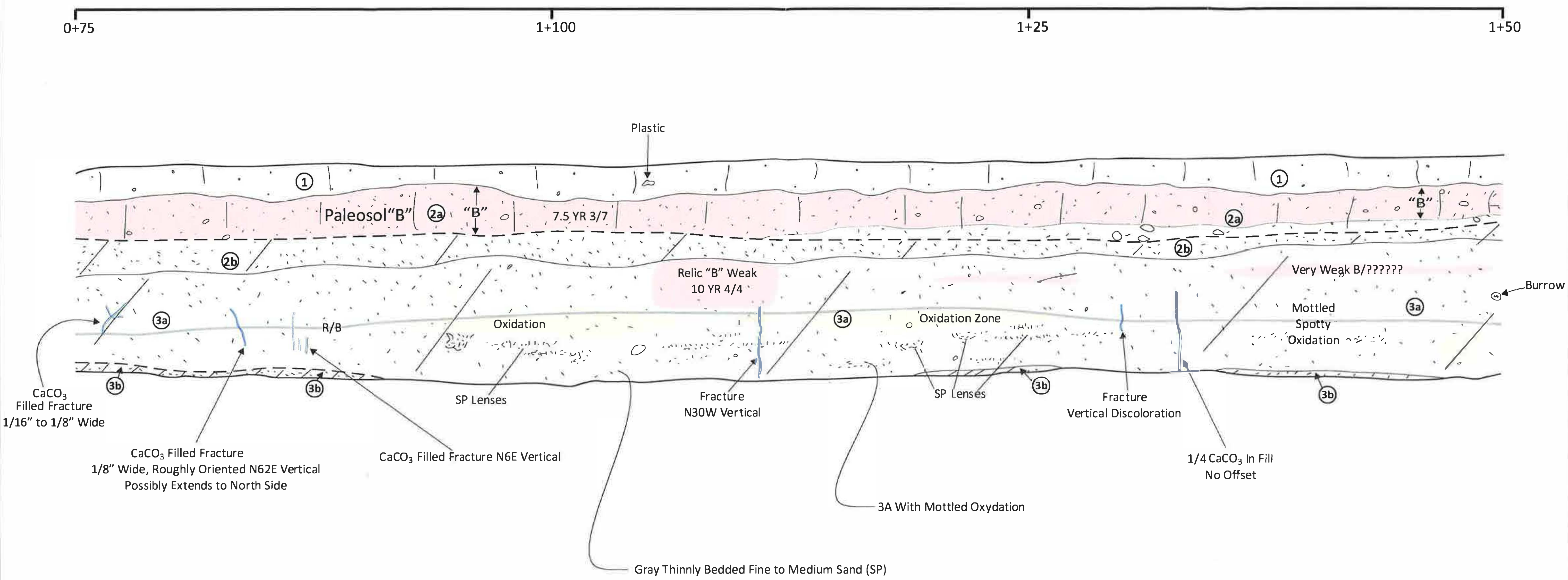
Approximate Scale: 1" = 5'
0 5 Feet 10 Feet

Plate 5
T-1 Trench Log
Proposed Bamiyan Marketplace
15749 Grand Avenue
Lake Elsinore, Riverside County, California
Earth Systems

1/17/2019	File No.: 302169-001

T-1

Trench Orientation
N48E



Relic Palrosol 1+10'
Silty Fine to Medium Sand 10 YR 4/4
Weakly Cemented
Medium Dry Strength
Mottled With Weak Oxydation

LEGEND

Bt - Horizon — Lithologic Contact

Weak B - Horizon — Geologic Contact

Approximate Scale: 1" = 5'

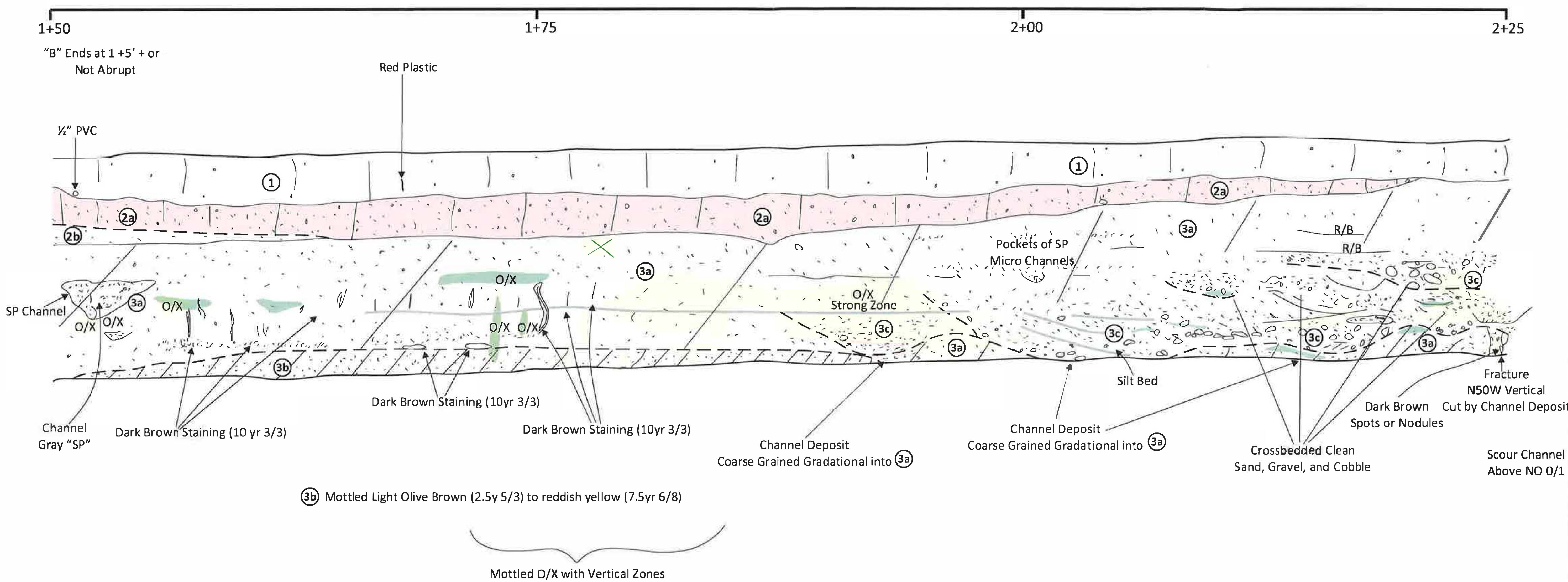
0 5 Feet 10 Feet

Plate 6 T-1 Trench Log	
Proposed Bamiyan Marketplace 15749 Grand Avenue Lake Elsinore, Riverside County, California	
Earth Systems	
1/17/2019	File No.: 302169-001

T-1

Trench Orientation

N48E



LEGEND

Bt - Horizon

Weak B - Horizon

Lithologic Contact

Geologic Contact

Approximate Scale: 1" = 5'

0 5 Feet 10 Feet

Plate 7
T-1 Trench Log

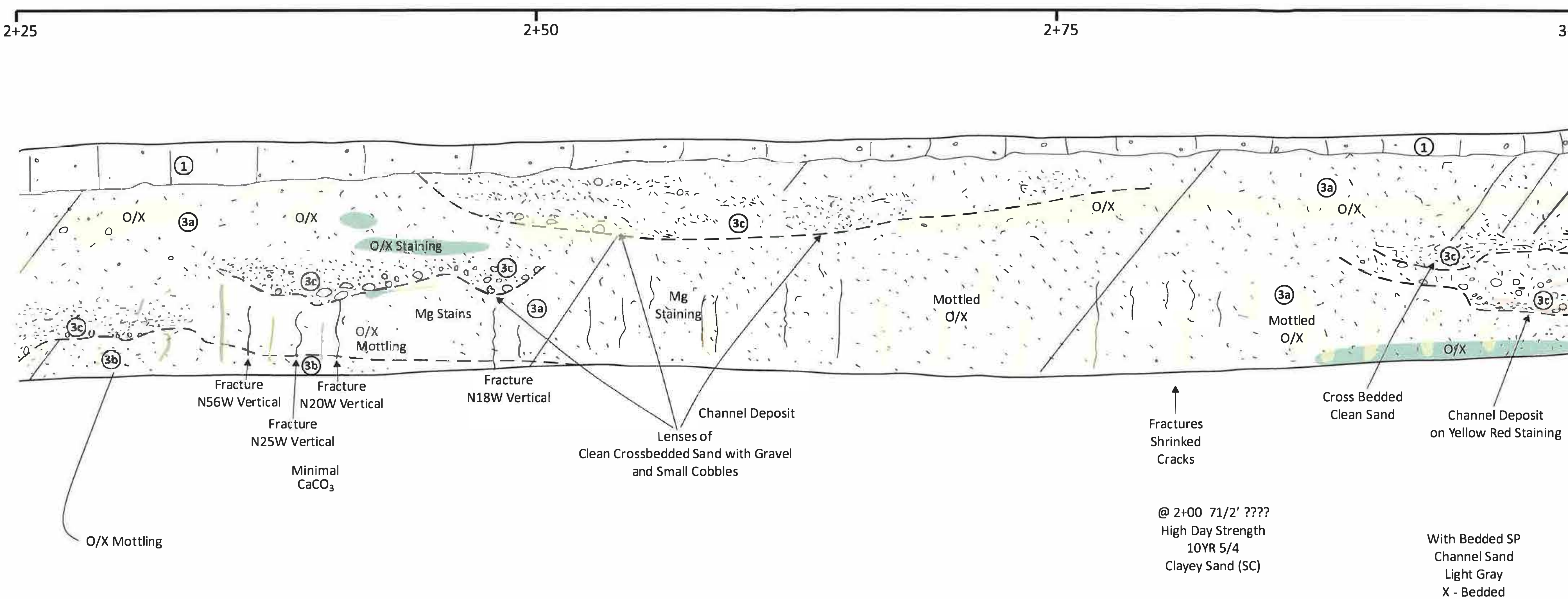
Proposed Bamiyan Marketplace
15749 Grand Avenue
Lake Elsinore, Riverside County, California

Earth Systems

1/17/2019 | File No.: 302169-001

T-1

Trench Orientation
N48E



LEGEND

Bt - Horizon

Weak B - Horizon

Lithologic Contact

Geologic Contact

Approximate Scale: 1" = 5'

0 5 Feet 10 Feet

Plate 8
T-1 Trench Log

Proposed Bamiyan Marketplace
15749 Grand Avenue
Lake Elsinore, Riverside County, California

Earth Systems

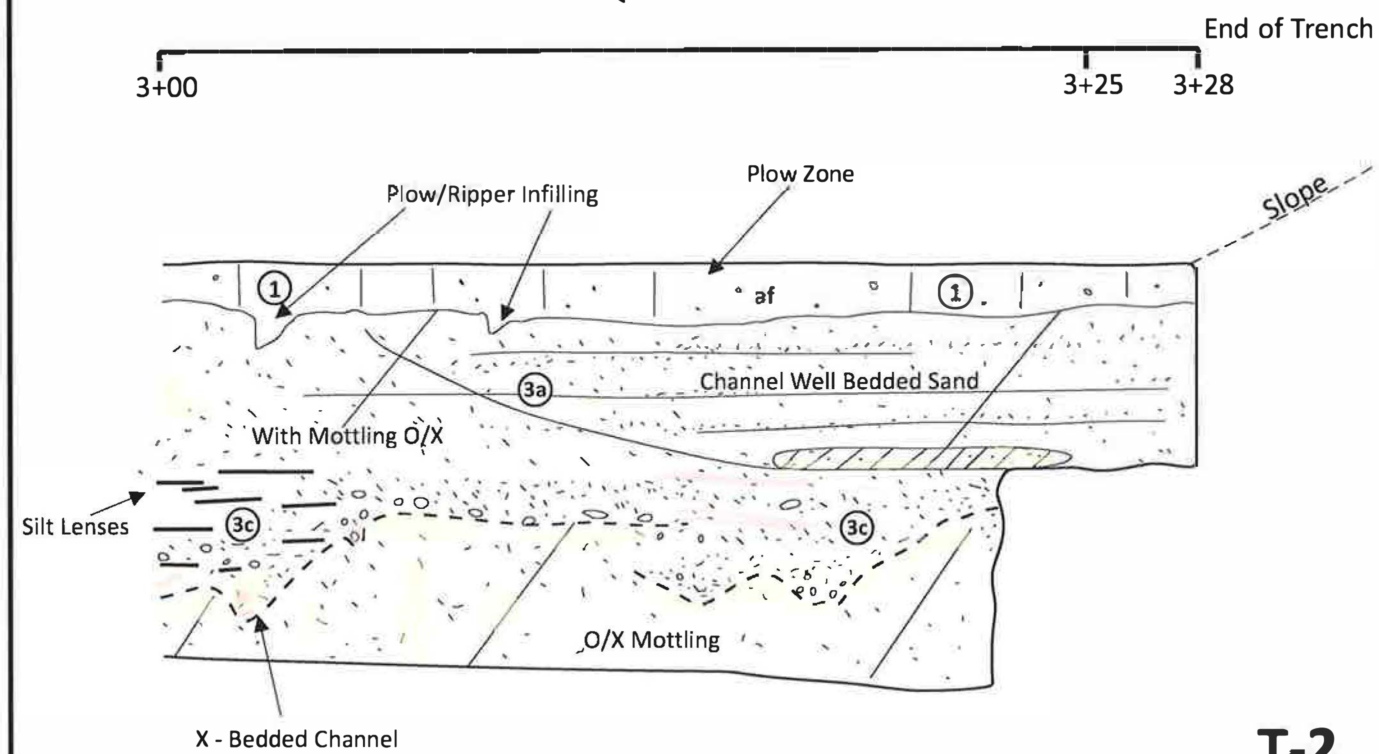
1/17/2019

File No.: 302169-001

T-1

Trench Orientation

N48E

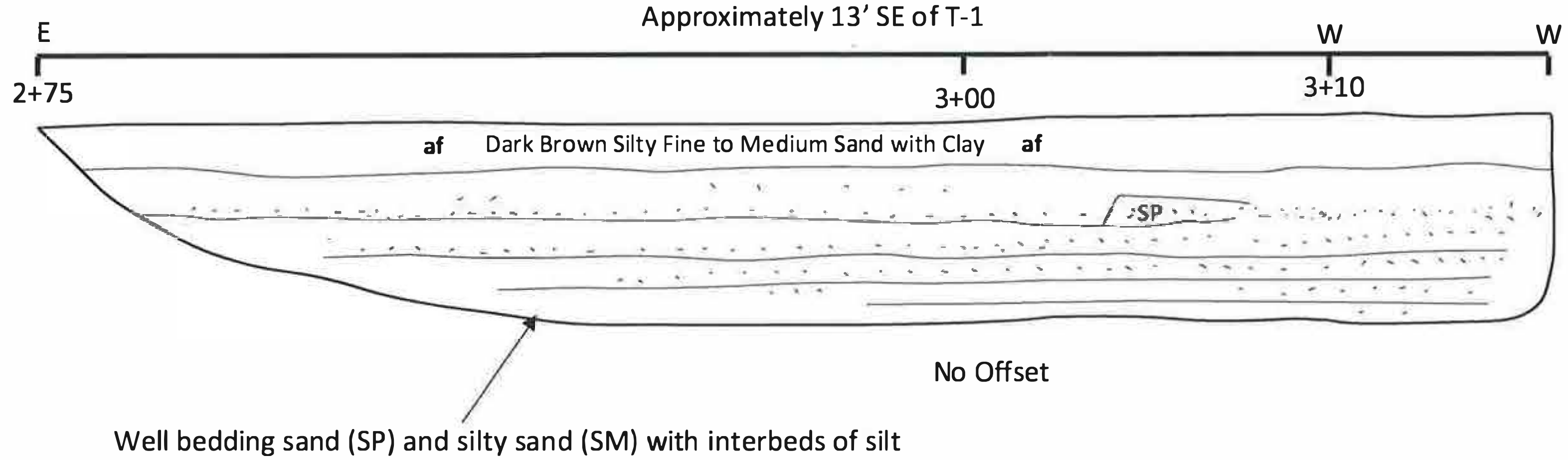


T-2

Trench Orientation

N52E

Approximately 13' SE of T-1



No O/S

O/X within silt and SM beds

LEGEND

	Bt - Horizon		Lithologic Contact
	Weak B - Horizon		Geologic Contact

Approximate Scale: 1" = 5'

0 5 Feet 10 Feet

Plate 9

T-1 & T-2 Trench Logs

Proposed Bamiyan Marketplace

15749 Grand Avenue

Lake Elsinore, Riverside County, California

Earth Systems

1/17/2019	File No.: 302169-001