State of California
Department of General Services
Real Estate Services Division
707 3rd Street, Suite 4-129
West Sacramento, California 95605

Geotechnical Engineering Report
Proposed Waterline Replacement
Patton State Hospital
3102 E. Highland Avenue
Patton, San Bernardino County, California

January 30, 2021
Revised February 11, 2021

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File No.: 304308-001
Doc. No.: 21-01-709

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State of California
Department of General Services (DGS)
Real Estate Services Division
707 3rd Street, Suite 4-129
West Sacramento, CA 95605

Attention: Mr. Kevin Nelson

Subject: **Geotechnical Engineering Report**

Project: **Proposed Waterline Replacement**
Patton State Hospital
3102 E. Highland Avenue
Patton, San Bernardino County, California

Earth Systems Pacific (Earth Systems) is pleased to present this geotechnical engineering report for the Proposed Waterline Replacement project located at Patton State Hospital located at 3102 E. Highland Avenue in the city of San Bernardino (Patton), San Bernardino County, California. This report presents our findings and recommendations for the geotechnical and geologic aspects of the project incorporating the information provided to our office. The site is suitable for the proposed pipe installation, 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 is revised to include DGS review requests regarding pipe abandonment and thrust block bearing allowance.

This report completes geotechnical tasks in accordance with our proposal PER-20-9-001R, dated September 16, 2020 and the State of California Agreement #5901-A Task Order #10. Other post-report services that may be required, such as specifications and plan reviews and/or consultation are additional services and will be billed in accordance to Earth Systems Fee Schedule in affect at the time the services are requested. Unless requested in writing, the client is responsible for distributing this report to the appropriate governing agency or other members of the design team.

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

Respectfully submitted,
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Section 1
INTRODUCTION

1.1 Project Description

This geotechnical engineering report has been prepared for the proposed Waterline Replacement project to be constructed at Patton State Hospital located at 3102 E. Highland Avenue in the city of San Bernardino (Patton), San Bernardino County, California. The general site location is presented on Plate 1 in Appendix A. We understand that the project will consist of replacing a leaking 14-inch water line that resides within the State of California property. The existing water line runs beneath a solar array and will need to be abandoned in support of a new alignment that extends from the check valves on Orange Street to the first set of valves on the west side of the solar array. We have assumed the pipeline will be on the order of 3 to 4 feet deep and be made of C900 Polyvinyl Chloride (PVC), as depicted on Plate 2. Plate 2 presents the general waterline alignment location. It is expected that conventional cut and cover trenching will be performed for most of the proposed pipeline alignment; however, tunneling Jack and Bore, or Horizontal Directional Drilling may be used at a drainage crossing along the pipeline alignment (see Plate 2 between borings B-2 and B-3 trending north-south). Pictures of the drainage are presented in Appendix A.

1.2 Site Description

The proposed Waterline Replacement project is approximately 1,630 feet in length and will extend from the check valves on Orange Street to the first set of valves on the west side of the solar array within the Patton State Hospital site. The site is generally flat, with elevations ranging from 1,395 to 1,450 feet, and slopes towards the southwest. The site is bound by North Orange Street to the east, Patton State Hospital to the south and west, and Serrano Middle School and residential development to the north. Based on the plans provided, there is an existing 14-inch waterline to be abandoned in place that runs beneath a solar array. Boring logs in Appendix A present surface and subsurface conditions at our exploration locations. Surface conditions generally consisted of exposed soils or gravel surfacing along the alignment.

1.3 Purpose and Scope of Services

Earth Systems services were to evaluate the site soil conditions at our boring locations and to provide professional geotechnical opinions and recommendations regarding the proposed waterline construction. The scope of services for this report included:

Field Work

1. A visual site assessment was made by Earth Systems' representative regarding surficially observed site conditions. In addition, Earth Systems reviewed select geological and geotechnical literature pertaining to the project area.
2. Proposed boring locations were marked in the field and Underground Service Alert notified of our intent to excavation. Private utility locating was also performed for greater assurance of not encountering a buried utility in our drilling.
3. Earth Systems drilled and logged five (5) exploratory borings at the locations indicated on the provided site plan (see Plate 2) to maximum depths of approximately 11½ and 51½ feet below the existing ground surface. Exposed soil profiles were observed relative to soil samples collected and groundwater conditions observed. Samples of the surface and subsurface materials were collected at various intervals, logged by our staff, and returned to our laboratory. The borings were backfilled with soil derived from the drilling.

Laboratory Analyses

Laboratory testing was performed on selected soil samples obtained from the exploratory borings. The test results aided in the classification and evaluation of the pertinent engineering properties of the various soils encountered. Testing included density and moisture content, particle size analysis, consolidation/collapse potential, maximum density/optimum moisture content, Sand Equivalent, Expansion Index, direct shear, R-Value, and chemical testing.

Engineering Analysis, Drafting, and Report Writing

Earth Systems conducted an engineering analysis of the data generated from the exploration and testing and prepared this written report presenting our findings and recommendations in the body of this report related to the following:

- A description of the proposed project including a vicinity map and site plan showing the approximate exploration locations;
- Logs of the exploration borings with engineering descriptions of the subsurface conditions encountered;
- A description of the surface and subsurface site conditions including groundwater conditions, perched water tables, and presence of rock as encountered in our field exploration;
- Results of our laboratory testing;
- Recommendations for site preparation and earthwork including stripping, grubbing, compaction criteria, imported fill criteria, and suitability of the onsite soils for use as fill and bedding.
- Discussions on effects of shallow water tables and recommendations for dewatering if shallow groundwater is apparent in the exploration borings.
- Estimated shrinkage and bulking factors for earthwork construction. Our estimated factors were provided as an estimated range of values and were limited to onsite soils and initial site grading;

- Recommendations for shoring and trench stability;
- Recommendations for Site Class;
- Recommendations for temporary excavations including OSHA type soil parameters and trench layouts;
- Recommendations for asphaltic concrete and concrete pavements;
- Recommendations for soil bearing strength and lateral earth pressures;

In addition, our report also addressed the following:

- A description of the geologic setting and possible associated geology-related hazards, including liquefaction, subsidence, and seismic settlement;
- A discussion of regional geology, site seismicity, and regional 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 seiches;
- A discussion of site conditions, including the geotechnical suitability of the site for the type of construction proposed, including settlement potentials of pipe bearing subgrades;
- A seismic analysis including recommendations for geotechnical seismic design coefficients in accordance with the 2019 California Building Code (CBC) for the General Procedure;
- Discussion of anticipated excavation conditions;
- Recommendations for underground utility trench backfill and bedding criteria;
- Recommendations for collapsible or expansive soils;
- Recommendations for dust control; and
- Recommendations for existing pipeline abandonment;
- Discussion of corrosion potential;
- An appendix, which will include a summary of the field exploration and laboratory testing program.

Section 2
METHODS OF EXPLORATION AND TESTING

2.1 Field Exploration

The subsurface exploration program included advancing five exploratory borings. The borings were drilled to depths ranging from approximately 11½ to 51½ feet below existing grades using a Mobile B-61 truck-mounted drill rig equipped with 8-inch hollow-stem augers provided by Cal Pac Drilling of Calimesa, California. The borings were advanced to observe soil profiles and obtain samples for laboratory testing. The approximate boring locations are shown on Plate 2, in Appendix A. The locations shown are approximate, established by consumer grade Global Positioning System (GPS) accurate to ± 15 feet in conjunction with pacing based on landmarks.

An engineer 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 a 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 drive the MC sampler within the borings was recorded.

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

Design parameters provided by Earth Systems in this report have considered an estimated 72% hammer efficiency based on data provided by the drilling subcontractor and accepted SP 117A criteria. The number of blows necessary to drive either a SPT sampler or a MC type ring sampler within the borings was recorded. Since the MC sampler was used in our field exploration to collect ring samples, the N-values using the California sampler can be roughly correlated to SPT N-values using a conversion factor that may vary from about 0.5 to 0.7. In general, a conversion factor of approximately 0.63 from a study at the Port of Los Angeles (Zueger and McNeilan, 1998 per SP 117A) is considered satisfactory. A value of 0.63 was applied in our calculations for this project.

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

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

2.2 Laboratory Testing

Samples were reviewed along with field logs to select those that would be analyzed further. Those selected for laboratory testing include, but were not limited to, soils that would be exposed and those deemed to be within the influence of the proposed structures. Test results are presented in graphic and tabular form in Appendix B of this report. Testing was performed in general accordance with American Society for Testing and Materials (ASTM) or other appropriate test procedure. A selected sample was 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 and 1140).
- 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 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 content of near 50% saturation. The samples were then submerged in water for 24 hours and the amount of expansion recorded with a dial indicator (ASTM D 4829).
- Sand Equivalency testing to evaluate fine grained soil impacts to backfill (CTM 217).
- R-Value to evaluate pavement support characteristics (CTM 301).
- Chemical Analyses (Soluble Sulfates and Chlorides (ASTM D 4327), pH (ASTM G 187), and Electrical Resistivity/Conductivity (APHA 2320-B) to evaluate the potential for adverse effects of the soil on concrete and steel.

Section 3
DISCUSSION

3.1 Soil Conditions

The field exploration indicates that soils at our exploration locations along the proposed water alignment predominately consist of silt and sand with varying amounts of sand, silt and a minor amount of silty clay to the maximum depth of exploration of 51½ feet below the ground surface. The soils are classified as sandy silt, silt, silty sand, sand with silt, and silty clay (Unified Soils Classification System symbols of ML, SM, SP-SM, and CL). Blow counts recorded indicate that the coarse grained (sandy) soils encountered are very loose to very dense. Site soil moisture observations varied between damp to moist with lab moistures ranging between 4 and 12 percent. A detailed description of the observed earth units soils is provided on the boring logs in Appendix A. Site soils are classified as Type C in accordance with CalOSHA.

Site soil layers are expected to have ground behavior as described in terms of the Tunnelman's Ground Classification, first described by Terzaghi (1950) and later modified by Heuer (1974). The Tunnelman's Ground Classification for the expected soils is presented in the following table.

Classification	Behavior	Typical Soil Types
Raveling - Slow Raveling - Fast Raveling	Chunks or flakes of materials begin to drop out of the arch or walls sometime after the ground has been exposed, due to loosening, over-stress and "brittle" fracture. In fast raveling ground, the process starts within a few minutes, otherwise the ground is slow raveling.	Residual soils or sand with small amounts of binder may be fast raveling below the water table, slow raveling above.
Running	Granular materials without cohesion are unstable at a slope greater than their angle of repose (approx. 30 - 35 degrees). When exposed at steep slopes they run like granulated sugar or dune sand until the slope flattens to the angle of repose.	Clean, dry granular materials. Apparent cohesion in moist sand, or weak cementation in any granular soil, may allow the material to stand for a brief period of raveling before it breaks down and runs. Such behavior is cohesive running.

Based upon the information gathered from our borings in the project area, our experience and observations in the direct project area, it is our opinion that the soil conditions expected in the vicinity of the proposed tunneling, and possible associated sending and receiving pits, are generally consistent with the alluvial materials encountered throughout the project area. The pit excavation and the casing tunnel are expected to be excavated in and be installed through damp, cohesionless sand with silt and silty sand and similar coarse-grained type soils. Deposits of gravel and cobbles may be encountered during tunneling.

The site lies within an area of high potential for wind erosion. Fine particulate matter (PM_{10}) can create an air quality hazard if dust is blowing. Watering the surface keeping it wet and placing silt fencing and wind breaks normally reduces the potential for this hazard.

3.2 Groundwater

No groundwater or perched water was encountered during our field exploration (maximum depth 51½ feet). Groundwater depths in the site area are generally greater than 100 feet below the ground surface. 1965 data (Fife, 1974) suggests that groundwater levels in the site vicinity in 1960 were approximately 350 feet deep. However, it is anticipated that groundwater levels in the immediate vicinity of the site have been and are currently over 100 feet deep.

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

- Well No. 01N03W29R001S is located approximately 2,350 feet southeast of the project site. The surface elevation of this well is approximately 1,372 feet and the groundwater reading as measured from 1972 was 1,042 feet.
- Well No. 01N03W32C001S is located approximately 3,075 feet southwest of the project site. The surface elevation of this well is approximately 1,289 feet and the groundwater readings as measured from 1912 to 1969 varied from 966 to 1,159 feet.
- Well No. 01N03W29N001S is located approximately 3,790 feet southwest of the project site. The surface elevation of this well is approximately 1,294 feet and the groundwater readings as measured from 1968 to 2007 varied from 964 to 1,002 feet.

Based on the above data, groundwater is not anticipated to be encountered during construction and is expected to be deeper than 100 feet at the site such that liquefaction is not a concern. Fluctuations of the groundwater level and localized zones of increased soil moisture subsurface content may be anticipated during and following the rainy season or from irrigation. Increased soil moisture or flowing water could be encountered in the drainage channel area if water is present or was recently present.

3.3 Collapse/Consolidation Potential

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

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

Table 1
Collapse Potential Values

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

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

Table 2
Collapsible Soil Hazard Ranking System

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

The results of collapse potential tests performed on five selected samples from depths ranging from 5 to 15 feet below the ground surface indicated a collapse potential on the order of 1.3 to 4.6 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 2%, which is the typical standard of care based on ASTM D5333 or where soil collapse becomes a concern for structural soils (Less than 113 pcf dry density) (County of Los Angeles, 2013).

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

1. Susceptible soil types were evaluated for collapse potential.
2. Soils are generally granular in nature, but cementation was observed in some samples in the form of calcium carbonate which was similar where observed amongst samples.
3. Pinhole voids were observed after HCl reaction tests conducted indicating loss of calcium carbonate in the soil matrix.
4. Soil collapse at the site appears to be related to in-place density (relative compaction) and cementation; however, a higher pre-collapse dry density (115 pcf) for 1% collapse was calculated. This was found to be higher than typical minimum guidelines as the cementing agent (calcium carbonate) has similar specific gravities when non-soluble but creates voids when dissolved.
5. Earth Systems performed evaluation providing collapse potential based on dry density for all samples.

Based on the above criteria and our field and laboratory findings, we estimate there is a “Moderate” collapse severity potential from soil layers between 5 and 15 ft bgs at the locations tested. Assuming the construction is accomplished according to Section 5.1 of this report, we estimate the collapse potential is approximately 1 inch.

3.4 Expansive Soils

Expansive soils are characterized by their ability to undergo significant volume change (shrink or swell) due to variations in moisture content. Changes in soil moisture content can result from rainfall, landscape irrigation, utility leakage, roof drainage, perched groundwater, drought, or other factors, and may cause unacceptable settlement or heave of structures, concrete slabs supported-on-grade, or pavements supported over these materials. Depending on the extent and location below finished subgrade, expansive soils can have a detrimental effect on structures. Based on our laboratory testing and experience with the project, the expansion potential of the on-site soils is generally “very low” as defined by ASTM D 4829 and the 2019 California Building Code. In general, the majority of site soils observed within the pipe zone were granular and “very low” in expansion.

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

3.5 Corrosivity

One sample of the near-surface soils within the site was collected and tested for potential corrosion of concrete and ferrous metals. Soils in the upper 0 to 5 feet were tested as in-situ drive samples. The tests were conducted in general accordance with the ASTM Standard Test Methods to evaluate pH, resistivity, and water-soluble sulfate and chloride content. The test results are presented in Appendix B. These tests should be considered as only an indicator of corrosivity for the samples tested. Other earth materials found on site may be more, less, or of a similar corrosive nature.

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

Table 3

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

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

Table 4

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

Caltrans considers a site corrosive if: Chloride concentration is greater than or equal to 500 ppm, sulfate concentration is greater than or equal to 2,000 ppm, or the pH is less than 5.5. As well, minimum resistivity less than 1,000 Ohm-cm is considered potentially corrosive. Test results show a pH value of 7.4, chloride content of 6.5 ppm, sulfate content of 12 ppm and minimum resistivity of 5,600 Ohm-cm. Although Earth Systems does not practice corrosion engineering, the corrosion values from the soils tested are normally considered as being mildly corrosive to buried metals and as possessing a "negligible" exposure to sulfate attack for concrete as defined in American Concrete Institute (ACI, 2011) 318, Section 4.3. The results of all chemical testing have been provided in Appendix B. The above values can potentially change based on several factors, such as importing soil from another job site and the quality of construction water used during grading and subsequent landscape irrigation.

3.6 Sand Equivalent

Sand Equivalent (SE) tests were performed on three samples collected from the upper 5 feet of soils encountered along the proposed alignments. The tests were conducted in general accordance with Caltrans Test Method 217. This test method provides the procedure for measuring the relative proportions of detrimental fine dust or clay-like material in soil or fine aggregates. A summary of the test results is presented in the table below:

Table 5
Sand Equivalent Test Results

Sample ID	Sample Location	Soil Type	Sand Equivalent (SE)
1	B1 @ 2.5’	SM	15
2	B2 @ 0-5’	SM	11
3	B5 @ 0-5’	SM	25

Test results for the alignment show Sand Equivalent values from 11 to 25. The soils tested were silty sand (SM). This soil type has Sand Equivalent values worse than typical specification (greater than 30 needed), therefore is not suitable for bedding or pipe zone material. Other soil types onsite are expected to be worse in respect to Sand Equivalent, and also not suitable for bedding or pipe zone.

3.7 Geologic Setting

Regional Geology: The project is in the northern-most portion of the Peninsular Ranges geomorphic province, which in the general site area is characterized by the San Bernardino Valley, San Andreas fault, San Jacinto fault, and Santa Ana River. The San Bernardino Valley is bounded on the northeast by the San Bernardino Mountains, and on the south by the Crafton Hills and the “Badlands”. The project area is located along the northeast margin of the valley, just southwest of the base of the San Bernardino Mountains and the San Andreas fault.

Local Geology: The Patton Hospital waterline project is located on the sloping alluvial plain characteristic of the Highland/Patton area. Younger Holocene sediments and Pleistocene older alluvium underlie the site. No active faults are mapped in the immediate vicinity of the waterline project; however, the San Bernardino segment of the San Andreas fault is located approximately 1,100 to 1,900 feet northeast of the water line alignment.

3.8 Geologic Hazards

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

3.8.1 Seismic Hazards

Surface Fault Rupture: The project site does not lie within a currently delineated State of California, *Alquist-Priolo* Earthquake Fault Zone (CGS 2018) for the nearby San Andreas fault. No other well-delineated fault lines cross through the project area, as shown on the California Geological Survey (CGS) Fault Activity Map (Jennings, 2010); Therefore, active surface fault rupture is not anticipated along the pipeline alignment.

Historic Seismicity: Approximately 30 Holocene-active faults or seismic zones lie within 60 miles of the project site (see Table A-1). The primary seismic hazard to the site is strong ground shaking from earthquakes along regional faults including the San Andreas fault, San Jacinto fault, Eastern California shear zone, and the many faults within the Los Angeles basin. The San Bernardino segment of the San Andreas fault is located approximately 0.2 miles northeast of the waterline. The San Jacinto fault is located approximately 7 miles southwest of the project.

The San Bernardino area is located within seismically active southern California where approximately historically 37 earthquakes of magnitude 5.5 or greater have occurred within 60 miles of the site. Many of the major historic earthquakes felt in the vicinity of the San Bernardino area have originated from nearby faults including the San Andreas and San Jacinto fault zones. These include the 1812 Wrightwood and 1857 Fort Tejon earthquakes, and multiple earthquakes along the San Jacinto fault zone in 1858, 1894, 1899, 1918, and 1923. Other earthquakes originating on regional southern California faults include 1933 Long Beach, 1992 Landers/Big Bear, 1994 Northridge, 1999 Hector Mine earthquakes.

Seismic Risk: While accurate earthquake predictions are not possible, various agencies have conducted statistical risk analyses. In 2013, the California Geological Survey (CGS) and the United States Geological Survey (USGS) presented new earthquake forecasts for California (USGS UCERF3). We have used these maps in our evaluation of the seismic risk at the site. The recent Working Group of California Earthquake Probabilities (WGCEP, 2014) estimated a 43 percent conditional probability that a magnitude 6.7 or greater earthquake may occur in 30 years (2014 as base year) along the Mojave segment of the San Andreas fault near Cajon Pass. The revised estimate for an 8+ magnitude earthquake along the Mojave segment of the San Andreas fault is about 17%.

There is an estimated 32 percent conditional probability that a magnitude 6.7 or greater earthquake may occur in 30 years (2014 as base year) along the local North and/or South San Bernardino segments of the San Andreas fault. The revised estimate for an 8+ magnitude earthquake along the nearby North or South San Bernardino segments of the San Andreas fault are about 8 to 9%.

The WGCEP estimated a 14 percent conditional probability that a magnitude 6.7 or greater earthquake may occur in 30 years (2014 as base year) along the proximal segment of the San Jacinto fault.

The primary seismic risk (ground shaking) at the site is a potential earthquake along the San Andreas fault that is about 0.2 miles northeast from the project. Geologists believe that the San Andreas fault has characteristic earthquakes that result from rupture of each fault segment. The estimated mean characteristic earthquake is magnitude 7.9 for the Mojave segment of the fault and a 7.5 magnitude for a multi-segment rupture event along the North and South San Bernardino branches of the San Andreas fault.

Estimated peak ground accelerations (2% exceedance in 50 years) is approximately 1 g. Acceleration values provided are estimates only. Actual acceleration values may be more or less than those provided and could exceed 1 g assuming a maximum considered earthquake event occurs on the nearby San Andreas or San Jacinto faults. Vertical accelerations are typically 1/3 to 2/3 of the horizontal accelerations but can equal or exceed the horizontal accelerations depending upon the local site effects and amplification.

3.8.2 Secondary Hazards

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

Seiches: Two water storage tanks are upgradient of the pipeline to the north-northwest. However, site topographic gradients are downward to the southwest. Also, local improvements including residential developments, a school, and paved streets would divert surface water flow, such that the potential for seiche-related flooding is currently considered low.

Landslides and Debris Flows: The project area is not in an area of anticipated landslides or debris flows, with the exception of the onsite drainage channel which is susceptible to debris flow, and based on our observations has had previous water flows (see pictures in Appendix A).

Soil Liquefaction, Dry Seismic Settlement, 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, 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 is low because a groundwater research indicates water is generally more than 50 feet below the ground surface historically and perched water conditions were not encountered, and were of a low potential to develop. The potential for lateral spreading is considered low due to non-liquefiable geologic materials.

3.8.3 Other Geologic Hazards

Ground Subsidence: The proposed pipeline is within a "susceptible" subsidence area as Holocene alluvial soils are present along the alignment. However, the pipeline alignment is in an area where similar geologic mediums are present (younger alluvium and older alluvium). Thus, some subsidence is probable along the pipeline alignment, although subsidence is anticipated to be areal in extent with low differential settlement potentials.

Flooding and Erosion: The proposed waterline lies within a designated *Other Flood Areas Zone X* "Areas of 0.2% annual chance flood; areas of 1% annual chance flood with average depths of less than 1 foot or with drainage areas less than 1 square mile; and areas protected by levees from 1% annual chance flood." This is shown on FEMA Map Number 06071C7965H revised August 28, 2008. The project site is in an area where sheet flooding and erosion could occur. Appropriate project design by the civil engineer, construction, and maintenance can minimize the sheet flooding potential. Site soils are granular and highly susceptible to wind and water erosion, as well as creep and failure on sloping terrain.

Section 4
CONCLUSIONS

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

General:

- From a geotechnical perspective, the site is suitable for the proposed development, provided the recommendations in this report are followed in the design and construction of this project.

Geotechnical Constraints and Mitigation:

- The primary geologic hazard is severe ground shaking from earthquakes originating on local and regional faults. A major earthquake above magnitude 7 originating on local segments of the nearby San Andreas fault zone and San Jacinto fault zone would be the critical seismic events that may affect the pipeline within the design life of the proposed improvement. Engineered design and earthquake-resistant construction increase safety and allow development of seismic areas.
- The potential for surface fault rupture is considered low.
- The underlying geologic condition for most of the alignment for seismic design is Site Class D for alluvial conditions.
- The potential for debris flows is considered moderate. Erosion of soils is considered a moderate to high potential.
- Other geologic hazards, including liquefaction and seismic settlement are considered low or negligible on this site.
- The soils encountered at the points of exploration generally included loose to very dense silty sands. These soils were generally damp to moist. The alignment soils are considered to be moderately prone to caving due to their sandy nature.
- The potential for soil collapse is low to moderate.
- Based on our laboratory testing and experience with the project, the Expansion Index of the onsite soil is “very low” as defined by ASTM D 4829. Samples of soils should be tested to confirm or modify these findings.
- The primary geologic hazards for tunneling and pit excavation are the damp to moist, cohesionless soils with possible oversize material which will be encountered. These soils have a high potential to ravel and run which can cause caving of the excavation and settlement above tunnels. Engineered design and construction can increase safety and allow development.
- Based on current conditions, groundwater is not anticipated to be encountered during construction.
- Laboratory testing of a soil sample showed potentially mild corrosivity to buried metallic elements and “negligible” for sulfate exposure to concrete. See Section 3.5 for further

information. Site soils should be reviewed by an engineer competent in corrosion evaluation.

- The soils are susceptible to wind and water erosion. Preventative measures to reduce erosion should be incorporated into site grading plans. Dust control should also be implemented during construction. Site grading should be in strict compliance with the requirements of the local Air Quality Management District (AQMD).
- Using the Cal/OSHA standards and general soil information obtained from the field exploration, classification of the near surface on-site soils will likely be characterized as Type C. Actual classification of site-specific soil type per Cal/OSHA specifications as they pertain to trench safety should be based on real-time observations and determinations of exposed soils by the Competent Person during grading and trenching operations.
- The boring excavations were advanced with moderate effort within the existing on-site soils due to loose to very dense soils with some oversize material observed. The onsite soils within the shallow excavations proposed are expected to be excavatable with conventional to heavy duty power equipment, depending on install depth.
- The unprocessed native soils are not typical of that used for bedding and pipe zone backfill, and import could be required if dictated by the pipeline designer.
- Test results for the alignment show Sand Equivalent values of 11 to 25. The soils tested were silty sand (SM). Onsite soils are not suitable for bedding or pipe zone material.
- The native soil is suitable for use as structural or intermediate trench backfill (excluding bedding and pipe zone), provided it is, stripped free of significant organic or deleterious matter and oversize materials.
- For general recommendations, the Standard Specifications for Public Works Construction ("Greenbook") are considered to be applicable. Where there is discrepancy with this report, the more stringent specification should be applied.

Section 5
RECOMMENDATIONS

5.1 Site Development-Trenching

A representative of Earth Systems Pacific (Earth Systems) should observe site clearing, grading, and the bottoms of excavations before placing fill and backfill. Local variations in soil conditions may warrant increasing the depth of overexcavation and recompaction.

Clearing and Grubbing: At the start of site grading, existing vegetation, pavements, foundations, non-engineered fill, construction debris, trash, and abandoned underground utilities should be removed from the proposed trenching area. Organic growth should be stripped from the surface and removed from the construction area. Areas disturbed during demolition and clearing should be properly backfilled and compacted as described below. Dust control should also be implemented during construction. Site grading should be in strict compliance with the requirements of the local Air Quality Management District (AQMD).

Subsequent to clearing and grubbing operations, areas to receive fill should be stripped of loose or soft earth materials until a uniform, firm subgrade is exposed, as evaluated by the geotechnical engineer or geologist, or their representative.

Existing Pipe Abandonment: The existing water line runs beneath the solar array and will need to be abandoned. We recommend a cement grout (fluidized, flowable, self-leveling, non-shrink grout) be placed under pressure to backfill the abandoned pipe. Grout should be pre-qualified through material type and strength/shrinkage data provided to the design team prior to use and approval.

In order to backfill the pipe with typical equipment, we recommend the pipe be exposed every 250 to 500 feet at an access point, have air vents and bulkheads added at each end and the pipe filled with the aforementioned grout (minimum 100 psi compressive strength). Grout pumping should continue until it comes out the vents. After the grout has set, the bulkheads at the access points should be removed and visual observation performed to verify the grout completely filled the pipe to the top of the pipe. The pipe ends should then be capped and the next section of pipe abandoned until the entire alignment is filled. Methods or lengths of run between access points which deviate from the above typical methodology should be reviewed by the project engineer for conformance to the intent of these recommendations.

We recommend pipe abandonment be done by a specialty contractor who is pre-qualified to do such work. Prequalification should include demonstration of multiple specific experience with the type of abandonment and crew proposed.

Shrinkage: The compaction related shrinkage factor for trench zone earthwork is expected to range from -1% to 16% for the upper excavated or scarified *site* soils (negative shrinkage equals bulking). This estimate is based on compactive effort to achieve an average relative compaction of about 92%.

Based upon 7 in-place densities evaluated, the average computed shrinkage is 8% with one standard deviation of 7%. Shrinkage, bulking, and construction related subsidence are highly dependent on and may vary with contractor methods for compaction. Losses from site clearing, oversize material, removal of unsuitable bedding and pipe zone materials, and removal of existing site improvements will affect earthwork quantity calculations and were not considered in the above soil compaction shrinkage and should be considered by the project design engineer.

5.2 Excavations and Pipeline Construction

Trench Safety: Excavations should be made in accordance with CalOSHA requirements. Construction site safety is the sole responsibility of the contractor. Soils were generally damp to moist, predominantly coarse grained (sandy), with potential slaking, raveling, and running in various strata to depth. Where excavations over 4 feet deep are planned, lateral bracing or appropriate cut slopes of 1.5:1 (horizontal:vertical) should be provided. No surcharge loads from stockpiled soils or construction materials should be allowed within a horizontal distance measured from the top of the excavation slope and equal to the depth of the excavation. The boring excavations were advanced with moderate effort within the existing on-site soils encountered.

Using the Cal/OSHA standards and general soil information obtained from the field exploration, classification of the near surface on-site soils will likely be characterized as Type C. Actual classification of site-specific soil type per Cal/OSHA specifications as they pertain to trench safety should be based on real-time observations and determinations of exposed soils by the Competent Person during grading and trenching operations. Due to the dry cohesionless site soil encountered, caving and running soils should be anticipated to depth. If soil and groundwater conditions are encountered during construction which differ from those described, our firm should be notified immediately in order that a review may be made, and supplemental recommendations, if required, can be provided.

The contractor should carefully review the boring logs in this report, and perform their own assessment of potential construction difficulties, and methods should be selected accordingly. The method of excavation and support is ultimately left to the contractor experienced with trench excavation.

Trench Excavation: Excavate trenches to ensure that sides will be stable under all working conditions. Slope trench walls or provide supports in conformance with all local and national standards for safety. Open only as much trench as can be safely maintained by available equipment. Backfill all trenches as soon as practicable, but not later than the end of each working day.

Dust Control: The proposed waterline alignment lies within an area of moderate high to 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 30-40 percent.

Shoring and Earth Pressures: Shoring may be required where soil conditions, space or other restrictions do not allow a sloped excavation. A braced or cantilevered shoring system may be used. Trench boxes should not be placed below or within the pipe zone elevation as their removal may loosen compacted backfill. Positive trench shoring 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) for Level Conditions	
Cantilevered	**Braced**
46	69

The values provided above assume a level ground surface adjacent to the top of the shoring and do not include a factor of safety. For sloping conditions (up to 2:1, h:v) adjacent to the excavation, 10 pcf should be added to above pressures. Steeper slopes may not be stable. All sloping conditions above the excavation should have the excavation protected from slope failure or debris moving downslope into the excavations. 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.

Alternatively, for a simple evaluation, shoring may be designed for ultimate active conditions (unrestrained wall) with a phi angle of 30 degrees, 0 psf cohesion, and 136 pcf total unit weight of soil (Active pressure coefficient of 0.333 and At-Rest pressure coefficient of 0.50). For passive conditions, a passive resistance of 300 pcf may be used (passive earth pressure coefficient k_p = 3.0, 136 pcf total unit weight).

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

Cantilevered shoring must extend to a sufficient depth below the excavation bottom to provide the required lateral resistance. We recommend required embedment depths be determined using methods for evaluating sheet pile walls and based on the principles of force and moment equilibrium. For this method, the allowable passive pressure against shoring, which extends below the level of excavation, may be assumed to be equivalent to a fluid weighing 300 pcf. Additionally, we recommend a factor of safety of at least 1.2 be applied to the calculated embedment depth and that passive pressure be limited to 2,250 psf.

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

Earth Systems should be retained during all site demolition and clearing and grading operations to monitor site conditions; substantiate proper use of materials; evaluate compaction operations; and verify that the recommendations contained herein are met.

<u>Pipe Subgrade, Bedding and Pipe-Zone Backfill</u>: Due to the potential for hydrocollapsible soils, the pipeline subgrade bottom should be compacted to a minimum of 90% relative compaction (ASTM D 1557) or to a firm condition at over optimum moisture content as evaluated by the project engineer or his representative for a depth of at least 12 inches below the bedding. Bedding and Pipe zone material shall be classified as sand per the USCS (SP, SW, SP-SM, SW-SM) with less than 5 percent fines (passing #200 sieve), a sand equivalent of 30 or more, and a maximum particle size of ¼ inch. Gravel should not comprise more than 15 percent of the backfill. The native soils are not typical of that used for bedding and pipe zone backfill and import would be required unless the pipe manufacture or designer allows soils of the types encountered and mechanical compaction is utilized. At least 4 inches of bedding over the bottom of the trench, or any protrusions, should be placed below the pipe. Bedding and pipe-zone material should be placed in maximum 6-inch loose lifts and be compacted to at least 90% relative compaction (ASTM D 1557) at over optimum moisture content. Compaction should be assured in the pipe haunches.

The subsurface materials anticipated may contain large quantities of oversize materials that may not be suitable as pipe subgrade. The final subgrade surface should be flat, firm, uniform and free of loose materials. Protruding oversize particles if any, should be removed from the trench bottom and replaced with compacted bedding materials.

<u>Trench-Zone Backfill</u>: The native soil is suitable for use as trench zone backfill (structural, intermediate and manholes) 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. Large oversize rock are not suitable for trench backfill.

Disposal alternatives for oversize materials encountered during trench excavation for the pipeline may include crushing for re-use as construction material or placement in designated rock disposal areas outside of construction areas. Other uses for oversize materials may include stockpiling for future use as project area landscaping.

The trench zone backfill material should be compacted to at least 90% relative compaction (ASTM D 1557) near its optimum moisture content for the trench zone and 95% for the street zone (upper 12 inches) where below pavement. Compaction should be verified by testing.

Trench-Zone backfill should be placed in maximum 8-inch lifts (loose) and compacted to at least its stated relative compaction (ASTM D 1557) at near its optimum moisture content. Trench zone (intermediate and structural) lifts may be increased to 24 inches in thickness if compacting with a vibratory sheeps-foot or similar vibratory excavator-mounted wheeled roller trench compactor while under the full-time observation of the project soils testing lab. Compaction should be verified by testing.

<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 pipeline with respect to the diameter of the pipeline, the less vertical load is applied to the pipeline. 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 pipeline. This is accounted by a load factor that is multiplied to the design strength of the pipeline. 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. Backfill should be considered to have a phi angle of 30 degrees, 0 psf cohesion, and 136 pcf <u>total</u> unit weight and 142 pcf saturated unit weight. The coefficient of friction between backfill and native soils may be taken as 0.35.

<u>General Trench Backfill and Compaction Recommendations</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.).

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 should not be performed.

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 1-sack cement-sand slurry (at least 1 foot as measured from the top of pipe) placed in maximum 3 foot lifts and allowed to cure 24 hours prior to subsequent lift placement. Care should be taken to not float the pipe during and after slurry placement. Backfill operations should be observed and tested to monitor compliance with these recommendations. The trench bottom should be in a firm condition prior to placing pipe, bedding, or fill and be compacted to at least 90% relative compaction (ASTM D 1557).

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. If gravel is used, it should be fully wrapped on all sides, top, and bottom, with filter fabric such as Mirafi 140N. Loss of soil may cause damaging settlement. NOTE: Rocks greater than 3 inches in diameter should not be incorporated within utility trench backfill.

All soils should be moisture conditioned prior to application of compactive effort. 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. We recommend backfill be compacted in appropriate moisture conditioned lifts. Typical compaction lifts should be approved by the geotechnical engineer; however, typical moisture conditioned lifts for hand operated equipment ("jumping jacks, wackers, etc.) are 4 to 6 inches of loose lift. Typical walk behind steel drum or sheepsfoot vibratory compactors is 8 inches to 1 foot. Vibratory sheepsfoot wheels on the back of an excavator can typically compact properly moistened soil 1 to 2 feet in loose thickness.

Native soils used for fill or compacted in-place are expected to be sensitive to the addition of moisture (water). Introduction or application of excessive water will cause the site soils to become pumping and unstable and not suitable. Application of water for moisture conditioning should be carefully monitored by the contractor.

A program of compaction testing, including frequency and method of test, should be developed by the project geotechnical engineer at the time of grading. Acceptable methods of test may include Nuclear methods such as those outlined in ASTM D 6938 (Standard Test Methods for In-Place Density and Water Content of Soil and Soil-Aggregate by Nuclear Methods) or correlated hand-probing. Full time testing and observation of backfill placement is recommended during construction. Compaction should be verified by testing.

Soils which are found to have expansive potential greater than "very low" after grading and processing will require differing engineering requirements which should be provided on a case by case basis for each specific location.

Backfill Settlement: The post-construction settlement of backfill soils following site preparation and fill construction, as described in previous sections of this report, is estimated to be approximately 0.3 to 0.4 percent of the fill thickness for fills placed and compacted in accordance with the recommendations provided. This settlement could cause an elevation differential between existing soils and newly placed fill. Subgrade settlement could be on the order of 1 inch where water is introduced or percolates into the subgrade beneath the pipe. Pipe selection should have flexibility and allow for settlement movement. We have recommended subgrade compaction to reduce the potential, but due to depths of susceptible soils and magnitude of potential, hydrocollapse could still occur.

Thrust Blocks: For design of thrust blocks, the following design parameters may be used for thrust blocks embedded in firm soil, compacted backfill, or slurry, considering at least 2½ feet of cover. Where thrust block bearing soils which are exposed are not firm, they should be excavated to firm material or be recompacted to at least 85% relative compaction:

- Allowable Bearing: 1,500 psf based upon the consistency of the soils encountered. We understand an *Office of the State Fire Marshal, Notes for Thrust Block Restraints* guidance document is being utilized by the client. That document is being utilized for the determination of thrust block bearing areas, and is based upon a 2,000 psf allowable soil bearing pressure for the tabulated required minimum thrust block bearing areas. For use of that document and the allowable bearing of 1,500 psf for this project, we recommend the tabulated bearing areas be increased in a linear fashion as the ratio of 2,000 psf to 1,500 psf (i.e. increase the tabulated areas based on 2,000 psf in the cited document by 1.33 times). This is considered to apply for all Conditions I through VIII as presented on the *Thrust Block Restraints* guidance document and extrapolated to 14" pipe size (or other).
- Passive Pressure: The passive resistance of compacted fill may be assumed to be equal to the pressure developed by an equivalent fluid with a density of 300 pcf. The maximum passive resistance should not exceed 1,500 psf. A one-third increase in the passive value may be used for seismic loads. The passive resistance of the subsurface soils will diminish or be non-existent if trench sidewalls slough, cave, or are overwidened during or following excavations. If this condition is encountered, our firm should be notified to review the condition and provide remedial recommendations, if warranted.
- Vertical Uplift Resistance: Vertical uplift resistance may consider a soil unit weight of 100 pounds per cubic foot. The upper 1 foot of soil should not be considered when calculating passive pressure unless confined by overlying asphalt concrete pavement or Portland cement concrete slab. 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.
- Friction Coefficient: An allowable coefficient of friction of 0.35 may be used between the bottom of cast-in-place thrust blocks and compacted fill.

5.3 Seismic Design Criteria

This project may be subject to severe ground shaking due to potential fault movements along regional faults. The underlying geologic condition for most of the alignment for seismic design is Site Class D for alluvial conditions. As such, the *minimum* seismic design for structures should comply with the 2019 edition of the CBC using the seismic coefficients summarized in the table shown below or in Appendix A in more detail.

Table 7
2019 CBC (ASCE 7-16) Seismic Parameters*

Site Coordinates: 34.1418°N and 117.2147°W	
Site Class:	D
Seismic Design Category	E
Maximum Considered Earthquake (MCE) Ground Motion	
Short Period Spectral Response S_s:	2.597 g
1 second Spectral Response, S_1:	1.048 g
Site Coefficient, F_a:	1.00
Site Coefficient, F_v:	1.70
PGA_M	1.188 g
Design Earthquake Ground Motion	
Short Period Spectral Response, S_{DS}	1.731 g
1 second Spectral Response, S_{D1}	1.188 g

The intent of the Code 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.

5.4 Site Drainage

Positive drainage should be maintained away from the excavations to prevent ponding and subsequent saturation of the nearby soils. Drainage should be maintained for graded and paved areas. Water should not pond on or near paved areas.

- Trenches should be maintained to provide adequate drainage to reduce the adverse effects of long-term standing water and erosive sheet flow over their surface and edges. Prolonged standing water will saturate the upper soils which may become weak, allowing plastic deformation. Additionally, uncontrolled flow or unregulated flow over roadway surfaces can lead to erosion and gullies. Water control and conveyance is possibly the most important factor in the life of a roadway. Crown or slope and site drainage should be maintained both during project construction and over the entire life of the project.

5.5 Asphalt Concrete (AC) Roads

Alternatives for asphalt concrete pavements are given below. The following preliminary pavements sections are provided for estimation purposes only. Two traffic indices were assumed for preliminary design (TI of 5.0 and 7.0). The project civil engineer should review the assumed traffic indices to determine if and where they are appropriate for use at the project.

The R-Value of the soils encountered and tested was 54. During grading, samples of the actual pavement subgrade should be observed or tested for R-Value, and the pavement sections refined as needed. Asphalt concrete pavement sections were calculated using Caltrans software CalFP Version 1.5. Based on the assumed traffic indices and assuming a minimum R-Value of 54, the following preliminary pavement sections with AC and Base are provided.

Where patches are made in existing AC or aggregate base, the newly repaved section should be at least 1-inch greater thickness than the existing AC, and 1 inch greater in aggregate base thickness. Trench joints within AC paving should be sealed as recommended within, and be over-ground, or overcut, at least 12 inches laterally beyond the trench width and be repaved.

Table 8
Preliminary Pavement Sections

R-Value of Subgrade Soils - 54 (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 Index 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.

The above pavement recommendations are contingent on the following:

- Pavement Area Preparation: Subsequent to the removal of debris and deleterious materials, and backfilling, the subgrade should be scarified and moisture conditioned, to near optimum moisture (if previously unprocessed) and compacted to at least 95% relative compaction (ASTM D 1557) for a depth of at least 12 inches below existing grade or finish subgrade (whichever is deeper). Compacted fill should be placed to finish subgrade elevation. Compaction should be verified by testing.
- All over-excavations should extend to a depth where the project geologist, engineer or his representative has deemed the exposed soils as being suitable for receiving compacted fill. The materials exposed at the bottom of excavations should be observed by a geotechnical engineer or geologist from our office prior to the placement of any compacted fill soils. Additional removals may be required as a result of observation and/or testing of the exposed subgrade subsequent to the required over-excavation.
- 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.

- Subgrade soils should be compacted near or slightly over optimum moisture content.
- 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).
- Asphaltic concrete should be Caltrans, ½-in. or ¾-in. maximum-medium grading and compacted to a minimum of 95% of the Marshall density or equivalent.
- Within the structural pavement section areas, positive drainage (both surface and subsurface) should be provided. In no instance should water be allowed to pond on the pavement, especially at joints between curb/gutters and the pavement section. Roadway performance depends greatly on how well runoff water drains from the site. Saturated subgrade soils and base will lead to premature roadway failure. This drainage should be maintained both during construction and over the entire life of the project.
- Existing street repair subsequent to utility installation or construction of turnouts should follow the guidelines of Riverside County or the appropriate governing agency.
- Where new asphalt roadways will be installed against existing roadways or the roadway is repaired, the repaired asphalt concrete pavement section should be designed and constructed to have at least the AC pavement (plus 1 inch) and aggregate base section as the original pavement section thickness (for both AC and base), the minimum section specified by the governing agency, or upon the newly calculated pavement sections presented within, whichever is greater.
- Proper methods, such as hot-sealing or caulking, should be employed to limit water infiltration into the asphalt concrete pavement base course and/or subgrade at construction/expansion joints and/or between existing and reconstructed pavement 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 utility trench excavation, should not be spread or placed over compacted finished grade soils unless subsequently compacted to at least 95% of the maximum dry unit weight, as evaluated by ASTM D 1557 test procedure, at near optimum moisture content, if placed under areas designated for pavement.
- The appropriate pavement design section depends primarily on the shear strength of the subgrade soil exposed after grading and anticipated traffic over the useful life of the pavement. R-value testing or confirmation observation should be performed during grading to verify and/or modify the preliminary pavement sections presented within this report. Pavement designs assume that heavy construction traffic will not be allowed on base cap or finished pavement sections.

5.6 Design and Construction Recommendations for Tunneling Operations

We understand that several alternative methods of tunnel excavation and support may be considered based on the anticipated ground conditions along the alignment. The method of excavation and support is ultimately left to the contractor. We anticipate that tunneling may employ a variety of methods, each with advantages and disadvantages. The primary methods available for excavation include open face shields, mechanized tunnel boring machines, microtunneling, and bore and jacking.

In general, we consider ground conditions and the proposed ground cover at the pipe-jacking/microtunneling (pipe-jacking) alignment location will be suitable for tunnel operations with strict and careful construction; however, performance of tunneling operations is strongly dependent on construction methods and procedures. The tunneling operations should be constructed only by contractors highly experienced in this type of construction under the anticipated conditions, and under strict construction monitoring and quality control. Full-time observation by the geotechnical engineer or his representative is strongly recommended.

The tunneling operations will be conducted in soils anticipated to consist of dry to wet sand with varying amounts of silt, gravel and possible cobbles. Along the tunnels we anticipate slow to fast raveling and loose running ground conditions for the granular soils in the tunnel crown. Slower raveling is anticipated where medium dense soils are encountered with some apparent cohesion due to moisture content and fines content. Fast raveling is anticipated locally where very loose to loose sands with less than 12 percent fines and low moisture content are encountered. Clean, dry, cohesionless sands and gravels are prone to running ground conditions. Due to the nature of the alluvial soils at the site, some fine to coarse gravel, cobbles, and boulders could occur in isolated lenses or pockets. Hard rock or mixed face conditions are not anticipated. Groundwater is not expected to be encountered on most of the alignment. Seasonal shallow groundwater is anticipated at creek and drainage locations.

Due to the anticipated raveling to running ground conditions, we anticipate ground control techniques may be required. The contractor should carefully review the boring logs in this report, and perform their own assessment of potential construction difficulties, and installation methods should be selected accordingly. We recommend that the contractor's actual method of construction be reviewed and approved by the geotechnical engineer prior to construction to verify that the installation method is consistent with the design assumptions.

A primary concern for the project is the potential for ground settlement above the tunnel due to ground losses during tunneling. Especially due to vibration from nearby traffic. Ground losses may occur as the result of soil movement in front of the excavation by means of raveling, caving, or flowing ground, or may also occur as a result of soil movement around the cutting edge of the tunneling machine and the tail of the machine. Ground losses may also occur as the result of soil movement downward toward the support system as it leaves the tail of the machine. We recommend that the ground surface and other vital structures (utilities, buildings, roadways, etc.) be monitored for settlement and signs of distress prior to, during and following tunnel operations.

The contractor should have an onsite means of immediately controlling any potential caving or other ground loss conditions which may occur in the expected site soils in order to prevent any damage to utilities or roadway above the tunneling operations. Additionally, the contractor should provide for construction techniques to control raveling, and loose running conditions through proper equipment and installation selection and ground improvement techniques, such as pressure grouting, if required.

Based upon the anticipated construction methods and in addition to the recommendations presented within, general construction recommendations are provided below:

- Use a lubricant between the casing and in-situ soils to aid in pipe thrusting and reduce adhesion.
- Site soils are granular and abrasive. Pipe design should account for these abrasive conditions.
- Minimize, if possible, large magnitude vibrations of the pipe that could cause consolidation of the soils surrounding the pipe.
- To minimize disturbance, attempt to provide at least two diameters of cover between the proposed hole or permanent casing and the ground surface for settlement sensitive structures, and on the order of one to one and one-half diameters of cover between the casing and any existing utilities (depending on sensitivity of the existing utility). Where this is not possible, the tunneling should be a lossless technique between the outer casing wall and surrounding soil.
- Provide near continuous mining if possible.
- Under no circumstances should the casing be withdrawn, or upper soils left to their own support.
- Do not over-cut the bore to a diameter larger than the diameter of the proposed permanent casing.
- Provide for sealing of the permanent casing ends or complete pipe grouting to prevent the migration of soils and water into the casing void space and minimize potential ground loss.

The methods presented above may or may not apply to all pipe-jacking/tunneling situations. Solutions to problems are greatly dependent on soil conditions, jacking method, and pipe installation procedures. The tunneling contractor, in coordination with the project owner, geotechnical engineer, and any governing agency should make all final determinations in regard to solutions to problems encountered during tunneling activities.

<u>Design Parameters for Tunneling Operations</u>: Provided below are geotechnical soil parameters for tunneling design.

Methods of installation will influence the frictional resistance between the casing wall and alignment soils. Preliminary frictional resistance between the proposed casing and the alignment soils are presented in the table below. These values do not include a factor of safety and it is possible that greater frictional resistance could be encountered.

Lateral Sliding Resistance Coefficient of Friction: Smooth Steel to Soil	0.25
Lateral Sliding Resistance Coefficient of Friction: Ductile Iron or PVC to Soil	0.3
Lateral Sliding Resistance Coefficient of Friction: Smooth Textured Concrete to Soil	0.35

Allowable passive jacking resistance of jacking pit sidewalls can be estimated using an equivalent fluid weight of 300 pounds per cubic foot (pcf). The total allowable pressure should not exceed the 1,500 pounds per square foot (psf). It is expected that vertical excavations for the open cuts will be shored, although sloped excavations can also be considered if space limitations permit. Shoring for vertical cuts may consist of soldier piles and lagging, or sheet piles with horizontal struts for internal bracing designed as recommended earlier in this report. Backfill of the pits should be as per earlier sections of this report for Trench Backfill. Design recommendations presented previously are also considered to apply.

<u>Observation and Testing</u>: 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 construction processes, proper geotechnical observation and testing during construction is imperative to allow the geotechnical engineer the opportunity to verify assumptions made during the design process. Therefore, we recommend that Earth Systems be retained during the construction of the proposed tunneling operations 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 commission.

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.

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, time of our services, 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.

As the Geotechnical Engineer of Record for this project, 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.

The recommendations provided in this report are based on the assumption that Earth Systems will be retained to provide observation during the construction phase to evaluate our recommendations in relation to the apparent site conditions at that time. If we are not accorded this observation, Earth Systems assumes no responsibility for the suitability of our recommendations. In addition, if there are any changes in the field to the plans and specifications, the Client must obtain written approval from Earth Systems' engineer that such changes do not affect our recommendations. Failure to do so will vitiate Earth Systems' recommendations.

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 pipeline alignment 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 and backfill.
- Consultation as needed during construction.

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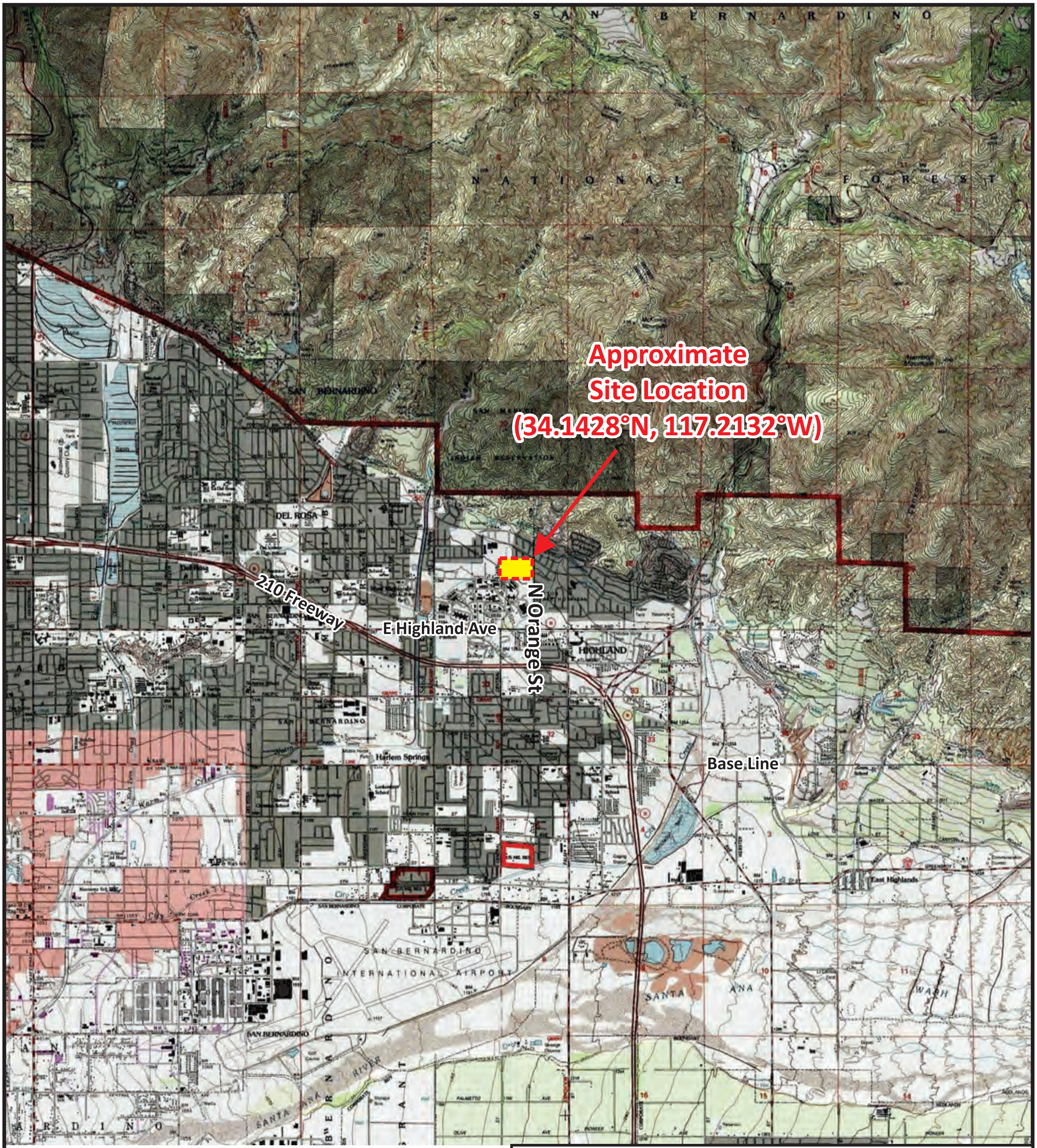
APPENDIX A

Plate 1 – Site Vicinity Map
Plate 2 – Exploration Location Map
Plate 3 – Regional Geologic Map
Table A-1 – Fault Parameters
Table A-2 - Historical Earthquakes
General Procedure Seismic Design Values
Site Class Estimator
Terms and Symbols Used on Boring Logs
Soil Classification System
Logs of Borings
Photos of Drainage Channel



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

LEGEND

Approximate Site Location

Approximate Scale: 1" = 1 Mile

0 1 Mile 2 Miles

Plate 1
Site Vicinity Map

Patton State Hospital Waterline Replacement
3102 East Highland Avenue
Patton, San Bernardino County, California

Earth Systems

1/30/2021	File No.: 304308-001

EXHIBIT A

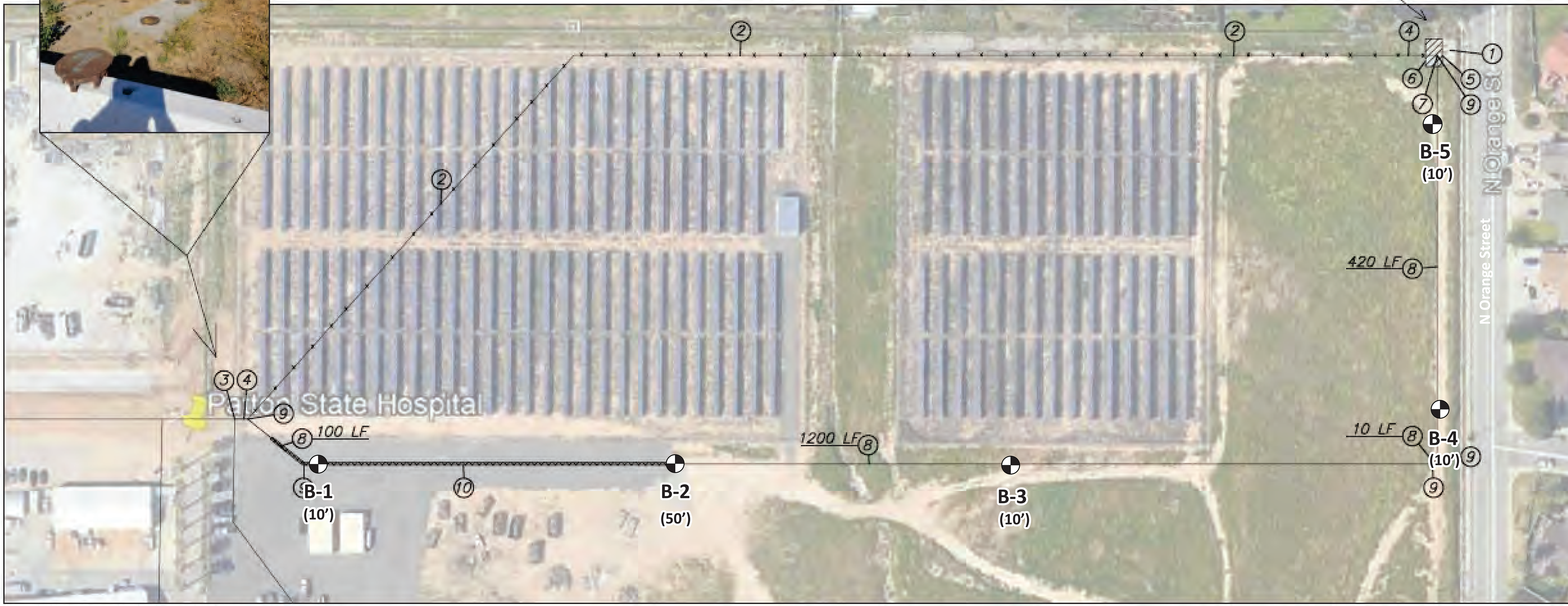
WATERLINE REPLACEMENT CONCEPTUAL LAYOUT

PATTON STATE HOSPITAL

3102 EAST HIGHLAND AVENUE

PATTON, CA 92369

SEPTEMBER 2020



CONSTRUCTION NOTES:

(1) EXISTING 8" DDVA (2 EA) TO REMAIN FOR REUSE.

(5) SAWCUT AND REMOVE 500 SF CONCRETE PAD UNDER DDVA'S. REPLACE ONCE DONE.

(8) INSTALL 14" C900 WATERLINE IN 3' WIDE AND 4' DEEP TRENCH WITH WARNING TAPE

ASSUME = IN ADDITION TO THE CONSTRUCTION NOTES:
1.) $25,000 FOR VERTICAL DEFLECTION FOR UTILITY CONFLICTS.
2.) $10,000 FOR UNKNOWNS.

N

Approximate Scale: 1" = 100'

0' 100' 200'

LEGEND

B-5
(10')

Approximate Exploration Locations
(Depth in Feet)

Plate 2
Exploration Location Map

Patton State Hospital Waterline Replacement
3102 East Highland Avenue
Patton, San Bernardino County, California

Earth Systems

1/30/2021 | File No.: 304308-001

Source: Exhibit A Waterline Replacement Conceptual Layout dated 9/5/2020.

LEGEND

- Dune sand
- Alluvium
- Lake deposits
- Glacial deposits
- River terrace deposits
- Pleistocene marine and marine terrace deposits
- Pleistocene volcanic rocks: Qpv[r]-rhyolite, Qpv[a]-andesite, Qpv[b]-basalt, Qpv[p]-pyroclastic rocks
- Pleistocene nonmarine
- Plio-Pleistocene nonmarine
- Quaternary and/or Pliocene cinder cones
- Undivided Pliocene nonmarine
- Upper Pliocene marine
- Middle and/or lower Pliocene nonmarine
- Middle and/or lower Pliocene marine
- Pliocene volcanic rocks: Pv[r]-rhyolite, Pv[a]-andesite, Pv[b]-basalt, Pv[p]-pyroclastic rocks
- Undivided Miocene nonmarine
- Upper Miocene nonmarine
- Upper Miocene marine
- Middle Miocene marine
- Lower Miocene marine
- Miocene volcanic rocks: Mv[r]-rhyolite, Mv[a]-andesite, Mv[b]-basalt, Mv[p]-pyroclastic rocks
- Oligocene nonmarine
- Eocene nonmarine
- Eocene marine
- Paleocene marine
- Tertiary nonmarine
- Tertiary marine
- Tertiary intrusive (hypabyssal) rocks: Ti[r]-rhyolite, Ti[a]-andesite, Ti[b]-basalt
- Tertiary lake deposits
- Tertiary volcanic rocks: Tv[r]-rhyolite, Tv[a]-andesite, Tv[b]-basalt, Tv[p]-pyroclastic rocks
- Upper Cretaceous marine
- Upper Jurasssic marine
- Mesozoic granitic rocks
- Mesozoic basic intrusive rocks
- Mesozoic ultrabasic intrusive rocks
- Jurassic-Triassic metavolcanic rocks
- Pre-Cretaceous metamorphic rocks (ls=limestone)
- Pre-Cretaceous metasedimentary rocks
- Pre-Cretaceous metavolcanic rocks
- Pre-Cenozoic granitic and metamorphic rocks
- Precambrian igneous and metamorphic rock complex
- Undivided Precambrian metamorphic rocks: pCg=gneiss, pCs=schist, pCls=limestone and/or dolomite
- Undivided Precambrian granitic rocks

Source: CGS Geologic Map of California, Santa Ana & San Bernardino

Plate 3
Regional Geologic Map

Patton State Hospital Waterline Replacement
3102 East Highland Avenue
Patton, San Bernardino County, California

Earth Systems

1/30/2021	File No.: 304308-001

Table A-1
Fault Parameters

Fault Section Name	**Distance** (miles)	(km)	Upper Seis. Depth (km)	Lower Seis. Depth (km)	Avg Dip Angle (deg.)	Avg Dip Direction (deg.)	Avg Rake (deg.)	Trace Length (km)	Fault Type	**Mean Mag**	Mean Return Interval (years)	Slip Rate (mm/yr)
San Andreas (San Bernardino S) FM3.1, 3.2	0.2	0.3	0.0	12.8	90	210	180	43	A	**7.6**	150	29
San Andreas, (North Branch, Mill Creek) FM3.1,	0.5	0.7	0.0	18.2	76	204	180	106	A	**7.6**	219	34
San Andreas (San Bernardino N) FM3.1, 3.2	0.7	1.1	0.0	12.8	90	212	180	35	A	**7.5**	103	27
San Jacinto (San Bernardino) FM3.1, 3.2	6.7	10.8	0.0	16.1	90	225	180	37	A	**7.6**	219	17
San Jacinto (Lytle Creek connector) FM3.1, 3.2	6.9	11.1	0.0	16.1	90	na	na	23	B'	**6.7**		
San Jacinto (San Jacinto Valley) rev FM3.1, 3.2	8.7	14.0	0.0	16.1	90	223	180	18	A	**7.6**	219	18
Cleghorn FM3.1, 3.2	9.5	15.2	0.0	15.5	90	187	0	25	B	**6.7**		3
Fontana (Seismicity) FM3.1, 3.2	11.1	17.9	0.0	16.3	80	313	na	24	B'	**6.7**		
North Frontal (West) FM3.1, 3.2	12.5	20.1	0.0	15.7	49	171	90	50	B	**7.2**		1
Cucamonga FM3.1, 3.2	13.2	21.3	0.0	7.8	45	347	90	28	B	**6.6**		5
San Gorgonio Pass FM3.1, 3.2	13.7	22.1	0.0	18.5	60	11	na	29	B'	**6.9**		
San Jacinto (San Jacinto Valley, stepover)	17.9	28.8	0.0	16.1	90	224	180	24	A	**7.6**	219	9
San Jacinto (Stepovers Combined) FM3.1, 3.2	17.9	28.8	0.0	16.5	90	229	180	25	A	**7.5**	110	4
Mission Creek FM3.1, 3.2	19.7	31.8	0.0	17.7	65	5	180	31	B'	**6.9**		
San Andreas (Mojave S) FM3.1, 3.2	22.6	36.3	0.0	13.1	90	206	180	98	A	**7.7**	102	34
San Gabriel (Extension) FM3.1, 3.2	25.2	40.6	0.0	14.7	61	6	180	62	B'	**7.2**		
Helendale-So Lockhart FM3.1, 3.2	25.8	41.5	0.0	12.8	90	51	180	114	B	**7.4**		0.6
San Andreas (San Gorgonio Pass-Garnet HIll) FM	27.0	43.4	0.0	12.8	58	20	180	56	A	**7.6**	219	24
San Jose FM3.1, 3.2	27.2	43.8	0.0	15.8	74	334	30	20	B	**6.6**		0.5
North Frontal (East) FM3.1, 3.2	28.2	45.4	0.0	16.6	41	187	90	28	B	**6.9**		0.5
Pinto Mtn FM3.1, 3.2	28.9	46.5	0.0	15.5	90	175	0	74	B	**7.2**		2.5
Chino, alt 2, FM3.2	29.6	47.6	0.0	13.4	65	234	150	29	B	**6.7**		1
Chino, alt 1, FM3.1	29.6	47.7	0.0	9.0	50	236	150	24	B	**6.6**		1
Elsinore (Glen Ivy) rev FM3.1, 3.2	29.7	47.7	0.0	13.2	90	218	180	26	A	**7.0**	222	3
Sierra Madre FM3.1, 3.2	30.0	48.4	0.0	14.2	53	19	90	57	B	**7.2**		2
Whittier, alt 1, FM3.1	30.5	49.0	0.0	12.4	70	24	150	46	A	**7.1**	530	4
Whittier, alt 2 FM3.2	30.5	49.0	0.0	14.1	75	24	150	46	A	**6.9**	165	4
Yorba Linda FM3.1, 3.2	32.3	52.0	0.0	13.3	90	153	na	18	B'	**6.5**		
Elsinore (Glen Ivy stepover) FM3.1, 3.2	32.8	52.8	0.0	13.2	90	216	180	11	A	**7.1**	322	15
Elsinore (Stepovers Combined) FM3.1, 3.2	32.8	52.8	0.0	13.7	90	224	180	12	A	**7.6**	725	5
San Jacinto (Anza) rev FM3.1, 3.2	32.9	52.9	0.0	16.8	90	216	180	46	A	**7.6**	219	17
Lenwood-Lockhart-Old Woman Springs FM3.1, 3	36.0	57.9	0.0	13.2	90	43	180	145	B	**7.5**		0.9
Clamshell-Sawpit FM3.1, 3.2	36.7	59.0	0.0	14.0	50	334	90	16	B	**6.6**		0.5
Peralta Hills FM3.1, 3.2	36.7	59.1	0.3	14.0	50	3	na	14	B'	**6.5**		
Elsinore (Temecula) rev FM3.1, 3.2	36.8	59.2	0.0	14.2	90	230	180	40	A	**7.4**	431	3
Richfield FM3.1, 3.2	38.4	61.7	2.5	12.9	28	353	na	6	B'	**6.2**		
Johnson Valley (No) 2011 rev FM3.1, 3.2	40.1	64.5	0.0	15.9	90	51	180	52	B	**6.8**		0.6
Puente Hills, FM3.1	40.2	64.7	5.0	13.0	25	20	90	44	B	**7.1**		0.7
Puente Hills (Coyote Hills) FM3.2	41.0	65.9	2.8	14.6	26	358	90	17	B	**6.8**		0.7
Raymond FM3.1, 3.2	44.3	71.3	0.0	15.6	79	348	60	22	B	**6.7**		1.5

Reference: USGS OFR 2013-1165 (CGS SP 228) Based on Site Coordinates of 34.14182 Latitude, -117.21472 Longitude

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

Site Coordinates: 34.142 N 117.215 W

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

Day	Year	Epicenter Latitude (Degrees)	Epicenter Longitude (Degrees)	Distance from Site (mi)	Magnitude M_W
9/20	*1907	34.20	117.10	7.7	**5.8**
7/23	*1923	34.00	117.25	10.0	**6.2**
12/16	1858	34.20	117.40	11.3	**6.0**
7/22	1899	34.20	117.40	11.3	**5.9**
10/16	1999	34.24	117.04	12.1	**5.6**
11/22	1880	34.00	117.00	15.7	**5.5**
12/19	1880	34.00	117.00	15.7	**5.9**
6/14	1892	34.20	117.50	16.8	**5.5**
1/16	1930	34.20	116.90	18.4	**5.5**
7/22	1899	34.30	117.50	19.6	**6.4**
6/28	1992	34.16	116.85	20.9	**5.5**
6/28	1992	34.20	116.83	22.3	**6.5**
7/30	1894	34.30	117.60	24.6	**6.2**
12/25	*1899	33.80	117.00	26.6	**6.7**
2/28	1990	34.14	117.70	27.7	**5.7**
12/8	1812	34.37	117.65	29.4	**7.5**
2/7	1889	34.10	116.70	29.6	**5.6**
4/21	*1918	33.75	117.00	29.7	**6.8**
5/15	1910	33.70	117.40	32.3	**6.0**
7/8	1986	34.00	116.61	35.9	**6.0**
8/28	1889	34.20	117.90	39.4	**5.6**
6/28	1992	34.20	116.44	44.4	**7.3**
6/28	1991	34.27	117.99	45.1	**5.6**
7/28	1769	34.00	118.00	46.0	**6.0**
6/28	1992	34.13	116.41	46.0	**5.8**
3/15	1979	34.33	116.44	46.1	**5.5**
6/29	1992	34.10	116.40	46.7	**5.7**
6/6	1918	33.60	116.70	47.6	**5.5**
10/2	1928	33.60	116.70	47.6	**5.5**
10/1	1987	34.06	118.08	49.8	**5.9**
7/11	1855	34.10	118.10	50.7	**6.0**
4/99	1803	34.20	118.10	50.7	**5.5**
6/28	1992	34.12	116.32	51.2	**5.7**
4/23	1992	33.96	116.32	52.7	**6.2**
1/16	1857	34.52	118.04	53.8	**6.3**
3/11	1933	33.64	117.97	55.4	**6.4**
12/4	1948	34.00	116.23	57.2	**6.0**
4/11	1910	33.50	116.50	60.4	**5.8**
9/5	1928	35.00	117.00	60.5	**5.5**
10/16	1999	34.59	116.27	62.1	**7.1**

From full earthquake catalog in USGS OFR 2008-1437h as updated with current events through 2019. For events with an asterisk, alternate solutions are given in

General Procedure Seismic Design Values

2019 California Building Code (CBC) (ASCE 7-16) Seismic Design Parameters

(Values presented should only be used by a Structural Engineer to determine if the exception in 11.4.8 (ASCE 7-16) can be used)

			CBC Reference	ASCE 7-16 Reference
Seismic Design Category		**E**		
Site Class		**D**	Table 1613.5.6	Table 11.6-2
Latitude:		34.142	Table 1613.5.2	Table 20.3-1
Longitude:		-117.215		
Maximum Considered Earthquake (MCE) Ground Motion				
Short Period Spectral Reponse	S_S	**2.597 g**	Figure 1613.5	Figure 22-1
1 second Spectral Response	S_1	**1.048 g**	Figure 1613.5	Figure 22-2
Site Coefficient	F_a	1.00	Table 1613.5.3(1)	Table 11.4-1
Site Coefficient	F_v	1.70	Table 1613.5.3(2)	Table 11-4.2
	S_{MS}	2.597 g	$= F_a * S_S$	
	S_{M1}	1.782 g	$= F_v * S_1$	
Design Earthquake Ground Motion				
Short Period Spectral Reponse	S_{DS}	**1.731 g**	$= 2/3 * S_{MS}$	
1 second Spectral Response	S_{D1}	**1.188 g**	$= 2/3 * S_{M1}$	

Site Specific Evaluation May Be Required Due to Site Class = D or E and S1>=0.2. The Presented SDS and SD1 are NOT Valid Unless the Exception of ASCE7-16, Section 11.4.8 Applies

To		0.14 sec	$= 0.2 * S_{D1} / S_{DS}$
Ts (11.4.8 ASCE 7-16 Exception Assumed)		0.69 sec	$= S_{D1} / S_{DS}$
Risk Category	II		Table 1604.5
Seismic Importance Factor		1.00	
F_{PGA}		1.10	
PGA_M		**1.19**	
Vertical Coefficient (C_V)		1.50	Table 11.9-1

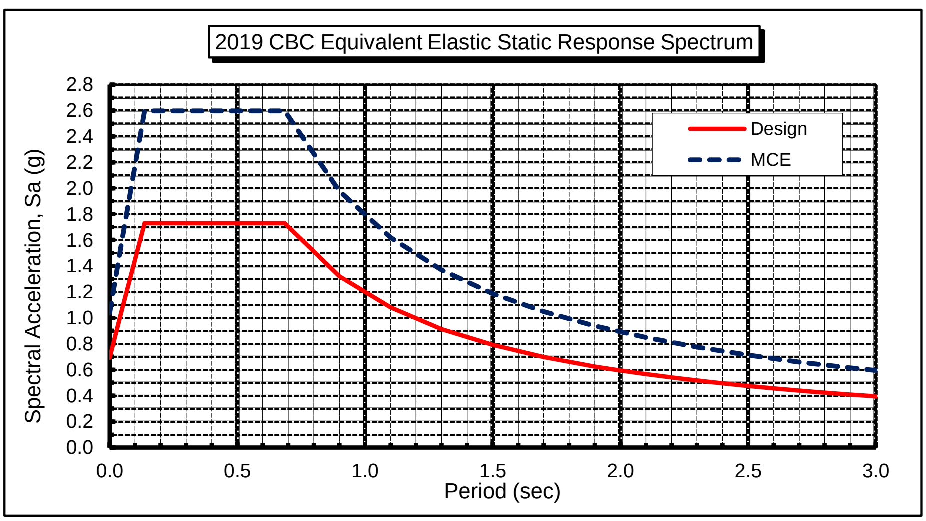


Table 11.5-1 Design

Period T (sec)	Sa (g)
0.00	0.693
0.05	1.071
0.14	1.731
0.69	1.731
0.90	1.320
1.10	1.080
1.30	0.914
1.50	0.792
1.70	0.699
1.90	0.625
2.10	0.566
2.30	0.516
2.50	0.475
2.70	0.440
2.90	0.410
3.10	0.383

Boring No.	B-2	Project and Number	Patton Waterline Rep	304308-001

ESSW Field Staff	
Drilling Company	CalPac
Drilling Method	6-8" H S A; HSA Inner Diameter: 3"
Site Latitude (North)	Decimal Degrees: 34.1418
Site Longitude (West)	Decimal Degrees: 117.2147
Date Drilled	1/4/2021
Hammer Weight (lbs)	140
Hammer Drop (inches)	30
Hammer Efficiency (E_M)	72
Borehole Correction (Cb)*	1
Sampler Correction Mod Cal to SPT	0.63
Sampler Liner Correction (Cs)	1.2 Applied if SPT Sampler Used; 1.0 Applied if Cal Sampler Used
Rod Length Above Ground (ft)	3
Depth to Estimate Vs Over (ft)*	100
*N_{sub} Value Desired For Column 6	70

*inside diameter of Hollow Stem Auger

*Caltrans Estimation Method

*Only Used for Calculating Nsub otherwise not used by program (i.e.N50, N70, N80, etc)

Calculation Results	
Ave. SPT N60HE-value (blows/ft)	17 (Based on Upper 50 feet)
Ave. Shear Wave Velocity (ft/sec)	798 (Based on Upper 50 feet)
Soil Profile Type (Site Class)	D (Based on Upper 50 feet)
Ave. Friction Angle (degrees)	33 (Based on Upper 50 feet)
Estimated Shear Wave Velocity ** Based on Depth Less than 100' ft	797 (ft/sec Upper 100 feet)
Soil Profile Type (Site Class)**	D Based on Ave. Shear Wave Velocity (ft/sec) 243 (m/sec Upper 100 feet)
Ave. Field SPT N-value (blows/ft)	14.1 (Based on Upper 50 feet)
Ave. Field SPT N-value (blows/ft)	21.6 (Based on Upper 100 feet)
Soil Profile Type (Site Class)**	D Based on Ave. Field Blow Count 22 (Upper 100 feet)

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

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

1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
Bottom of Layer Depth (ft)	Blow Count***	Type of Sampler	d_i (feet)	N_{60} (blows/ft)	N70 (blows/ft)	N_{60HE} (blows/ft)	V_{si}** (m/sec)	V_{si} (ft/sec)	Φ_i (degrees)	d_i/N_{60HEi}	d_i/V_{si}	d_i/Φ_i	Consistency if Coarse Grained (Based on ASTM and Corrected for N60)	Consistency if Fine Grained (Based on ASTM and Corrected for N60)
2.5	6	c	2.5	3.40	2.92	4.54	155.81	511.06	26.46	0.55115	0.00489	0.094498	Very Loose	Soft
5.0	6	c	2.5	3.40	2.92	4.54	155.81	511.06	26.46	0.55115	0.00489	0.094498	Very Loose	Soft
7.5	19	c	2.5	10.77	9.23	14.36	217.66	713.92	30.74	0.17405	0.00350	0.081318	Medium Dense	Stiff
10.0	32	c	2.5	18.14	15.55	24.19	253.18	830.43	33.20	0.10334	0.00301	0.075312	Medium Dense	Very Stiff
15.0	19	c	5.0	12.21	10.47	14.36	217.66	713.92	30.74	0.34809	0.00700	0.162635	Medium Dense	Stiff
20.0	24	c	5.0	17.24	14.77	18.14	232.92	763.96	31.80	0.27557	0.00654	0.157244	Medium Dense	Very Stiff
25.0	25	c	5.0	17.96	15.39	18.90	235.69	773.06	31.99	0.26455	0.00647	0.156303	Medium Dense	Very Stiff
30.0	31	c	5.0	23.44	20.09	23.44	250.86	822.82	33.04	0.21335	0.00608	0.151353	Medium Dense	Very Stiff
35.0	48	c	5.0	36.29	31.10	36.29	284.77	934.05	35.37	0.13779	0.00535	0.141369	Dense	Hard
40.0	23	s	5.0	33.12	28.39	27.60	263.04	862.79	33.87	0.18116	0.00580	0.147604	Dense	Hard
45.0	44	s	5.0	63.36	54.31	52.80	317.49	1041.37	37.61	0.09470	0.00480	0.13293	Very Dense	Hard
50.0	62	s	5.0	89.28	76.53	74.40	350.69	1150.26	39.89	0.06720	0.00435	0.125352	Very Dense	Hard
		Total:	50.0	"d" Feet			Total:			2.96209	0.06269	1.520414		

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

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

Consistency classification based upon ASCE 1996

BOREHOLE, SAMPLER, AND ROD CORRECTION FACTORS

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

Adapted from Skempton (1986).

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

DESCRIPTIVE SOIL CLASSIFICATION

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

SOIL GRAIN SIZE

U.S. STANDARD SIEVE

12” 3” 3/4” 4 10 40 200

BOULDERS	COBBLES	GRAVEL		SAND			SILT	CLAY
		COARSE	FINE	COARSE	MEDIUM	FINE		

305 76.2 19.1 4.76 2.00 0.42 0.074 0.002

SOIL GRAIN SIZE IN MILLIMETERS

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

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

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

CONSISTENCY OF COHESIVE SOILS (CLAY OR CLAYEY SOILS)

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

MOISTURE DENSITY

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

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

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

MOISTURE CONDITION

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

RELATIVE PROPORTIONS

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

PLASTICITY

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

LOG KEY SYMBOLS

Bulk, Bag or Grab Sample

Standard Penetration Split Spoon Sampler (2” outside diameter)

Modified California Sampler (3” outside diameter)

No Recovery

GROUNDWATER LEVEL

Water Level (measured or after drilling)

Water Level (during drilling)

Terms and Symbols Used on Boring Logs

Earth Systems

MAJOR DIVISIONS			GRAPHIC SYMBOL	LETTER SYMBOL	TYPICAL DESCRIPTIONS
COARSE GRAINED SOILS More than 50% of material is 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

Earth Systems

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

Boring No. B-1

Project Name: Patton Waterline Replacement
Project Number 304308-001
Boring Location: See Plate 2

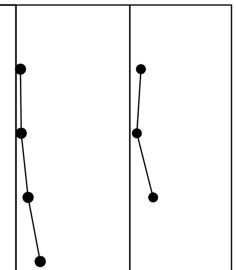
Drilling Date: January 4, 2021
Drilling Method: Mobile B-61 w/autohammer
Drill Type: 8" HSA
Logged By: R. Howe

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: dark brown, very loose, damp, fine grained sand, trace clay, low plasticity, gravel surface
2.5	MOD Calif.	5,3,3			111	9	
5	MOD Calif.	3,3,4			107	10	moist, slight plasticity
7.5	MOD Calif.	4,7,9			123	10	loose
10	MOD Calif.	10,15,17					non plastic

Graphic Trend
Blow Count Dry Density



Total Depth 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-2

Project Name: Patton Waterline Replacement
Project Number 304308-001
Boring Location: See Plate 2

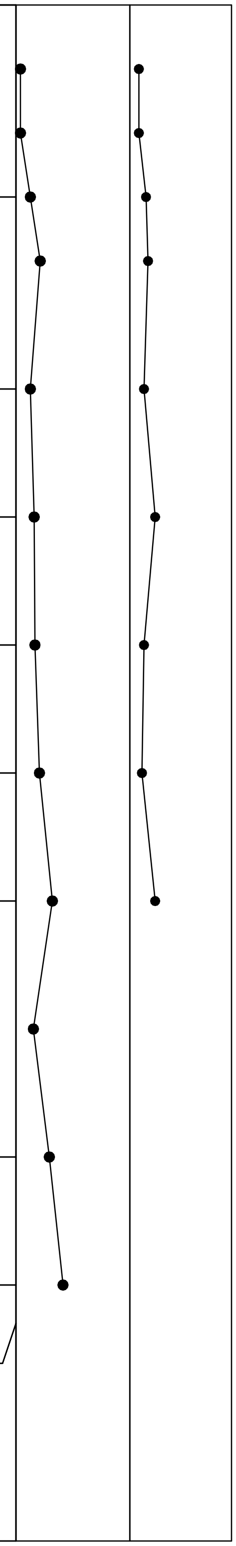
Drilling Date: January 4, 2021
Drilling Method: Mobile B-61 w/autohammer
Drill Type: 8" HSA
Logged By: R. Howe

Description of Units

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

Depth (Ft.)	Sample Type (Bulk / SPT / MOD Calif.)	Penetration Resistance (Blows/6")	USCS	Dry Density (pcf)	Moisture Content (%)	Description of Units
0	Bulk (0–4.5)		SM			SILTY SAND: dark brown, very loose, damp, fine to coarse grained sand, trace fine gravel, trace clay
2.5	MOD Calif.	3,3,3		109	6	
5	MOD Calif.	3,3,3		109	6	
7.5	MOD Calif.	3,8,11	SM	116	10	SILTY SAND: dark brown, medium dense, damp, fine grained sand, trace clay, slight plasticity
10	MOD Calif.	9,14,18		118	9	
15	MOD Calif.	5,8,11	ML	114	10	SILT: reddish brown, stiff, damp, trace fine grained sand, trace clay, slight plasticity
20	MOD Calif.	6,11,13	ML	125	9	SILT: brown, very stiff, damp, trace fine grained sand and clay, slight plasticity
25	MOD Calif.	7,10,15	SP-SM	114	5	POORLY GRADED SAND WITH SILT: light brown, medium dense, damp, fine grained sand
30	MOD Calif.	9,12,19	SM	112	4	SILTY SAND: light brown, medium dense, damp, fine grained sand, trace coarse sand, some fine gravel
35	MOD Calif.	10,24,24	SP-SM	125	7	POORLY GRADED SAND WITH SILT: red brown, dense, damp, fine to medium grained sand, trace fine gravel
40	SPT	10,11,12				
45	SPT	12,15,29	SP-SM			POORLY GRADED SAND WITH SILT: light brown, very dense, damp, fine to medium grained sand, trace fine gravel
50	SPT	22,22,40	SW-SM			WELL GRADED SAND WITH SILT: gray to brown, very dense, damp, fine to coarse grained sand

Graphic Trend
Blow Count Dry Density



Total Depth 51-1/2 feet
Backfilled with cuttings
No groundwater encountered

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

Boring No. B-3

Project Name: Patton Waterline Replacement
Project Number 304308-001
Boring Location: See Plate 2

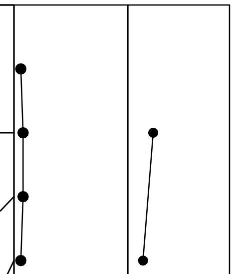
Drilling Date: January 4, 2021
Drilling Method: Mobile B-61 w/autohammer
Drill Type: 8" HSA
Logged By: R. Howe

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			ML			SANDY SILT: dark brown, firm, moist, trace fine grained sand, trace clay, slight plasticity
2.5	MOD Calif.	6,5,4				
5	MOD Calif.	4,6,6	ML	125	10	CLAYEY SILT: light brown, firm, damp, trace fine grained sand, medium plasticity
7.5	MOD Calif.	5,5,7	SM			SILTY SAND: red brown, loose, damp, fine to medium grained sand, trace CaCo3 granules
10	MOD Calif.	4,4,5	CL	115	11	SILTY CLAY: red brown, firm, moist, trace fine grained sand, medium plasticity

Graphic Trend
Blow Count Dry Density



Total Depth 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: Patton Waterline Replacement
Project Number 304308-001
Boring Location: See Plate 2

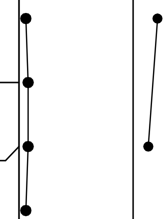
Drilling Date: January 4, 2021
Drilling Method: Mobile B-61 w/autohammer
Drill Type: 8" HSA
Logged By: R. Howe

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			ML			SANDY CLAYEY SILT: reddish brown, stiff, damp, fine grained sand, low plasticity
	MOD Calif.	6,7,8		124	9	
5	MOD Calif.	6,6,7	SM			SILTY SAND: brown, loose, moist, fine grained sand, trace CaCo3 granules
	MOD Calif.	5,5,8	CL	115	12	SILTY CLAY: brown, firm, damp, trace fine grained sand, CaCo3 granules
10	MOD Calif.	4,5,6				

Graphic Trend
Blow Count Dry Density



Total Depth 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-5
Project Name: Patton Waterline Replacement
Project Number 304308-001
Boring Location: See Plate 2

Drilling Date: January 4, 2021
Drilling Method: Mobile B-61 w/autohammer
Drill Type: 8" HSA
Logged By: R. Howe

Description of Units

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

Depth (Ft.)	Sample Type (Bulk / SPT / MOD Calif.)	Penetration Resistance (Blows/6")	Symbol	USCS	Dry Density (pcf)	Moisture Content (%)	Description of Units
0	Bulk			SM			SILTY SAND: brown, loose, damp, fine to coarse grained sand, low plasticity, CaCo3 granules
	MOD Calif.	6,5,6			120	8	
5	MOD Calif.	4,6,6					
	MOD Calif.	5,5,6		CL	115	9	SANDY SILTY CLAY: brown, firm, damp, fine grained sand, low plasticity
10	MOD Calif.	5,5,6		SP-SM			POORLY GRADED SAND WITH SILT: brown, loose, damp, fine to medium grained sand

Graphic Trend
Blow Count Dry Density

Total Depth 11-1/2 feet
Backfilled with cuttings
No groundwater encountered

Photos of Drainage Channel

Notes: The ditch is in poor shape and has debris as well as what appears to be makeshift erosion control. In one area there appears to be a buried utility. It is unclear if this is an actual utility or debris, but it appeared more like it might actually be a utility. It appeared to be cement coated, possible asbestos cement. In the South Side Close up bottom left corner you can see the pipe as well as in the AC Pipe photo. Oversize rock material is visible intermixed in the soils and debris.

North Side of Crossing

South Side of Crossing


South Side Close-Up


AC Pipe

Crossing

North Side Looking at Crossing

APPENDIX B

Laboratory Test Results

File No.: 304308-001 January 30, 2021
Lab No.: 21-002

UNIT DENSITIES AND MOISTURE CONTENT ASTM D2937 & D2216

Job Name: Patton Waterline Replacement

Sample Location	Depth (feet)	Unit Dry Density (pcf)	Moisture Content (%)	USCS Group Symbol
B1	2.5	111	9	SM
B1	5	107	10	SM
B1	7.5	123	10	SM
B2	2.5	109	6	SM
B2	5	109	6	SM
B2	7.5	116	10	SM
B2	10	118	9	SM
B2	15	114	10	ML
B2	20	125	9	ML
B2	25	114	5	SP-SM
B2	30	112	4	SM
B2	35	125	7	SP-SM
B3	5	125	10	ML
B3	10	115	11	CL
B4	2.5	124	9	ML
B4	7.5	115	12	CL
B5	2.5	120	8	SM
B5	7.5	115	9	CL

File No.: 304308-001 1/30/2021

Lab No.: 21-002

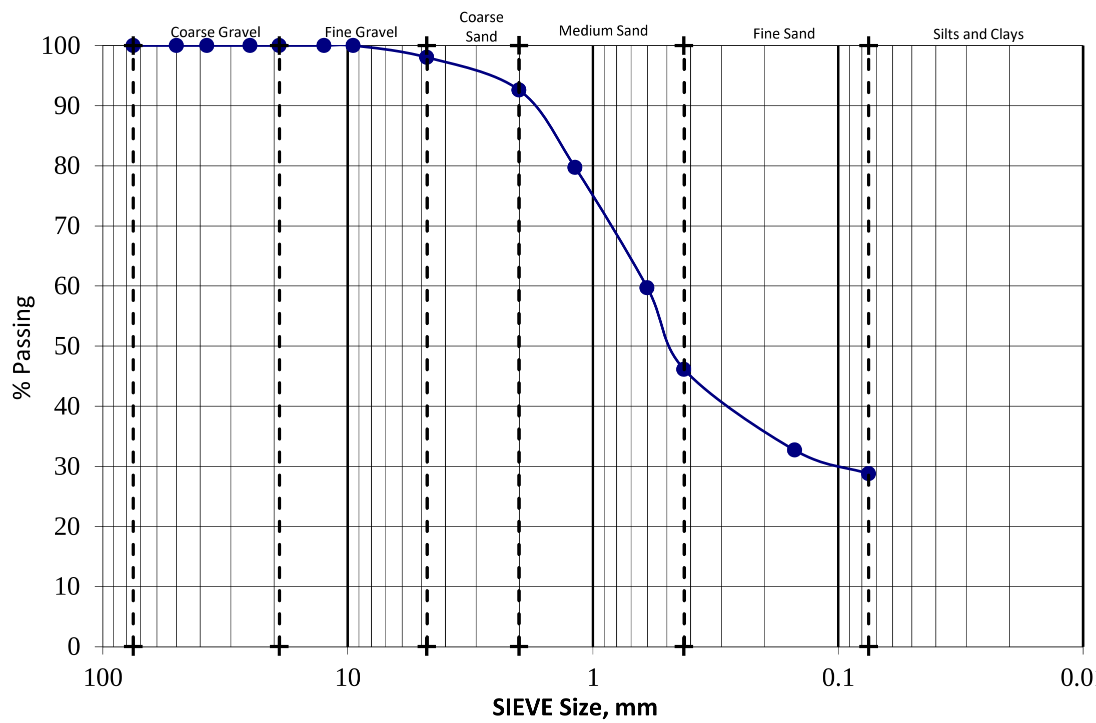
SIEVE ANALYSIS ASTM D6913

Job Name: Patton Waterline Replacement

Sample ID: B3 @ 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	98
#10	93
#16	80
#30	60
#40	46
#100	33
#200	28.8



% Coarse Gravel:	0	% Coarse Sand:	5			
% Fine Gravel:	2	% Medium Sand:	46	Cu:	NA	
		% Fine Sand:	17	Cc:	NA	Gradation
% Total Gravel	2	% Total Sand	69	% Fines:	28.8	NA

File No.: 304308-001 1/30/2021

Lab No.: 21-002

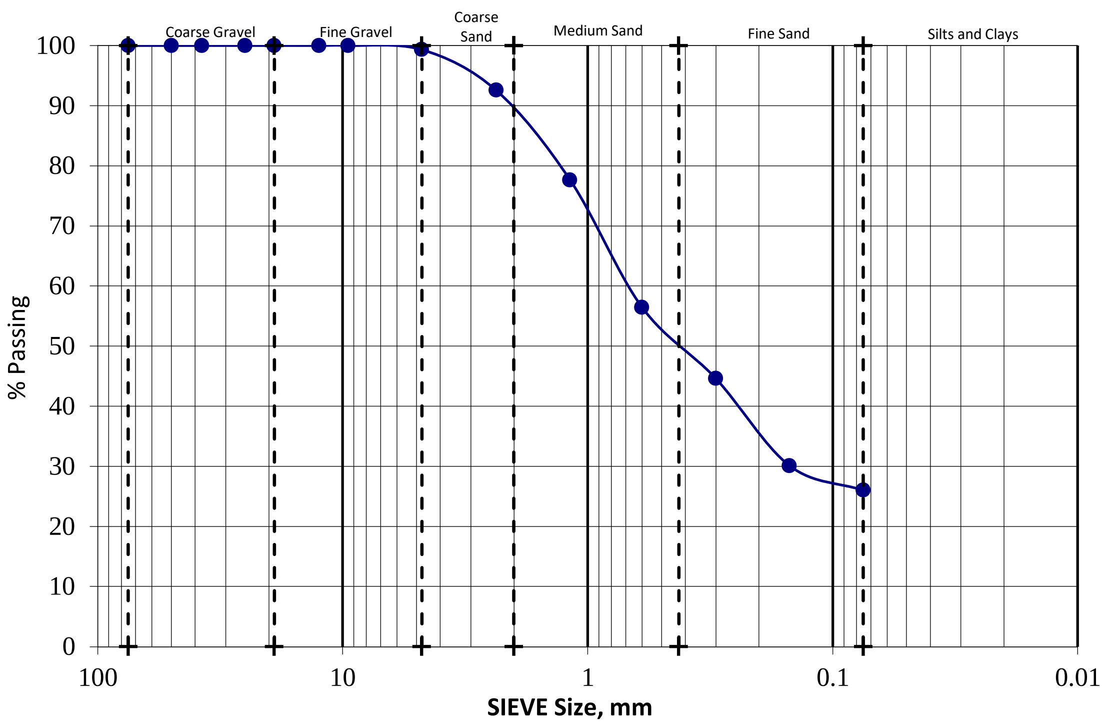
SIEVE ANALYSIS ASTM D6913

Job Name: Patton Waterline Replacement

Sample ID: B5 @ 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	99
#8	93
#16	78
#30	56
#50	45
#100	30
#200	26.0



% Coarse Gravel:	0	% Coarse Sand:	11				
% Fine Gravel:	1	% Medium Sand:	38	Cu:	NA		
		% Fine Sand:	24	Cc:	NA	Gradation	
% Total Gravel	1	% Total Sand	73	% Fines:	26.0	NA	

File No.: 304308-001 January 30, 2021

Job Name: Patton Waterline Replacement

Lab Number: 21-002

ASTM D-1140 or Earth Systems Method (circle one)

AMOUNT PASSING NO. 200 SIEVE (Earth Systems Method Transfers Sample until water runs clear)

Sample Location	Depth (feet)	Fines Content (%)	USCS Group Symbol	Soaking Time
B1	2.5	37.4	SM	10
B2	2.5	20.4	SM	10
B2	7.5	36.2	SM	10

File No.: 304308-001
Lab No.: 21-002

January 30, 2021

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

Patton Waterline Replacement
B1 @ 5 feet

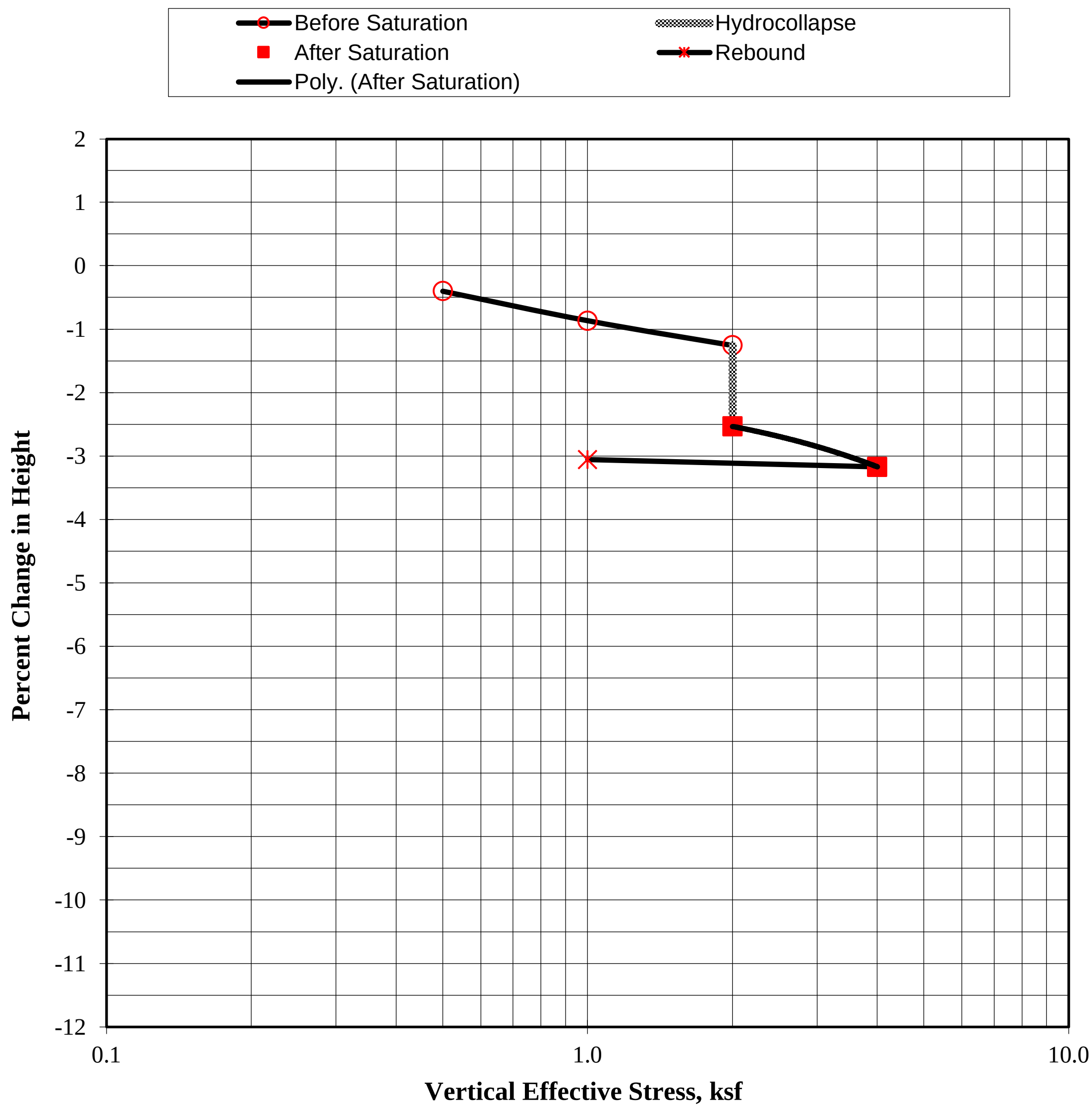
Silty Sand (SM)

Ring Sample

Initial Dry Density: 121.5 pcf
Initial Moisture: 9.5%
Specific Gravity: 2.67
Initial Void Ratio: 0.372

Hydrocollapse: 1.3% @ 2.0 ksf

% Change in Height vs Normal Pressure Diagram



File No.: 304308-001 January 30, 2021
Lab No.: 21-002

CONSOLIDATION TEST ASTM D 2435 & D 5333

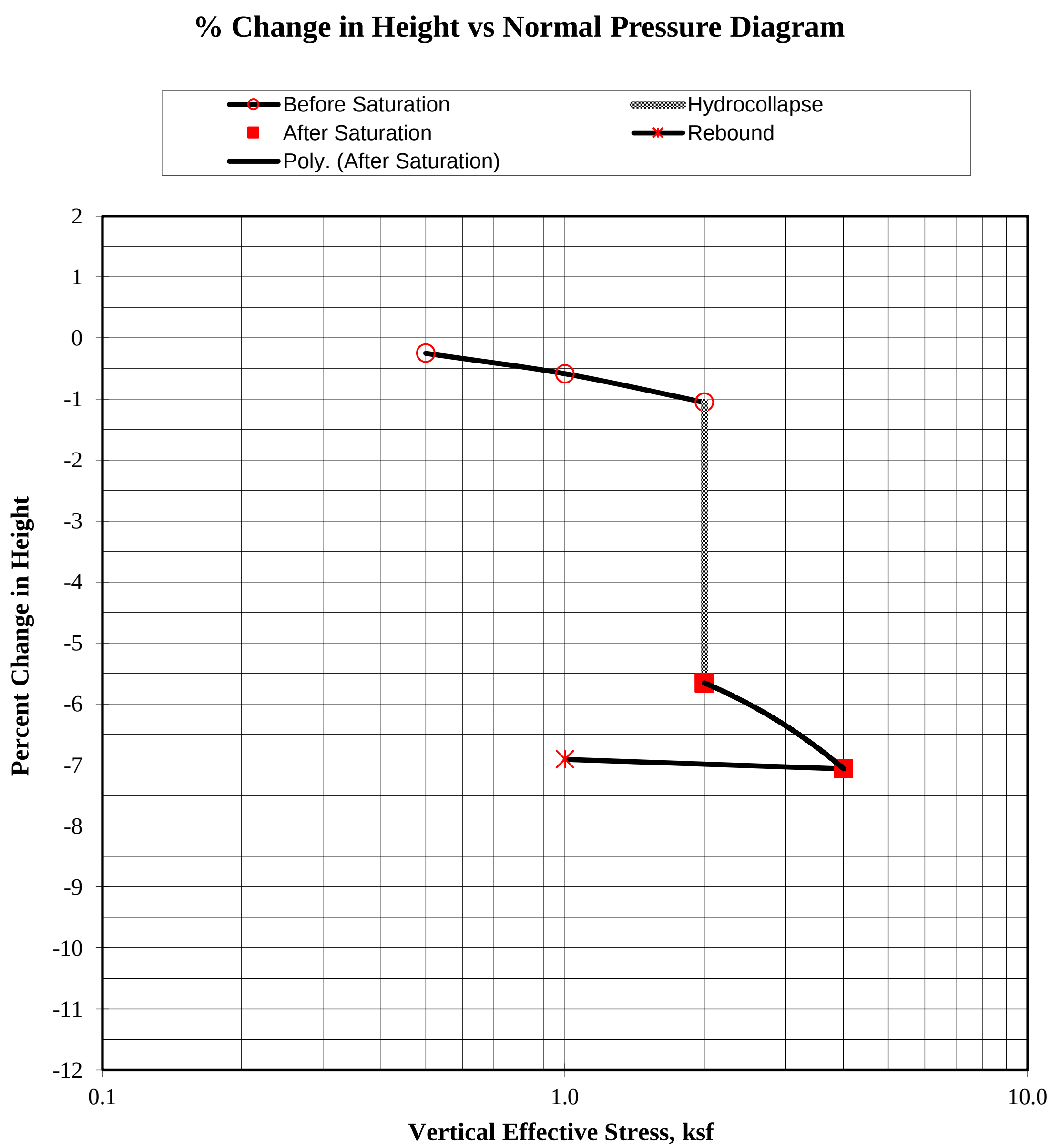
Patton Waterline Replacement
B2 @ 5 feet

Silty Sand (SM)

Ring Sample

Initial Dry Density: 109.4 pcf
Initial Moisture: 12.0%
Specific Gravity: 2.67
Initial Void Ratio: 0.524

Hydrocollapse: 4.6% @ 2.0 ksf



File No.: 304308-001
Lab No.: 21-002

January 30, 2021

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

Patton Waterline Replacement
B2 @ 15 feet

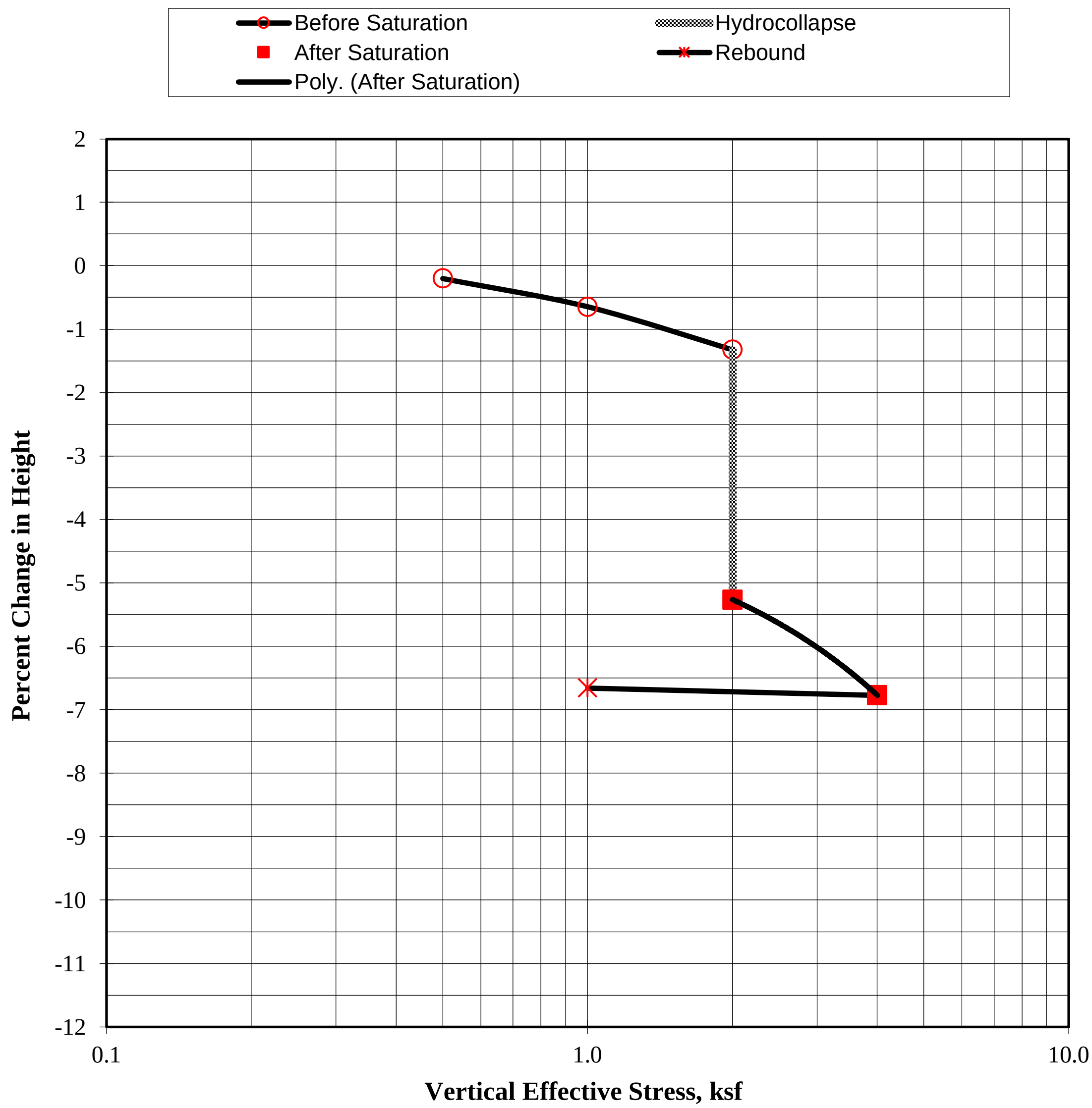
Silt (ML)

Ring Sample

Initial Dry Density: 108.0 pcf
Initial Moisture: 10.8%
Specific Gravity: 2.67
Initial Void Ratio: 0.543

Hydrocollapse: 3.9% @ 2.0 ksf

% Change in Height vs Normal Pressure Diagram



File No.: 304308-001 January 30, 2021
Lab No.: 21-002

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

Patton Waterline Replacement
B3 @ 10 feet

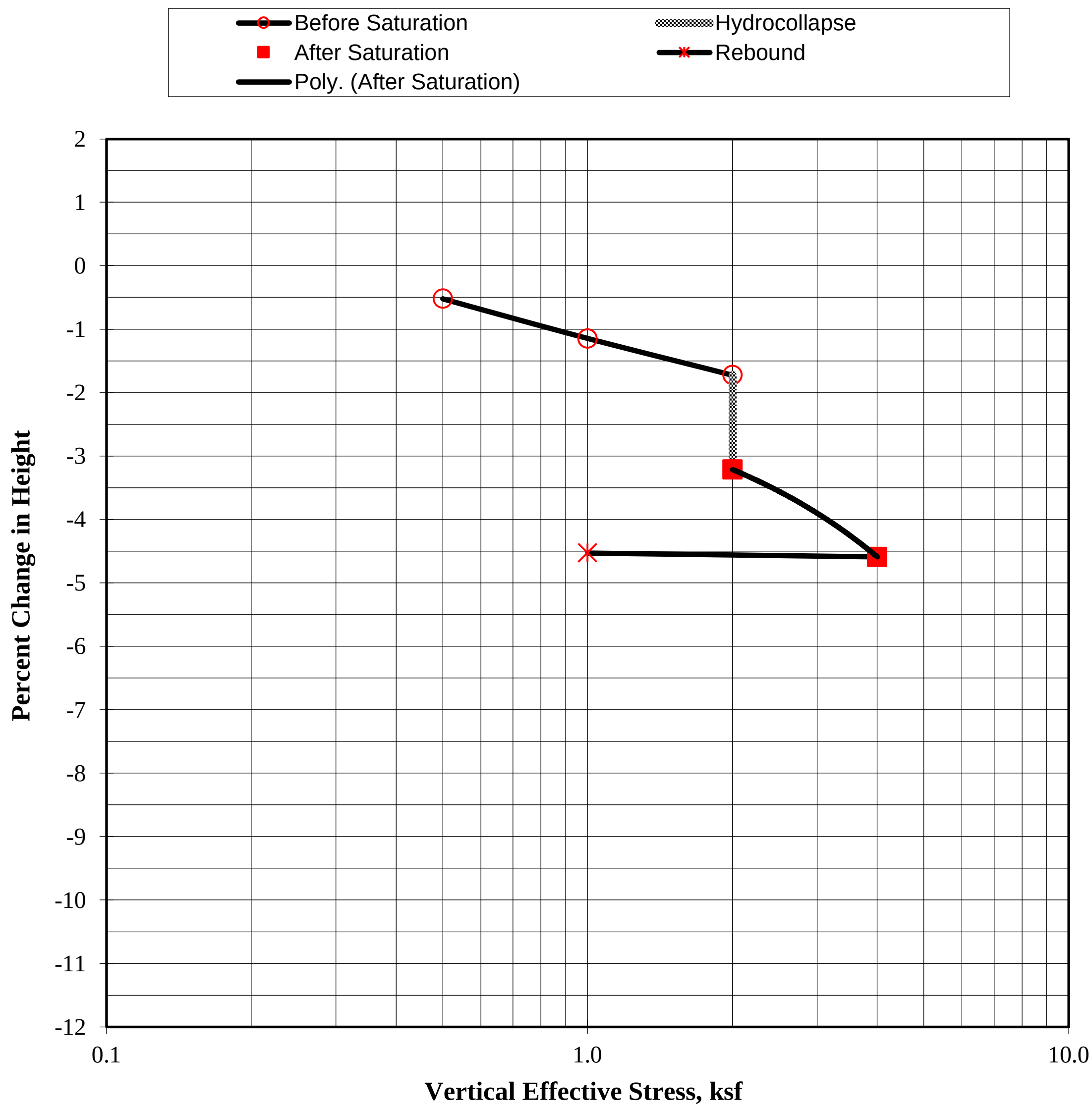
Silty Clay (CL)

Ring Sample

Initial Dry Density: 106.2 pcf
Initial Moisture: 12.8%
Specific Gravity: 2.67
Initial Void Ratio: 0.570

Hydrocollapse: 1.5% @ 2.0 ksf

% Change in Height vs Normal Pressure Diagram



File No.: 304308-001
Lab No.: 21-002

January 30, 2021

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

Patton Waterline Replacement
B5 @ 7 1/2 feet

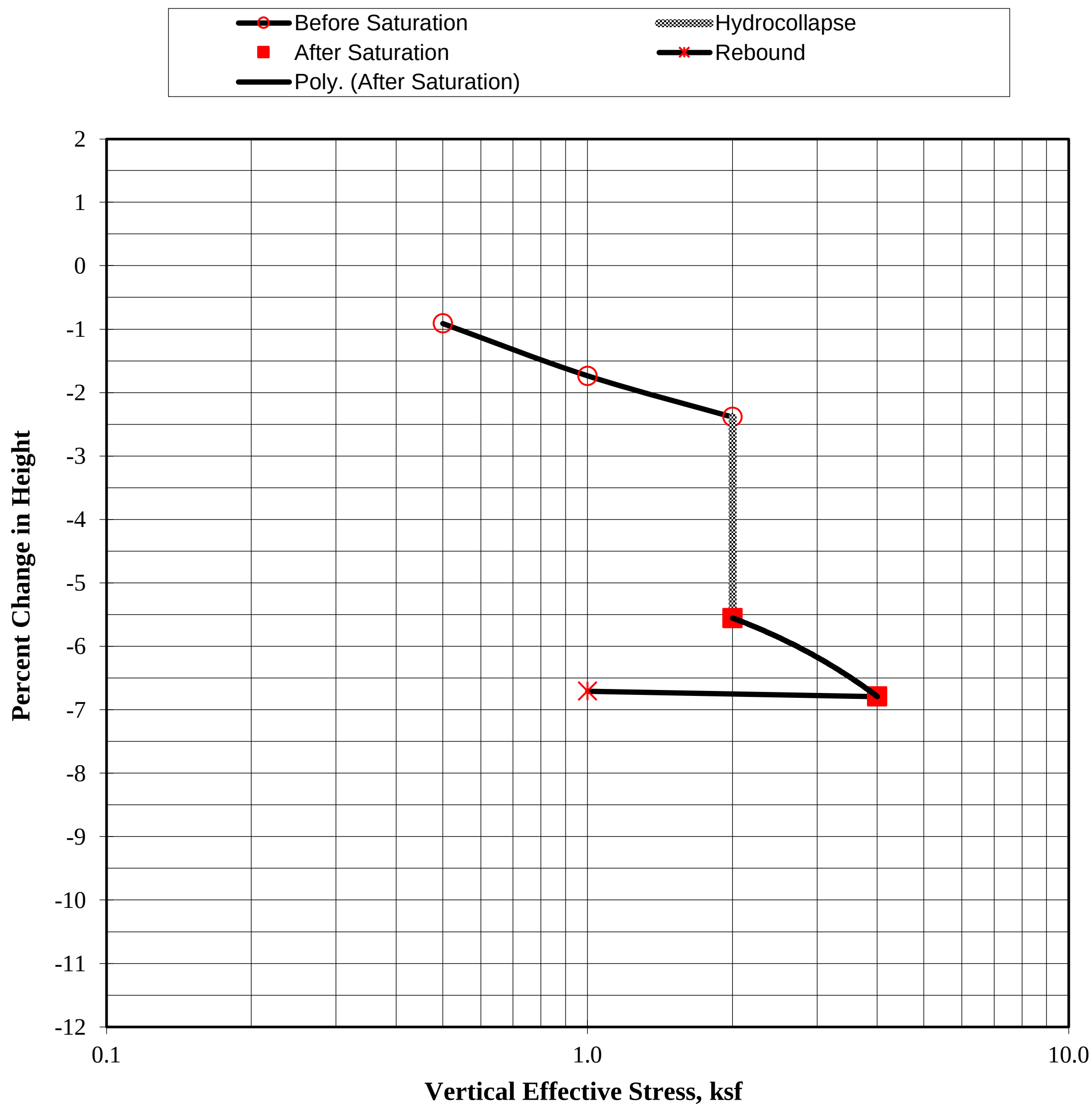
Sandy Silty Clay (CL)

Ring Sample

Initial Dry Density: 111.6 pcf
Initial Moisture: 1.1%
Specific Gravity: 2.67
Initial Void Ratio: 0.494

Hydrocollapse: 3.2% @ 2.0 ksf

% Change in Height vs Normal Pressure Diagram



File No.: 304308-001 January 30, 2021
Lab No.: 21-002

EXPANSION INDEX **ASTM D-4829**

Job Name: Patton Waterline Replacement
Sample ID: B3 @ 5 feet
Soil Description: Clayey Silt (ML)

Initial Moisture, %:	7.0	
Initial Compacted Dry Density, pcf:	121.9	
Initial Saturation, %:	50	*
Final Moisture, %:	11.9	
Volumetric Swell, %:	-0.2	
Expansion Index, EI:	**0**	**Very Low**

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

January 30, 2021

MAXIMUM DRY DENSITY / OPTIMUM MOISTURE

ASTM D 1557 (Modified)

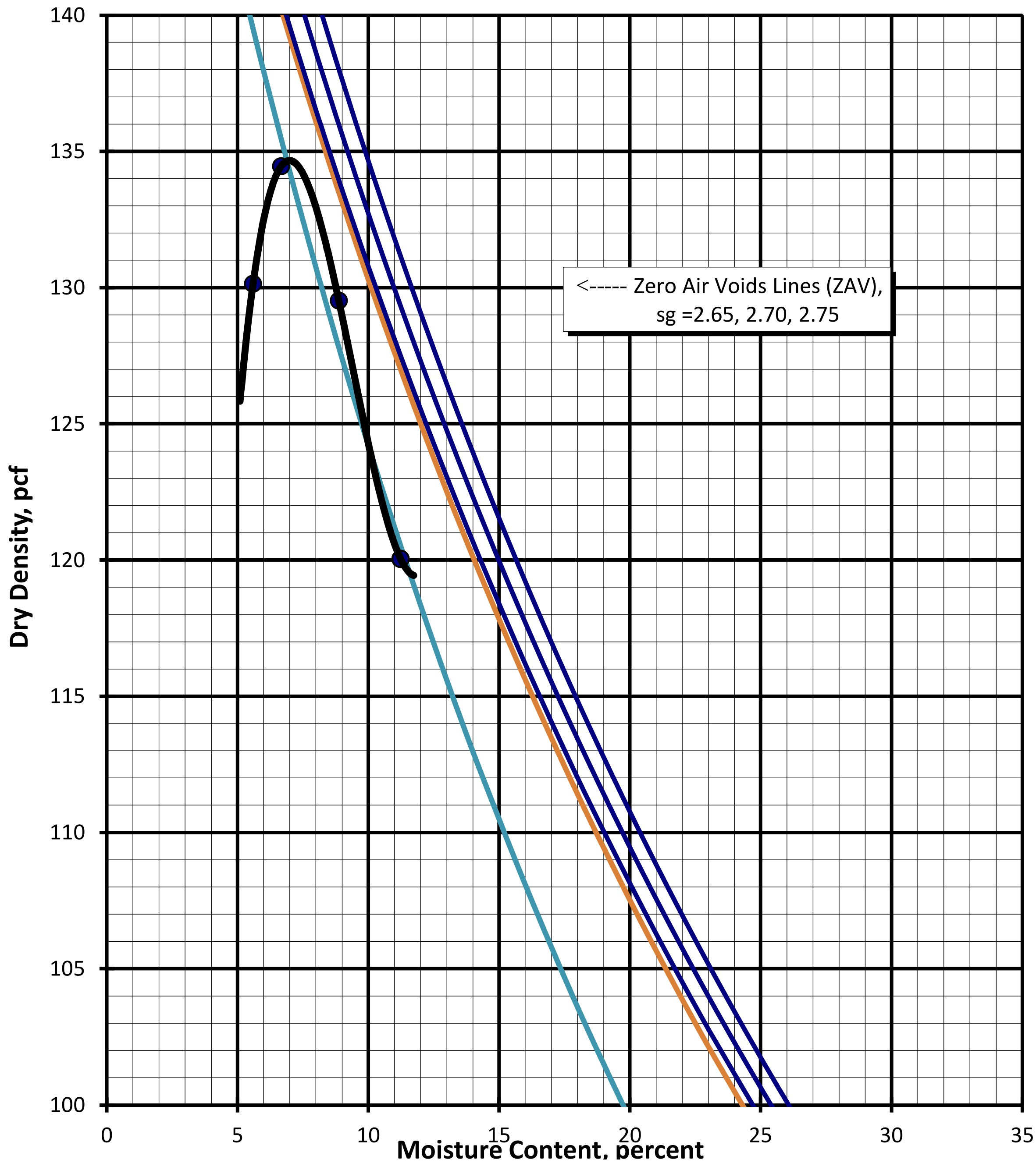
Job Name: Patton Waterline Replacement
Sample ID: B2 @ 0-5 feet
Location:

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

Description: Dark Brown Silty F-C Sand (SM)

Maximum Dry Density: **134.7 pcf**
Optimum Moisture: **7%**

Sieve Size	% Retained (Cumulative)
3/4"	0.2
3/8"	1.6
#4	5.9



SAND EQUIVALENT/DURABILITY INDEX

Cal Test 217

JOB NUMBER:	304308-001	DATE:	1/30/2021
PROJECT NAME:	Patton Waterline Replacement	TECH:	JM
CLIENT:		LAB NUMBER:	21-002

CONTRACTOR:
SUPPLIER:

SAMPLE I.D.: B1 @ 2 1/2 feet
SAMPLE LOCATION:
SAMPLE DESCRIPTION: Silty Sand (SM)

TEST NUMBER:	1	2	3
START SOAK:	12:38:00 PM	#####	12:44:00 PM
START SHAKE:	12:48:00 PM	#####	12:54:00 PM
FINISH IRRIGATION:	12:49:00 PM	#####	12:55:00 PM
FINAL READING:	1:09:00 PM	#####	1:15:00 PM
CLAY READING:	11.1	11.2	11.5
SAND READING:	1.5	1.6	1.6
SAND EQUIVALENT:	14	15	14

SE = Sand Reading/Clay Reading x 100

AVERAGE OF 3 POINTS: **15**

Earth Systems
79-811 Country Club Drive, Suite B | Bermuda Dunes, CA 92203 | Ph: 760.345.1588 | www.earthsystems.com

SAND EQUIVALENT/DURABILITY INDEX

Cal Test 217

JOB NUMBER:	304308-001	DATE:	1/30/2021
PROJECT NAME:	Patton Waterline Replacement	TECH:	AL
CLIENT:		LAB NUMBER:	21-002

CONTRACTOR:

SUPPLIER:

SAMPLE I.D.: B2 @ 0-5 feet

SAMPLE LOCATION:

SAMPLE DESCRIPTION: Silty Sand (SM)

TEST NUMBER:	1	2	3
START SOAK:	1:15:00 PM	########	1:21:00 PM
START SHAKE:	1:25:00 PM	########	1:31:00 PM
FINISH IRRIGATION:	1:26:00 PM	########	1:32:00 PM
FINAL READING:	1:46:00 PM	########	1:52:00 PM
CLAY READING:	13.7	14.0	13.9
SAND READING:	1.5	1.4	1.3
SAND EQUIVALENT:	11	10	10

SE = Sand Reading/Clay Reading x 100

AVERAGE OF 3 POINTS: **11**

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SAND EQUIVALENT/DURABILITY INDEX

Cal Test 217

JOB NUMBER:	304308-001	DATE:	1/30/2021
PROJECT NAME:	Patton Waterline Replacement	TECH:	JM
CLIENT:		LAB NUMBER:	21-002

CONTRACTOR:

SUPPLIER:

SAMPLE I.D.: B5 @ 0-5 feet

SAMPLE LOCATION:

SAMPLE DESCRIPTION: Silty Sand (SM)_

TEST NUMBER:	1	2	3
START SOAK:	1:48:00 PM	#######	1:54:00 PM
START SHAKE:	1:58:00 PM	#######	2:04:00 PM
FINISH IRRIGATION:	1:59:00 PM	#######	2:05:00 PM
FINAL READING:	2:19:00 PM	#######	2:25:00 PM
CLAY READING:	9.9	10.0	9.9
SAND READING:	2.4	2.4	2.4
SAND EQUIVALENT:	25	24	25

SE = Sand Reading/Clay Reading x 100

AVERAGE OF 3 POINTS: **25**

File No.: 304308-001 1/30/2021

Lab No.: 21-002

SOIL CHEMICAL ANALYSES

Job Name:	Patton Waterline Replacement
Job No.:	304308-001
Sample ID:	B5
Sample Location:	0-5
Resistivity (Units)	
as-received (ohm-cm)	15,200
saturated (ohm-cm)	5,600
pH	7.4
Electrical Conductivity (mS/cm)	0.06
Chemical Analyses	
Cations	
calcium Ca^{2+} (mg/kg)	27
magnesium Mg^{2+} (mg/kg)	10
sodium Na^{1+} (mg/kg)	19
potassium K^{1+} (mg/kg)	3.7
ammonium NH41+ (mg/kg)	ND
Anions	
carbonate CO_3^{2-} (mg/kg)	ND
bicarbonate HCO_3^{1-}(mg/kg)	183
fluoride F^{1-} (mg/kg)	2.1
chloride Cl^{1-} (mg/kg)	6.5
sulfate SO_4^{2-} (mg/kg)	12
nitrate NO31- (mg/kg)	26
phosphate PO_4^{3-} (mg/kg)	ND
Other Tests	
sulfide S^{2-} (qual)	na
Redox (mV)	na

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

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

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

General Guidelines for Soil Corrosivity

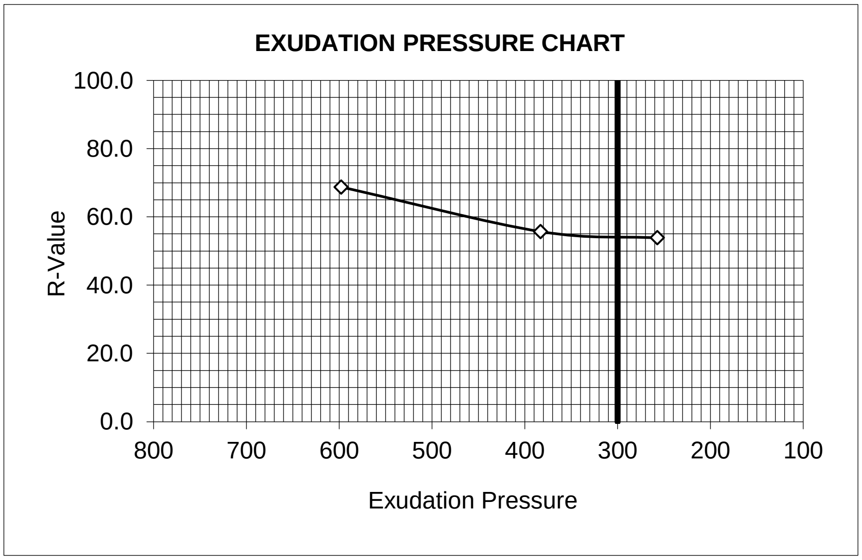
Chemical Agent	Amount in Soil	Degree of Corrosivity
Soluble Sulfates[1]	0 -1,000 mg/Kg (ppm) [0-.1%]	Low
	1,000 - 2,000 mg/Kg (ppm) [0.1-0.2%]	Moderate
	2,000 - 20,000 mg/Kg (ppm) [0.2-2.0%]	Severe
	> 20,000 mg/Kg (ppm) [>2.0%]	Very Severe
Resistivity[2] (Saturated)	0- 900 ohm-cm	Very Severely Corrosive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

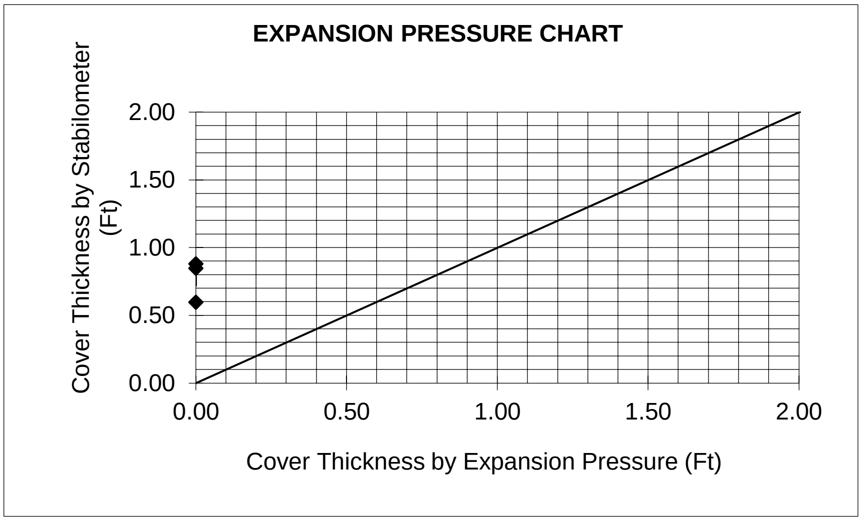
1 - General corrosivity to concrete elements. American Concrete Institute (ACI) Water Soluble Sulfate in Soil by Weight, ACI 318, Tables 4.2.2 - Exposure Conditions and Table 4.3.1 - Requirements for Concrete Exposed to Sulfate-Containing Solutions. *It is recommended that concrete be proportioned in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

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

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

January 30, 2021 **File No.: 304308-001**



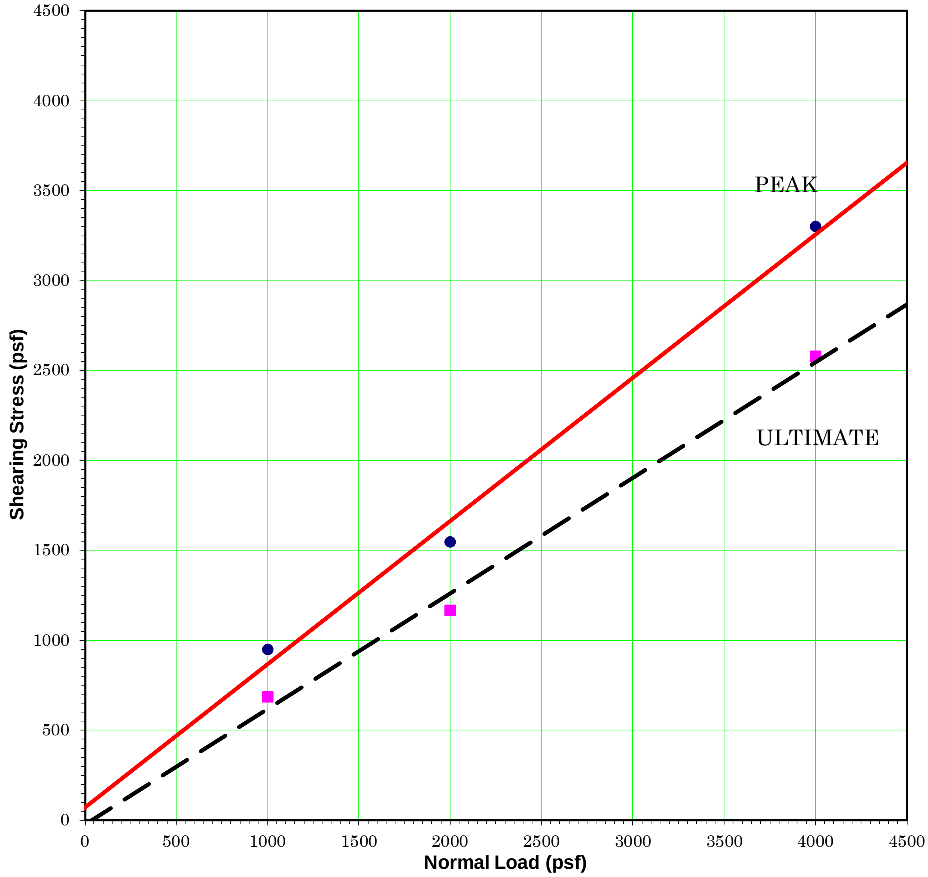


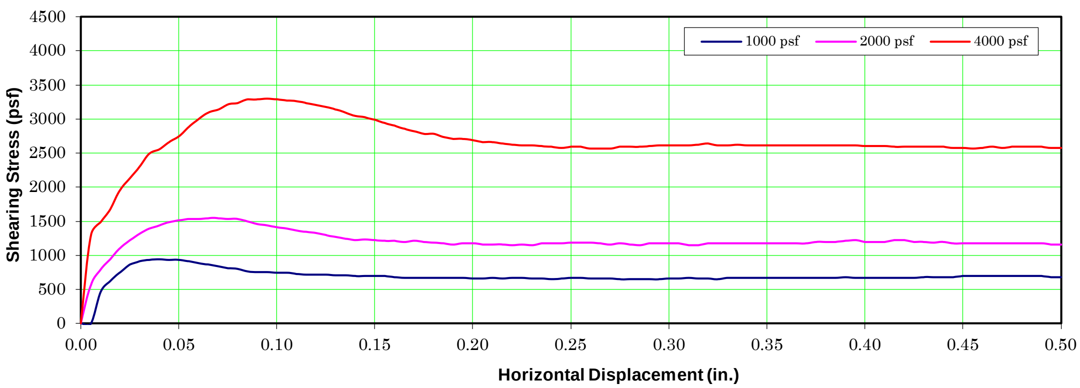
JOB NAME: Patton Waterline Replacement
SAMPLE I. D.: B2 @ 0-5 feet
SOIL DESCRIPTION: Silty Sand (SM)

SPECIMEN NUMBER	D	E	F
EXUDATION PRESSURE	598	383	257
RESISTANCE VALUE	68.8	55.7	53.9
EXPANSION DIAL(0.0001")	0	0	0.0001
EXPANSION PRESSURE (PSF)	0.0	0.0	0.0
% MOISTURE AT TEST	7.0	7.8	8.6
DRY DENSITY AT TEST	128.9	134.2	131.6

R-VALUE @ 300 PSI EXUDATION	**54**
R-VALUE by Expansion Pressure*	**N/A**

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





DIRECT SHEAR DATA*

Sample Location:	B2 @ 0-5 feet		
Material:	Silty Sand (SM)		
Dry Density (pcf):	134.7 Remolded to 90% of Max	121.2	
	Initial	Final	
Moisture Content (%):	7	11.0	
Saturation (%):	79	100	
	Peak	Ultimate	
ϕ Angle of Friction (degrees):	39	33	
c Cohesive Strength (psf):	70	0	
Test Type: Peak and Ulitimate			
Shear Rate (in/min): 0.007			

* Test Method: ASTM D-3080

DIRECT SHEAR TEST

Patton Waterline Replacement

Patton, California 92369

Earth Systems Pacific

1/30/2021	304308-001