Geotechnical Evaluation and Geologic Hazard Assessment

Ukiah New Courthouse Building

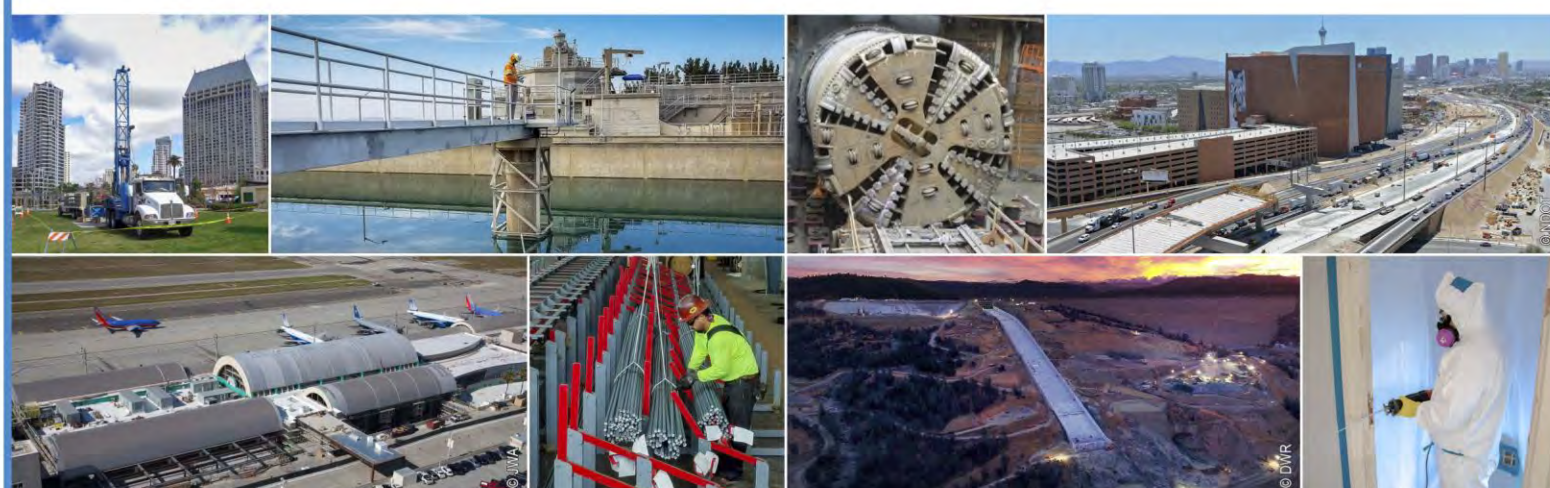
300 East Perkins Street

Ukiah, California

Judicial Council of California

455 Golden Gate Avenue | San Francisco, California 94102

October 12, 2022 | Project No. 404353001



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Geotechnical Evaluation and Geologic Hazard Assessment

Ukiah New Courthouse Building
300 East Perkins Street
Ukiah, California

Facilities Services | Administrative Division
Judicial Council of California
455 Golden Gate Avenue | San Francisco, California 94102

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REGISTERED PROFESSIONAL ENGINEER
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1 INTRODUCTION

In accordance with your authorization, we have performed a geotechnical evaluation and geologic hazard assessment for the proposed Ukiah New Courthouse Building project located at 300 East Perkins Street in Ukiah, California (Figure 1). The scope of our evaluation is conducted for Cannon Design on behalf of Judicial Council of California, and in accordance with our proposal to Cannon Design dated March 3, 2022. The purpose of our study is to evaluate the subsurface conditions for the project and provide recommendations for the design and construction of the proposed improvements.

2 SCOPE OF SERVICES

Our scope of services performed for this study includes the following:

- Review of readily available geologic and seismic literature pertinent to the project area, including geologic maps and reports, regional fault maps, seismic hazard maps, and aerial photography.
- Review Professional Service Industries, Inc.'s 2011 geotechnical investigation report at the project site and incorporate their findings in this geotechnical evaluation.
- Site reconnaissance to observe the general site conditions and to mark the locations for our subsurface exploration.
- Coordination with Underground Service Alert (USA) to locate underground utilities in the vicinity of our subsurface exploration.
- A private underground utility survey to further check the exploration locations for underground utility conflicts.
- Subsurface evaluation consisting of drilling, logging, and sampling of four (4) hollow stem auger borings advanced to depths of up to 51.5 feet.
- Performance of a geophysical Refraction Microtremor (ReMi) survey to evaluate subsurface variations in shear wave velocity for seismic site classification.
- Laboratory testing on selected samples to evaluate in-situ soil moisture content and dry density, percent passing #200 sieve, grain size distribution, Atterberg limits, expansion index, unconfined compression strength, consolidation characteristics, R-value, and soil corrosivity.
- Compilation and engineering analysis of the field and laboratory data, and the findings from our background review, subsurface evaluation, and laboratory testing.
- Preparation of this geotechnical evaluation and geologic hazards assessment report presenting our findings and conclusions regarding the subsurface conditions encountered at the project site, and our geotechnical recommendations for the design and construction of the proposed improvements.

3 SITE AND PROJECT DESCRIPTION

The proposed Ukiah New Courthouse Building is located at 300 East Perkins Street in Ukiah, California (Figure 1). The project site is bounded by East Perkins Street to the north, residential units to the south, commercial warehouses and buildings to the west, and Leslie Street to the east (Figures 1 and 2). In addition, a Union Pacific Rail Road (UPRR) track was observed extending in a north-south direction along the western periphery of the site. The ground surface across the site is relatively flat with elevations varying from about 607 to 612 feet above mean sea level [MSL] (Google, 2022).

Based on the information provided to us, we understand that the project will consist of a new 3-story courthouse building encompassing 82,000 square feet (Figure 2). We understand the building will be supported on continuous perimeter foundation and isolated spread footings. Other associated improvements include surface parking at the northern and southern portions of the site, and improvements for adjacent roadways (Courthouse Road and Clay Street).

4 FIELD EXPLORATION AND LABORATORY TESTING

4.1 Field Exploration

Our field exploration conducted for this study includes a site reconnaissance and subsurface explorations at the proposed site. The subsurface exploration, conducted on September 13 and 14, 2022, consists of four (4) hollow-stem auger borings (B-1 through B-4), shown on Figure 2. Prior to drilling, our scope includes notifying USA North 811 for field marking of the existing utilities and contracting a private utility survey to further assess and locate any utilities that may conflict with the exploration locations.

The borings extend to depths of up to about 51.5 feet below existing grade. Logging of the subsurface conditions exposed in the borings and collecting disturbed, relatively undisturbed and bulk soil samples, as performed by a representative of Ninyo & Moore, are summarized on the boring logs. The materials encountered in the borings are classified and logged in accordance with the Unified Soil Classification System (USCS). The collected soil samples have been transported to our laboratory, and geotechnical laboratory testing has been performed on selected samples. The hollow-stem auger borings have been backfilled with grout after completion of drilling. Detailed logs of the borings and sampling procedures are presented in Appendix A.

Data from a previous geotechnical evaluation performed at the subject site in 2011 by Professional Service Industries, Inc., including boring logs and laboratory test results, are incorporated in our current evaluation to the extent possible. The pertinent boring logs and laboratory test results are presented in Appendix F.

4.2 Laboratory Testing

Geotechnical laboratory testing of soil samples recovered from the borings include tests to evaluate in-situ soil moisture content and dry density, percent passing #200 sieve, grain size distribution, Atterberg limits, expansion index, consolidation characteristics, unconfined compression strength, R-value, and soil corrosivity. The results of the in-place moisture content and dry density tests are shown at the corresponding sample depths on the boring logs in Appendix A. The results of the laboratory tests performed are presented in Appendix B and Appendix C.

4.3 Geophysical Survey

Geophysical Refraction Microtremor (ReMi) survey has been performed at the site on September 14, 2022. The purpose of the study is to evaluate the subsurface shear-wave velocity at the site in order to select the appropriate seismic site class. The ReMi survey uses the passive seismic method of Microtremor Array Measurements (MAM) and consists of one linear profile of seismic data collection at the project site at the location shown on Figure 2. The method provides a shear wave velocity model to a depth of approximately 100 feet which is then used to calculate the average shear velocity (V_{s100}) to select the seismic site class. The seismic model results are provided in Appendix D.

5 GEOLOGIC AND SUBSURFACE CONDITIONS

Our findings regarding regional geologic setting, site geology, subsurface stratigraphy, and groundwater conditions at the subject site are provided in the following sections.

5.1 Regional Geologic Setting

The project is located in Ukiah valley (about 3 miles southwest of Mendocino Lake), which is in-turn located in the Coast Ranges geomorphic province of California. The Coast Ranges are comprised of several mountain ranges and structural valleys formed by tectonic processes commonly found around the Circum-Pacific belt. Basement rocks have been sheared, faulted, metamorphosed, and uplifted, and are separated by thick blankets of Cretaceous and Cenozoic sediments that fill structural valleys. The Coast Ranges has several ranges that trend northwest, parallel to major strike-slip faults such as the San Andreas, Hayward, and Calaveras (Figure 3). Additionally, the Maacama fault zone closest to the project site consists of right-lateral faults, and "downslope movement of unstable terrain has produced topographic features sometimes confused with fault-produced topography, while obliterating some features actually created as the result of fault movements” (CDMG, 1981).

5.2 Site Geology

Regional geologic map by Jennings and Strand indicates that the site is underlain by Holocene age deposits, which typically consist of alluvial gravel, sand, and clay. A map of the regional geology is presented as Figure 4 (Jennings and Strand, 1960).

5.3 Subsurface Conditions

The following sections provide a generalized description of the materials encountered during our subsurface exploration at the project site. More detailed descriptions are presented on the boring logs in Appendix A.

5.3.1 Alluvium

Alluvium encountered in Borings B-1 through B-4 to the total depths explored, generally consists of brown to dark gray, moist to wet, stiff to hard, sandy silt, lean clay and silty clay; brown, moist to wet, medium dense to very dense, poorly graded sand, well-graded sand, silty and clayey sand; and brown, moist to wet, loose to very dense, poorly graded gravel, well-graded gravel, silty and clayey gravel.

5.4 Groundwater

Groundwater is logged in our borings at a depth as shallow as about 25 feet at the subject site. Review of the previous study at the site (PSI, 2011) indicate that groundwater was encountered at a depth as shallow as about 5 feet below grade.

Fluctuations in the groundwater level across the site and over time may occur due to seasonal precipitation, variations in topography or subsurface hydrogeologic conditions, or as a result of changes to nearby irrigation practices or groundwater pumping. In addition, seeps may be encountered at elevations above the observed groundwater levels due to perched groundwater conditions, leaking pipes, preferential drainage, or other factors not evident at the time of our exploration.

6 GEOLOGIC HAZARDS AND CONSIDERATIONS

This study considers a number of issues relevant to the proposed construction, including seismic hazards, landsliding and slope stability, regional land subsidence, flooding and dam inundation, static settlement, expansive soils, corrosive soils, and excavation characteristics. These issues are discussed in the following subsections.

6.1 Seismic Hazards

The seismic hazards considered in this study include the potential for ground rupture due to faulting, seismic ground shaking, liquefaction, and dynamic settlement. These potential hazards are discussed in the following subsections.

6.1.1 Historical Seismicity

The site is located in a seismically active region. Figure 3 presents the location of the site relative to the epicenters of historic earthquakes with magnitudes of 5.0 or more from 1800 to 2000. The Maacama fault zone is located about 1¼ miles northeast of the project site.

6.1.2 Faulting and Ground Surface Rupture

Fault mapping by California Geological Survey (formerly California Division of Mines and Geology) indicates the site is not located within an Alquist-Priolo Earthquake Fault Zone established by the State Geologist (CDMG, 1982) to delineate regions of potential ground surface rupture adjacent to active faults. As defined by the California Geological Survey (CGS), active faults are faults that have caused surface displacement within Holocene time, or within approximately the last 11,700 years (CGS, 2007 and 2018). The closest fault rupture hazard zone is the Maacama fault zone (RGH, 2001) which is located about 1¼ miles northeast of the site (Figure 5).

Based on our review of the referenced seismic hazard and geologic maps, known active faults are not mapped on the site and the site is not located within a fault-rupture hazard zone. Therefore, the probability of damage from surface fault rupture is considered to be low.

6.1.3 Site-Specific Ground Motion Hazard Analysis

Considering the proximity of the site to active faults capable of producing a maximum moment magnitude of 6.0 or more, the project area has a high potential for experiencing strong ground motion. The 2019 California Building Code (CBC) specifies that the risk-targeted maximum considered earthquake (MCE_R) ground motion response accelerations be used to evaluate seismic loads for design of buildings and other structures. Per the 2019 CBC, a site-specific ground motion hazard analysis shall be performed for structures on Site Class D with a mapped MCE_R, 5 percent damped, spectral response acceleration parameter at a period of 1 second (S_1) greater than or equal to 0.2g in accordance with Sections 21.2 and 21.3 of the American Society of Civil Engineers (ASCE) Publication 7-16 (2016) for the Minimum Design

Loads and Associated Criteria for Building and Other Structures. Our calculates indicate that the S_1 for the site is equal to 0.763g using the 2022 Structural Engineers Association of California [SEAOC]/Office of Statewide Health Planning and Development [OSHPD] seismic design tool (web-based); therefore, we have performed a site-specific ground motion hazard analysis for the project area.

The site-specific ground motion hazard analysis consists of the review of available seismologic information for nearby faults and performance of probabilistic seismic hazard analysis (PSHA) and deterministic seismic hazard analysis (DSHA) to develop acceleration response spectrum (ARS) curves corresponding to the MCE_R for 5 percent damping. Prior to the site-specific ground motion hazard analysis, we obtain the mapped seismic ground motion values and develop the general MCE_R response spectrum for 5 percent damping in accordance with Section 11.4 of ASCE 7-16 (SEAOC/OSHPD, 2022). Based on the data obtained from the ReMi survey (Appendix D), the average shear wave velocity for the upper 30 meters of soil (V_{s30}) utilized in the analysis is 315 meters per second (1032 feet per second).

We have used the 2014 new generation attenuation (NGA) West-2 relationships to evaluate the site-specific ground motions. The NGA relationships used for developing the probabilistic and deterministic response spectra are by Chiou and Youngs (2014), Campbell and Bozorgnia (2014), Boore, Stewart, Seyhan, and Atkinson (2014), and Abrahamson, Silva, and Kamai (2014). We have performed the PSHA using Open Seismic Hazard Analysis software developed by USGS (USGS, 2022a), and the DSHA (Seyhan, 2015) using the Calculation of Weighted Average 2014 NGA Models spreadsheet by the Pacific Earthquake Engineering Research Center (PEER).

PSHA includes analyses for earthquake hazards having a 2 percent chance of being exceeded in 50 years multiplied by the risk coefficients per ASCE 7-16. The maximum rotated components of ground motions are considered in PSHA with 5 percent damping. For the DSHA, we have analyzed accelerations from characteristic earthquakes on active faults within the region using the USGS Unified Hazard Tool application (USGS, 2022b). Our evaluation indicates a magnitude 7.4 event on the Maacama fault with a rupture distance of 2.01 kilometers from the site is the controlling earthquake. Hence, the deterministic seismic hazard analysis for the site uses this event along with corrections to the spectral accelerations for the 84^{th} percentile of the maximum rotated component of ground motion with 5 percent damping.

The site-specific MCE_R response spectrum is taken as the lesser of the spectral response acceleration at any period from the PSHA and DSHA, and the site-specific general response spectrum is determined by taking two-thirds of the MCE_R response spectrum with some conditions in accordance with Section 21.3 of ASCE 7-16. Figure 6 presents the site-specific MCE_R response spectrum and the site-specific design response spectrum. The general mapped design response spectrum calculated in accordance with Section 11.4 of ASCE 7-16 is also presented on Figure 6 for comparison. The site-specific spectral response acceleration parameters, consistent with the 2019 CBC, are provided in Section 8.1 for the evaluation of seismic loads on buildings and other structures. The calculated site-specific maximum considered earthquake geometric mean (MCE_G) peak ground acceleration, PGA_M, is 0.919g. The site-specific ground motion analyses for the subject project are provided in Appendix E.

6.1.4 Liquefaction and Strain Softening

The strong vibratory motions generated by earthquakes can trigger a rapid loss of shear strength in saturated, loose, granular soils of low plasticity (liquefaction) or in wet, sensitive, cohesive soils (strain softening). Liquefaction and strain softening can result in a loss of foundation bearing capacity, or lateral spreading of sloping or unconfined ground. Liquefaction can also generate sand boils leading to subsidence at the ground surface. Liquefaction (or strain softening) is generally not a concern at depths more than 50 feet below ground surface.

Regional studies of liquefaction susceptibility (Cal OES, 2022) indicate that the site is not located within an area identified as a liquefaction zone. The seismic hazard zone map prepared by the California Geological Survey (CGS, 2001) indicates that the subject site does not fall within a mapped liquefaction hazard zone (Figure 5).

The subsurface exploration indicates layers of sand and fine-grained soil of low plasticity below groundwater level. Our analyses include evaluation of the potential for liquefaction in accordance with the methods presented by Boulanger and Idriss (2014) using the data collected during our subsurface exploration. Our analysis assumes a design groundwater elevation of 5 feet below the ground surface, and considers a seismic event producing a PGA of 0.919g resulting from a Magnitude 7.4 earthquake. The results of our analysis indicate that thin layers of coarse-grained soils and non-plastic sandy silts encountered below the assumed groundwater level will liquefy under the considered ground motion. Other consequences of liquefaction, including dynamic settlement, sand-boil-induced ground subsidence, and lateral spreading, are addressed in the following sections.

Due to the primarily granular nature of subsurface soils encountered in our borings at the project site, the soils are not considered sensitive. As such, we do not regard seismically induced strain-softening behavior as a design consideration.

6.1.5 Dynamic Settlement

The strong vibratory motion associated with earthquakes can also dynamically compact loose granular soil, leading to surficial settlements. Dynamic settlement may occur in both dry and saturated sand and silt.

Our analysis includes evaluating the potential for dynamic settlement due to liquefaction of saturated soil using the data collected during our field investigation and the methodology of Boulanger and Idriss (2014). Our analysis considers a Magnitude 7.4 earthquake producing a PGA of 0.919g and a design groundwater elevation of 5 feet below the ground surface (PSI, 2011). The results of our analysis indicate that the free-field total dynamic settlement following the considered seismic event will be up to about 5 inches with most of the settlement occurring within the upper 30 feet. The liquefaction densification settlement varies from about 3½ to 4¼ inches, and dry sand settlement in the upper five feet varies from about ¼ to 1½ inches. Further, based on our analyses, we judge the risk of sand-boil-induced ground subsidence is moderate to high which can increase the magnitude of differential settlement affecting isolated shallow foundations. Recommendations are provided for compaction of soils within the upper five feet along with reinforced shallow foundations. If needed to further mitigate dynamic settlement, we provide recommendations for deep foundations or deep soil improvement.

Following recommended surficial earthwork and using reinforced shallow footing foundations, differential dynamic settlement is estimated to be up to about 2 inches over a horizontal distance of approximately 30 feet. For reinforced concrete mat foundations, we estimate differential dynamic settlement would be up to about 2 inches across the mat foundation. If deep foundations or deep soil improvements are implemented, differential dynamic settlement is estimated to be about ½ inch over a horizontal distance of approximately 30 feet. Note that estimated dynamic settlement is in additional to estimated static settlement as discussed in Section 6.4.

6.1.6 Lateral Spread

In addition to vertical displacements, seismic ground shaking can induce horizontal displacements as surficial soil deposits spread laterally by floating atop liquefied subsurface layers. Lateral spread can occur on sloping ground or on flat ground adjacent to an exposed face. Based on the site's relatively flat topography, lack of a nearby free face, and our liquefaction analysis, we do not anticipate that lateral spreading will occur near the proposed structure following a significant seismic event.

6.2 Landsliding and Slope Stability

Regional geologic mapping (USGS, 1979) does not show any landslides on or near the project site. The ground surface at the site is relatively flat. Based on the existing topography and our review of existing maps and literature, we do not regard landslides or slope stability as design considerations for this project.

6.3 Flooding and Inundation Due to Dam Failure

Our review of Federal Emergency Management Agency (FEMA) Flood Insurance Rate Maps (FEMA, 2009) found that the site is within the 100 and 500-year flood zones, and located in Zone X, areas of 1% annual chance flood with average depth less than one foot or drainage areas of less than on square mile. Based on these findings, the potential for flooding is moderate to high and should be further evaluated by experts in that field.

Properties located downstream of dams can be inundated with flood waters if the dams were to fail. Dam owners are required to prepare inundation maps showing the limits of flooding caused by dam failure. The closest major dam to the site is the Mendocino Middle Dam, which is located approximately 3.5 miles southeast of the project site. The dam is operated by the county of Mendocino. Mendocino Middle Dam regulates water flow into Mill Creek and a failure or breach of the dam caused by an earthquake or other event would result in flooding along the Mill Creek corridor. Based on maps provided by the State of California Department of Water Resources (DWR, 2022), the project site is located outside of the inundation path for Mendocino Middle Dam and other smaller dams in the local area.

6.4 Static Settlement

Although building loads were not available at the time of this report, based on the results of our subsurface evaluation and laboratory testing, static settlement due to sustained loading is not a design consideration. Static settlement due to building loads is anticipated to be on the order of 1 inch with a differential settlement of ½ inch over 30 feet.

6.5 Expansive Soils

Some clay minerals undergo volume changes upon wetting or drying. Unsaturated soil containing those minerals will shrink/swell with the removal/addition of water. The heaving pressures associated with this expansion can damage structures and flatwork. Laboratory testing performed on a sample of the near-surface soil to evaluate the expansion characteristics in accordance with the American Society of Testing and Materials (ASTM) Standard D 4829 (Expansion Index) indicates an expansion index of 19. These results are indicative of a very low expansion characteristic. Based on results of our laboratory testing of surficial soils, we do not regard expansive soils as design considerations for the project.

6.6 Corrosive/Deleterious Soil

Corrosivity analysis, performed by CERCO Analytical, Inc. of Concord, California on one sample of the near-surface soil, indicates "moderately corrosive" based on resistivity test results. CERCO Analytical' s report (see Appendix C) includes the following recommendation: "All buried iron, steel, cast iron, ductile iron, galvanized steel and dielectric coated steel or iron should be properly protected against corrosion depending upon the critical nature of the structure. All buried metallic pressure piping such as ductile iron firewater pipelines should be protected against corrosion." Please refer to the CERCO Analytical report included in Appendix C for more information regarding their test results and brief evaluation.

6.7 Excavation Characteristics

We anticipate that the project will involve excavations of up to a several feet for construction of the proposed improvements. The surficial soils encountered during our subsurface exploration generally consisted of stiff to very stiff sandy silt; and loose silty gravel.

We anticipate that conventional earthmoving and drilling equipment in good working condition should be able to make the proposed excavations. Excavations in fill materials, if encountered, may encounter obstructions consisting of debris, rubble, abandoned structures, utilities or over-sized materials that may require special handling or demolition equipment for removal. Near vertical cuts may not be stable particularly if the excavation encounters sand or gravel, is exposed to rainfall or runoff, extends near the groundwater level or encounters seeping groundwater. Excavation subgrade may become unstable if exposed to wet conditions. Recommendations for excavation stabilization are presented in Section 8.3.7. Where encountered, moist excavated materials will need to be dried out before reuse as fill.

7 CONCLUSIONS

Based on our review of the referenced background data (including the previous evaluation by PSI for the site), our site field reconnaissance, subsurface evaluation, and laboratory testing, it is our opinion that proposed construction is feasible from a geotechnical standpoint. Geotechnical considerations include the following:

- Our subsurface exploration encountered alluvium that generally consists of brown to dark gray, moist to wet, stiff to hard, silt, lean clay and silty clay; brown, moist to wet, loose to very dense, intermittent layers of sand and gravel mixtures.

- Groundwater, as encountered in our exploratory borings at the site, is logged at a depth as shallow as 25 feet below grade. Variation and fluctuation in groundwater levels should be anticipated as discussed in Section 5.4. Review of the previous evaluation for the site (PSI, 2011) indicates that the groundwater level is approximately 5 feet below the existing grade.

- Based on our review of the referenced geologic maps, the project site is not underlain by known active faults (i.e., faults that exhibit evidence of surface displacement in the last 11,700 years). Therefore, the potential for ground surface rupture because of faulting at the site is considered low.

- The site could experience relatively intense ground shaking due to a significant earthquake event resulting in liquefaction and dynamic settlement.

- The site is not located within a mapped liquefaction hazard zone. However, based on the subsurface soils and shallow groundwater encountered in our borings (and review of PSI, 2011 report), the results of our analyses for liquefaction potential indicate that loose to medium dense granular layers of soil below the assumed groundwater level will liquefy as a result of the considered ground motion.

- The results of our dynamic settlement analysis indicate that the total dynamic settlement resulting from the considered ground motion will be up to about 5 inches with most of the settlement occurring within the upper 30 feet. Recommendations are provided for compaction of soils within the upper five feet along with reinforced shallow foundations. If needed to further mitigate dynamic settlement, we provide recommendations for deep foundations or deep soil improvement. We estimate differential dynamic settlement of approximately 2 inches or ½ inch over a lateral distance of about 30 feet for reinforced shallow footing foundation or deep mitigation, respectively. For reinforced concrete mat foundations, we estimate differential dynamic settlement would be up to about 2 inches across the mat foundation. We anticipate that the proposed improvements can be designed to accommodate this level of dynamic settlement.

- Based on the site's relatively flat topography, lack of a nearby free face, and our liquefaction analysis, lateral spread is not a design consideration for this project.

- Landslides and slope stability are not design considerations for this project.

- The subject site is in a flood hazard Zone X (FEMA, 2009). This is indicative of areas of 1% annual chance flood with average depth less than one foot or drainage areas of less than on square mile.

- Static settlement should be tolerable for the proposed improvements provided that the proposed structures are supported on foundations that conform with our recommendations.

- The results of our laboratory testing indicate that the subsurface materials have a very low expansion characteristic, and expansive soils are not a design consideration for the project.

- Based on the results of the soil corrosivity tests during this study, the soils are considered to be moderately corrosive (Appendix C). A corrosion engineer may be consulted to provide specific guidance on protective measures to mitigate corrosion.

- Excavations that remain unsupported and exposed to water, or encounter seepage, or granular soil may be unstable and prone to sloughing. Recommendations for excavation stabilization are provided.

8 RECOMMENDATIONS

The following sections present our geotechnical recommendations for the design and construction of the proposed improvements. The project improvements should be designed and constructed in accordance with these recommendations, applicable codes, and appropriate construction practices.

8.1 Seismic Design Criteria

Ninyo & Moore has performed a site-specific ground motion analysis in accordance with the procedure in Chapter 21 of ASCE Standard 7-16. The assumptions and methodology for this analysis are discussed in Section 6.1.3. Seismic Site Class D was selected based on the results of the ReMi Survey (Appendix D). The design response spectrum based on the site-specific ground motion analysis is presented on Figure 6 and the corresponding seismic design criteria are summarized in Table 1. The spectral ordinates and seismic coefficients based on the mapped values of the risk-targeted spectral response acceleration, consistent with Section 11.4 of ASCE Standard 7-16, are also presented in Table 1 (SEAOC & OSHPD, 2022). In conformance with the 2019 California Building Code, the spectral ordinates and seismic coefficients consistent with Section 11.4 of ASCE Standard 7-16 may be used for seismic design provided that new structures are designed by the equivalent lateral force (ELF) procedure as per Section 12.8 of ASCE Standard 7-16. Otherwise, the seismic design criteria and design response spectrum consistent with the site-specific ground motion analysis in Table 1 and Figure 6, respectively, should be used for seismic design per the 2019 California Building Code.

Table 1 – California Building Code Seismic Design Criteria

Seismic Design Parameter	Site Specific	ASCE 7-16
Site Class	D	D
Site Coefficient, F_a	-	1.0
Site Coefficient, F_v	-	-
Mapped Spectral Response Acceleration at 0.2-second period, S_S	-	1.99 g
Mapped Spectral Response Acceleration at 1.0-second period, S_1	-	0.763 g
Site-Adjusted Spectral Acceleration at 0.2-second period, S_{MS}	1.592 g	1.99 g
Site-Adjusted Spectral Acceleration at 1.0-second period, S_{M1}	1.038 g	-

Table 1 – California Building Code Seismic Design Criteria		
Seismic Design Parameter	**Site Specific**	**ASCE 7-16**
Design Spectral Response Acceleration at 0.2-second Period, S_{DS}	1.061 g	1.327 g
Design Spectral Response Acceleration at 1.0-second Period, S_{D1}	0.692 g	-
Seismic Design Category for Risk Category I, II, or III	II	II

8.2 Foundations

Foundations should be designed in accordance with structural considerations and our geotechnical recommendations. In addition, requirements of the governing jurisdictions, practices of the Structural Engineers Association of California, and applicable building codes should be considered in the design of the structures. The proposed building may be supported on reinforced shallow foundations, deep foundations, or shallow foundations supported on densified soils, depending upon structural allowances for tolerable settlement, as presented in Section 6.1.5.

8.2.1 Spread Footings

Footings should be supported on subgrade prepared as per the recommendations in Section 8.3.5. The footings may be designed using the criteria listed in Table 2. Ninyo & Moore should observe the footing excavations to evaluate bearing materials and subgrade condition.

Table 2 – Recommended Bearing Design Parameters for Footings					
Footing	**Sustained Loads**	**Footing Widths**	**Bearing Depth[1]**	**Allowable Bearing Capacity[2]**	**Static Settlement**
Wall Footing	6 kips/foot or less	18 inches or more	2 feet or more	3,000 psf	1-inch total ½ inch differential over 30 feet
Column Footing	80 kips or less	24-48 inches	2 feet or more	3,000 psf	1-inch total ½ inch differential over 30 feet

Notes:
1 Below the adjacent finish grade and the existing grade.
2 Net allowable bearing capacity in pounds per square foot with Safety Factor of 2 or more. Allowable bearing capacity may be increased by one-third for wind or seismic load combinations.

Structures supported on footings consistent with these recommendations should be designed for the total and differential settlements listed in Table 2 for sustained loads.

The spread footings should be reinforced with deformed steel bars as detailed by the project structural engineer. All footings should be tied together with grade beams or tie beams. Where footings are located adjacent to utility trenches or other excavations, the footing bearing surfaces should bear below an imaginary plane extending upward from the bottom edge of the adjacent trench/excavation at a 1½:1 (horizontal to vertical) angle above the bottom edge of the footing. Footings should be deepened or excavation depths reduced as-needed. Footing bottoms should not be sloped more than 1-unit vertical to 10 units horizontal. Wall footings may be stepped provided that the bearing grade differential between adjacent steps does not exceed 18 inches and the slope of a series of such steps does not exceed 1-unit vertical to 2 units horizontal.

A lateral bearing pressure of 300 psf per foot of depth may be used to evaluate the resistance of footings to lateral loads. The recommended lateral bearing pressure is for level and gently sloping ground conditions where the ground slope adjacent to the foundation is 5 percent or less. The lateral bearing pressure should be neglected to a depth of 12 inches where the ground adjacent to the foundation is not covered by a slab or pavement. The lateral bearing pressure may be increased by one-third when considering loads of short duration such as wind or seismic forces. A friction coefficient of 0.30 may be assumed for evaluating frictional resistance to lateral loads for footings bearing on compacted native soils.

8.2.2 Mat Foundations

Reinforced concrete mat foundations supported by an 18-inch minimum zone of adequately placed and compacted aggregate base (in accordance with Section 8.3.6) in the upper 5 feet can be used to reduce differential settlement at the subject site. The PT slabs or reinforced concrete mat foundations should be designed in accordance with the Post-Tensioning Institute publication PTI DC 10.5-19. Mat foundations on the order of 18 inches thick may be designed using an allowable bearing capacity of 3,000 psf. The allowable bearing capacity may be increased by one-third for short duration loads, such as wind or seismic. The mat should be designed to span an unsupported length of 10 feet.

The recommended vertical modulus of subgrade reaction, k_{v1}, for use in design of flexible mat foundations is 35 pounds per cubic inch (pci) applicable for a 12-inch-square loaded area. For actual foundation sizes, the subgrade modulus should be reduced using the following formula:

$$k_v = k_{v1} \times \frac{1}{B} \qquad Equation\ 1$$

Where for a uniformly loaded mat,

$$k_v = vertical\ modulus\ of\ subgrade\ reaction\ for\ actual\ foundation\ width$$

$$k_{v1} = vertical\ modulus\ of\ subgrade\ reaction\ for\ 1 - foot - square\ loaded\ area$$
$$= 35\ pci$$

$$B = foundation\ width\ in\ feet$$

For point loads on a mat, the vertical modulus of subgrade reaction need not be reduced using the formula above for the entire width of the mat but rather some equivalent width which is related to the flexural stiffness of the mat relative to the underlying soil subgrade stiffness and may be estimated using the following formula:

$$B' = 14 \times t \leq B \qquad Equation\ 2$$

Where,

$$B' = equivalent\ foundation\ width\ in\ feet\ to\ be\ used\ in\ Equation\ 1\ for\ B$$

$$t = thickness\ of\ mat\ or\ slab\ in\ feet$$

8.2.3 Drilled Piers

Drilled piers should be at least 16-inch diameter and drilled at least 35 feet below existing grade. To evaluate resistance to downward axial loads, drilled piers may be designed for an allowable side friction of 200 pounds per square foot (psf) to a depth of 12 feet below lowest adjacent finished grade and 400 psf below a depth of 12 feet. Resistance to upward axial loads can be considered as 50 percent of the downward allowable axial capacity. The allowable side friction may be increased by one-third when considering loads of short duration, such as wind or seismic loads. Drilled piers should be spaced at least 3 diameters on-center.

An allowable lateral bearing pressure of 300 psf per foot depth up to 3,000 psf may be used to evaluate resistance to lateral loads and overturning moments. The allowable lateral bearing pressure may be increased by a factor of two for isolated pole footings that can accommodate ½ inch of lateral deflection at the ground surface.

Drilled piers in a row perpendicular to the direction of lateral loading do not need to be reduced for group effects where the center-to-center pier spacing is equivalent to 3 or more pier diameters. A reduction in the lateral resistance due to group effects should be considered for piers in a column parallel to the direction of loading where the center-to-center spacing between adjacent piers in the column is less than eight pier diameters. The reduction in lateral resistance due to group effects for piers in a column parallel to the direction of loading is influenced by the number of piers in the column and the spacing between piers. We can provide reductions for lateral loading, if needed.

Drilled pier excavations should be cleaned of loose material prior to pouring concrete. We anticipate groundwater will be encountered within the pier excavations. Drilled pier excavations that encounter groundwater or cohesionless soil may be unstable, and may need to be stabilized by temporary casing or use of drilling mud. Standing water should be removed from the pier excavation, or the concrete should be delivered to the bottom of the excavation, below the water surface, by tremie pipe. Casing should be removed from the excavation as the concrete is placed. Concrete should be placed in the piers in a manner that reduces the potential for segregation of the components.

8.2.4 Ground Improvement

For reduction of dynamic settlement associated with liquefaction, ground improvement can be performed to a minimum depth of 30 feet below existing grade. Ground improvements methods could include pressure grouting, drilled displacement columns, or similar methods of increasing the density of the granular subsurface materials. Proposed methods of ground improvement and post-improvement evaluation of settlement should be submitted to the geotechnical engineer for review and acceptance. The contractor should perform post-improvement subsurface exploration to evaluate and confirm that the improved soils have achieved suitable densification, and analysis of post-improvement settlement should be performed by a licensed engineer.

Following completion and acceptance of ground improvement, structures can be supported on shallow foundations designed and constructed in accordance with the recommendations provided in this report.

8.2.5 Slabs-on-Grade

Building floor slabs should be designed by the project structural engineer based on the anticipated loading conditions. The subgrade should be prepared in accordance with Section 8.3.5. Where a vapor retarding system is not used, slabs should be constructed on 6 inches, or more, of aggregate base and placed in accordance with Section 8.3.6. The slab should be reinforced with deformed steel bars. We recommend that masonry briquettes or plastic chairs be used to aid in the correct placement of slab reinforcement. A vapor retarder is recommended in areas where moisture-sensitive floor coverings or conditioned environments are anticipated. Joints consistent with ACI guidelines (ACI, 2021) may be constructed at periodic intervals to reduce the potential for random cracking of the slab.

8.3 Earthwork Recommendations

Earthwork should be performed in accordance with the requirements of applicable governing agencies and the recommendations presented below. Ninyo & Moore should observe foundation excavations and earthwork operations. Evaluations performed by Ninyo & Moore during the course of construction operations may result in new recommendations, which could supersede the recommendations in this section.

8.3.1 Pre-Construction Conference

We recommend that a pre-construction conference be held to discuss the recommendations presented in the report. Representatives of the City, the design engineer, Ninyo & Moore, and the contractor should be in attendance to discuss project schedule and earthwork requirements.

8.3.2 Site Preparation

Site preparation should begin with the removal of existing vegetation, utility lines, debris and other deleterious materials from areas to be graded. Tree stumps and roots should be removed to such a depth that organic material is generally not present. Clearing and grubbing should extend at least 5 feet laterally beyond the proposed excavation and fill areas. Rubble and excavated materials that do not meet criteria for use as fill should be disposed of in an appropriate landfill. Existing utilities in the work area should be relocated away from the proposed structures. Existing utilities to be abandoned should be removed, crushed in place, or backfilled with grout.

Within the new foundation footprint and extending at least five feet beyond the foundation areas, existing site soils should be excavated to a depth of 5 feet. If groundwater is encountered in the excavation, open-graded ¾-inch crushed rock wrapped in filter fabric (e.g. Mirafi 140N, or similar) should be placed in the bottom of the excavation to at least 12 inches above the depth of groundwater. We estimate that at least 12-inches of crushed rock may be required considering that groundwater has been encountered at a depth of 5 feet below grade at the site.

Excavations resulting from removal of buried utilities, tree stumps, obstructions, or required overexcavation in foundation areas should be backfilled with compacted fill in accordance with the recommendations in the following sections.

8.3.3 Observation and Removals

Prior to placement of fill, or the placement of forms or reinforcement for foundations, the client should request an evaluation of the exposed subgrade by Ninyo & Moore. Materials that are considered unsuitable shall be excavated under the observation of Ninyo & Moore in accordance with the recommendations in this section or supplemental recommendations by the geotechnical engineer.

Unsuitable materials include, but may not be limited to dry, loose, soft, wet, expansive, organic, or compressible natural soil, and undocumented or otherwise deleterious fill materials. Unsuitable materials should be removed from trench bottoms and below bearing surfaces to a depth at which suitable foundation subgrade, as evaluated in the field by Ninyo & Moore, is exposed.

8.3.4 Material Recommendations

Materials used during earthwork, grading, and paving operations should comply with the requirements listed in Table 3. Materials should be evaluated by the geotechnical engineer for suitability prior to import to the site or use. The contractor should notify the geotechnical consultant 72 hours prior to import of materials or use of on-site materials to permit time for sampling, testing, and evaluation of the proposed materials. On-site materials may need to be dried out before re-use as fill. The contractor should be responsible for the uniformity of import material brought to the site.

Table 3 – Recommended Material Requirements

Material and Use	Source	Requirements[1,2,3]
General Fill: - for uses not otherwise specified	Import	Close-graded with 35 percent or more passing No. 4 sieve and either: Expansion Index of 50 or less, Plasticity Index of 12 or less, or less than 10 percent, by dry weight, passing No. 200 sieve
	On-site borrow	No additional requirements[1]
Aggregate Base	Import	Class II; CSS[4] Section 26-1.02
Controlled Low Strength Material (CLSM)	Import	CSS[4] Section 19-3.02F
Pipe/Conduit Bedding and Pipe Zone Material -material below pipe invert to 12 inches above pipe	Import	90 to 100 percent (by mass) should pass No. 4 sieve, and 5 percent or less should pass No. 200 sieve

Table 3 – Recommended Material Requirements		
Material and Use	Source	Requirements[1,2,3]
Trench Backfill - above bedding material	Import or on-site borrow	As per general fill and excluding rock/lumps retained on 4-inch sieve or 2-inch sieve in top 12 inches
Controlled Low Strength Material (CLSM)	Import	CSS[4] Section 19-3.02G

Notes:

1 In general, fill should be free of rocks or lumps in excess of 6-inches diameter, trash, debris, roots, vegetation or other deleterious material.
2 In general, import fill should be tested or documented to be non-corrosive[3] and free from hazardous materials in concentrations above levels of concern.
3 Non-corrosive as defined by the Corrosion Guidelines (Caltrans, 2021).
4 CSS is California Standard Specifications (Caltrans, 2018).

8.3.5 Subgrade Preparation

Subgrade in trenches and below slabs, footings, flatwork, or fill should be prepared as per the recommendations in Table 4. Prepared subgrade should be maintained in a moist (but not saturated) condition by the periodic sprinkling of water prior to placement of additional overlying fill. Subgrade that has been permitted to dry out and loosen or develop desiccation cracking, should be scarified, moisture-conditioned, and recompacted as per the requirements in this report. Note that subgrade preparation as outlined in this section does not apply if groundwater or saturated subgrade conditions are encountered (see Section 8.3.2).

Table 4 – Subgrade Preparation Recommendations	
Subgrade Location	**Source**
Footings	• After clearing per Section 8.3.2, check for unsuitable materials as per Section 8.3.3. • If unsuitable material is encountered, remove and replace with CLSM or aggregate base placed and compacted per Section 8.3.6. • Scarify and moisture condition exposed subgrade as-needed to achieve a moisture content 2 points or more above the optimum as evaluated by ASTM D1557. Compact exposed subgrade per Section 8.3.6. • Keep in moist condition by sprinkling water.
Below fill, slabs, pavement, and flatwork	• After clearing per Section 8.3.2, check for unsuitable materials as per Section 8.3.3. • Scarify top 8 inches then moisture-condition and compact as per Section 8.3.6. • Keep in moist condition by sprinkling water.
Utility Trenches	• After clearing per Section 8.3.2, check for unsuitable materials as per Section 8.3.3. • Remove or compact loose/soft material.

8.3.6 Fill Placement and Compaction

Fill and backfill should be compacted in horizontal lifts in conformance with the recommendations presented in Table 5. The allowable uncompacted thickness of each lift of fill depends on the type of compaction equipment utilized, but generally should not exceed 8 inches in loose thickness.

Table 5 – Fill Placement and Compaction Recommendations

Fill Type	Location	Compacted Density[1]	Moisture Content[2]
Subgrade	Below pavement (within 12 inches of finished subgrade)	95 percent	+ 2 percent
	Below foundations or fill and in locations not already specified	90 percent	+ 2 percent
General Fill	Below pavement (within 12 inches of finished subgrade)	95 percent	+ 2 percent
	In locations not already specified	90 percent	+ 2 percent
Bedding and Pipe Zone Fill	Material below invert to 12 inches above pipe or conduit	90 percent	Near Optimum
Trench Backfill	Top 18 inches below finish subgrade for areas subject to vehicular loading	95 percent	+ 2 percent
	In locations not already specified	90 percent	+ 2 percent
Aggregate Base	Below slabs or pavement	95 percent	Near Optimum

Notes:
1 Expressed as percent relative compaction or ratio of field density to reference density (typically on a dry density basis for soil and aggregate). The reference density of soil and aggregate should be evaluated by ASTM D 1557.
2 Target moisture content at compaction relative to the optimum as evaluated by ASTM D 1557

Compacted fill should be maintained in a moist (but not saturated) condition by the periodic sprinkling of water prior to placement of additional overlying fill. Fill that has been permitted to dry out and loosen or develop desiccation cracking, should be scarified, moisture-conditioned, and recompacted as per the requirements above.

8.3.7 Temporary Excavations and Shoring

Trench excavations shall be stabilized in accordance with the Excavation Rules and Regulations (29 Code of Federal Regulations [CFR], Part 1926) stipulated by the Occupational Safety and Health Administration (OSHA) regulations (OSHA, 2021). Stabilization shall consist of shoring sidewalls or laying slopes back.

Dewatering pits or sumps should be used to depress the groundwater level (if encountered) below the bottom of the excavation. Table 6 lists the OSHA material type classifications and corresponding allowable temporary slope layback inclinations for soil deposits that may be encountered on site. Alternatively, an internally-braced shoring system or trench shield conforming to the OSHA Excavation Rules and Regulations (29 CFR, Part 1926) may be used to stabilize excavation sidewalls during construction. The lateral earth pressures listed in Table 6 may be used to design or select the internally-braced shoring system or trench shield. The recommendations listed in this table are based upon the limited subsurface data provided by our subsurface exploration and reflect the influence of the environmental conditions that existed at the time of our exploration. Excavation stability, material classifications, allowable slopes, and shoring pressures should be re-evaluated and revised, as-needed, during construction. Excavations, shoring systems and the surrounding areas should be evaluated daily by a competent person for indications of possible instability or collapse.

Table 6 – OSHA Material Classifications and Allowable Slopes

Formation	OSHA Classification	Allowable Temporary Slope[1,2,3]	Lateral Earth Pressure on Shoring[4] (psf)
Fine-grained Alluvium (above groundwater)	Type B	1h:1v (45°)	45×D + 72

Notes:
1 Allowable slope for excavations less than 20 feet deep. Excavation sidewalls in cohesive soil may be benched to meet the allowable slope criteria (measured from the bottom edge of the excavation). The allowable bench height is 4 feet. The bench at the bottom of the excavation may protrude above the allowable slope criteria.
2 In layered soil, layers shall not be sloped steeper than the layer below.
3 Temporary excavations less than 5 feet deep may be made with vertical side slopes and remain un-shored if judged to be stable by a competent person (29 CFR, Part 1926.650).
4 'D' is depth of excavation for excavations up to 20 feet deep. Includes a surface surcharge equivalent to two feet of soil.

The shoring system should be designed or selected by a suitably qualified individual or specialty subcontractor. The shoring parameters presented in this report are preliminary design criteria, and the designer should evaluate the adequacy of these parameters and make appropriate modifications for their design. We recommend that the contractor take appropriate measures to protect workers. OSHA requirements pertaining to worker safety should be observed.

Excavations made in close proximity to existing structures may undermine the foundation of those structures and/or cause soil movement related distress to the existing structures. Stabilization techniques for excavations in close proximity to existing structures will need to account for the additional loads imposed on the shoring system and appropriate setback distances for temporary slopes. The geotechnical engineer should be consulted for additional recommendations if the proposed excavations cross below a plane extending down and away from the foundation bearing surfaces of the adjacent structure at an angle of 2:1 (horizontal to vertical).

The excavation bottoms may encounter wet, loose material which may be subject to pumping under heavy equipment loads. The contractor should be prepared to stabilize the bottom of the excavations. In general, unstable bottom conditions may be mitigated by using a stabilizing geogrid, overexcavating the excavation bottom to suitable depths and replacing with compacted fill, or other suitable method. Additionally, aeration of wet soils should be anticipated.

8.3.8 Construction Dewatering

Groundwater was encountered during our subsurface exploration at a depth as shallow as approximately 25 feet. Review of the previous evaluation (PSI, 2011) at the subject site indicates that groundwater level in the site vicinity is about 5 feet below the ground surface. Variations in groundwater levels across the site and over time should be anticipated. Water intrusion into the excavations may occur as a result of groundwater intrusion or surface runoff. The contractor should be prepared to take appropriate dewatering measures in the event that water intrudes into the excavations. Sump pits, trenches, or similar measures should be used to depress the water level below the bottom of the excavation. Considerations for construction dewatering should include anticipated drawdown, volume of pumping, potential for settlement, and groundwater discharge. Disposal of groundwater should be performed in accordance with the guidelines of the Regional Water Quality Control Board.

8.3.9 Utility Trenches

Trenches constructed for the installation of underground utilities should be stabilized in accordance with our recommendations in Section 8.3.7. Utility trenches should be backfilled with materials that conform to our recommendations in Section 8.3.4. Trench backfill, bedding, and pipe zone fill should be compacted in accordance with Section 8.3.6 of this report. Bedding and pipe zone fill should be shoveled under pipe haunches and compacted by manual or mechanical, hand-held tampers. Trench backfill should be compacted by mechanical means. Densification of trench backfill by flooding or jetting should not be permitted.

Trenches should not be excavated adjacent to footings. If trenches are to be excavated near a continuous footing, the bottom of the trench should be located above a 2:1 (horizontal to vertical) plane projected downward from the bottom of the footing. Utility lines that cross beneath footings should be evaluated on a case-by-case basis, but in general encased in concrete or CLSM below the footing for a distance equivalent to the depth of the excavation.

8.4 Retaining Walls

Walls with drained backfill conditions may be designed for active or at-rest equivalent fluid earth pressures of 40 or 60 psf per foot depth, respectively, with level backfill consisting of site soils or granular select import. Walls that yield or deflect may be designed for active earth pressures. Wall deflection equivalent to about ½ percent of wall height may be needed to reduce at-rest earth pressures to active earth pressures for compacted backfill. An equivalent fluid pressure of 22H psf may be used to evaluate seismic earth pressures on retaining walls. For rising backfill conditions, the active or at-rest equivalent fluid earth pressures may be increased by 1 psf per foot depth per degree of inclination. Walls retaining broken back slopes may be evaluated by considering the slope height to be included as part of the wall height, or by considering the slope angle to be the slope of a plane extending from the toe of the slope at the back of the wall to the ground surface at a lateral distance behind the wall equivalent to twice the wall height. Wall height should be evaluated as the vertical distance above the wall footing to the ground surface at the heel of the wall.

A hydrostatic pressure equivalent to 62 psf per foot depth below the historic high groundwater level should be considered for retaining walls that extended below the historic high-water level. Hydrostatic pressures may be neglected for walls above the historic high-water level, provided that suitable drainage of the retained soil is provided. A drainage system should be provided behind the wall and connected to an appropriate outlet.

8.5 Pavement Sections

We understand that the project includes asphalt concrete pavement sections for on-site parking and access ways; and off-site adjacent roadways (Courthouse Road and Clay Street). Ninyo & Moore conducted an analysis to evaluate appropriate asphalt pavement structural sections following the methodology presented in the Highway Design Manual (Caltrans, 2020).

Projected traffic and anticipated vehicle loading data were not available at the time of our pavement evaluation and we did not evaluate a traffic index for the project. Pavement sections were evaluated for a range of traffic indices. The designer may interpolate between the values provided once a traffic index has been selected.

The design R-value of 52 (based on laboratory test results) was used for evaluate the pavement sections. The pavement sections were designed for a 20-year service life presuming that periodic maintenance, including crack sealing and resurfacing will be performed during the service life of the pavement. Premature deterioration may occur without periodic maintenance. Our recommendations for the pavement sections are presented in Table 7.

Paving operations and base preparation should be observed and tested by Ninyo & Moore. Subgrade enhancement geotextiles, where utilized, should be rolled out flat and tight, without folds or wrinkles, over prepared subgrade in the direction of travel. The geotextile should be pinned to the subgrade with nails and washers or u-shaped sod staples. Adjacent rolls should overlap 12 inches or more. Abutting rolls should overlap in the direction of fill placement to reduce the potential for peeling of the geotextile during fill placement. Aggregate base fill should be pushed over the geotextile into position and compacted. To reduce the potential for displacement of the geotextile or deterioration of the subgrade, construction equipment should not operate on the geotextile with 6 inches of aggregate base cover.

Table 7 – Asphalt Concrete Pavement Structural Sections

Design R-Value	Traffic Index	Alternative 1	Alternative 2
52	3	3 inches AC	2 inches AC 4 inches AB
52	6	5½ inches AC	3½ inches AC 4 inches AB
52	9	9 inches AC	5½ inches AC 7 inches AB

Notes:
[1] AC is Type A, Dense-Graded Hot Mix Asphalt complying with Caltrans Standard Specification 39-2 (2018).
[2] AB is Class II Aggregate Base complying with Caltrans Standard Specification 26-1.02 (2018).

Pavements should be sloped so that runoff is diverted to an appropriate collector (concrete gutter, swale, or area drain) to reduce the potential for ponding of water on the pavement. Concentration of runoff over asphalt pavement should be discouraged.

8.6 Exterior Flatwork

Pedestrian sidewalks, walkways, and other flatwork constructed of Portland cement concrete should consist of no less than 4 inches of concrete over 6 inches of aggregate base. The concrete thickness should be increased to 6 inches at driveways. Criteria for aggregate base are presented in Section 8.3.4. Recommendations for subgrade preparation and fill placement are provided in Sections 8.3.5 and 8.3.6, respectively.

Concrete flatwork should be appropriately jointed to reduce the random occurrence of cracks. Joints should be laid out in a square pattern at consistent intervals. Contraction, construction, and isolation joints should be detailed and constructed in accordance with the guidelines of American

Concrete Institute (ACI) Committee 301 (ACI, 2021). We recommend a contraction joint spacing of no more than 12 feet for driveways and no more than 8 feet for other flatwork.

Concrete flatwork may be reinforced with deformed steel bars to reduce the potential for differential slab movement, should cracking occur between joints. The reinforcing steel should have a nominal diameter of ⅜-inch or more and should be detailed by the engineer based on the anticipated loading and flatwork usage. Slabs reinforced with distributed steel should be 5 inches thick (or more). Masonry briquettes or plastic chairs should be used to maintain the position of the reinforcement in the upper portion of the slab during concrete placement.

8.7 Concrete Placement

Concrete in contact with soil or water that contains high concentrations of soluble sulfates can be subject to chemical and/or physical deterioration. The sulfate ion concentration was measured to be 19 mg/kg and is determined to be insufficient to damage reinforced concrete structures. However, due to the potential variability in soil conditions across the site, we recommend that Type V cement with a water/cement ratio of 0.45 or less be considered for the project.

8.8 Moisture Vapor Retarder

The migration of moisture through slabs underlying enclosed spaces or overlain by moisture sensitive floor coverings should be discouraged by providing a moisture vapor retarding system between the subgrade soil and the bottom of slabs. We recommend that the moisture vapor retarding system consist of a 4-inch-thick capillary break, overlain by a 15-mil-thick plastic membrane. The capillary break should be constructed of clean, compacted, open-graded crushed rock or angular gravel of ¾-inch nominal size. To reduce the potential for slab curling and cracking, an appropriate concrete mix with low shrinkage characteristics and a low water-to-cementitious-materials ratio should be specified. In addition, the concrete should be delivered and placed in accordance with ASTM C94 with attention to concrete temperature and elapsed time from batching to placement, and the slab should be cured in accordance with the ACI Manual of Concrete Practice (ACI, 2021), as appropriate. The plastic membrane should conform to the requirements in the latest version of ASTM Standard E 1745 for a Class A membrane. The bottom of the moisture barrier system should be higher in elevation than the exterior grade, if possible. Positive drainage should be established and maintained adjacent to foundations and flatwork.

8.9 Drainage and Site Maintenance

Surface drainage on the site should generally be provided so that water is diverted away from structures and is not permitted to pond. Positive drainage should be established adjacent to structures to divert surface water to an appropriate collector (graded swale, v-ditch, or area drain) with a suitable outlet. Drainage gradients should be 2 percent or more a distance of 5 feet or more from the structure for impervious surfaces and 5 percent or more a distance of 10 feet or more from the structure for pervious surfaces. Slope, pad, and roof drainage (from adjacent structures) should be collected and diverted to suitable discharge areas away from structures or other slopes by non-erodible devices (e.g., gutters, downspouts, concrete swales, etc.). Graded swales, v-ditches, or curb and gutter should be provided at the site perimeter to restrict flow of surface water onto and off of the site. Slopes should be vegetated or otherwise armored to reduce potential for erosion of soil. Drainage structures should be periodically cleaned out and repaired, as-needed, to maintain appropriate site drainage patterns.

Landscaping adjacent to foundations should include vegetation with low-water demands and irrigation should be limited to that which is needed to sustain the plants. Trees should be restricted from the areas adjacent to foundations a distance equivalent to the canopy radius of the mature tree. Bioretention areas should not be located within a distance of 20 feet from structure foundations.

Care should be taken by the contractor during grading to preserve any berms, drainage terraces, interceptor swales or other drainage devices on or adjacent to the project area. Drainage patterns established at the time of grading should be maintained for the life of the project.

8.10 Review of Construction Plans

The recommendations provided in this report are based on preliminary design information for the proposed construction. We recommend that a copy of the plans be provided to Ninyo & Moore for review before bidding to check the interpretation of our recommendations and that the designed improvements are consistent with our assumptions. It should be noted that, upon review of these documents, some recommendations presented in this report might be revised or modified to meet the project requirements.

8.11 Construction Observation and Testing

The recommendations provided in this report are based on subsurface conditions encountered in exploratory borings. During construction, the geotechnical engineer or his representative in the field should be allowed to check the exposed subsurface conditions. During construction, the geotechnical engineer or his representative's duties should include, but not limited to:

- Pre-Construction meeting.
- Check for unsuitable materials and observe foundation excavations.

- Observe preparation and compaction of subgrade.
- Check and test imported materials prior to import to site or use as fill.
- Observe placement and compaction of fill.
- Perform field density tests to evaluate fill and subgrade compaction.
- Observe placement of reinforcing steel and concrete in drilled piers and slabs.
- .Review proposed method of deep soil densification, as appropriate.

The recommendations provided in this report assume that Ninyo & Moore will be retained as the geotechnical consultant during the construction phase of the project. If another geotechnical consultant is selected, the selected consultant should provide a letter to the architect and the owner (with a copy to Ninyo & Moore) indicating that they fully understand Ninyo & Moore's recommendations, and that they are in full agreement with the recommendations contained in this report.

9 LIMITATIONS

The field evaluation, laboratory testing, geotechnical analyses, and assessment of geologic hazards presented in this report have been conducted in general accordance with current practice and the standard of care exercised by geotechnical consultants performing similar tasks in the project area at the time this report was prepared. No warranty, expressed or implied, is made regarding the conclusions, recommendations, and opinions presented in this report. There is no evaluation detailed enough to reveal every subsurface condition. Variations may exist, and conditions not observed or described in this report may be encountered during construction. Uncertainties relative to subsurface conditions can be reduced through additional subsurface exploration. Additional subsurface evaluation will be performed upon request. Please also note that our evaluation was limited to assessment of the geotechnical aspects of the project, and did not include evaluation of structural issues, environmental concerns, or the presence of hazardous materials.

This document is intended to be used only in its entirety. No portion of the document, by itself, is designed to completely represent any aspect of the project described herein. Ninyo & Moore should be contacted if the reader requires additional information or has questions regarding the content, interpretations presented, or completeness of this document.

This report is intended for design purposes only. It does not provide sufficient data to prepare an accurate bid by contractors. It is suggested that the bidders and their geotechnical consultant perform an independent evaluation of the subsurface conditions in the project areas. The independent evaluations may include, but not be limited to, review of other geotechnical reports prepared for the adjacent areas, site reconnaissance, and additional exploration and laboratory testing.

Our conclusions, recommendations, and opinions are based on an analysis of the observed site conditions. If geotechnical conditions different from those described in this report are encountered, our office should be notified and additional recommendations, if warranted, will be provided upon request. It should be understood that the conditions of a site could change with time as a result of natural processes or the activities of man at the subject site or nearby sites. In addition, changes to the applicable laws, regulations, codes, and standards of practice may occur due to government action or the broadening of knowledge. The findings of this report may, therefore, be invalidated over time, in part or in whole, by changes over which Ninyo & Moore has no control.

This report is intended exclusively for use by the client. Any use or reuse of the findings, conclusions, and/or recommendations of this report by parties other than the client is undertaken at said parties' sole risk.

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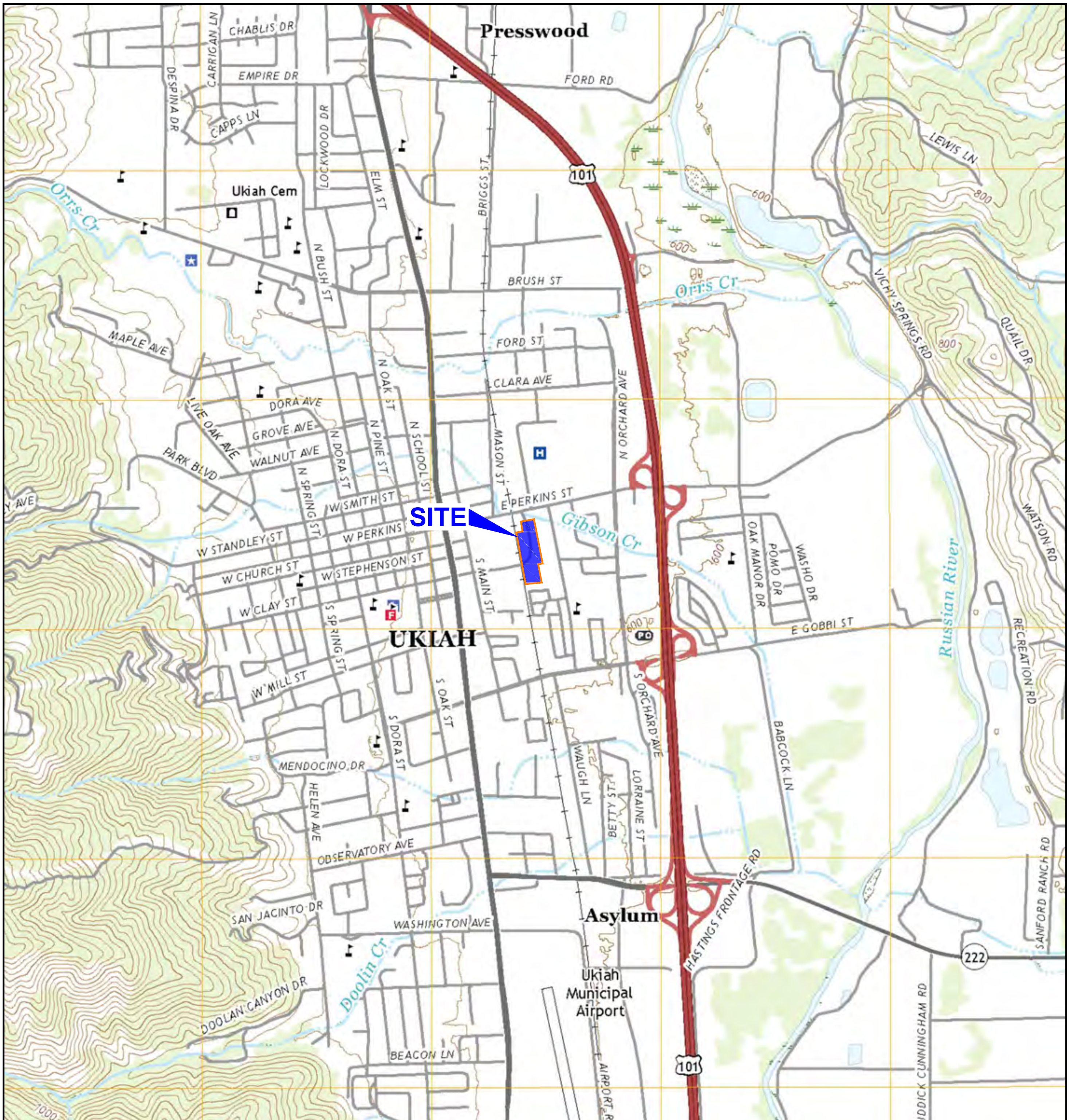
FIGURES



NOTE: DIMENSIONS, DIRECTIONS, AND LOCATIONS ARE APPROXIMATE I REFERENCE: USGS, 2018

N

SCALE (FEET)

0 2,000 4,000

FIGURE 1

SITE LOCATION

UKIAH NEW COURTHOUSE BUILDING
300 EAST PERKINS STREET
UKIAH, CALIFORNIA
404353001 I 10/22

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404353001.dwg 09/26/2022 AEK

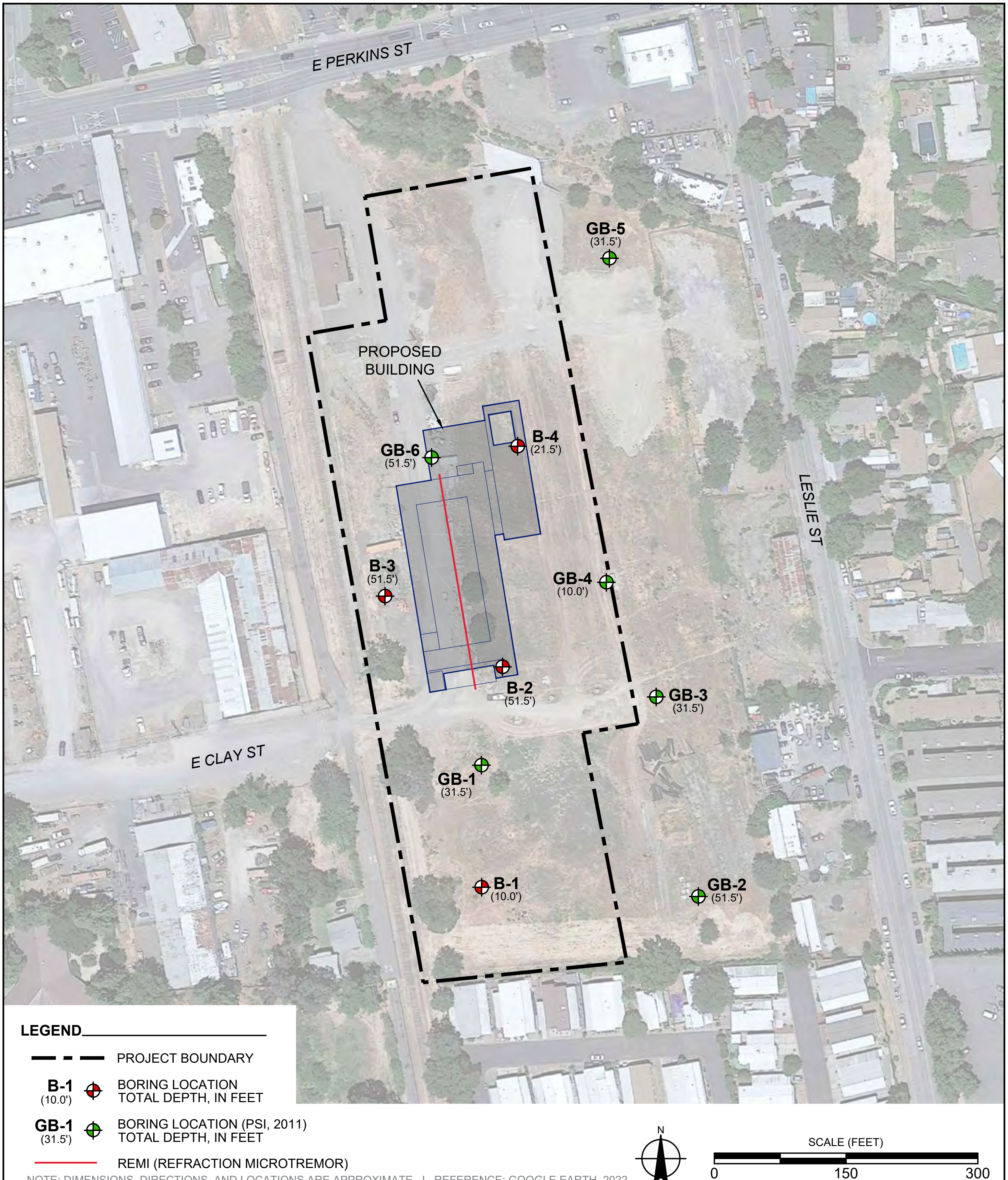

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FIGURE 2

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EXPLORATION LOCATIONS

UKIAH NEW COURTHOUSE BUILDING
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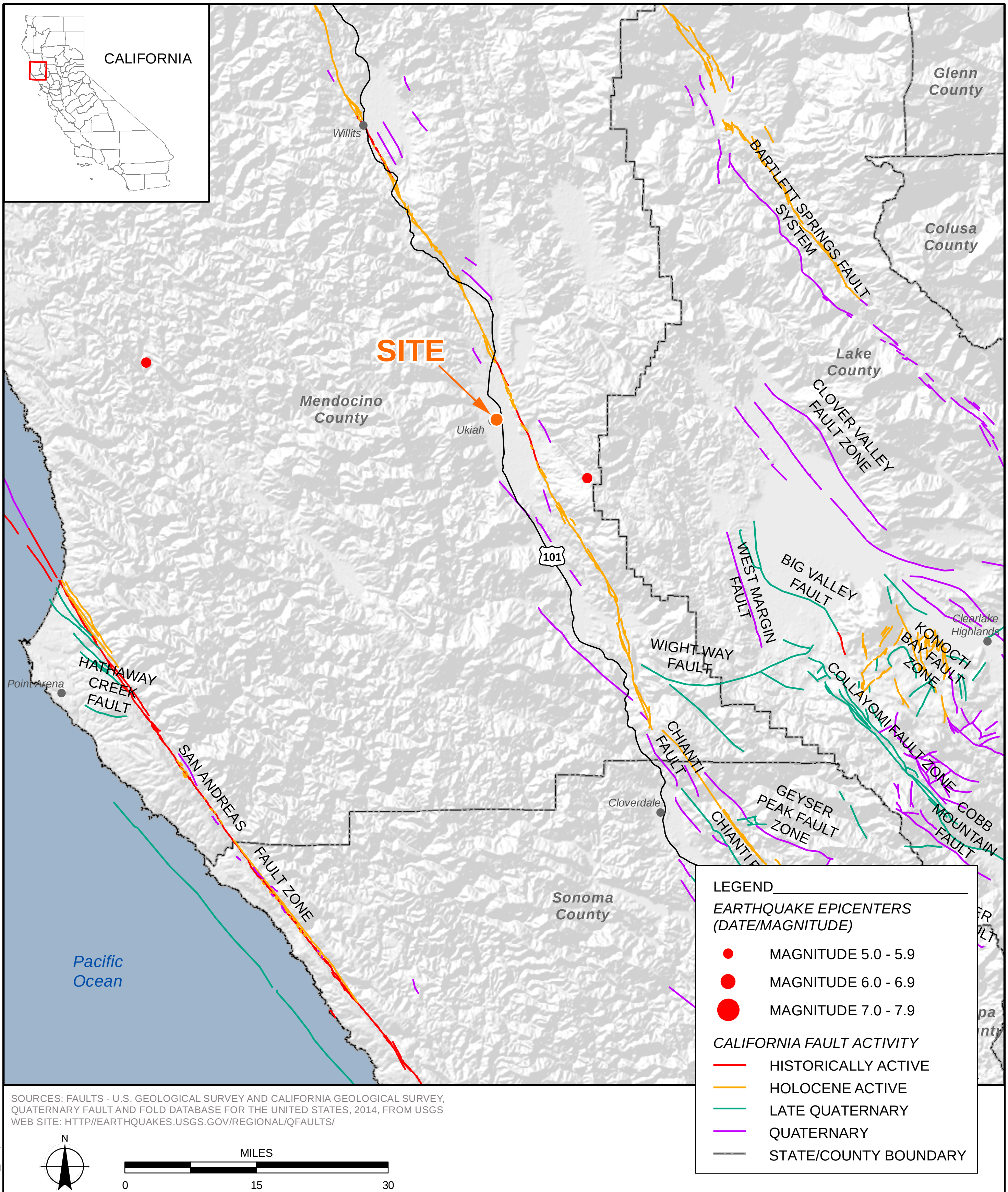
404353001 I 10/22



SOURCES: FAULTS - U.S. GEOLOGICAL SURVEY AND CALIFORNIA GEOLOGICAL SURVEY, QUATERNARY FAULT AND FOLD DATABASE FOR THE UNITED STATES, 2014, FROM USGS WEB SITE: HTTP//EARTHQUAKES.USGS.GOV/REGIONAL/QFAULTS/

NOTE: DIRECTIONS, DIMENSIONS, AND LOCATIONS ARE APPROXIMATE

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FIGURE 3

FAULT LOCATIONS AND EARTHQUAKE EPICENTERS

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300 EAST PERKINS STREET, UKIAH, CALIFORNIA

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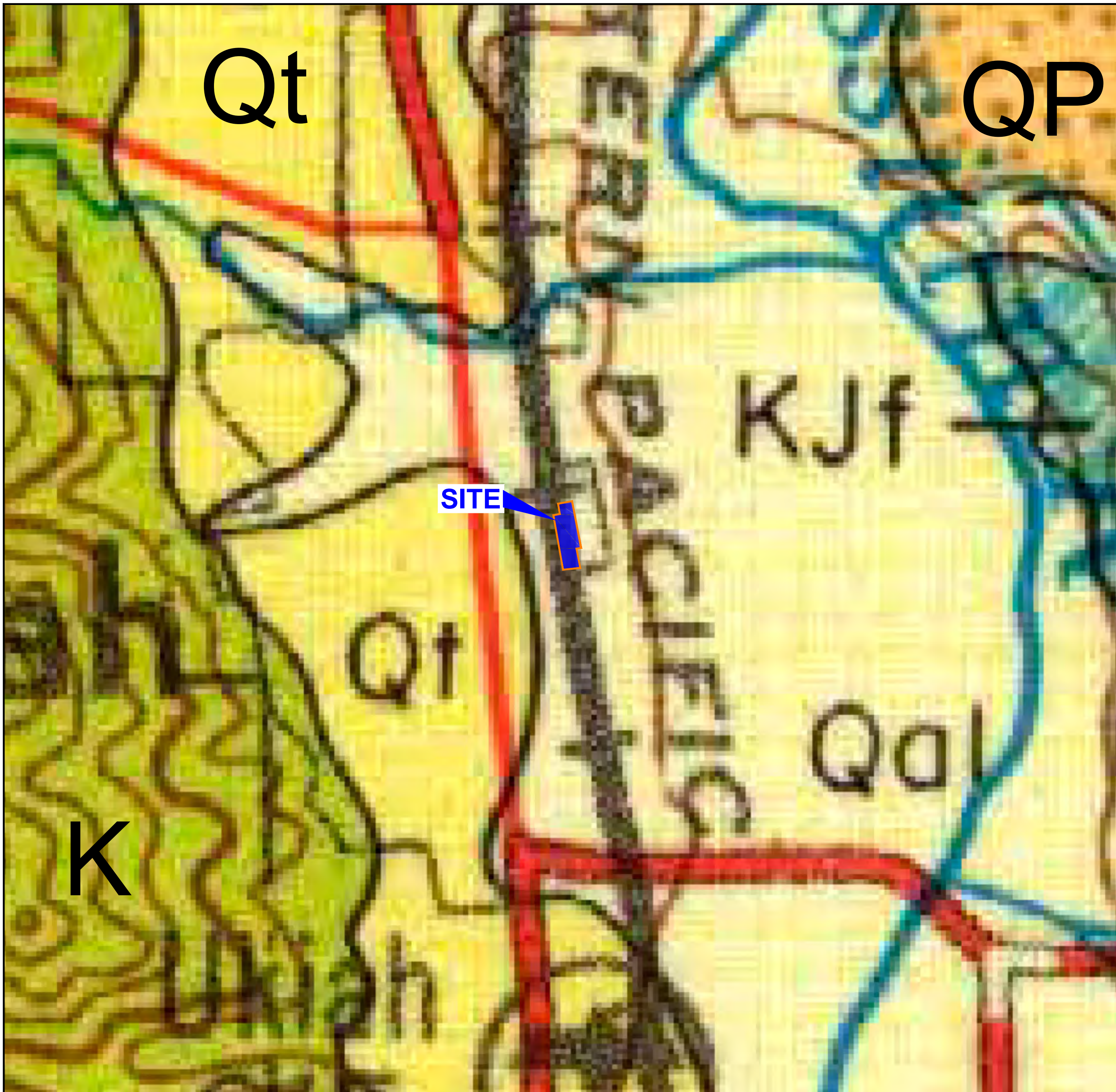

LEGEND

Qal	ALLUVIUM (HOLOCENE)	K	UNDIVIDED CRETACEOUS MARINE (CRETACEOUS)		THRUST FAULT
Qt	QUATERNARY NONMARINE DEPOSITS (PLEISTOCENE)	KJf	FRANCISCAN FORMATION (CRETACEOUS)		FAULT
QP	OLDER DUNE SAND (HOLOCENE)				GEOLOGIC CONTACT
					STRIKE AND DIP OF BEDDING

NOTE: DIMENSIONS, DIRECTIONS, AND LOCATIONS ARE APPROXIMATE
REFERENCE: USGS, 1960

N

SCALE (FEET)
0 2,000 4,000

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FIGURE 4

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REGIONAL GEOLOGY

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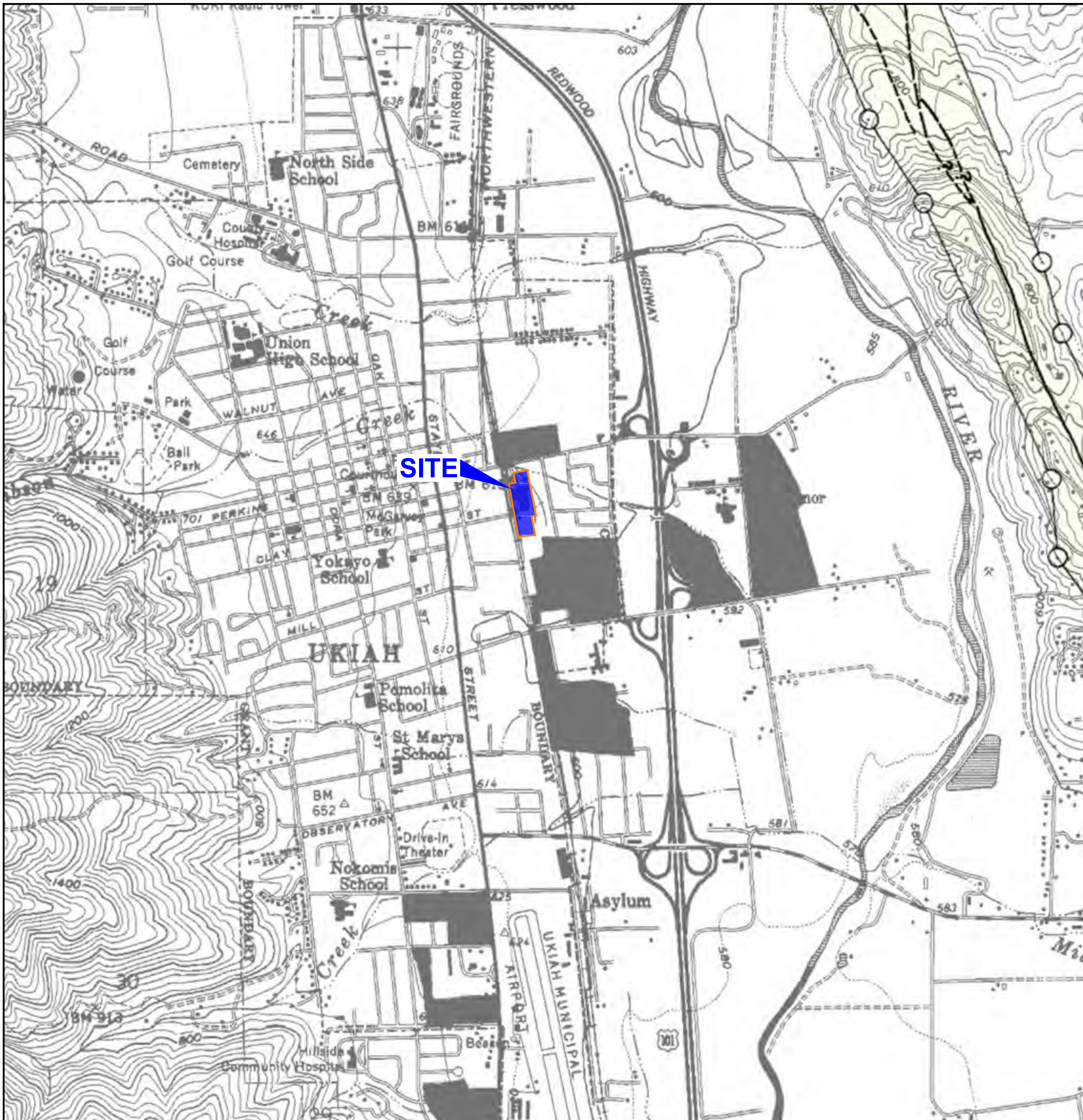

LEGEND

EARTHQUAKE FAULT ZONES:
Zone boundaries are delineated by straight-line segments; the boundaries define the zone encompassing active faults that constitute a potential hazard to structures from surface faulting or fault creep such that avoidance as described in Public Resources Code Section 2621.5(a) would be required.

NOTE: DIMENSIONS, DIRECTIONS, AND LOCATIONS ARE APPROXIMATE | REFERENCE: CGS, 1982

N

SCALE (FEET)
0 2,000 4,000

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FIGURE 5

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SEISMIC HAZARD ZONES

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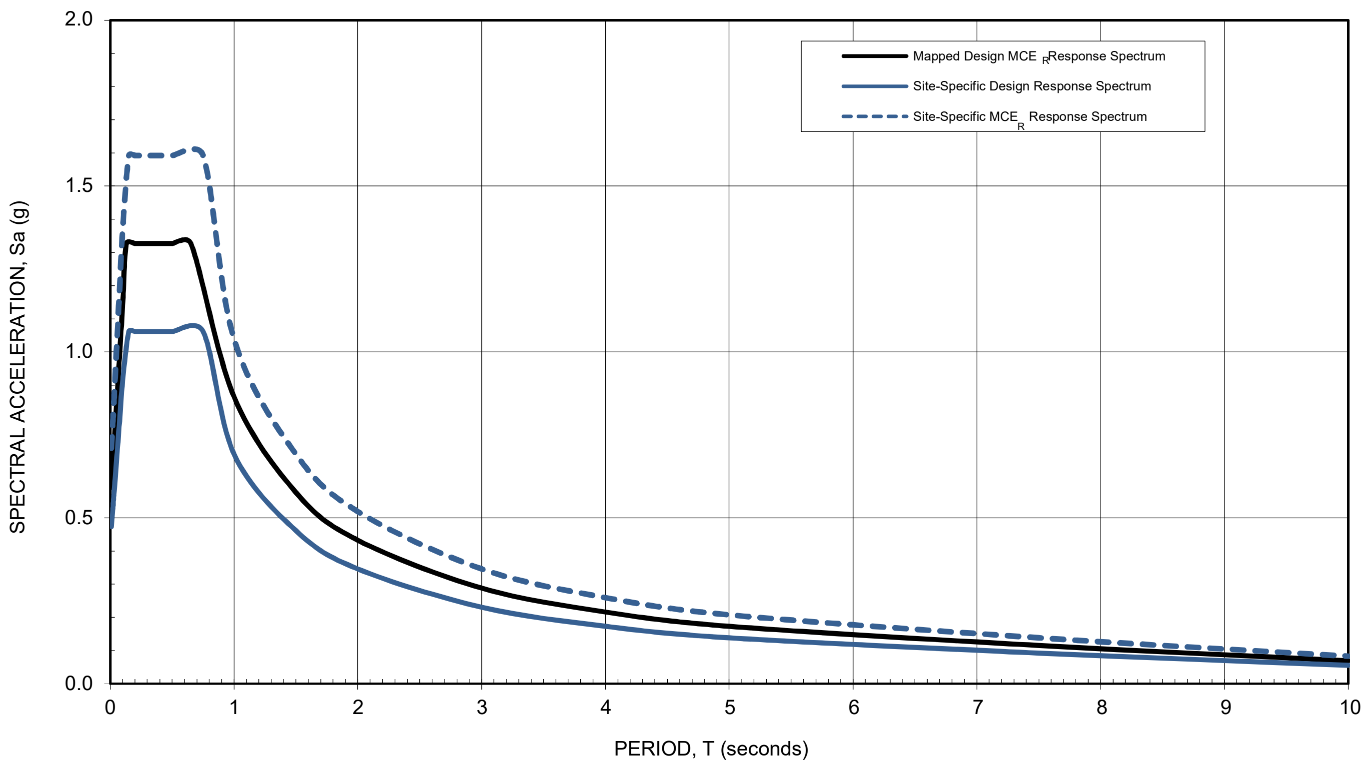
PERIOD (seconds)	SITE-SPECIFIC MCE_R RESPONSE SPECTRUM Sa (g)	SITE-SPECIFIC DESIGN RESPONSE SPECTRUM Sa (g)
0.010	0.710	0.473
0.020	0.783	0.522
0.030	0.857	0.571
0.050	1.003	0.669
0.075	1.186	0.791
0.100	1.370	0.913
0.150	1.592	1.061
0.200	1.592	1.061
0.250	1.592	1.061
0.300	1.592	1.061
0.400	1.592	1.061

PERIOD (seconds)	SITE-SPECIFIC MCE_R RESPONSE SPECTRUM Sa (g)	SITE-SPECIFIC DESIGN RESPONSE SPECTRUM Sa (g)
0.500	1.592	1.061
0.750	1.592	1.061
1.000	1.038	0.692
1.500	0.692	0.461
2.000	0.519	0.346
3.000	0.346	0.231
4.000	0.259	0.173
5.000	0.208	0.138
7.500	0.138	0.092
10.000	0.083	0.055

S_{DS} = 1.061 g	S_{D1} = 0.692 g	S_{MS} = 1.592 g	S_{M1} = 1.038 g	PGA_M = 0.735 g



NOTES:

1 The probabilistic ground motion spectral response accelerations are based on the risk-targeted Maximum Considered Earthquake (MCE_R) having a 2% probability of exceedance in 50 years in the maximum direction using the Chiou & Youngs (2014), Campbell & Bozorgnia (2014), Boore et al. (2014), and Abrahamson et al. (2014) attenuation relationships and the risk coefficients.

2 The deterministic ground motion spectral response accelerations are for the 84th percentile of the geometric mean values in the maximum direction using the Chiou & Youngs (2014), Campbell & Bozorgnia (2014), Boore et al. (2014), and Abrahamson et al. (2014) attenuation relationships for deep soil sites considering a Mw 7.4 event on the Maacama Fault located 2.01 kilometers from the site. It conforms with the lower bound limit per ASCE 7-16 Section 21.2.2.

3 The Site-Specific MCE_R Response Spectrum is the lesser of spectral ordinates of deterministic and probabilistic accelerations at each period per ASCE 7-16 Section 21.2.3. The Site-Specific Design Response Spectrum conforms with lower bound limit per ASCE 7-16 Section 21.3.

4 The Mapped Design MCE_R Response Spectrum is computed from mapped spectral ordinates modified for Site Class D (stiff soil profile) per ASCE 7-16 Section 11.4. It is presented for the sake of comparison.

FIGURE 6

ACCELERATION RESPONSE SPECTRA

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APPENDIX A

Boring Logs

APPENDIX A

BORING LOGS

Field Procedure for the Collection of Disturbed Samples
Disturbed soil samples were obtained in the field using the following methods.

Bulk Samples
Bulk samples of representative earth materials were obtained from the exploratory borings. The samples were bagged and transported to the laboratory for testing.

The Standard Penetration Test (SPT) Sampler
Disturbed drive samples of earth materials were obtained by means of a Standard Penetration Test sampler. The sampler is composed of a split barrel with an external diameter of 2 inches and an unlined internal diameter of 1-3/8 inches. The sampler was driven into the ground 18 inches or the interval recorded on the boring log where driving refusal occurred, with a 140-pound hammer falling relatively freely from a height of 30 inches in general accordance with ASTM D 1586. The blow counts were recorded for every 6 inches of penetration; the blow counts reported on the logs are those for the last 12 inches of penetration or the interval reported. Soil samples were observed and removed from the sampler, bagged, sealed and transported to the laboratory for testing.

Field Procedure for the Collection of Relatively Undisturbed Samples
Relatively undisturbed soil samples were obtained in the field using the following method.

Modified Split-Barrel Drive Sampler
Relatively undisturbed soil samples were obtained in the field using a modified split-barrel drive sampler. The sampler, with an external diameter of 3.0 inches, was lined with 6-inch long, thin wall stainless steel liners with inside diameters of approximately 2.4 inches. The sample barrel was driven into the ground with a 140-pound hammer with a drop height of 30 inches in general accordance with ASTM D 3550. The driving weight was permitted to fall relatively freely. The sampler was driven into the ground 18 inches or the interval recorded on the boring log where driving refusal occurred The approximate length of the fall, the weight of the hammer, and the number of blows for the last 12 inches of penetration or the interval reported presented on the boring logs. The blow counts are recorded as an index to the relative resistance of the materials sampled. The samples were removed from the sample barrel in the liners, sealed, and transported to the laboratory for testing.

BORING LOG EXPLANATION SHEET

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	
0								Bulk sample.
								Modified split-barrel drive sampler.
								No recovery with modified split-barrel drive sampler.
								Sample retained by others.
								Standard Penetration Test (SPT).
5								No recovery with a SPT.
			XX/XX					Shelby tube sample. Distance pushed in inches/length of sample recovered in inches.
								No recovery with Shelby tube sampler.
								Continuous Push Sample.
								Seepage.
10								Groundwater encountered during drilling.
								Groundwater measured after drilling.
							SM	MAJOR MATERIAL TYPE (SOIL): Solid line denotes unit change.
							CL	Dashed line denotes material change. Attitudes: Strike/Dip b: Bedding c: Contact j: Joint f: Fracture F: Fault cs: Clay Seam s: Shear bss: Basal Slide Surface sf: Shear Fracture sz: Shear Zone sbs: Shear Bedding Surface
15								
								The total depth line is a solid line that is drawn at the bottom of the boring.
20								

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BORING LOG

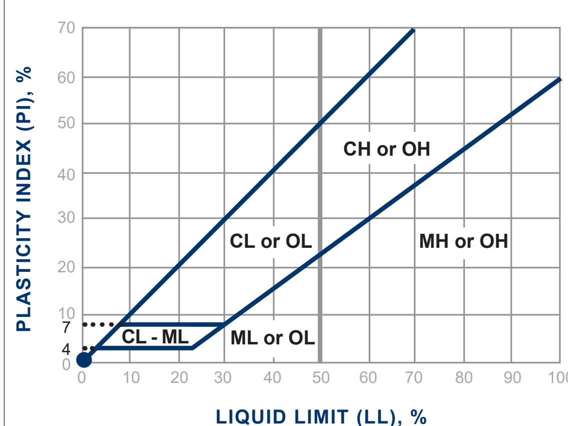
Soil Classification Chart Per ASTM D 2488

Primary Divisions			Secondary Divisions: Group Symbol	Secondary Divisions: Group Name
COARSE-GRAINED SOILS more than 50% retained on No. 200 sieve	**GRAVEL** more than 50% of coarse fraction retained on No. 4 sieve	CLEAN GRAVEL less than 5% fines	GW	well-graded GRAVEL
			GP	poorly graded GRAVEL
		GRAVEL with DUAL CLASSIFICATIONS 5% to 12% fines	GW-GM	well-graded GRAVEL with silt
			GP-GM	poorly graded GRAVEL with silt
			GW-GC	well-graded GRAVEL with clay
			GP-GC	poorly graded GRAVEL with
		GRAVEL with FINES more than 12% fines	GM	silty GRAVEL
			GC	clayey GRAVEL
			GC-GM	silty, clayey GRAVEL
	SAND 50% or more of coarse fraction passes No. 4 sieve	CLEAN SAND less than 5% fines	SW	well-graded SAND
			SP	poorly graded SAND
		SAND with DUAL CLASSIFICATIONS 5% to 12% fines	SW-SM	well-graded SAND with silt
			SP-SM	poorly graded SAND with silt
			SW-SC	well-graded SAND with clay
			SP-SC	poorly graded SAND with clay
		SAND with FINES more than 12% fines	SM	silty SAND
			SC	clayey SAND
			SC-SM	silty, clayey SAND
FINE-GRAINED SOILS 50% or more passes No. 200 sieve	**SILT and CLAY** liquid limit less than 50%	INORGANIC	CL	lean CLAY
			ML	SILT
			CL-ML	silty CLAY
		ORGANIC	OL (PI > 4)	organic CLAY
			OL (PI < 4)	organic SILT
	SILT and CLAY liquid limit 50% or more	INORGANIC	CH	fat CLAY
			MH	elastic SILT
		ORGANIC	OH (plots on or above "A"-line)	organic CLAY
			OH (plots below "A"-line)	organic SILT
	Highly Organic Soils		PT	Peat

Grain Size

Description		Sieve Size	Grain Size	Approximate Size
Boulders		> 12"	> 12"	Larger than basketball-sized
Cobbles		3 - 12"	3 - 12"	Fist-sized to basketball-sized
Gravel	Coarse	3/4 - 3"	3/4 - 3"	Thumb-sized to fist-sized
	Fine	#4 - 3/4"	0.19 - 0.75"	Pea-sized to thumb-sized
Sand	Coarse	#10 - #4	0.079 - 0.19"	Rock-salt-sized to pea-sized
	Medium	#40 - #10	0.017 - 0.079"	Sugar-sized to rock-salt-sized
	Fine	#200 - #40	0.0029 - 0.017"	Flour-sized to sugar-sized
Fines		Passing #200	< 0.0029"	Flour-sized and smaller

Plasticity Chart



Apparent Density - Coarse-Grained Soil

Apparent Density	Spooling Cable or Cathead: SPT (blows/foot)	Spooling Cable or Cathead: Modified Split Barrel (blows/foot)	Automatic Trip Hammer: SPT (blows/foot)	Automatic Trip Hammer: Modified Split Barrel (blows/foot)
Very Loose	≤ 4	≤ 8	≤ 3	≤ 5
Loose	5 - 10	9 - 21	4 - 7	6 - 14
Medium Dense	11 - 30	22 - 63	8 - 20	15 - 42
Dense	31 - 50	64 - 105	21 - 33	43 - 70
Very Dense	> 50	> 105	> 33	> 70

Consistency - Fine-Grained Soil

Consis-tency	Spooling Cable or Cathead: SPT (blows/foot)	Spooling Cable or Cathead: Modified Split Barrel (blows/foot)	Automatic Trip Hammer: SPT (blows/foot)	Automatic Trip Hammer: Modified Split Barrel (blows/foot)
Very Soft	< 2	< 3	< 1	< 2
Soft	2 - 4	3 - 5	1 - 3	2 - 3
Firm	5 - 8	6 - 10	4 - 5	4 - 6
Stiff	9 - 15	11 - 20	6 - 10	7 - 13
Very Stiff	16 - 30	21 - 39	11 - 20	14 - 26
Hard	> 30	> 39	> 20	> 26

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USCS METHOD OF SOIL CLASSIFICATION

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DATE DRILLED 9/14/22 BORING NO. B-1 GROUND ELEVATION 609' ± (MSL) SHEET 1 OF 1 METHOD OF DRILLING 6" HSA DR10K1 Truck Mounted Rig (Clear Heart Drilling) DRIVE WEIGHT 140 lbs (automatic) DROP 30 inches SAMPLED BY JW LOGGED BY JW REVIEWED BY RPM,MKW **DESCRIPTION/INTERPRETATION**
0							ML	ALLUVIUM: Brown, moist, stiff to very stiff, sandy SILT; trace gravel.
			15	11.7	87.8			
			19					Increase in clay content.
			30				CL	Brown, moist, very stiff to hard, lean CLAY.
10								Total Depth = 10 feet. Boring was backfilled with cement grout. Notes: Groundwater, was not encountered below existing grade. It may rise to a higher level due to seasonal variations in precipitation and several other factors as discussed in the report. The ground elevation shown above is an estimation only. It is based on our interpretations of published maps and other documents reviewed for the purposes of this evaluation. It is not sufficiently accurate for preparing construction bids and design documents (Google Earth, 2022).
20								
30								
40								

DATE DRILLED 9/14/22 BORING NO. B-2

GROUND ELEVATION 609' ± (MSL) SHEET 1 OF 2

METHOD OF DRILLING 6" HSA DR10K1 Truck Mounted Rig (Clear Heart Drilling)

DRIVE WEIGHT 140 lbs (automatic) DROP 30 inches

SAMPLED BY JW LOGGED BY JW REVIEWED BY RPM,MKW

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0							ML	ALLUVIUM: Brown, moist, very stiff, sandy SILT; few to little coarse gravel.
			21					
			10	12.6	103.7		CL-ML	Brown, moist, stiff, silty CLAY; few fine gravel.
							GW	Brown, moist, medium dense, well-graded GRAVEL with sand.
10			22					
			19				GP-GM	Brown, moist, medium dense, poorly-graded GRAVEL with silt and sand.
							SW-SM	Brown, moist, medium dense, well-graded SAND with silt and gravel.
20			24				GC	Brown, moist, medium dense, clayey GRAVEL.
							SC	Brown, wet, medium dense, clayey SAND.
			17					Dark gray.
30			31				SM	Brown, wet, dense, silty SAND.
			48				GW	Brown, wet, dense to very dense, well-graded GRAVEL with sand.
40								

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DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
40		▼	52				GW	ALLUVIUM (Continued): Brown, wet, dense to very dense, well-graded GRAVEL with sand.
			44					
50		▼	39					
								Total Depth = 51.5 feet.
								Boring was backfilled with cement grout.
								Notes: Groundwater, was encountered at 29 feet below existing grade. It may rise to a higher level due to seasonal variations in precipitation and several other factors as discussed in the report.
60								The ground elevation shown above is an estimation only. It is based on our interpretations of published maps and other documents reviewed for the purposes of this evaluation. It is not sufficiently accurate for preparing construction bids and design documents (Google Earth, 2022).
70								
80								

DATE DRILLED 9/14/22 BORING NO. B-2

GROUND ELEVATION 609' ± (MSL) SHEET 2 OF 2

METHOD OF DRILLING 6" HSA DR10K1 Truck Mounted Rig (Clear Heart Drilling)

DRIVE WEIGHT 140 lbs (automatic) DROP 30 inches

SAMPLED BY JW LOGGED BY JW REVIEWED BY RPM,MKW

DATE DRILLED	9/13/22
BORING NO.	B-3
GROUND ELEVATION	611' ± (MSL)
SHEET	1 OF 2
METHOD OF DRILLING	6" HSA DR10K1 Truck Mounted Rig (Clear Heart Drilling)
DRIVE WEIGHT	140 lbs (automatic)
DROP	30 inches
SAMPLED BY	JW
LOGGED BY	JW
REVIEWED BY	RPM,MKW

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0							GM	ALLUVIUM: Brown, moist, loose, silty GRAVEL with sand.
			9	14.3	98.1		ML	Brown, moist, stiff, sandy SILT. PP=2 tsf
			10	17.0	102.0		CL	Brown, moist, stiff, lean CLAY. PP=2.5 tsf
			17	18.3	106.1			Very stiff.
10			18					Few gravel.
							GW	Brown, moist, medium dense, well-graded GRAVEL with sand.
							GP	Brown, moist, medium dense, poorly-graded GRAVEL with sand.
			30				CL	Brown with light gray mottling, moist, hard, lean CLAY; few sand. PP=4.5 tsf
20			24	22.5	103.4			Very stiff.
							ML	Brown, moist, very stiff, sandy SILT.
			21				SM	Brown, wet, medium dense, silty SAND. PP=4 tsf
30			12	20.4			CL	Dark gray, wet, very stiff, lean CLAY. PP=3 tsf
			51				GC	Brown, wet, very dense, clayey GRAVEL.
40								

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300 E PERKINS STREET, UKIAH, CALIFORNIA
404353001 | 10/2022

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DATE DRILLED 9/13/22 BORING NO. B-3 GROUND ELEVATION 611' ± (MSL) SHEET 2 OF 2 METHOD OF DRILLING 6" HSA DR10K1 Truck Mounted Rig (Clear Heart Drilling) DRIVE WEIGHT 140 lbs (automatic) DROP 30 inches SAMPLED BY JW LOGGED BY JW REVIEWED BY RPM,MKW **DESCRIPTION/INTERPRETATION**
40			11				SW-SC	<u>ALLUVIUM (Continued):</u> Brown, wet, medium dense, well-graded SAND with clay and gravel.
							GP	Brown, wet, very dense, poorly-graded GRAVEL.
			80/11.8"				SP	Brown, wet, very dense, poorly-graded SAND; few to little gravel.
							GP	Brown, wet, dense, poorly-graded GRAVEL with sand.
50			36					
								Total Depth = 51.5 feet. Boring was backfilled with cement grout. <u>Notes</u>: Groundwater, was encountered at 25 feet below existing grade. It may rise to a higher level due to seasonal variations in precipitation and several other factors as discussed in the report. The ground elevation shown above is an estimation only. It is based on our interpretations of published maps and other documents reviewed for the purposes of this evaluation. It is not sufficiently accurate for preparing construction bids and design documents (Google Earth, 2022).
60								
70								
80								

DATE DRILLED	9/14/22	BORING NO.	B-4
GROUND ELEVATION	610' ± (MSL)	SHEET 1 OF 1	
METHOD OF DRILLING	6" HSA DR10K1 Truck Mounted Rig (Clear Heart Drilling)		
DRIVE WEIGHT	140 lbs (automatic)	DROP	30 inches
SAMPLED BY JW	LOGGED BY JW	REVIEWED BY	RPM,MKW

DEPTH (feet)	SAMPLES Bulk / Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0						ML	ALLUVIUM: Brown, moist, very stiff, sandy SILT; few to little gravel.
		20					Increase in clay content.
		18	13.2	97.4		CL-ML	Brown, moist, very stiff, silty CLAY with gravel.
						GW-GM	Brown, moist, medium dense, well-graded GRAVEL with silt and sand.
10		24					
		25					Medium dense.
20		39				GC	Brown, moist, dense, clayey GRAVEL with sand.
							Total Depth = 21.5 feet. Boring was backfilled with cement grout. Notes: Groundwater was not encountered below existing grade. It may rise to a higher level due to seasonal variations in precipitation and several other factors as discussed in the report. The ground elevation shown above is an estimation only. It is based on our interpretations of published maps and other documents reviewed for the purposes of this evaluation. It is not sufficiently accurate for preparing construction bids and design documents (Google Earth, 2022).
30							
40							

APPENDIX B

Laboratory Testing

APPENDIX B

LABORATORY TESTING

Classification
Soils were visually and texturally classified in accordance with the Unified Soil Classification System (USCS) in accordance with ASTM D 2488. Soil classifications are indicated on the logs of the exploratory borings in Appendix A.

Moisture Content
The moisture content of samples obtained from the exploratory boring was evaluated in accordance with ASTM D 2216. The test results are presented on the boring log in Appendix A.

In Place Density Tests
The dry density of relatively undisturbed samples obtained from the exploratory boring was evaluated in accordance with ASTM D 2937. The test results are presented on the log of the exploratory boring in Appendix A.

200 Wash
Evaluations of the percentage of particles finer than the No. 200 sieve in selected soil samples were performed in accordance with ASTM D 1140. The results of the tests are presented on Figures B-1.

Gradation Analysis
Gradation analysis tests were performed on selected representative soil samples in accordance with ASTM D 422. The grain-size distribution curves are shown on Figure B-2 through Figure B-6. This test results were utilized in evaluating the soil classifications in accordance with the USCS.

Atterberg Limits
Tests were performed on selected representative fine-grained soil samples to evaluate the liquid limit, plastic limit, and plasticity index in accordance with ASTM D 4318. The test results were used to evaluate the soil classification in accordance with the Unified Soil Classification System (USCS). The test results and classifications are shown on Figure B-7.

Consolidation Tests
Consolidation test was performed on a selected relatively undisturbed soil sample in accordance with ASTM D 2435. The sample was inundated during testing to represent adverse field conditions. The percent of consolidation for each load cycle was recorded as a ratio of the amount of vertical compression to the original height of the sample. The results of the test are summarized on Figure B-8.

Expansion Index Tests
The expansion index of selected materials was evaluated in accordance with ASTM D 4829. The specimen was molded under a specified compactive energy at approximately 50 percent saturation (plus or minus 1 percent). The prepared 1-inch thick by 4-inch diameter specimens were loaded with a surcharge of 144 pounds per square foot and inundated with tap water. Readings of volumetric swell were made for a period of 24 hours. The results of this tests are presented on Figure B-9.

Unconfined Compression Test
Unconfined compression tests were performed on relatively undisturbed samples in accordance with ASTM D 2166. The test results are shown on Figure B-10.

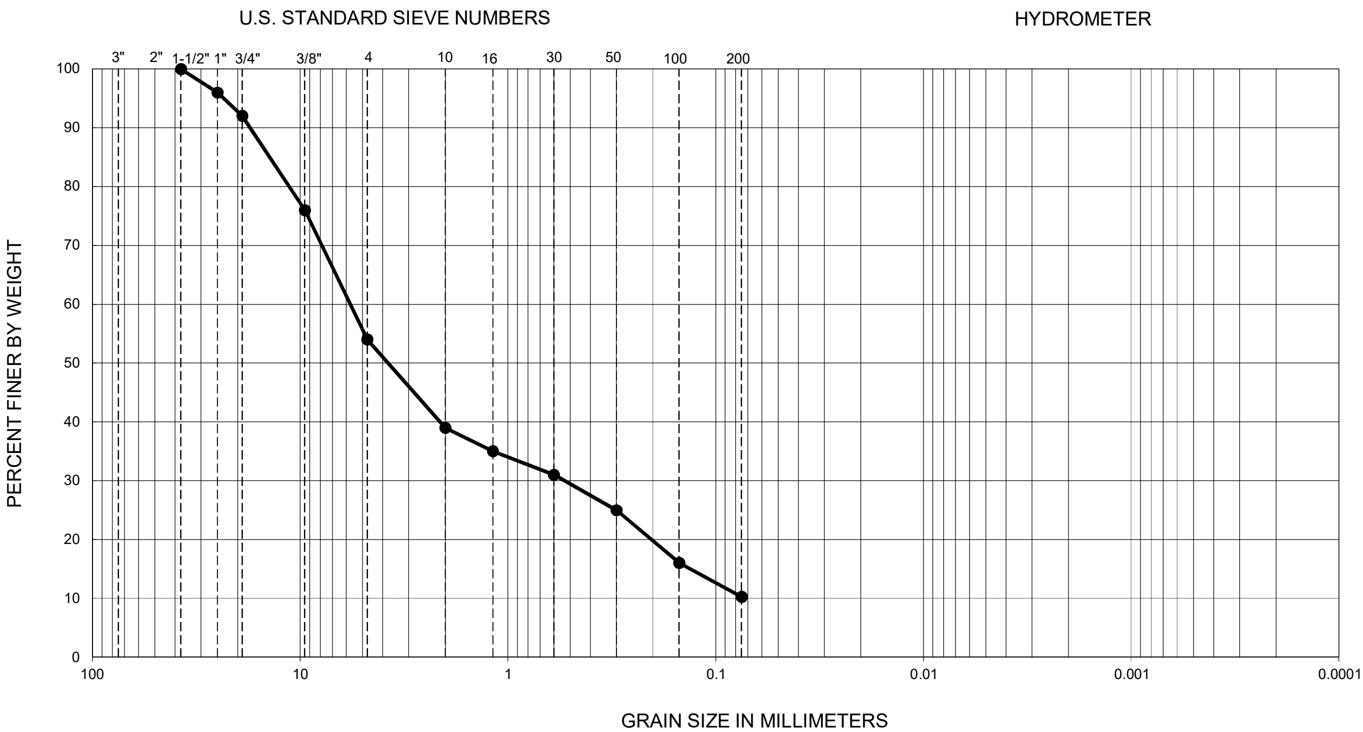
R-Value

The resistance value, or R-value, for site soils was evaluated in accordance with California Test (CT) 301. Samples were prepared and evaluated for exudation pressure and expansion pressure. The equilibrium R-value is reported as the lesser or more conservative of the two calculated results. The test results are shown on Figure B-11.

SAMPLE LOCATION	SAMPLE DEPTH (ft)	DESCRIPTION	PERCENT PASSING NO. 4	PERCENT PASSING NO. 200	USCS (TOTAL SAMPLE)
B-2	16.0-16.5	Well-graded SAND with silt and gravel	60	10	SW-SM
B-3	25.5-26.0	Silty SAND	100	45	SM

PERFORMED IN ACCORDANCE WITH 1140

GRAVEL		SAND			FINES	
Coarse	Fine	Coarse	Medium	Fine	SILT	CLAY



Symbol	Sample Location	Depth (ft)	Liquid Limit	Plastic Limit	Plasticity Index	D_{10}	D_{30}	D_{60}	C_u	C_c	Passing No. 200 (percent)	USCS
●	B-2	15.5-16.0	--	--	--	0.07	0.55	6.05	82.7	0.7	10	GP-GM

PERFORMED IN ACCORDANCE WITH ASTM D 422 / D6913

Soak Time: 2.0

Group Name: Poorly graded GRAVEL with silt and sand

% Gravel 46

% Sand 44

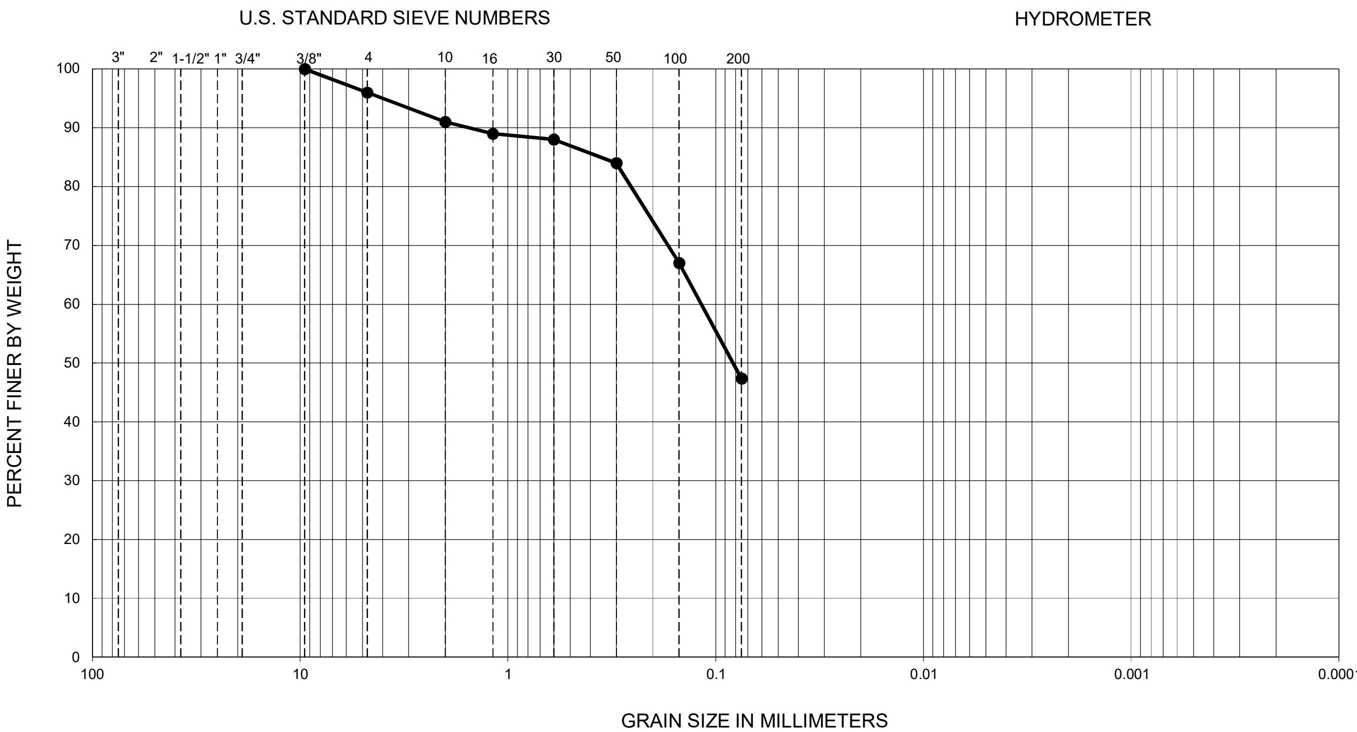
% Fines 10

FIGURE B-2

GRADATION TEST RESULTS

UKIAH COURTHOUSE BUILDING
300 EAST PERKINS STREET, UKIAH, CALIFORNIA

GRAVEL		SAND			FINES	
Coarse	Fine	Coarse	Medium	Fine	SILT	CLAY



Symbol	Sample Location	Depth (ft)	Liquid Limit	Plastic Limit	Plasticity Index	D_{10}	D_{30}	D_{60}	C_u	C_c	Passing No. 200 (percent)	USCS
●	B-2	25.0-25.5	--	--	--	--	--	0.12	--	--	47	SC

PERFORMED IN ACCORDANCE WITH ASTM D 422 / D6913

Soak Time: 2.1

Group Name: Clayey SAND

% Gravel 4

% Sand 49

% Fines 47

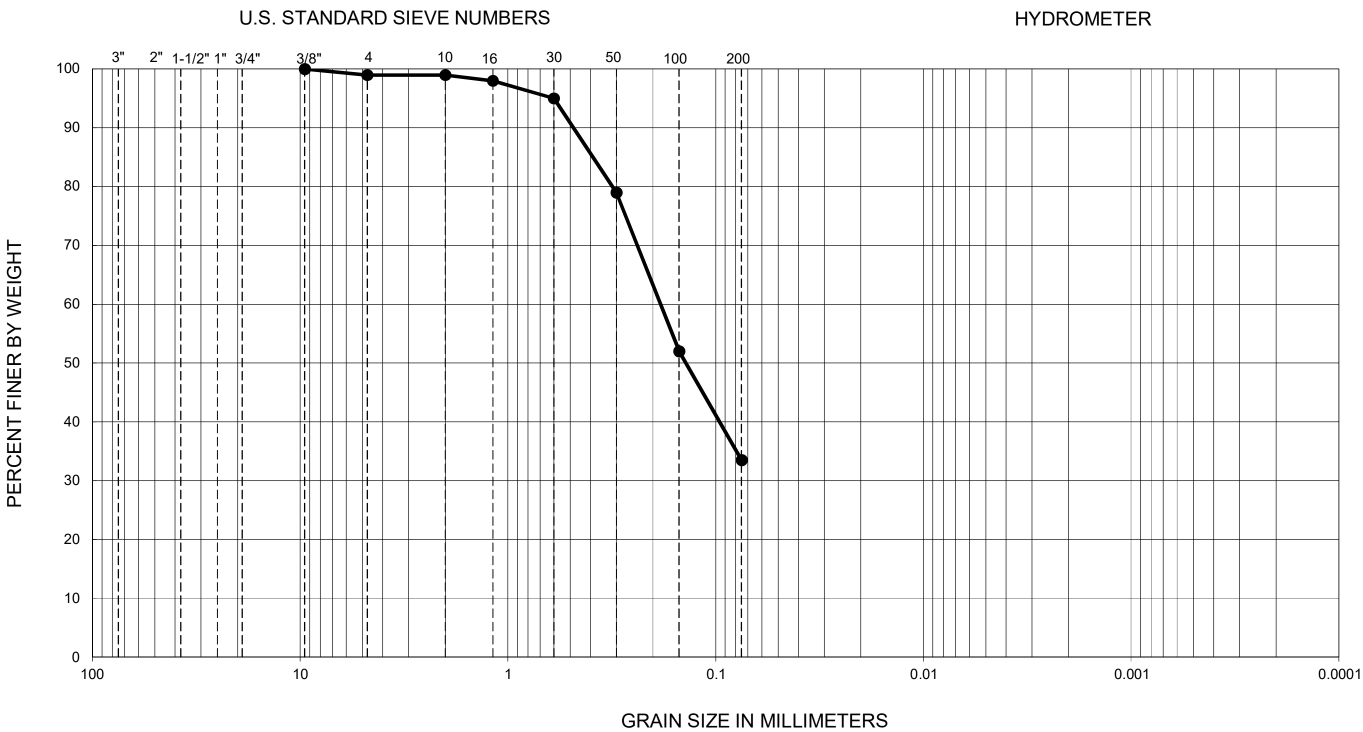
FIGURE B-3

GRADATION TEST RESULTS

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

UKIAH COURTHOUSE BUILDING
300 EAST PERKINS STREET, UKIAH, CALIFORNIA

GRAVEL		SAND			FINES	
Coarse	Fine	Coarse	Medium	Fine	SILT	CLAY



Symbol	Sample Location	Depth (ft)	Liquid Limit	Plastic Limit	Plasticity Index	D_{10}	D_{30}	D_{60}	C_u	C_c	Passing No. 200 (percent)	USCS
●	B-2	30.5-31.0	--	--	--	--	--	0.19	--	--	34	SM

PERFORMED IN ACCORDANCE WITH ASTM D 422 / D6913

Soak Time: 2.2

Group Name: Silty SAND

% Gravel 1

% Sand 65

% Fines 34

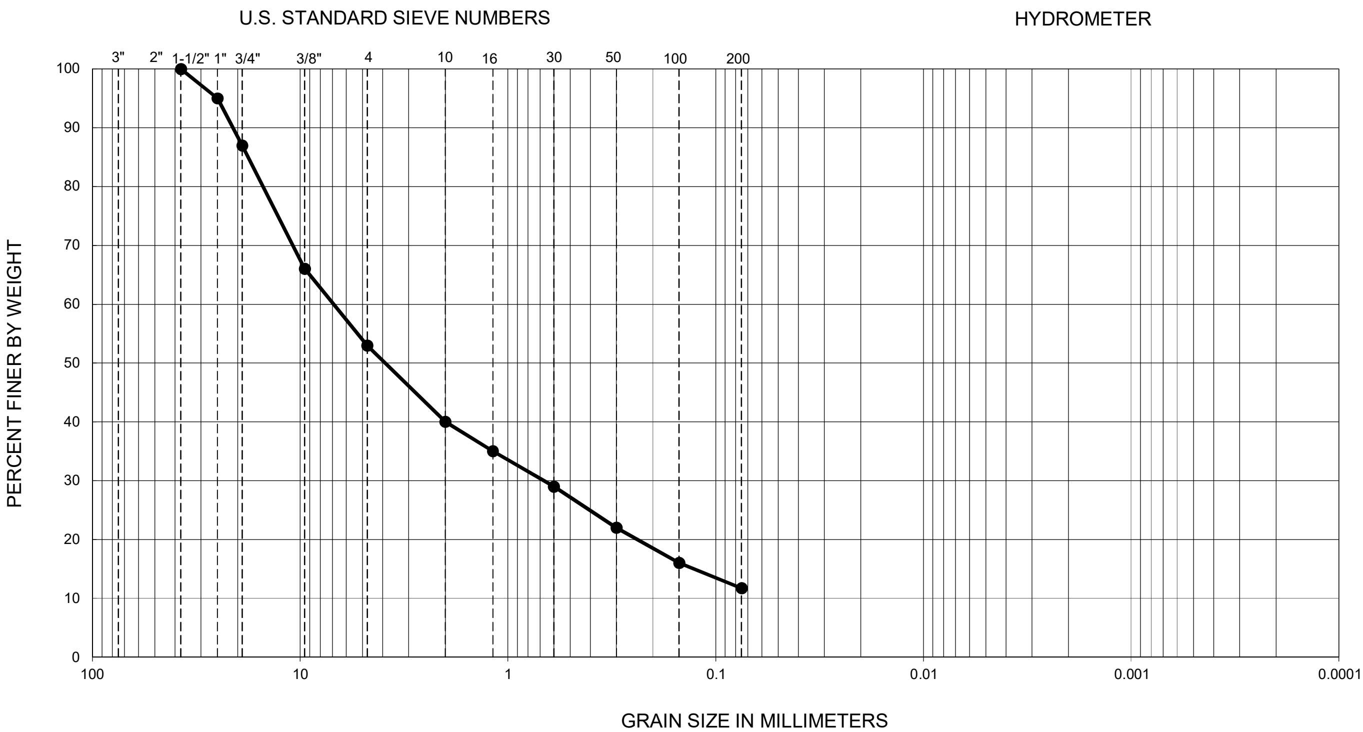
FIGURE B-4

GRADATION TEST RESULTS

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

UKIAH COURTHOUSE BUILDING
300 EAST PERKINS STREET, UKIAH, CALIFORNIA

GRAVEL		SAND			FINES	
Coarse	Fine	Coarse	Medium	Fine	SILT	CLAY



Symbol	Sample Location	Depth (ft)	Liquid Limit	Plastic Limit	Plasticity Index	D_{10}	D_{30}	D_{60}	C_u	C_c	Passing No. 200 (percent)	USCS
●	B-3	40.0-41.5	--	--	--	0.06	0.77	4.75	73.7	1.9	12	SW-SC

PERFORMED IN ACCORDANCE WITH ASTM D 422 / D6913

Soak Time: 2.3

Group Name: Well-graded SAND with clay and gravel

% Gravel 40

% Sand 48

% Fines 12

FIGURE B-5

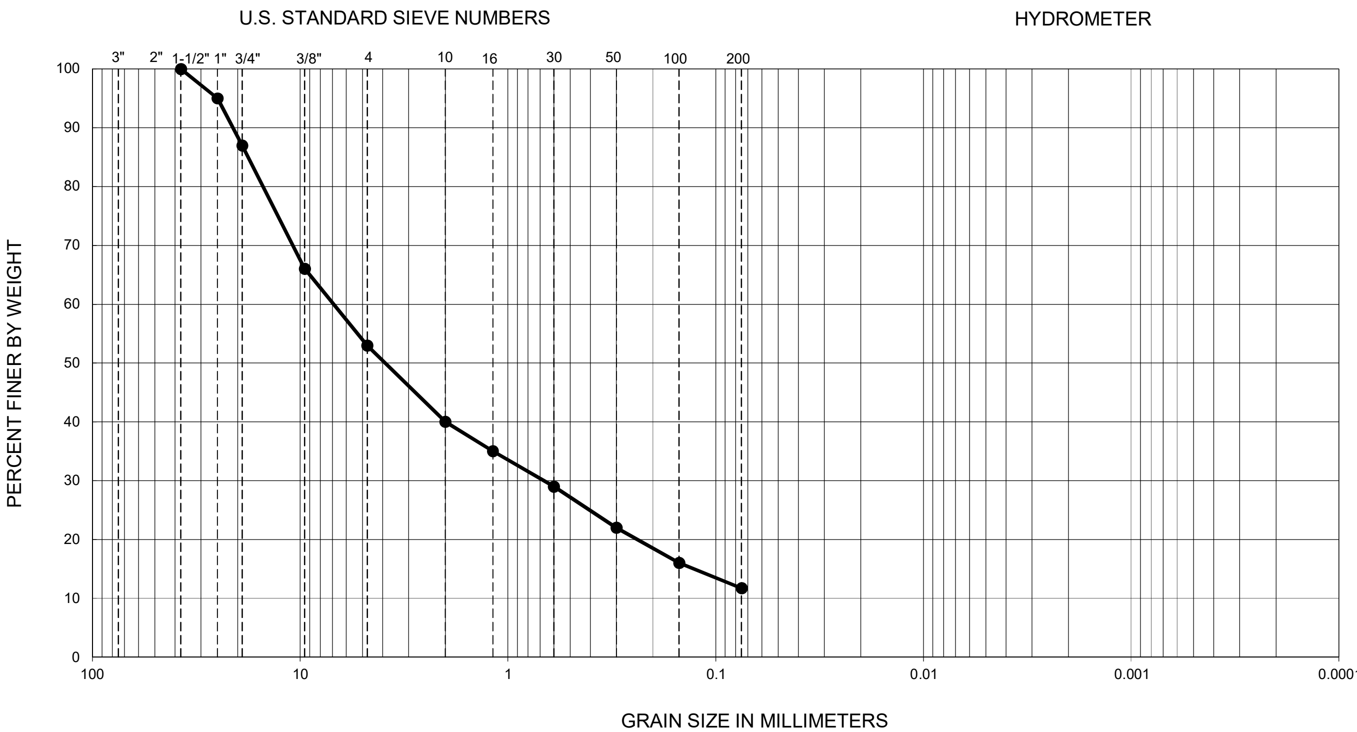
GRADATION TEST RESULTS

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

UKIAH COURTHOUSE BUILDING
300 EAST PERKINS STREET, UKIAH, CALIFORNIA

404353001 | 10/22

GRAVEL		SAND			FINES	
Coarse	Fine	Coarse	Medium	Fine	SILT	CLAY



Symbol	Sample Location	Depth (ft)	Liquid Limit	Plastic Limit	Plasticity Index	D_{10}	D_{30}	D_{60}	C_u	C_c	Passing No. 200 (percent)	USCS
●	B-4	15.0-16.5	--	--	--	0.06	0.70	7.31	114.0	1.0	12	GW-GM

PERFORMED IN ACCORDANCE WITH ASTM D 422 / D6913

Soak Time: 2.4

Group Name: Well-graded GRAVEL with silt and sand

% Gravel 47

% Sand 41

% Fines 12

FIGURE B-6

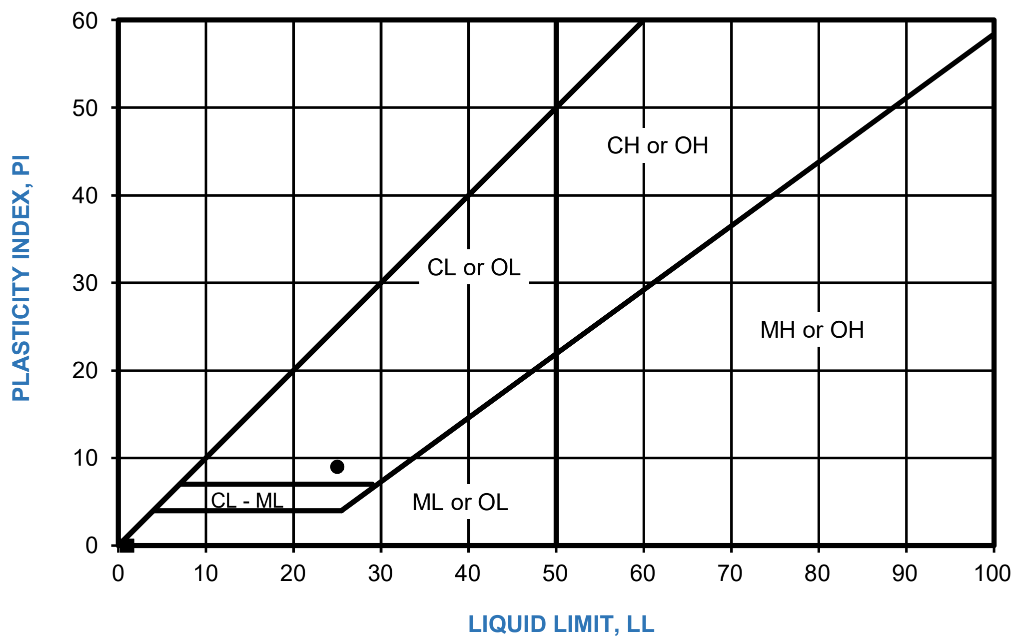
GRADATION TEST RESULTS

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

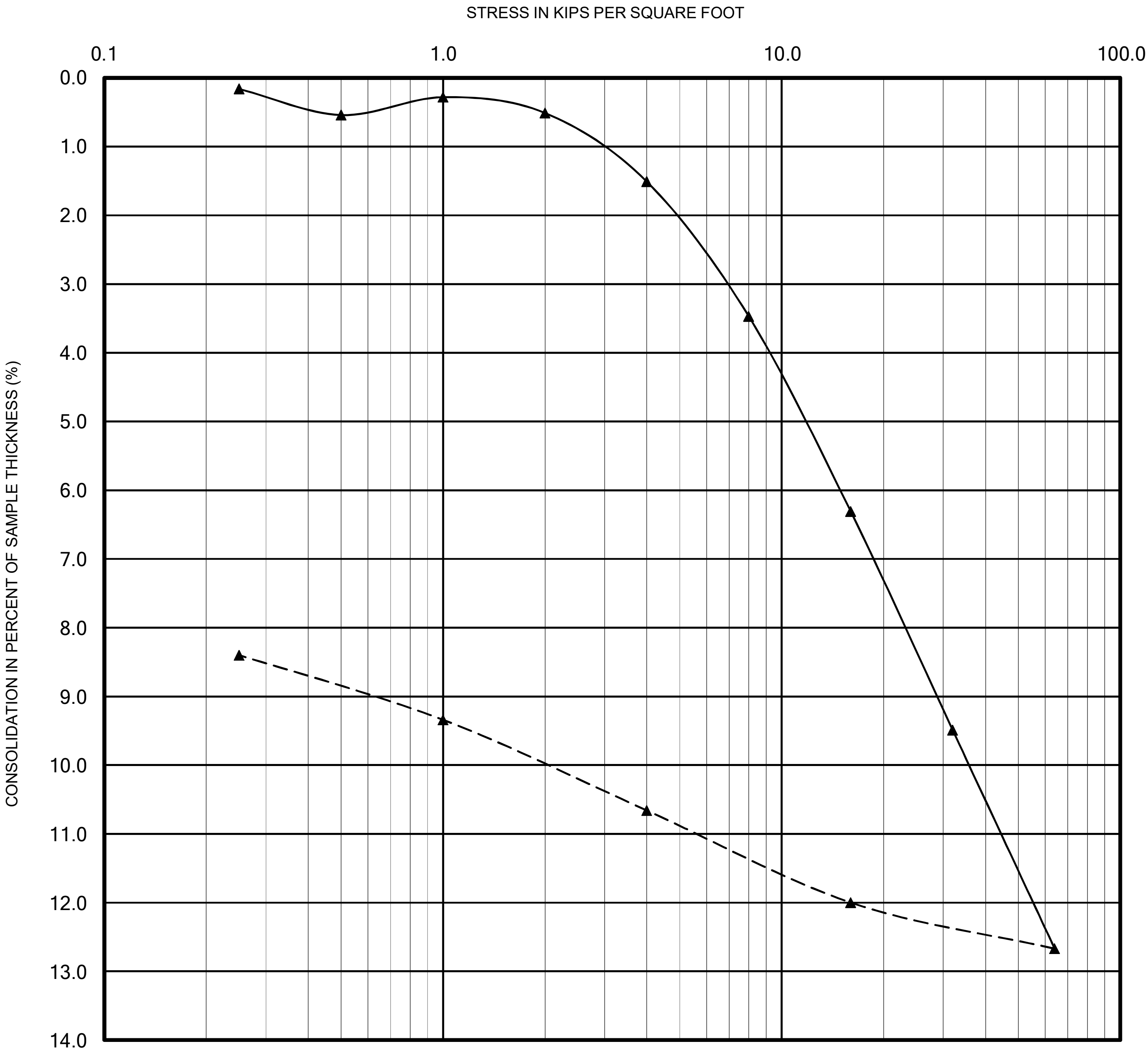
UKIAH COURTHOUSE BUILDING
300 EAST PERKINS STREET, UKIAH, CALIFORNIA

SYMBOL	LOCATION	DEPTH (ft)	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX	USCS CLASSIFICATION (Fraction Finer Than No. 40 Sieve)	USCS
●	B-2	20.0-21.5	25	16	9	CL	GC
	B-3	2.5-3.0	NP	NP	NP	ML	ML

NP - INDICATES NON-PLASTIC



PERFORMED IN ACCORDANCE WITH ASTM D 4318



Loading After Inundation		Sample Location	B-3
Rebound Cycle		Depth (ft)	6.0-6.5
		Soil Type	CL

PERFORMED IN ACCORDANCE WITH ASTM D 2435

FIGURE B-8

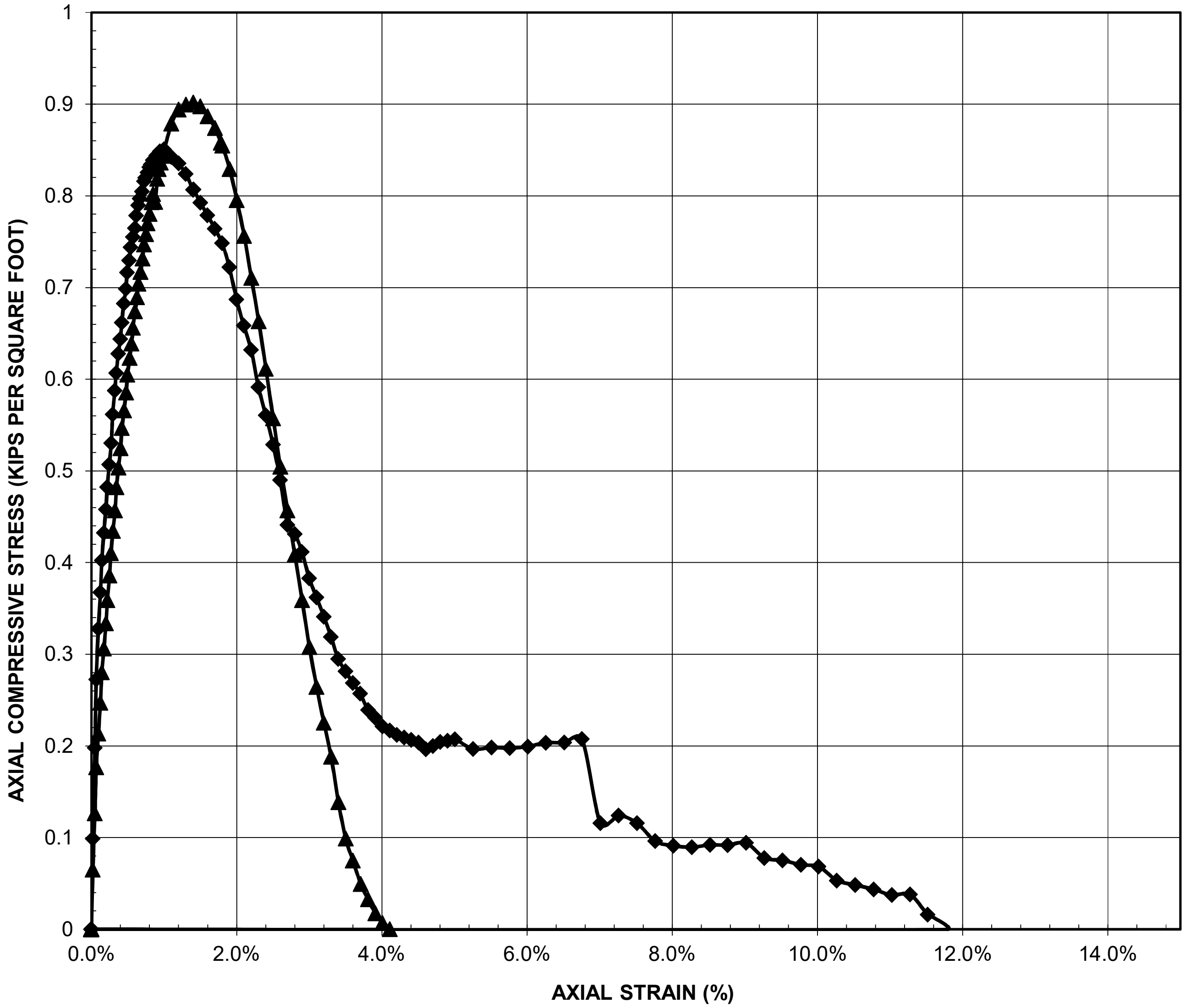
CONSOLIDATION TEST RESULTS

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

UKIAH COURTHOUSE BUILDING
300 EAST PERKINS STREET, UKIAH, CALIFORNIA

SAMPLE LOCATION	SAMPLE DEPTH (ft)	INITIAL MOISTURE (percent)	COMPACTED DRY DENSITY (pcf)	FINAL MOISTURE (percent)	VOLUMETRIC SWELL (in)	EXPANSION INDEX	POTENTIAL EXPANSION
B-2	0.0-5.0	8.1	117.4	15.7	0.019	19	Very Low

PERFORMED IN ACCORDANCE WITH ASTM D 4829



SYMBOL	DESCRIPTION	SOIL TYPE	SAMPLE LOCATION	SAMPLE DEPTH (ft.)	MOISTURE CONTENT w, (%)	DRY DENSITY γ_d, (pcf)	STRAIN RATE (%/min.)	UNDRAINED SHEAR STR s_u, (ksf)
◆	Silty CLAY	CL-ML	B-2	5.5-6.0'	12.6	103.7	1.00	0.43
▲	Sandy SILT	ML	B-3	3.0-3.5'	14.3	98.1	1.00	0.45

FIGURE B-10

UNCONFINED COMPRESSION RESULTS

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

UKIAH COURTHOUSE BUILDING
300 EAST PERKINS STREET, UKIAH, CALIFORNIA
404353001 | 10/22

SAMPLE LOCATION	SAMPLE DEPTH (ft)	SOIL TYPE	R-VALUE
B-1	0.0-5.0	Sandy SILT	52.0

PERFORMED IN ACCORDANCE WITH ASTM D 2844/CT 301

APPENDIX C

Corrosivity Testing (CERCO Analytical)

CERCO analytical
1100 Willow Pass Court, Suite A
Concord, CA 94520-1006
925 **462 2771** Fax. 925 **462 2775**
www.cercoanalytical.com

29 September, 2022

Job No. 2209046
Cust. No.13270

Mr. Rathna Mothkuri
Ninyo & Moore
2149 O'Toole Avenue, Suite 30
San Jose, CA 95131

Subject: Project No.: 404353001
Project Name: Ukiah Courthouse Building, 300 East Perkins Street, Ukiah, CA
Corrosivity Analysis – ASTM Test Methods

Dear Mr. Mothkuri:

Pursuant to your request, CERCO Analytical has analyzed the soil sample submitted on September 22, 2022. Based on the analytical results, this brief corrosivity evaluation is enclosed for your consideration.

Based upon the resistivity measurement, this sample is classified as "moderately corrosive". All buried iron, steel, cast iron, ductile iron, galvanized steel and dielectric coated steel or iron should be properly protected against corrosion depending upon the critical nature of the structure. All buried metallic pressure piping such as ductile iron firewater pipelines should be protected against corrosion.

The chloride ion concentration reflects none detected with a reporting limit of 15 mg/kg.

The sulfate ion concentration is 19 mg/kg and is determined to be insufficient to damage reinforced concrete structures and cement mortar-coated steel at this location.

The pH of the soil is 7.61, which does not present corrosion problems for buried iron, steel, mortar-coated steel and reinforced concrete structures.

The redox potential is 330-mV and is indicative of potentially "slightly corrosive" soils resulting from anaerobic soil conditions.

This corrosivity evaluation is based on general corrosion engineering standards and is non-specific in nature. For specific long-term corrosion control design recommendations or consultation, please call *JDH Corrosion Consultants, Inc. at (925) 927-6630.*

We appreciate the opportunity of working with you on this project. If you have any questions, or if you require further information, please do not hesitate to contact us.

Very truly yours,
CERCO ANALYTICAL, INC.

Sheri Moore

for, J. Darby Howard, Jr., P.E.
President

JDH/jdl
Enclosure

CERCO analytical
1100 Willow Pass Court, Suite A
Concord, CA 94520-1006
925 462 2771 Fax. 925 462 2775
www.cercoanalytical.com

Client: Ninyo & Moore
Client's Project No.: 404353001
Client's Project Name: Ukiah Courthouse Building, 300 East Perkings Street, Ukiah, CA
Date Sampled: 14-Sep-22
Date Received: 22-Sep-22
Matrix: Soil
Authorization: Signed Chain of Custody

Date of Report: 29-Sep-2022

Job/Sample No.	Sample I.D.	Redox (mV)	pH	Conductivity (umhos/cm)*	Resistivity (100% Saturation) (ohms-cm)	Sulfide (mg/kg)*	Chloride (mg/kg)*	Sulfate (mg/kg)*
2209046-001	B-4/0.0-5.0'	330	7.61	-	2,900	-	N.D.	19

	Redox	pH	Conductivity	Resistivity	Sulfide	Chloride	Sulfate
Method:	ASTM D1498	ASTM D4972	ASTM D1125M	ASTM G57	ASTM D4658M	ASTM D4327	ASTM D4327
Reporting Limit:	-	-	10	-	50	15	15
Date Analyzed:	27-Jun-2022	27-Sep-2022	-	27-Sep-2022	-	28-Sep-2022	28-Sep-2022

Sherri Moore
Sherri Moore
Chemist

* Results Reported on "As Received" Basis
N.D. - None Detected

Quality Control Summary - All laboratory quality control parameters were found to be within established limits

APPENDIX D

Geophysical Survey

APPENDIX D

GEOPHYSICAL SURVEY

Seismic Site Classification

A seismic survey using passive surface wave techniques was performed at the site on September 14, 2022 (Figure 2). The purpose of the study was to evaluate the subsurface shear-wave velocity at a representative location. The passive seismic method carried out included Micro-tremor Array Measurements (MAM) and consisted of two single linear profiles of seismic data collection. The method provided a shear wave velocity model to a depth of approximately 100 feet below the ground surface (bgs) and Vs_{100} for seismic site classification (CBC, 2019). The following sections provide a summary of the methods and analyses used in our study. The seismic model results are provided in Figure D-1.

Field Methods

A Geode 24–Channel Seismograph (Geometrics Inc., San Jose, CA) was used for MAM surveying, with 4.5 Hertz (Hz) vertical component geophone placement every 10 feet for a total profile length of 230 feet. Approximately twenty records were collected, with a record length of 30 seconds (s) and 2 millisecond (ms) sample interval. The field data were digitally recorded in SEG2 format, reviewed in the field for data quality, saved to a hard disk, and documented.

Data Processing and Modeling

The MAM seismic data were processed using SeisImager (Geometrics Inc., San Jose, CA) seismic processing software. The dispersive characteristics of surface waves are used to evaluate the subsurface velocity at depth. Longer wavelength (that is, longer-period and lower-frequency) surface waves travel deeper and thus contain more information about deeper velocity structure. Shorter wavelength (that is, shorter-period and higher-frequency) surface waves travel shallower and thus contain more information about shallower velocity structure. The dispersion is dependent on the material properties, such as surface wave velocity, relative material densities, and Poisson's ratio. An inversion is performed on the collected passive seismic shear wave records within SeisImager to produce a model of the variation in shear wave velocities with depth. The following data processing flow was used to calculate Average Shear-wave Velocities (AVS) to a depth of approximately 100 feet (V_s100).

- Collated records into list file and edited any bad channels or records,
- Applied 2D Spatial Auto Correlation (SPAC); using a linear array and 24 geophones at 10 feet spacing,
- Phase velocity frequency transformation from 2 to 20 Hz,

- Automated velocity picks of raw phase velocity were calculated and updated manually,
- Created an initial model and carried out a non-linear Least Squares Method (LSM) inversion to produce a final shear wave velocity model; convergence of the inversion was judged whether the model achieved an RMS <5% within 5-7 iterations,
- Calculated V_s100 using final shear wave velocity model.

Results

Shear wave data resolution generally decreases with depth, due to the loss of sensitivity of the dispersion curve to changes in shear wave velocity as depth increases. Figure showing our MAM seismic modeling results is provided in Figures D-1. The layered model in Figure D-1 indicates our interpretation of the approximate changes in shear wave velocity vertically with depth across the surveyed location.

The model results indicate Vs_{100} value of **1032 feet/sec** and a Seismic Site Classification of **Class D** accordingly.

Figure D-1 404353001 : Vs Model, Line 1

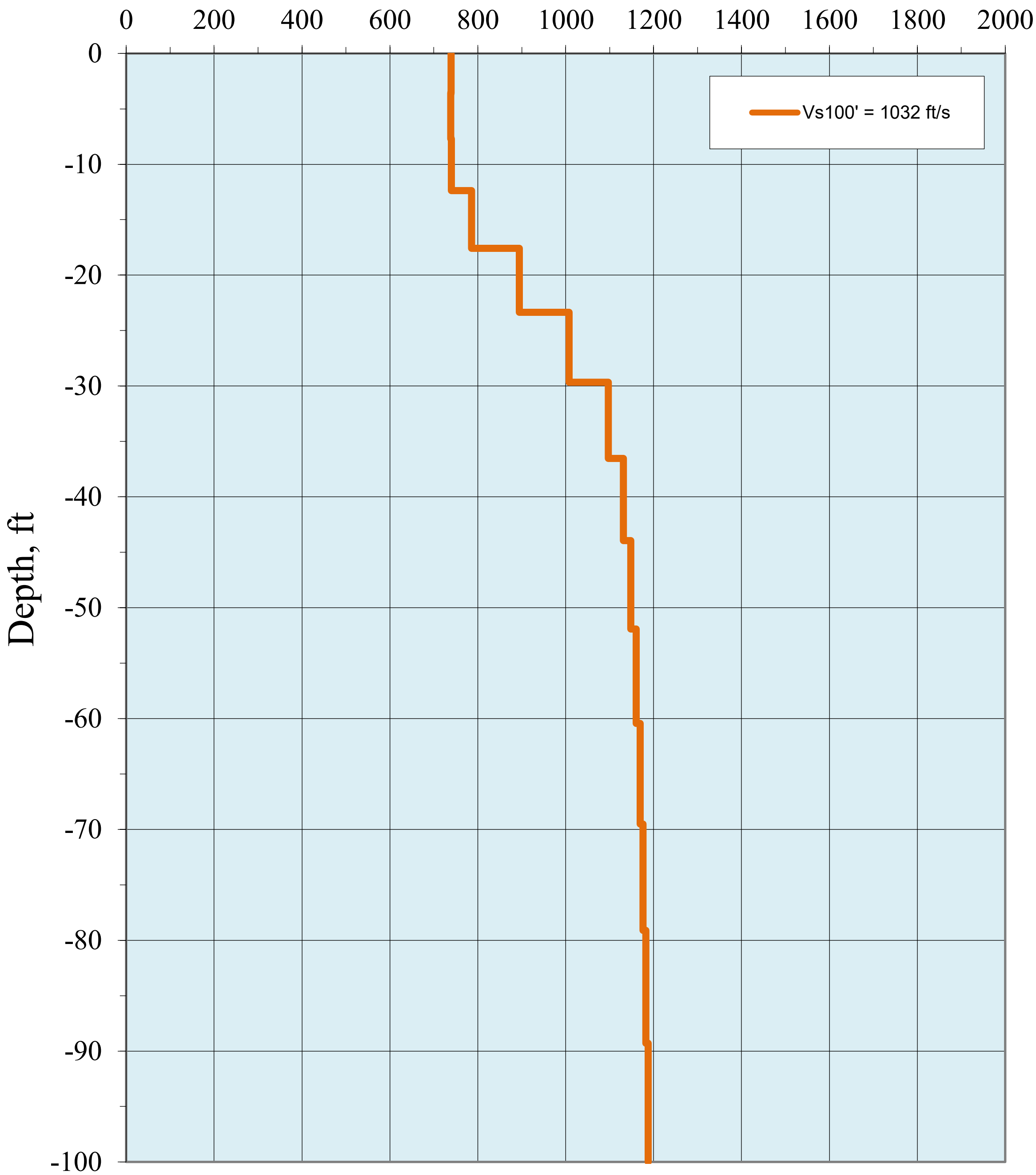


Figure D-1: Shear-Wave Velocity, ft/s

APPENDIX E

Site-Specific Seismic Calculations

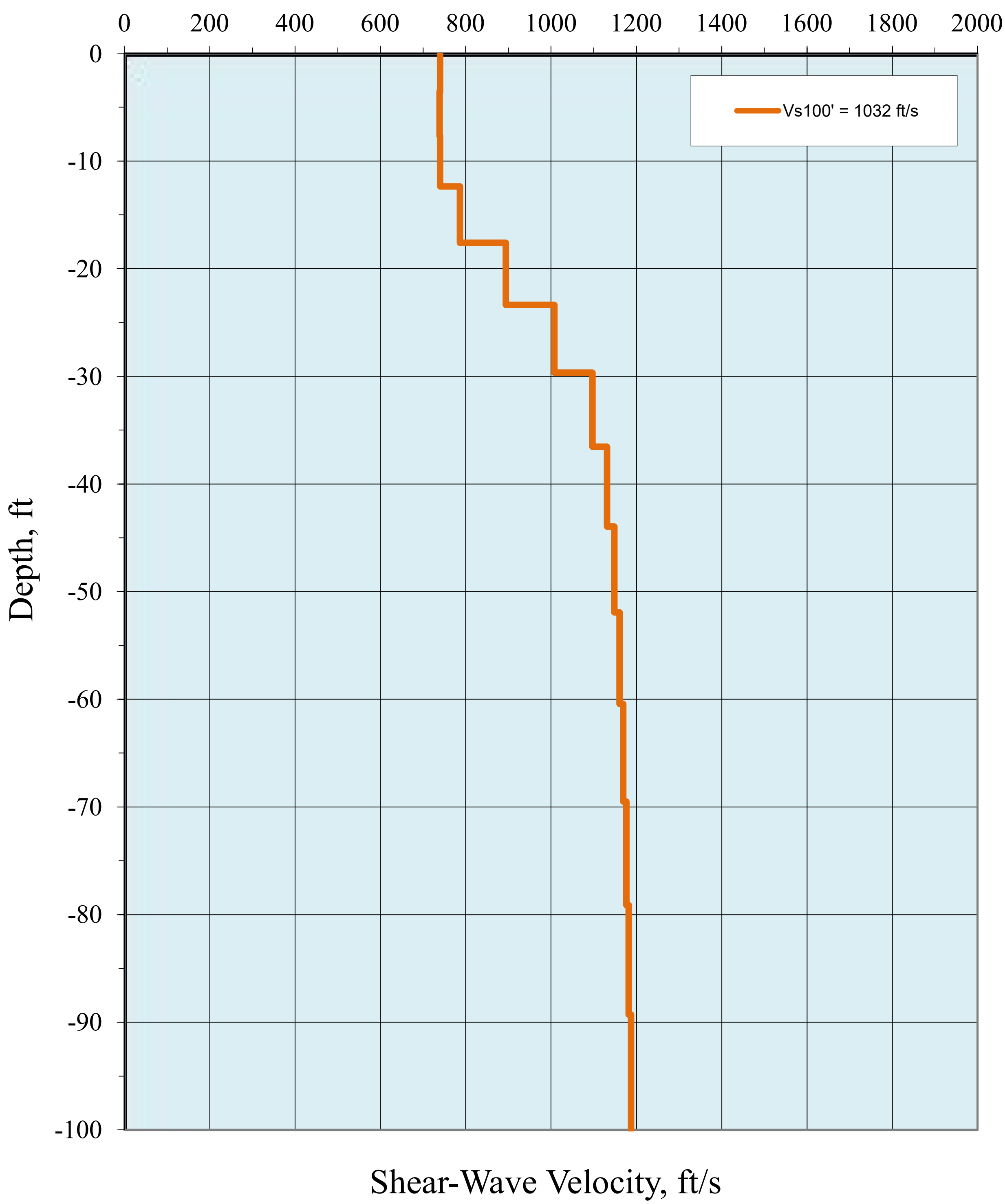
Maacama 2011 CFM	CASE A		
Fault ID	66		Caltrans_Fault_Database Excel Sheet
Maximum Magnitute (Mmax)	7.4		Caltrans_Fault_Database Excel Sheet
Fault Type	strike slip		Caltrans_Fault_Database Excel Sheet
Fault Dip	63	degree	Caltrans_Fault_Database Excel Sheet
Dip Direction	E		Caltrans_Fault_Database Excel Sheet
Bottom of rupture plane	9.4	km	Caltrans_Fault_Database Excel Sheet
Top of Rupture Plane (Ztor)	0	km	Caltrans_Fault_Database Excel Sheet
Fault Strike Heading	#N/A	degrees	Google Earth
Azimuth alpha heading	#N/A	degrees	Google Earth
Azimuth, alpha (see figure)	-90	degrees	
Rx (negative if opposite direction from dip)	-2.01	km	Google Earth
Map width of rupture plane, Wm	4.79	km	
Width of rupture plane, W	10.55	km	
Rjb	2.01	km	
Rrup1	2.01	km	
Ry	0	km	
Rrup	2.01	km	
Fnorm	0		
Frev	0		
Fhw	0		

San Andreas (North Coast) 2011 CFM	CASE B		
Fault ID	80		Caltrans_Fault_Database Excel Sheet
Maximum Magnitute (Mmax)	8		Caltrans_Fault_Database Excel Sheet
Fault Type	Strike Slip		Caltrans_Fault_Database Excel Sheet
Fault Dip	90	degree	Caltrans_Fault_Database Excel Sheet
Dip Direction	V		Caltrans_Fault_Database Excel Sheet
Bottom of rupture plane	11.1	km	Caltrans_Fault_Database Excel Sheet
Top of Rupture Plane (Ztor)	0	km	Caltrans_Fault_Database Excel Sheet
Fault Strike Heading	#N/A	degrees	Google Earth
Azimuth alpha heading	#N/A	degrees	Google Earth
Azimuth, alpha (see figure)	90	degrees	
Rx (negative if opposite direction from dip)	44.17	km	Google Earth
Map width of rupture plane, Wm	0.00	km	
Width of rupture plane, W	11.10	km	
Rjb	44.17	km	
Rrup1	45.54	km	
Ry	0	km	
Rrup	44.17	km	
Fnorm	0		1 for normal, 0 otherwise
Frev	0		1 for thrust/reverse, 0 otherwise
Fhw	0		1 for site on hanging wall side (dip direction s

Bartlett Springs 2011 CFM

	CASE C		
Fault ID	53		Caltrans_Fault_Database Excel Sheet
Maximum Magnitute (Mmax)	7.2		Caltrans_Fault_Database Excel Sheet
Fault Type	Strike Slip		Caltrans_Fault_Database Excel Sheet
Fault Dip	90	degree	Caltrans_Fault_Database Excel Sheet
Dip Direction	V		Caltrans_Fault_Database Excel Sheet
Bottom of rupture plane	11.8	km	Caltrans_Fault_Database Excel Sheet
Top of Rupture Plane (Ztor)	0	km	Caltrans_Fault_Database Excel Sheet
Fault Strike Heading	#N/A	degrees	Google Earth
Azimuth alpha heading	#N/A	degrees	Google Earth
Azimuth, alpha (see figure)	90	degrees	
Rx (negative if opposite direction from dip)	37.18	km	Google Earth
Map width of rupture plane, Wm	0.00	km	
Width of rupture plane, W	11.80	km	
Rjb	37.18	km	
Rrup1	39.01	km	
Ry	0	km	
Rrup	37.18	km	
Fnorm	0		
Frev	0		
Fhw	0		

404353001 : Vs Model, Line 1



U.S. Geological Survey - Earthquake Hazards Program

Unified Hazard Tool

Please do not use this tool to obtain ground motion parameter values for the design code reference documents covered by the U.S. Seismic Design Maps web tools (e.g., the International Building Code and the ASCE 7 or 41 Standard). The values returned by the two applications are not identical.

Input

Edition

Dynamic: Conterminous U.S. 2014 (u…

Latitude

Decimal degrees

39.14925

Longitude

Decimal degrees, negative values for western longitudes

-123.20297

Site Class

259 m/s (Site class D)

Spectral Period

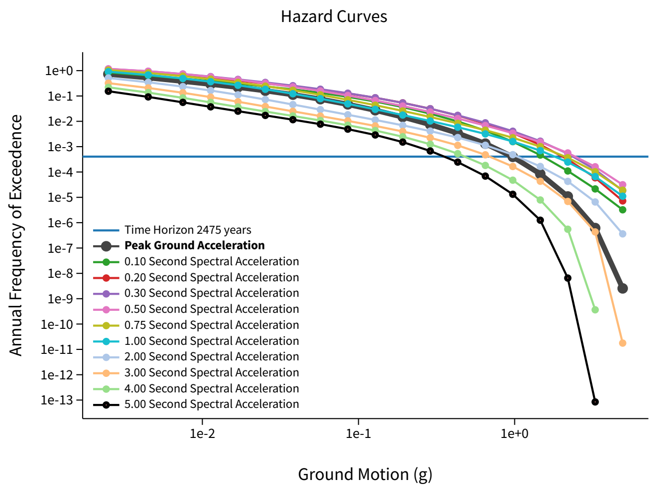
Peak Ground Acceleration

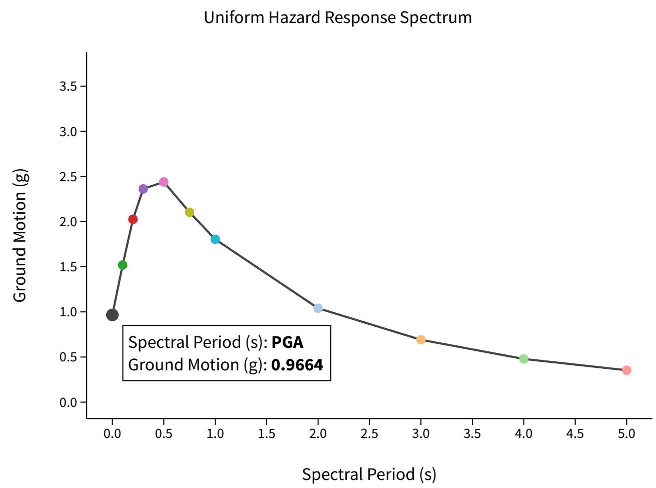
Time Horizon

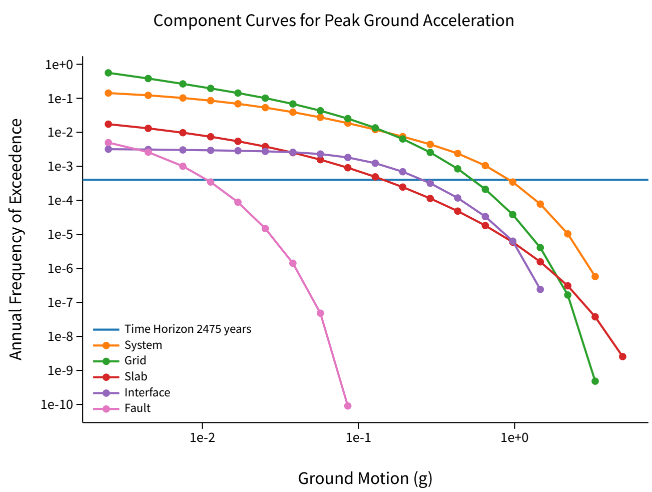
Return period in years

2475

Hazard Curve





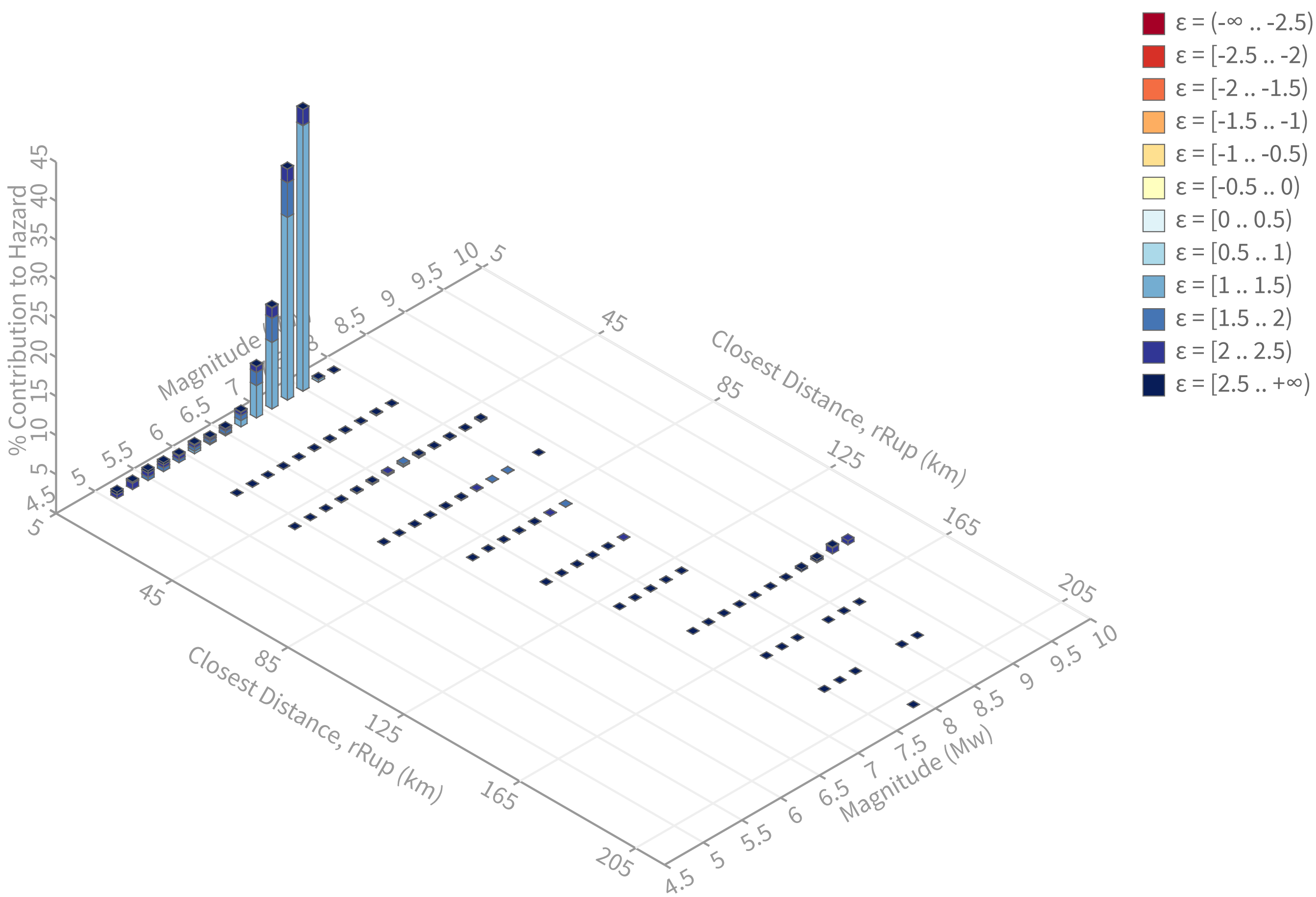


View Raw Data

Deaggregation

Component

Total



Summary statistics for, Deaggregation: Total

Deaggregation targets

Return period: 2475 yrs
Exceedance rate: 0.0004040404 yr^{-1}
PGA ground motion: 0.96642191 g

Recovered targets

Return period: 3143.7409 yrs
Exceedance rate: 0.00031809237 yr^{-1}

Totals

Binned: 100 %
Residual: 0 %
Trace: 0.28 %

Mean (over all sources)

m: 7.22
r: 8.07 km
ε_0: 1.44 σ

Mode (largest m-r bin)

m: 7.46
r: 3.4 km
ε_0: 1.28 σ
Contribution: 36.06 %

Mode (largest m-r-ε_0 bin)

m: 7.46
r: 3.4 km
ε_0: 1.23 σ
Contribution: 33.99 %

Discretization

r: min = 0.0, max = 1000.0, Δ = 20.0 km
m: min = 4.4, max = 9.4, Δ = 0.2
ε: min = -3.0, max = 3.0, Δ = 0.5 σ

Epsilon keys

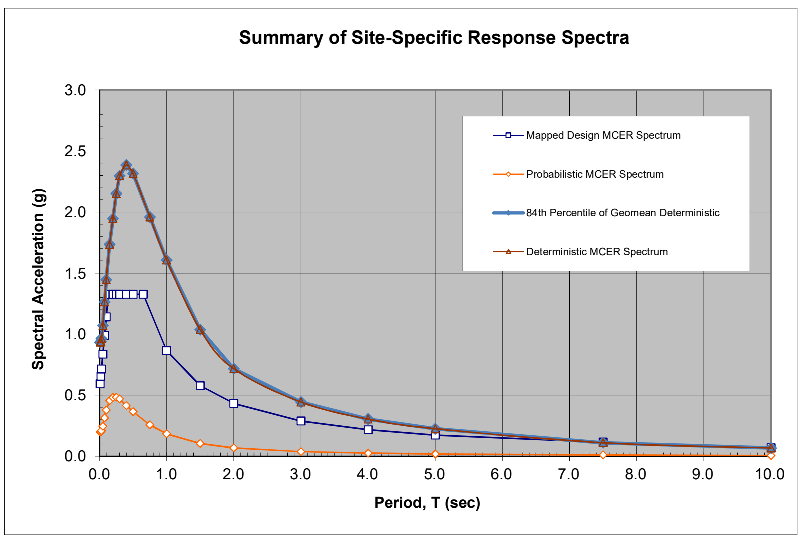
ε0: [-∞ .. -2.5)
ε1: [-2.5 .. -2.0)
ε2: [-2.0 .. -1.5)
ε3: [-1.5 .. -1.0)
ε4: [-1.0 .. -0.5)
ε5: [-0.5 .. 0.0)
ε6: [0.0 .. 0.5)
ε7: [0.5 .. 1.0)
ε8: [1.0 .. 1.5)
ε9: [1.5 .. 2.0)
ε10: [2.0 .. 2.5)
ε11: [2.5 .. +∞]

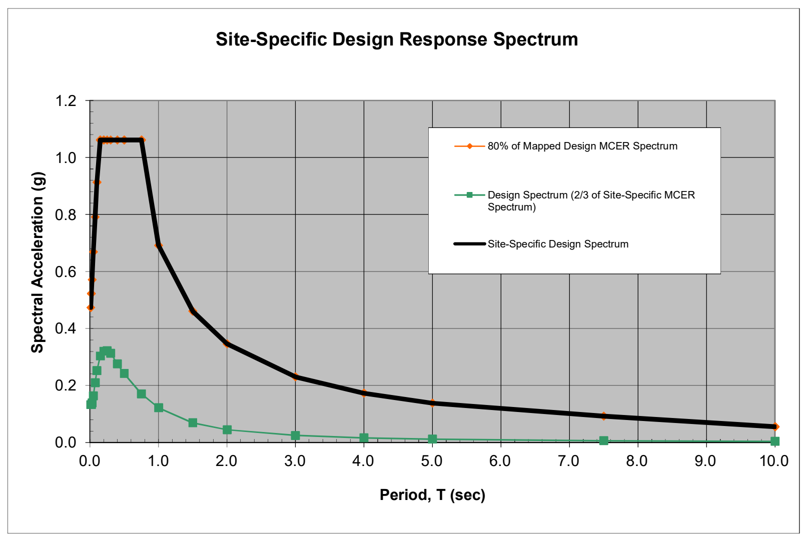
Deaggregation Contributors

Source Set ↳ Source	Type	r	m	ε_0	lon	lat	az	%
UC33brAvg_FM32	System							44.06
Maacama [14]		3.39	7.32	1.31	123.169°W	39.161°N	65.39	40.64
Maacama [13]		4.33	6.91	1.50	123.156°W	39.141°N	103.37	1.37
UC33brAvg_FM31	System							43.09
Maacama [14]		3.39	7.33	1.31	123.169°W	39.161°N	65.39	39.95
Maacama [13]		4.33	6.90	1.50	123.156°W	39.141°N	103.37	1.24
UC33brAvg_FM31 (opt)	Grid							4.45
PointSourceFinite: -123.203, 39.199		7.36	5.69	2.05	123.203°W	39.199°N	0.00	1.31
PointSourceFinite: -123.203, 39.199		7.36	5.69	2.04	123.203°W	39.199°N	0.00	1.22
UC33brAvg_FM32 (opt)	Grid							4.45
PointSourceFinite: -123.203, 39.199		7.36	5.69	2.05	123.203°W	39.199°N	0.00	1.31
PointSourceFinite: -123.203, 39.199		7.36	5.69	2.04	123.203°W	39.199°N	0.00	1.21

Site Class	Mapped Spectral Response Acceleration Parameters		Site Coefficients		Spectral Response Acceleration Parameters Adjusted for Site Effects		Design Spectral Response Acceleration Parameters		Period			Vs30 (m/sec)	Risk Coefficients			PGA		Site-Specific MCE_G PGA
	Ss (g)	S1 (g)	Fa	Fv	Sms (g)	Sm1 (g)	Sds (g)	Sd1 (g)	To (sec)	Ts (sec)	TL (sec)		C_{RS}	C_{R1}	Ratio	F_{PGA}	PGA_M (g)	PGA_M (g)
D	1.99	0.763	1.000	1.700	1.990	1.297	1.327	0.865	0.130	0.652	8.0	314	0.883	0.877	-0.0075	1.1	0.919	0.735
Det. Limit			0.800													0.8		
80% Design	1.000	2.500	1.000	1.700	1.990	1.297	1.327	0.865	**0.130**	**0.652**								
D (stiff soil profile)		4.000																

Mapped Design MCE_R Response Spectrum (Fa and Fv per Section 11.4 of ASCE 7-16)		Period (sec)	Probabilistic MCE_R Response Spectrum				Deterministic MCE_R Response Spectrum				Site-Specific MCE_R Response Spectrum	Site-Specific MCE_R Response Spectrum - 150% Limit of Design	Mapped Design MCE_R Response Spectrum (Fa and Fv per Section 21.3 of ASCE 7-16)	80% of Mapped Design MCE_R Response Spectrum (Fa and Fv per Section 21.3 of ASCE 7-16)	Site-Specific Design Response Spectrum	Site-Specific Design Response Spectrum with 80% Limit
			Geomean 2% in 50 Years	Max Horiz Direction Response to Geomean	Geomean 2% in 50 Years Rotated	1% Chance of Collapse in 50 Years (Method 1)	84th Percentile of Geomean	Max Horiz Direction Response to Geomean	84th Percentile of Geomean Rotated	Deterministic Limit on Response Spectrum						
Period (sec)	Sa (g)		Sa (g)	Max/Mean	Sa (g)	Sa (g)	Sa (g)	Max/Mean	Sa (g)	Sa (g)	Sa (g)	Sa (g)	Sa (g)	Sa (g)	Sa (g)	Sa (g)
0.01	0.592	0.01	0.205	1.100	0.225	0.199	0.848	1.100	0.932	0.932	0.199	0.710	0.592	0.473	0.133	**0.473**
0.02	0.653	0.02	0.205	1.100	0.226	0.199	0.854	1.100	0.940	0.940	0.199	0.783	0.653	0.522	0.133	**0.522**
0.03	0.714	0.03	0.215	1.100	0.236	0.209	0.875	1.100	0.962	0.962	0.209	0.857	0.714	0.571	0.139	**0.571**
0.05	0.836	0.05	0.253	1.100	0.279	0.246	0.973	1.100	1.070	1.070	0.246	1.003	0.836	0.669	0.164	**0.669**
0.075	0.989	0.075	0.324	1.100	0.356	0.314	1.147	1.100	1.262	1.262	0.314	1.186	0.989	0.791	0.210	**0.791**
0.1	1.141	0.1	0.389	1.100	0.428	0.378	1.313	1.100	1.444	1.444	0.378	1.370	1.141	0.913	0.252	**0.913**
0.130	1.327	0.15	0.469	1.100	0.516	0.455	1.576	1.100	1.733	1.733	0.455	1.592	1.327	1.061	0.304	**1.061**
0.2	1.327	0.2	0.495	1.100	0.544	0.480	1.769	1.100	1.946	1.946	0.480	1.592	1.327	1.061	0.320	**1.061**
0.25	1.327	0.25	0.492	1.113	0.547	0.483	1.934	1.113	2.151	2.151	0.483	1.592	1.327	1.061	0.322	**1.061**
0.3	1.327	0.3	0.473	1.125	0.532	0.469	2.041	1.125	2.296	2.296	0.469	1.592	1.327	1.061	0.313	**1.061**
0.4	1.327	0.4	0.409	1.150	0.470	0.414	2.074	1.150	2.385	2.385	0.414	1.592	1.327	1.061	0.276	**1.061**
0.5	1.327	0.5	0.352	1.175	0.413	0.364	1.970	1.175	2.315	2.315	0.364	1.592	1.327	1.061	0.243	**1.061**
0.652	1.327	0.75	0.235	1.238	0.291	0.255	1.583	1.238	1.959	1.959	0.255	1.592	1.327	1.061	0.170	**1.061**
1	0.865	1	0.161	1.300	0.209	0.183	1.236	1.300	1.607	1.607	0.183	1.038	0.865	0.692	0.122	**0.692**
1.5	0.576	1.5	0.090	1.325	0.119	0.104	0.782	1.325	1.036	1.036	0.104	0.692	0.576	0.461	0.069	**0.461**
2	0.432	2	0.057	1.350	0.077	0.067	0.530	1.350	0.716	0.716	0.067	0.519	0.432	0.346	0.045	**0.346**
3	0.288	3	0.030	1.400	0.042	0.037	0.317	1.400	0.443	0.443	0.037	0.346	0.288	0.231	0.025	**0.231**
4	0.216	4	0.019	1.450	0.028	0.025	0.209	1.450	0.303	0.303	0.025	0.259	0.216	0.173	0.016	**0.173**
5	0.173	5	0.014	1.500	0.020	0.018	0.150	1.500	0.225	0.225	0.018	0.208	0.173	0.138	0.012	**0.138**
7.5	0.115	7.5	0.007	1.500	0.011	0.009	0.074	1.500	0.110	0.110	0.009	0.138	0.115	0.092	0.006	**0.092**
10	0.069	10	0.004	1.500	0.006	0.006	0.044	1.500	0.066	0.066	0.006	0.083	0.069	0.055	0.004	**0.055**





APPENDIX F

Boring Logs and Laboratory Test Results from Previous Evaluation (PSI, 2011)

BORING GB-1

CLIENT: WESTON SOLUTIONS	PSI PROJECT NO.: 575-249
PROJECT: FORMER UKIAH STATION	BORING TYPE: 8-INCH DIA. H.S.A. w/AUTO-HAMMER
LOCATION: 309 E. PERKINS STREET, UKIAH, CALIFORNIA	ELEVATION: EXISTING GRADE
DATE DRILLED: JANUARY 6, 2011	LOGGED BY: FRANK POSS

DEPTH (ft)	SAMPLE NO.	SAMPLE	SOIL DESCRIPTION	BLOWS PER FOOT	% PASSING #200 SIEVE	LIQUID LIMIT	PLASTIC LIMIT	MOISTURE CONTENT (%)	UNIT DRY WT. (p.c.f.)	REMARKS
	SS-1		Clayey SILT (ML), brown, very soft, moist, some sand and gravel	8				21	104	Torvane = 0.2 tsf.
5										
	SP-2		Silty CLAY (CL), green-brown, very soft, wet	5		24	17	16		Groundwater at 6 feet. Torvane = 0.2 tsf.
	SS-3		SAND (SP), brown, medium dense, wet, fine to coarse sand, some gravel	19						
10										
	SP-4		Gravelly SAND (SP), red brown, dense, wet, fine to very coarse sand, some silt	31				11		
15										
	SP-5		GRAVEL (GP), dark brown, dense, wet, some sand and silt	35						
20										
	SP-6		Silty GRAVEL (GM), light brown, very dense, wet, some clay and sand	54						
25										

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FIGURE NO.
A-1a

BORING GB-1 (cont.)

CLIENT: WESTON SOLUTIONS	PSI PROJECT NO.: 575-249
PROJECT: FORMER UKIAH STATION	BORING TYPE: 8-INCH DIA. H.S.A. w/AUTO-HAMMER
LOCATION: 309 E. PERKINS STREET, UKIAH, CALIFORNIA	ELEVATION: EXISTING GRADE
DATE DRILLED: JANUARY 6, 2011	LOGGED BY: FRANK POSS

DEPTH (ft)	SAMPLE NO.	SAMPLE	SOIL DESCRIPTION	BLOWS PER FOOT	% PASSING #200 SIEVE	LIQUID LIMIT	PLASTIC LIMIT	MOISTURE CONTENT (%)	UNIT DRY WT. (p.c.f.)	REMARKS
	SP-7		SILT (ML), brown, very soft, moist, some very fine grained sand and clay	8						Torvane = 0.2 tsf.
30										
	SP-8		Sandy GRAVEL (GP), dark brown, very dense, wet, some silt, fine to coarse grained sand	57						
			End of boring at 30 feet below grade - sampled to 31.5 feet. Groundwater was encountered at 6 feet below grade. Borehole backfilled with cement grout							
35										
40										
45										
50										

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FIGURE NO.
A-1b

BORING GB-2

CLIENT: WESTON SOLUTIONS	PSI PROJECT NO.: 575-249
PROJECT: FORMER UKIAH STATION	BORING TYPE: 8-INCH DIA. H.S.A. w/AUTO-HAMMER
LOCATION: 309 E. PERKINS STREET, UKIAH, CALIFORNIA	ELEVATION: EXISTING GRADE
DATE DRILLED: JANUARY 6, 2011	LOGGED BY: FRANK POSS

DEPTH (ft)	SAMPLE NO.	SAMPLE	SOIL DESCRIPTION	BLOWS PER FOOT	% PASSING #200 SIEVE	LIQUID LIMIT	PLASTIC LIMIT	MOISTURE CONTENT (%)	UNIT DRY WT. (p.c.f.)	REMARKS
	SS-1		Silty CLAY (CL), brown, soft, moist	11						Torvane = 0.3 tsf.
5										
	SP-2		same as above; very soft, wet	8		26	17	20		Groundwater at 6 feet. Torvane = 0.2 tsf.
	SS-3		Silty SAND (SM), dark brown, medium dense, wet, fine to coarse sand, some gravel	19	20			20	103	
10										
	SP-4		Sandy GRAVEL (GP), dark brown, medium dense, wet, fine to coarse sand	27						
15										
	SP-5		same as above, dense	46						
20										
	SP-6		Silty CLAY (CL), orange brown, very stiff, moist, some gravel at 20.5 feet	16				12		Qp = 2.5 tsf.
25										

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FIGURE NO.
A-2a

BORING GB-2 (cont.)

CLIENT: WESTON SOLUTIONS	PSI PROJECT NO.: 575-249
PROJECT: FORMER UKIAH STATION	BORING TYPE: 8-INCH DIA. H.S.A. w/AUTO-HAMMER
LOCATION: 309 E. PERKINS STREET, UKIAH, CALIFORNIA	ELEVATION: EXISTING GRADE
DATE DRILLED: JANUARY 6, 2011	LOGGED BY: FRANK POSS

DEPTH (ft)	SAMPLE NO.	SAMPLE	SOIL DESCRIPTION	BLOWS PER FOOT	% PASSING #200 SIEVE	LIQUID LIMIT	PLASTIC LIMIT	MOISTURE CONTENT (%)	UNIT DRY WT. (p.c.f.)	REMARKS
	SP-7		as above; mottled blue green, firm	20						Qp = 1.0 tsf.
30										
	SP-8		Sandy Gravel (GP), brown, very dense, wet, fine to coarse sand, some silt	80						Sandier at 30.5
35										
	SP-9		As above; some cobbles	50/6				12		
40										
	SP-10		As above	50/4						
45										
	SP-11		CLAY (CL), blue/green brown mottled, very stiff, moist, some gravel	23						Qp = 2.5 tsf.
50										
	SP-12		Gravelly CLAY (CL), blue/green brown mottled, hard, moist, some sand	50						
			End of boring at 50 feet below grade - sampled to 51.5 feet. Groundwater was encountered at 6 feet below grade. Borehole backfilled with cement grout							

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FIGURE NO.
A-2b

BORING GB-3

CLIENT: WESTON SOLUTIONS	PSI PROJECT NO.: 575-249
PROJECT: FORMER UKIAH STATION	BORING TYPE: 8-INCH DIA. H.S.A. w/AUTO-HAMMER
LOCATION: 309 E. PERKINS STREET, UKIAH, CALIFORNIA	ELEVATION: EXISTING GRADE
DATE DRILLED: JANUARY 7, 2011	LOGGED BY: FRANK POSS

DEPTH (ft)	SAMPLE NO.	SAMPLE	SOIL DESCRIPTION	BLOWS PER FOOT	% PASSING #200 SIEVE	LIQUID LIMIT	PLASTIC LIMIT	MOISTURE CONTENT (%)	UNIT DRY WT. (p.c.f.)	REMARKS
	SP-1		Clayey SILT (ML), brown, very soft, wet, some fine sand	3				20		Torvane = 0.05 tsf.
5										
	SS-2		Gravelly CLAY (CL), dark brown, very soft, moist, some silt	6						Torvane = 0.05 tsf.
										Groundwater at 7.5 feet.
	SP-3		Clayey GRAVEL (GC), dark brown, dense, wet, some sand and silt	24						
10										
	SS-4		Clayey SAND (SC), dark brown, dense, wet, silt and gravel	34	23			12		
15										
	SP-5		Sandy GRAVEL (GP), brown, dense, wet, some cobbles and clay	48						
20										
	SP-6		Clayey GRAVEL (GC), light brown, medium dense very moist, some sand	29				12		
25										

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FIGURE NO.
A-3a

BORING GB-3 (cont.)

CLIENT: WESTON SOLUTIONS	PSI PROJECT NO.: 575-249
PROJECT: FORMER UKIAH STATION	BORING TYPE: 8-INCH DIA. H.S.A. w/AUTO-HAMMER
LOCATION: 309 E. PERKINS STREET, UKIAH, CALIFORNIA	ELEVATION: EXISTING GRADE
DATE DRILLED: JANUARY 7, 2011	LOGGED BY: FRANK POSS

DEPTH (ft)	SAMPLE NO.	SAMPLE	SOIL DESCRIPTION	BLOWS PER FOOT	% PASSING #200 SIEVE	LIQUID LIMIT	PLASTIC LIMIT	MOISTURE CONTENT (%)	UNIT DRY WT. (p.c.f.)	REMARKS
	SP-7		Silty CLAY (CL), mottled green/gray brown, very soft, moist	8						Qp = 2.5 tsf.
30										
	SP-8		Sandy GRAVEL (GP), brown, very dense, wet, some clay and cobbles	50/5						
			End of boring at 30 feet below grade - sampled to 31.5 feet. Groundwater was encountered at 7.5 feet below grade. Borehole backfilled with cement grout							
35										
40										
45										
50										

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FIGURE NO.
A-3b

BORING GB-4

CLIENT: WESTON SOLUTIONS	PSI PROJECT NO.: 575-249
PROJECT: FORMER UKIAH STATION	BORING TYPE: 8-INCH DIA. H.S.A. w/AUTO-HAMMER
LOCATION: 309 E. PERKINS STREET, UKIAH, CALIFORNIA	ELEVATION: EXISTING GRADE
DATE DRILLED: JANUARY 7, 2011	LOGGED BY: FRANK POSS

DEPTH (ft)	SAMPLE NO.	SAMPLE	SOIL DESCRIPTION	BLOWS PER FOOT	% PASSING #200 SIEVE	LIQUID LIMIT	PLASTIC LIMIT	MOISTURE CONTENT (%)	UNIT DRY WT. (p.c.f.)	REMARKS
	SS-1		Clayey GRAVEL (GC), dark brown, loose, wet, some sand and silt (FILL)	5						
5										Groundwater at 5 feet.
	SP-2		CLAY (CL), dark brown, very soft, wet	0				21		Small sample recovery
	SS-3		Clayey GRAVEL (GC), brown, medium dense, wet, some sand	15						wood fragments
10			Boring terminated at 10 feet due to drill refusal Groundwater was encountered at 5 feet below grade. Borehole backfilled with cement grout							
15										
20										
25										

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FIGURE NO.
A-4

BORING GB-5

CLIENT: WESTON SOLUTIONS	PSI PROJECT NO.: 575-249
PROJECT: FORMER UKIAH STATION	BORING TYPE: 8-INCH DIA. H.S.A. w/AUTO-HAMMER
LOCATION: 309 E. PERKINS STREET, UKIAH, CALIFORNIA	ELEVATION: EXISTING GRADE
DATE DRILLED: JANUARY 7, 2011	LOGGED BY: FRANK POSS

DEPTH (ft)	SAMPLE NO.	SAMPLE	SOIL DESCRIPTION	BLOWS PER FOOT	% PASSING #200 SIEVE	LIQUID LIMIT	PLASTIC LIMIT	MOISTURE CONTENT (%)	UNIT DRY WT. (p.c.f.)	REMARKS
	SP-1		CLAY (CL), dark brown, firm, moist, some silt and sand	6						No recovery
5										
	SS-2		As above; soft, wet	14						Torvane = 0.4 tsf.
										Groundwater at 8.0 feet.
	SP-3		Sandy GRAVEL (GP), brown, medium dense, wet, some silt and cobbles	28				14		
10										
	SS-4		As above; dense, coarser material, some clay	39	10			9	125	
15										
	SP-5		Clayey GRAVEL (GC), red brown, dense, wet, some sand and cobbles	40						
20										
	SP-6		Gravelly SAND (SP), dark brown, medium dense, wet, fine to coarse sand, some silt and clay	23				13		
25										

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FIGURE NO.
A-5a

BORING GB-5 (cont.)

CLIENT: WESTON SOLUTIONS	PSI PROJECT NO.: 575-249
PROJECT: FORMER UKIAH STATION	BORING TYPE: 8-INCH DIA. H.S.A. w/AUTO-HAMMER
LOCATION: 309 E. PERKINS STREET, UKIAH, CALIFORNIA	ELEVATION: EXISTING GRADE
DATE DRILLED: JANUARY 7, 2011	LOGGED BY: FRANK POSS

DEPTH (ft)	SAMPLE NO.	SAMPLE	SOIL DESCRIPTION	BLOWS PER FOOT	% PASSING #200 SIEVE	LIQUID LIMIT	PLASTIC LIMIT	MOISTURE CONTENT (%)	UNIT DRY WT. (p.c.f.)	REMARKS
	SP-7		CLAY (CL), light brown, hard, moist, some silt	36						
30	SP-8		Sandy GRAVEL (GP), dark brown, very dense, wet, some silt	90						
			End of boring at 30 feet below grade - sampled to 31.5 feet. Groundwater was encountered at 8.0 feet below grade. Borehole backfilled with cement grout							
35										
40										
45										
50										

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FIGURE NO.
A-5b

BORING GB-6

CLIENT: WESTON SOLUTIONS	PSI PROJECT NO.: 575-249
PROJECT: FORMER UKIAH STATION	BORING TYPE: 8-INCH DIA. H.S.A. w/AUTO-HAMMER
LOCATION: 309 E. PERKINS STREET, UKIAH, CALIFORNIA	ELEVATION: EXISTING GRADE
DATE DRILLED: JANUARY 6, 2011	LOGGED BY: FRANK POSS

DEPTH (ft)	SAMPLE NO.	SAMPLE	SOIL DESCRIPTION	BLOWS PER FOOT	% PASSING #200 SIEVE	LIQUID LIMIT	PLASTIC LIMIT	MOISTURE CONTENT (%)	UNIT DRY WT. (p.c.f.)	REMARKS
	SP-1		CLAY (CL), dark brown, soft, moist, some sand, silt gravel	10						Torvane = 0.4 tsf.
5										
	SS-2		As above; wet	13						Groundwater at 6.0 feet. Torvane = 0.5 tsf.
	SP-3		Gravelly SAND (SP), red brown, dense, fine to coarse sand, some cobbles	32				11		
10										
	SS-4		GRAVEL (GP), brown, dense, wet, some sand, clay, and cobbles	37				10	133	
15										
	SP-5		Sandy GRAVEL (GP), brown, dense, wet, some cobbles	36						
20										
	SP-6		Clayey SILT (ML), green brown, soft, moist, some ffine sand	9						Torvane = 0.3 tsf.
25										

Information To Build On
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FIGURE NO.
A-6a

BORING GB-6 (cont.)

CLIENT: WESTON SOLUTIONS	PSI PROJECT NO.: 575-249
PROJECT: FORMER UKIAH STATION	BORING TYPE: 8-INCH DIA. H.S.A. w/AUTO-HAMMER
LOCATION: 309 E. PERKINS STREET, UKIAH, CALIFORNIA	ELEVATION: EXISTING GRADE
DATE DRILLED: JANUARY 6, 2011	LOGGED BY: FRANK POSS

DEPTH (ft)	SAMPLE NO.	SAMPLE	SOIL DESCRIPTION	BLOWS PER FOOT	% PASSING #200 SIEVE	LIQUID LIMIT	PLASTIC LIMIT	MOISTURE CONTENT (%)	UNIT DRY WT. (p.c.f.)	REMARKS
	SP-7		Sandy GRAVEL (GP), brown, dense, wet	32						
30										
	SP-8		Gravelly SAND (SP), brown, very dense, fine to coarse sand, some cobbles	59				13		
35										
	SP-9		As above	95						
40										
	SP-10		As above	50/6						
45										
	SP-11		CLAY (CL), blue/green brown mottled, very stiff, moist, some gravel	20						Qp = 2.5 tsf.
50										
	SP-12		no recovery	38						driller noted that no change in drilling pressure from 45 to 50 feet
			End of boring at 50 feet below grade - sampled to 51.5 feet. Groundwater was encountered at 6 feet below grade. Borehole backfilled with cement grout							

psi Information To Build On
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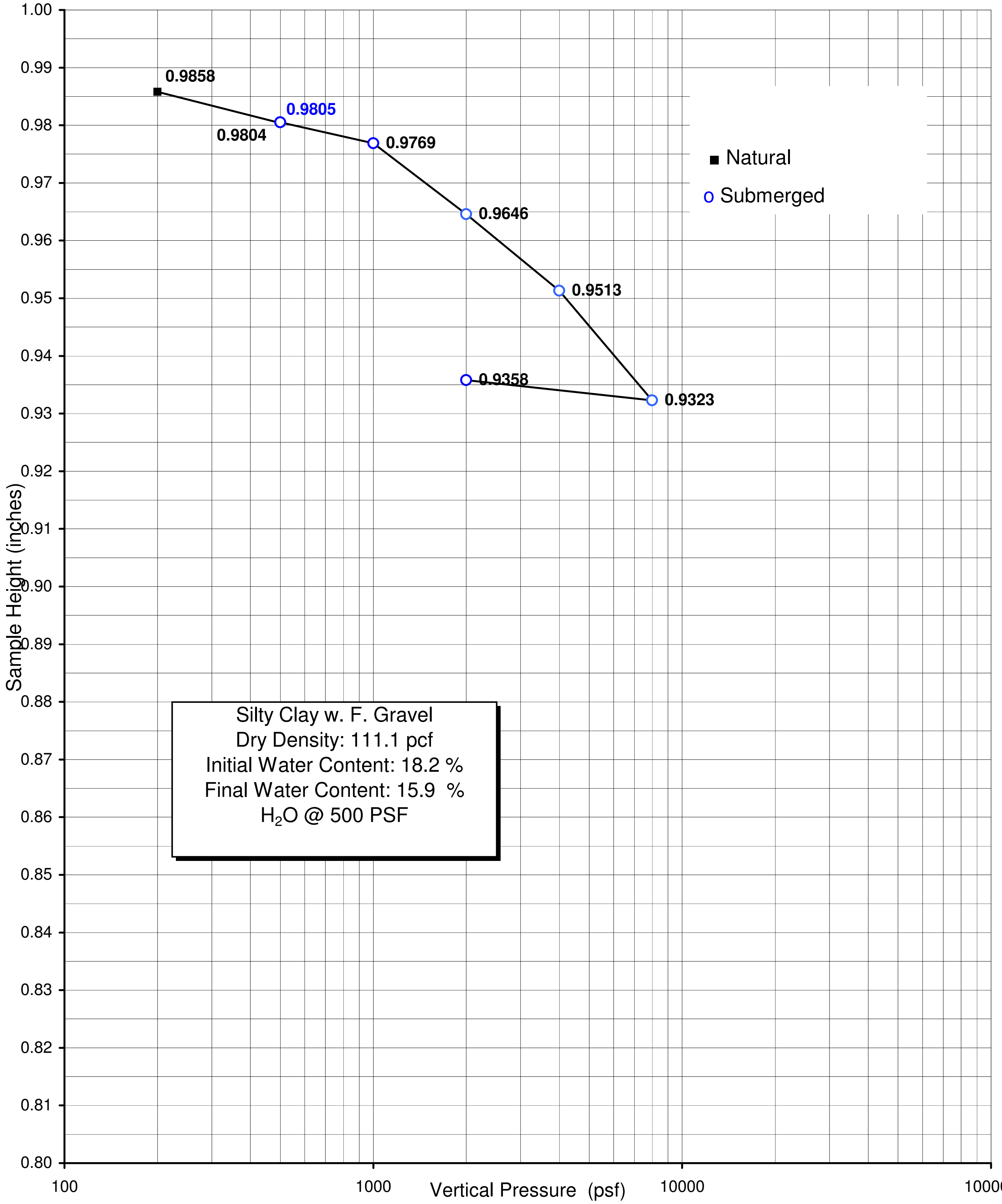
FIGURE NO.
A-6b

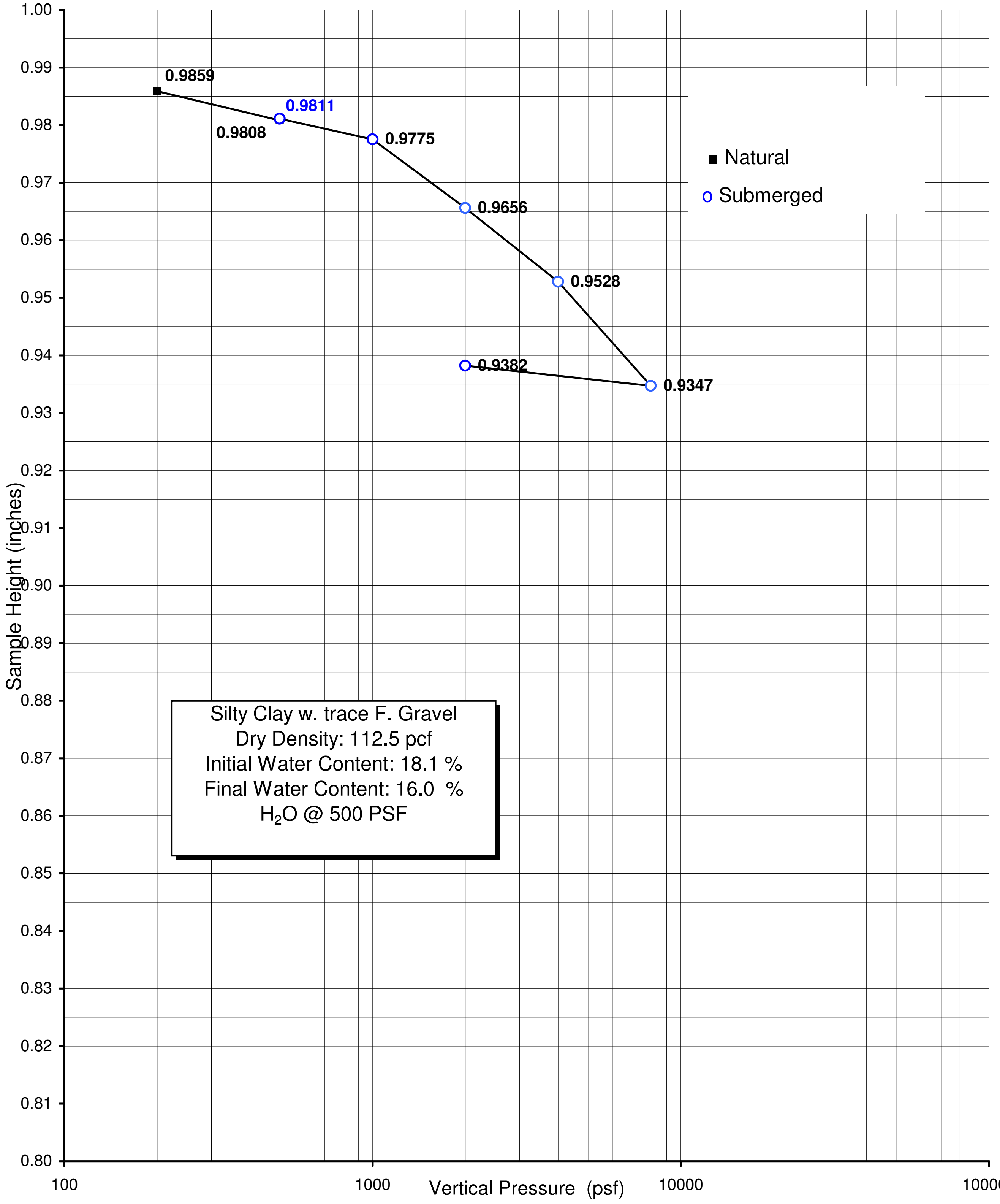
CONSOLIDATION TEST - ASTM D2435

Boring / Sample No. B-3 / SS-2	Depth: 5.0'	Date 01-17-11



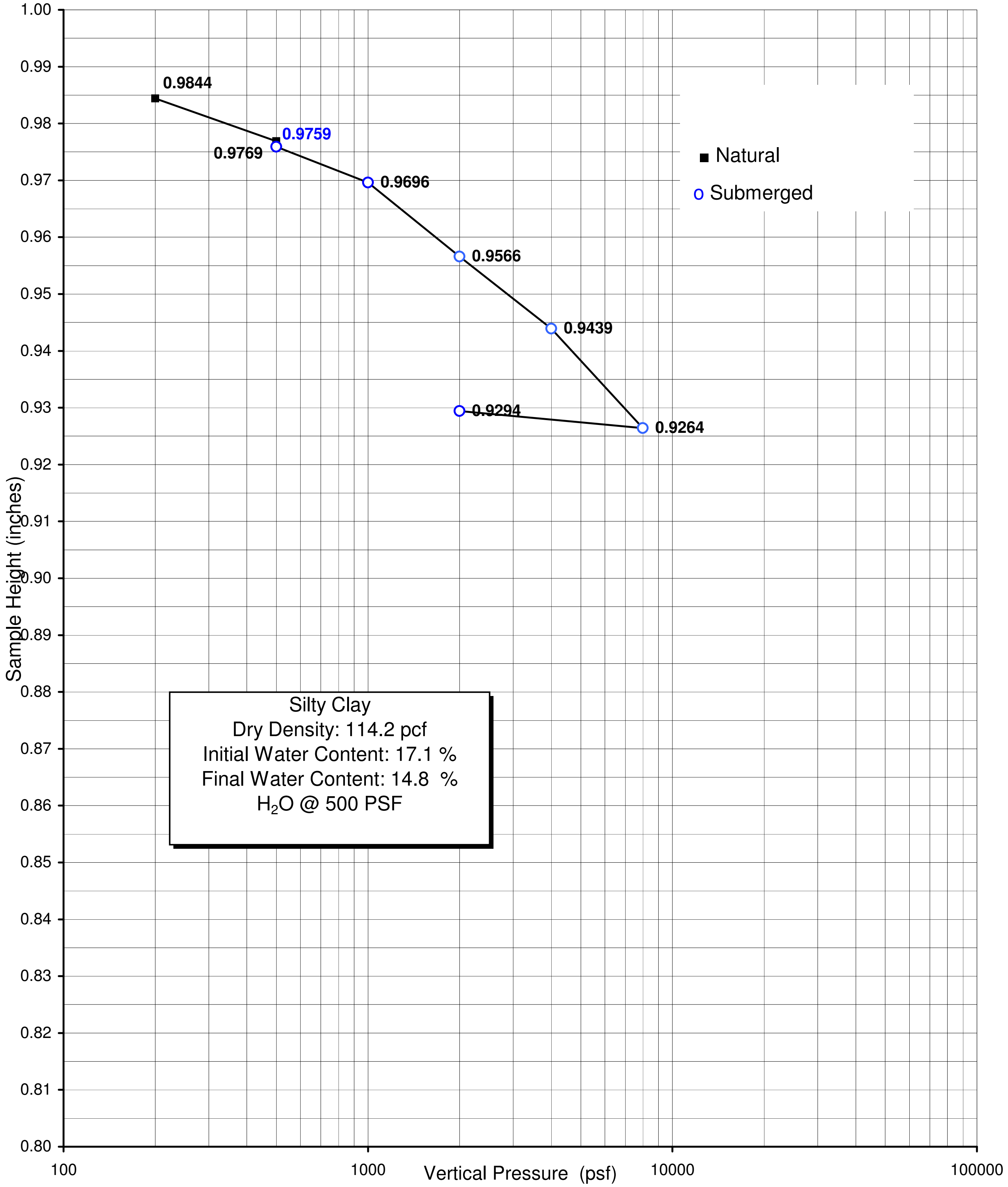
CONSOLIDATION TEST - ASTM D2435

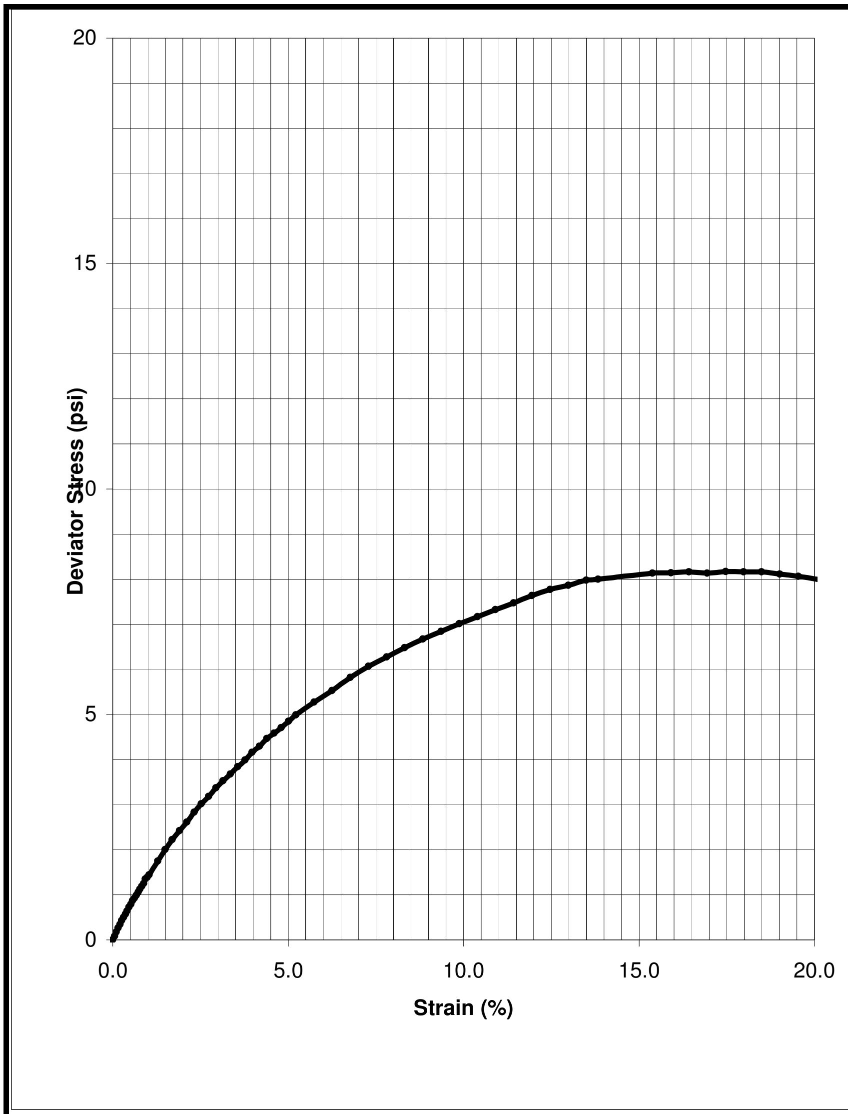
Boring / Sample No. B-5 / SS-2	Depth: 5.0'	Date 01-17-11



CONSOLIDATION TEST - ASTM D2435

Boring / Sample No. B-6 / SS-2	Depth: 5.0'	Date 01-17-11



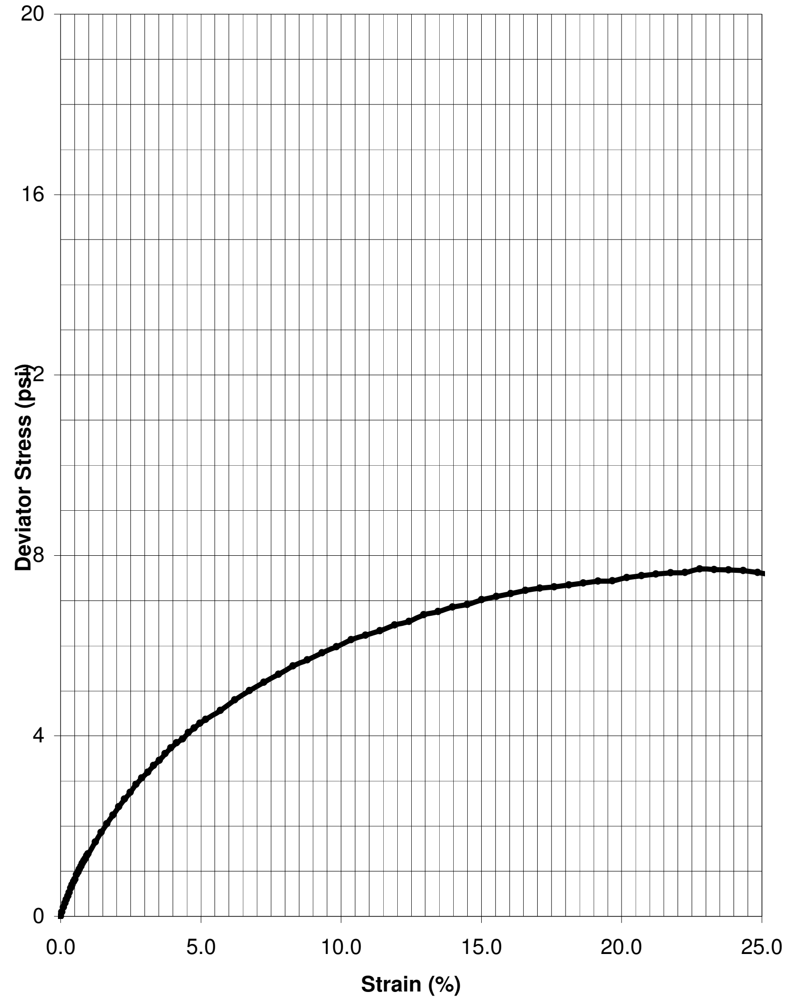


Soil Description: Brown, Silty Clay

Type of Specimen: *Undisturbed*

SPECIMEN			A	B	C
INITIAL	Wet Density (pcf)		***124.5***		
	Water Content (%)		***24.2***		
	Dry Density (pcf)		***100.2***		
FINAL	Wet Density (pcf)				
	Water Content (%)				
	Dry Density (pcf)				
SATURATION STAGE	Initial pwp	psi			
	Saturated pwp	psi			
	Final Cell pressure	psi			
	B value				
CONSOLIDATION STAGE	Cell Pressure	psi			
	Back Pressure	psi			
	Initial pwp	psi			
	Final pwp	psi			
COMPRESSION STAGE	Cell Pressure	psi			
	Initial pwp	psi			
	Initial σ'_3	psi			
	Strain Rate (in./min.)		***0.005***		
FAILURE CONDITION	**Strain %**		***17.5***		
	$(\sigma_1 - \sigma_3)_f$	**psi**	***8.17***		
	$(\sigma_1 / \sigma_3)_f$	**psi**			
	σ_3	**psi**	***3.60***		
	σ_{1f}	**psi**	***11.77***		
	c_v				
	m_v				
	k				
SAMPLE SIZE			D =	2.41	in.
			H =	5.0	in.

UNCONSOLIDATED UNDRAINED TRIAXIAL COMPRESSION TEST	Job Name PSI #575-249	Date: 1-26-11
ASTM D-2850	Job No. 2008-026	Sample : B-2/SS-1



Before

After

Soil Description: Brown, Silty Clay w. trace F. Gravel

Type of Specimen: *Undisturbed*

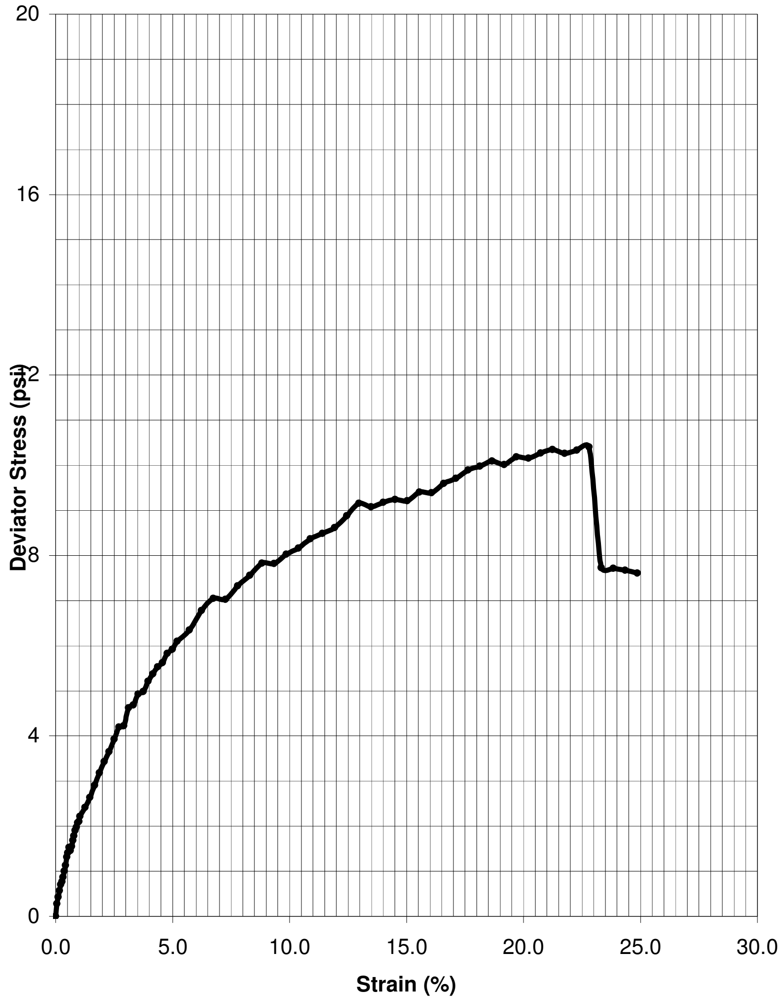
SPECIMEN			A	B	C
INITIAL	Wet Density (pcf)		***129.0***		
	Water Content (%)		***19.7***		
	Dry Density (pcf)		***107.8***		
FINAL	Wet Density (pcf)				
	Water Content (%)				
	Dry Density (pcf)				
SATURATION STAGE	Initial pwp	psi			
	Saturated pwp	psi			
	Final Cell pressure	psi			
	B value				
CONSOLIDATIO N STAGE	Cell Pressure	psi			
	Back Pressure	psi			
	Initial pwp	psi			
	Final pwp	psi			
COMPRESSION STAGE	Cell Pressure	psi			
	Initial pwp	psi			
	Initial σ'_3	psi			
	Strain Rate (in./min.)		***0.005***		
FAILURE CONDITION	**Strain %**		***23.8***		
	$(\sigma_1 - \sigma_3)_f$	**psi**	***7.68***		
	$(\sigma_1 / \sigma_3)_f$	**psi**			
	σ_3	**psi**	***3.60***		
	σ_{1f}	**psi**	***11.28***		
	c_v				
	m_v				
	k				
SAMPLE SIZE			D =	2.41	in.
			H =	4.6	in.

UNCONSOLIDATED UNDRAINED TRIAXIAL COMPRESSION TEST

ASTM D-2850

Job Name PSI #575-249	Date: 1-27-11
Job No. 2008-026	Sample : B-5/SS-2

Geo**L**ogic **A**ssociates



Before

After

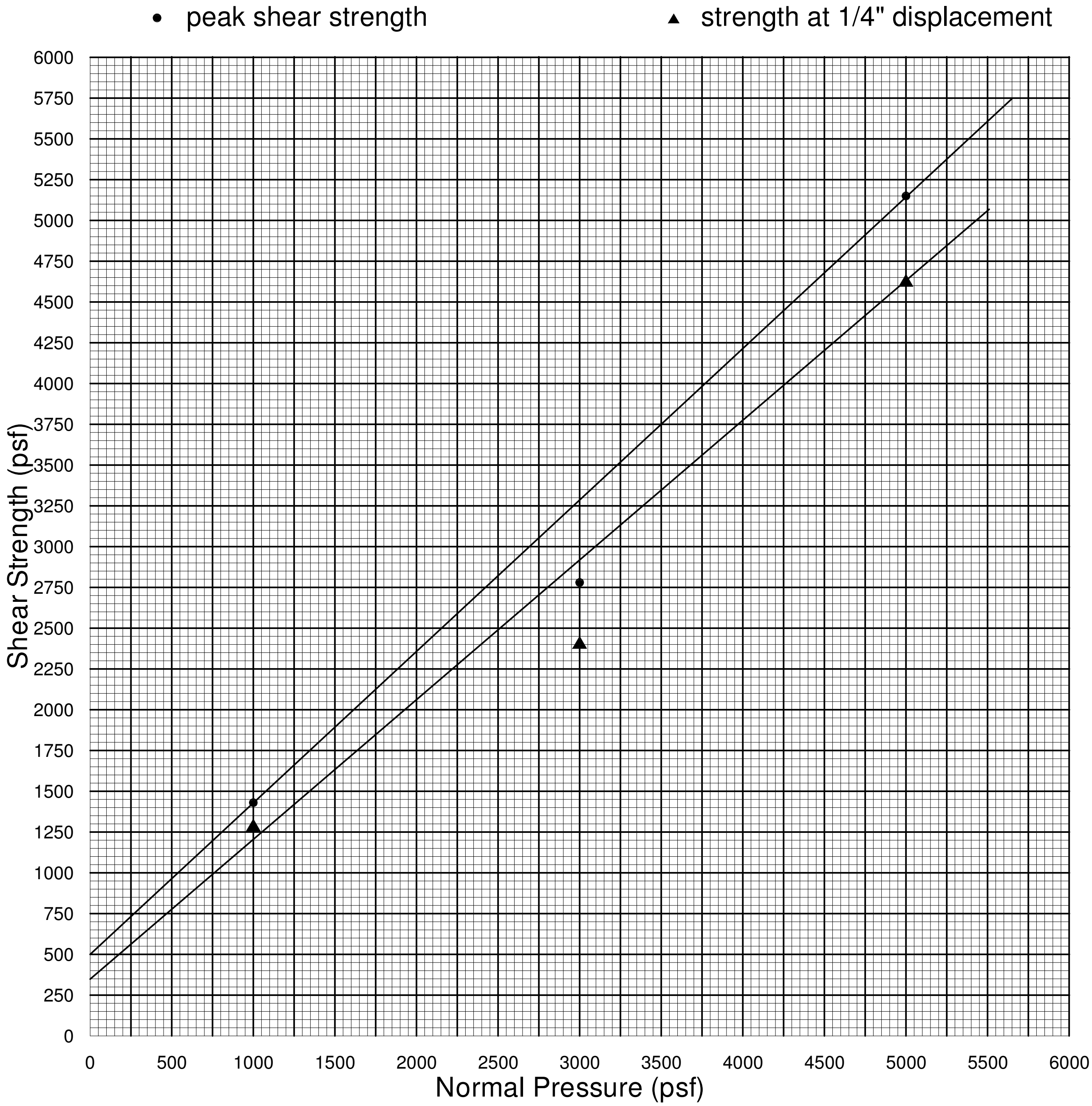
Soil Description: Brown, Silty Clay w. trace F. Gravel

Type of Specimen: *Undisturbed*

	SPECIMEN		A	B	C
INITIAL	Wet Density (pcf)		***133.6***		
	Water Content (%)		***18.3***		
	Dry Density (pcf)		***112.9***		
FINAL	Wet Density (pcf)				
	Water Content (%)				
	Dry Density (pcf)				
SATURATION STAGE	Initial pwp	psi			
	Saturated pwp	psi			
	Final Cell pressure	psi			
	B value				
CONSOLIDATION STAGE	Cell Pressure	psi			
	Back Pressure	psi			
	Initial pwp	psi			
	Final pwp	psi			
COMPRESSION STAGE	Cell Pressure	psi			
	Initial pwp	psi			
	Initial σ'_3	psi			
	Strain Rate (in./min.)		***0.005***		
FAILURE CONDITION	**Strain %**		***22.8***		
	$(\sigma_1 - \sigma_3)_f$	**psi**	***10.40***		
	$(\sigma_1 / \sigma_3)_f$	**psi**			
	σ_3	**psi**	***3.60***		
	σ_{1f}	**psi**	***14.00***		
	c_v				
	m_v				
	k				
SAMPLE SIZE			D =	2.41	in.
			H =	4.8	in.

UNCONSOLIDATED UNDRAINED TRIAXIAL COMPRESSION TEST	Job Name PSI #575-249	Date: 1-27-11
ASTM D-2850	Job No. 2008-026	Sample : B-6/SS-2

DIRECT SHEAR TEST - ASTM D-3080

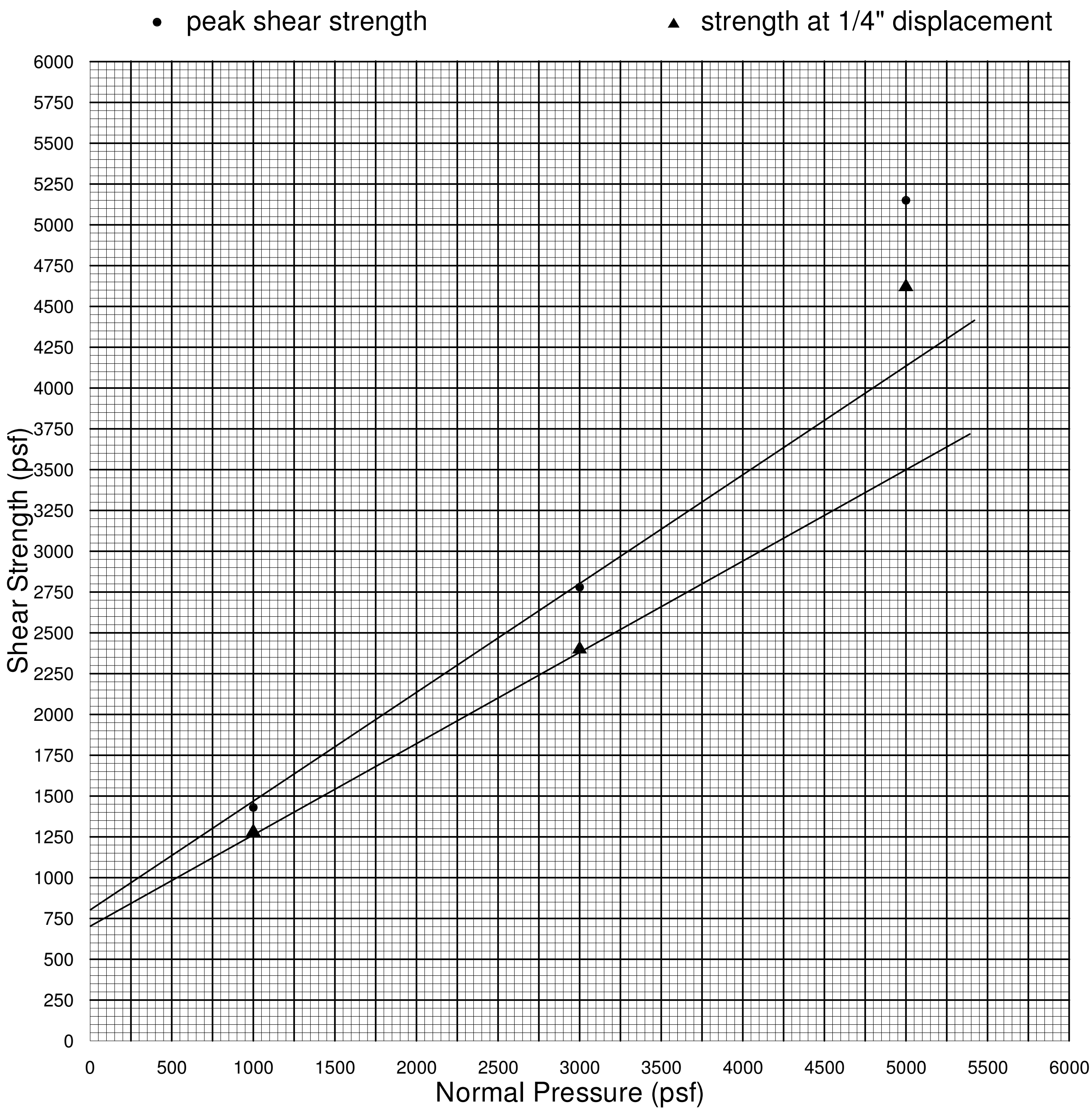


Strain Rate: 0.0042 in. / min.

Sample	Type	Description	Dry Density (pcf)	Initial W.C (%)	Final W.C. (%)
B1-SS3 @ 7.5'	Undisturbed & Saturated	F.M. Clayey Sand w. Gravel	121.4	10.8	13.9

Normal Pressure (psf)	Peak Shear Strength (psf)	Ultimate Shear Strength (psf)
1000	1430 @ 0.1600"	1280
3000	2780 @ 0.1150"	2400
5000	5150 @ 0.1500"	4620
	C = 500 psf	C = 350 psf
	ϕ = 43 deg.	ϕ = 40 deg.

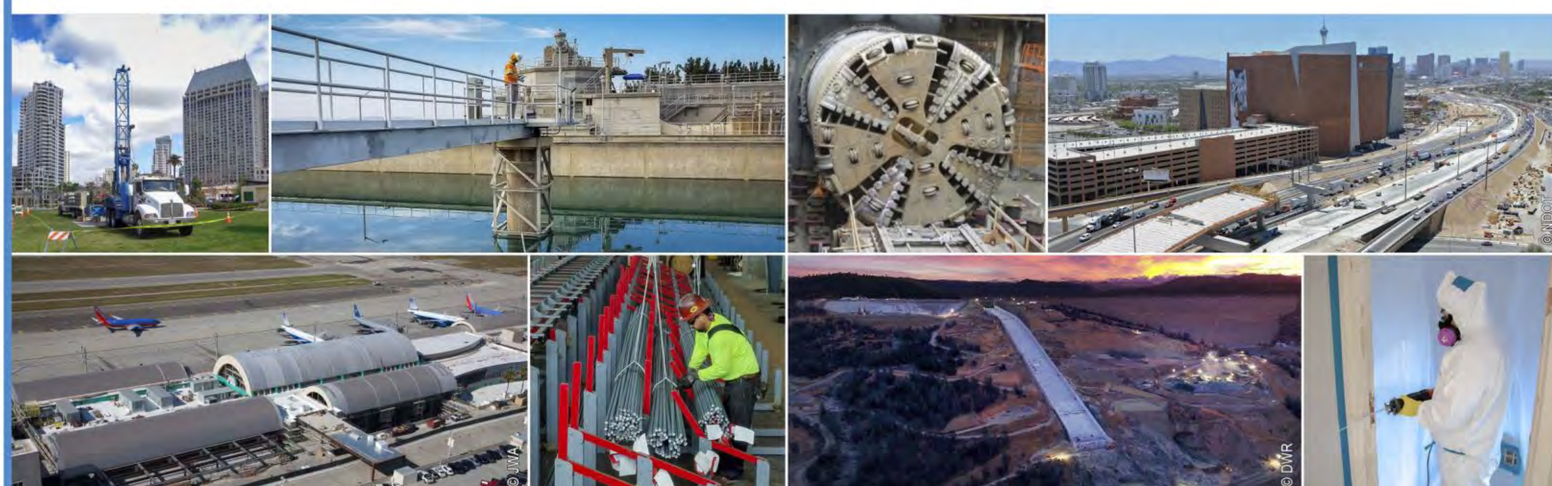
DIRECT SHEAR TEST - ASTM D-3080



Strain Rate: 0.0042 in. / min.

Sample	Type	Description	Dry Density (pcf)	Initial W.C (%)	Final W.C. (%)
B1-SS3 @ 7.5'	Undisturbed & Saturated	F.M. Clayey Sand w. Gravel	121.4	10.8	13.9

Normal Pressure (psf)	Peak Shear Strength (psf)	Ultimate Shear Strength (psf)
1000	1430 @ 0.1600"	1280
3000	2780 @ 0.1150"	2400
5000	5150 @ 0.1500"	4620
	C = 750 psf	C = 700 psf
	ϕ = 34 deg.	ϕ = 30 deg.



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