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DRAFT
GEOTECHNICAL ENGINEERING AND GEOHAZARD INVESTIGATION
PROPOSED TEACHER WORKFORCE HOUSING PROJECT
SANTA CRUZ CITY SCHOOLS
NATURAL BRIDGES HIGH SCHOOL / CAREER TRAINING CENTER CAMPUS
313 SWIFT STREET
SANTA CRUZ, CALIFORNIA

Project Number: E95815.01

For:

Santa Cruz City Schools
c/o Mr. Trevor Miller
Director of Facility Services
536 Palm Street
Santa Cruz, California 95060

February 14, 2024

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February 14, 2024 E95815.01

Santa Cruz City Schools (SCCS)
c/o Mr. Trevor Miller
Director of Facility Services
536 Palm Street
Santa Cruz, California 95060

Subject: Draft: Geotechnical Engineering and Geohazard Investigation
Proposed Teacher Workforce Housing Project
Santa Cruz City Schools
Natural Bridges High School / Career Training Center Campus
313 Swift Street
Santa Cruz, California

Dear Mr. Miller:

We are pleased to submit this geotechnical engineering and geohazard investigation report conducted for the proposed three-story Teacher Workforce Housing building to be located at the Natural Bridges High School / Career Training Center Campus at an address of 313 Swift Street, in Santa Cruz, California.

We recommend that those portions of the plans and specifications that pertain to earthwork and foundations be reviewed by Moore Twining Associates, Inc. (Moore Twining) to determine if they are consistent with our recommendations. This service is not a part of this current contractual agreement; however, the client should provide these documents for our review prior to their issuance for construction bidding purposes.

We appreciate the opportunity to be of service to Santa Cruz City Schools. If you have any questions regarding this report, or if we can be of further assistance, please contact us at your convenience.

Sincerely,
MOORE TWINING ASSOCIATES, INC.

DRAFT

Shaun Reich, EIT
Project Engineer
Geotechnical Engineering Division

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SANTA CRUZ, CALIFORNIA

Project Number: E95815.01

1.0 INTRODUCTION

We are pleased to submit this geotechnical engineering and geohazard investigation report conducted for the proposed three-story Teacher Workforce Housing project to be constructed at the Natural Bridges High School / Career Training Center Campus for Santa Cruz City Schools located at 313 Swift Street in Santa Cruz, California. Moore Twining Associates, Inc. (Moore Twining) was authorized to conduct this investigation by Santa Cruz City Schools.

The contents of this report include the purpose of the investigation and the scope of services provided. The existing site features and anticipated construction are discussed. In addition, a description of the investigative procedures used and the subsequent findings obtained are presented. Finally, this report provides a summary of evaluations, geomorphic and geologic setting, geologic hazards, seismic hazards, general conclusions and related recommendations. The report appendices contain the drawings (Appendix A); the logs of borings (Appendix B); the results of laboratory tests (Appendix C), and a summary of the liquefaction analysis (Appendix D1).

The Geotechnical Engineering Division of Moore Twining, performed the investigation.

2.0 PURPOSE AND SCOPE OF INVESTIGATION

2.1 Purpose: The purpose of this investigation was to conduct a field exploration and laboratory testing program, evaluate the data collected during the field and laboratory portions of the investigation, and provide the following:

2.1.1 Recommendations for site preparation, including placement, moisture conditioning, and compaction of engineered fill soils;

2.1.2 Geotechnical engineering parameters for use in design of foundations and slabs on grade;

2.1.3 Descriptions of potential geologic/seismic hazards in accordance with the requirements of the 2022 California Building Code and California Geological Survey – Note No. 48;

2.1.4 Presentation of seismic coefficients in accordance with the 2022 CBC;

2.1.5 Conclusions regarding soil corrosion potential.

This report is provided specifically for the proposed improvements described in the Anticipated Construction section of this report. The purpose of our investigation was to provide geotechnical engineering parameters for use in design of foundations, slabs-on-grade, and preparation of related construction documents. The intent of the investigation was also to evaluate potential geohazards. This investigation did not include in-place density tests, percolation testing, an environmental investigation, or an environmental audit.

2.2 <u>Scope</u>: Our proposal, dated February 21, 2023, outlined the scope of our services. The actions undertaken during the investigation are summarized as follows:

2.2.1 The “19six SCCS Board Presentation-Workforce Housing”, dated January 11, 2023, showing the location of the proposed building within the campus, was reviewed.

2.2.2 A Site Plan Schematic titled, “SCCS WFH unit Count_remix_rev3", undated, provided via email on February 17, 2023, prepared by 19six Architects, was reviewed. It should be further noted that at the time of the preparation of this report, the site plan had not finalized.

2.2.3 Satellite images of the site between the years 1993 and 2022 from online sources, were reviewed. In addition, historical aerial photographs of the site for various years between 1928 and 2020 were reviewed from an in-progress report entitled, “Phase I Environmental Site Assessment, Santa Cruz City Schools, 313 Swift Street, Santa Cruz, California 95060” prepared by Moore Twining’s Environmental Division.

2.2.4 A draft report entitled, “Geotechnical Engineering Investigation, Proposed Artificial Turf Project, Natural Bridges Elementary School, 255 Swift Street, Santa Cruz, California,” dated June 7, 2019, prepared by Moore Twining Associates, Inc. (Moore Twining Project No. E95805.01), was reviewed.

2.2.5 A site reconnaissance and subsurface exploration were conducted by a Moore Twining staff engineer.

2.2.6 Laboratory tests were conducted to determine selected physical and engineering properties of the subsurface soils.

2.2.7 Mr. Ralph Le Roux (19-Six Architects) was consulted during the investigation.

2.2.8 The data obtained from the investigation were evaluated to develop an understanding of the subsurface soil conditions and engineering properties of the subsurface soils encountered.

2.2.9 This report was prepared.

3.0 BACKGROUND INFORMATION

The site history and previous studies, site description and the anticipated construction are summarized in the following subsections.

3.1 Site History and Previous Studies: Based on Moore Twining's in-progress Phase 1 Environmental Site Assessment and review of historical aerial photographs, city directories, topographic maps and Sanborn maps, the subject property was vacant, undeveloped land until the late 1920's/early 1930's. During the early 1930's to the early 1940's, row crops were grown on the subject property. The 1968 historical aerial photograph shows the subject property was developed with two structures, a paved driveway and parking areas. Sometime between March 2002 and March 2003, in the area where the three-story Workforce Housing project is planned, the aerial images indicate that several buildings were removed, an asphalt concrete paved driveway and cul-de-sac was added west and southwest of an existing parking lot area, and buildings were added west and south of the cul-de-sac and also south of the parking lot area. Sometime between September 2022 and July 2023, the buildings on the west and south sides of the cul-de-sac were removed.

Moore Twining's in-progress Phase 1 Environmental Site Assessment indicated the subject property was historically used by the National Guard Armory for vehicle and equipment maintenance. In May 2002, a Preliminary Endangerment Assessment (PEA) was conducted by Shaw Environmental and Infrastructure (Shaw, former IT Corporation) to address the concerns associated with the former National Guard Armory activities. Moore Twining's in-progress Phase 1 Environmental Site Assessment further indicated that elevated concentrations of some polycyclic aromatic hydrocarbons (PAHs) were identified in the vicinity of a former grease rack, but no other constituents of concern (COCs) were identified above their respective screening levels at the time of Shaw's PEA. No further action was recommended and approved by Department Toxic Substances Control (DTSC) in July 2002.

Moore Twining previously prepared a report for an "Artificial Turf Project" located at the adjacent "Natural Bridges Elementary School", located immediately south of the subject site at an address of 225 Swift Street, Santa Cruz, California, dated June 7, 2019. The exploration included advancing four (4) borings with a CME-75 drill rig within the area to support the artificial turf field to depths of about 14½ feet to about 21 feet below site grade (BSG). The borings generally encountered silty sands and clayey sand to depths of about 5 to 14½ feet BSG. In general, the surface soils to depths of 2 to 5 feet BSG were considered very loose. The silty sands and clayey sands were further underlain by poorly graded sand. Weathered claystone bedrock was encountered at depths of 5 feet to 15½ feet BSG. The weathered claystone extended to the maximum depth of exploration, about 21 feet BSG. Groundwater was generally encountered at depths of 6.7 to 7.5 feet BSG during drilling; and after being left open for several hours, groundwater stabilized to depths ranging from about 3.4 to 6 feet BSG. The report further recommends that since a 10-foot clear distance is not present from ground elevation to groundwater depth, stormwater infiltration was not recommended. In addition, due to the medium expansion potential and high subgrade moisture content, measures such as chemical treatment of subgrade were anticipated to dry the soils for stabilization purposes and reducing the expansion potential of subgrade which may be sensitive to heave. The report recommended non expansive fill materials below slabs on grade to reduce potential impacts from the expansive soils.

A report entitled, "Geotechnical Investigation, Natural Bridges Elementary School, 255 Swift Street, Santa Cruz, CA," dated November 16, 2017, prepared by GeoTrinity Consultants, Inc. (GeoTrinity), was reviewed. The report was prepared for restoration of the existing gymnasium in the southeast portion of the adjacent elementary school campus and for development of surrounding walkways, patios, and underground utilities. The investigation included two (2) exploratory borings near the northwest and southwest corners of the existing gymnasium to depths of about 16 feet below site grade (BSG). The boring logs indicate that clayey sand soils were encountered to depths of about 10 to 12 feet, underlain by weathered bedrock extending to the maximum depth explored, about 16 feet BSG. The bedrock was described as consisting of dark gray siltstone. The report indicated that groundwater was encountered at approximately 9 feet BSG, but the boreholes were not left open for a sufficient period of time to establish equilibrium groundwater conditions. Site preparation recommendations were provided in the report for preparation of subgrade to receive slabs-on-grade or pavements. For the gymnasium building, the report provided recommendations for foundation design for continuous footings to be at least 12 inches wide, extend to a minimum of 18 inches below surrounding grade surface, and be designed for an allowable bearing pressure of 2,800 pounds per square foot for dead loads. The interior slab-on-grade was recommended to be underlain by 24 inches of imported, non-expansive compacted fill.

No additional previous reports of geotechnical engineering investigations, geologic hazards investigations, compaction testing or environmental studies conducted within the subject school campus were provided for review during this investigation. If available, these reports should be provided for review and consideration for this project.

3.2 Site Description: The proposed Teacher Workforce Housing project is planned to be located within the area of Natural Bridges High School and the Career Training Center campus (see Drawing No. 1 in Appendix A). The site is located at about 36.954407 degrees latitude and -122.049663 degrees longitude.

Based on our review of online sources, the current Santa Cruz City Schools Natural Bridges High School and Career Training Center has been assigned a Santa Cruz County Assessor's Parcel Number (APN) of 003-161-32. The site area comprises approximately 3.45 acres of a 4.08 acre parcel, and has an address of 313 Swift Street, in Santa Cruz, California.

Some of the proposed buildings will be constructed within the footprint of previous buildings as shown on Drawing No. 2 in Appendix A.

The proposed area to support the proposed Teacher Workforce Housing building area currently supports Portland cement concrete curb and sidewalk, site lighting, and an asphalt concrete paved area previously supporting several modular buildings with associated underground utilities. In addition, the northwestern portion of the site was observed to contain a chain-link fenced area that was used for various equipment, storage containers, and material storage. The overall fenced area was also observed to support a dense growth of weeds/grasses, landscaping grass, bushes, and mature trees.

The area proposed to support the proposed Teacher Workforce Housing buildings is bounded to the north by commercial properties and Delaware Avenue beyond, to the west by commercial properties, tennis courts and Sergeant Derby Park, to the south by Natural Bridges Elementary School, and to the east by improvements associated with Natural Bridges High School with Swift Street and residential property beyond.

The existing asphalt concrete pavement and Portland cement concrete flatwork surrounding the previous modular buildings were observed to be in a fair condition. A saw-cut trench and several utilities were observed within the area previously supporting the modular buildings. The two (2) existing buildings and surrounding flatwork (Portland cement concrete and asphalt concrete pavements) located near the entrance to the property were observed to be in a fair condition.

Review of a groundwater monitoring well report obtained from the online Geotracker application indicates that a monitoring well exists, or previously existed within the area north of the existing Natural Bridges High School building. It should be further noted that the monitoring well recorded a high groundwater reading of about 3 feet below site grade in March of 1998, 1999, and 2011.

The subject property is relatively flat and online imagery indicates elevations at the project site range from about 51 feet to 55 feet above mean sea level (AMSL).

3.3 Anticipated Construction: The following project description is based on an undated site plan schematic titled "SCCS WFH unit count_remix_rev3" (provided via email on February 17, 2023), prepared by 19six Architects, in addition to review of online satellite imagery and our experience with similar projects. It should be noted that at the time of preparation of this report, the project is in preliminary status and therefore the site plan is not finalized.

The Site Plan Schematic prepared by 19six Architects indicates a three-story Teacher Workforce Housing building is planned in the central and western portions of the parcel located at 313 Swift Street, in Santa Cruz, California, with an assumed first floor footprint of the building of about 23,804 +/- square feet in area. Existing improvements such as pavements, concrete flatwork, existing buildings, etc. will be removed as part of the project.

Based on similar construction, the proposed three-story Workforce Housing building is anticipated to have a maximum column loads on the order of 50 kips and a maximum wall load of less than 3.0 kip per lineal foot. However, the actual design loads were not known at the time of this investigation.

Additional construction is anticipated to consist of concrete flatwork, underground utilities, pavements, and landscaping.

Due to the relatively flat area where the proposed building is planned, no significant cuts and fills are anticipated to achieve finished grades.

4.0 INVESTIGATIVE PROCEDURES

The field exploration and laboratory testing programs conducted for this investigation are summarized in the following subsections. The field exploration, number of test borings, sampling, and laboratory testing were performed in accordance with our proposal, dated February 21, 2023.

4.1 Field Exploration: The field exploration included a site reconnaissance, drilling test borings, conducting standard penetration tests, and soil sampling. Prior to the field exploration, the borings werre marked and the site was marked for Underground Service Alert (USA).

4.1.1 Site Reconnaissance: The site reconnaissance was conducted by a Moore Twining staff engineer on May 10 and 11, 2023. The reconnaissance consisted of walking the areas proposed for the building and noting visible surface features. The features noted are described in the "Site Description" and "Existing Conditions" sections of this report.

4.1.2 <u>Drilling Test Borings</u>: On May 10 and 11, 2023, eight (8) test borings were drilled for this investigation within the proposed building and pavement areas.

The test borings were drilled using a CME-75 truck mounted drill rig to a maximum depth of about 22.3 feet below site grade (BSG), at which auger refusal occurred due to competent bedrock. The drilling rig was equipped with 6⅝-inch diameter hollow-stem augers. The test boring locations are shown on the test boring location map provided as Drawing No. 2 in Appendix A.

The soils encountered in the test borings were logged during drilling by a representative of Moore Twining. The field soil classification was in accordance with the Unified Soil Classification System and consisted of particle size, color, and other distinguishing features. Soil samples were collected and returned to our laboratory for classification and soil mechanics testing. The presence and depth of free water, if any, in the test borings were noted and recorded during the drilling.

Test boring locations were determined by field measurement with reference to existing site features. Elevations of the borings were not surveyed as a part of the investigation. In accordance with the requirements of Santa Cruz County Environmental Health Department, the boreholes were backfilled with neat cement.

4.1.3 <u>Soil Sampling</u>: In the borings, standard penetration tests were conducted, and both disturbed and relatively undisturbed soil samples were obtained.

The standard penetration resistance, N-value, is defined as the number of blows required to drive a standard split barrel sampler into the soil. The standard split barrel sampler (SPT) has a 2 inch O.D. and a 1-⅜ inch inside diameter (I.D.). The sampler is driven by a 140 pound weight free falling 30 inches. The sampler is lowered to the bottom of the bore hole and set by driving it an initial 6 inches. It is then driven an additional 12 inches and the number of blows required to advance the sampler the additional 12 inches is recorded as the N-value.

Relatively undisturbed soil samples for laboratory tests were obtained by driving a California modified split barrel sampler into the soil. The soil was retained in stainless steel rings, 2.5 inches O.D. and 1 inch in height. The lower 6 inch portion of the samples were placed in close-fitting, plastic, air-tight containers which, in turn, were placed in cushioned boxes for transport to the laboratory.

Soil samples obtained were taken to Moore Twining's laboratory for classification and testing.

4.2 <u>Laboratory Testing</u>: The laboratory testing was programmed to determine selected physical and engineering properties of the samples tested. The tests were conducted on disturbed and relatively undisturbed samples considered representative of the subsurface material encountered.

The results of laboratory tests are summarized in Appendix C. These data, along with the field observations, were used to prepare the final test boring logs in Appendix B.

5.0 FINDINGS AND RESULTS

The findings and results of the field exploration and laboratory testing are summarized in the following subsections.

5.1 Subsurface Profile: The subsurface soils encountered in boring B-1 and borings B-3 through B-6 consisted of undocumented fill soils extending to depths ranging from about 1½ to 3½ feet BSG. The undocumented fill soils consisted of silty sands, silty clayey sands with gravel, and sandy lean clays. The undocumented fill soils typically contained gravel and/or debris such as brick and concrete debris. The undocumented fill soils were underlain by native soils consisting of various interbedded, discontinuous layers of sandy lean clays, clayey sands, poorly graded sands with silt, silty sands, poorly graded sands, and sandy fat clays which extended to depths of about 10 to 15 feet BSG.

Where native soils were encountered at the ground surface in borings B-2, B-7 and B-8, the native soils consisted of either silty, clayey sands or sandy lean clays which extended to a depth of about 5 feet BSG. Below a depth of 5 feet, the soils encountered consisted of various interbedded, discontinuous layers of clayey sands with gravel and sandy lean clays which extended to 12 feet BSG in boring B-2 or to the maximum depth explored of 6½ feet BSG in boring B-7.

Where encountered at depths of about 10 to 15 feet BSG, the native Marine terrace deposits (Qtm), were underlain by Santa Cruz mudstone (Tsc) that generally consisted of slightly friable, moderately weathered and thinly bedded siltstone to the maximum depth explored, about 22⅓ feet BSG. It should be noted that an apparent hydrocarbon type odor was noted within various samples, but was generally more noticeable within the siltstone samples.

5.2 Soil Engineering Properties: The engineering properties of the subsurface soils encountered during this investigation are summarized below.

Silty Sand Fill Soils - The silty sand fill soils encountered were loose, as indicated by a standard penetration resistance, N-value, of 6 blows per foot. The moisture content of one sample tested was 8.0 percent.

Sandy Lean Clay Fill Soils - The sandy lean clay fill soils encountered were described as medium stiff to stiff, as indicated by standard penetration resistance, N-values, of 5 and 10 blows per foot. The moisture content of a sample tested was 14.9 percent.

Silty Clayey Sand with Gravel Fill Soils - The silty clayey sand with gravel fill soils encountered was loose, as indicated by a standard penetration resistance, N-value of 10 blows per foot. The moisture content of a sample tested was 6.7 percent. An Atterberg Limits test conducted on a near surface bulk sample collected from the surface to a depth of 1½ feet BSG from boring B-5 indicated a liquid limit of 24 and a plasticity index of 7.

Native Sandy Lean Clays- The native sandy lean clays encountered were described as very soft to hard, as indicated by standard penetration resistance, N-values, ranging from 3 to greater than 50 blows per foot. The moisture content of the samples tested ranged from about 11.7 to 31.7 percent. Two (2) relatively undisturbed samples revealed dry densities of 99.5 and 103.9 pounds per cubic foot. The results of a direct shear test conducted on a sample collected from boring B-3 at a depth of 3½ to 5 ft BSG indicated an internal angle of friction of 33 degrees with a cohesion value of 360 pounds per square foot. An Atterberg Limits test conducted on a near surface bulk sample collected from boring B-1 at depths of 2 to 3½ feet BSG indicated a liquid limit of 22 and a plasticity index of 7. The result of one (1) consolidation test conducted on a sample collected from Boring B-3 at a depth of 3½ to 5 feet BSG indicated medium compressibility characteristics (about 7.3 percent consolidation under a load of 16 kips per square foot).

Native Clayey Sands and Clayey Sands with Gravel- The native clayey sands encountered were described as loose to dense, as indicated by standard penetration resistance, N-values, ranging from 9 to 16 blows per foot, and SPT equivalent N-values (estimated by driving a California Modified split barrel sampler) of 14 and 32 blows per foot. The moisture contents of the samples tested ranged from 18.7 to 30 percent. Two (2) relatively undisturbed samples revealed dry densities of 103.1 and 103.8 pounds per cubic foot. The result of one (1) consolidation test conducted on a sample collected from Boring B-1 at a depth of 5 to 6½ feet BSG indicated moderate compressibility characteristics (about 6 percent consolidation under a load of 16 kips per square foot).

Native Poorly Graded Sands and Poorly Graded Sands with Silt- The native poorly graded sands and poorly graded sands with silt encountered were described as loose to medium dense, as indicated by SPT equivalent N-values (estimated by driving a California Modified split barrel sampler) of 9 and 26 blows per foot. The moisture contents of the samples tested ranged from about 3.2 to 25.0 percent. Two (2) relatively undisturbed samples revealed dry densities of 94.8 and 111.7 pounds per cubic foot. The results of a direct shear test conducted on a sample collected from boring B-5 at a depth of 10 to 11½ ft BSG indicated an internal angle of friction of 32 degrees with a cohesion value of 250 pounds per square foot. The result of one (1) consolidation test conducted on a sample collected from boring B-3 at depths of about 3½ to 5 feet BSG indicated low compressibility characteristics (about 3.8 percent consolidation under a load of 16 kips per square foot).

Native Silty Clayey Sands: The silty clayey sands encountered were described as very loose to loose, as indicated by standard penetration resistance, N-values, ranging from 2 to 4 blows per foot The moisture content of a sample tested was about 20 percent. An Atterberg Limits test conducted on a near surface bulk sample collected from boring B-2 the surface to a depth of 1 to 2½ feet BSG indicated a liquid limit of 19 and a plasticity index of 4.

Native Silty Sands - The native silty sands encountered were described as very loose to medium dense, as indicated by a standard penetration resistance, N-value, ranging from 9 to 19 blows per foot, and an SPT equivalent N-value (estimated by driving a California Modified split barrel sampler) 1 blow per foot. The moisture contents of the samples tested ranged from about 20.9 to 23.4 percent. Two (2) relatively undisturbed sample revealed dry densities of about 89.1 and 92.7 pounds per cubic foot. The result of one (1) consolidation test conducted on a sample collected from Boring B-3 at a depth of 10 to 11½ feet BSG indicated low compressibility characteristics (About 4.5 percent consolidation under a load of 16 kips per square foot).

Native Sandy Fat Clay Soils - The native sandy fat clay soils encountered were described as medium stiff, as indicated by standard penetration resistance, N-value, of 7 blows per foot. The moisture content of a sample tested was about 28.4 percent.

Santa Cruz Mudstone (Siltstone): The siltstone encountered was described as very dense as indicated by standard penetration resistance, N-values, of greater than 50 blows per foot. The moisture content of the samples tested ranged from about 25.5 to 39.9 percent.

Expansion Index Tests: The results of an expansion index test conducted on a near surface silty clayey sand sample from boring B-2 indicated an expansion index value of 18. The results of an expansion index test conducted on a near surface sample containing a mixture of sandy lean clay fill and native clayey sand with gravel indicated an expansion index value of 5.

Maximum Density-Optimum Moisture Determination: A maximum density-optimum moisture determination conducted on a sample containing a mixture of silty sand fill and native sandy lean clay collected from depths of 0 to 5 feet BSG from boring B-1 indicated a maximum dry density of 123.8 pounds per cubic foot at an optimum moisture content of 8.8 percent. A maximum density-optimum moisture determination conducted on a silty clayey sand with gravel sample collected from depths of 0 to 3½ feet BSG from boring B-5 indicated a maximum dry density of 126.2 pounds per cubic foot at an optimum moisture content of 8.4 percent.

Resistance-value (R-value) Tests: Three (3) R-value tests were performed for the new pavements proposed for the subject project. An R-value test conducted on a sample containing a mixture of silty sand fill and native sandy lean clay sample collected from depths of 0 to 5 feet BSG from boring B-6 indicated an R-value of 38. An R-value test conducted on a sandy lean clay sample collected from depths of 0 to 5 feet BSG from boring B-7 indicated an R-value of 33. An R-value test conducted on a sandy lean clay sample collected from depths of 0 to 5 feet BSG from boring B-8 indicated an R-value of 29.

Chemical Tests: Chemical tests were performed on two (2) near surface samples. A sample collected from boring B-1 indicated a pH value of 6.8; a minimum resistivity value of 8,900 ohm-centimeters; and less than 0.0040 percent by weight concentrations of chloride and sulfate. A sample collected from boring B-4 indicated a pH value of 6.8; a minimum resistivity value of 4,300 ohm-centimeters; 0.0071 percent by weight concentration of sulfate; and less than 0.0040 percent by weight concentration of chloride.

5.3 Groundwater Conditions: Groundwater was encountered in the majority of the borings at depths of about 3 to 6 feet BSG during drilling.

Based on our review of the groundwater data from wells on the California Department of Water Resources Sustainable Groundwater Management Act (SGMA) Data View Application and California State Water Resources Control Board Geotracker Application were reviewed. The SGMA application was found to not contain groundwater depth data points within the localized region surrounding the subject project site with relation to proximity or elevation.

Review of a groundwater monitoring well report from an open case file obtained from the online Geotracker application for the address 411 Swift Street indicated that a monitoring well, located within the northeast portion of the subject site, indicated a groundwater depth of about 3 feet below site grade in March of 1998, 1999, and 2011.

It should be recognized, however, that groundwater is dependent upon seasonal precipitation, irrigation, land use, and climatic conditions as well as other factors. Therefore, observations at the time of the field investigation may vary from those encountered both during the construction phase and the design life of the project. Actual groundwater levels during construction could be higher or lower than indicated in this report. The evaluation of such factors was beyond the scope of this investigation and report.

6.0 GEOLOGIC AND SEISMIC HAZARDS EVALUATION

6.1 Geomorphic and Geologic Setting: The City of Santa Cruz, General Plan Update, Geology and Seismology indicates: "*The City of Santa Cruz is located on the southwestern slope of the central Santa Cruz Mountains, part of the Coast Ranges physiographic province of California. The northwest-southeast structural grain of the Coast Ranges is controlled by a complex of active faults within the San Andreas fault system. Southwest of the San Andreas fault in the Santa Cruz area, the Coast Ranges, including the Santa Cruz Mountains, are underlain by a large, northwest-trending, fault-bounded, elongate prism of granitic and metamorphic basement rocks. The granitic and metamorphic basement is Cretaceous in age, or older and is overlain by a sequence of dominantly marine sedimentary rocks of Paleocene to Pliocene age and non-marine sediments of Pleistocene and Holocene age. The older sedimentary rocks are moderately to strongly deformed, with steep-limbed folds and several generations of faults associated with uplift of the Santa Cruz Mountains.*

The region is tectonically active, that is, it is subject to forces causing the earth's crust to deform. The deformation can occur as movement on active faults, folding of layered rocks, or down warping or uplifting of portions of the crust. All these processes are active in the Santa Cruz area. The Santa Cruz Mountains are cut by several active faults, of which the San Andreas is the most important. Along the coast, the ongoing tectonic activity is most evident in the gradual uplift of the coastline, as indicated by the series of uplifted marine terraces that sculpt the coastline. The Loma Prieta earthquake of 1989 and its aftershocks are recent reminders of the geologic unrest in the region."

The Geologic Map of the Monterey 30'x60' Quadrangle and Adjacent Areas, California, compiled by David L. Wagner, H. Gary Greene, George Saucedo, and Cynthia L. Pridmore, 2002, indicates that the area supporting the subject project site is underlain by Pleistocene-age Marine Terrace Deposits (Qmt). Wagner, Greene, Saucedo, and Pridmore describe many different types of coastal terrace deposits but generally describe them as gravel, sand, silt, and clay deposited on wave cut surfaces. Siltstone rock, interpreted to be the Santa Cruz mudstone was encountered below the Marine Terrace Deposits.

Regional geologic and site geologic maps are included as Drawing Nos. 3 and 6 in Appendix A, respectively, and geologic cross sections through the building area is included as Drawing No. 7 in Appendix A of this report.

6.2 Tectonics and Seismicity: Numerous active and potentially active faults are located in the site region and contribute to design seismic ground motion estimates. An "active fault" is defined, for the purpose of this evaluation, as a fault that has had surface displacement within the Holocene Epoch (about the last 11,700 years). Potentially active faults also contribute to the seismicity of the site. A widely accepted definition of "potentially active" is a fault that does not show evidence of displacement during the Holocene, but shows evidence of displacement occurring less than 1.6 million years ago (after the beginning of the Pleistocene Epoch). Since the Quaternary Period includes the Holocene and Pleistocene Epochs, Quaternary Faults are those that have documented latest fault displacement within the Holocene or Pleistocene Epochs, undifferentiated. Faults showing evidence of displacement older than 1.6 million years are usually classified as "Pre-Quaternary" and "inactive." Pre-Quaternary faults do not impact the site seismicity.

The City of Santa Cruz, General Plan Update indicates, "*Locally, the San Andreas, Zayante-Vergeles, and San Gregario faults, and the Monterey Bay Tularcitos fault zone are thought to present a significant seismic hazard to the City. These faults are associated with Holocene activity (movement in the last 11,000 years) and are therefore considered to be active.*" The City of Santa Cruz, General Plan Update also indicated that the most severe historical earthquakes to affect the local Santa Cruz area are the 1906 San Francisco earthquake and the 1989 Loma Prieta earthquake, with Richter magnitudes of about 8.3 and 7.1, respectively. Drawing No. 4 in Appendix A of this report shows the active and potentially active faults near the site from the 2010 Fault Activity Map of California: California Geological Survey, Geologic Data Map No. 6, prepared by C.W. Jennings and W.A. Bryant.

Based on the results of deaggregation of the 2,475 year return period event (2 percent probability of exceedance in 50 years) using the United State Geological Survey deaggregation website (https://earthquake.usgs.gov/hazards/interactive/), Dynamic Conterminous U.S. 2014 Edition (update, v4.2.0); the nearby San Andreas (Santa Cruz Mountains Section), the San Gregario, and Seaside-Monterey Section segment of the Monterey Bay-Tularcitos fault zone are the predominant contributing source of seismicity for the site area. The Reliz fault zone, Zayante-Vergeles fault zone, and gridded sources contribute decreasingly lower percentages of the total seismicity of the site area.

7.0 EVALUATIONS

The data and methodology used to develop conclusions and recommendations for project design and preparation of construction specifications are summarized in the following subsections. The evaluation was based upon the subsurface soil conditions determined from the field exploration and laboratory testing program and our understanding of the proposed construction. The conclusions obtained from the results of our evaluations are described in the Conclusions section of this report.

7.1 GEOLOGIC HAZARDS: The potential geologic hazards of landslides and conditional geologic hazards are described in the following subsections.

7.1.1 Landslides: The potential for landslides to impact the site were reviewed based on the Local Hazard Mitigation Plan, 2021 - 2026, Geologic Hazards Map for Santa Cruz County application (website listed in the References Section of this report). The site is relatively flat and the mapping indicates the project area is not within the landslide hazard areas or areas with slopes greater than 50 percent. Figure 5 from the City of Santa Cruz, General Plan Update, Geology and Seismology, also shows the site and surrounding area as being mapped outside of any mapped definitive, probable or questionable landslides. Consequently, the potential impacts to the site from active landsliding is considered low.

7.1.2 Conditional Geologic Hazards: Conditional geologic hazards, as identified in section 31 of California Geological Survey Note 48, are discussed in the following subsections.

7.1.2.1 Hazardous Materials: Hazardous materials such as methane gas, hydrogen-sulfide gas and tar seeps are not known to be present in the project area and are not considered to be a concern at the subject site.

7.1.2.2 Volcanic Activity: California includes six regions with a history of late Pleistocene volcanic eruptions that are subject to hazards from future eruptions (Miller, 1989). Of these six regions, none of these areas are located within 100 miles of the site. The nearest regions with a history of late Pleistocene volcanic eruptions are the Mono-Lake Long Valley area, located about 175 miles east-northeast of the site. Based on review of Plate 1 (Miller, 1989), the subject site is not located within any designated volcanic hazard zones (such as flowage, pyroclastic flows, debris flows, or ash fall hazards). The closest volcanic hazard to the site is indicated to be ash fall hazard from an eruption in the Mono Lake-Long Valley Area, impacting areas greater than 127 miles northeast of the subject site, as measured from an area between Buck Meadows and Harden Flat.

Considering the distance from the site, the potential for lava flows or significant ash falls at the site during the design life of the project is considered low.

7.1.2.3 **Flooding**: The Flood Insurance Rate Map, Community Panel number 06087C0333F, distributed by the Federal Emergency Management Agency (FEMA), dated September 29, 2017, indicates the site is not located in a designated Flood Hazard Zone. The Community panel shows the nearest designated Flood Hazard Zone is about ½ mile west of the site near the Moore Creek.

Based on our review of the County of Santa Cruz Local Hazard Mitigation Plan, 2021 Update, Chapter 10 Dam Failure, prepared by the County of Santa Cruz Planning Department, the site is not located in or near a dam failure inundation area. Figure 22 "Newell Creek Dam Inundation Area" from the County of Santa Cruz Local Hazard Mitigation Plan, 2021 Update, Chapter 10 Dam Failure shows that inundation from breach of the Newell Dam would affect areas greater than 1 mile northeast of the site.

7.1.2.4 **Tsunamis and Seiches**: Tsunamis are waves generated in oceans from seismic activity. The potential for tsunamis to impact the site was researched by reviewing the Tsunami Hazard Area Map, County of Santa Cruz, dated October 7, 2022, produced collectively by the California Governor's Office of Emergency Services, the California Geological Survey, AECOM Technical Services, and the Tsunami Research Center at the University of Southern California . The Santa Cruz County Tsunami Hazard Areas map indicates that the site area is not located in a mapped area of tsunami inundation. The nearest areas that would be affected by a tsunami are shown on the Tsunami Hazard Area Map to be greater than ¼ mile from the subject site. Thus, the potential for a tsunami to impact the site is considered to be low.

A seiche is a wave generated by the periodic oscillation of a body of water whose period is a function of the resonant characteristics of the containing basin as controlled by its physical dimensions. These periods generally range from a few minutes to an hour or more. The closest body of water is the Antonelli Pond (within the pathway of Moore Creek) that is about ½ mile west-northwest of the site and is about 17 feet lower in elevation than the subject site. Due to the distance from this body of water, and the elevation of this body of water relative to the subject site, seiches are not considered a significant hazard at the site.

7.1.2.5 **Radon Gas:** The Indoor Radon Potential map prepared by the California Geological Survey (online map application www.maps.conservation.ca.gov/cgs/radon), was reviewed to evaluate the potential for exposure to radon gas at the site. The map includes data for the Monterey County area based on Special Report 216, titled "Radon Potential In Santa Cruz County," prepared by the California Geological Survey, dated 2010. The map identifies a low potential for indoor radon in the site area, which is defined as "0 to 4.9 percent of home indoor-radon measurements are likely to exceed the U.S. EPA recommended action level of 4 picocuries per liter."

The California Geologic Survey (CGS) on the California Department of Conservation (CDC) website was also reviewed for California Indoor Radon Levels Sorted by an interactive map. The interactive map shows the site as being mapped in a green-colored zone which is defined as a low radon potential zone.

In addition, the California Department of Public Health Services (CDPH) on the California Geologic Survey (CGS) website was also reviewed for California Indoor Radon Levels Sorted by Zip Code, last updated February 2017. Based on the zip code for the site (95060), the CGS website indicates only 39 of the 538 measurements (approximately 7.2 percent) were 4 picocuries per liter or higher.

7.1.2.6 **Naturally Occurring Asbestos:** Asbestos occurs in soil and rock naturally in certain geologic settings in California. It has been documented that inhalation of asbestos fibers may cause negative health effects. Most commonly, naturally occurring asbestos is associated with serpentinite and partially serpentized ultramafic rocks. Ultramafic rocks are scattered throughout much of the Coast Range region. The document: "Reported Historic Asbestos Mines, Historic Asbestos Prospects, and Other Occurrences of Asbestos in California;" Open-File Report 2011-1188, prepared by the U.S. Department of the Interior, U.S Geological Survey, dated August, 2011, does not identify ultramafic rock in the site vicinity. The referenced report indicates the closest occurrences of ultramafic rocks are about 14 miles northeast of the site.

Thus, based on the cited literature and our site observations, it is our opinion that the potential to encounter naturally occurring asbestos containing rock during construction of the project is very low.

7.1.2.7 **Hydrocollapse:** As a part of this investigation, three (3) consolidation tests were conducted on relatively undisturbed near surface soils within the upper 6½ feet BSG and one (1) consolidation test was conducted on a sample collected from depths of 10 to 11½ feet BSG. The test results indicated none, or insignificant collapse potential. Given these results, the recommendations for preparation of the site, and the geologic nature of the subsurface materials, hydrocollapse is not considered a significant concern for this project.

7.1.2.8 **Regional Subsidence:** Based on our review of an online map published by the California Water Science Center, the site is not located in an area of recorded subsidence (see https://ca.water.usgs.gov/land_subsidence/california-subsidence-areas.html).

7.1.2.9 **Clays and cyclic softening:** Potential for cyclic softening of the clays encountered was evaluated by comparing the plasticity index of the soils encountered with the lower limit value that would indicate a susceptibility to softening during seismic shaking. One of the shallow sandy silty clay samples tested from depths of 2 to 3½ feet BSG from boring B-1 indicated a plasticity index of 7. The plasticity index of the clay layer tested is less than 12, which indicates a potential for cyclic softening. However, provided the recommendations included in this report to mitigate potential liquefaction are followed, the potential for cyclic softening of clays to affect the proposed building foundation support will be low.

7.2 SEISMIC HAZARDS: The potential for fault ground rupture, seismic groundshaking and seismic coefficients/earthquake spectral response acceleration design values, and liquefaction and seismic settlement are described in the following subsections.

7.2.1 Active Faulting and Surface Fault Rupture: Earthquakes are caused by the sudden displacement of earth along faults with a consequent release of stored strain energy. The fault slippage can often extend to the ground surface where it manifests in abrupt relative ground displacement across a rupture surface. Damage resulting directly from fault rupture ground displacement occurs only where structures are located astride a rupture surface with relative displacement.

The project site is not located in an Alquist-Priolo Earthquake Fault Zone or a known local earthquake fault rupture hazard zone. The locations of active faults were evaluated using the 2010 Fault Activity Map of California: California Geological Survey, Geologic Data Map No. 6, prepared by Jennings and Bryant. A map depicting the major active faults in the vicinity of the site is included on Drawing No. 4 in Appendix A of this report.

The Fault Activity Map of California indicates the nearest fault to the site is the Ben Lomond Fault, which is located about 1.5 miles east of the proposed Workforce Housing building project. The Fault Activity Map of California shows the Ben Lomond Fault as extending north and south from a portion of the Zayante Fault Section, extending as far south as Felton, several miles north of Santa Cruz, extending the fault southward through the city of Santa Cruz extending into Monterey Bay. The location of the Ben Lomond Fault relative to the site is shown on Drawing No. 4 using the 2010 Fault Activity Map of California: California Geological Survey, Geologic Data Map No. 6, prepared by Jennings and Bryant. The following was provided by the City of Santa Cruz, General Plan Update, Geology and Seismology "*Most of the movement occurred on this fault prior to about six million years ago and it is not presently considered to be active (Stanley and McCaffrey, 1983; Cao et al., 2003)*". The southern most portion of the Ben Lomond fault is about 1.5 miles east of the subject site and is colored green on the Fault Activity Map of California, indicating Late Quaternary fault displacement (during the past 700,000 years), see Drawing No. 4 in Appendix A of this report. Considering that this portion of the Ben Lomond fault had fault displacement greater than 11,700 years ago, the fault is not considered to be an "active" fault.

The nearest known "active" fault, the Monterey Bay - Tularcitos Fault Zone, is located about 4¾ miles southwest of the project site.

Considering the distance to the nearest known "active" fault (Monterey Bay - Tularcitos Fault Zone), the potential for surface fault rupture at the site due to a known "active" fault is considered low.

7.2.2 Ground Shaking: For any given earthquake, the rock in the immediate vicinity will respond with a certain maximum acceleration and with a predominant period that depends on the nature of the rock and the source mechanism. Away from the focus of the earthquake, the ground motions begin to attenuate. The way in which the earthquake wave is altered depends to a great degree on source characteristics and to a lesser degree on the travel path.

A summary of our review of historic seismic activity relative to the site is included below.

7.2.2.1 Historic Seismic Activity: The general area of the site has experienced recurring seismic activity. The City of Santa Cruz, General Plan Update indicated that the most severe historical earthquakes to affect the local Santa Cruz area are the 1906 San Francisco earthquake and the 1989 Loma Prieta earthquake, with Richter magnitudes of about 8.3 and 7.1, respectively. Based on historical earthquake data obtained from the U.S. Geological Survey's earthquake database system, approximately 119 historical earthquakes with magnitude 4.5 or greater have been recorded from 1900 through August 29, 2023 within about 50 miles of the site. A map showing the location of the project site with relation to the approximate historical earthquake epicenter locations and magnitude category is presented on Drawing No. 5 in Appendix A of this report.

The nearest earthquake event (estimated magnitude of 4.5 and estimated peak ground acceleration at the subject site of 0.142g) found during the search occurred about 4.1 miles northeast of the site (2km east-southeast of Pasatiempo area) on February 12, 1938. The largest magnitude earthquake identified within the 50 mile search radius was the 6.9 magnitude Loma Prieta earthquake, which occurred on October 17, 1989, approximately 10.9 miles northeast of the site (estimated peak ground acceleration at the subject site of about 0.298g).

7.2.2.2 Seismic Design Parameters: It is our understanding that the 2022 CBC will be used for structural design, and that seismic site coefficients are needed for design.

Based on the 2022 CBC, a Site Class D was assigned for the on-site soil conditions with standard penetration resistance, N-values, averaging between 15 and 50 blows per foot in the upper 100 feet below site grade.

A Maximum Considered Earthquake (geometric mean) peak ground acceleration adjusted for site effects (PGA_M) of 0.741g was determined for the site using the Ground Motion Parameter Calculator provided by SEOAC and OSHPD (http://seismicmaps.org)

7.2.3 Liquefaction and Seismic Settlement: Liquefaction and seismic settlement are conditions that can occur under seismic shaking from earthquake events. Liquefaction describes a phenomenon in which a saturated, cohesionless soil loses strength during an earthquake as a result of induced shearing strains. Lateral and vertical movements of the soil mass, combined with loss of bearing can result. Fine, well sorted, loose sand, shallow groundwater conditions, higher intensity earthquakes, and particularly long duration of ground shaking are the requisite conditions for liquefaction.

The site is not located in an area which has been mapped by the State of California for seismic hazards such as liquefaction. The “City of Santa Cruz, Local Hazard Mitigation Plan, Five Year Update 2018-2023", Chapter 4, Figure 6 - “Areas in Santa Cruz Potentially Vulnerable to Liquefaction” indicates the subject site is located within a “Low” susceptibility area for liquefaction,

However, based on the low standard penetration test, N-values, and the granular nature of many of the soils within the upper 10 feet encountered during this investigation, and the presence of shallow groundwater, the potential for liquefaction or seismic settlement to impact the project is moderate to high.

For the purpose of the liquefaction and seismic settlement analyses, an historic high groundwater of 2½ feet AMSL was used. Liquefaction and seismic settlement analyses were conducted based on soil properties revealed by the subsurface exploration in accordance with the NCEER (Youd, et. al. 2001) method using the computer program LiquefyPro, developed by CivilTech Software.

A Maximum Considered Earthquake (geometric mean) peak ground acceleration adjusted for site effects (PGAM) of 0.741g was determined for the site using the Ground Motion Parameter Calculator provided by SEAOC and OSHPD (http://seismicmaps.org). A Maximum Considered Earthquake magnitude of 6.27 was applied in the analysis based on deaggregation analysis (United States Geological Survey deaggregation website, Dynamic Conterminous U.S. 2014, V4.2.0).

Soil parameters, such as wet unit weight, SPT blow (N) count, and fines content were input from laboratory testing and the auger drill boring data for the soil layers encountered throughout the depths explored

The findings of the liquefaction analyses indicate that numerous layers of granular soils, with thicknesses ranging from less than a foot to about 3 feet thick within the upper about 10 to 15 feet BSG, are susceptible to liquefaction as a result of the Maximum Considered Earthquake. The depths of the layers of liquefiable soil from boring B-4 occur within the upper 13½ feet BSG and include loose clayey sand with gravel and very loose silty sand with gravel. In addition, soils susceptible to liquefaction occur within the upper ~6 feet in many other borings. The results of the seismic settlement analyses indicated a total seismic settlement of about 3 to 4 inches due to liquefaction., Based on the analysis, the liquefiable soils are within the zone of influence of the proposed building foundations. This could result in partial loss/reduction of structural support of the foundations. Thus, mitigation of liquefaction will be required for support of the structure as discussed in the following Section 7.2.4 of this report.

7.2.4 <u>Mitigation of Bearing Support Failure due to Liquefaction:</u> Liquefaction and partial loss/reduction of foundation bearing support is a potential impact resulting from the design level seismic event. Based on the potential impacts of liquefaction, specialty deep ground improvement methods are recommended in this report to mitigate the potential for liquefaction and foundation bearing failure as well as to provide support of the foundations and slab on grade. It should be noted that due to the relatively close proximity of existing building structures to the subject site, vibratory installation methods may adversely effect surrounding building structures and improvements by inducing differential settlement. In addition, the site/surrounding subsurface environmental conditions may need to be considered with regard to selection of the type of deep ground modification. From a geotechnical engineering standpoint, if vibration to adjacent buildings is a concern, deep ground modification such as rigid inclusions or deep cement soil mixing could be used to mitigate the effects of liquefaction as well as to provide structural support of the proposed building and floor slab loads for the structure. These ground modification methods are typically designed and constructed by specialty ground modification contractors. In order to use the deep ground modification elements for support of the structure and floor slab loads, the foundation design will need to be coordinated with the design of the deep ground modification system. The building foundation system and slab on grade may need to be stiffened in order to span between ground modification elements.

As an alternative to deep ground modification, the structure could be designed with a structural floor slab and supported on pile foundations extending into the underlying sedimentary rock for support below the liquefiable soils. However, deep foundations would need to be designed for downdrag loading due to liquefaction. In addition, deep foundation systems would need to provide temporary support of the drilled shafts due to the shallow groundwater conditions and subsurface soil conditions encountered. If a deep foundation system were used, methods such as auger cast piles would generally be well suited for foundation construction since the drilling tool provides casing of the drilled hole during construction. The costs, and advantages and disadvantages of a deep foundation system compared with deep ground modification would need to be evaluated to determine the preferred approach. It should be noted that if deep ground modification is not conducted to densify and/or reinforce the soils to mitigate liquefaction, elements such as new underground utilities below the building would not be protected from the effects of liquefaction, unless underground utility services were also supported on a deep foundation.

Other approaches such as a mat foundation were also considered; however, mat foundations would be subject to loss/reduction of bearing capacity in the event of liquefaction.

7.3 **SOIL ENGINEERING:** The data and methodology used to develop conclusions and recommendations for project design and preparation of construction specifications are summarized in the following subsections. The evaluation was based upon the subsurface soil conditions encountered during this investigation and our understanding of the proposed construction. The conclusions obtained from the results of our evaluations are described in the Conclusions section of this report.

7.3.1 **Existing Surface Improvements:** At the time of our field exploration, the area of the proposed three-story Workforce Housing project was occupied by various improvements including asphalt concrete pavement, Portland cement sidewalk and curb, underground utilities, various stored materials, and areas supporting weeds/landscaping, bushes, and mature trees. In addition, underground utilities are located within the area of proposed construction. Removal of these existing features will be required as part of the proposed project. As part of the site preparation, all soils which are disturbed as a result of the removal of the existing surface and subsurface improvements should be excavated to expose undisturbed native soils. All excavations should be backfilled with compacted engineered fill.

7.3.2 **Undocumented Fills:** Fill soils were encountered in boring B-1 and borings B-3 through B-6 as extending to depths of about 1½ to 3½ feet BSG. The fill soils generally consisted of very loose to loose silty sands, loose silty clayey sands with gravel, and medium stiff to stiff sandy lean clays. As part of site preparation, the undocumented fills should be fully removed from areas of all new buildings and site improvements. Debris such as brick and concrete were encountered in one of the borings. Where encountered, debris such as brick and concrete should be removed from soils to be reused as engineered fill, and the debris should be disposed of off-site in a legal manner. Removal of underground utilities should include removal of all associated trench backfill soils. Contractors should assume that the existing trench backfill soils extend from the bottom of the utility to the ground surface at a 1H:1V inclination.

7.3.3 **Wet Soils and Shallow Groundwater:** Due to the shallow groundwater conditions, excavations are anticipated to expose wet, unstable soils. In addition, dewatering and stabilization of wet unstable soils should be anticipated at the bottom of excavations. Where unstable soil conditions are experienced, methods such as chemical (i.e., lime) treatment of the soil, or use of a bridge lift with aggregate base and a geotextile stabilization fabric such as Mirafi 600X or equivalent, may be required to achieve a stable condition for subgrade preparation. Excavated soils to be reused as engineered fill will also likely need to be aerated, mixed with drier soils, chemically treated to be reused as engineered fill, or replaced with drier imported granular fill soils that meet the recommendations of this report.

7.3.4 Expansive Soils: One of the potential geotechnical hazards evaluated at this site is the expansion potential of the near surface soils. Over time, expansive soils will experience cyclic drying and wetting as the dry and wet seasons pass. Expansive soils experience volumetric changes (shrink/swell) as the moisture content of the clayey soils fluctuate. These shrink/swell cycles can impact foundations and lightly loaded slabs-on-grade when not designed for the anticipated expansive soil pressures. Expansive soils cause more damage to structures, particularly light buildings and pavements, than any other natural hazard, including earthquakes and floods (Jones and Holtz, 1973). Expansion potential may not manifest itself until months or years after construction. The potential for damage to slabs-on-grade and foundations supported on expansive soils can be reduced by placing non-expansive fill below the slabs-on-grade.

In evaluation of the potential for expansive soils at the site, expansion index testing was performed on a representative sample of the near surface soils which are anticipated to be within the zone of influence of the planned improvements. The expansion index testing was performed in accordance with ASTM D4829 and the results are included in Appendix C of this report. The results of the expansion index (EI) testing indicated that the near surface clayey samples tested have a very low expansion potential (EI = 5 and 18). However, for constructability, this report recommends that slabs on grade are underlain by at least 6 inches of Class 2 aggregate base.

7.3.5 Asphalt Concrete (AC) Pavements: Recommendations for onsite asphalt concrete pavement structural sections are presented in the "Recommendations" section of this report. The structural sections were designed using the gravel equivalent method in accordance with the California Department of Transportation Highways Design Manual. The analysis was based on traffic index values ranging from 5.0 to 8.0. The appropriate paving section should be determined by the project civil engineer or applicable design professional based on the actual vehicle loading (traffic index) values. If traffic loading is anticipated to be greater than assumed, the pavement sections should be re-evaluated.

It should be noted that if the pavements are constructed prior to the building construction, the additional construction truck traffic should be considered in the selection of the traffic index value. If more frequent or heavier traffic is anticipated and higher Traffic Index values are needed, Moore Twining should be contacted to provide additional pavement section designs.

The results of the R-value tests from near surface samples collected from borings B-6 through B-8 (drilled in proposed pavement areas) indicated R-values of 29, 33, and 38.

Based on the results of the testing and the procedures in the Caltrans Highway Design Manual, an R-value of 30 was used for the pavement design.

7.3.6 Portland Cement Concrete (PCC) Pavements: Recommendations for Portland cement concrete (PCC) pavement structural sections are presented in the "Recommendations" section of this report. The PCC pavement sections are based upon the amount and type of traffic loads being considered and the strength of the subgrade soils which will support the pavement. The measure of the amount and type of traffic loads are based upon an index of equivalent axle loads (EAL) from the loading of heavy trucks called a traffic index (T.I).

The results of R-value testing performed in accordance with ASTM Test Method were used to estimate the pavement subgrade modulus. Based on a design R-value of 30, a modulus of subgrade reaction, K-value, for the pavement section, of 185 psi/in was used for PCC placed directly on 4 inches of aggregate base.

The pavement design application StreetPave www.pavementdesigner.org ("Street" Analysis), which is consistent with ACI 330.2R ("Guide for the Design and Construction of Concrete Site Paving for Industrial and Trucking Facilities"), was utilized to determine the recommended PCC section thicknesses. A range of truck traffic and a design period of 20 years were used as inputs in the design.

7.3.7 Soil Corrosion: The risk of corrosion of construction materials relates to the potential for soil-induced chemical reaction. Corrosion is a naturally occurring process whereby the surface of a metallic structure is oxidized or reduced to a corrosion product such as iron oxide (i.e., rust). The metallic surface is attacked through the migration of ions and loses its original strength by the thinning of the member.

Soils make up a complex environment for potential metallic corrosion. The corrosion potential of a soil depends on numerous factors including soil resistivity, texture, acidity, field moisture and chemical concentrations. In order to evaluate the potential for corrosion of metallic objects in contact with the onsite soils, chemical testing of soil samples was performed by Moore Twining as part of this report. The test results are included in Appendix C of this report. Conclusions regarding the corrosion potential of the soils tested are included in the Conclusions section of this report based on the National Association of Corrosion Engineers (NACE) corrosion severity ratings listed in Table No. 1, below.

Table No. 1

Soil Resistivity (ohm cm)	Corrosion Potential Rating
>20,000	Essentially non-corrosive
10,000 - 20,000	Mildly corrosive
5,000 - 10,000	Moderately corrosive
3,000 - 5,000	Corrosive
1,000 - 3,000	Highly corrosive
<1,000	Extremely corrosive

The results of soil sample analyses indicate for samples tested in the area of the proposed building that the near-surface soils exhibit a "corrosive" to "moderately corrosive" corrosion potential to buried metal objects. Appropriate corrosion protection should be provided for buried improvements. If piping or concrete are placed in contact with imported soils, these soils should be analyzed to evaluate the corrosion potential of these soils.

The soil corrosion data should be provided to the manufacturers or suppliers of materials that will be in contact with soils (pipes or ferrous metal objects, etc.) to provide assistance in selecting the protection and materials for the proposed products or materials. If the manufacturers or suppliers cannot determine if materials are compatible with the soil corrosion conditions, a professional consultant, i.e., a corrosion engineer, with experience in corrosion protection should be consulted to provide design parameters. Moore Twining does not provide corrosion engineering services.

7.3.8 Sulfate Attack of Concrete: Degradation of concrete in contact with soils due to sulfate attack involves complex physical and chemical processes. When sulfate attack occurs, these processes can reduce the durability of concrete by altering the chemical and microstructural nature of the cement paste. Sulfate attack is dependent on a variety of conditions including concrete quality, exposure to sulfates in soil, groundwater and environmental factors. The standard practice for geotechnical engineers in evaluation of the soils anticipated to be in contact with structural concrete is to perform laboratory testing to determine the concentrations of sulfates present in the soils. The test results are then compared with the exposure classes in Table 19.3.1.1 of ACI 318 to provide guidelines for concrete exposed to soils containing sulfates. It should be noted that other exposure conditions such as the presence of: seawater, groundwater with elevated concentrations of dissolved sulfates, or materials other than soils can result in sulfate exposure categories to concrete that are higher than the concentrations of sulfate in soil. The design engineer will need to determine whether other potential sources of sulfate exposure need to be considered other than exposure to sulfates in soil. The sulfate exposure classes for soils from Table 19.3.1.1 are summarized in the below table.

Table No. 2
ACI Exposure Categories for Water Soluble Sulfate in Soils

Sulfate Exposure Class (per ACI 318)	Water Soluble Sulfate in Soil (Percent by Mass)
S0	Less than 0.10 Percent
S1	0.10 to Less than 0.20 Percent
S2	0.20 to Less than or Equal to 2.00 Percent
S3	Greater than 2.00 Percent

Common methods used to resist the potential for degradation of concrete due to sulfate attack from soils include, but are not limited to the use of sulfate-resisting cements, air-entrainment and reduced water to cement ratios. The laboratory test results for sulfates are included in Appendix C of this report. Conclusions regarding the sulfate test results are included in the Conclusions section of this report.

8.0 CONCLUSIONS

Based on the data collected during the field and laboratory investigation, our geotechnical experience in the vicinity of the project site, and our understanding of the anticipated construction, the following conclusions are presented.

8.1 The site is considered suitable for the proposed construction with regard to support of the proposed building, provided the recommendations contained in this report are followed. It should be noted that due to liquefaction and shallow groundwater conditions, measures such as deep ground modification or pile foundations are recommended in this report to mitigate potential geotechnical hazards. The recommended design consultation and observation of foundations and earthwork activities by Moore Twining are integral to this conclusion.

8.2 Fill soils were encountered in boring B-1 and borings B-3 through B-6 as extending to depths of about 1½ to 3½ feet BSG. The fill soils generally consisted of very loose to loose silty sands, loose silty clayey sands with gravel, and medium stiff to stiff sandy lean clays. The undocumented fill soils typically contained gravel and/or debris such as brick and concrete debris. The undocumented fill soils were underlain by native soils consisting of various interbedded, discontinuous layers of very soft to hard sandy lean clays, medium dense clayey sands, medium dense poorly graded sands with silt, very loose to medium dense silty sands, loose poorly graded sand, and medium stiff sandy fat clays which extended to depths of about 10 to 15 feet BSG where the siltstone rock was generally encountered.

Where native soils were encountered at the ground surface in borings B-2, B-7 and B-8, the near surface soils consisted of either very loose silty clayey sands or very soft to medium stiff sandy lean clays. These near surface soils were underlain by various interbedded, discontinuous layers of dense clayey sands with gravel and very stiff to hard sandy lean clays over siltstone.

Where encountered, the native soils were underlain by Santa Cruz mudstone that generally consisted of slightly friable, moderately weathered and thinly bedded hard siltstone which extended to the maximum depth explored.

8.3 In order to reduce the potential for excessive static settlement of surface improvements which are sensitive to settlement, this report recommends over-excavation of the existing undocumented fill soils to support new surface improvements on compacted engineered fill soils.

8.4 The near surface soils tested exhibited a very low expansion potential, moderate compressibility characteristics, and fair support characteristics for pavements when compacted as engineered fill.

8.5 The results of the liquefaction analysis indicates relatively thick layers of granular soils below the groundwater level are susceptible to liquefaction in the event of the design earthquake ground motion. Due to the potential for liquefaction of relatively shallow layers, this could result in partial loss/reduction of structural support of the foundations. Due to the shallow depth of liquefaction, this report recommends either the use of specialty ground modification to support spread foundations; or deep foundations to reduce potential impacts from loss or reduction of foundation bearing capacity in the event of liquefaction. These approaches are discussed in Section 7.2.4 of this report.

8.6 During our May 10 and 11, 2023 investigation, groundwater was encountered in the majority of the borings at depths of about 3 to 6 feet BSG during drilling. Due to the shallow groundwater conditions, contractors should anticipate dewatering and control and management of groundwater will be required for construction of the project. In addition, buoyancy effects and appropriate waterproofing measures should be considered for design and construction.

8.7 Contractors should anticipate that the onsite soils will be overly moist and will require drying such as by chemical soil treatment prior to being used as engineered fill. Also, it should be anticipated that the base of excavations will require stabilization due to wet, unstable soil conditions.

8.8 The results of corrosion testing indicated that the near-surface soils exhibit a “moderately corrosive to corrosive” corrosion potential to buried metal objects. Based on Table 19.3.1.1 - Exposure categories and classes from Chapter 19 of ACI 318, the sulfate concentration from chemical testing of soil samples falls in the S0 classification.

8.9 Other than liquefaction, based on our assessment of potential geologic and seismic hazards, no geologic or seismic hazards, were identified as significant potential impacts to the project.

9.0 RECOMMENDATIONS

Based on the evaluation of the field and laboratory data and our geotechnical experience in the vicinity of the project, we present the following recommendations for use in the project design and construction. However, this report should be considered in its entirety. When applying the recommendations for design, the background information, procedures used, findings and conclusions should be considered. The recommended design consultation and construction monitoring by Moore Twining are integral to the proper application of the recommendations.

Where the requirements of a governing agency, utility agency or product manufacturer differ from the recommendations of this report, the more stringent recommendations should be applied to the project.

9.1 General

9.1.1 The project design will need to address measures to address liquefaction and loss of foundation bearing support for the proposed structure. Due to the presence of relatively shallow liquefaction, specialty ground modification or deep foundations are recommended to address potential impacts from reduction or loss of foundation bearing support. If specialty deep ground modification is used to mitigate liquefaction, the deep ground modification should also be designed to provide direct support of all static building and floor slab loads. Designs for ground modification should be provided to Moore Twining for review and comment.

9.1.2 Moore Twining should be retained to review the grading and foundation plans before the plans are released for bidding purposes so that any relevant recommendations can be presented. Additional or alternative recommendations may be warranted based on review of the foundation plans.

9.1.3 A preconstruction meeting including, as a minimum, the general contractor, foundation subcontractors, architect, owner and Moore Twining should be scheduled at least one week prior to the start of demolition. The purpose of the meeting should be to discuss critical project issues, concerns and scheduling.

9.1.4 The contractor should be responsible for protecting existing facilities from damage which are to remain. Any damage shall be repaired by the contractor at no cost to the owner(s).

9.1.5 Shallow groundwater conditions are anticipated at the site. Thus, construction dewatering should be anticipated for the project for excavations which may encroach the high groundwater conditions. Dewatering of excavations should be conducted such that the bottom of the excavations are relatively dry during placement of all engineered fill. Dewatering should be conducted in accordance with all applicable regulatory and permitting requirements.

9.1.6 Subsurface improvements should be designed for bouyant conditions based on the shallow groundwater.

9.1.7 Due to the shallow groundwater conditions, high moisture contents should be anticipated for the near surface soils. Where overly moist, and wet, unstable soil conditions are experienced, drying such as by chemical (i.e., lime, cement, etc.) treatment should be anticipated. In addition, over-excavation to an additional depth of 12 inches and placement of a bridge lift of bridging rock and a geotextile stabilization fabric such as Mirafi 600X, may be required to achieve a stable soil condition. The actual method employed to stabilize the bottom of the excavation should be submitted to, and approved by Moore Twining at the time of construction.

9.1.8 After review of the geotechnical report and the construction documents, the Contractor(s) bidding on this project should determine if the data are sufficient for accurate bid purposes. If the data are not sufficient, the Contractor should conduct, or retain a qualified geotechnical engineer to conduct, supplemental studies and collect more data as required to prepare accurate bids.

9.1.9 A demolition plan should be prepared to identify existing improvements such existing asphalt concrete, Portland cement hardscaping/sidewalk, concrete curb, underground utilities, that are to be demolished and/or removed as part of the project.

9.1.10 The contractor should use appropriate equipment such as low-pressure equipment, steel tracks, etc. to achieve the required over-excavation, compaction and site preparation to minimize rutting and subgrade instability.

9.2 Site Grading and Drainage

9.2.1 It is critical to develop and maintain site grades which will drain surface and roof runoff away from foundations and floor slabs - both during and after construction. Adjacent exterior finished grades should be sloped a minimum of four (4) percent for a distance of at least ten (10) feet away from the structure, as necessary to establish positive drainage and preclude ponding of water adjacent to foundations, or as required to comply with local, state or federal requirements, whichever is more stringent. Paved surfaces may be sloped at a minimum of 1 percent.

9.2.2 Surface water must not be allowed to pond adjacent to the building foundations, including during construction. It is recommended to provide rain gutters and direct all water from roof drains into closed conduits that are connected to an acceptable discharge area away from the building foundations, upon an impervious surface that will direct water away into a storm drain, or directly into the site storm drain system.

9.2.3 In general, it is not recommended to place landscape or planted areas with high irrigation demand directly adjacent to the building foundations, retaining walls, and/or interior slabs-on-grade. Plants with low irrigation demand should be used. Provisions to reduce excessive irrigation should also be incorporated into design and operation.

9.2.4 Trees should be setback from proposed structures at least 10 feet or a distance equal to the anticipated drip line radius of the mature tree. For example, if a tree has an anticipated drip-line diameter of 30 feet, the tree should be planted at least 15 feet away (radius) from proposed or existing buildings.

9.2.5 Landscaping after construction should direct rainfall and irrigation runoff away from the structure and should establish positive drainage of water away from the structure. Care should be taken to maintain a leak-free sprinkler system.

9.2.6 Landscape and planter areas should be irrigated using low flow irrigation (such as drip, bubblers or mist type emitters). The use of plants with minimal water requirements are recommended.

9.2.7 Perimeter curbs should be extended to the bottom of the aggregate base where irrigated landscape areas meet pavements. This should reduce moisture from irrigation and runoff from migrating into the aggregate base section and reducing the life of the pavements.

9.2.8 Assessment for stormwater infiltration was not included in the scope of our investigation. However, due to the presence of shallow groundwater, infiltration systems are not recommended at this site.

9.3 Deep Ground Modification for Liquefaction Mitigation

9.3.1 The following general recommendations were prepared for design and construction of deep ground modification to mitigate liquefaction and to provide direct support of the foundation and floor slab loads for the structure. On a preliminary basis, this report provides recommendations regarding the use of aggregate piers or rigid inclusions for mitigation of liquefaction. In the event other methods of deep ground modification are desired to be considered, Moore Twining should be notified and requested to review and comment on other methods. The site/surrounding subsurface environmental conditions may need to be considered with regard to selection of the type of deep ground modification. As an alternative to use of deep ground modification, the structure could be supported using a structural slab with a pier and gradebeam foundation system. Refer to Section 9.9 of this report.

9.3.2 Ground modification along with over-excavation and compaction of the subgrade soils will be required to reduce the estimated total and differential static settlement to a maximum of 1 inch total and ½ inch differential and to reduce the total seismic settlement to a maximum of ¾ inch and ½ inch differential in 30 feet, or other limits as specified by the project structural engineer based on the details of the proposed structure. The deep ground modification activity shall be conducted in accordance with this report, the plans, the specifications, and the City of Santa Cruz requirements, whichever is the most stringent. In the event a select fill material is required for load transfer for support of the slab on grade, the load transfer platform design requirements should be detailed by the design build contractor.

9.3.3 The sequencing of the work, bulking of soils resulting from the work, construction of a stable, working platform and any requirements of the site preparation necessary to provide a level work area and equipment access, etc. should be coordinated with the general and earthwork contractors and included in the scope of work.

9.3.4 In addition to liquefaction mitigation, the deep ground modification elements will be required to provide direct support of all new foundations, the interior slab on grade and all walkways adjacent to the building. The design shall achieve a minimum allowable bearing capacity of 3,000 pounds per square foot. The final extent of the treatment areas shall be determined by the design-build contractor in accordance with the recommendations of this report. It is recommended the area required for ground modification include the building and all foundation areas, sidewalks adjacent to the building, and any other area deemed to be required by the design professional for post-earthquake functionality, etc.

9.3.5 The deep ground modification program should include post ground modification testing to verify the soil densification and composite ground modification element/section stiffness that was assumed for design is actually achieved. The methods of verification of the post ground modification condition (i.e., load testing, cone penetration tests, soil borings, etc.) shall be developed by the design-build engineer, but shall include at least three cone penetration tests; or borings with nearly continuous standard penetration tests. The design-build contractor shall submit a final report signed and stamped by the design build engineer at the conclusion of the work including an analysis of the post ground modification geotechnical data and analysis to validate that the ground modification achieved the specified performance criteria. The final report shall include a conclusion that the deep ground modification work is suitable for support of the planned improvements and provide any supplemental recommendations that are deemed necessary by the design build engineer.

9.3.6 The Contractor shall provide a deep ground modification design submittal for review by the owner and design professionals. The submittal shall include an independent engineering analysis using the data provided in this report, and any additional data deemed necessary by the design build engineer, indicating that the proposed ground modification will achieve the required settlements. This assessment should be prepared by a professional engineer registered in the State of California. The submittal shall include the deep ground modification procedures, a plan showing the deep ground modification design layout in relation to the proposed improvements, the design/specifications for the load transfer platform, the procedures that will be used to verify the performance requirements of the deep ground modification design, and the quality control methods to be employed by the contractor.

9.3.7 The deep ground modification will need to extend to an appropriate elevation such that the deep ground modification elements will be suitable for direct support of the proposed foundations as well as to address the effects of liquefaction for the building foundations and slabs on grade.

9.3.8 Due to soil disturbance caused by the installation process, upon successful completion of the ground modification, the existing surface soils should be over-excavated in the treatment area as recommended in the Site Preparation section of this report to establish a compacted subgrade condition.

9.3.9 At the conclusion of the installation of deep ground modification work, the Contractor shall prepare an as-built plan showing the locations, diameters, depths, etc. for all the deep ground modification elements.

9.3.10 The Contractor will be responsible for quality control of the installation of the deep ground modification program. The project owner will also hire a consultant to perform sampling and testing of the materials for quality assurance purposes.

9.4 Site Demolition and Preparation

9.4.1 All surface topsoil, vegetation and organics identified should be removed from all work areas. The general depth of stripping should be sufficiently deep to remove the roots larger than ¼ inch in diameter or any accumulation of organic matter that will result in an organic content more than 3 percent by weight should be removed and not used as engineered fill. Stripping should be reviewed by Moore Twining at the time of construction and should extend laterally a minimum of 10 feet outside areas of planned excavation. Deeper stripping may be required in localized areas. These materials will not be suitable for use as engineered fill; however, stripped topsoil may be stockpiled and reused in landscape areas at the discretion of the owner.

9.4.2 As part of the site preparation, existing surface and subsurface improvements such as foundations, slabs on grade, pavements, curb and gutter, stormwater collection structures, hardscaping, underground utilities, buried structures, and associated backfill will need to be removed. During removal of underground utilities and subsurface structures, the backfill soils should also be removed. The utility pipe or conduits to be abandoned should be completely removed and disposed of off-site. Care should be taken to over-excavate all soils which are disturbed from the demolition activities prior to backfilling the excavations with engineered fill. All excavations should be backfilled with engineered fill in accordance with the recommendations of this report, under the observation and testing of Moore Twining. In addition, all undocumented fill soils should be removed from the areas of the proposed improvements. The resulting excavations should be cleaned of all backfill and organic material, the exposed undisturbed native soils should then be scarified to a depth of 8 inches, moisture conditioned, then compacted as engineered fill, and the excavation backfilled with engineered fill.

9.4.3 For areas to receive fill soils outside the building pad preparation limits, after site stripping, removal of root systems and removal of existing surface and subsurface improvements, the areas should be over-excavated to a minimum of 6 inches below preconstruction site grade, to a minimum of 6 inches below the improvements to be removed, and to the depth required to remove existing undocumented fills (if any), whichever is greater. The exposed bottom of the excavation should be scarified to a minimum of 8 inches, moisture conditioned to between 1 and 3 percent above optimum moisture content and compacted as engineered fill to a stable condition.

9.4.4 Following stripping and removal of existing surface and subsurface improvements (if any), the building pad area should be over-excavated to a depth of 1.5 feet below site grade, to at least 1 foot below subsurface structures to be removed, to the depth required to remove all soils which are disturbed from demolition work, to the depth required to excavate all undocumented fill, and as required for the deep ground modification design, whichever is greater. As a minimum, the horizontal limit of the excavation for preparation of the building should include the entire footprint of the building, all foundations, adjacent walkways, to a minimum of 5 feet horizontally beyond the foundations, and to the horizontal extent of the deep ground modification limits shown on the plans, whichever is greater. Upon approval of the excavation by Moore Twining based on visual observations and the survey data provided by the contractor, the bottom of the excavation should be aerated, scarified to a depth of 8 inches, moisture conditioned to between optimum and three (3) percent above optimum moisture content, and compacted to a minimum of 92 percent relative compaction prior to placement of fill soils. Where instability occurs at the bottom of the excavation, the bottom of the excavation should be stabilized in accordance with the recommendations of this report. The above recommendations are intended for subgrade preparation where deep ground modification is used for support. In the event the proposed structure is supported on a pier and gradebeam foundation system, the undocumented fills would not require excavation.

Due to the soil disturbance caused by the deep ground modification installation process, after the deep ground modification work, the subgrade soils should be re-prepared by compacting the soils disturbed from the deep ground modification work. The recompaction work and final documentation/verification of the compaction of the building pad area should be verified by Moore Twining by observation and testing.

9.4.5 It is recommended that extra care be taken by the contractor to ensure that the horizontal and vertical extent of the over-excavation and compaction conform to the site preparation recommendations presented in this report. Moore Twining is not responsible for measuring and verifying the horizontal and vertical extent of over-excavation and compaction. The contractor should verify in writing to the owner and Moore Twining that the horizontal and vertical over-excavation limits were completed in conformance with the recommendations of this report, the project plans, and the specifications (the most stringent applies). It is recommended that this verification be performed by a licensed surveyor. This verification should be provided prior to requesting pad certification from Moore Twining or excavating for foundations.

9.4.6 Miscellaneous lightly loaded foundations (such as retaining walls, sound walls, trash enclosures, screen walls, monument signs, etc.) should be evaluated on a case by case basis to develop supplemental recommendations for site preparation and foundation design. In lieu of a case by case evaluation, following stripping and removal of existing surface and subsurface improvements, miscellaneous foundations should be over-excavated to a minimum of 6 inches below the bottom of the foundations, to a minimum depth to remove undocumented fill, and to at least 12 inches below the existing improvement to be removed, whichever is greater. The zone of over-excavation should extend at least 5 feet beyond the edge of foundations on all sides. Upon approval of the over-excavation limits by Moore Twining, the soils at the bottom of the excavation should be scarified to a minimum depth of 12 inches, moisture conditioned to slightly above optimum moisture content and compacted as engineered fill to a minimum of 92 percent of the maximum dry density determined in accordance with ASTM D1557. Areas of instability should be stabilized in accordance with the recommendations of this report.

9.4.7 After site stripping, removal of root systems and removal of existing surface and subsurface improvements, the areas to receive exterior concrete slabs on grade outside the building pad preparation limits and pavements should be over-excavated to a minimum of 12 inches below preconstruction site grade, to a minimum of 12 inches below the proposed subgrade, to a minimum of 6 inches below improvements to be removed, and to the depth required to remove undocumented fills (if any), whichever is greater. The exposed bottom of the excavation should be scarified to a minimum of 12 inches, moisture conditioned to between 1 and 4 percent above optimum moisture content and compacted as engineered fill to a stable condition. The zone of over-excavation, scarification and compaction (overbuild zone) should extend laterally a minimum of 3 feet outside the perimeters of pavements/slabs located outside the building pad preparation limits, or by a horizontal distance equal to the depth of fill, whichever is greater. Areas of instability should be stabilized in accordance with the recommendations of this report.

9.4.8 All fill required to bring the site to final grades should be placed as engineered fill. In addition, all native soils over-excavated should be compacted as engineered fill.

9.4.9 Any wells encountered or scheduled for demolition should be abandoned per state and local requirements. The contractor should obtain an abandonment permit from the local environmental health department, and issue certificates of destruction to the owner and Moore Twining upon completion. At a minimum, wells in building areas (and within 5 feet of building perimeters) should have their casings removed to a depth of at least 8 feet below preconstruction site grades or finished pad grades, whichever is deeper. In parking lot or landscape areas, the casings should be removed to a depth of at least 5 feet below site grades or finished grades. The wells should be capped with concrete and the resulting excavations should be backfilled as engineered fill.

9.4.10 The moisture content and density of the compacted soils should be maintained until the placement of concrete. If soft or unstable soils are encountered during excavation or compaction operations, our firm should be notified so the soils conditions can be examined and additional recommendations provided to address the pliant areas.

9.4.11 Final grading shall produce a subgrade ready to receive a slab-on-grade which is smooth, planar, and resistant to rutting. The finished pad (before aggregate base is placed) shall not depress more than one-half (½) inch under the wheels of a fully loaded water truck, or equivalent loading. If depressions more than one-half (½) inch occur, the contractor shall perform remedial grading to achieve this requirement at no cost to the owner.

9.4.12 The Contractor should be responsible for the disposal of concrete, asphalt concrete, soil, spoils, etc. that must be exported from the site. Individuals, facilities, agencies, etc. may require analytical testing and other assessments of these materials to determine if these materials are acceptable for the intended use by the receiving party. The Contractor is responsible to perform the tests, assessments, etc. necessary to determine the appropriate method of disposal. In addition, the Contractor is responsible for all costs to dispose of these materials in a legal manner.

9.5 Engineered Fill

9.5.1 The near surface soils encountered included sandy lean clays, clayey sand with varying amounts of gravel, silty sands, and silty clayey sands, with a very low expansion potential. The on-site near surface soils that have an expansion index less than 30, are free of debris and organics (less than 3 percent by dry weight) and do not contain particles larger than 3 inches in the greatest dimension are suitable for use as engineered fill, provided the soils are properly aerated or moisture conditioned to achieve the moisture contents recommended in this report for use as engineered fill.

If clayey soils with an expansion index of greater than 30 are encountered during grading, these soils should be placed below a depth of 3 feet below the lowest adjacent site grades for the building (or used in non-structural landscape areas of the site). The interior slabs on grade, adjacent walkways, exterior slabs on grade that are outside the building pad preparation limits and Portland cement concrete pavements should be underlain by a minimum of 6 inches of non-recycled Class 2 aggregate base, or greater as required by the deep ground modification design. If soils other than those considered in this report are encountered, Moore Twining should be notified to provide alternate recommendations.

9.5.2 It should be noted that high moisture contents should be anticipated for the near surface soils. Thus, drying of the soil by methods such as chemical treatment should be anticipated where wet soils occur.

9.5.3 The compactibility of the native soils is dependent upon the moisture contents, subgrade conditions, degree of mixing, type of equipment, as well as other factors. The evaluation of such factors was beyond the scope of this report; therefore, we recommend that they be evaluated by the contractor during preparation of bids and construction of the project.

9.5.4 Imported, non-expansive fill soil should be non-recycled and granular in nature with the following acceptance criteria recommended.

Percent Passing 3-Inch Sieve	100
Percent Passing No. 4 Sieve	75 - 100
Percent Passing No. 200 Sieve	15 - 40
Expansion Index (ASTM D4829)	Less than 15
R-value	Greater than 30*
Organics	Less than 3 percent by weight
Sulfates	< 0.05 percent by weight
Min. Resistivity	> 5,000 ohms-cm

* For Pavement Areas Only

9.5.5 Prior to importing fill, the Contractor shall submit test data that demonstrates that the proposed import soils comply with the recommended criteria for both geotechnical and environmental compliance. Also, prior to being transported to the site, the import material shall be certified by the Contractor and the supplier (to the satisfaction of the Inspector of Record) that the soils do not contain any environmental contaminates regulated by local, state or federal agencies having jurisdiction. This certification shall consist of, as a minimum, analytical data specific to the source of the import material in accordance with the Department of Toxic Substances Control, “Informational Advisory, Clean Imported Fill Material,” dated October 2001. The list of constituents to be tested for the fill source shall be submitted to the Owner for review and approval prior to the Contractor testing the fill. In lieu of sampling and testing aggregate base materials (or bedding sand) from virgin sand and gravel sources, a letter stating that the aggregate base (or bedding sand) comprises materials entirely from natural (virgin) sources and that the aggregate base (or bedding sand) is non-contaminated may be provided by the Contractor. After approval of the Contractor's submittal and prior to being transported to the site, the import fill material shall also be tested and approved by Moore Twining for the above geotechnical characteristics. The Contractor shall allow a minimum of seven (7) working days for each import source to be tested by Moore Twining for compliance with the geotechnical characteristics specified herein.

9.5.6 Recycled asphalt concrete materials should not be used as fill below the building.

9.5.7 Native onsite soils used as engineered fill should be placed in loose lifts approximately 8 inches thick or less, dried or moisture conditioned to between 1 and 3 percent above optimum moisture content, and compacted to a minimum of 90 percent relative compaction (ASTM D1557), with exception that the upper 12 inches of subgrade should be compacted to a minimum of 95 percent relative compaction. Additional lifts should not be placed if the previous lift did not meet the required dry density or if soil conditions are not stable.

9.5.8 Imported, non-expansive fill and any on-site soils that are non-expansive should be placed in loose lifts approximately 8 inches thick or less, moisture-conditioned to between optimum and three (3) percent above optimum moisture content, and compacted to a minimum of 92 percent relative compaction (ASTM D1557), with exception that the upper 12 inches of subgrade should be compacted to a minimum of 95 percent relative compaction. Additional lifts should not be placed if the previous lift did not meet the required dry density or if soil conditions are not stable.

9.5.9 Utility trench backfill should be moisture conditioned and compacted in accordance with the recommendations for engineered fill as recommended in this report. The contractor should use appropriate equipment and methods to avoid damage to utilities and/or structures during placement and compaction of the backfill materials. Lift thickness can be increased if the contractor can demonstrate the minimum compaction requirements can be achieved.

9.5.10 Aggregate base shall comply with State of California Department of Transportation requirements for Caltrans Class 2 aggregate base, with exception that aggregate base used below the building slab should be non-recycled. Aggregate base used below exterior slab and pavement areas shall comply with State of California Department of Transportation requirements for Caltrans Class 2 aggregate base but may include recycled materials. Documentation that the aggregate base to be used for the project meets the Class 2 aggregate base requirements (R-value, gradation, sand equivalent, durability, etc.) should be provided to the Owner. All aggregate base should be compacted to a minimum of 95 percent relative compaction.

9.5.11 Open graded gravel and rock material such as ¾-inch crushed rock or ½-inch crushed rock should not be used as backfill including trench backfill. In the event gravel or rock is required by a regulatory agency for use as backfill, all open graded materials shall be fully encased in a geotextile filter fabric, such as Mirafi 140N, to prevent migration of fine grained soils into the porous material. Gravel and rock cannot be used without the written approval of Moore Twining. If the contractor elects to use crushed rock (and if approved by Moore Twining), the contractor will be responsible for slurry cut off walls at the locations directed by Moore Twining. Materials such as crushed rock should be placed in thin (less than 8 inches) lifts and each lift should be compacted with a minimum of three (3) passes with a vibratory compactor.

9.5.12 In-place density testing should be conducted in accordance with ASTM D 6938 (nuclear methods) at a frequency of at least:

Area	Minimum Test Frequency
Mass Fills or Building Pad Subgrade	1 test per 2,500 square feet per compacted lift
Pavements	1 test per 5,000 square feet per compacted lift
Sidewalks	1 test per 100 lineal feet per compacted lift
Utility Lines	1 test per 150 feet per compacted lift

9.6 Shallow Spread Foundations Supported on Deep Ground Modification

In order to mitigate the liquefaction potential described in this report, this report provides alternative recommendations for supporting the foundations and floor slab loads on either: 1) shallow foundations combined with deep ground modification, or 2) drilled pier foundations. Recommendations for deep ground modification are presented in section 9.3 of this report. This section includes recommendations for shallow foundations supported directly on deep ground modification elements. Section 9.9 of this report includes recommendations for supporting the foundations and floor slab loads on pile foundations.

9.6.1 This report recommends use of a specialty deep ground modification program to address concerns with liquefaction and to provide direct support of the foundations and floor slab loads. The foundation design should be coordinated with the design of the ground modification system.

9.6.2 After it has been determined that the installation of the deep ground modification has been completed in accordance with the specified performance criteria of this report and the building pad has been prepared in accordance with the site preparation recommendations of this report, structural loads may be supported directly on the deep ground modification elements. On a preliminary basis, spread and continuous footings supported directly on the deep ground modification elements, may be designed for a maximum net allowable soil bearing pressure of 3,000 pounds per square foot for dead-plus-live loads and the foundation and floor slab elements should be designed with sufficient stiffness to transfer all loads to the ground modification elements. It is expected that higher allowable bearing pressures would be obtainable when the deep ground modification design is conducted, subject to determination of the ground modification design engineer. The details of the foundation design will need to be coordinated with the design of the deep ground modification. This value may be increased by one-third for short duration wind or seismic loads. Final determination of the allowable settlement and bearing capacity should be determined by the design build ground modification engineer.

9.6.3 A structural engineer experienced in foundation design should recommend the thickness, design details and concrete specifications for the foundations and slabs on grade based on: 1) a total static settlement of 1 inch, 2) a differential static settlement of ½ inch in 30 linear feet, 3) a total seismic settlement of ¾ inch, and 4) a differential seismic settlement of ½ inch in 30 lineal feet. The deep ground modification activity shall be conducted in accordance with the recommendations of this report (refer to Section 9.3), the plans, the specifications, and the City of Santa Cruz requirements, whichever is the most stringent. If different settlement limits are required by the structural engineer, the tolerable limits for the structure/foundations should be specified by the structural engineer.

9.6.4 Perimeter foundations should have a minimum depth of 18 inches below the bottom of the slab-on-grade and 18 inches below the lowest finished adjacent grade. Interior foundations should have a minimum depth of 18 inches below the bottom of the slab-on-grade. All footings should have a minimum width of 18 inches, regardless of load.

9.6.5 The foundations should be continuous around the perimeter of the structure to reduce moisture migration beneath the structure. Continuous perimeter foundations should be extended through doorways and/or openings that are not needed for support of loads.

9.6.6 Structural loads for miscellaneous lightly loaded (less than 1.5 kips per foot) foundations (such as retaining walls, sound walls, screen walls, monument and pylon signs, etc.) should be supported on engineered fill soils prepared in accordance with the recommendations in the "Site Preparation" section of this report. Spread and continuous footings for lightly loaded structures with a minimum depth of 18 inches below finished grade may be designed for a maximum net allowable soil bearing pressure of 1,500 pounds per square foot for dead-plus-live loads. This value may be increased by one-third for short duration wind or seismic loads. It should be noted that miscellaneous lightly loaded structures would be subject to the effects of liquefaction.

9.6.7 The following seismic factors were developed for the site using the Ground Motion Parameter Calculator available from SEAOC and OSHPD (http://seismicmaps.org) in accordance with the latest CBC, using a site latitude of 36.954407 degrees, and a longitude of -122.049663 degrees and a Site Class D (Stiff Soil). The data provided in the following Table No. 3 are based upon the procedures of ASCE 7-16 and were not determined based upon a ground motion hazard analysis. A Site Class D was selected for the project since liquefaction will be mitigated either by deep ground modification or use of deep foundations. In addition, the building period is anticipated to be low. The structural engineer should review the values in the following Table and determine whether a ground motion hazard analysis is required for the project considering the seismic design category, structural details, and requirements of ASCE 7-16 (Section 11.4.8 and other applicable sections). If required, Moore Twining should be notified and requested to conduct the additional analysis, develop updated seismic factors for the project, and update the following values.

TABLE NO. 3

Seismic Factor	2022 CBC Value
Site Class	D
Maximum Considered Earthquake (geometric mean) peak ground acceleration adjusted for site effects (PGA_M)	0.741
Mapped Maximum Considered Earthquake (geometric mean) peak ground acceleration ASCE 7-16 (PGA)	0.674
Spectral Response At Short Period (0.2 Second), Ss	1.598
Spectral Response At 1-Second Period, S_1	0.605
Site Coefficient (based on Spectral Response At Short Period), Fa	1.0
Site Coefficient (based on spectral response at 1-second period) Fv	See Note
Maximum considered earthquake spectral response acceleration for short period, S_{MS}	1.598
Maximum considered earthquake spectral response acceleration at 1 second, S_{M1}	See Note

TABLE NO. 3

Seismic Factor	2022 CBC Value
Five percent damped design spectral response accelerations for short period, S_{DS}	1.065
Five percent damped design spectral response accelerations at 1-second period, S_{D1}	0.686
$T_S = S_{D1} / S_{DS}$	0.644

Note: Requires ground motion hazard analysis per ASCE Section 21.2 (ASCE 7-16, Section 11.4.8), unless an Exception of Section 11.4.8 of ASCE 7-16 is applicable for the building design.
Ts and S_{D1} calculated using Long-Period Site Coefficient, Fv, from Table 11.4-2 of ASCE 7-16 Supplement 3.

9.6.8 Foundation excavations should be observed by Moore Twining prior to the placement of steel reinforcement and concrete to verify conformance with the intent of the recommendations of this report. In addition, the design-build ground modification engineer should confirm the condition of the prepared subgrade (inclusive of cushion material, load transfer platforms, etc.) satisfies the requirements of their design and specifications. The Contractor is responsible for proper notification to Moore Twining and receipt of written confirmation of these observations prior to placement of steel reinforcement.

9.6.9 Foundation excavations or exposed soils should not be left uncovered and allowed to dry such that the moisture content of the soils is less what is recommended in this report or drying produces cracks in the soils. The exposed soils, such as sidewalls, excavation bottoms, etc. should be continuously moistened to maintain the moisture content at least one percent above optimum until concrete is placed.

9.7 Frictional Coefficient and Earth Pressures

9.7.1 The bottom surface area of concrete footings or concrete slabs in direct contact with the subgrade soils prepared as recommended in this report can be used to resist lateral loads. An allowable coefficient of friction of 0.30 can be used for design. In areas where slabs are underlain by a synthetic moisture vapor membrane, an allowable coefficient of friction of 0.10 can be used for design.

9.7.2 The allowable passive resistance of the native soils and engineered fill may be assumed to be equal to the pressure developed by a fluid with a density of 275 pounds per cubic foot. The upper 6 inches of subgrade in landscape areas should be neglected in determining the total passive resistance.

9.7.3 Due to the fines content, the near surface soils are not recommended for use as backfill of retaining walls. Backfill of retaining walls within a zone defined by a 1 Horizontal to 1 Vertical plane from the back of the wall foundation to the ground surface should consist of imported, non-expansive granular fill meeting the recommendations included in Section 9.8.2 of this report. This requirement should be depicted on the project plans. The active and at-rest pressures of the imported, non-expansive granular engineered fill may be assumed to be equal to the pressures developed by fluid with a density of 45 and 65 pounds per cubic foot, respectively. These pressures assume a level ground surface, drained conditions and do not include the surcharge effects of construction equipment, loads imposed by nearby foundations and roadways and hydrostatic water pressure.

9.7.4 The wall designer should determine if seismic increments are required for retaining wall design. If seismic increments are required for retaining wall design, Moore Twining should be contacted for recommendations for seismic geotechnical design considerations for the retaining structures.

9.7.5 The at-rest pressure should be used in determining lateral earth pressures against walls which are not free to deflect. For walls which are free to deflect at least one percent of the wall height at the top, the active earth pressure may be used.

9.7.6 The above earth pressures assume that the backfill soils will be drained. Therefore, all retaining walls should incorporate the use of a backdrain as recommended in this report.

9.8 Retaining Walls

9.8.1 Retaining wall plans, when available, should be reviewed by Moore Twining to evaluate the actual backfill materials, proposed construction, drainage conditions, and other design geotechnical parameters.

9.8.2 Imported, granular, fill used for backfill of retaining walls from the bottom of the wall footing at a 1 horizontal to 1 vertical gradient to the surface should meet the following requirements:

Percent Passing 3-Inch Sieve	100
Percent Passing No. 4 Sieve	75 - 100
Percent Passing No. 200 Sieve	0 - 20
Plasticity Index	Less than 5

9.8.3 The import fill material should be tested for conformance with the recommended material characteristics.

9.8.4 Segmented wall design (if any) should be conducted by a California licensed geotechnical engineer familiar with segmented wall design and having successfully designed at least three walls at sites with similar soil conditions. None of the data included in this report should be used for segmented wall design. A design level geotechnical report should be conducted to provide wall design parameters. If the designer uses the data in this report for wall design, the designer assumes the sole risk for this data. The wall designer should perform sufficient observations of the wall construction to certify that the wall was constructed in accordance with the design plans and specifications.

9.8.5 As a minimum, retaining walls should be constructed with a drainage system including a perforated pipe surrounded by at least 1 cubic foot of 3/4 inch crushed rock or Class 2 permeable material per lineal foot of drain pipe. In addition, retaining walls associated with the subsurface portion of the structure should include a full height geocomposite drainage board, or a minimum 12 inch wide section of drain rock encapsulated in a geotextile fabric. If open graded materials such as crushed rock are used as drain material surrounding drain pipes, these materials should be fully encased in filter fabric such as Mirafi 140 N and vibrated in place to a non-yielding condition under the observation of the geotechnical engineer. A Caltrans Class 2 permeable material, installed without the use of filter fabric, is preferable to open graded material as it presents a lower potential for clogging than the filter fabric. Class 2 permeable material should be compacted to 95 percent relative compaction in accordance with ASTM D1557. Drain pipes should be located near the wall to adequately reduce the potential for hydrostatic pressures behind the wall. Drainage should be directed to pipes which gravity drain to closed pipes of the storm drain or subdrain system. Drain pipe outlet invert elevations should be sufficient (a bypass should be constructed if necessary) to preclude hydrostatic surcharge to the wall in the event the storm drain system did not function properly. Clean out and inspection points should be incorporated into the drain system. Drainage should be directed to the site storm drain system. The drainage system should be designed by the wall designer and detailed on the plans. In landscape or uncovered areas, the top of the drainage section should be covered with a filter fabric and a minimum of 12 inches of onsite soils compacted as engineered fill.

9.8.6 It is recommended to use lighter hand operated or walk behind compaction equipment in the zone equal to one wall height behind the wall to reduce the potential for damage to the wall during construction. Heavier compaction equipment could cause loads in excess of design loads which could result in cracking, excessive rotation, or failure of a retaining structure. The contractor is responsible for damage to the wall caused by improper compaction methods behind the wall.

9.8.7 If retaining walls are to be finished with dry wall, plaster, decorative stone, etc., or if effervescence is undesirable, waterproofing measures should be applied to walls. Waterproofing systems should be designed by a qualified professional.

9.8.8 Retaining walls may be subject to lateral loading from pressures exerted from the soils, groundwater, foundations, and vehicular traffic loads, adjacent to the walls. In addition to earth pressures, lateral loads due to slabs-on-grade, footings, or traffic above the base of the walls should be included in design of the walls. The designer should take into consideration the allowable settlements for the improvements to be supported by the retaining wall.

9.9 Pile Foundations

As an alternative to supporting the foundations and floor slab loads on shallow foundations over deep ground modification to mitigate the potential effects of liquefaction, general recommendations are provided below for use of pile foundations to support the structure and floor slab loads. **Design of the deep foundations will need to include analysis of the seismic design loading and will need to consider the additional down drag loading/reduced load capacity associated with the potential for liquefaction.**

Based on the subsurface conditions and difficulties with deep foundation installation at this site, foundation support should utilize drilled shafts with temporary casing such as auger cast piles that utilize casing systems as part of the drill tooling. If these systems are utilized, it is recommended a design build foundation contractor develop estimates of pile capacity and minimum embedment into the underlying formational material based upon a load testing program. In addition, specifications for pile integrity testing should be incorporated into the test program to validate the contractors installation methods. For the purpose of this report, geotechnical parameters for use in final design should be determined by the design build pile contractor using the data contained in this report and additional studies, as required by the design build contractor. A detailed submittal should be prepared and submitted to the owner and design team for review and consideration.

9.9.1 A civil or structural engineer registered in the state of California should design the foundation system to resist shear, moment, and axial (tension and compression) loads based on the recommendations in this report and the structural loading requirements for the piles.

9.9.2 It is assumed the design engineer will prepare a specification for the construction of the deep foundations as part of the construction documents. The specifications should be consistent with the recommendations included in this report, ACI 336.1, the project plans, and the requirements of the City of Santa Cruz.

9.9.3 Due to the variation in the depth of the undocumented fill and sedimentary rock material, variable foundation depths will be required throughout the site that will need to be adjusted depending on the depth of rock encountered. The variation in depth to bearing materials at the site will result in difficulty in establishing accurate probable tip elevations. Therefore, pile lengths will need to be adjusted in the field depending on the actual depth to rock materials encountered. Accordingly, structural design and detailing for the deep foundation reinforcement should be conducted to allow the most ease of field adjustment of reinforcing steel length, by both cutting or splicing of reinforcing steel cages.

9.9.4 A structural engineer experienced in foundation design should recommend the thickness, design details and concrete specifications for the foundations based on a total static settlement of 1 inch, a differential static settlement of ½ inch between foundations, and a differential settlement of ½ inch over a distance of 30 feet.

9.9.5 Piles should be placed no closer than three pile diameters, center-to-center. For alternate spacing, the capacity of piles in groups should be reduced using appropriate group reduction formulas.

9.9.6 The deep foundation design should consider the variability in rock elevations and the foundation plans should identify the minimum foundation embedment into rock as required to achieve the design loads for the foundations. These values may also be considered for initial capacity estimates for auger cast piles.

9.9.7 Downdrag loading/reduced pile capacity needs to be considered within and above the liquefiable soil zones were ignored for the seismic condition. A detailed analysis should be provided as part of the design build contractor's submittal.

9.9.8 P-multipliers should be applied for lateral analysis of pile groups for lateral load directions parallel to a pile row in accordance with Section 10 of the California Amendments to the AASHTO LRFD Bridge Design Specifications, prepared by the State of California Department of Transportation.

9.9.9 Based on the California Amendments to the AASHTO LRFD Bridge Design Specifications, if the lateral loading direction for a single row of piles is perpendicular to the row of piles, P-multipliers of 0.80, 0.90 and 1.0 shall be used for pile spacing of 2.5D, 3D and 4D, respectively, where "D" is the pile diameter.

9.9.10 The type and strength of the concrete used for construction of the pile foundations should be specified by the design engineer. Where foundation concrete is placed below water, the compressive strength requirements of the concrete mix design should be increased by 1,000 pounds per square inch above the minimum design requirement.

9.9.11 The slump requirements specified for the foundation concrete and the reinforcing steel spacing for foundations should be designed to ensure concrete will flow.

9.10 Interior Floor Slab Construction and Vapor Retarding Membrane

9.10.1 All slabs on grade located within the building pad preparation limits (i.e. interior floor slabs and slabs on grade adjacent to the building) should be underlain by a minimum of 6 inches of aggregate base over subgrade soils (and deep ground modification elements, if applicable) prepared as recommended in this report.

9.10.2 The recommendations provided herein are intended only for the design of interior concrete slabs-on-grade and their proposed uses, which do not include construction equipment. The building contractor should assess the slab section and determine its adequacy to support any proposed construction traffic.

9.10.3 The slabs and underlying subgrade should be constructed in accordance with current American Concrete Institute (ACI) standards.

9.10.4 A vapor retarder should be placed below interior building slabs where moisture could permeate into the interior and create problems. Refer to the American Concrete Institute's Guide to Concrete Floor and Slab Construction (ACI 302.1R) for selection and installation of moisture vapor retarders. It is recommended that a Stegowrap 15 vapor retarder be used where moisture could permeate into the interior and create problems, such as where flooring or floor slab applications will contain moisture sensitive materials (or other slab applications or uses). The vapor retarder should overlay the compacted 6 inch layer of aggregate base. It should be noted that placing the PCC slab directly on the vapor retarder may increase the potential for cracking and curling; however, ACI recommends the placement of the vapor retarding membrane directly below the slab unless a watertight roofing system is in place prior to slab construction to reduce the amount vapor emission through the slab-on-grade. It is recommended that the slab be moist cured for a minimum of 7 days to reduce the potential for excessive cracking.

The underslab membrane should have a high puncture resistance (minimum of approximately 2,400 grams of puncture resistance), high abrasion resistance, rot resistant, and mildew resistant. It is recommended that the membrane be selected in accordance with the current ASTM C 755, Standard Practice For Selection of Vapor Retarder For Thermal Insulation and conform to the current ASTM E 1745 Plastic Water Vapor Retarders Used in Contact with Soil or Granular Fill under Concrete Slabs and ASTM E 154 Standard Test Methods for Water Vapor Retarders Used in Contact with Earth Under Concrete Slabs, on Waters, or as Ground Cover. It is recommended that the vapor barrier installation conform to the current ACI Manual of Concrete Practice, Guide for Concrete Floor and Slab Construction (302.1R), Addendum, Vapor Retarder Location and current ASTM E 1643, Standard Practice for Installation of Water Vapor Retarders Used In Contact with Earth or Granular Fill Under Concrete Slabs. In addition, it is recommended that the manufacturer of floor covering, floor covering adhesive or other slab material applications be consulted to determine if the manufacturers have additional recommendations regarding the design and construction of the slab-on-grade, testing of the slab-on-grade, slab preparation, application of the adhesive, installation of the floor covering and maintenance requirements. It should be noted that the recommendations presented in this report are not intended to achieve a specific vapor emission rate.

9.10.5 The membrane should be installed so that there are no holes or uncovered areas. All seams should be overlapped and sealed with manufacturer approved tape continuous at the laps so they are vapor tight. All perimeter edges of the membrane, such as pipe penetrations, interior and exterior footings, joints, etc.) should be caulked per manufacturer's recommendations.

9.10.6 Tears or punctures that may occur in the membrane should be repaired prior to placement of concrete per manufacturer's recommendations. Once repaired, the membrane should be inspected by the contractor and the owner to verify adequate compliance with manufacturer's recommendations.

9.10.7 The manufacturer's requirements vary regarding the surface and cover material around the placed membrane. Vapor retarding membranes should be installed in accordance with the manufacturers' specifications.

9.10.8 Additional measures to reduce moisture migration should be implemented for floors that will receive moisture sensitive coverings. These include: 1) constructing a less pervious concrete floor slab by maintaining a water-cement ratio of 0.52 or less in the concrete for slabs-on-grade, 2) ensuring that all seams and utility protrusions are sealed with tape to create a "water tight" moisture barrier, 3) placing concrete walkways or pavements adjacent to the structure, 4) providing adequate drainage away from the structure, 5) moist cure the slabs for at least 7 days, and 6) locating lawns, irrigated landscape areas, and flower beds away from the structure.

9.10.9 The Contractor shall test the moisture vapor transmission through the slab, the pH, internal relative humidity of the floor slab, etc., at a frequency and method as specified by the flooring manufacturer, adhesive manufacturer, underlayment manufacturer, etc. or as required by the plans and specifications, whichever is most stringent. The tests should be conducted in accordance with the applicable ASTM test methods. The results of vapor transmission tests, pH tests, internal relative humidity tests of the floor slab, ambient building conditions, etc. should be within floor manufacturer's, adhesive manufacturer's and underlayment manufacturer's specifications at the time the floor is placed. It is recommended that the floor, adhesive and underlayment manufacturers and subcontractor review and approve the test data prior to floor covering installation.

9.10.10 It should be noted that the placement and compaction of the Class 2 AB, the vapor retarding membrane installation, protection, etc., and the placement, curing, etc. of concrete should be in accordance with the project geotechnical engineering report, applicable ACI requirements, and the manufacturer's requirements.

9.10.11 If the subgrade is prepared and then disturbed by equipment workers, weather or other source, we recommend that the exposed subgrade to receive slabs be tested to verify adequate compaction. If adequate compaction is not verified, the disturbed subgrade should be over-excavated, scarified, and compacted as engineered fill. This condition should be verified prior to installation of aggregate base and prior to construction of the slabs-on-grade.

9.11 Exterior Slabs-On-Grade

The recommendations for exterior slabs provided below are not intended for use for slabs subjected to vehicular traffic, rather lightly loaded sidewalks, curbs, and planters, etc.

9.11.1 Exterior improvements that subject the subgrade soils to a sustained load greater than 150 pounds per square foot should be prepared in accordance with recommendations presented in this report for interior slabs-on-grade. Moore Twining can provide alternative design recommendations for exterior slabs, if requested. The following recommendations consider an allowable settlement/heave of 1 inch total and ½ inch differential.

9.11.2 Subgrade soils for exterior slabs should be prepared as recommended in the "Site Demolition and Preparation" section of this report. Upon completion of the over-excavation and compaction of the subgrade soils, exterior slabs should be supported on a minimum of 6 inches of Class 2 aggregate base. Engineered fill soils should be placed below all exterior slabs on grade in accordance with the recommendations provided in the "Site Preparation" section of this report.

9.11.3 The subgrade soils should be not be allowed to dry prior to placement of the non-expansive fill. The moisture content of the prepared subgrade soils below the imported, non-expansive fill should be verified to be between 1 and 4 percent above optimum moisture content to a minimum depth of 12 inches within 48 hours of placement of the non-expansive fill below slab-on-grade and curbs and gutters. If necessary to achieve the recommended moisture content, the subgrade should be over-excavated, moisture conditioned as necessary and compacted as engineered fill.

9.11.4 The exterior slabs-on-grade adjacent to landscape areas should be designed with thickened edges which extend at least the bottom of the aggregate base below exterior slabs.

9.11.5 Since exterior sidewalks, curbs, etc. are typically constructed at the end of the construction process, the moisture conditioning conducted during earthwork can revert to natural dry conditions. Placing aggregate base materials and/or concrete walks and finish work over dry or slightly moist subgrade should be avoided. It is recommended that the general contractor notify Moore Twining to conduct in-place moisture and density tests prior to placing aggregate base and concrete flatwork. Written test results indicating passing density and moisture tests should be in the general contractor's possession prior to placing concrete for exterior flatwork.

9.12 Asphaltic Concrete (AC) Pavements

9.12.1 Asphalt concrete pavement should be supported on subgrade soils prepared in accordance with the recommendations provided under the subsection entitled, "Site Preparation." The upper 12 inches of subgrade below the pavement section should be compacted to a minimum of 95 percent of the maximum dry density as determined by ASTM D1557. The moisture content of the subgrade should be verified prior to placement of the aggregate base.

9.12.2 The pavement sections in Table No. 4 are based on an R-value of 30 for the new asphalt pavements. The analysis was based on traffic indices ranging from 5.0 to 8.0 at half point increments. These indices are provided as a general guide. The project civil engineer should determine the appropriate traffic index. Moore Twining should be contacted if additional pavement section designs are needed. It should be noted that if pavements are constructed prior to the construction of the proposed buildings, higher traffic index values should be used in design.

Table No. 4
Two-Layer Asphaltic Concrete Pavement Sections

Traffic Index	AC Thickness, inches	AB Thickness, inches	Compacted Subgrade (inches)
5.0	2.5	6.5	12
5.5	3.0	7.0	12
6.0	3.0	8.5	12
6.5	3.5	9.0	12
7.0	4.0	9.5	12
7.5	4.0	11.0	12
8.0	4.5	11.5	12

AC - Asphalt Concrete compacted as recommended under section 9.12.9 of this report
AB - Class 2 Aggregate Base with a minimum R-value of 78 compacted to at least 95 percent relative compaction (ASTM D1557)
Subgrade - Subgrade soils compacted to at least 95 percent relative compaction (ASTM D-1557)

9.12.3 The curbs where pavements meet irrigated landscape areas or uncovered open areas should be extended at least to the bottom of the aggregate base section. This should reduce subgrade moisture from irrigation and runoff from migrating into the base section and reducing the life of the pavements.

9.12.4 If actual pavement subgrade materials are significantly different from those tested for this study due to unanticipated grading or soil importing, the pavement sections should be re-evaluated for the changed subgrade conditions.

9.12.5 If the paved areas are to be used during construction, or if the type and frequency of traffic are greater than assumed in design, the pavement sections should be re-evaluated for the anticipated traffic.

9.12.6 Pavement section design assumes that proper maintenance, such as sealing and repair of localized distress, will be performed on an as needed basis for longevity and safety.

9.12.7 Pavement materials and construction method should conform to the current California Standard Specification Requirements.

9.12.8 It is recommended that the base course of asphaltic concrete consist of a ¾ inch maximum medium gradation. The top course or wear course should consist of a ½ inch maximum medium gradation.

9.12.9 The asphaltic concrete, including the joint density, should be compacted to an average relative compaction of 93 percent, with no single test value being below a relative compaction of 91 percent and no single test value being above a relative compaction of 97 percent of the referenced laboratory density according to ASTM D2041.

9.12.10 The asphalt concrete should comply with requirements for a Type "A" asphalt concrete in accordance with the current State of California Department of Transportation (Caltrans) Standard Specification, or the requirements of the governing agency, whichever is more stringent. The Contractor shall provide an asphalt concrete mix design prepared and signed by a California registered civil engineer and approved by Moore Twining prior to construction.

9.13 Portland Cement Concrete (PCC) Pavements

Recommendations for Portland Cement Concrete pavement structural sections are presented in the following subsections. The PCC pavement design assumes a minimum modulus of rupture of 500 psi. The design professional should specify where Portland cement concrete pavements are used based on the anticipated type and frequency of traffic.

9.13.1 The subgrade soils for Portland cement concrete pavements should be over-excavated and compacted as recommended in the “Site Preparation” section of the recommendations in this report. As part of the final preparation, the upper 12 inches of the subgrade soils should be moisture conditioned and compacted to a minimum of 95 percent of the maximum dry density determined in accordance with ASTM D 1557.

9.13.2 The following preliminary Portland cement concrete pavement sections have been prepared for average daily truck traffic ranging from 2 to 21 trucks per day which corresponds to Traffic Indices ranging from about 6 to 8. The design pavement sections should be selected by the civil engineer based on the anticipated traffic loading. If the paved areas are to be used during construction, or if the type and frequency of traffic are greater than assumed in design, the pavement section should be re-evaluated for the anticipated traffic. The design structural sections were prepared based on the procedures outlined in the pavement design application www.pavementdesigner.org (Street Analysis), and assuming the following: 1) minimum modulus of rupture of 500 psi for the concrete, 2) a design life of 20 years, 3) load transfer by aggregate interlock or dowels, 4) concrete shoulder, 5) a load safety factor of 1.1, 6) truck loading consisting of a maximum single axle weight of 16,000 pounds, and a maximum tandem axle weight of 34,000 pounds.

Table No. 5
Portland Cement Concrete Pavements

Traffic Index	ADTT	PCC Thickness (inches)	Aggregate Base (inches)	Compacted Subgrade (inches)
6.0	2	6.0	4.0	12.0
7.0	7	6.0	4.0	12.0
8.0	21	6.5	4.0	12.0

ADTT - Average Daily Truck Traffic based on a loaded garbage/dumpster truck
PCC - Portland Cement Concrete (minimum Modulus of Rupture=500 psi)
Subgrade - Subgrade soils compacted to at least 95 percent relative compaction (ASTM D-1557)

9.13.3 The PCC pavement should be constructed in accordance with the American Concrete Institute requirements, the requirements of the project plans and specifications, whichever is the most stringent. The pavement design engineer should include appropriate construction details and specifications for construction joints, contraction joints, joint filler, concrete specifications, curing methods, etc.

9.13.4 Concrete used for PCC pavements shall possess a minimum flexural strength (modulus of rupture) of 500 pounds per square inch. A minimum compressive strength of 3,500 pounds per square inch, or greater as required by the pavement designer, is recommended. Specifications for the concrete to reduce the effects of excessive shrinkage, such as maximum water requirements for the concrete mix, allowable shrinkage limits, contraction joint construction requirements, etc. should be provided by the designer of the PCC pavement.

9.13.5 Jointing is one of the most critical aspects of the PCC pavement design and construction. Joint spacing, joint type and load transfer devices have significant impacts on the pavement design and performance. Thus, the detailing of joints needs to be considered carefully and applied with clear details on the project plans by the pavement designer/detailer. Positive load transfer devices such as dowels are commonly used at contraction joints whenever the designer cannot be assured aggregate interlock will be maintained.

9.13.6 Specifications for the concrete mixtures used in the PCC pavement to reduce the effects of excessive shrinkage (such as curling and excessive shrinkage at joints), including maximum water requirements for the concrete mix, allowable shrinkage limits, curing methods, etc. should be provided by the designer/detailer of the PCC slabs. In addition, as noted in Section 9.13.5, contraction joint requirements should be detailed by the designer/detailer of the PCC pavement to maintain stability. The minimum PCC thickness noted in this report assumes aggregate interlock occurs at contraction joints. However, curling and excessive shrinkage can disengage aggregate interlock and allow greater pavement deflection at free edges. The design engineer should decide if aggregate interlock is appropriate or specify joint reinforcement.

9.13.7 The pavement section thickness design provided above assumes the design and construction will include sufficient load transfer at construction joints. Coated dowels or load transfer devices are recommended for construction joints to transfer loads. The joint details should be detailed by the pavement design engineer and provided on the plans.

9.13.8 Contraction and construction joints should include a joint filler/sealer to prevent migration of water into the subgrade soils. The type of joint filler should be specified by the pavement designer. The joint sealer and filler material should be maintained throughout the life of the pavement.

9.13.9 Contraction joints should have a depth of at least one-fourth the slab thickness, e.g., 1.5-inch for a 6-inch slab. Specifications for contraction joint spacing, timing and depth of sawcuts should be included in the plans and specifications.

9.13.10 Stresses are anticipated to be greater at the edges and construction joints of the pavement section. A thickened edge is recommended on the outside of slabs subjected to wheel loads.

9.13.11 Joint spacing in feet should not exceed twice the slab thickness in inches, e.g., 12 feet by 12 feet for a 6-inch slab thickness. Regardless of slab thickness, joint spacing should not exceed 15 feet.

9.13.12 Lay out joints to form square panels. When this is not practical, rectangular panels can be used if the long dimension is no more than 1.5 times the short.

9.13.13 Isolation (expansion) joints should extend the full depth and should be used only to isolate fixed objects abutting or within paved areas.

9.13.14 Pavement section design assumes that proper maintenance such as sealing and repair of localized distress will be performed on a periodic basis.

9.14 Temporary Excavations and Shoring

9.14.1 It is the responsibility of the Contractor to provide safe working conditions with respect to excavation slope stability. The Contractor is responsible for site slope safety, and classification of materials for excavation purposes, and maintaining slopes in a safe manner during construction. The grades classification and height recommendations presented for temporary slopes are for consideration in preparing budget estimates and evaluating construction procedures. Temporary cut slopes should not be steeper than 1.5:1, horizontal to vertical, and flatter if possible. Temporary excavations should be constructed in accordance with CAL OSHA requirements.

9.14.2 In no case should excavations extend below a 2H to 1V zone below utilities, foundations and/or floor slabs which are to remain after construction. Excavations which are required to be advanced below the 2H to 1V envelope should be shored to support the soils, foundations, and slabs.

9.14.3 Excavation stability should be monitored by the Contractor. Slope gradient estimates provided in this report do not relieve the Contractor of the responsibility for excavation safety. In the event that tension cracks or distress to the structure occurs, during or after excavation, the owners and Moore Twining should be notified immediately and the Contractor should take appropriate actions to minimize further damage or injury.

9.15 Utility Trenches

9.15.1 The utility trench subgrade should be prepared by excavation of a neat trench without disturbance to the bottom of the trench. If sidewalls are unstable, the Contractor shall either slope the excavation to create a stable sidewall or shore the excavation. All trench subgrade soils disturbed during excavation, such as by accidental over-excavation of the trench bottom, or by excavation equipment with cutting teeth, should be compacted to a minimum of 90 percent relative compaction prior to placement of bedding material. The Contractor is responsible for notifying Moore Twining when these conditions occur and arrange for Moore Twining to observe and test these areas prior to placement of pipe bedding. The Contractor shall use such equipment as necessary to achieve a smooth undisturbed native soil surface at the bottom of the trench with no loose material at the bottom of the trench. The Contractor shall either remove all loose soils or compact the loose soils as engineered fill prior to placement of bedding, pipe and backfill of the trench.

9.15.2 The trench width, type of pipe bedding, the type of initial backfill, and the compaction requirements of bedding and initial backfill material for utility trenches (storm drainage, sewer, water, electrical, gas, cable, phone, irrigation, etc.) should be specified by the project Civil Engineer or applicable design professional in compliance with the manufacturer's requirements, governing agency requirements and this report, whichever is more stringent. The contractor is responsible for contacting the governing agency to determine the requirements for pipe bedding, pipe zone and final backfill. The contractor is responsible for notifying the Owner and Moore Twining if the requirements of the agency and this report conflict, the most stringent applies. For flexible polyvinylchloride (PVC) pipes, these requirements should be in accordance with the manufacturer's requirements or ASTM D-2321, whichever is more stringent, assuming a hydraulic gradient exists (gravel, rock, crushed gravel, etc. cannot be used as backfill on the project). The width of the trench should provide a minimum clearance of 8 inches between the sidewalls of the pipe and the trench, or as necessary to provide a trench width that is 12 inches greater than 1.25 times the outside diameter of the pipe, whichever is greater. As a minimum, the pipe bedding should consist of 4 inches of compacted (92 percent relative compaction) select sand with a minimum sand equivalent of 30 and meeting the following requirements: 100 percent passing the 1/4 inch sieve, a minimum of 90 percent passing the No. 4 sieve and not more than 10 percent passing the No. 200 sieve. The haunches and initial backfill (12 inches above the top of pipe) should consist of a select sand meeting these sand equivalent and gradation requirements that is placed in maximum 6-inch thick lifts and compacted to a minimum relative compaction of 92 percent using hand equipment. The final fill (12 inches above the pipe to the surface) should be on-site or imported, non-expansive materials moisture conditioned and compacted as engineered fill. The project civil engineer should take measures to control migration of moisture in the trenches such as slurry collars, etc.

9.15.3 If ribbed or corrugated HDPE or metal pipes are used on the project, then the backfill should consist of select sand with a minimum sand equivalent of 30, 100 percent passing the 1/4 inch sieve, a minimum of 90 percent passing the No. 4 sieve and not more than 10 percent passing the No. 200 sieve. The sand shall be placed in maximum 6-inch thick lifts, extending to at least 1 foot above the top of pipe, and compacted to a minimum relative compaction of 92 percent using hand equipment. Prior to placement of the pipe, as a minimum, the pipe bedding should consist of 4 inches of compacted (92 percent relative compaction) sand meeting the above sand equivalent and gradation requirements for select sand bedding. The width of the trench should meet the requirements of ASTM D2321 listed in Table No. 6 (minimum manufacturer requirements), or as necessary to provide sufficient space to achieve the required compaction, whichever is greater. As an alternative to the trench width recommended above and the use of the select sand bedding, a lesser trench width for HDPE pipes may be used if the trench is backfilled with a 2-sack sand-cement slurry from the bottom of the trench to 1 foot above the top of the pipe.

Table No. 6
Minimum Trench Widths for HDPE Pipe with
Sand Bedding Initial Backfill

Inside Diameter of HDPE Pipe (inches)	Outside Diameter of HDPE Pipe (inches)	Minimum Trench Width (inches) per ASTM D2321
12	14.2	30
18	21.5	39
24	28.4	48
36	41.4	64
48	55	80

9.15.4 Open graded gravel and rock material such as ¾-inch crushed rock or ½-inch crushed rock should not be used as trench backfill. In the event gravel or rock is required by a regulatory agency for use as backfill, all open graded materials shall be fully encased in a geotextile filter fabric, such as Mirafi 140N, to prevent migration of fine grained soils into the porous material. Gravel and rock cannot be used without the written approval of Moore Twining. If the contractor elects to use crushed rock (and if approved by Moore Twining), the contractor will be responsible for slurry cut off walls at the locations directed by Moore Twining. Crushed rock should be placed in thin (less than 8 inch) lifts and densified with a minimum of three (3) passes using a vibratory compactor.

9.15.5 Utility trench backfill placed in or adjacent to building areas, exterior slabs or pavements should be placed in 8 inch lifts, moisture conditioned and compacted in accordance with the recommendations for engineered fill. Lift thickness can be increased if the contractor can demonstrate the minimum compaction requirements can be achieved. The contractor should use appropriate equipment and methods to avoid damage to utilities and/or structures during placement and compaction of the backfill materials.

9.15.6 Jetting of trench backfill is not allowed to compact the backfill soils.

9.15.7 Where utility trenches extend from the exterior to the interior limits of the building, lean concrete should be used as backfill material for a minimum distance of 2 feet laterally on each side of the exterior building line to prevent the trench from acting as a conduit to exterior surface water.

9.15.8 Storm drains and/or utility lines should be designed to be “watertight.” If encountered, leaks should be immediately repaired. Leaking storm drain and/or utility lines could result in trench failure, sloughing and/or soil movement causing damage to surface and subsurface structures, pavements, flatwork, etc. In addition, landscaping irrigation systems should be monitored for leaks. The Contractor is required to video inspect or pressure test the wet utilities prior to placement of foundations, slabs-on-grade or pavements to verify that the pipelines are constructed properly and are “watertight.” The Contractor shall provide the Owner a copy of the results of the testing. The Contractor is required to repair all noted deficiencies at no cost to the owner.

9.15.9 All utility trenches should be compacted in accordance with the recommendations of this report for engineered fill.

9.15.10 Utility trenches should not be constructed within a zone defined by a line that extends at an inclination of 2 horizontal to 1 vertical downward from the bottom of building foundations.

9.16 Corrosion Protection

9.16.1 Based on the National Association of Corrosion Engineers corrosion severity rating listed in Section 7.3.7 and the analytical results of soil samples, the soils are “corrosive” to “moderately corrosive” to ferrous alloy pipes, as indicated by resistivity value of 4,300 to 8,900 ohm-centimeters and pH values of 6.8 and 6.8. Buried metal objects should be protected in accordance with the manufacturer's recommendations based on the“corrosive” corrosion potential of the soil. The evaluation was limited to the effects of soils to metal objects; corrosion due to other potential sources, such as stray currents and groundwater, was not evaluated.

9.16.2 Based on Table 19.3.1.1 - Exposure categories and classes from Chapter 19 of ACI 318, the sulfate concentration from chemical testing of soil samples falls in the S0 classification. Therefore, restrictions are not required regarding the type, water-to-cement ratio, or strength of the concrete used for foundation and slabs due to the sulfate content. However, a low water to cement ratio of 0.52 lb./lb. or less in the concrete for slabs-on-grade is recommended to reduce the potential for shrinkage cracking and curling of slabs.

9.16.3 These soil corrosion data should be provided to the manufacturers or suppliers of materials that will be in contact with soils (pipes or ferrous metal objects, etc.) to provide assistance in selecting the protection and materials for the proposed products or materials. If the manufacturers or suppliers cannot determine if materials are compatible with the soil corrosion conditions, a professional consultant, i.e., a corrosion engineer, with experience in corrosion protection should be consulted to design parameters. Moore Twining is not a corrosion engineer; thus, we do not provide recommendations for mitigation of corrosive soil conditions. It is recommended that a corrosion engineer be consulted for the site specific conditions.

10.0 DESIGN CONSULTATION

10.1 Moore Twining should be retained to review those portions of the contract drawings and specifications that pertain to earthwork and foundations prior to finalization to determine whether they are consistent with our recommendations. This service is not part of this current contractual agreement.

10.2 If Moore Twining is not retained for review, we assume no liability for the misinterpretation of our conclusions and recommendations. This review is documented by a formal plan/specification review report provided by Moore Twining.

11.0 CONSTRUCTION MONITORING

11.1 It is recommended that Moore Twining be retained to observe the excavation, earthwork, and foundation phases of work to determine that the subsurface conditions are compatible with those used in the analysis and design.

11.2 Moore Twining can conduct the necessary observation and field testing to provide results so that action necessary to remedy indicated deficiencies can be taken in accordance with the plans and specifications. Upon completion of the work, a written summary of our observations, field testing and conclusions will be provided regarding the conformance of the completed work to the intent of the plans and specifications. This service is not, however, part of this current contractual agreement.

11.3 In the event that the earthwork operations for this project are conducted such that the construction sequence is not continuous, (or if construction operations disturb the surface soils) it is recommended that the exposed subgrade that will receive floor slabs be tested to verify adequate compaction and/or moisture conditioning. If adequate compaction or moisture contents are not verified, the fill soils should be over-excavated, scarified, moisture conditioned and compacted are recommended in the Recommendations of this report.

11.4 The construction monitoring is an integral part of this investigation. This phase of the work provides Moore Twining the opportunity to verify the subsurface conditions interpolated from the soil borings and make alternative recommendations if the conditions differ from those anticipated.

11.5 If Moore Twining is not retained to provide engineering observation and field-testing services during construction activities related to earthwork, foundations, pavements and trenches; then, Moore Twining will not be responsible for compliance of any aspect of the construction with our recommendations or performance of the structures or improvements if the recommendations of this report are not followed. It is recommended that if a firm other than Moore Twining is selected to conduct these services that they provide evidence of professional liability insurance satisfactory to the owner and review this report. After their review, the firm should, in writing, state that they understand the conclusions and recommendations of this report and agree to conduct sufficient observations and testing to ensure the construction complies with this report's recommendations. Moore Twining should be notified, in writing, if another firm is selected to conduct observations and field-testing services prior to construction.

11.6 Upon the completion of work, a final report should be prepared by Moore Twining. This report is essential to ensure that the recommendations presented are incorporated into the project construction, and to note any deviations from the project plans and specifications. The client should notify Moore Twining upon the completion of work to prepare a final report summarizing the observations during site preparation activities relative to the recommendations of this report. This service is not, however, part of this current contractual agreement.

12.0 NOTIFICATION AND LIMITATIONS

12.1 The conclusions and recommendations presented in this report are based on the information provided regarding the proposed construction, and the results of the field and laboratory investigation, combined with interpolation of the subsurface conditions.

12.2 If variations or undesirable conditions are encountered during construction, Moore Twining should be notified promptly so that these conditions can be reviewed and our recommendations reconsidered where necessary. It should be noted that unexpected conditions frequently require additional expenditures for proper construction of the project.

12.3 If the proposed construction is relocated or redesigned, or if there is a substantial lapse of time between the submission of our report and the start of work (more than 12 months) at the site, or if conditions have changed due to natural cause or construction operations at or adjacent to the site, the conclusions and recommendations contained in this report should be considered invalid unless the changes are reviewed and our conclusions and recommendations modified or approved in writing.

12.4 Changed site conditions, or relocation of proposed structure(s), may require additional field and laboratory investigations to determine if our conclusions and recommendations are applicable considering the changed conditions or time lapse.

12.5 The conclusions and recommendations contained in this report are valid only for the project discussed in the Anticipated Construction section of this report. The use of the information and recommendations contained in this report for structure on this site not discussed herein is not recommended. The entity or entities that use or cause to use this report or any portion thereof for another structure or site not covered by this report shall hold Moore Twining, its officers and employees harmless from any and all claims and provide Moore Twining's defense in the event of a claim.

12.6 This report is issued with the understanding that it is the responsibility of the client to transmit the information and recommendations of this report to developers, owners, buyers, architects, engineers, designers, contractors, subcontractors, and other parties having interest in the project so that the steps necessary to carry out these recommendations in the design, construction and maintenance of the project are taken by the appropriate party.

12.7 This report presents the results of a geotechnical engineering investigation and geologic/seismic hazards investigation only and should not be construed as an environmental audit or study.

12.8 Our professional services were performed, our findings obtained, and our recommendations prepared in accordance with generally-accepted engineering principles and practices. This warranty is in lieu of all other warranties either expressed or implied.

12.9 Reliance on this report by a third party (i.e., that is not a party to our written agreement) is at the party's sole risk. If the project and/or site are purchased by another party, the purchaser must obtain written authorization and sign an agreement with Moore Twining in order to rely upon the information provided in this report for design or construction of the project.

13.0 CLOSING

We appreciate the opportunity to be of service. If you have any questions regarding this report, or if we can be of further assistance, please contact us at your convenience.

Sincerely,

MOORE TWINING ASSOCIATES, INC.
Geotechnical Engineering Division

DRAFT

Shaun Reich, EIT
Project Engineer

DRAFT

Allen H. Harker, CEG
Certified Engineering Geologist

DRAFT

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Registered Geotechnical Engineer
Division Manager

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United States Geological Survey; Unified Hazard Tool https://earthquake.usgs.gov/hazards/interactive/

United States Geological Survey; U.S. Seismic Design Maps Application https://earthquake.usgs.gov/designmaps/us/application.php

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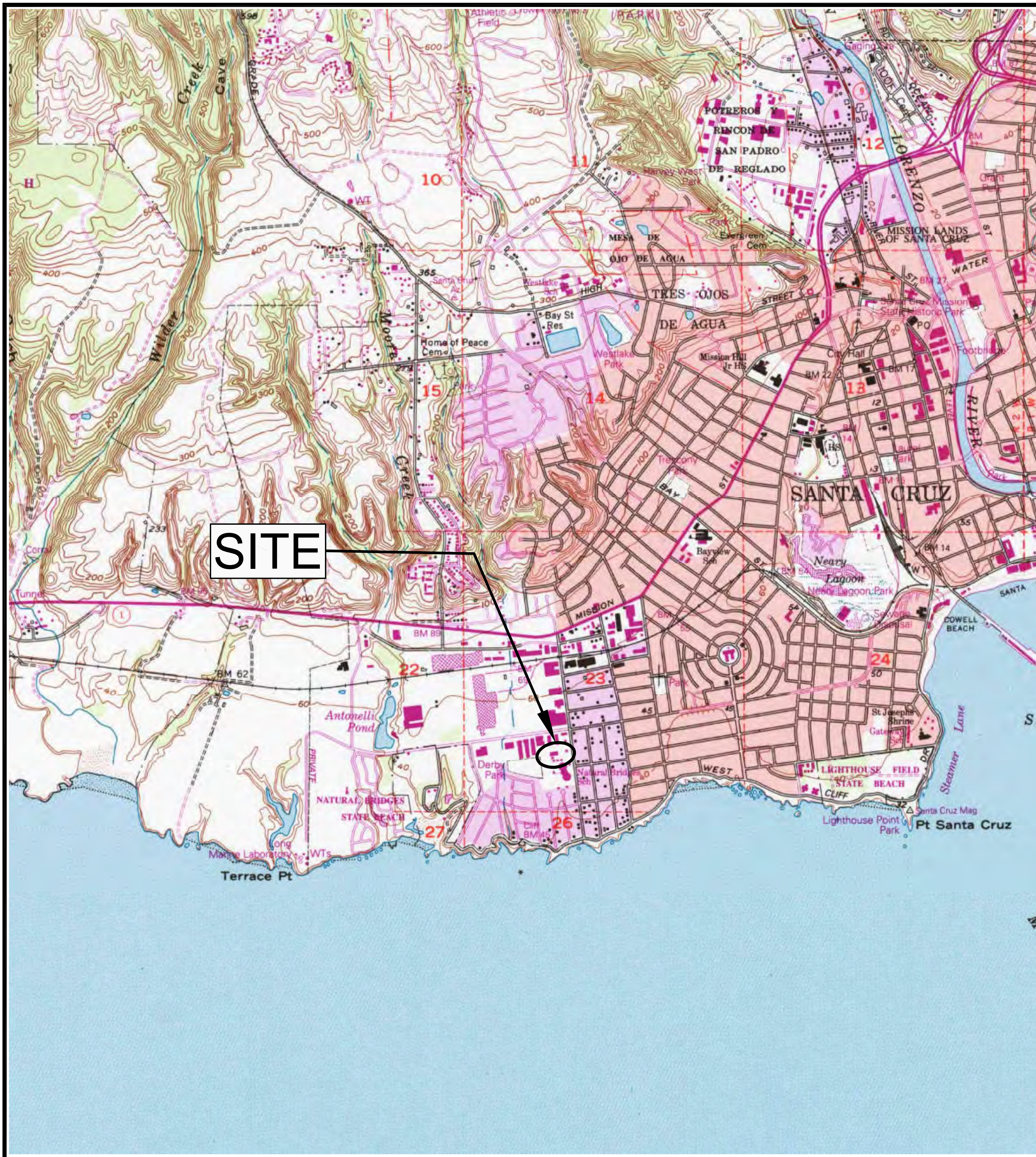
APPENDIX A

DRAWINGS

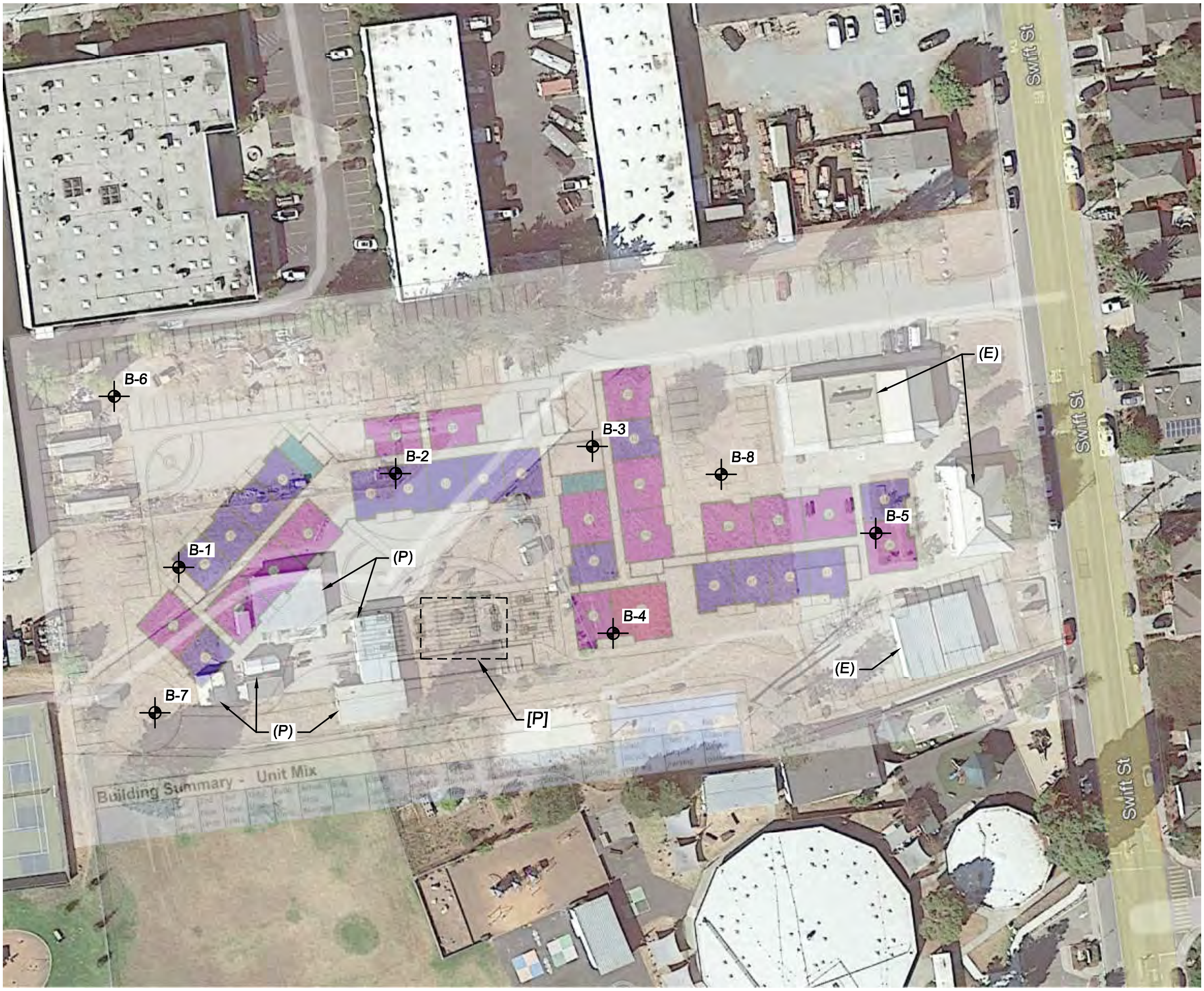
Drawing No. 1 - Site Location Map
Drawing No. 2 - Test Boring Location Map
Drawing No. 3 - Regional Geologic Map
Drawing No. 4 - Map of Major Active and Potentially Active Faults Relative to Site
Drawing No. 5 - Historical Earthquake Epicenter Map
Drawing No. 6 - Site Geologic Map
Drawing No. 7 - Soil Profile Cross Section A-A'
Drawing No. 8 - Soil Profile Cross Section B-B'



SOURCE: U.S.G.S. TOPOGRAPHIC MAP, 7 ½ MINUTE SERIES
SANTA CRUZ, CALIFORNIA QUADRANGLE, 1954, PHOTOREVISED 1994

0 2500
APPROXIMATE SCALE IN FEET

SITE LOCATION MAP PROPOSED THREE-STORY WORKFORCE HOUSING PROJECT SANTA CRUZ CITY SCHOOLS 313 SWIFT STREET SANTA CRUZ COUNTY, CALIFORNIA	FILE NO.: 95815-01-01	DATE: 07/05/2023	MOORE TWINING ASSOCIATES, INC.
	DRAWN BY: JE	APPROVED BY:	
	PROJECT NO. E95815.01	DRAWING NO. 1	



APPROXIMATE TEST BORING LOCATION

(E) EXISTING BUILDING

(P) PREVIOUS BUILDING

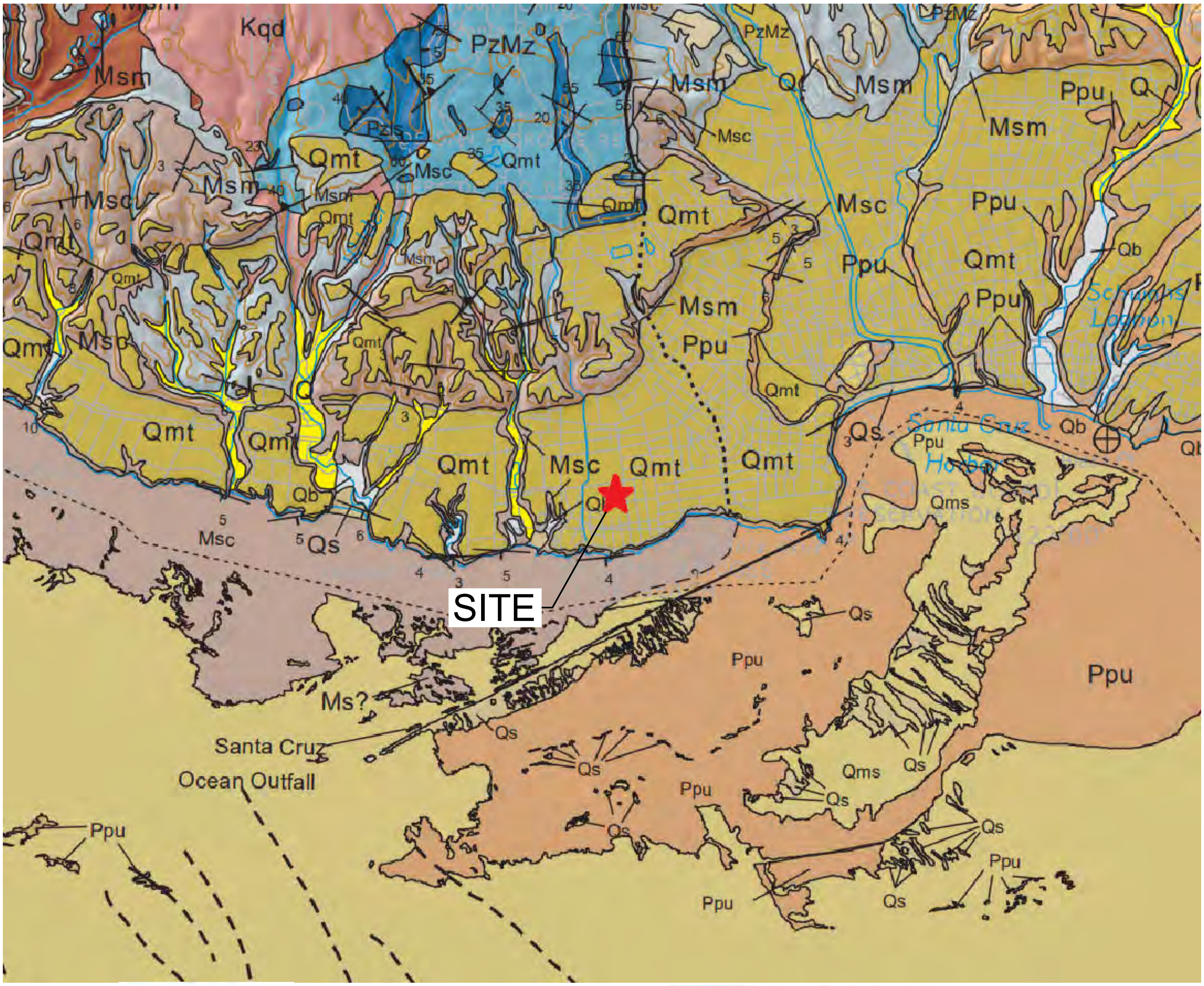
[P] PREVIOUS BUILDING-APPROXIMATE LOCATION

N

0 100

APPROXIMATE SCALE IN FEET

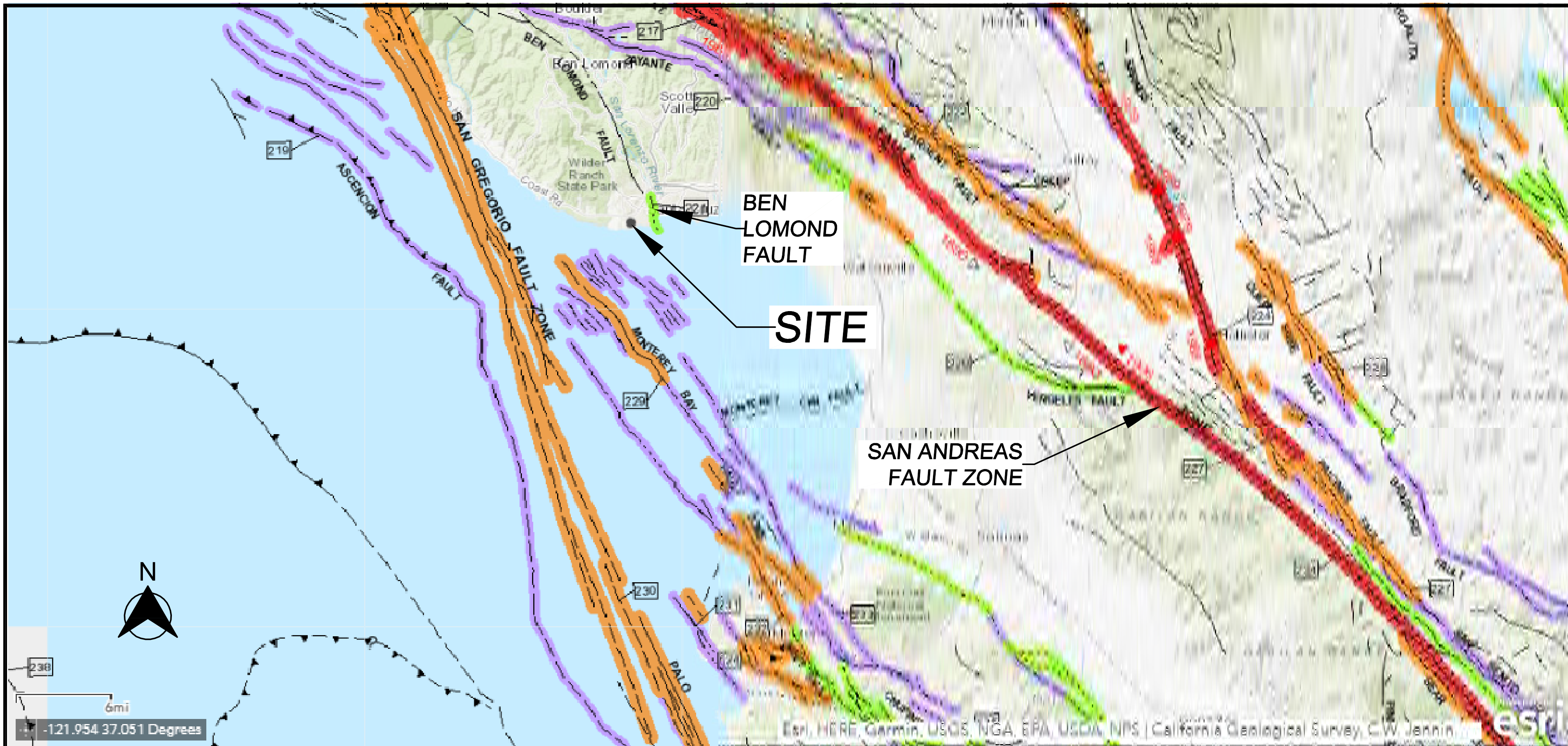
TEST BORING LOCATION MAP PROPOSED THREE-STORY WORKFORCE HOUSING PROJECT SANTA CRUZ CITY SCHOOLS 313 SWIFT STREET SANTA CRUZ COUNTY, CALIFORNIA	FILE NO. 95815-01-02	DATE DRAWN: 08/30/2023	MOORE TWINING ASSOCIATES, INC.
	DRAWN BY: RM	APPROVED BY:	
	PROJECT NO. E95815.01	DRAWING NO. 2	



Symbol	Description
Q	Alluvium
Qmt	Marine terrace deposits
Qs	Sand
Qb	Basin deposits
Ppu	Purisima Formation
Msm	Santa Margarita Sandstone (Mv-Basalt interbed)
Msc	Santa Cruz Mudstone
Kqd	Quartz diorite
PzMz	Prebatholithic metasedimentary rocks (Pzls - Carbonate rocks)
msc	Schist of Sierra de Salinas

REFERENCE: GEOLOGIC MAP OF MONTEREY 30'x60' QUADRANGLE AND ADJACENT AREAS, CALIFORNIA; COMPILED BY DAVID L. WAGNER, H. GARY GREENE, GEORGE J. SAUCEDO AND CYNTHIA L PRIDMORE; 2002

REGIONAL GEOLOGIC MAP PROPOSED THREE-STORY WORKFORCE HOUSING PROJECT SANTA CRUZ CITY SCHOOLS 313 SWIFT STREET SANTA CRUZ COUNTY, CALIFORNIA	FILE NO. 95815-01-02	DATE DRAWN: 08/30/2023	MOORE TWINING ASSOCIATES, INC.
	DRAWN BY: RM	APPROVED BY:	
	PROJECT NO. E95815.01	DRAWING NO. 3	



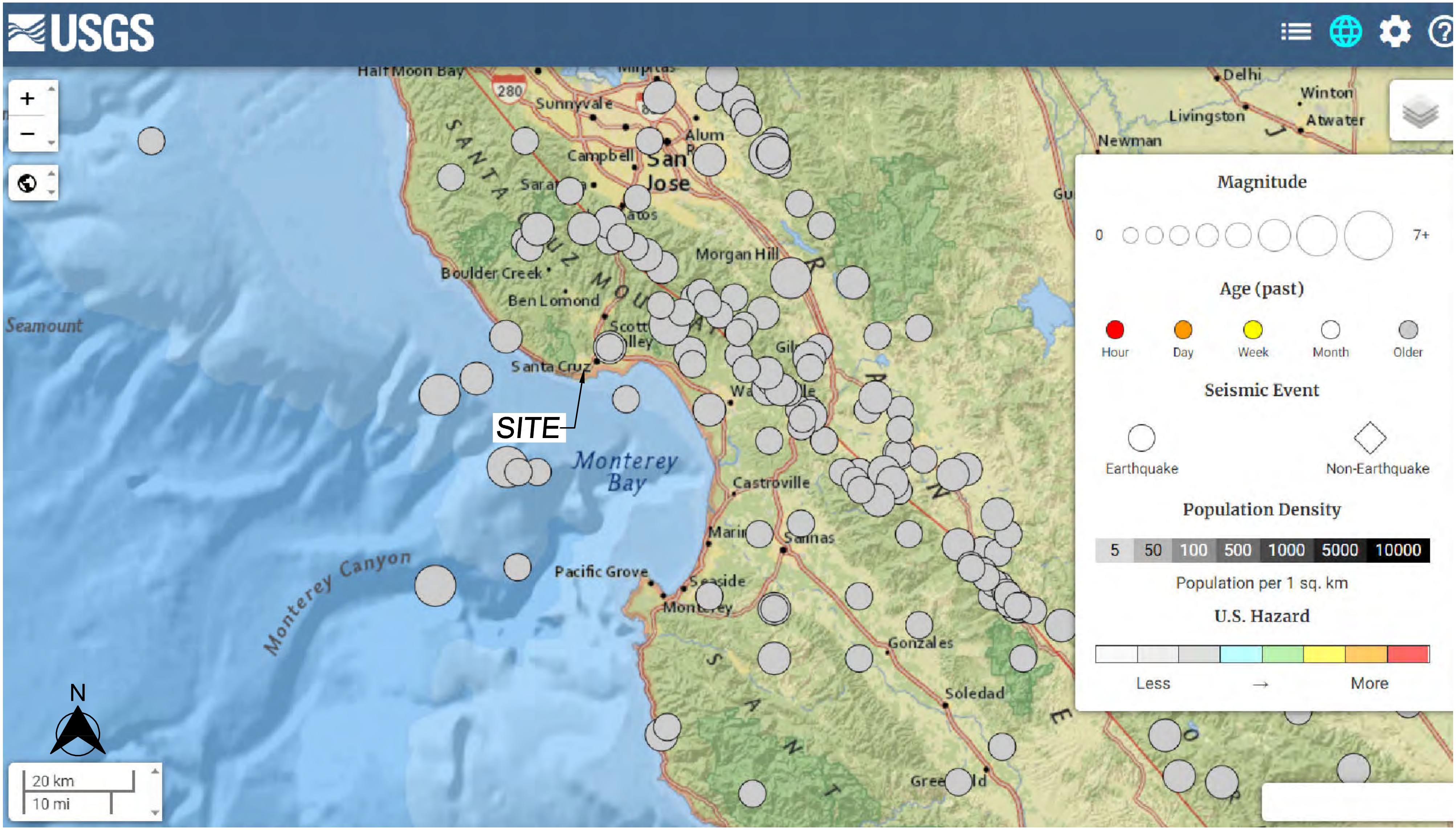
LEGEND

- Fault along which historic (last 200 years) displacement has occurred
- Holocene fault displacement (during past 11,700 years) without historic record
- Late Quaternary fault displacement (during past 700,000 years)
- Quaternary fault (age undifferentiated)

REFERENCE: JENNINGS, CW., AND BRYANT, W.A., 2010 FAULT ACTIVITY MAP OF CALIFORNIA: CALIFORNIA GEOLOGICAL SURVEY, GEOLOGIC DATA MAP NO. 6

MAP OF ACTIVE AND POTENTIALLY ACTIVE FAULTS RELATIVE TO SITE PROPOSED THREE-STORY WORKFORCE HOUSING PROJECT SANTA CRUZ CITY SCHOOLS 313 SWIFT STREET SANTA CRUZ COUNTY, CALIFORNIA	FILE NO. 95815-01-02	DATE DRAWN: 08/30/2023
	DRAWN BY: RM	APPROVED BY:
	PROJECT NO. E95815.01	DRAWING NO. 4

MOORE TWINING ASSOCIATES, INC.



HISTORICAL EARTHQUAKE EPICENTER MAP PROPOSED THREE-STORY WORKFORCE HOUSING PROJECT SANTA CRUZ CITY SCHOOLS 313 SWIFT STREET SANTA CRUZ COUNTY, CALIFORNIA	FILE NO. 95815-01-02	DATE DRAWN: 08/30/2023
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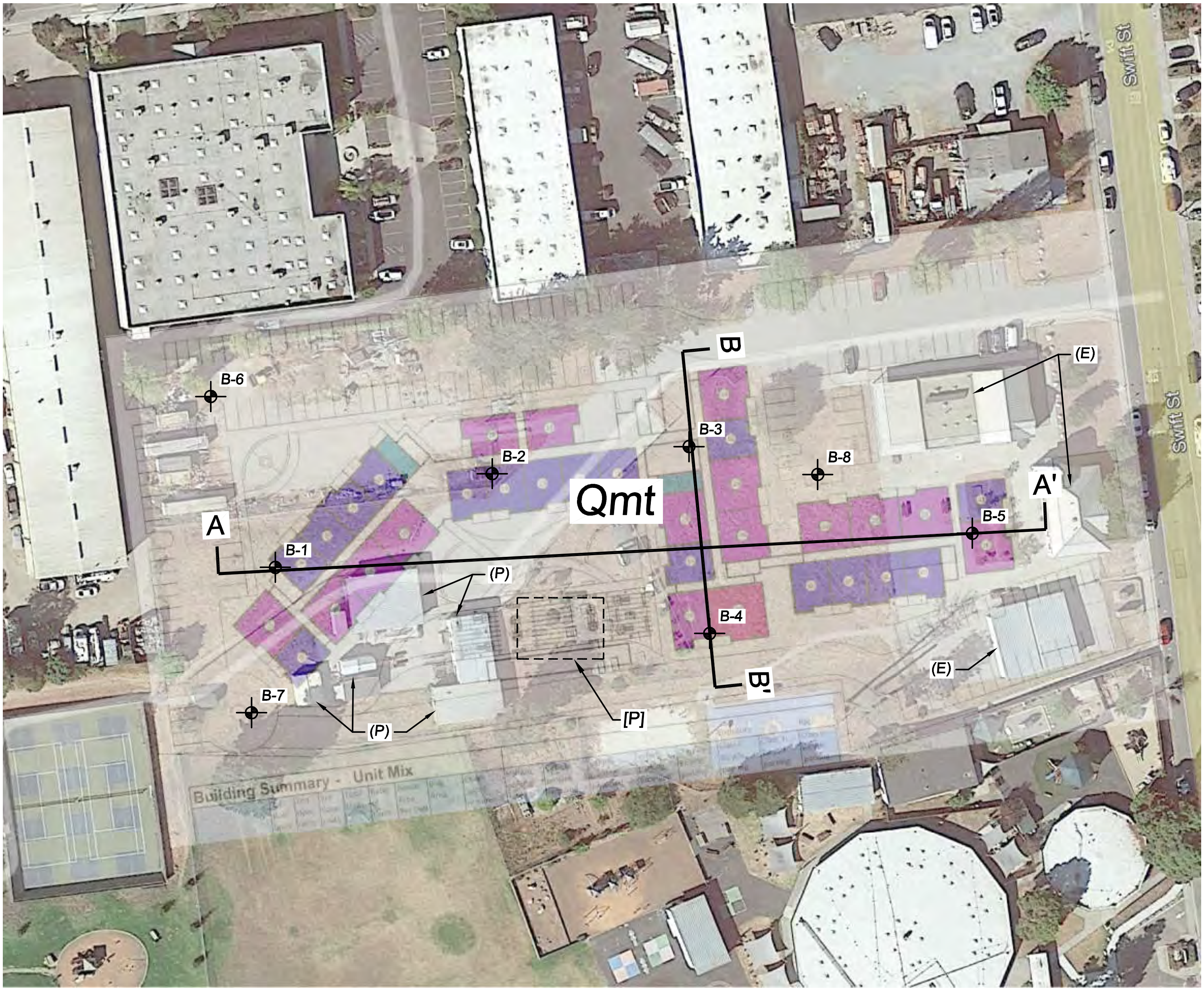
	PROJECT NO. E95815.01	DRAWING NO. 5

MOORE TWINING ASSOCIATES, INC.



APPROXIMATE TEST BORING LOCATION

(E) EXISTING BUILDING

(P) PREVIOUS BUILDING

[P] PREVIOUS BUILDING-APPROXIMATE LOCATION

A A' CROSS SECTION

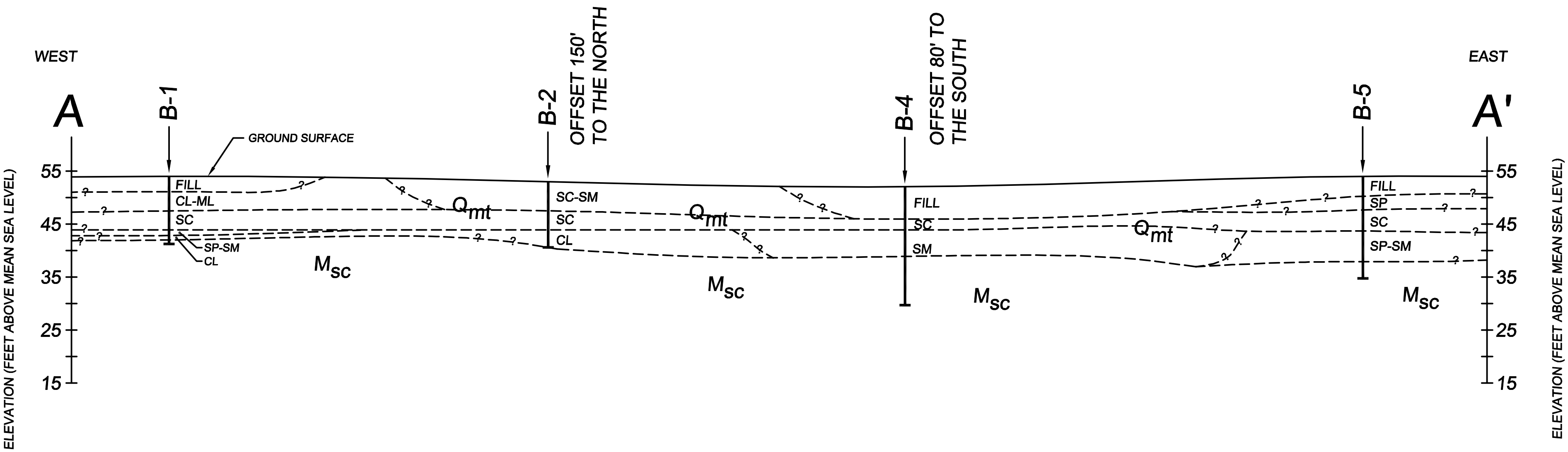
Qmt: MARINE TERRACE DEPOSITS

N

0 100

APPROXIMATE SCALE IN FEET

SITE GEOLOGIC MAP PROPOSED THREE-STORY WORKFORCE HOUSING PROJECT SANTA CRUZ CITY SCHOOLS 313 SWIFT STREET SANTA CRUZ COUNTY, CALIFORNIA	FILE NO. 95815-01-02	DATE DRAWN: 08/30/2023	MOORE TWINING ASSOCIATES, INC.
	DRAWN BY: RM	APPROVED BY:	
	PROJECT NO. E95815.01	DRAWING NO. 6	



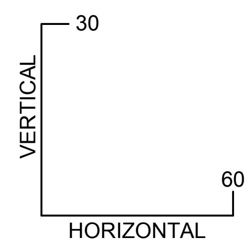
GEOLOGIC UNITS

Q_{mt} = MARINE TERRACE DEPOSIT

M_{sc} = SANTA CRUZ MUDSTONE

SOIL CLASSIFICATION

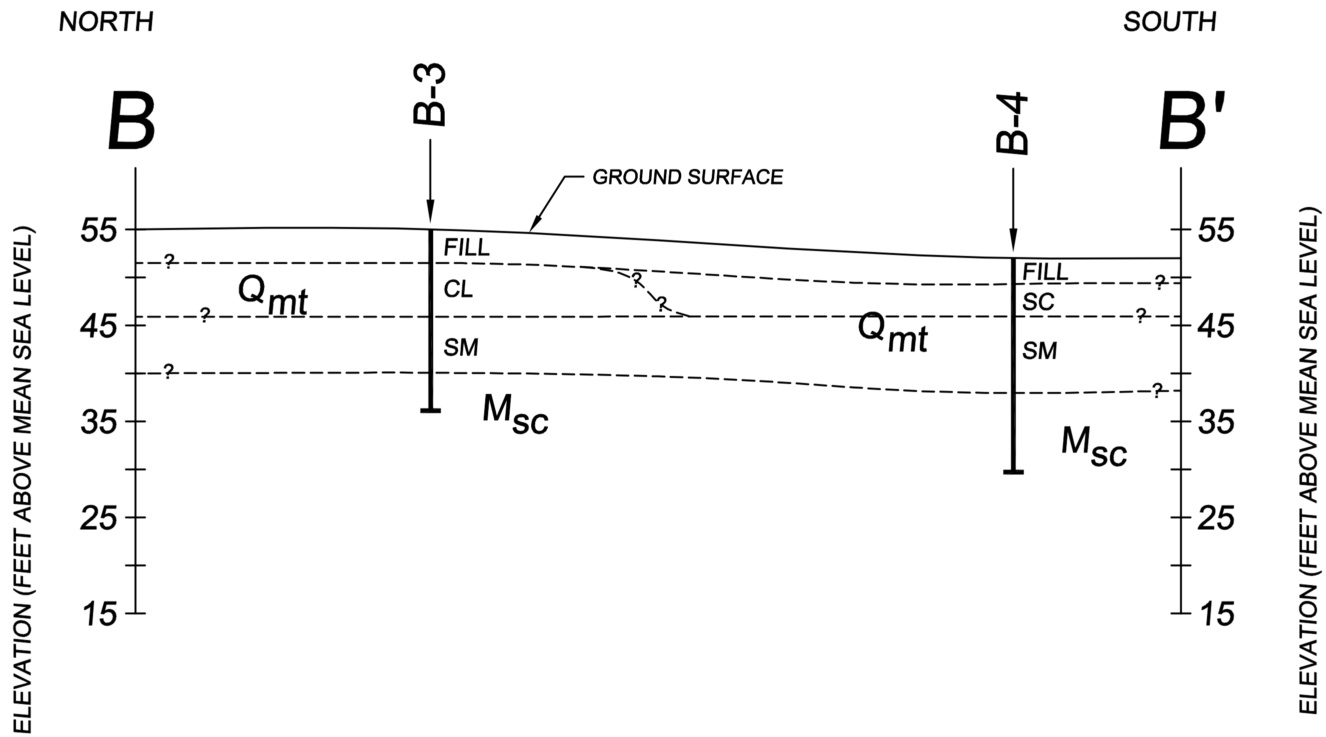
FILL:	FILL
SM:	SILTY SANDS
SC:	CLAYEY SANDS
SP:	POORLY GRADED SANDS
SC-SM:	SILTY CLAYEY SAND
CL:	SANDY LEAN CLAY
CL-SM:	SANDY SILTY CLAY
SP-SM:	POORLY GRADED SAND WITH SILT



APPROXIMATE SCALE IN FEET
2X HORIZONTAL EXAGGERATION

GEOLOGIC SOIL PROFILE CROSS SECTION A-A' PROPOSED THREE-STORY WORKFORCE HOUSING PROJECT SANTA CRUZ CITY SCHOOLS 313 SWIFT STREET SANTA CRUZ COUNTY, CALIFORNIA	FILE NO. 95815-01-02	DATE DRAWN: 08/30/2023
	DRAWN BY: RM	APPROVED BY:
	PROJECT NO. E95815.01	DRAWING NO. 7

MOORE TWINING ASSOCIATES, INC.



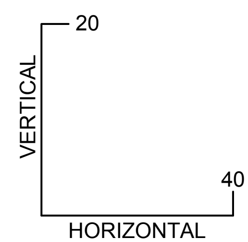
GEOLOGIC UNITS

Q_{mt} = MARINE TERRACE DEPOSIT

M_{sc} = SANTA CRUZ MUDSTONE

SOIL CLASSIFICATION

FILL: FILL
SM: SILTY SANDS AND SILTY SANDS WITH GRAVEL
SC: CLAYEY SAND
CL: SANDY LEAN CLAY



APPROXIMATE SCALE IN FEET
2X HORIZONTAL EXAGGERATION

GEOLOGIC SOIL PROFILE CROSS SECTION B-B'
PROPOSED THREE-STORY WORKFORCE HOUSING PROJECT
SANTA CRUZ CITY SCHOOLS
313 SWIFT STREET
SANTA CRUZ COUNTY, CALIFORNIA

FILE NO. 95815-01-02	DATE DRAWN: 08/30/2023
DRAWN BY: RM	APPROVED BY:
PROJECT NO. E95815.01	DRAWING NO. 8

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APPENDIX B

LOG OF TEST BORINGS

This appendix contains the final boring logs. The logs represents our interpretation of the contents of the field logs and the results of the field and laboratory tests.

The logs and related information depict subsurface conditions only at these locations and at the particular time designated on the logs. Soil conditions at other locations may differ from conditions occurring at the test boring locations. Also, the passage of time may result in changes in the soil conditions.

In addition, an explanation of the abbreviations used in the preparation of the logs and a description of the Unified Soil Classification System are provided at the end of Appendix B.

Test Boring: B-1

Project: Proposed SCCS Workforce Housing Project

Project Number: E95815.01

Drilled By: J.C.

Drill Type: CME 75

Auger Type: Hollow Stem Auger

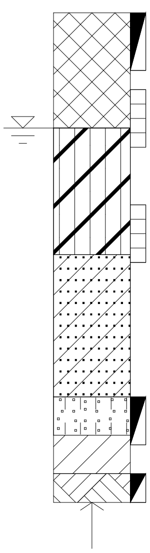
Hammer Type: 140 Pound Auto Trip

Logged By: A.V.

Date: May 11, 2023

Elevation:

Depth to Groundwater First Encountered During Drilling: 3 feet



ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
0	4/6 3/6 3/6 3/6 2/6 3/6	FILL	FILL - SILTY SAND; loose, moist, fine to coarse grained sand with gravel, brown	@ 0-5 ft BSG SR = 8,900 ohm-cm Cl < 0.0040% SS < 0.0040% pH = 6.8	6 5	8.0 19.2
5	2/6" 6/6" 16/6"	CL-ML	NATIVE - SANDY SILTY CLAY; soft, wet, low plasticity, dark brown, increasing sand with depth	@ 2.0 ft BSG DD = 99.5 pcf PI = 7 LL = 22	22	18.7
		SC	CLAYEY SAND; medium dense, wet, fine to coarse grained sand, light brown	@ 5.0 ft BSG DD = 103.1 pcf		
10	3/6" 12/6" 50/3"	SP-SM	POORLY GRADED SAND WITH SILT; medium dense, wet, fine to medium with trace coarse grained sand, brown		>50	18.4
		CL	SANDY LEAN CLAY; hard, moist, low plasticity, brown and gray, with fine sand			31.7
	42/6" 50/3"	ROCK	SILTSTONE; slightly friable, hard, damp to moist, gray, moderately weathered, thinly bedded		>50	
			Auger refusal at 12.75 feet BSG due to competent bedrock			

Notes: *Flat and grass surface. Groundwater measured at a depth of 5 feet BSG after drilling.*

Figure Number

Test Boring: B-2

Project: Proposed SCCS Workforce Housing Project

Project Number: E95815.01

Drilled By: J.C.

Drill Type: CME 75

Auger Type: Hollow Stem Auger

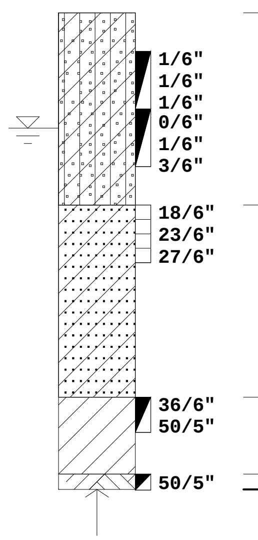
Hammer Type: 140 Pound Auto Trip

Logged By: A.V.

Date: May 11, 2023

Elevation:

Depth to Groundwater First Encountered During Drilling: 3 feet



ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
0	1/6" 1/6" 1/6" 0/6" 1/6" 3/6"	SC-SM	SILTY CLAYEY SAND; very loose, low plasticity, with fine sand, dark brown Loose, wet	@ 0-5 ft BSG EI = 18 @ 1 ft BSG PI = 4 LL = 19 +4 = 0.5% Sand = 53.2% -200 = 46.3% @ 2.5 ft BSG No recovery	2 4	20.2
5	18/6" 23/6" 27/6"	SC	CLAYEY SAND WITH GRAVEL; dense, moist, fine to medium grained sand and gravel, brown	@ 5.0 ft BSG DD = 103.8 pcf	50	25.7
10	36/6" 50/5"	CL	SANDY LEAN CLAY; hard, moist, low plasticity, light brown, with thin lenses of very dense siltstone		>50	31.3
	50/5"	ROCK	SILTSTONE; slightly friable, hard, moist, gray, thinly bedded, moderately weathered		>50	
15			Bottom of Boring at 12.4 feet Auger refusal at 12.4 feet BSG due to competent bedrock			
20						
25						

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.

Test Boring: B-3

Project: Proposed SCCS Workforce Housing Project

Project Number: E95815.01

Drilled By: J.C.

Drill Type: CME 75

Auger Type: Hollow Stem Auger

Hammer Type: 140 Pound Auto Trip

Logged By: A.V.

Date: May 10, 2023

Elevation:

Depth to Groundwater First Encountered During Drilling: 3.5 feet

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
0	2/6" 4/6" 6/6"	FILL	FILL - SANDY LEAN CLAY; stiff, moist, low plasticity, dark brown, some gravel and red brick and concrete debris		10	
5	3/6" 4/6" 7/6"	CL	Medium stiff NATIVE - SANDY LEAN CLAY; medium stiff, wet, low plasticity, dark brown, slight petroleum odor	@ 3.5 ft BSG DD = 103.9pcf ø = 33° c = 360 psf	11	15.8
10	6/6" 6/6" 3/6"	SM	SILTY SAND; loose, wet, fine to coarse grained sand and trace gravel, brown, trace clay		9	23.3
	5/6" 7/6" 7/6"		Increase in sand, without gravel	@ 10 ft BSG DD = 89.1pcf +4 = 0.0% Sand = 81.7% -200 = 17.2%	14	23.4
15	37/6" 50/2"	ROCK	SILTSTONE; slightly friable, hard, damp to moist, brownish-gray, thinly bedded		>50	
	50/5"		increased competency, decreased weathering	Strong hydrocarbon odor	>50	
20			Auger refusal at 18.9 feet BSG due to competent bedrock			
25						

Notes: *Flat and grass surface. Groundwater measured at a depth of 6 feet BSG after drilling.*

Figure Number

MOORE TWINING ASSOCIATES, INC.

Since 1898

Test Boring: B-4

Project: Proposed SCCS Workforce Housing Project

Project Number: E95815.01

Drilled By: J.C.

Drill Type: CME 75

Auger Type: Hollow Stem Auger

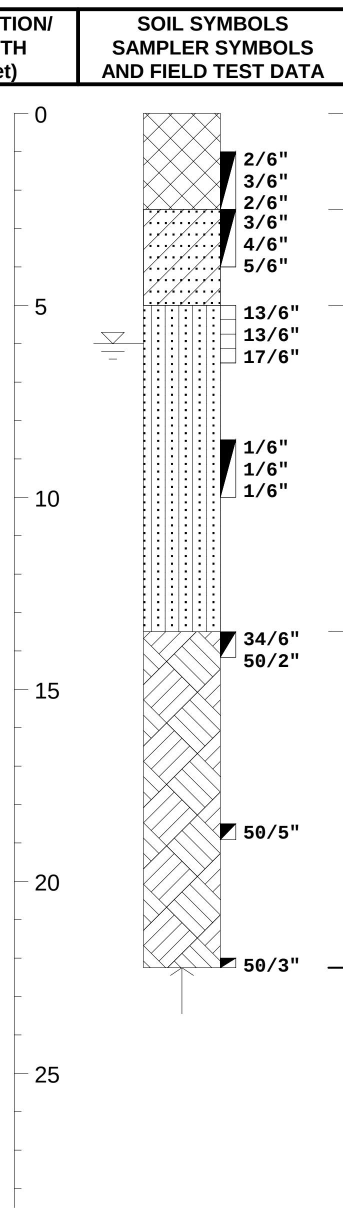
Hammer Type: 140 Pound Auto Trip

Logged By: A.V.

Date: May 10, 2023

Elevation:

Depth to Groundwater First Encountered During Drilling: 6 feet



USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
FILL	FILL - SANDY LEAN CLAY; loose, moist, low plasticity, dark grey, some gravel, with silt	@ 0-5 ft BSG EI = 5 SR = 4,300ohm-cm Cl < 0.0040% SS = 0.0071% pH=6.8	5	14.9
SC	NATIVE - CLAYEY SAND WITH GRAVEL; loose, moist, low plasticity, brown, fine to coarse grained sand and gravel		9	18.8
SM	SILTY SAND WITH GRAVEL; medium dense, wet, fine to coarse grained sand and gravel, brown, with shale fragments	@ 2.5 ft BSG +4 = 25.9% Sand = 51.7% -200 = 22.4%	30	20.9
	Decrease in fines content Very loose, wet, fine to coarse grained sand and gravel, brown, trace clay	@ 5.0 ft BSG DD = 92.7pcf @ 8.5 ft BSG +4 = 29.7% Sand = 53.6% -200 = 16.7%	2	
ROCK	SILTSTONE; moist, hard, friable, gray, moderately weathered, thinly bedded	@ 13.5 ft BSG +4=0.0% Sand=3.9% -200=96.1%	>50	30.9
	Decreased weathering, decreased moisture, slightly friable		>50	28.2
	Auger refusal at 22.25 feet BSG due to competent bedrock	Moderate hydrocarbon odor	>50	39.9

Notes: *Flat and grass surface. Groundwater measured at a depth of 5 feet BSG after drilling.*

Figure Number

MOORE TWINING ASSOCIATES, INC.

Test Boring: B-5

Project: Proposed SCCS Workforce Housing Project

Project Number: E95815.01

Drilled By: J.C.

Drill Type: CME 75

Auger Type: Hollow Stem Auger

Hammer Type: 140 Pound Auto Trip

Logged By: A.V.

Date: May 10, 2023

Elevation:

Depth to Groundwater First Encountered During Drilling: N/E

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
0	2/6" 4/6" 6/6"	FILL	FILL - SILTY, CLAYEY SAND WITH GRAVEL; loose, moist, fine to coarse grained sand and gravel, brown	@ 0 ft BSG PI = 7 LL = 24 +4 = 19.8% Sand = 50.7% -200 = 29.5%	10	6.7
	6/6" 6/6" 10/6"	SP	NATIVE - POORLY GRADED SAND; loose, moist, fine to coarse grained sand, brown		16	3.2
5	3/6" 7/6" 9/6"	SC	CLAYEY SAND WITH GRAVEL; medium dense, moist, fine to coarse grained sand, orange-brown	@ 3.5 ft BSG DD = 111.7pcf	16	30.0
10	7/6" 20/6" 20/6"	SP-SM	POORLY GRADED SAND WITH SILT; medium dense, very moist, fine to medium with trace coarse grained sand, brown	@ 10 ft BSG DD = 94.8pcf ø = 25° c = 620psf	40	25.0
15	17/6" 50/4"	ROCK	SILTSTONE; hard, moist, friable, highly weathered, gray, thinly bedded		>50	26.7
	26/6" 50/3"		Decreased moisture, moderately weathered		>50	25.5
20			Auger refusal encountered at 19.25 feet BSG due to competent bedrock			
25						

Notes: *Flat and grass surface. Groundwater measured at a depth of 12 feet BSG after drilling.*

Figure Number

MOORE TWINING ASSOCIATES, INC.

Test Boring: B-6

Project: Proposed SCCS Workforce Housing Project

Project Number: E95815.01

Drilled By: J.C.

Drill Type: CME 75

Auger Type: Hollow Stem Auger

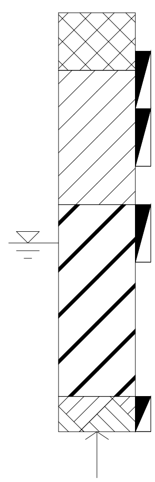
Hammer Type: 140 Pound Auto Trip

Logged By: A.V.

Date: May 11, 2023

Elevation:

Depth to Groundwater First Encountered During Drilling: 6 feet



ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
0	1/6" 1/6" 2/6"	FILL	FILL - SILTY SAND; moist, fine to medium grained sand, dark brown, some gravel, grass roots		3	18.2
	3/6" 3/6" 3/6"	CL	NATIVE - SANDY LEAN CLAY; very soft, moist, with fine grained sand, dark brown, low to medium plasticity		6	24.2
5	5/6" 2/6" 5/6"	CH	Soft, with some gravel		7	28.4
			FAT CLAY; medium stiff, very moist to wet, with trace fine grained sand, brown with iron-oxide staining, high plasticity			
10	35/6" 50/5"	ROCK	SILTSTONE; hard, low moisture, moderately weathered, dark brown, thinly bedded	Slight petroleum odor	>50	
			Auger refusal at 10.9 feet BSG due to competent bedrock			
15						
20						
25						

Notes: *Flat. Groundwater measured at a depth of 5 feet BSG after drilling.*

Figure Number

MOORE TWINING ASSOCIATES, INC.

Since 1898

Test Boring: B-7

Project: Proposed SCCS Workforce Housing Project

Project Number: E95815.01

Drilled By: J.C.

Drill Type: CME 75

Auger Type: Hollow Stem Auger

Hammer Type: 140 Pound Auto Trip

Logged By: A.V.

Date: May 11, 2023

Elevation:

Depth to Groundwater First Encountered During Drilling: 5 feet

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
0	1/6" 1/6" 1/6"	CL	SANDY LEAN CLAY; very soft, moist, with fine grained sand, low to medium plasticity, dark brown, with silt		2	18.1
	1/6" 1/6" 1/6"				2	18.9
			Slight increase in sand, low plasticity			
			At 2.5 ft BSG - Very moist			
5	6/6" 10/6" 20/6"	CL	SANDY LEAN CLAY; very stiff, moist, brown with iron-oxide staining, with shale fragments		30	23.4
			Bottom of Boring B-7 at 6.5 feet BSG			
10						
15						
20						
25						

Notes: *Flat and grass surface. Groundwater measured at a depth of 5 feet BSG after drilling.*

Figure Number

MOORE TWINING ASSOCIATES, INC.

Since 1898

Test Boring: B-8

Project: Proposed SCCS Workforce Housing Project

Project Number: E95815.01

Logged By: A.V.

Drilled By: J.C.

Date: May 10, 2023

Drill Type: CME 75

Elevation:

Auger Type: Hollow Stem Auger

Depth to Groundwater First Encountered During Drilling: N/E

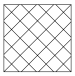
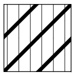
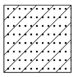
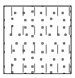
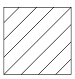
Hammer Type: 140 Pound Auto Trip

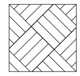
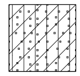
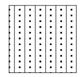
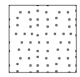
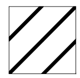
ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
0	2/6" 3/6" 3/6"	CL	SANDY LEAN CLAY; medium stiff, moist, with fine grained sand and gravel, medium plasticity, dark brown, grass roots in upper 6 inches		6	11.7
	2/6" 2/6" 3/6"		Increased fine grained sand, low plasticity		5	17.5
5			Bottom of Boring B-8 at 5 feet BSG			
10						
15						
20						
25						

Notes: *Flat and grass surface*

Figure Number

KEY TO SYMBOLS

Symbol	Description
Strata symbols	
	Fill
	Sandy, Silty Clay
	Clayey Sand
	Poorly Graded Sand with Silt
	Lean Clay

Symbol	Description
	Sedimentary Rock
	Silty, Clayey Sand
	Silty Sand
	Poorly Graded Sand
	Fat clay

Notes:

1. Exploratory borings were drilled on April 10 and 11, 2023 using a CME 75 drill rig equipped with 6-5/8" outside diameter hollow stem augers.

2. Groundwater was encountered during drilling of the borings, see logs.

3. Boring locations were measured or paced from existing features. Elevations were provided from a Topographic Survey prepared by Carroll Engineering.

4. These logs are subject to the limitations, conclusions, and recommendations in this report.

5. The "N-value" reported for the California Modified Split Barrel Sampler is the uncorrected field blow count. This value should not be interpreted as an SPT equivalent N-value.

6. Results of tests conducted on samples recovered are reported on the logs.

DD =	Natural dry density (pcf)	LL =	Liquid Limit (%)
+4 =	Percent retained on the No. 4 sieve(%)	PI =	Plasticity Index (%)
-200 =	Percent passing the No. 200 sieve (%)	EI =	Expansion Index
Sand =	Percent passing the No. 4 sieve and retained on No. 200 sieve (%)	Gravel =	Percent passing 3-inch & retained on No. 4 sieves(%)
pH =	Soil pH	SR =	Soil resistivity (ohms-cm)
SS =	Soluble sulfates (%)	Cl =	Soluble chlorides (%)
ø =	Internal Angle of Friction (degrees)	c =	Cohesion (psf)
pcf =	Pounds per cubic foot	psf =	Pounds per square foot
O.D. =	Outside diameter	AMSL =	Above mean sea level
N/A =	Not applicable	N/E =	Not encountered
BSG =	Below Site Grade Elevation	NV =	Non-Viscous
NP =	Non-Plastic		

KEY TO SYMBOLS

Symbol	Description
Misc. Symbols	
	Water table during drilling
	Drill rejection
Soil Samplers	
	Standard penetration test
	California Modified split barrel ring sampler

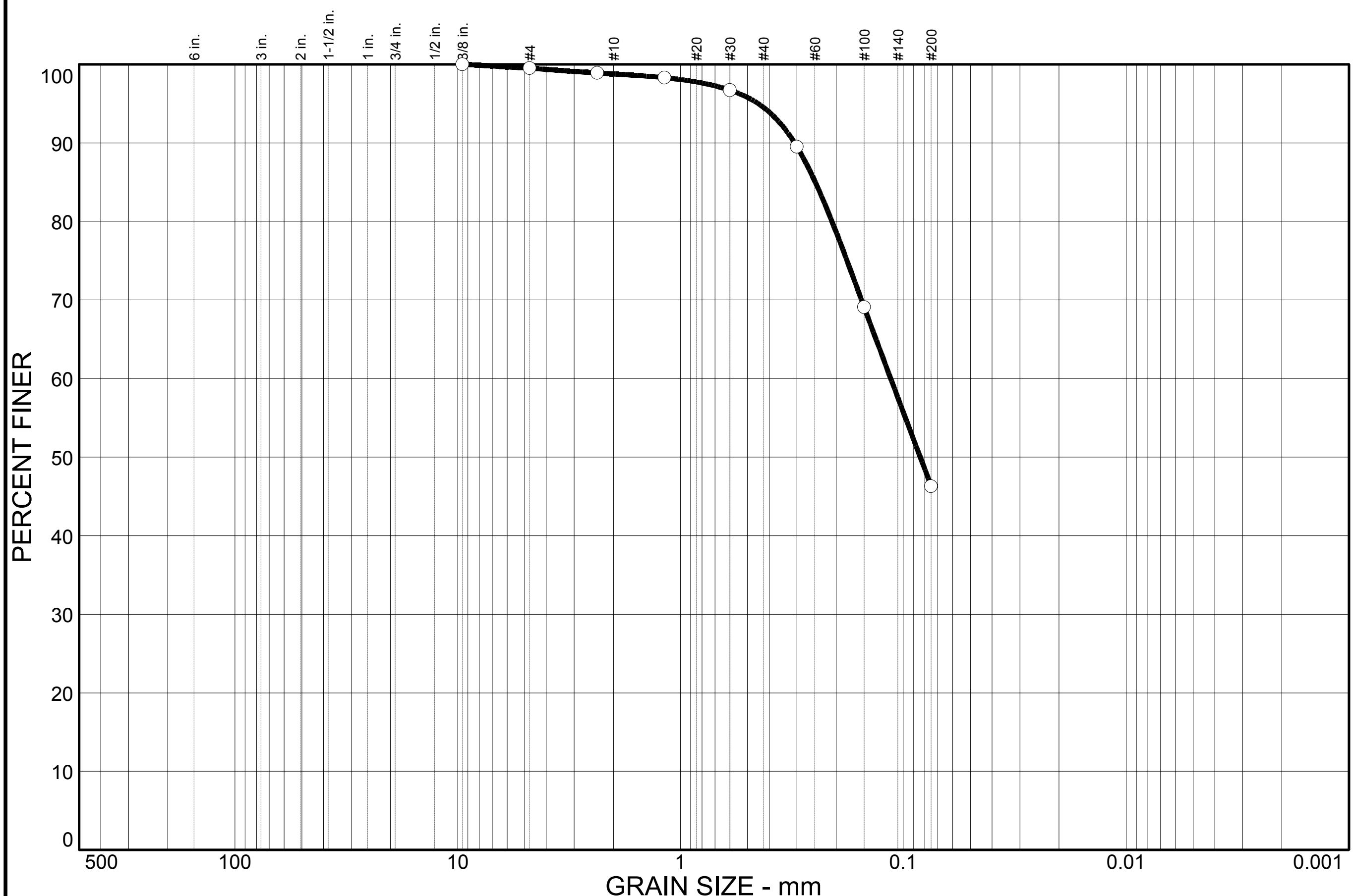
APPENDIX C

RESULTS OF LABORATORY TESTS

This appendix contains the individual results of the following tests. The results of the moisture content and dry density tests are included on the test boring logs in Appendix B. These data, along with the field observations, were used to prepare the final test boring logs in Appendix B.

These Included:	To Determine:
Moisture Content (ASTM D2216)	Moisture contents representative of field conditions at the time the sample was taken.
Dry Density (ASTM D2216)	Dry unit weight of sample representative of in-situ or in-place undisturbed condition.
Grain-Size Distribution (ASTM D422)	Size and distribution of soil particles, i.e., sand, gravel and fines (silt and clay).
Atterberg Limits (ASTM D4318)	Determines the moisture contents where the soil behaves as a viscous material (liquid limit) and the moisture content at which the soil reaches a plastic state (plastic limit)
Direct Shear (ASTM D3080)	Soil shearing strength under varying loads and/or moisture conditions.
Sulfate Content (ASTM D4327)	Percentage of water-soluble sulfate as (SO4) in soil samples. Used as an indication of the relative degree of sulfate attack on concrete and for selecting the cement type.
Expansion Index (ASTM D4829)	Swell potential of soil with increases in moisture content.
Moisture-Density Relationship (ASTM D698)	The optimum (best) moisture content for compacting soil and the maximum dry unit weight (density) for a given compactive effort.
Resistance-Value (ASTM D2844)	The capacity of a subgrade or subbase to support a pavement section designed to carry a specified traffic load.
Chloride Content (Cal Test 422)	Percentage of soluble chloride in soil. Used to evaluate the potential attack on encased reinforcing steel.
Sulfate Content (Cal Test 417)	Percentage of water-soluble sulfate as (SO4) in soil samples. Used as an indication of the relative degree of sulfate attack on concrete and for selecting the cement type.
Minimum Resistivity (ASTM G187)	The minimum resistivity in ohm-centimeters of a soil. Used to determine the corrosion potential for buried metal objects
pH (Cal Test 643)	The acidity or alkalinity of subgrade material.

Particle Size Distribution Report



% COBBLES	% GRAVEL		% SAND			% FINES	
	CRS.	FINE	CRS.	MEDIUM	FINE	SILT	CLAY
0.0	0.0	0.5	0.7	4.2	48.3	46.3	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3/8 in.	100.0		
#4	99.5		
#8	98.9		
#16	98.3		
#30	96.7		
#50	89.5		
#100	69.1		
#200	46.3		

* (no specification provided)

Material Description

Silty, clayey sand

Atterberg Limits

PL= 15 LL= 19 PI= 4

Coefficients

D_{85}= 0.248 D_{60}= 0.114 D_{50}= 0.0840
D_{30}= D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= SC-SM AASHTO=

Remarks

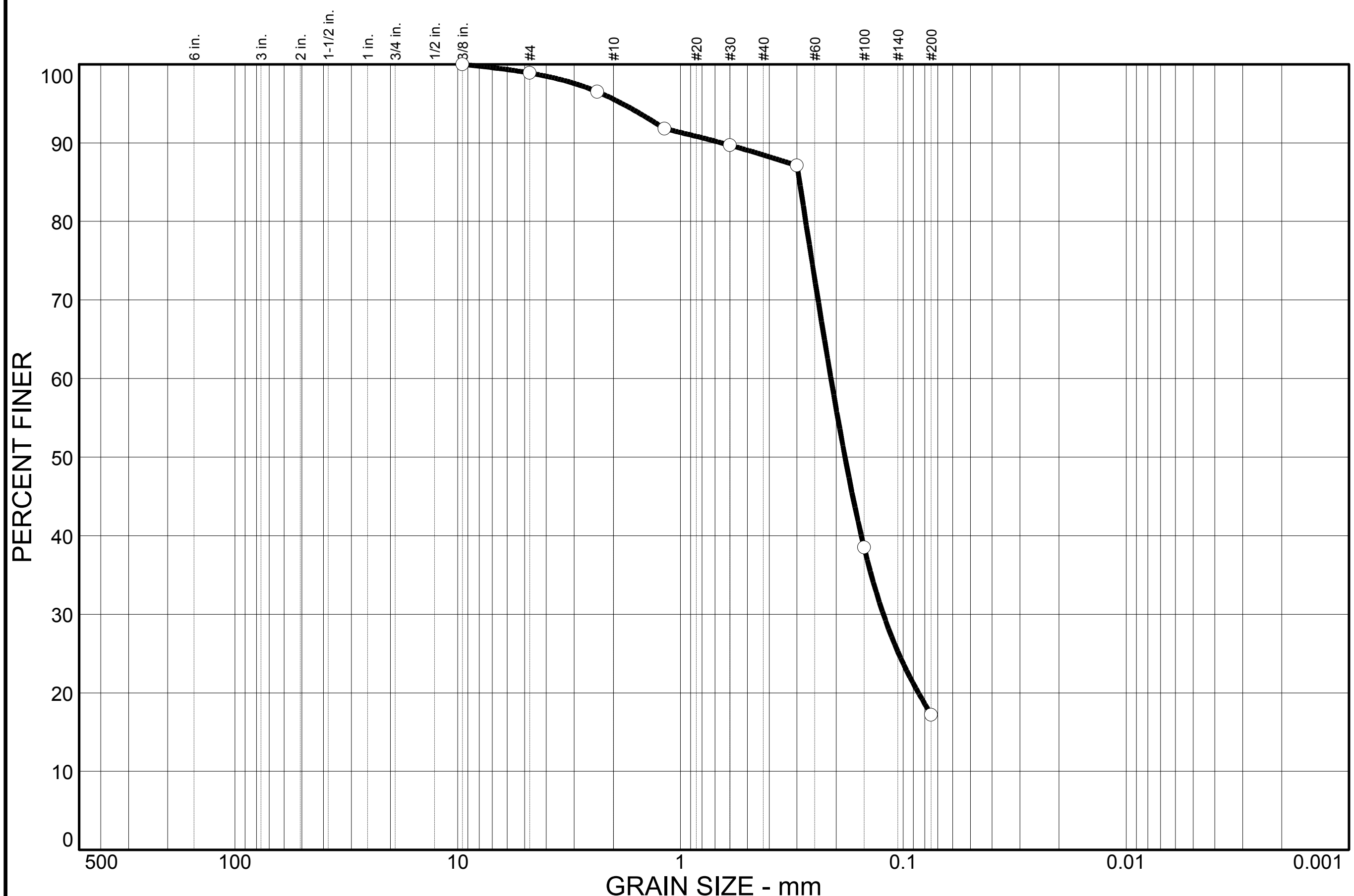
Sample No.: B-2 **Source of Sample:** **Date:** 5/11/23
Location: **Elev./Depth:** 1-2.5'

Moore Twining Associates, Inc.

Fresno, CA

Client: Santa Cruz City Schools
Project: SCCS Workforce Housing
Project No: E95815.01 **Figure**

Particle Size Distribution Report



% COBBLES	% GRAVEL		% SAND			% FINES	
	CRS.	FINE	CRS.	MEDIUM	FINE	SILT	CLAY
0.0	0.0	1.1	3.3	7.1	71.3	17.2	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3/8 in.	100.0		
#4	98.9		
#8	96.5		
#16	91.8		
#30	89.7		
#50	87.1		
#100	38.5		
#200	17.2		

* (no specification provided)

Material Description

Silty sand

Atterberg Limits

PL= LL= PI=

Coefficients

D_{85}= 0.292 D_{60}= 0.211 D_{50}= 0.183
D_{30}= 0.123 D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= SM AASHTO=

Remarks

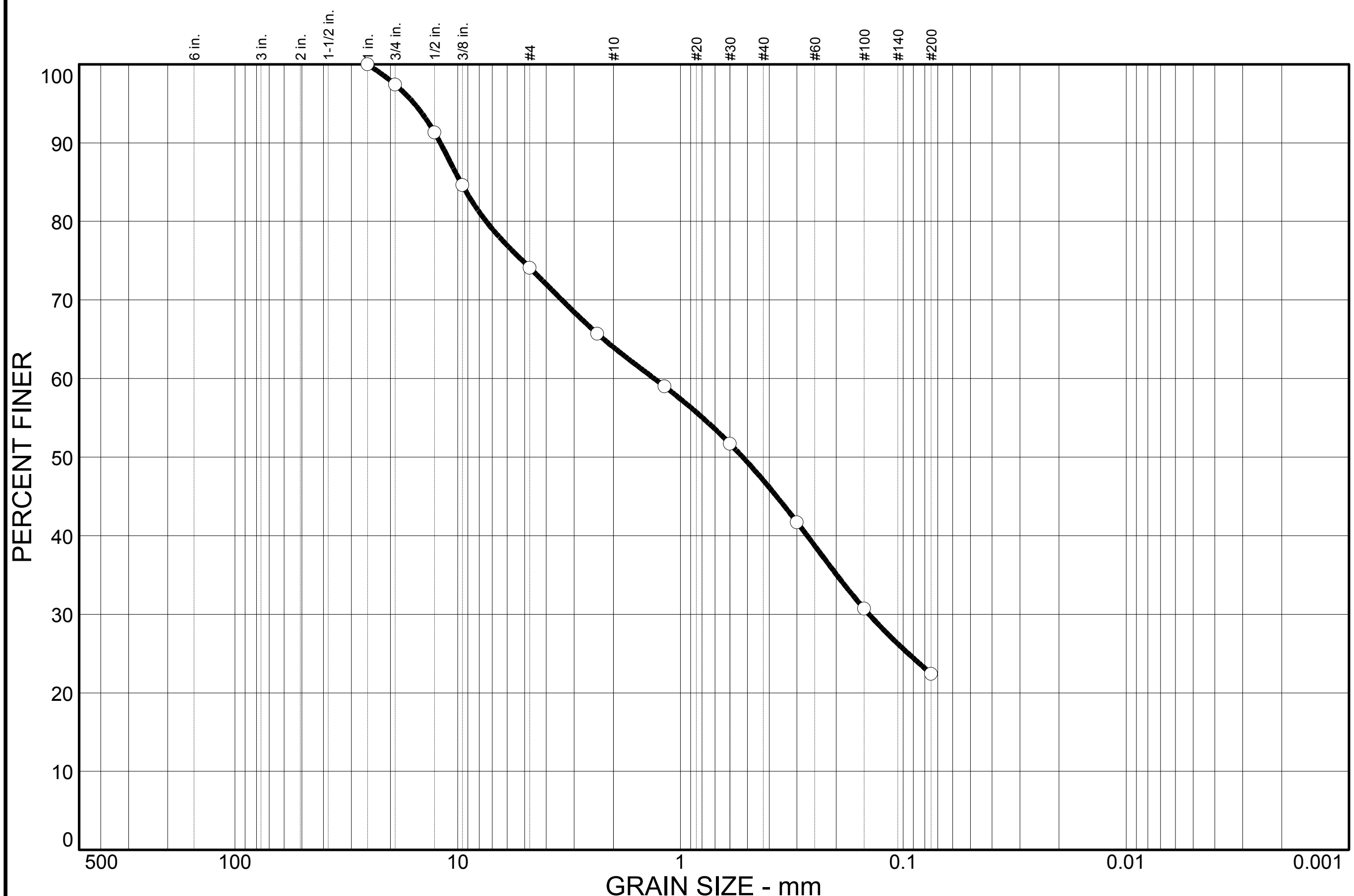
Sample No.: B-3 **Source of Sample:** **Date:** 5/11/23
Location: **Elev./Depth:** 10-11.5'

Moore Twining Associates, Inc.

Fresno, CA

Client: Santa Cruz City Schools
Project: SCCS Workforce Housing
Project No: E95815.01 **Figure**

Particle Size Distribution Report



% COBBLES	% GRAVEL		% SAND			% FINES	
	CRS.	FINE	CRS.	MEDIUM	FINE	SILT	CLAY
0.0	2.6	23.3	10.1	16.9	24.7	22.4	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1 in.	100.0		
3/4 in.	97.4		
1/2 in.	91.3		
3/8 in.	84.6		
#4	74.1		
#8	65.7		
#16	59.0		
#30	51.7		
#50	41.7		
#100	30.7		
#200	22.4		

* (no specification provided)

Material Description

Clayey sand with gravel

Atterberg Limits

PL= LL= PI=

Coefficients

D_{85}= 9.70 D_{60}= 1.31 D_{50}= 0.526
D_{30}= 0.143 D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= SC AASHTO=

Remarks

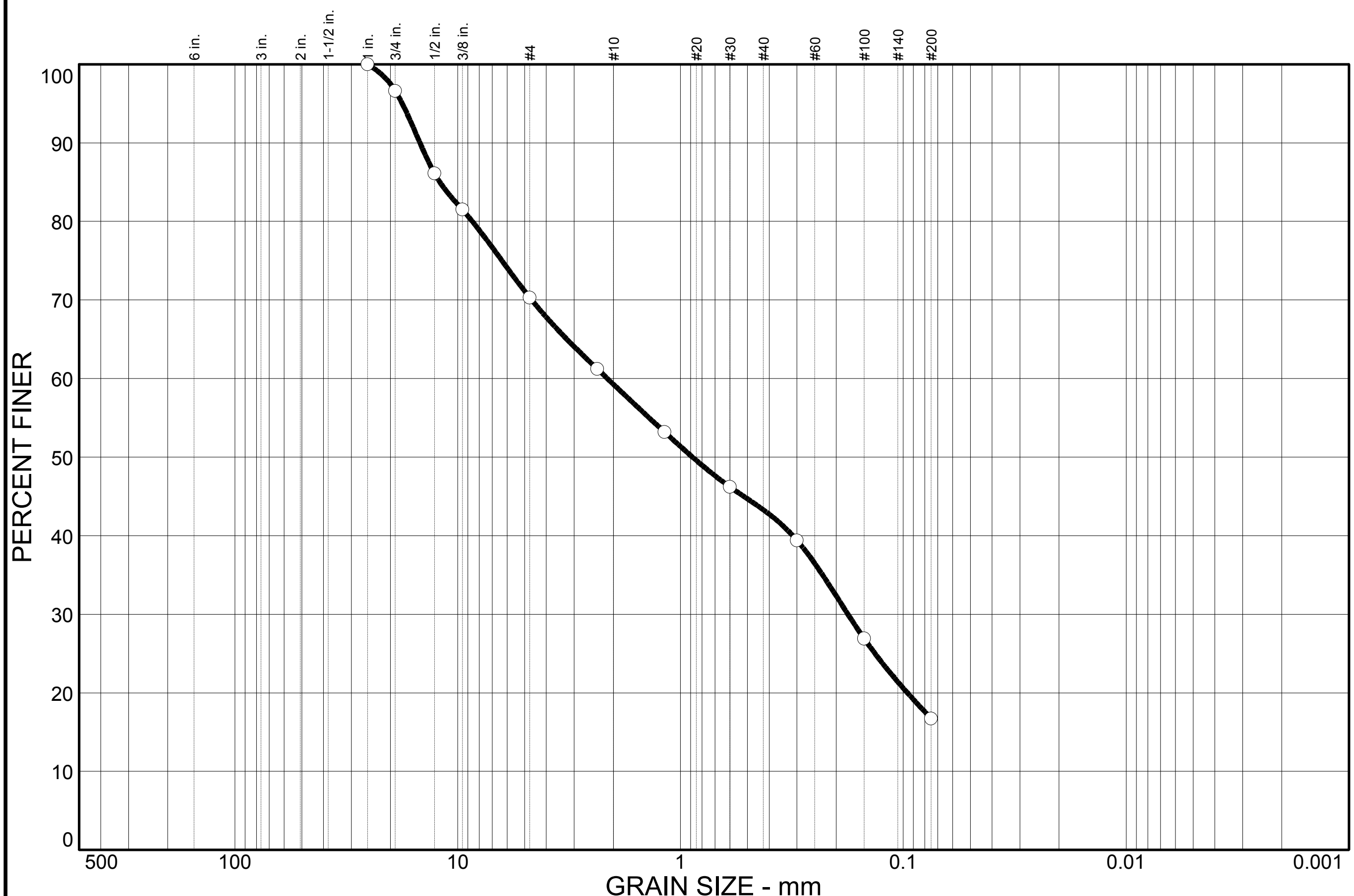
Sample No.: B-4 **Source of Sample:** **Date:** 5/11/23
Location: **Elev./Depth:** 2.5-4'

Moore Twining Associates, Inc.

Fresno, CA

Client: Santa Cruz City Schools
Project: SCCS Workforce Housing
Project No: E95815.01 **Figure**

Particle Size Distribution Report



% COBBLES	% GRAVEL		% SAND			% FINES	
	CRS.	FINE	CRS.	MEDIUM	FINE	SILT	CLAY
0.0	3.4	26.3	11.0	16.0	26.6	16.7	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1 in.	100.0		
3/4 in.	96.6		
1/2 in.	86.1		
3/8 in.	81.5		
#4	70.3		
#8	61.2		
#16	53.2		
#30	46.2		
#50	39.4		
#100	26.9		
#200	16.7		

* (no specification provided)

Material Description

Silty sand with gravel

Atterberg Limits

PL= LL= PI=

Coefficients

D_{85}= 12.0 D_{60}= 2.13 D_{50}= 0.883
D_{30}= 0.177 D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= SM AASHTO=

Remarks

Sample No.: B-4 **Source of Sample:** **Date:** 5/11/23
Location: **Elev./Depth:** 8.5-10'

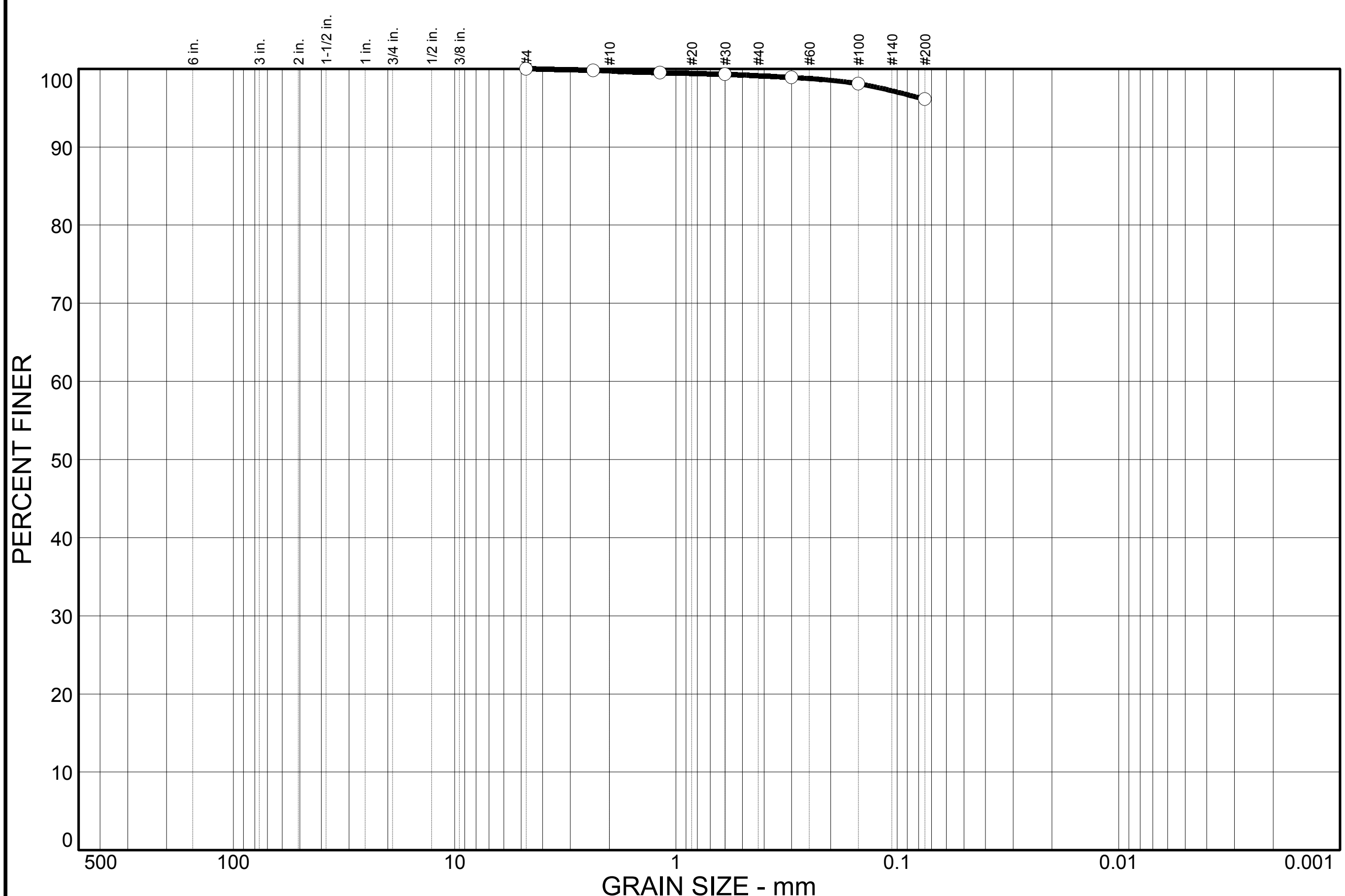
Moore Twining Associates, Inc.

Fresno, CA

Client: Santa Cruz City Schools
Project: SCCS Workforce Housing

Project No: E95815.01 **Figure**

Particle Size Distribution Report



% COBBLES	% GRAVEL		% SAND			% FINES	
	CRS.	FINE	CRS.	MEDIUM	FINE	SILT	CLAY
0.0	0.0	0.0	0.3	0.6	3.0	96.1	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#4	100.0		
#8	99.8		
#16	99.5		
#30	99.3		
#50	98.9		
#100	98.1		
#200	96.1		

* (no specification provided)

Material Description

Marine terrace deposits - Siltstone

Atterberg Limits

PL= LL= PI=

Coefficients

D_{85}= D_{60}= D_{50}=
D_{30}= D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= Qmt AASHTO=

Remarks

Sample No.: B-4 **Source of Sample:** **Date:** 5/11/23
Location: **Elev./Depth:** 13.5-15'

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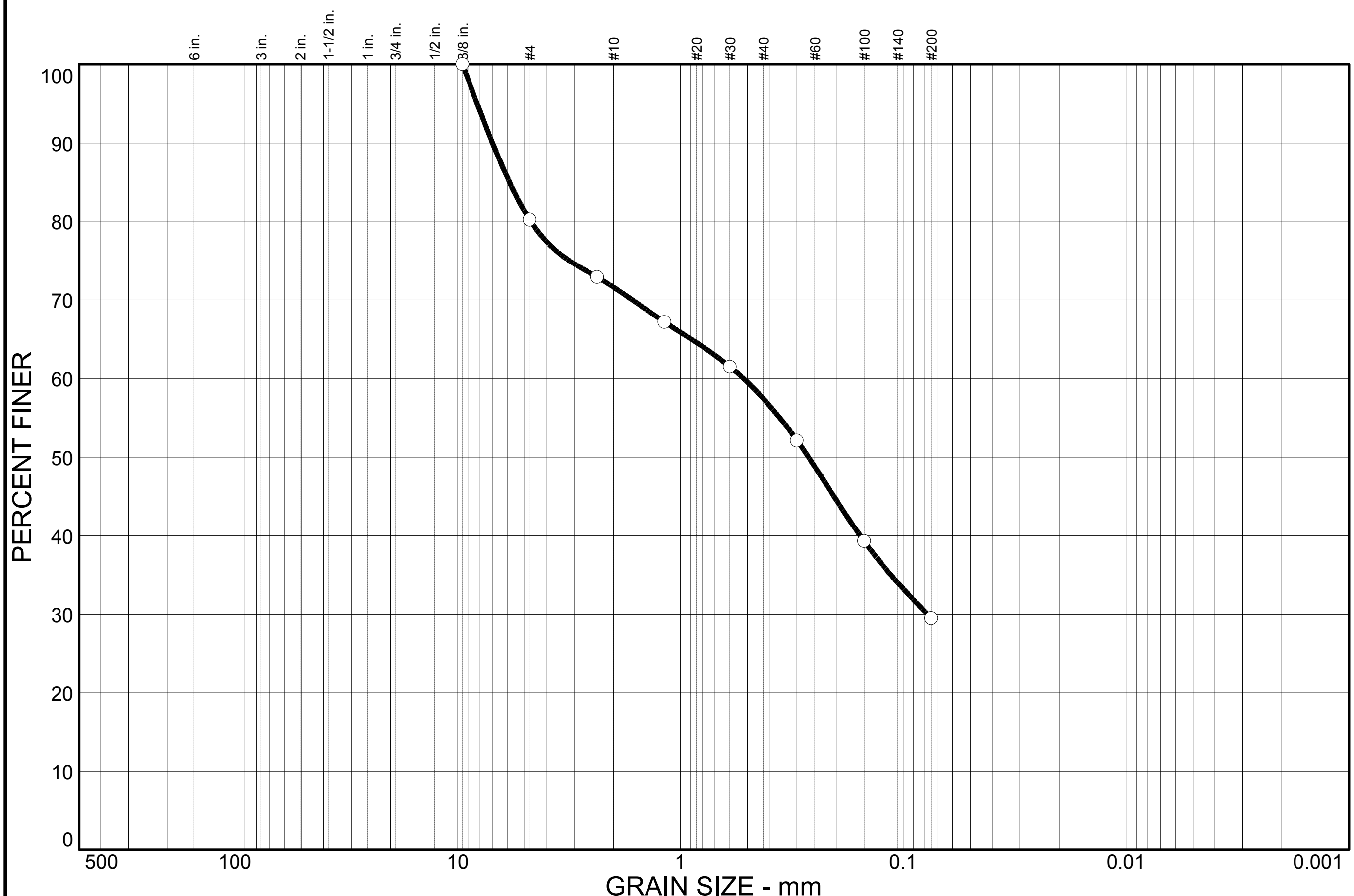
Fresno, CA

Client: Santa Cruz City Schools
Project: SCCS Workforce Housing
Project No: E95815.01 **Figure**

Particle Size Distribution Report



% COBBLES	% GRAVEL		% SAND			% FINES	
	CRS.	FINE	CRS.	MEDIUM	FINE	SILT	CLAY
0.0	0.0	19.8	8.6	14.2	27.9	29.5	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3/8 in.	100.0		
#4	80.2		
#8	72.9		
#16	67.2		
#30	61.5		
#50	52.1		
#100	39.3		
#200	29.5		

* (no specification provided)

Material Description

FILL - Silty, clayey sand with gravel

Atterberg Limits

PL= 17 LL= 24 PI= 7

Coefficients

D_{85}= 5.86 D_{60}= 0.522 D_{50}= 0.267
D_{30}= 0.0780 D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= SC-SM AASHTO=

Remarks

Sample No.: B-5 **Source of Sample:** **Date:** 5/11/23
Location: **Elev./Depth:** 0-1.5'

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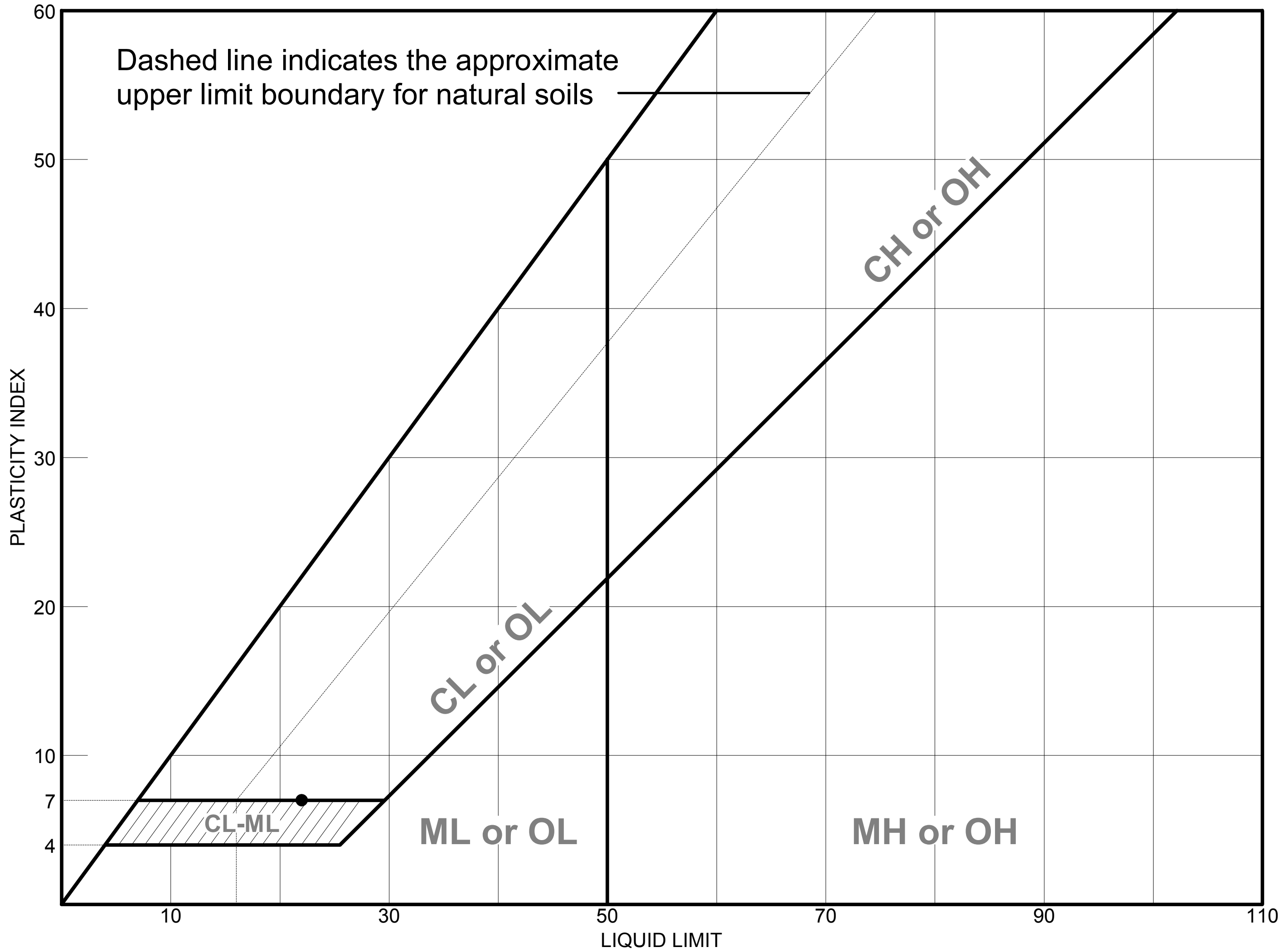
Client: Santa Cruz City Schools
Project: SCCS Workforce Housing

Project No: E95815.01 **Figure**

LIQUID AND PLASTIC LIMITS TEST REPORT



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	Fill - Silty sand and Native - Sandy silty clay	22	15	7			CL-ML

Project No. E95815.01 **Client:** Santa Cruz City Schools

Project: SCCS Workforce Housing

● **Source:** **Sample No.:** B-1 **Elev./Depth:** 2-3.5'

Remarks:

●

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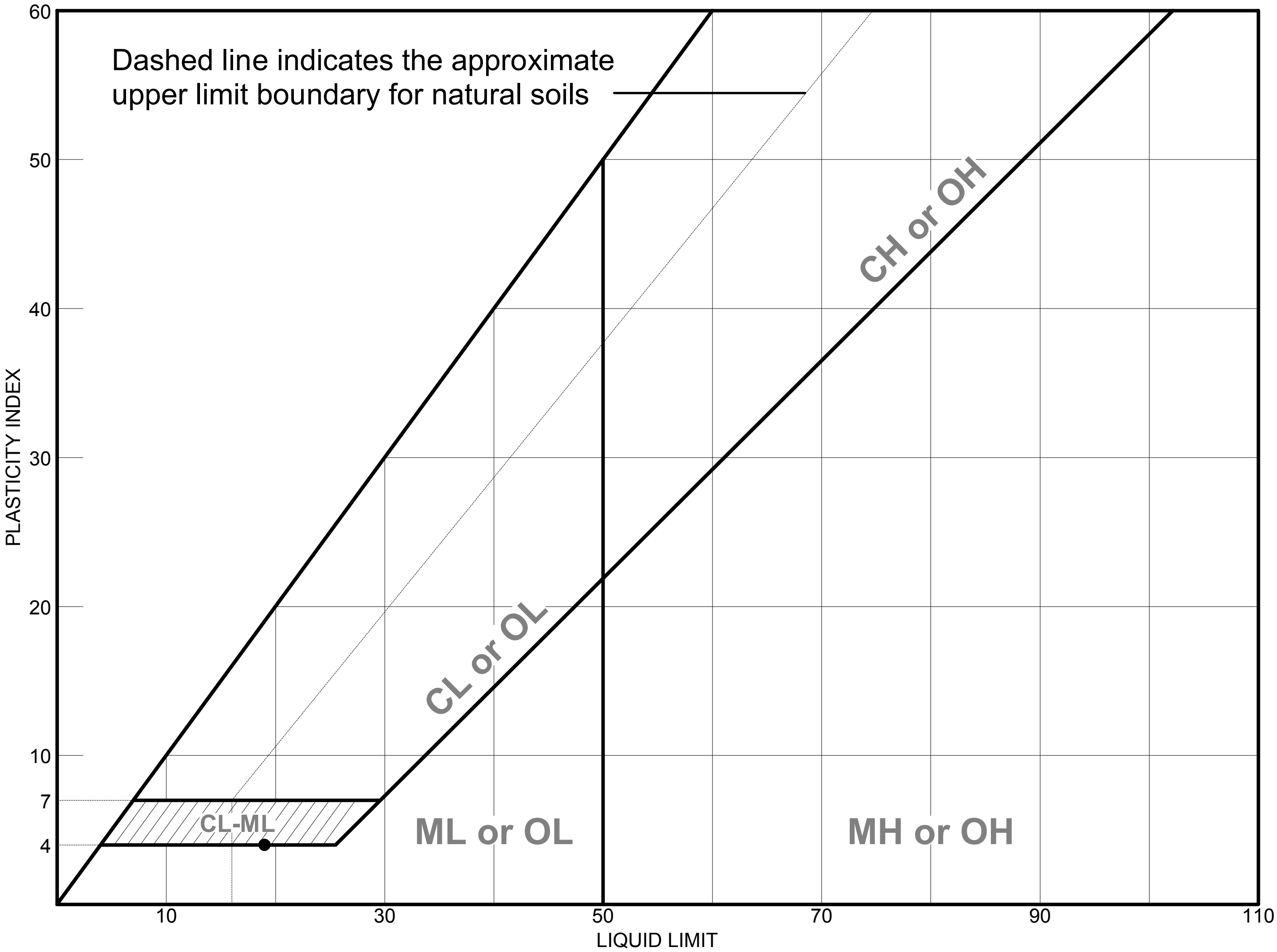
Figure

LIQUID AND PLASTIC LIMITS TEST REPORT



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	Silty, clayey sand	19	15	4	94.5	46.3	SC-SM

Project No. E95815.01 **Client:** Santa Cruz City Schools

Project: SCCS Workforce Housing

● **Source:** **Sample No.:** B-2 **Elev./Depth:** 1-2.5'

Remarks:

●

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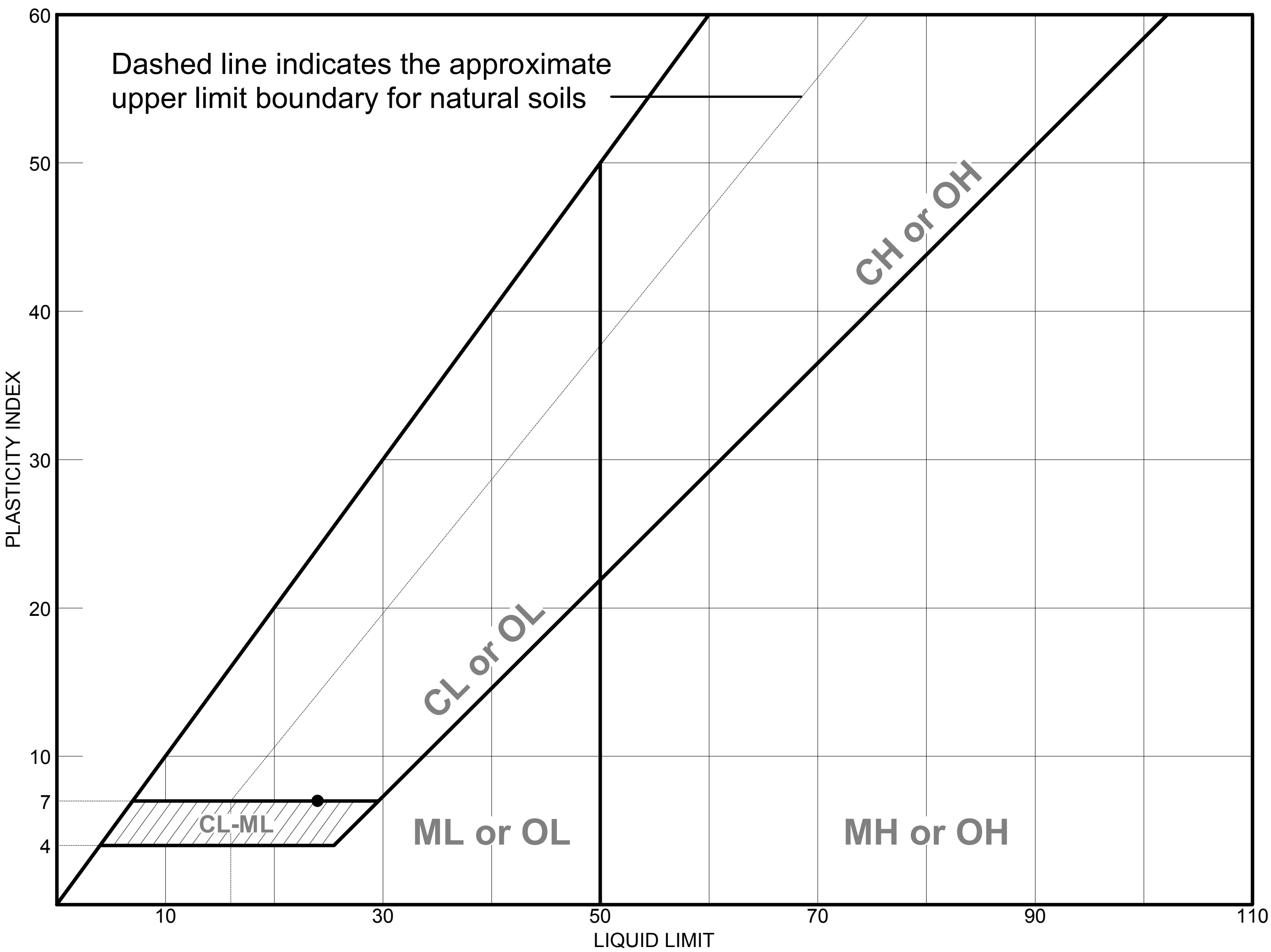
Figure

LIQUID AND PLASTIC LIMITS TEST REPORT



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	FILL - Silty, clayey sand with gravel	24	17	7	57.4	29.5	FILL

Project No. E95815.01 **Client:** Santa Cruz City Schools

Project: SCCS Workforce Housing

● **Source:** **Sample No.:** B-5 **Elev./Depth:** 0-1.5'

Remarks:

●

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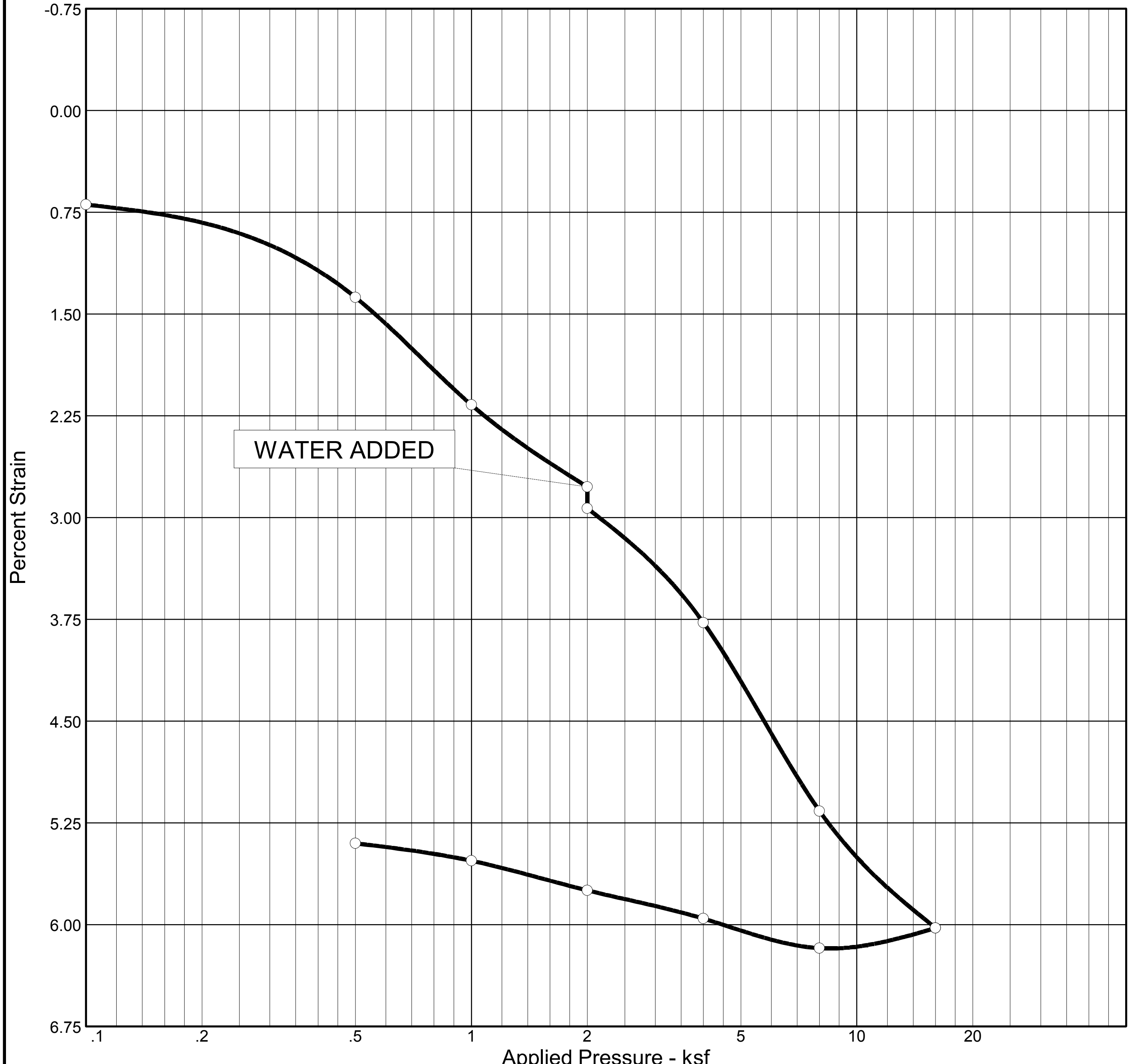
Figure

CONSOLIDATION TEST REPORT



Natural		Dry Dens. (pcf)	LL	PI	Sp. Gr.	Overburden (ksf)	P_c (ksf)	C_c	C_s	Swell Press. (ksf)	Clpse. %	e_o
Sat.	Moist.											
90.0 %	18.1 %	108.0			2.65		0.49	0.04	0.01		0.1	0.532

MATERIAL DESCRIPTION	USCS	AASHTO
Clayey sand	SC	

Project No. E95815.01 **Client:** Santa Cruz City Schools

Project: SCCS Workforce Housing

Source: **Sample No.:** B-1 **Elev./Depth:** 5-6.5'

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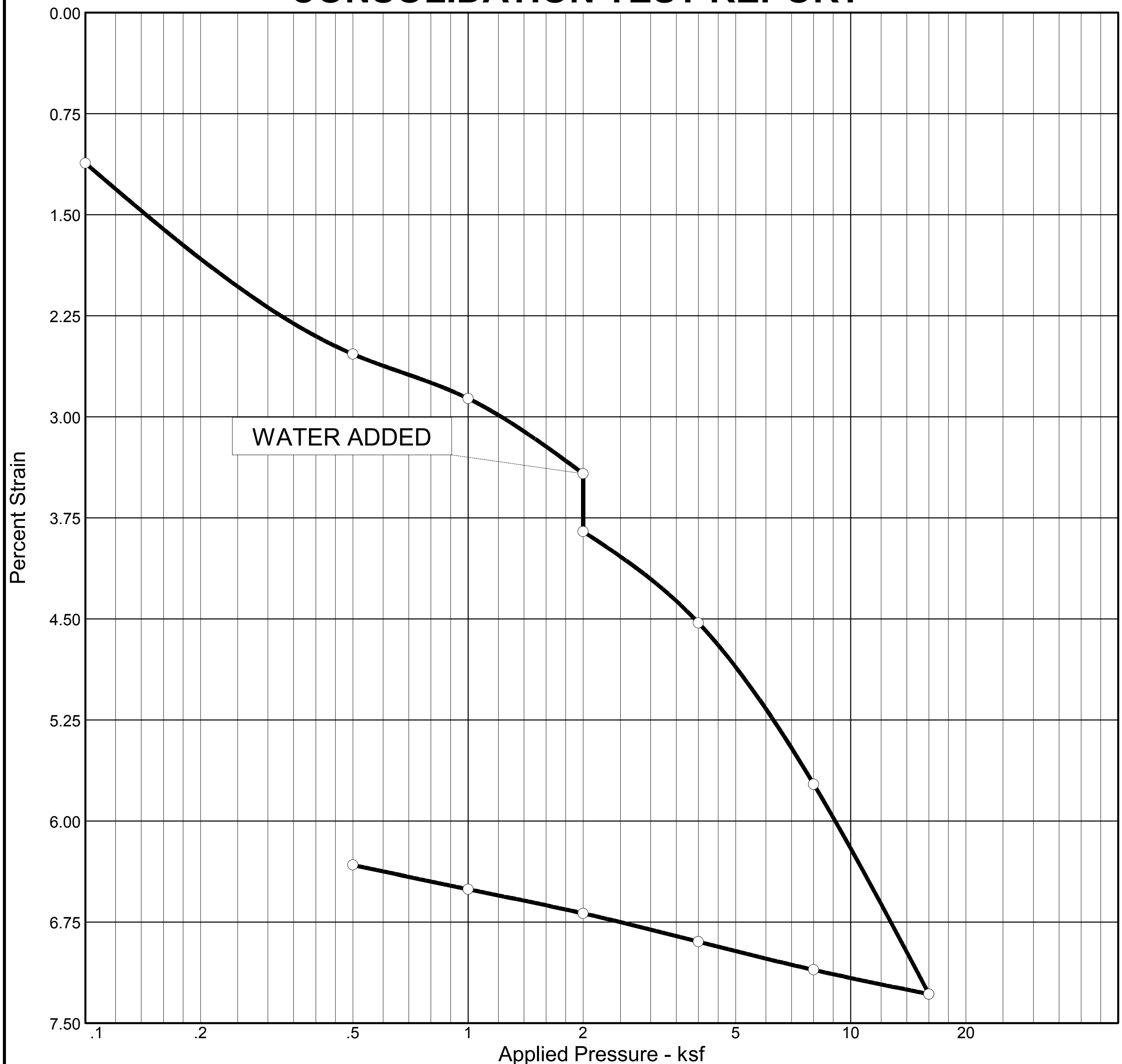
Remarks:

Figure

CONSOLIDATION TEST REPORT



Natural		Dry Dens. (pcf)	LL	PI	Sp. Gr.	Overburden (ksf)	P_c (ksf)	C_c	C_s	Swell Press. (ksf)	Clpse. %	e_o
Sat.	Moist.											
86.2 %	17.3 %	108.1			2.65		5.14	0.08	0.01		0.5	0.531

MATERIAL DESCRIPTION	USCS	AASHTO
Sandy lean clay	CL	

Project No. E95815.01 **Client:** Santa Cruz City Schools

Project: SCCS Workforce Housing

Source: **Sample No.:** B-3 **Elev./Depth:** 3.5-5'

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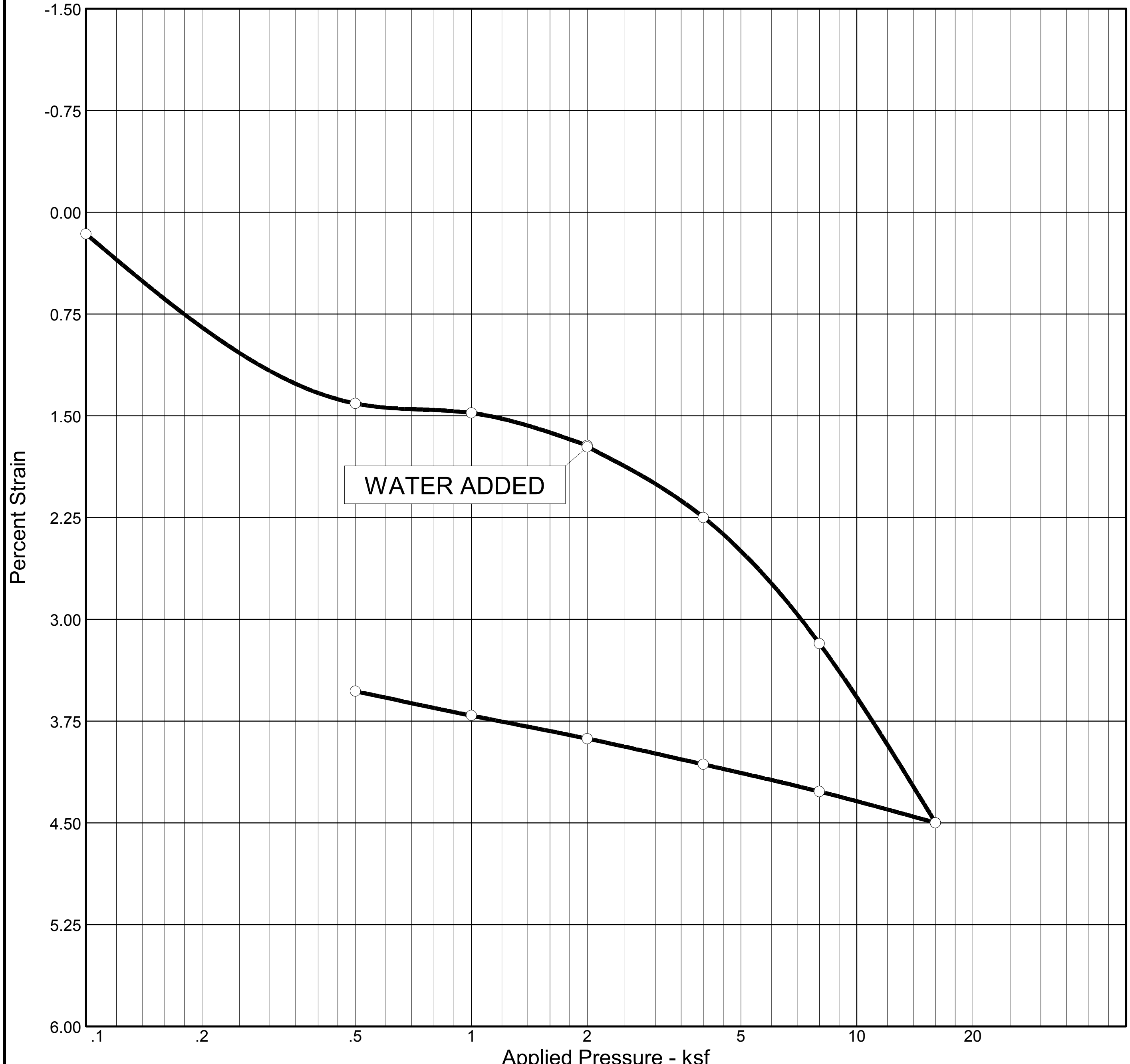
Remarks:

Figure

CONSOLIDATION TEST REPORT



Natural		Dry Dens. (pcf)	LL	PI	Sp. Gr.	Overburden (ksf)	P_c (ksf)	C_c	C_s	Swell Press. (ksf)	Swell %	e_o
Sat.	Moist.											
105.8 %	28.6 %	96.3			2.65		3.70	0.08	0.01			0.718

MATERIAL DESCRIPTION	USCS	AASHTO
Silty sand	SM	

Project No. E95815.01 **Client:** Santa Cruz City Schools

Project: SCCS Workforce Housing

Source: **Sample No.:** B-3 **Elev./Depth:** 10-11.5'

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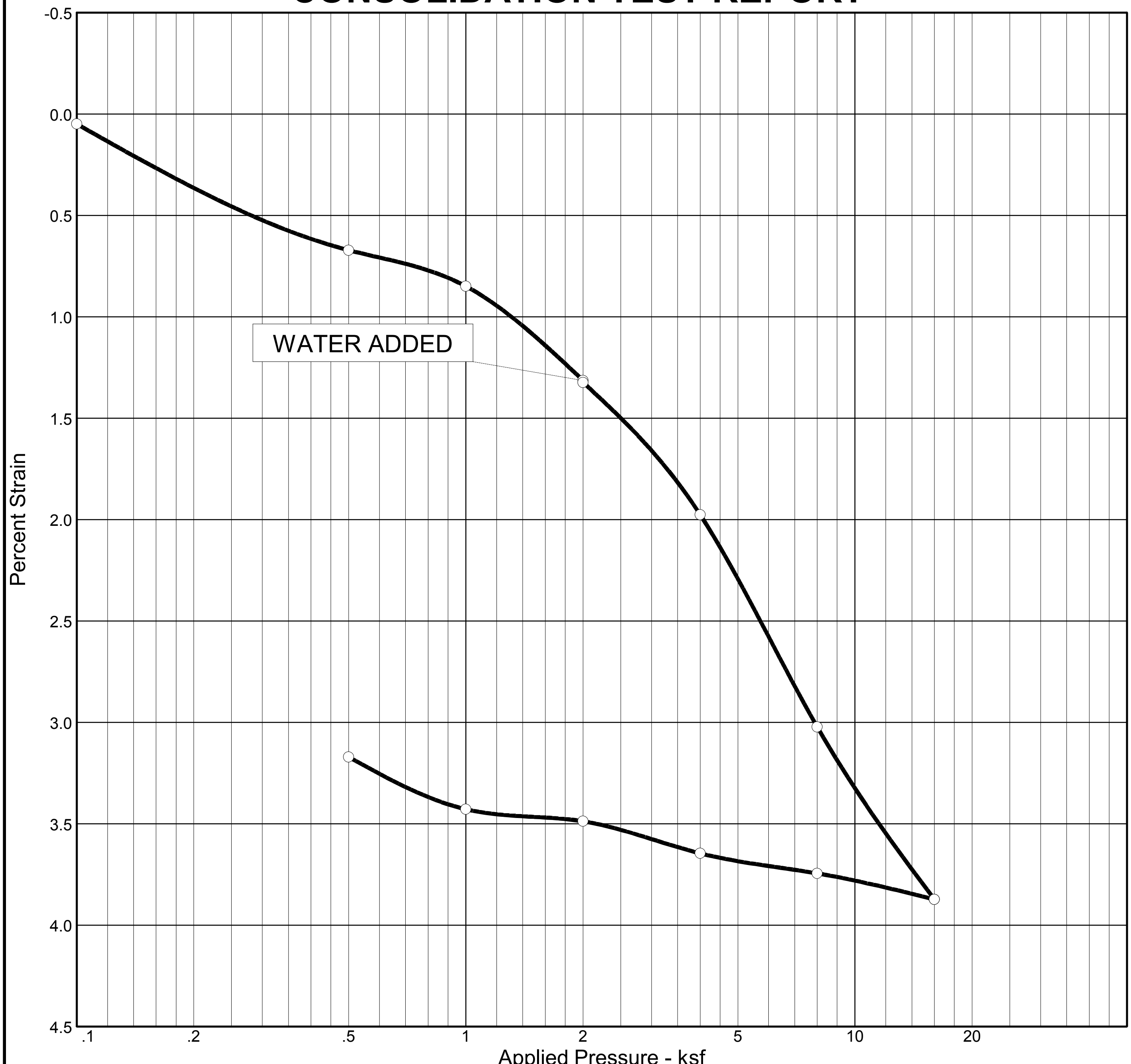
Remarks:

Figure

CONSOLIDATION TEST REPORT



Natural		Dry Dens. (pcf)	LL	PI	Sp. Gr.	Overburden (ksf)	P_c (ksf)	C_c	C_s	Swell Press. (ksf)	Swell %	e_o
Sat.	Moist.											
97.2 %	19.7 %	107.7			2.65		1.33	0.04	0.01			0.537

MATERIAL DESCRIPTION	USCS	AASHTO
Poorly graded sand	SP	

Project No. E95815.01 **Client:** Santa Cruz City Schools

Project: SCCS Workforce Housing

Source: **Sample No.:** B-5 **Elev./Depth:** 3.5-5'

Remarks:

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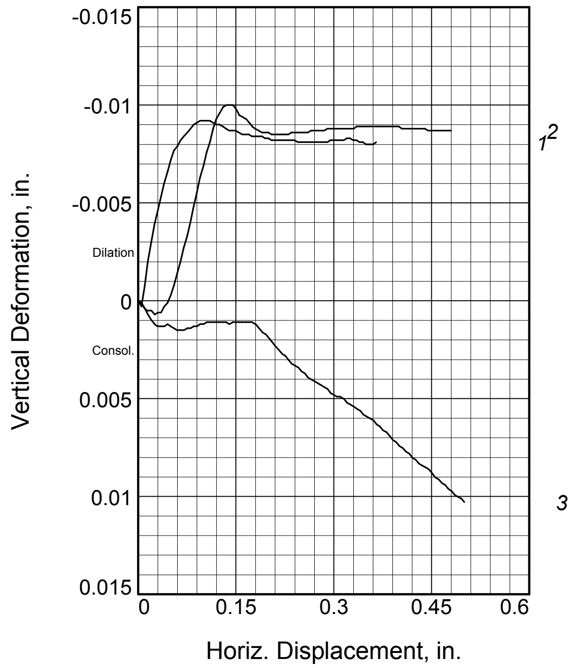
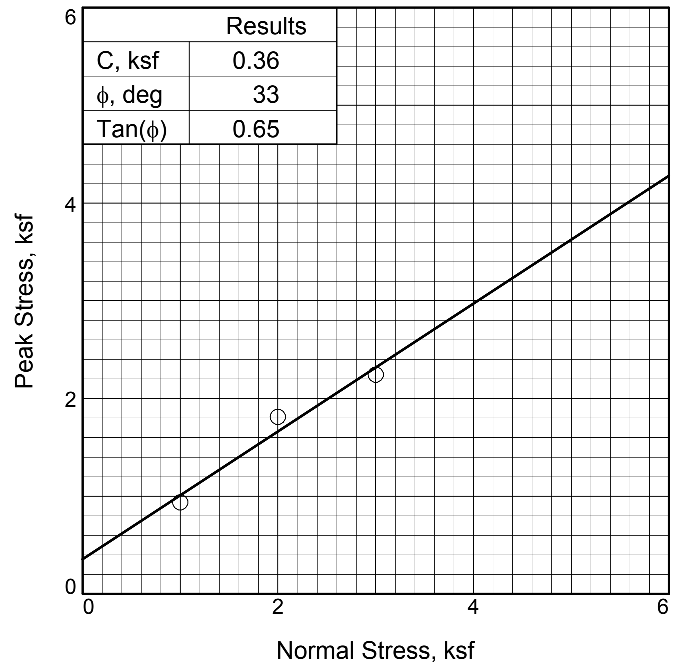
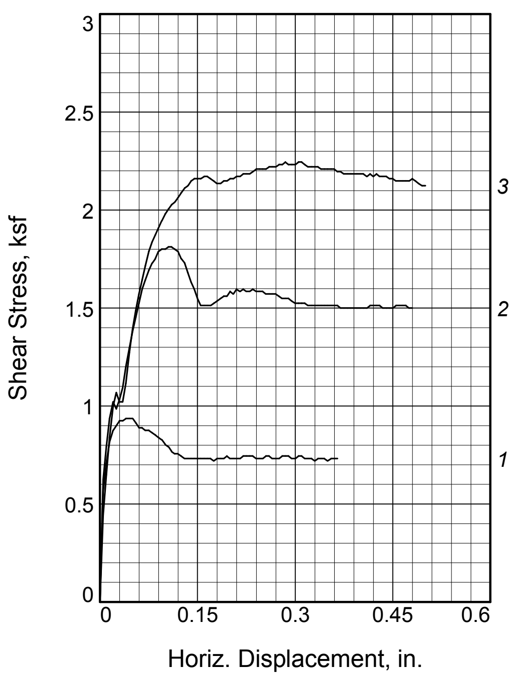
Figure







	Sample No.	1	2	3
Initial	Water Content, %	17.8	16.2	18.1
	Dry Density, pcf	111.4	115.5	109.3
	Saturation, %	96.9	99.2	93.2
	Void Ratio	0.4857	0.4322	0.5134
	Diameter, in.	2.42	2.42	2.42
	Height, in.	1.00	1.00	1.00
At Test	Water Content, %	17.7	16.3	19.2
	Dry Density, pcf	111.5	115.7	109.5
	Saturation, %	96.9	100.7	99.4
	Void Ratio	0.4835	0.4299	0.5107
	Diameter, in.	2.42	2.42	2.42
	Height, in.	1.00	1.00	1.00
	Normal Stress, ksf	1.00	2.00	3.00
	Peak Stress, ksf	0.94	1.81	2.24
	Displacement, in.	0.04	0.11	0.28
	Ultimate Stress, ksf			
	Displacement, in.			
	Strain at peak, %	1.7	4.3	11.8

Sample Type: Sandy lean clay

Description:

Specific Gravity= 2.65

Remarks:

Figure ________

Client: Santa Cruz City Schools

Project: SCCS Workforce Housing

Sample Number: B-3 **Depth:** 3.5-5'

Proj. No.: E95815.01 **Date Sampled:** May □□, □□□□

DIRECT SHEAR TEST REPORT

Moore Twining Associates, Inc.

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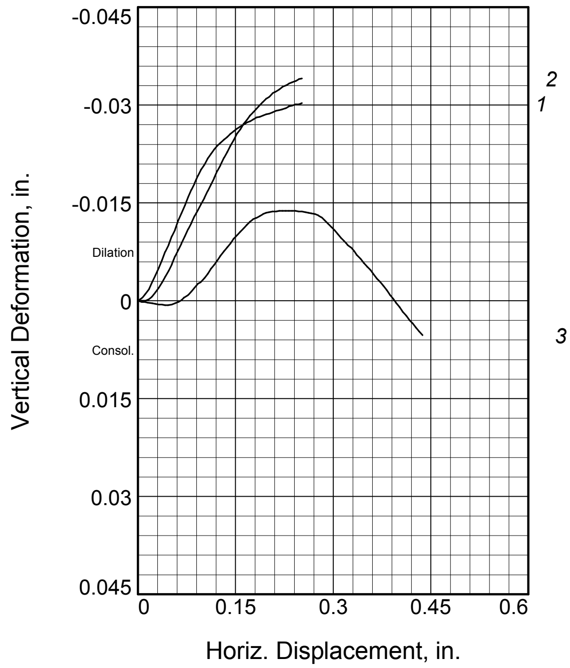
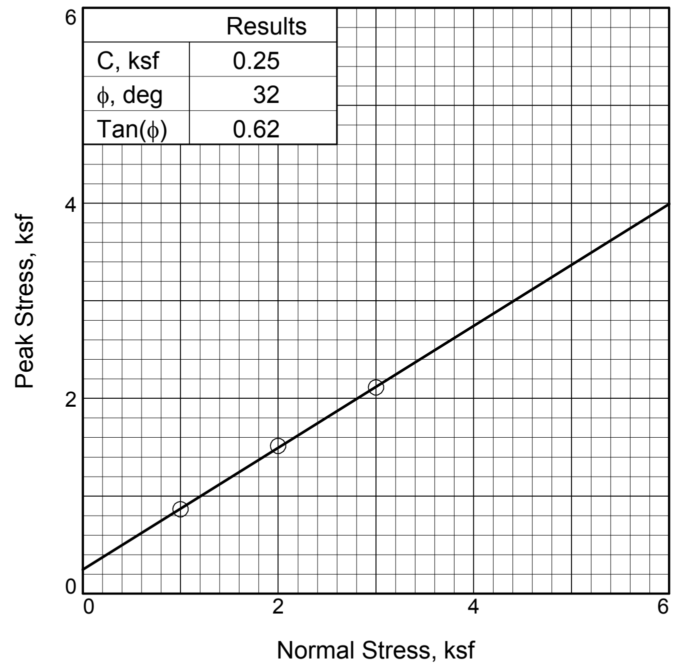
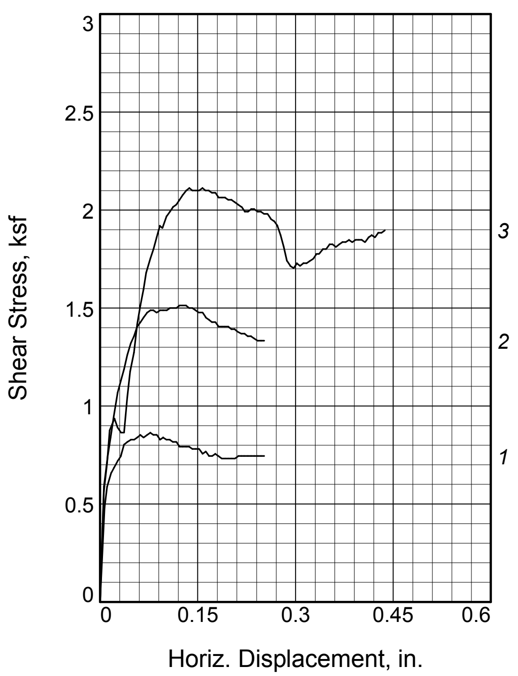




Results	
C, ksf	0.25
ϕ, deg	32
Tan(ϕ)	0.62



	Sample No.	1	2	3
Initial	Water Content, %	30.6	33.9	33.5
	Dry Density, pcf	94.8	90.8	90.3
	Saturation, %	108.8	109.5	106.7
	Void Ratio	0.7448	0.8214	0.8311
	Diameter, in.	2.42	2.42	2.42
	Height, in.	1.00	1.00	1.00
At Test	Water Content, %	28.5	29.8	29.9
	Dry Density, pcf	95.0	91.0	90.6
	Saturation, %	101.7	96.6	96.1
	Void Ratio	0.7422	0.8181	0.8257
	Diameter, in.	2.42	2.42	2.42
	Height, in.	1.00	1.00	1.00
	Normal Stress, ksf	1.00	2.00	3.00
	Peak Stress, ksf	0.86	1.51	2.11
	Displacement, in.	0.08	0.12	0.14
	Ultimate Stress, ksf			
	Displacement, in.			
	Strain at peak, %	3.2	5.0	5.7

Sample Type: SP-SM

Description: Poorly graded sand with silt

Specific Gravity= 2.65

Remarks:

Figure ________

Client: Santa Cruz City Schools

Project: SCCS Workforce Housing

Sample Number: B-5 **Depth:** 10-11.5'

Proj. No.: E95815.01 **Date Sampled:** 5/11/23

DIRECT SHEAR TEST REPORT

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Fresno, CA

MOORE TWINING ASSOCIATES, INC.

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EXPANSION INDEX TEST, ASTM D4829

MTA PROJECT NAME:	SCCS Workforce Housing	REPORT DATE:	7/1/2023
		TEST DATE:	6/15/2023
MTA PROJECT NO.:	E95815.01		
SAMPLE I.D.:	B-2 @ 0-5'		
SAMPLED BY:	AV		
SAMPLE DATE:	5/11/2023	TESTED BY:	CJ

MATERIALS DESCRIPTION: Silty clayey sand

% PASSING # 4 SIEVE 100

Initial Moisture Determination:		Final Moisture Determination:	
Pan + Wet Soil Wt., gm	250.0	Wet Soil Wt., lbs	0.9665
Pan + Dry Soil Wt., gm	229.2	Dry Soil Wt., lbs	0.8202
Pan Wt., gm	0.0		
Initial % Moisture Content	9.1	Final % Moisture Content	17.8

Initial Expansion Data:		**Final Expansion Data:**	
Ring + Sample Wt., lbs	0.8946	Ring + Sample Wt., lbs	0.9665
Ring Wt., lbs	0.0000	Ring Wt., lbs	0.0000
Remolded Wt., lbs	0.8946	Remolded Wt., lbs	0.9665
Remolded Wet Density, pcf	123.0	Remolded Wet Density, pcf	130.5
Remolded Dry Density, pcf	112.8	Remolded Dry Density, pcf	110.8

Expansion Data:		Initial Volume	Final Volume
		0.00727222	0.007404
Initial Gage Reading, in:	0.2480		
Final Gage Reading, in:	0.2661		
Expansion, in:	0.0181		
Expansion Index	18	**Comments:**	**Very Low Expansion Potential**

Classification of Expansive Soils. (Table No.1 From ASTM D4829)

Expansion Index	Potential Expansion
0-20	Very Low
21-50	Low
51-90	Medium
91-130	High
>130	Very High

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EXPANSION INDEX TEST, ASTM D4829

MTA PROJECT NAME:	SCCS Workforce Housing	REPORT DATE:	7/1/2023
		TEST DATE:	6/3/2023
MTA PROJECT NO.:	E95815.01		
SAMPLE I.D.:	B-4 @ 0-5'		
SAMPLED BY:	AV		
SAMPLE DATE:	5/11/2023	TESTED BY:	CJ

MATERIALS DESCRIPTION: Fill - Sandy lean clay to Native - Clayey sand with gravel

% PASSING # 4 SIEVE 100

Initial Moisture Determination:		Final Moisture Determination:	
Pan + Wet Soil Wt., gm	250.0	Wet Soil Wt., lbs	0.8862
Pan + Dry Soil Wt., gm	219.3	Dry Soil Wt., lbs	0.7147
Pan Wt., gm	0.0		
Initial % Moisture Content	14.0	Final % Moisture Content	24.0

Initial Expansion Data:		**Final Expansion Data:**	
Ring + Sample Wt., lbs	0.8148	Ring + Sample Wt., lbs	0.8862
Ring Wt., lbs	0.0000	Ring Wt., lbs	0.0000
Remolded Wt., lbs	0.8148	Remolded Wt., lbs	0.8862
Remolded Wet Density, pcf	112.0	Remolded Wet Density, pcf	121.2
Remolded Dry Density, pcf	98.3	Remolded Dry Density, pcf	97.8

Expansion Data:		Initial Volume	Final Volume
		0.00727222	0.00731
Initial Gage Reading, in:	0.0266		
Final Gage Reading, in:	0.0318		
Expansion, in:	0.0052		
Expansion Index	5	**Comments:**	**Very Low Expansion Potential**

Classification of Expansive Soils. (Table No.1 From ASTM D4829)

Expansion Index	Potential Expansion
0-20	Very Low
21-50	Low
51-90	Medium
91-130	High
>130	Very High

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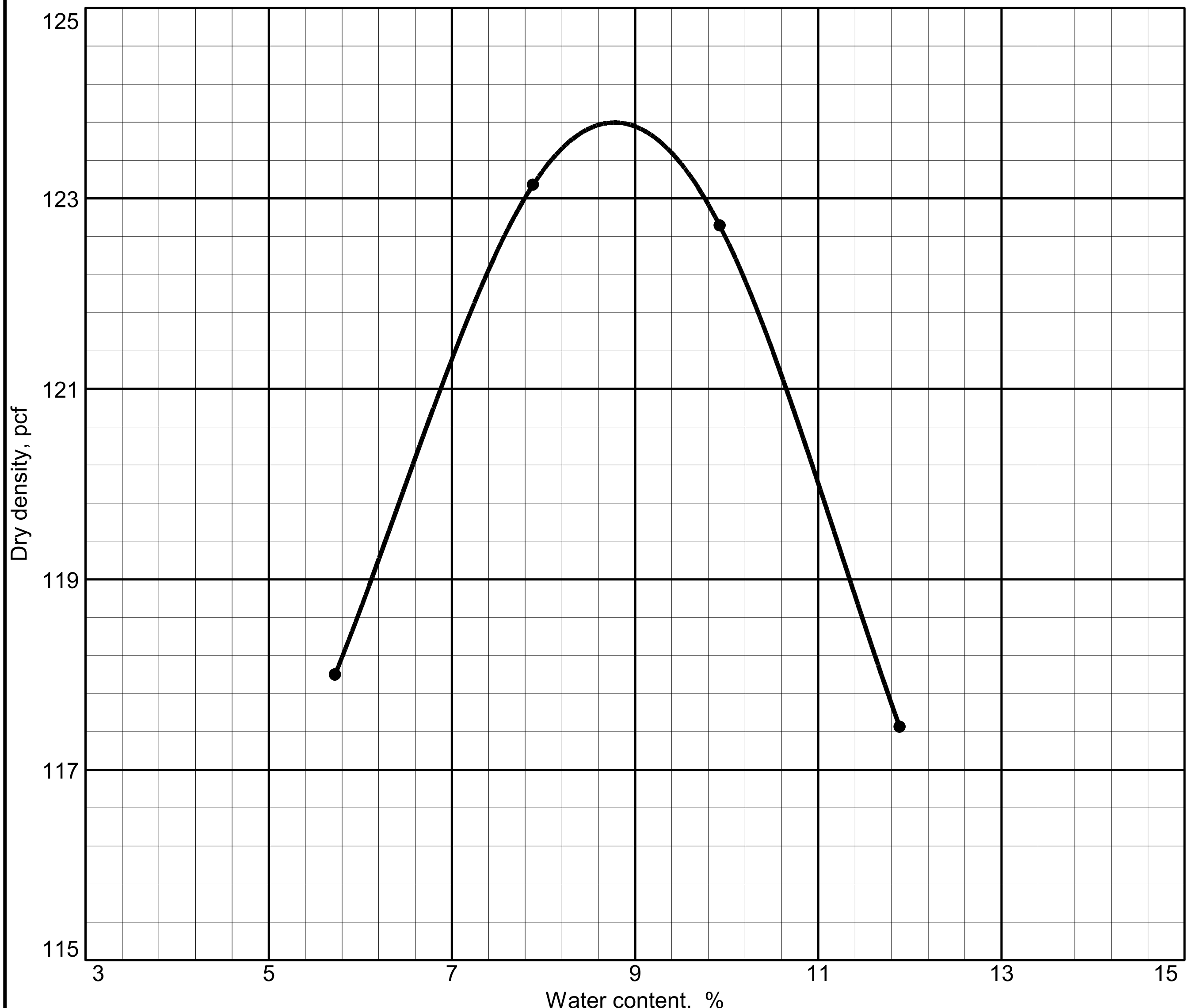
COMPACTION TEST REPORT



Test specification: ASTM D 1557-12 Method B Modified

Elev/ Depth	Classification		Nat. Moist.	Sp.G.	LL	PI	% > 3/8 in.	% < No.200
	USCS	AASHTO						
0-5'	Fill to CL-ML							

TEST RESULTS	MATERIAL DESCRIPTION
Maximum dry density = 123.8 pcf Optimum moisture = 8.8 %	Fill - Silty sand to Native - Sandy silty clay

Project No. E95815.01 **Client:** Santa Cruz City Schools

Project: SCCS Workforce Housing

● **Source:** **Sample No.:** B-1 **Elev./Depth:** 0-5'

Remarks:

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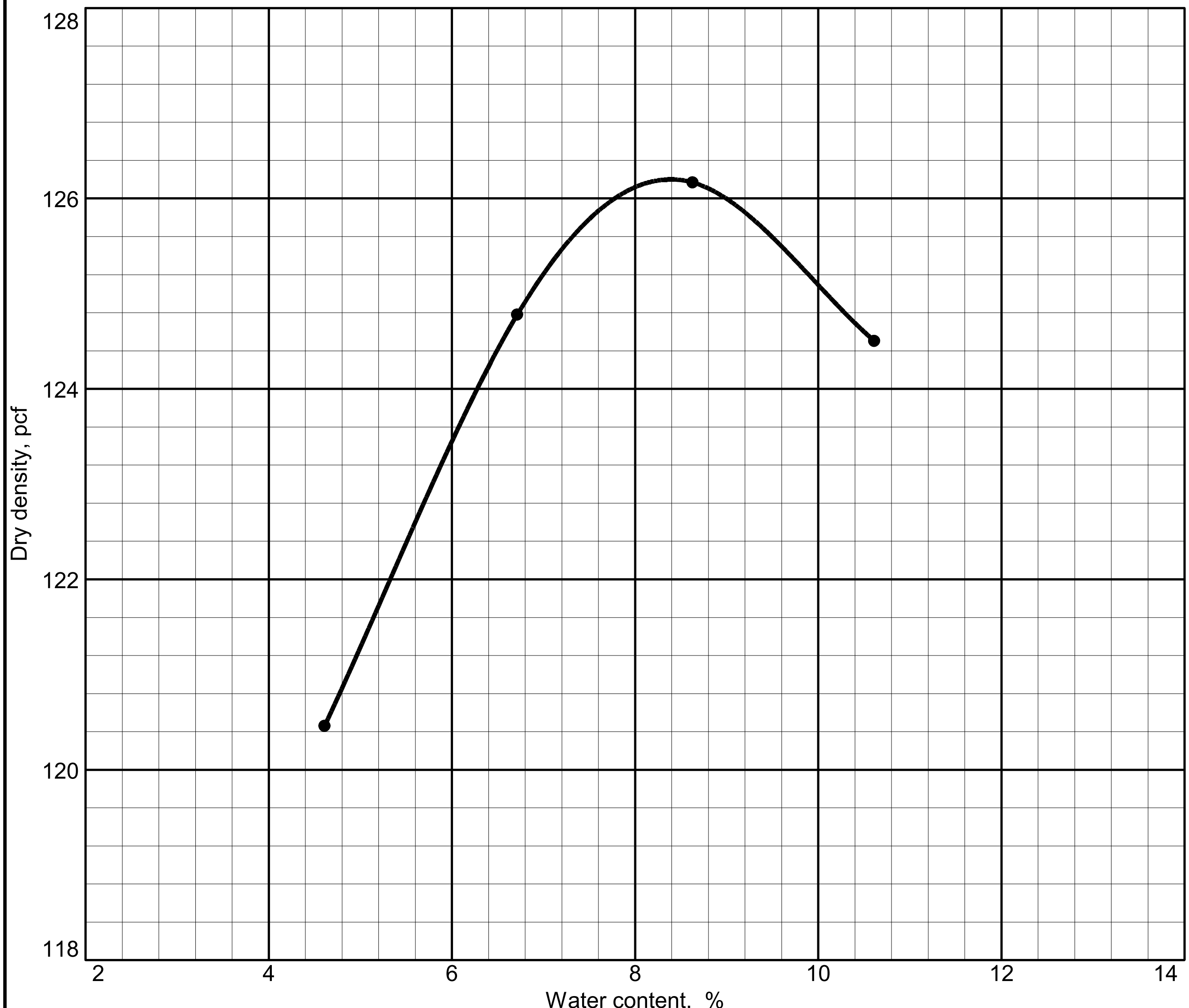
Figure

COMPACTION TEST REPORT



Test specification: ASTM D 1557-12 Method B Modified

Elev/ Depth	Classification		Nat. Moist.	Sp.G.	LL	PI	% > 3/8 in.	% < No.200
	USCS	AASHTO						
0-3.5'	Fill							

TEST RESULTS	MATERIAL DESCRIPTION
Maximum dry density = 126.2 pcf Optimum moisture = 8.4 %	Fill - Silty, clayey sand with gravel

Project No. E95815.01 **Client:** Santa Cruz City Schools

Project: SCCS Workforce Housing

● **Source:** **Sample No.:** B-5 **Elev./Depth:** 0-3.5'

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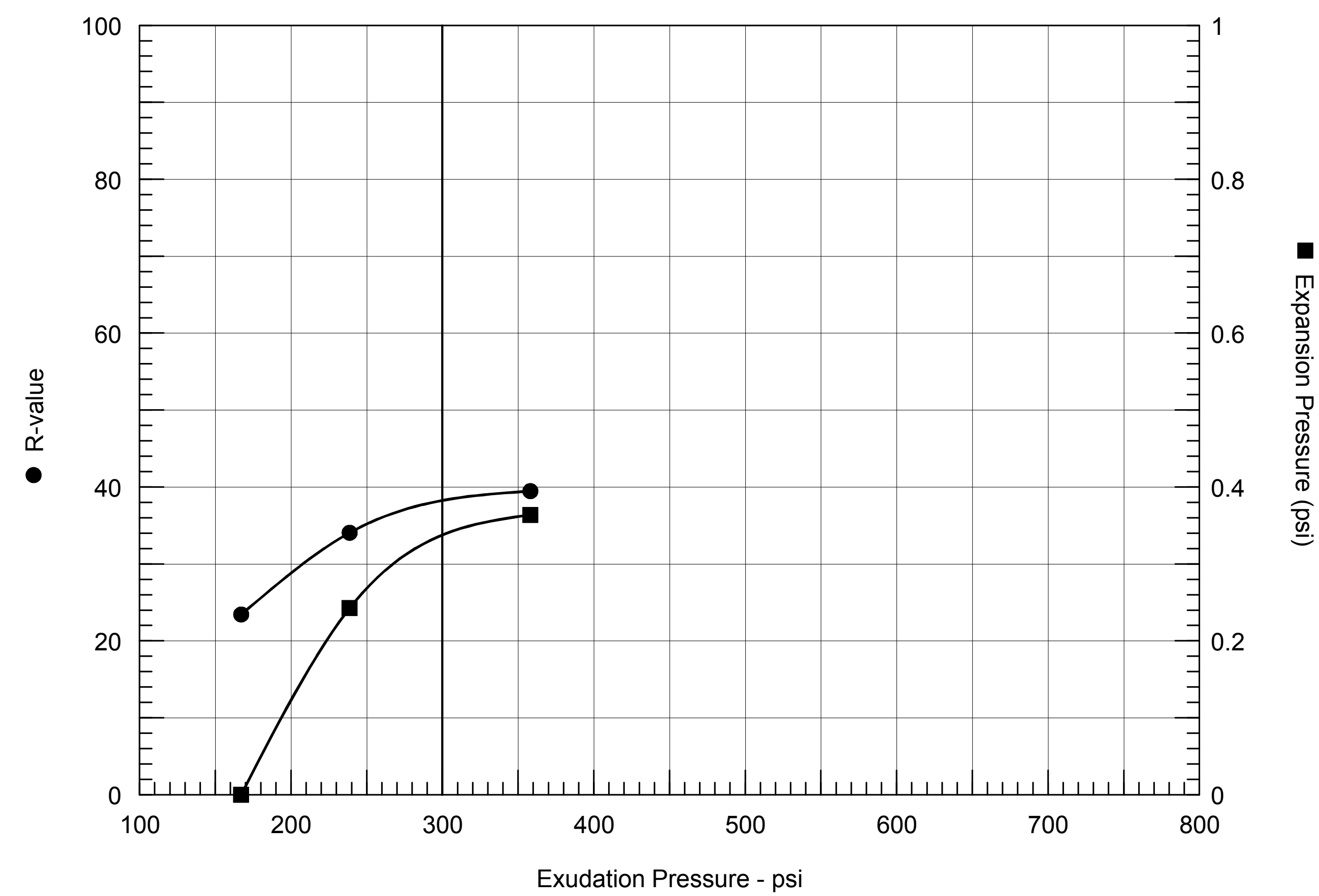
Remarks:

Figure

R-VALUE TEST REPORT



Resistance R-Value and Expansion Pressure - ASTM D 2844

No.	Compact. Pressure psi	Density pcf	Moist. %	Expansion Pressure psi	Horizontal Press. psi @ 160 psi	Sample Height in.	Exud. Pressure psi	R Value	R Value Corr.
1	150	122.9	12.0	0.24	82	2.41	239	36	34
2	250	126.1	11.0	0.36	71	2.35	358	43	39
3	100	120.7	13.0	0.00	104	2.46	167	23	23

Test Results	Material Description
R-value at 300 psi exudation pressure = 38 **Exp. pressure at 300 psi exudation pressure =** 0.34 psi	Fill - Silty sand to Native - Sandy lean clay

Project No.: E95815.01

Project:SCCS Workforce Housing

Sample Number: B-6 **Depth:** 0-5'

Date: 7/11/2023

R-VALUE TEST REPORT

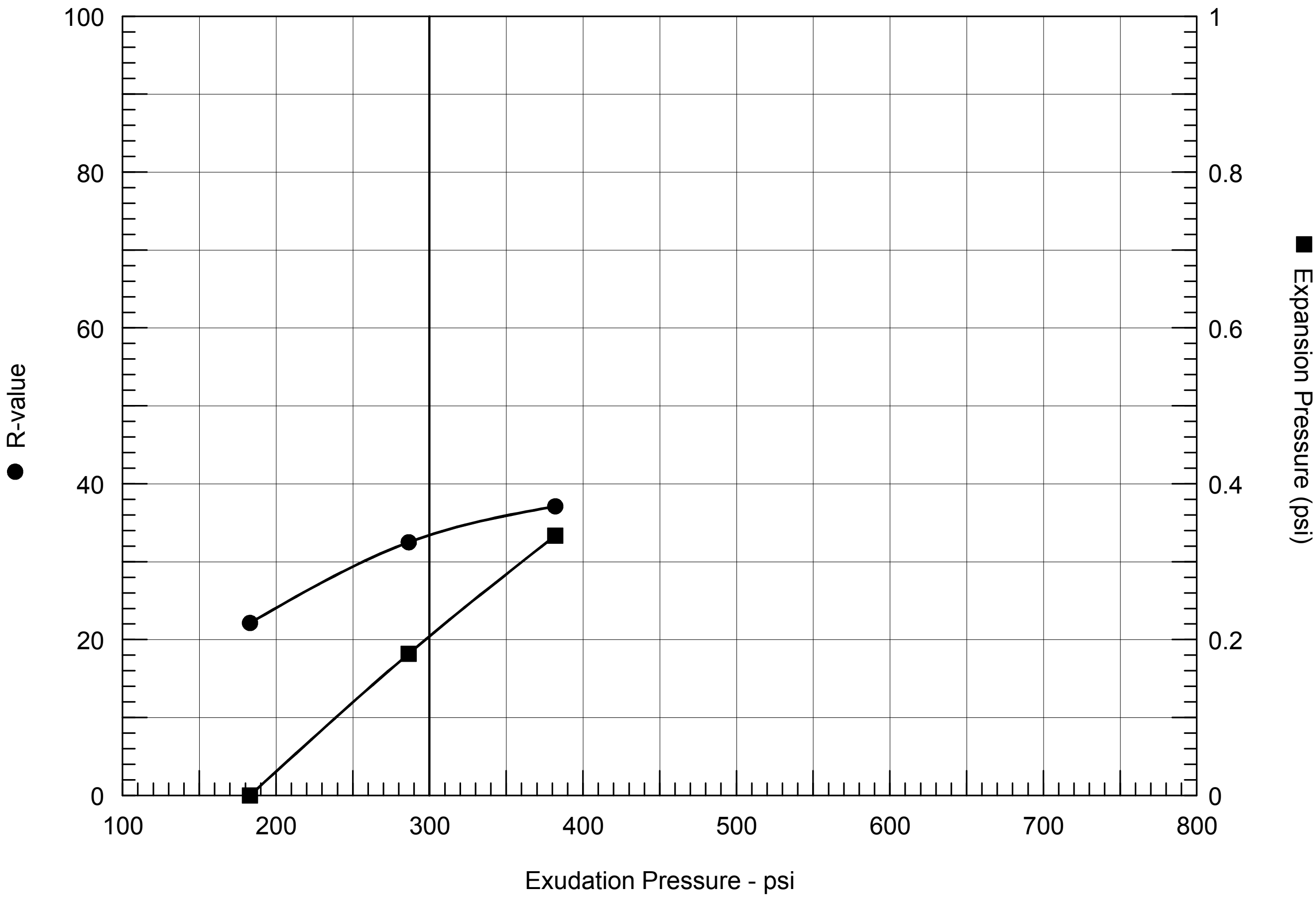
Moore Twining Associates, Inc.

Tested by: MS
Checked by: MS

Remarks:

Figure N/A

R-VALUE TEST REPORT



Resistance R-Value and Expansion Pressure - ASTM D 2844

No.	Compact. Pressure psi	Density pcf	Moist. %	Expansion Pressure psi	Horizontal Press. psi @ 160 psi	Sample Height in.	Exud. Pressure psi	R Value	R Value Corr.
1	100	119.8	15.0	0.00	106	2.52	183	22	22
2	150	122.8	13.9	0.18	88	2.46	286	32	32
3	250	124.7	12.9	0.33	78	2.42	382	39	37

Test Results	Material Description
R-value at 300 psi exudation pressure = 33 **Exp. pressure at 300 psi exudation pressure =** 0.20 psi	Sandy lean clay

Project No.: E95815.01

Project:SCCS Workforce Housing

Sample Number: B-7 **Depth:** 0-5'

Date: 7/11/2023

R-VALUE TEST REPORT

Moore Twining Associates, Inc.

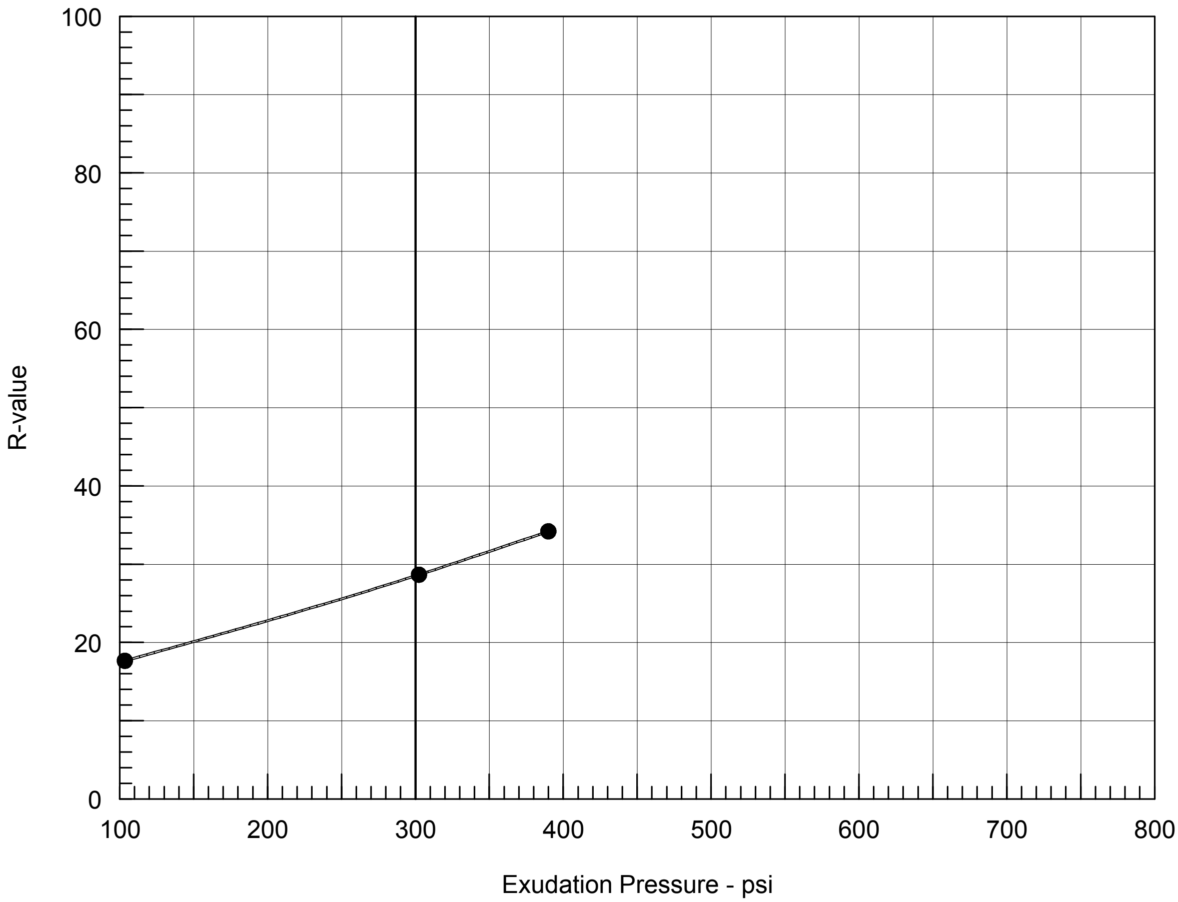
Tested by:

Checked by:

Remarks:

Figure ______

R-VALUE TEST REPORT



Resistance R-Value and Expansion Pressure - ASTM D 2844

No.	Compact. Pressure psi	Density pcf	Moist. %	Expansion Pressure psi	Horizontal Press. psi @ 160 psi	Sample Height in.	Exud. Pressure psi	R Value	R Value Corr.
1	30	111.4	16.5	0.00	120	2.60	103	17	18
2	250	117.4	14.2	0.00	96	2.47	302	29	29
3	300	119.5	13.2	0.00	84	2.42	390	36	34

Test Results	Material Description
R-value at 300 psi exudation pressure = 29	Sandy lean clay

Project No.: E95815.01

Project:SCCS Workforce Housing

Sample Number: B-8 **Depth:** 0-5'

Date: 7/11/2023

Tested by: MS

Checked by: MS

Remarks:

R-VALUE TEST REPORT

Moore Twining Associates, Inc.

Figure N/A

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Since 1898

Project Name:	SCCS Workforce Housing	Report Date:	7/1/2023
		Sample Date:	5/11/2023
Project Number:	E95815.01		
		Sampled By:	AV
Subject:	Minimum Resistivity, ASTM G187	Tested By:	RS
Material Description:	Fill - Silty sand to Native - Sandy silty clay	Test Date:	5/23/2023
Location:	B-1 @ 0-5'		

Laboratory Test Results, Minimum Resistivity - ASTM G187

Total Water Added, mls	Resistivity, Ohm-cm
75 mls	54,000
100 mls	38,000
125 mls	28,000
150 mls	21,000
175 mls	9,100
200 mls	8,900
225 mls	9,000

Remarks: Min. Resistivity is 8,900 Ohm-cm

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Since 1898

Project Name:	SCCS Workforce Housing	Report Date:	7/1/2023
		Sample Date:	5/11/2023
Project Number:	E95815.01		
		Sampled By:	AV
Subject:	Minimum Resistivity, ASTM G187	Tested By:	RS

Material Description: Fill - Sandy lean clay and Native - Clayey sand with gravel

Location: B-4 @ 0-5' Test Date: 6/1/2023

Laboratory Test Results, Minimum Resistivity - ASTM G187

Total Water Added, mls	Resistivity, Ohm-cm
100 mls	53,000
125 mls	37,000
150 mls	23,000
175 mls	10,800
200 mls	7,700
225 mls	5,900
250 mls	4,600
275 mls	4,500
300 mls	4,400
325 mls	4,300
350 mls	4,400

Remarks: Min. Resistivity is 4,300 Ohm-cm

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MOORE TWINING
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California ELAP Certificate #1371

2527 Fresno Street
Fresno, CA 93721
(559) 268-7021 Phone
(559) 268-0740 Fax

June 06, 2023

Work Order #: **JE22017**

Shaun Reich
MTA Geotechnical Division
2527 Fresno Street
Fresno, CA 93721

RE: SCCS Workforce Housing

Enclosed are the analytical results for samples received by our laboratory on **05/22/23** . For your reference, these analyses have been assigned laboratory work order number **JE22017**.

All analyses have been performed according to our laboratory's quality assurance program. All results are intended to be considered in their entirety, Moore Twining Associates, Inc. (MTA) is not responsible for use of less than complete reports. Results apply only to samples analyzed.

If you have any questions, please feel free to contact us at the number listed above.

Sincerely,

Moore Twining Associates, Inc.

Susan Federico
Client Services Representative

MOORE TWINING ASSOCIATES, INC.
California ELAP Certificate #1371

2527 Fresno Street
Fresno, CA 93721
(559) 268-7021 Phone
(559) 268-0740 Fax

MTA Geotechnical Division 2527 Fresno Street Fresno CA, 93721	**Project:** SCCS Workforce Housing **Project Number:** E95815.01 **Project Manager:** Shaun Reich	**Reported:** 06/06/2023

Analytical Report for the Following Samples

Sample ID	Notes	Laboratory ID	Matrix	Date Sampled	Date Received
B-1 @ 0'-5'		JE22017-01	Soil	05/11/23 00:00	05/22/23 08:44

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MTA Geotechnical Division 2527 Fresno Street Fresno CA, 93721	**Project:** SCCS Workforce Housing **Project Number:** E95815.01 **Project Manager:** Shaun Reich	**Reported:** 06/06/2023

B-1 @ 0'-5'

JE22017-01 (Soil) Sampled: 05/11/23 00:00

Analyte	Flag	Result	Reporting Limit	Units	Dilution	Batch	Prepared	Analyzed	Method
Inorganics									
Chloride		ND	40	mg/kg	20	B3F0212	06/05/23	06/06/23	Cal Test 422
Chloride		ND	0.0040	% by Weight	20	[CALC]	06/06/23	06/06/23	[CALC]
Sulfate as SO4		ND	0.0040	% by Weight	20	[CALC]	06/06/23	06/06/23	[CALC]
pH		**6.8**	0.10	pH Units	1	B3F0212	06/05/23	06/06/23	Cal Test 643
Sulfate as SO4		ND	40	mg/kg	20	B3F0212	06/05/23	06/06/23	Cal Test 417

Notes and Definitions

µg/L	micrograms per liter (parts per billion concentration units)
mg/L	milligrams per liter (parts per million concentration units)
mg/kg	milligrams per kilogram (parts per million concentration units)
ND	Analyte NOT DETECTED at or above the reporting limit
RPD	Relative Percent Difference
	Analysis of pH, filtration, and residual chlorine is to take place immediately after sampling in the field. If the test was performed in the laboratory, the hold time was exceeded. **(for aqueous matrices only)**

MOORE TWINING ASSOCIATES, INC.

CHAIN OF CUSTODY / ANALYSIS REQUEST

2527 FRESNO STREET • FRESNO, CA 93721 • PHONE (559) 268-7021 • FAX: (559) 268-0740

ANALYTICAL CHEMISTRY DIVISION
CALIFORNIA ELAP CERTIFICATION # 1371

WORK ORDER #: JE 22017
PAGE 01 OF 02

REPORT TO:	☐ INVOICE TO: ☐ REPORT COPY TO:	REPORTING:
ATTENTION: SHAUN R.	ATTENTION:	☒ STANDARD FORMAT ☐ PDF ☐ EDT (SWRCB) ☐ EXCEL
COMPANY NAME: GEOTECHNICAL DIVISION	COMPANY NAME:	☐ GEOTRACKER/COELT (LUFT) GLOBAL ID:
ADDRESS:	ADDRESS:	☐ COUNTY ENVIRONMENTAL HEALTH:
		☐ STATE WATER RESOURCES CONTROL BOARD:
PHONE:	PHONE:	☐ OTHER:
EMAIL / FAX:	EMAIL / FAX:	

SAMPLE INFORMATION	SAMPLE TYPES	PROJECT INFORMATION
SAMPLED BY (PRINT): ALAN V.	SOLID: BS - BIOSOLID CR - CERAMIC SL - SOIL/SOLID	CONTRACT / P.O. NO.:
SIGNATURE:	LIQUID: DW - DRINKING WATER GW - GROUND WATER OL - OIL SF - SURFACE WATER ST - STORM WATER WW - WASTEWATER	PROJECT: SCCS WORKFORCE HOUSING
☐ PUBLIC SYSTEM ☒ ROUTINE ☐ PRIVATE WELL ☐ REPEAT ☐ OTHER ☐ REPLACEMENT		PROJECT NUMBER: E95815.01
TURN AROUND TIME ☒ STANDARD ☐ RUSH, DUE ON:		PROJECT MANAGER: SHAUN R.

ANALYSIS REQUESTED

NOTES ON RECEIVED CONDITION:
☐ CUSTODY SEAL(S) BROKEN ☐ SAMPLES(S) DAMAGED
☐ ON ICE ☐ AMBIENT TEMP. ☐ INCORRECT PRESERVATION

LAB USE	CLIENT SAMPLE ID	DATE	TIME	TYPE	PH	SULFATE	CHLORIDE								STATION CODE
1	B-1 @ 0'-5'	05-11-2023	NA	SL	x	x	x								

COMMENTS / ADDITIONAL INSTRUCTIONS:

RELINQUISHED BY	COMPANY	DATE	TIME	RECEIVED BY	COMPANY
Boreal one		05-22-2023	08:44am	McCullough	MTA

Payment for services rendered as noted herein are due in full within 30 days from the date invoiced. If not so paid, account balances are deemed delinquent. Delinquent balances are subject to monthly service charges and interest specified in MTA's current Standard Terms and Conditions for Laboratory Services. The person signing for the Client/Company acknowledges that they are either the Client or an authorized agent to the Client, that the Client agrees to be responsible for payment for the services on this Chain of Custody and agrees to MTA's terms and conditions for laboratory services unless contractually bound otherwise. MTA's current terms and conditions can be obtained by contacting our accounting department at (559) 268-7021.

FL-SC-0005-04 A

Moore Twining Associates

Sample Integrity Page 2 of 2 WO# JE 22017 MTA Bottles: Yes or No

COC Info			
Was temperature within range? Chemistry ≤6°C Micro <10°C Temp____°C	Yes No N/A	Did all bottle labels agree with COC?	Yes No N/A
If samples were taken today, is there evidence that chilling has begun? Recvd____C°	Yes No N/A	Was a sufficient amount of sample received?	Yes No N/A
Did all bottles arrive unbroken and intact?	Yes No N/A	Were correct containers and preservatives received for the tests requested?	Yes No N/A
Do samples have a hold time <72 hours?	Yes No N/A		

Were there bubbles in VOA vials? **(Volatiles Only)**	Yes No N/A
Was PM notified of discrepancies? PM: By/Time:	Yes No N/A

Bottles Received															
125ml (A) 250ml (B) 1Liter (C) 40ml VOA (V)	1														
Bacti $Na_2S_2O_3$															
None (P)															
None (AG)															
None (CG) 500ml															
Cr6 Buffer (P) Borate Carbonate Buffer															
Dissolved Oxygen 300ml (P)															
HNO_3 (P)															
HCl (AG)															
H_2SO_4 (P)															
H_3PO_4 (AG)															
NaOH (P)															
NaOH + ZnAc (P)															
$Na_2S_2O_3$ (AG)															
$Na_2S_2O_3$ (CG)															
$Na_2S_2O_3$ 250ml (Brown P) 549															
Thio/K Citrate															
NH_4Cl (AG) 552															
Other:															
Client Own															
Low Level Hg/Metals Double Bag															
Plastic Bag	1														
Glass Jar: 125 / 250 / 500															
Soil Tube: Brass / Steel / Plastic															
5g Encore															
1G Cubitaner															

Comments

Filter or Split	Container	Preservative	Date/Time/Initials
S P F			
S P F			
S P F			
S P F			
S P F			

Labeled by ___ @____ Labels checked by: ___ @ 0940

FL-SC-0003-07

MOORE TWINING
ASSOCIATES, INC.
California ELAP Certificate #1371

2527 Fresno Street
Fresno, CA 93721
(559) 268-7021 Phone
(559) 268-0740 Fax

June 19, 2023

Work Order #: **JF01005**

Shaun Reich
MTA Geotechnical Division
2527 Fresno Street
Fresno, CA 93721

RE: SCCS Workforce Housing

Enclosed are the analytical results for samples received by our laboratory on **06/01/23** . For your reference, these analyses have been assigned laboratory work order number **JF01005**.

All analyses have been performed according to our laboratory's quality assurance program. All results are intended to be considered in their entirety, Moore Twining Associates, Inc. (MTA) is not responsible for use of less than complete reports. Results apply only to samples analyzed.

If you have any questions, please feel free to contact us at the number listed above.

Sincerely,

Moore Twining Associates, Inc.

Lauren Cox

Lauren Cox
Client Services Representative

California ELAP Certificate #1371

2527 Fresno Street
Fresno, CA 93721
(559) 268-7021 Phone
(559) 268-0740 Fax

MTA Geotechnical Division 2527 Fresno Street Fresno CA, 93721	**Project:** SCCS Workforce Housing **Project Number:** E95815.01 **Project Manager:** Shaun Reich	**Reported:** 06/19/2023

Analytical Report for the Following Samples

Sample ID	Notes	Laboratory ID	Matrix	Date Sampled	Date Received
B-4 @ 0'-5'		JF01005-01	Soil	05/10/23 00:00	06/01/23 10:40

MOORE TWINING ASSOCIATES, INC.
California ELAP Certificate #1371

2527 Fresno Street
Fresno, CA 93721
(559) 268-7021 Phone
(559) 268-0740 Fax

MTA Geotechnical Division	**Project:** SCCS Workforce Housing	**Reported:**
2527 Fresno Street	**Project Number:** E95815.01	06/19/2023
Fresno CA, 93721	**Project Manager:** Shaun Reich	

B-4 @ 0'-5'

JF01005-01 (Soil)

Analyte	Result	Reporting Limit	Units	Batch	Prepared	Analyzed	Method	Flag
Inorganics								
Chloride	ND	0.0040	% by Weight	[CALC]	06/16/23	06/16/23	[CALC]	
Chloride	ND	40	mg/kg	B3F1417	06/15/23	06/16/23	Cal Test 422	HT2
pH	**6.8**	0.10	pH Units	B3F1417	06/15/23	06/16/23	Cal Test 643	**HT2**
Sulfate as SO4	**0.0071**	0.0040	% by Weight	[CALC]	06/16/23	06/16/23	[CALC]	
Sulfate as SO4	**71**	40	mg/kg	B3F1417	06/15/23	06/16/23	Cal Test 417	**HT2**

Notes and Definitions

DUP1	A high RPD was observed between a sample and this sample's duplicate.
HT2	This sample was analyzed past the EPA recommended holding time for this parameter due to late delivery of the sample to the laboratory.
ND	Analyte NOT DETECTED at or above the reporting limit
RPD	Relative Percent Difference
mg/kg	milligrams per kilogram (parts per million concentration units)

MOORE TWINING ASSOCIATES, INC.

ANALYTICAL CHEMISTRY DIVISION
CALIFORNIA ELAP CERTIFICATION # 1371

CHAIN OF CUSTODY / ANALYSIS REQUEST

2527 FRESNO STREET • FRESNO, CA 93721 • PHONE (559) 268-7021 • FAX: (559) 268-0740

WORK ORDER #: ~~JE 31005~~ 3F01005
PAGE 01 OF ~~2~~

REPORT TO:
ATTENTION: SHAUN R.
COMPANY NAME: MTA GEOTECHNICAL
ADDRESS: DIVISION
PHONE:
EMAIL / FAX:

☐ **INVOICE TO:**
ATTENTION:
COMPANY NAME:
ADDRESS:
PHONE:
EMAIL / FAX:

☐ **REPORT COPY TO:**

REPORTING:
☒ STANDARD FORMAT ☐ PDF
☐ EDT (SWRCB) ☐ EXCEL
☐ GEOTRACKER/COELT (LUFT)
GLOBAL ID:
☐ COUNTY ENVIRONMENTAL HEALTH:
☐ STATE WATER RESOURCES CONTROL BOARD:
☐ OTHER:

SAMPLE INFORMATION
SAMPLED BY (PRINT): ALAN V.
SIGNATURE:

☐ PUBLIC SYSTEM ☒ ROUTINE
☐ PRIVATE WELL ☐ REPEAT
☐ OTHER ☐ REPLACEMENT

TURN AROUND TIME
☐ STANDARD
☐ RUSH, DUE ON:

SAMPLE TYPES
SOLID:
BS - BIOSOLID
CR - CERAMIC
SL - SOIL/SOLID
LIQUID:
DW - DRINKING WATER
GW - GROUND WATER
OL - OIL
SF - SURFACE WATER
ST - STORM WATER
WW - WASTEWATER

PROJECT INFORMATION
CONTRACT / P.O. NO.:
PROJECT: SCCS WORKFORCE HOUSING
PROJECT NUMBER: E95815.01
PROJECT MANAGER: SHAUN R.

ANALYSIS REQUESTED

LAB USE — NOTES ON RECEIVED CONDITION:
☐ CUSTODY SEAL(S) BROKEN ☐ SAMPLES(S) DAMAGED
☐ ON ICE ☑ AMBIENT TEMP. ☐ INCORRECT PRESERVATION

LAB USE	CLIENT SAMPLE ID	DATE	TIME	TYPE	PH	CHLORIDE	SULFATE								STATION CODE
1	B-4 @ 0'-5'	NA	NA	SL	x	x	x								

COMMENTS / ADDITIONAL INSTRUCTIONS: 05/10/2023 date on sample

RELINQUISHED BY	COMPANY	DATE	TIME	RECEIVED BY	COMPANY
BOUASONE PHOUIKHAMBAY MAT		06-01-2023	~~06-01-2023~~ 10:40 AM	[signature]	MTA

Payment for services rendered as noted herein are due in full within 30 days from the date invoiced. If not so paid, account balances are deemed delinquent. Delinquent balances are subject to monthly service charges and interest specified in MTA's current Standard Terms and Conditions for Laboratory Services. The person signing for the Client/Company acknowledges that they are either the Client or an authorized agent to the Client, that the Client agrees to be responsible for payment for the services on this Chain of Custody and agrees to MTA's terms and conditions for laboratory services unless contractually bound otherwise. MTA's current terms and conditions can be obtained by contacting our accounting department at (559) 268-7021.

FL-SC-0005-04 A

Moore Twining Associates

Sample Integrity Page 2 of 2 WO# SF01005 MTA Bottles: Yes or No

COC Info

Was temperature within range? Chemistry ≤6°C Micro <10°C Temp_____°C	Yes No N/A	Did all bottle labels agree with COC?	Yes No N/A	Were there bubbles in VOA vials? **(Volatiles Only)**	Yes No N/A
If samples were taken today, is there evidence that chilling has begun? Recvd_____C°	Yes No N/A	Was a sufficient amount of sample received?	Yes No N/A	Was PM notified of discrepancies? PM: By/Time:	Yes No N/A
Did all bottles arrive unbroken and intact?	Yes No N/A	Were correct containers and preservatives received for the tests requested?	Yes No N/A		
Do samples have a hold time <72 hours?	Yes No N/A				

Bottles Received

125ml (A) 250ml (B) 1Liter (C) 40ml VOA (V)	1														
Bacti $Na_2S_2O_3$															
None (P)															
None (AG)															
None (CG) 500ml															
Cr6 Buffer (P) Borate Carbonate Buffer															
Dissolved Oxygen 300ml (P)															
HNO_3 (P)															
HCl (AG)															
H_2SO_4 (P)															
H_3PO_4 (AG)															
NaOH (P)															
NaOH + ZnAc (P)															
$Na_2S_2O_3$ (AG)															
$Na_2S_2O_3$ (CG)															
$Na_2S_2O_3$ 250ml (Brown P) 549															
Thio/K Citrate															
NH_4Cl (AG) 552															
Other:															
Client Own															
Low Level Hg/Metals Double Bag															
Plastic Bag	1														
Glass Jar: 125 / 250 / 500															
Soil Tube: Brass / Steel / Plastic															
5g Encore															
1G Cubitaner															

Comments

Filter or Split	Container	Preservative	Date/Time/Initials
S P F			
S P F			
S P F			
S P F			
S P F			

Labeled by: [illegible] @_____ Labels checked by: [illegible] @ 1140

FL-SC-0003-07

APPENDIX D1

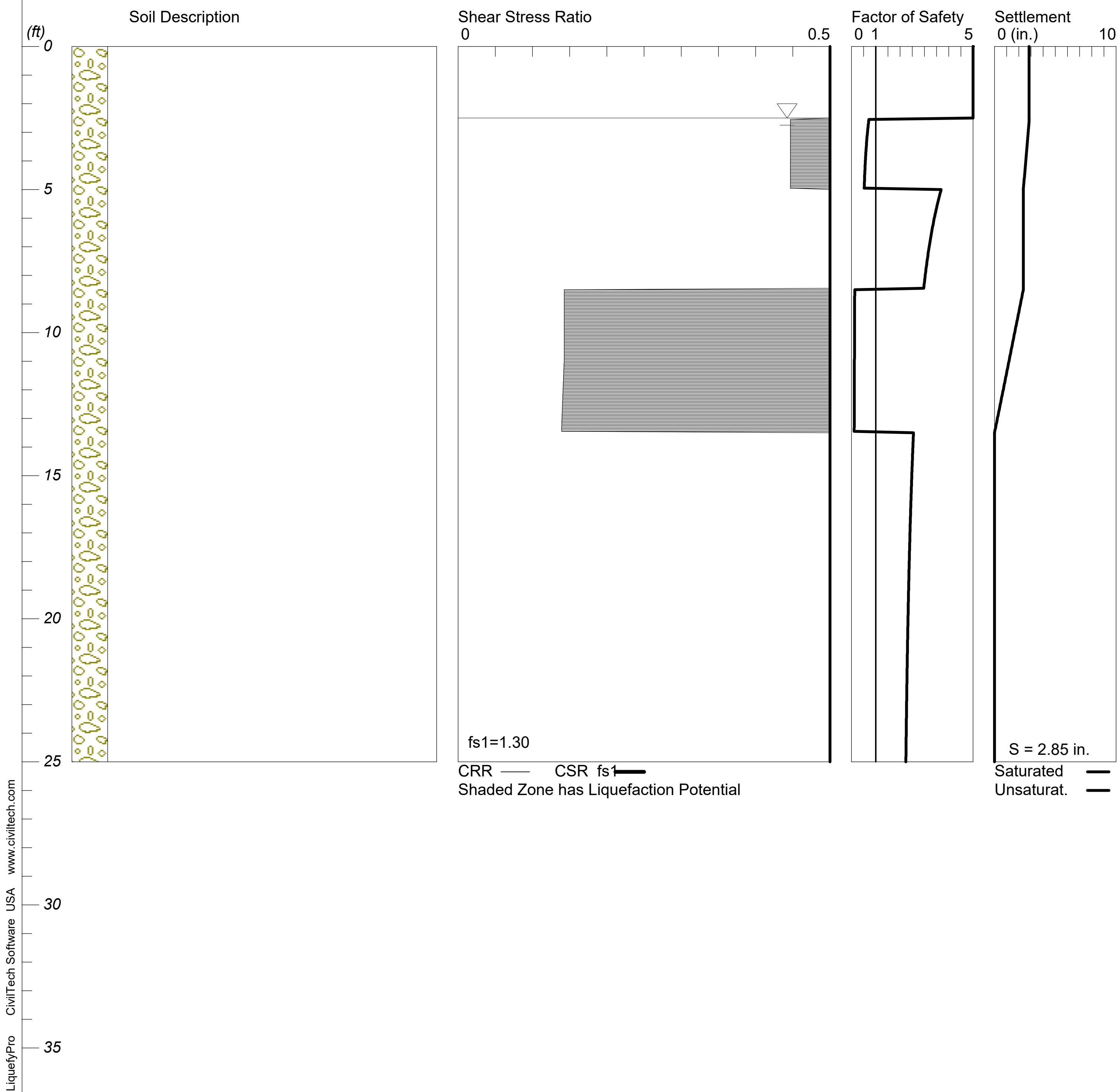
RESULTS OF LIQUEFACTION/SEISMIC SETTLEMENT CALCULATIONS

LIQUEFACTION ANALYSIS

Proposed SCCS Workforce Housing Project

Hole No.=B-4 Water Depth=2.5 ft

Magnitude=6.27
Acceleration=0.741g



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LIQUEFACTION ANALYSIS CALCULATION DETAILS

****

Title: Proposed SCCS Workforce Housing Project
Subtitle: E95815.01

Input Data:
Surface Elev.=
Hole No.=B-4
Depth of Hole=25.00 ft
Water Table during Earthquake= 2.50 ft
Water Table during In-Situ Testing= 5.00 ft
Max. Acceleration=0.74 g
Earthquake Magnitude=6.27
No-Liquefiable Soils: CL, OL are Non-Liq. Soil
1. SPT or BPT Calculation.
2. Settlement Analysis Method: Ishihara / Yoshimine
3. Fines Correction for Liquefaction: Idriss/Seed
4. Fine Correction for Settlement: During Liquefaction*
5. Settlement Calculation in: All zones*
6. Hammer Energy Ratio, Ce = 1.4
7. Borehole Diameter, Cb = 1
8. Sampling Method, Cs = 1.2
9. User request factor of safety (apply to CSR) , User = 1.3
Plot one CSR curve (fs1=User)
10. Average two input data between two Depths: No
* Recommended Options

In-Situ Test Data:

Depth ft	SPT	Gamma pcf	Fines %
0.00	5.00	110.00	NoLiq
2.50	9.00	110.00	22.00
5.00	20.00	92.70	22.00
8.50	2.00	92.70	17.00
13.50	50.00	92.70	96.00
18.50	50.00	92.70	96.00
25.00	50.00	92.70	96.00

Output Results:
Calculation segment, dz=0.050 ft
User defined Print Interval, dp=1.00 ft

Peak Ground Acceleration (PGA), a_max = 0.74g

CSR Calculation:

Depth ft	gamma pcf	sigma atm	gamma' pcf	sigma' atm	rd	mZ g	a(z) g	CSR	x fs1	=CSRfs
0.00	110.00	0.000	110.00	0.000	1.00	0.000	0.741	0.48	1.30	0.63
1.00	110.00	0.052	110.00	0.052	1.00	0.000	0.741	0.48	1.30	0.62
2.00	110.00	0.104	110.00	0.104	1.00	0.000	0.741	0.48	1.30	0.62
3.00	110.00	0.156	47.60	0.143	0.99	0.000	0.741	0.52	1.30	0.68
4.00	110.00	0.208	47.60	0.165	0.99	0.000	0.741	0.60	1.30	0.78
5.00	92.70	0.260	30.30	0.188	0.99	0.000	0.741	0.66	1.30	0.86
6.00	92.70	0.304	30.30	0.202	0.99	0.000	0.741	0.71	1.30	0.93
7.00	92.70	0.348	30.30	0.216	0.98	0.000	0.741	0.76	1.30	0.99
8.00	92.70	0.391	30.30	0.231	0.98	0.000	0.741	0.80	1.30	1.04
9.00	92.70	0.435	30.30	0.245	0.98	0.000	0.741	0.84	1.30	1.09
10.00	92.70	0.479	30.30	0.259	0.98	0.000	0.741	0.87	1.30	1.13
11.00	92.70	0.523	30.30	0.274	0.97	0.000	0.741	0.90	1.30	1.17
12.00	92.70	0.567	30.30	0.288	0.97	0.000	0.741	0.92	1.30	1.20
13.00	92.70	0.610	30.30	0.302	0.97	0.000	0.741	0.94	1.30	1.23
14.00	92.70	0.654	30.30	0.317	0.97	0.000	0.741	0.96	1.30	1.25
15.00	92.70	0.698	30.30	0.331	0.97	0.000	0.741	0.98	1.30	1.27
16.00	92.70	0.742	30.30	0.345	0.96	0.000	0.741	1.00	1.30	1.30
17.00	92.70	0.786	30.30	0.359	0.96	0.000	0.741	1.01	1.30	1.31
18.00	92.70	0.829	30.30	0.374	0.96	0.000	0.741	1.02	1.30	1.33
19.00	92.70	0.873	30.30	0.388	0.96	0.000	0.741	1.04	1.30	1.35
20.00	92.70	0.917	30.30	0.402	0.95	0.000	0.741	1.05	1.30	1.36
21.00	92.70	0.961	30.30	0.417	0.95	0.000	0.741	1.06	1.30	1.37
22.00	92.70	1.005	30.30	0.431	0.95	0.000	0.741	1.06	1.30	1.38
23.00	92.70	1.048	30.30	0.445	0.95	0.000	0.741	1.07	1.30	1.39
24.00	92.70	1.092	30.30	0.460	0.94	0.000	0.741	1.08	1.30	1.40
25.00	92.70	1.136	30.30	0.474	0.94	0.000	0.741	1.09	1.30	1.41

CSR is based on water table at 2.50 during earthquake

CRR Calculation from SPT or BPT data:

Depth ft	SPT	Cebs	Cr	sigma' atm	Cn	(N1)60	Fines %	d(N1)60	(N1)60f	CRR7.5
0.00	5.00	1.68	0.75	0.000	1.70	10.71	NoLiq	7.14	17.85	0.19
1.00	5.00	1.68	0.75	0.052	1.70	10.71	NoLiq	7.14	17.85	0.19
2.00	5.00	1.68	0.75	0.104	1.70	10.71	NoLiq	7.14	17.85	0.19
3.00	9.00	1.68	0.75	0.156	1.70	19.28	22.00	5.72	25.00	0.28
4.00	9.00	1.68	0.75	0.208	1.70	19.28	22.00	5.72	25.00	0.28
5.00	20.00	1.68	0.75	0.260	1.70	42.84	22.00	7.92	50.76	2.00

6.00	20.00	1.68	0.75	0.274	1.70	42.84	22.00	7.92	50.76	2.00
7.00	20.00	1.68	0.75	0.289	1.70	42.84	22.00	7.92	50.76	2.00
8.00	20.00	1.68	0.75	0.303	1.70	42.84	22.00	7.92	50.76	2.00
9.00	2.00	1.68	0.85	0.317	1.70	4.86	17.00	3.30	8.16	0.09
10.00	2.00	1.68	0.85	0.332	1.70	4.86	17.00	3.30	8.16	0.09
11.00	2.00	1.68	0.85	0.346	1.70	4.86	17.00	3.30	8.16	0.09
12.00	2.00	1.68	0.85	0.360	1.67	4.76	17.00	3.30	8.06	0.09
13.00	2.00	1.68	0.85	0.374	1.63	4.67	17.00	3.29	7.96	0.09
14.00	50.00	1.68	0.85	0.389	1.60	114.51	96.00	27.90	142.41	2.00
15.00	50.00	1.68	0.95	0.403	1.58	125.69	96.00	30.14	155.83	2.00
16.00	50.00	1.68	0.95	0.417	1.55	123.52	96.00	29.70	153.22	2.00
17.00	50.00	1.68	0.95	0.432	1.52	121.45	96.00	29.29	150.74	2.00
18.00	50.00	1.68	0.95	0.446	1.50	119.48	96.00	28.90	148.38	2.00
19.00	50.00	1.68	0.95	0.460	1.47	117.61	96.00	28.52	146.13	2.00
20.00	50.00	1.68	0.95	0.475	1.45	115.82	96.00	28.16	143.99	2.00
21.00	50.00	1.68	0.95	0.489	1.43	114.12	96.00	27.82	141.94	2.00
22.00	50.00	1.68	0.95	0.503	1.41	112.48	96.00	27.50	139.98	2.00
23.00	50.00	1.68	0.95	0.518	1.39	110.91	96.00	27.18	138.10	2.00
24.00	50.00	1.68	0.95	0.532	1.37	109.41	96.00	26.88	136.29	2.00
25.00	50.00	1.68	0.95	0.546	1.35	107.97	96.00	26.59	134.56	2.00

CRR is based on water table at 5.00 during In-Situ Testing

Factor of Safety, - Earthquake Magnitude= 6.27:

Depth ft	sigC' atm	CRR7.5	x Ksig	=CRRv	x MSF	=CRRm	CSRfs	F.S.=CRRm/CSRfs
0.00	0.00	0.19	1.00	0.19	1.58	2.00	0.63	5.00 ^
1.00	0.03	0.19	1.00	0.19	1.58	2.00	0.62	5.00 ^
2.00	0.07	0.19	1.00	0.19	1.58	2.00	0.62	5.00 ^
3.00	0.10	0.28	1.00	0.28	1.58	0.45	0.68	0.66 *
4.00	0.14	0.28	1.00	0.28	1.58	0.45	0.78	0.57 *
5.00	0.17	2.00	1.00	2.00	1.58	3.16	0.86	3.69
6.00	0.18	2.00	1.00	2.00	1.58	3.16	0.93	3.41
7.00	0.19	2.00	1.00	2.00	1.58	3.16	0.99	3.20
8.00	0.20	2.00	1.00	2.00	1.58	3.16	1.04	3.03
9.00	0.21	0.09	1.00	0.09	1.58	0.14	1.09	0.13 *
10.00	0.22	0.09	1.00	0.09	1.58	0.14	1.13	0.13 *
11.00	0.22	0.09	1.00	0.09	1.58	0.14	1.17	0.12 *
12.00	0.23	0.09	1.00	0.09	1.58	0.14	1.20	0.12 *
13.00	0.24	0.09	1.00	0.09	1.58	0.14	1.23	0.11 *
14.00	0.25	2.00	1.00	2.00	1.58	3.16	1.25	2.53
15.00	0.26	2.00	1.00	2.00	1.58	3.16	1.27	2.48
16.00	0.27	2.00	1.00	2.00	1.58	3.16	1.30	2.44
17.00	0.28	2.00	1.00	2.00	1.58	3.16	1.31	2.41
18.00	0.29	2.00	1.00	2.00	1.58	3.16	1.33	2.38
19.00	0.30	2.00	1.00	2.00	1.58	3.16	1.35	2.35
20.00	0.31	2.00	1.00	2.00	1.58	3.16	1.36	2.33
21.00	0.32	2.00	1.00	2.00	1.58	3.16	1.37	2.30
22.00	0.33	2.00	1.00	2.00	1.58	3.16	1.38	2.28
23.00	0.34	2.00	1.00	2.00	1.58	3.16	1.39	2.27
24.00	0.35	2.00	1.00	2.00	1.58	3.16	1.40	2.25
25.00	0.36	2.00	1.00	2.00	1.58	3.16	1.41	2.24

* F.S.<1: Liquefaction Potential Zone. (If above water table: F.S.=5)
^ No-liquefiable Soils or above Water Table.
(F.S. is limited to 5, CRR is limited to 2, CSR is limited to 2)

CPT convert to SPT for Settlement Analysis:
Fines Correction for Settlement Analysis:

Depth ft	Ic	qc/N60	qc1 atm	(N1)60	Fines %	d(N1)60	(N1)60s
0.00	-	-	-	17.85	NoLiq	0.00	17.85
1.00	-	-	-	17.85	NoLiq	0.00	17.85
2.00	-	-	-	17.85	NoLiq	0.00	17.85
3.00	-	-	-	25.00	22.00	0.00	25.00
4.00	-	-	-	25.00	22.00	0.00	25.00
5.00	-	-	-	50.76	22.00	0.00	50.76
6.00	-	-	-	50.76	22.00	0.00	50.76
7.00	-	-	-	50.76	22.00	0.00	50.76
8.00	-	-	-	50.76	22.00	0.00	50.76
9.00	-	-	-	8.16	17.00	0.00	8.16
10.00	-	-	-	8.16	17.00	0.00	8.16
11.00	-	-	-	8.16	17.00	0.00	8.16
12.00	-	-	-	8.06	17.00	0.00	8.06
13.00	-	-	-	7.96	17.00	0.00	7.96
14.00	-	-	-	100.00	96.00	0.00	100.00
15.00	-	-	-	100.00	96.00	0.00	100.00
16.00	-	-	-	100.00	96.00	0.00	100.00
17.00	-	-	-	100.00	96.00	0.00	100.00
18.00	-	-	-	100.00	96.00	0.00	100.00
19.00	-	-	-	100.00	96.00	0.00	100.00
20.00	-	-	-	100.00	96.00	0.00	100.00
21.00	-	-	-	100.00	96.00	0.00	100.00
22.00	-	-	-	100.00	96.00	0.00	100.00
23.00	-	-	-	100.00	96.00	0.00	100.00
24.00	-	-	-	100.00	96.00	0.00	100.00
25.00	-	-	-	100.00	96.00	0.00	100.00

(N1)60s has been fines corrected in liquefaction analysis, therefore d(N1)60=0.
Fines=NoLiq means the soils are not liquefiable.

Settlement of Saturated Sands:
Settlement Analysis Method: Ishihara / Yoshimine

Depth ft	CSRsf	/ MSF*	=CSRm	F.S.	Fines %	(N1)60s	Dr %	ec %	dsz in.	dsp in.	S in.
24.95	1.41	1.00	1.41	2.24	96.00	100.00	100.00	0.000	0.0E0	0.000	0.000
24.00	1.40	1.00	1.40	2.25	96.00	100.00	100.00	0.000	0.0E0	0.000	0.000
23.00	1.39	1.00	1.39	2.27	96.00	100.00	100.00	0.000	0.0E0	0.000	0.000
22.00	1.38	1.00	1.38	2.28	96.00	100.00	100.00	0.000	0.0E0	0.000	0.000
21.00	1.37	1.00	1.37	2.30	96.00	100.00	100.00	0.000	0.0E0	0.000	0.000
20.00	1.36	1.00	1.36	2.33	96.00	100.00	100.00	0.000	0.0E0	0.000	0.000
19.00	1.35	1.00	1.35	2.35	96.00	100.00	100.00	0.000	0.0E0	0.000	0.000
18.00	1.33	1.00	1.33	2.38	96.00	100.00	100.00	0.000	0.0E0	0.000	0.000
17.00	1.31	1.00	1.31	2.41	96.00	100.00	100.00	0.000	0.0E0	0.000	0.000
16.00	1.30	1.00	1.30	2.44	96.00	100.00	100.00	0.000	0.0E0	0.000	0.000
15.00	1.27	1.00	1.27	2.48	96.00	100.00	100.00	0.000	0.0E0	0.000	0.000
14.00	1.25	1.00	1.25	2.53	96.00	100.00	100.00	0.000	0.0E0	0.000	0.000
13.00	1.23	1.00	1.23	0.11	17.00	7.96	45.38	3.983	2.4E-2	0.239	0.239
12.00	1.20	1.00	1.20	0.12	17.00	8.06	45.64	3.957	2.4E-2	0.476	0.716
11.00	1.17	1.00	1.17	0.12	17.00	8.16	45.91	3.930	2.4E-2	0.473	1.189
10.00	1.13	1.00	1.13	0.13	17.00	8.16	45.91	3.930	2.4E-2	0.472	1.661
9.00	1.09	1.00	1.09	0.13	17.00	8.16	45.91	3.930	2.4E-2	0.472	2.132
8.00	1.04	1.00	1.04	3.03	22.00	50.76	100.00	0.000	0.0E0	0.236	2.368
7.00	0.99	1.00	0.99	3.20	22.00	50.76	100.00	0.000	0.0E0	0.000	2.368
6.00	0.93	1.00	0.93	3.41	22.00	50.76	100.00	0.000	0.0E0	0.000	2.368
5.00	0.86	1.00	0.86	3.69	22.00	50.76	100.00	0.000	0.0E0	0.000	2.368
4.00	0.78	1.00	0.78	0.57	22.00	25.00	79.72	1.703	1.0E-2	0.207	2.575
3.00	0.68	1.00	0.68	0.66	22.00	25.00	79.72	1.547	9.3E-3	0.197	2.772
2.55	0.62	1.00	0.62	0.72	22.00	25.00	79.72	1.401	8.4E-3	0.079	2.851

Settlement of Saturated Sands = 2.851 in.
qc1 and (N1)60 is after fines correction in liquefaction analysis
dsz is per each segment, dz = 0.05 ft
dsp is per each print interval, dp = 1.00 ft
S is cumulated settlement at this depth

Settlement of Unsaturated Sands:

Depth ft	sigma' atm	sigC' atm	(N1)60s	CSRsf	Gmax atm	g*Ge/Gm	g_eff	ec7.5 %	Cec	ec %	dsz in.	dsp in.	S in.
2.50	0.13	0.08	17.85	0.62	339.38	2.4E-4	1.0000	1.1414	0.69	0.7882	0.00E0	0.000	0.000
2.00	0.10	0.07	17.85	0.62	303.55	2.1E-4	0.1571	0.1793	0.69	0.1238	0.00E0	0.000	0.000
1.00	0.05	0.03	17.85	0.62	214.66	1.5E-4	0.0298	0.0340	0.69	0.0235	0.00E0	0.000	0.000
0.00	0.00	0.00	17.85	0.63	2.98	2.1E-6	0.0010	0.0012	0.69	0.0008	0.00E0	0.000	0.000

Settlement of Unsaturated Sands

Settlement of Unsaturated Sands = 0.000 in.
dsz is per each segment, dz = 0.05 ft
dsp is per each print interval, dp = 1.00 ft
S is cumulated settlement at this depth

Total Settlement of Saturated and Unsaturated Sands = 2.851 in.

Units: Unit: qc, fs, Stress or Pressure = atm (1.0581tsf); Unit Weight = pcf; Depth = ft; Settlement = in.

1 atm (atmosphere) = 1.0581 tsf(1 tsf = 1 ton/ft2 = 2 kip/ft2)
1 atm (atmosphere) = 101.325 kPa(1 kPa = 1 kN/m2 = 0.001 Mpa)

SPT	Field data from Standard Penetration Test (SPT)
BPT	Field data from Becker Penetration Test (BPT)
qc	Field data from Cone Penetration Test (CPT) [atm (tsf)]
fs	Friction from CPT testing [atm (tsf)]
Rf	Ratio of fs/qc (%)
gamma	Total unit weight of soil
gamma'	Effective unit weight of soil
Fines	Fines content [%]
D50	Mean grain size
Dr	Relative Density
sigma	Total vertical stress [atm]
sigma'	Effective vertical stress [atm]
sigC'	Effective confining pressure [atm]
rd	Acceleration reduction coefficient by Seed
a_max.	Peak Ground Acceleration (PGA) in ground surface
mZ	Linear acceleration reduction coefficient X depth
a_min.	Minimum acceleration under linear reduction, mZ
CRRv	CRR after overburden stress correction, CRRv=CRR7.5 * Ksig
CRR7.5	Cyclic resistance ratio (M=7.5)
Ksig	Overburden stress correction factor for CRR7.5
CRRm	After magnitude scaling correction CRRm=CRRv * MSF
MSF	Magnitude scaling factor from M=7.5 to user input M
CSR	Cyclic stress ratio induced by earthquake
CSRfs	CSRfs=CSR*fs1 (Default fs1=1)
fs1	First CSR curve in graphic defined in #9 of Advanced page
fs2	2nd CSR curve in graphic defined in #9 of Advanced page
F.S.	Calculated factor of safety against liquefaction F.S.=CRRm/CSRsf
Cebs	Energy Ratio, Borehole Dia., and Sampling Method Corrections
Cr	Rod Length Corrections
Cn	Overburden Pressure Correction
(N1)60	SPT after corrections, (N1)60=SPT * Cr * Cn * Cebs
d(N1)60	Fines correction of SPT
(N1)60f	(N1)60 after fines corrections, (N1)60f=(N1)60 + d(N1)60
Cq	Overburden stress correction factor
qc1	CPT after Overburden stress correction

dqc1	Fines correction of CPT
qc1f	CPT after Fines and Overburden correction, qc1f=qc1 + dqc1
qc1n	CPT after normalization in Robertson's method
Kc	Fine correction factor in Robertson's Method
qc1f	CPT after Fines correction in Robertson's Method
Ic	Soil type index in Suzuki's and Robertson's Methods
(N1)60s	(N1)60 after settlement fines corrections
CSRm	After magnitude scaling correction for Settlement calculation CSRm=CSRsf / MSF*
CSRfs	Cyclic stress ratio induced by earthquake with user inputed fs
MSF*	Scaling factor from CSR, MSF*=1, based on Item 2 of Page C.
ec	Volumetric strain for saturated sands
dz	Calculation segment, dz=0.050 ft
dsz	Settlement in each segment, dz
dp	User defined print interval
dsp	Settlement in each print interval, dp
Gmax	Shear Modulus at low strain
g_eff	gamma_eff, Effective shear Strain
g*Ge/Gm	gamma_eff * G_eff/G_max, Strain-modulus ratio
ec7.5	Volumetric Strain for magnitude=7.5
Cec	Magnitude correction factor for any magnitude
ec	Volumetric strain for unsaturated sands, ec=Cec * ec7.5
NoLiq	No-Liquefy Soils

References:

1. NCEER Workshop on Evaluation of Liquefaction Resistance of Soils. Youd, T.L., and Idriss, I.M., eds., Technical Report NCEER 97-0022.
SP117. Southern California Earthquake Center. Recommended Procedures for Implementation of DMG Special Publication 117, Guidelines for Analyzing and Mitigating Liquefaction in California. University of Southern California. March 1999.
2. RECENT ADVANCES IN SOIL LIQUEFACTION ENGINEERING AND SEISMIC SITE RESPONSE EVALUATION, Paper No. SPL-2, PROCEEDINGS: Fourth International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics, San Diego, CA, March 2001.
3. RECENT ADVANCES IN SOIL LIQUEFACTION ENGINEERING: A UNIFIED AND CONSISTENT FRAMEWORK, Earthquake Engineering Research Center, Report No. EERC 2003-06 by R.B Seed and etc. April 2003.

Note: Print Interval you selected does not show complete results. To get complete results, you should select 'Segment' in Print Interval (Item 12, Page C).

APPENDIX D2

ENERGY MEASUREMENT FOR DYNAMIC PENETROMETERS STANDARD PENETRATION TEST (SPT) FOR MOORE TWINING ASSOCIATES (MTA) CME75 DRILL RIG

GRL engineers, inc.
Dynamic Measurements and Analyses

Job No. 228076-1

Report on: Energy Measurement for Dynamic Penetrometers – Standard Penetration Test (SPT)

Truck 170 CME 75 Drill Rig Calibration
Fresno, CA

Prepared for Moore Twining Associates, Inc.
By Camilo Alvarez, PE & William Chambers CPEng

June 3, 2022

GRL Dynamic Measurements and Analyses
engineers, inc.

June 3, 2022

Allen Bushey
Moore Twining Associates, Inc

Re: Energy Measurement for Dynamic Penetrometers
Standard Penetration Test (SPT) on Truck 170 CME 75 drill rig
Fresno, CA. GRL Job No. 228076-1

Dear Mr. Allen Bushey:

This report transmits our findings from energy measurements and related data analysis conducted by GRL Engineers, Inc. (GRL) for your Truck 170 mounted CME 75 drill rig located Fresno, CA. One automatic hammer and penetrometer system was monitored during Standard Penetration Test (SPT) of the test borehole BH2. Dynamic testing summarized in this report was conducted on June 2, 2022.

The purpose in collecting the SPT energy measurements was to compute the energy transfer efficiency for a single SPT hammer. To meet this objective, an 8G Model, Pile Driving Analyzer® (PDA) utilizing the SPT Analyzer feature was used to acquire and process the dynamic test data. Additional information regarding the testing equipment and analytical procedures is provided in Appendix A.

Test Sequence

Using an instrumented AW-J rod for a Truck 170 mounted CME 75 drill rig at test borehole BH2, energy measurements were made at five sample depths for the drill rig. From BH2, the dynamic measurements were obtained from sample depths of 10.0, 12.5, 14.5, 18.0 and 20.0 ft. Each sample depth consisted of energy measurements of 18 inches of driving.

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Energy Transfer Measurements

A Model 8G Pile Driving Analyzer was used to take measurements of strain and acceleration. The strain and acceleration signals were conditioned and converted to forces and velocities by the PDA. The PDA interprets the measured dynamic data according to the Case Method equations. Force and velocity records from the PDA were also viewed graphically on an LCD screen to evaluate data quality. All force and velocity records were also digitally stored for subsequent analysis.

The maximum energy transferred to the rod (EMX) was calculated by integrating both the force and velocity records over time as follows:

$$EMX = \int F(t)V(t)dt$$

Where: $F(t)$ = the force at time t
$V(t)$ = the velocity at time t

The energy transfer ratio or efficiency is computed by dividing EMX by the theoretical SPT hammer energy of 350 lb-ft (computed from the product of the hammer weight, assumed to be the standard 140 lbs, and the fall height, assumed to be 2.5 ft). The SPT N values can then be corrected for a nominal 60% transfer efficiency, $N_{60,}$ as follows:

$$N_{60} = (e_m / 60)\ N_m$$

Where: e_m = the measured transfer ratio (ETR)
N_m = the measured SPT “N” value

Conclusions

Table 1 in Appendix B presents a summary of the average transferred energy and the energy transfer ratio for the single drill rig at each sample depth calculated using the *EMX* equation. Included in Table 1 are also average values of the hammer operating rate, maximum impact force and maximum velocity of the rod. The overall performance, which represents the average of data from all sample depths for each rig/rod type is also shown. Complete data, including the maximum, minimum and standard deviation for each sampling depth, is included in Appendix B.

For the Truck 170 mounted CME 75 drill rig at BH2, the average energy transfer ratio from individual sample depths ranged from 84.0% to 104.5%.

The average, overall transfer ratio (for all sampling depths weighted by N-values for each sample) were as follows:

SPT Rig (*Serial Number*)	Overall Transfer Efficiency	Hammer Operating Rate (BPM)
Truck 170 CME 75 drill rig at BH2	**92.6%**	**51.5**

Presented N_{60} values, provided in the Table 1 in Appendix B, does not account for any required corrections such as those for overburden or sampling spoon.

We appreciate the opportunity to be of assistance to you. Please do not hesitate to contact us if you have any questions regarding this report, or if we may be of further service.

Respectfully,

GRL Engineers, Inc.

REGISTERED PROFESSIONAL ENGINEER
Camilo A Alvarez
No. 67938
Exp. 6/30/19
Civil
STATE OF CALIFORNIA

Camilo Alvarez, P.E.

William Chambers, CPEng RPEQ

Summary of SPT Test Results

Project: MOORE TWINING-TRUCK 170-SPT SOIL SAMPLER, Test Date: 6/2/2022

FMX: Maximum Force
VMX: Maximum Velocity
BPM: Blows/Minute

EFV: Maximum Energy
ETR: Energy Transfer Ratio - Rated

Instr. Length ft	Blows Applied /6"	**N Value**	**N60 Value**	Average FMX kips	Average VMX ft/s	Average BPM bpm	Average EFV ft-lb	Average ETR %
13.59	**5-11-18**	**29**	**45**	27	17.0	52.8	316	90.4
18.59	**3-15-18**	**33**	**51**	27	17.4	53.0	327	93.5
18.59	**8-12-22**	**34**	**53**	28	18.4	51.3	344	98.3
			Overall Average Values:	27	17.7	52.4	330	94.2
			Standard Deviation:	1	1.3	0.8	17	5.0
			Overall Maximum Value:	30	20.0	53.2	366	104.5
			Overall Minimum Value:	25	14.2	51.0	294	84.0

Summary of SPT Test Results

Project: MOORE TWINING-TRUCK 170-SPT SOIL SAMPLER, Test Date: 6/2/2022

FMX: Maximum Force
VMX: Maximum Velocity
BPM: Blows/Minute

EFV: Maximum Energy
ETR: Energy Transfer Ratio - Rated

Instr. Length ft	Blows Applied /6"	**N Value**	**N60 Value**	Average FMX kips	Average VMX ft/s	Average BPM bpm	Average EFV ft-lb	Average ETR %
23.59	**4-9-13**	**22**	**32**	27	19.2	50.5	331	94.4
23.59	**7-14-16**	**30**	**44**	27	16.4	50.1	302	86.2
			Overall Average Values:	27	17.6	50.3	314	89.7
			Standard Deviation:	0	1.4	0.2	17	4.9
			Overall Maximum Value:	28	20.1	50.7	354	101.3
			Overall Minimum Value:	26	16.2	49.9	296	84.5

Appendix A

Introduction to SPT Dynamic Pile Testing

APPENDIX A
AN INTRODUCTION INTO SPT DYNAMIC PILE TESTING

1. BACKGROUND

The Standard Penetration Test is frequently conducted as an in-situ assessment of soil strength. This test requires that a 140 lb weight is dropped 30 inches onto a drive rod at whose bottom a sampler is usually installed. The sampler is driven for 18 inches; the number of blows required for the last 12 inches of driving is the so-called N-value. The N-value may be used as a strength indicator for foundation design or as a means of assessing the liquefaction potential of soils.

Obviously, the SPT hammer efficiency is an important consideration when using the N-values for design purposes. Measurements have indicated that the energy in the drive rod is sometimes only 30% and and may reach 90% of the potential or rated energy of the SPT hammer (E-rated = 0.35 kip-ft or 0.475 kJ). The type of hammer used to drive the rod is the main reason for these variations. On the average, the energy in the drive rod is 60% of the standard rated energy.

Because of the variability of energy, methods based on N-values are considered unreliable. However, measurements during SPT testing using the Case Method can be done on a routine basis and these measurements yield the transferred energy values. With measured energy, EMX, known, an adjustment of the measured N-value, N_m, can be made as follows.

$$N_{60} = N_m \, [E_m \, / \, (0.6E_r \,)] \qquad (1)$$

Thus, if the measured energy value is equal to the normally expected transferred energy of 60% of E-rated then the adjusted and measured N-values are identical. On the other hand, if the measured energy is only 30% then the adjusted blow count will be reduced by 50%.

2. DYNAMIC TESTING AND ANALYSIS METHODS APPLIED TO SPT

The Case Method of dynamic pile testing, named after the Case Institute of Technology where it was developed between 1964 and 1975, requires that a substantial ram mass (e.g. a pile driving hammer) impacts the pile top such that the pile undergoes at least a small permanent set. Thus, the method is also referred to as a “High Strain Method”. The Case Method requires dynamic measurements on the pile or shaft under the ram impact and then a calculation of various quantities. Conveniently, for SPT applications, the measurements and analyses are done by a single piece of equipment: the SPT Analyzer. The Pile Driving Analyzer® (PDA) is also suitable to perform these measurements and data processing.

A related analysis method is the “Wave Equation Analysis” which calculates a relationship between bearing capacity, pile stresses, transferred energy and field blow count. The GRLWEAP™ program performs this analysis and provides a complete set of helpful information and input data. This program can be used very effectively to simulate the SPT driving process.

3. MEASUREMENTS

GRL uses equipment manufactured by Pile Dynamics, Inc. The system includes either an SPT-Analyzer™ (SPTA) or a Pile Driving Analyzer® (PDA), an instrumented rod section and two accelerometers. SPT energy testing is very closely related to and borrows procedures from dynamic pile testing. Those interested in the basis of the SPT energy testing method may obtain extensive literature on dynamic pile testing from GRL Engineers, Inc.

3.1 SPT Analyzer or Pile Driving Analyzer

The basis for the results calculated by the SPTA or PDA are strain and acceleration measured in an instrumented rod section. These signals are converted to rod top force, F(t), and rod top velocity, v(t). The SPTA or PDA conditions, calibrates and displays these signals and immediately computes average pile force and velocity thereby eliminating bending effects. The product of these two measurements is then integrated over time which yields the energy transferred to the instrumented section as a function of time (see Section 4.1).

For convenience and accuracy, strain measurements are usually taken on an instrumented section of SPT drive rod. Ideally, the section properties of the instrumented rod and those of the drive rod are the same, however, using subs, other sections can also be utilized.

For the instrumented section, PDI provides a force calibration in such a way that the output of the instrumented rod is directly calculated without the need for an accurate elastic modulus or cross sectional area of the rod section.

The acceleration measurements are often demanding in the SPT environment, because of high frequency and high acceleration motion components. An experienced measurement engineer, therefore, has to evaluate the quality of this data before final conclusions are drawn from the numerical results calculated by SPTA or PDA.

SPTA or PDA records are taken while the standard N-value is acquired in the conventional manner. This then allows a direct correlation between N-value and average transferred energy.

3.2 HPA

The SPT hammer's ram velocity may be directly obtained using radar technology in the Hammer Performance Analyzer™. The impact velocity results can be automatically processed with a PC or recorded on a strip chart. HPA measurements yield a hammer kinetic energy, but not the energy transferred to the drive rod.

4 RECORD EVALUATION BY SPTA OR PDA

4.1 HAMMER PERFORMANCE

The PDA calculates the energy transferred to the pile top from:

$$E(t) = \int_0^t F(\tau)v(\tau)\, d\tau \qquad (2)$$

The maximum of the E(t) curve is often called **ENTHRU or EMX;** it is the most important quantity for an overall evaluation of the performance of a hammer and driving system. **EMX** allows for a classification of the hammer's performance when presented as, e_T, the rated transfer efficiency, also called energy transfer ratio (**ETR**) or global efficiency.

$$e_T = EMX/E_R \qquad (3)$$

where E_R is the hammer manufacturer's rated energy value or 0.35 kip-ft (0.475 kJ) in the case of the SPT hammer.

Often in the SPT literature one finds also reference to the EF2 energy. This evaluation is based on assumed proportionality between force and velocity (see also Section 5):

$$v(t) = F(t) / Z \qquad (4)$$

where Z = EA/c is the pile impedance, E is the elastic modulus, A is the cross sectional area and c is the speed of the stress wave in the pile material..

Combining equations 2 and 4 leads to

$$EF(t) = \int_0^t F(\tau)^2 / Z\, d\tau \qquad (5)$$

The EF2 transferred energy value is the EF-value at the time t = 2L/c, where L is the drive rod length and c is the stress wave speed in steel (16,800 ft/s or 5,124 m/s). Since the force is easier to measure than both force and velocity, Equation 5 is preferred by some test engineers. However, the EF method is fraught with errors and certain correction factors have to be applied to make it approximately correct. Among the error sources are the following:

- Proportionality is often violated prior to time 2L/c. The proportionality between force and velocity in a downward traveling wave only holds if the wave does not encounter a disturbance prior to reflecting off the pile toe. Such disturbances include a change in cross sectional area, an open or loose splice or joint, or resistance along the shaft.

- Using only one force measurement precludes a data quality check based on the proportionality between force and velocity. Thus, a force measurement that is for some reason in error may not be detectable, which will lead to errors in the EF2 value. Data quality checks will be discussed further in Section 5.

The use if EF2 is therefore not recommended but it is often included in result presentations for the sake of completeness.

4.2 STRESSES

During SPT monitoring, it is also of interest to monitor compressive stresses at both the top of the drive rod and at its bottom.

At the pile top (location of sensors) the maximum compression stress averaged over the rod's cross section, **CSX**, is directly obtained from the measurements. Note that this stress value refers to the instrumented section. If the rod has a different cross sectional area then the stress in the rod will be different from CSX.

The SPTA or PDA can also calculate, in an approximate manner, the force at the rod bottom, **CFB**. To obtain the corresponding stress, this force value should be divided by the appropriate cross sectional area, e.g. by the rod area just above the sampler or by the sampler area itself. Of course, non-uniform stress components as they might occur at the sampler tip due to a sloping rock are not considered in this calculation.

5. DATA QUALITY CHECKS

Quality data is the first and foremost requirement for accurate dynamic testing results. It is therefore important that the measurement engineer performing SPTA or PDA tests has the experience necessary to recognize measurement problems and take appropriate corrective action should problems develop. Fortunately, dynamic pile testing allows for certain data quality checks because two independent measurements are taken that have to conform to the so-called proportionality relationship.

As long as there is only a wave traveling in one direction, as is the case during impact when only a downward traveling wave exists in the rod, force and velocity measured at its top are proportional

$$F = v\,Z \qquad (5)$$

where Z is again the pile impedance, Z = EA/c. This relationship can also be expressed in terms of stress

$$\sigma = F/A = v\,(E/c) \qquad (6)$$

or strain

$$\epsilon = \sigma/E = v / c \qquad (7)$$

This means that the early portion of strain times wave speed must be equal to the velocity unless the proportionality is affected by high friction near the pile top or by a pile cross sectional change not far below the sensors. Checking the proportionality is an excellent means of assuring meaningful measurements but is only truly meaningful for perfectly uniform rods. Open or loose splices, for example, will lead to a non-proportionality. For SPT rods it is fortunate that usually no soil resistance acts along the shaft and for that reason, proportionality can exist until the stress wave returns from sampler top or rod bottom unless connectors are not sufficiently tightened or have a significant mass.

Velocity data quality can also be checked by looking at the final displacement, DFN, which is calculated from the acceleration by double integration. If the calculated final displacement is much higher or lower than indicated by the N-value, the accelerometer attachment may be loose or the sensor may be faulty. If major drift in the velocity is observed, the EMX value may be in error, even though proportionality from impact to time 2L/c exists. In this case, it may be useful to evaluate the energy transferred to the drill rod at time 2L/c, which is calculated by the PDA or SPTA as the E2E quantity.

Appendix B

SPT Analyses Results

Summary of SPT Test Results

Project: MOORE TWINING-TRUCK 170-SPT SOIL SAMPLER, Test Date: 6/2/2022

FMX: Maximum Force
VMX: Maximum Velocity
BPM: Blows/Minute
EFV: Maximum Energy
ETR: Energy Transfer Ratio - Rated

Instr. Length ft	Blows Applied /6"	**N Value**	**N60 Value**	Average FMX kips	Average VMX ft/s	Average BPM bpm	Average EFV ft-lb	Average ETR %
13.59	**5-11-18**	**29**	**45**	27	17.0	52.8	316	90.4
18.59	**3-15-18**	**33**	**51**	27	17.4	53.0	327	93.5
18.59	**8-12-22**	**34**	**53**	28	18.4	51.3	344	98.3
			Overall Average Values:	27	17.7	52.4	330	94.2
			Standard Deviation:	1	1.3	0.8	17	5.0
			Overall Maximum Value:	30	20.0	53.2	366	104.5
			Overall Minimum Value:	25	14.2	51.0	294	84.0

Summary of SPT Test Results

Project: MOORE TWINING-TRUCK 170-SPT SOIL SAMPLER, Test Date: 6/2/2022

FMX: Maximum Force
VMX: Maximum Velocity
BPM: Blows/Minute

EFV: Maximum Energy
ETR: Energy Transfer Ratio - Rated

Instr. Length ft	Blows Applied /6"	**N Value**	**N60 Value**	Average FMX kips	Average VMX ft/s	Average BPM bpm	Average EFV ft-lb	Average ETR %
23.59	**4-9-13**	**22**	**32**	27	19.2	50.5	331	94.4
23.59	**7-14-16**	**30**	**44**	27	16.4	50.1	302	86.2
			Overall Average Values:	27	17.6	50.3	314	89.7
			Standard Deviation:	0	1.4	0.2	17	4.9
			Overall Maximum Value:	28	20.1	50.7	354	101.3
			Overall Minimum Value:	26	16.2	49.9	296	84.5

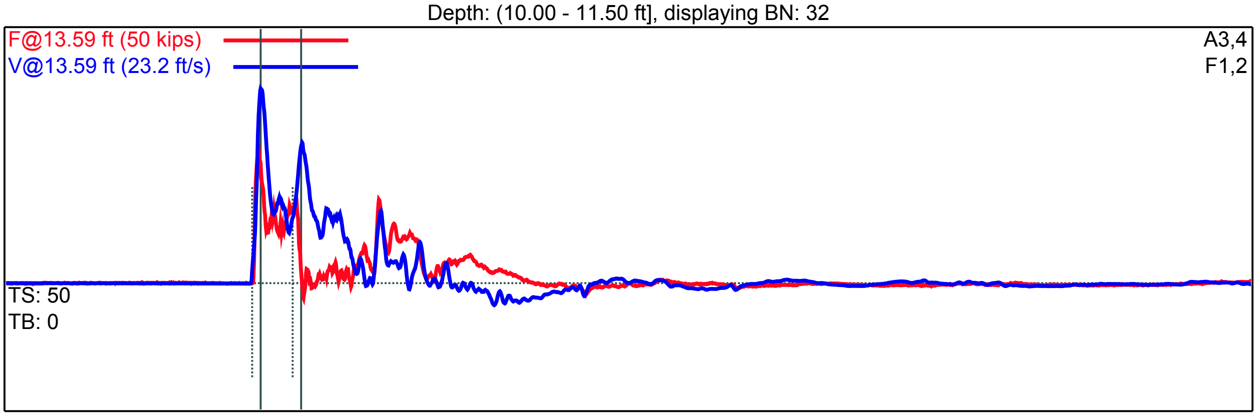
MOORE TWINING-TRUCK 170-SPT SOIL SAMPLER
pc
BH1

10-11.5
Interval start: 6/2/2022

AR: 1.21 in^2
LE: 13.59 ft
WS: 16807.9 ft/s

SP: 0.492 k/ft3
EM: 30000 ksi

Depth: (10.00 - 11.50 ft], displaying BN: 32



F1 : [652AWJ1] 223.3 PDICAL (1) FF1
F2 : [652AWJ2] 222.94 PDICAL (1) FF1

A3 (PR): [K10815] 418.274 mv/6.4v/5000g (1) VF1
A4 (PR): [K10814] 390 mv/6.4v/5000g (1) VF1

FMX: Maximum Force
VMX: Maximum Velocity
BPM: Blows/Minute

EFV: Maximum Energy
ETR: Energy Transfer Ratio - Rated

BL#	BC /6"	FMX kips	VMX ft/s	BPM bpm	EFV ft-lb	ETR %
1	5	26	18.6	1.9	249	71.2
2	5	26	17.3	52.9	299	85.5
3	5	27	17.7	52.7	312	89.2
4	5	27	17.0	53.1	304	86.9
5	5	28	16.6	52.8	313	89.5
6	11	27	16.0	53.0	299	85.5
7	11	27	16.6	52.8	317	90.5
8	11	27	15.8	52.8	308	87.9
9	11	28	16.4	53.0	312	89.3
10	11	27	15.4	52.8	310	88.5
11	11	27	16.8	53.0	319	91.2
12	11	27	16.2	52.8	313	89.4
13	11	26	16.0	53.1	316	90.2
14	11	27	16.6	52.9	312	89.3
15	11	28	16.9	53.0	326	93.2
16	11	26	17.0	52.8	325	92.9
17	18	26	16.9	52.9	313	89.4
18	18	26	17.2	52.8	327	93.3
19	18	27	17.4	52.8	318	90.9
20	18	27	17.6	52.8	326	93.0
21	18	27	17.4	52.8	325	93.0
22	18	27	17.5	52.8	322	91.9
23	18	27	17.5	52.7	322	91.9
24	18	27	17.6	52.8	321	91.6
25	18	27	17.6	52.7	324	92.6
26	18	27	17.1	52.9	318	90.7
27	18	28	17.6	52.7	310	88.7
28	18	27	16.9	52.7	319	91.1

29	18	28	17.8	52.7	321	91.6
30	18	27	16.9	52.7	307	87.8
31	18	27	18.0	52.8	315	89.9
32	18	27	17.6	52.7	312	89.2
33	18	26	17.2	52.8	303	86.7
34	18	26	17.6	52.6	313	89.5
	Average	27	17.0	52.8	316	90.4
	Std Dev	1	0.6	0.1	7	2.0
	Maximum	28	18.0	53.1	327	93.3
	Minimum	26	15.4	52.6	299	85.5

N-value: 29

BN: 34 5/11/18

Sample Interval Time: 37.53 seconds.

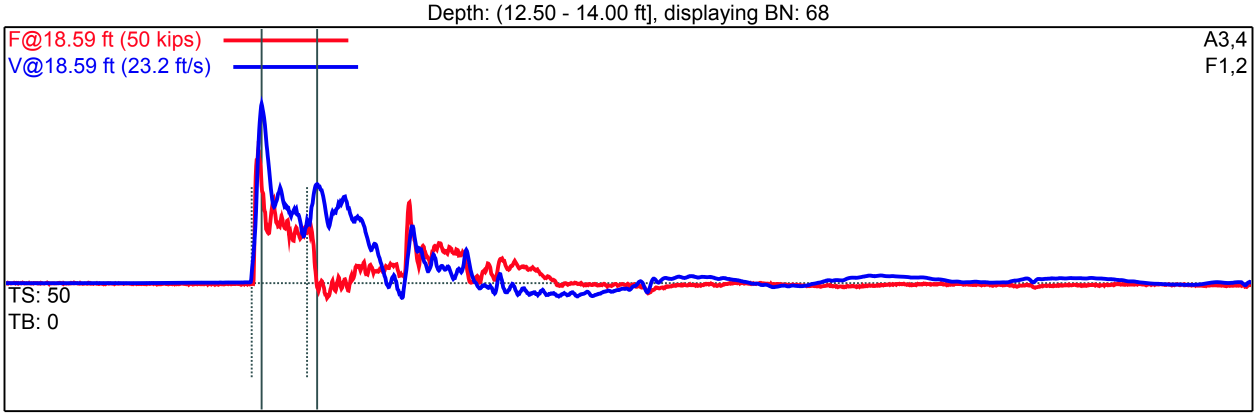
MOORE TWINING-TRUCK 170-SPT SOIL SAMPLER
pc
BH1

10-11.5
Interval start: 6/2/2022

AR: 1.21 in^2
LE: 18.59 ft
WS: 16807.9 ft/s

SP: 0.492 k/ft3
EM: 30000 ksi

Depth: (12.50 - 14.00 ft], displaying BN: 68



F1 : [652AWJ1] 223.3 PDICAL (1) FF1
F2 : [652AWJ2] 222.94 PDICAL (1) FF1

A3 (PR): [K10815] 418.274 mv/6.4v/5000g (1) VF1
A4 (PR): [K10814] 390 mv/6.4v/5000g (1) VF1

BL#	BC	FMX	VMX	BPM	EFV	ETR
	/6"	kips	ft/s	bpm	ft-lb	%
35	3	28	18.3	1.9	328	93.8
36	3	27	17.4	52.9	309	88.3
37	3	28	18.9	52.7	352	100.4
38	15	27	18.0	53.1	326	93.2
39	15	28	17.1	53.0	317	90.7
40	15	28	18.5	53.2	337	96.2
41	15	28	17.6	53.0	320	91.4
42	15	28	17.6	53.1	325	93.0
43	15	27	17.8	53.1	325	92.8
44	15	28	17.8	53.2	322	92.0
45	15	28	18.9	53.1	339	96.7
46	15	27	17.9	53.1	327	93.4
47	15	26	18.1	53.1	332	94.8
48	15	26	16.3	52.9	312	89.1
49	15	26	18.2	52.8	337	96.3
50	15	25	16.4	53.0	315	89.9
51	15	25	18.3	53.0	339	96.9
52	15	27	16.0	52.9	317	90.5
53	18	25	18.8	52.7	346	98.9
54	18	27	16.5	52.9	318	90.8
55	18	26	18.7	53.0	348	99.5
56	18	27	16.1	52.9	316	90.4
57	18	27	19.0	52.7	348	99.4
58	18	27	16.1	52.9	316	90.2
59	18	26	18.9	53.0	347	99.1
60	18	26	14.2	53.0	300	85.8
61	18	26	18.8	52.8	347	99.0
62	18	27	14.2	52.9	299	85.5
63	18	26	18.6	53.0	344	98.2
64	18	26	16.1	52.9	314	89.7
65	18	25	18.6	53.1	341	97.4

66	18	26	14.8	53.0	294	84.0
67	18	26	20.0	53.1	353	100.9
68	18	26	16.2	53.0	309	88.2
69	18	27	19.5	53.1	354	101.2
70	18	27	16.2	53.2	313	89.6
	Average	27	17.4	53.0	327	93.5
	Std Dev	1	1.5	0.1	16	4.6
	Maximum	28	20.0	53.2	354	101.2
	Minimum	25	14.2	52.7	294	84.0

N-value: 33

BN: 70 3 / 15 / 18

Sample Interval Time: 39.64 seconds.

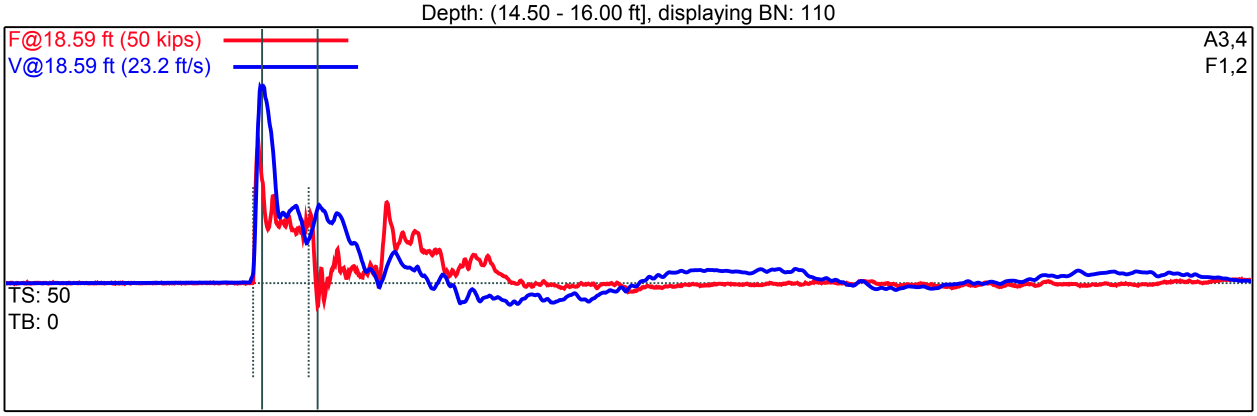
MOORE TWINING-TRUCK 170-SPT SOIL SAMPLER
pc
BH1

10-11.5
Interval start: 6/2/2022

AR: 1.21 in^2
LE: 18.59 ft
WS: 16807.9 ft/s

SP: 0.492 k/ft3
EM: 30000 ksi

Depth: (14.50 - 16.00 ft], displaying BN: 110



F1 : [652AWJ1] 223.3 PDICAL (1) FF1
F2 : [652AWJ2] 222.94 PDICAL (1) FF1

A3 (PR): [K10815] 418.274 mv/6.4v/5000g (1) VF1
A4 (PR): [K10814] 390 mv/6.4v/5000g (1) VF1

BL#	BC	FMX	VMX	BPM	EFV	ETR
	/6"	kips	ft/s	bpm	ft-lb	%
71	8	28	18.0	1.9	304	86.9
72	8	28	19.3	51.2	334	95.3
73	8	28	18.2	51.6	324	92.6
74	8	28	19.5	51.4	335	95.6
75	8	28	18.9	51.4	329	94.0
76	8	28	17.6	51.4	315	90.0
77	8	27	19.5	51.5	345	98.5
78	8	27	19.0	51.5	338	96.5
79	12	29	19.8	51.4	343	98.0
80	12	29	19.2	51.5	340	97.2
81	12	28	20.0	51.4	356	101.7
82	12	29	19.2	51.5	343	98.1
83	12	29	19.8	51.3	361	103.1
84	12	29	19.8	51.5	351	100.2
85	12	29	19.6	51.4	362	103.3
86	12	29	19.1	51.4	359	102.6
87	12	28	19.6	51.5	357	102.1
88	12	28	18.8	51.5	352	100.7
89	12	29	18.7	51.5	359	102.4
90	12	28	18.3	51.3	352	100.5
91	22	29	18.3	51.5	354	101.2
92	22	30	19.1	51.3	366	104.5
93	22	27	18.1	51.5	324	92.5
94	22	28	18.5	51.4	336	96.1
95	22	29	18.5	51.4	354	101.3
96	22	29	17.5	51.5	336	95.9
97	22	29	17.9	51.3	332	94.8
98	22	29	18.8	51.3	350	100.0
99	22	28	17.7	51.2	331	94.4
100	22	29	16.5	51.4	314	89.7
101	22	29	17.1	51.2	328	93.6

102	22	29	18.0	51.3	345	98.4
103	22	27	15.7	51.2	307	87.8
104	22	28	18.8	51.2	348	99.4
105	22	28	16.8	51.2	321	91.7
106	22	28	18.3	51.2	335	95.7
107	22	28	18.6	51.3	335	95.6
108	22	28	19.8	51.2	363	103.6
109	22	28	18.6	51.3	353	100.9
110	22	28	17.8	51.0	347	99.2
111	22	28	17.9	51.3	348	99.5
112	22	28	16.1	51.0	333	95.1
	Average	28	18.4	51.3	344	98.3
	Std Dev	1	1.1	0.1	14	4.1
	Maximum	30	20.0	51.5	366	104.5
	Minimum	27	15.7	51.0	307	87.8

N-value: 34

BN: 112 8 / 12/ 22

Sample Interval Time: 47.89 seconds.

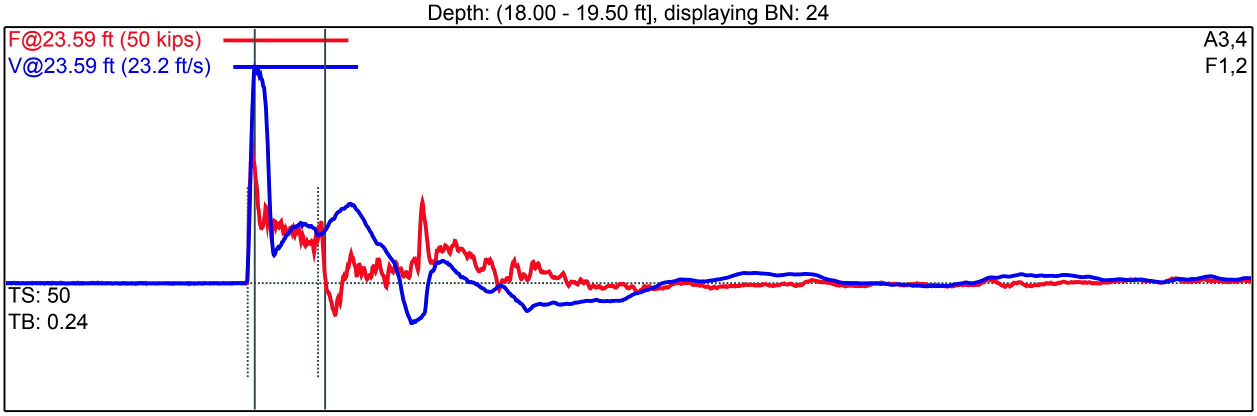
MOORE TWINING-TRUCK 170-SPT SOIL SAMPLER
pc
BH1

18.0- 19.5
Interval start: 6/2/2022

AR: 1.21 in^2
LE: 23.59 ft
WS: 16807.9 ft/s

SP: 0.492 k/ft3
EM: 30000 ksi

Depth: (18.00 - 19.50 ft], displaying BN: 24



F1 : [652AWJ1] 223.3 PDICAL (1) FF1
F2 : [652AWJ2] 222.94 PDICAL (1) FF1

A3 (PR): [K10815] 418.274 mv/6.4v/5000g (1) VF1
A4 (PR): [K10814] 390 mv/6.4v/5000g (1) VF1

FMX: Maximum Force
VMX: Maximum Velocity
BPM: Blows/Minute

EFV: Maximum Energy
ETR: Energy Transfer Ratio - Rated

BL#	BC	FMX	VMX	BPM	EFV	ETR
	/6"	kips	ft/s	bpm	ft-lb	%
2	4	27	19.5	50.5	344	98.2
3	4	27	19.6	50.5	352	100.5
4	4	27	19.7	50.7	361	103.1
5	9	26	19.8	50.5	343	97.9
6	9	27	20.1	50.7	354	101.3
7	9	27	19.8	50.5	353	101.0
8	9	27	19.9	50.5	352	100.6
9	9	26	20.0	50.4	337	96.3
10	9	26	19.6	50.7	321	91.6
11	9	26	19.1	50.4	311	88.8
12	9	27	19.8	50.4	343	98.1
13	9	27	17.8	50.5	309	88.4
14	13	27	19.5	50.5	341	97.5
15	13	27	18.3	50.5	305	87.1
16	13	27	18.7	50.6	322	92.1
17	13	26	18.8	50.5	330	94.3
18	13	27	19.4	50.4	330	94.4
19	13	27	18.8	50.5	337	96.2
20	13	27	18.6	50.5	333	95.3
21	13	26	19.2	50.5	331	94.7
22	13	26	18.6	50.5	316	90.3
23	13	27	19.4	50.4	332	94.9
24	13	27	19.5	50.6	326	93.2
25	13	27	19.6	50.5	323	92.3
26	13	27	18.5	50.7	320	91.4

Average	27	19.2	50.5	331	94.4
Std Dev	0	0.6	0.1	14	3.9
Maximum	27	20.1	50.7	354	101.3
Minimum	26	17.8	50.4	305	87.1

N-value: 22

BN: 26 4 / 9 /13

Sample Interval Time: 28.56 seconds.

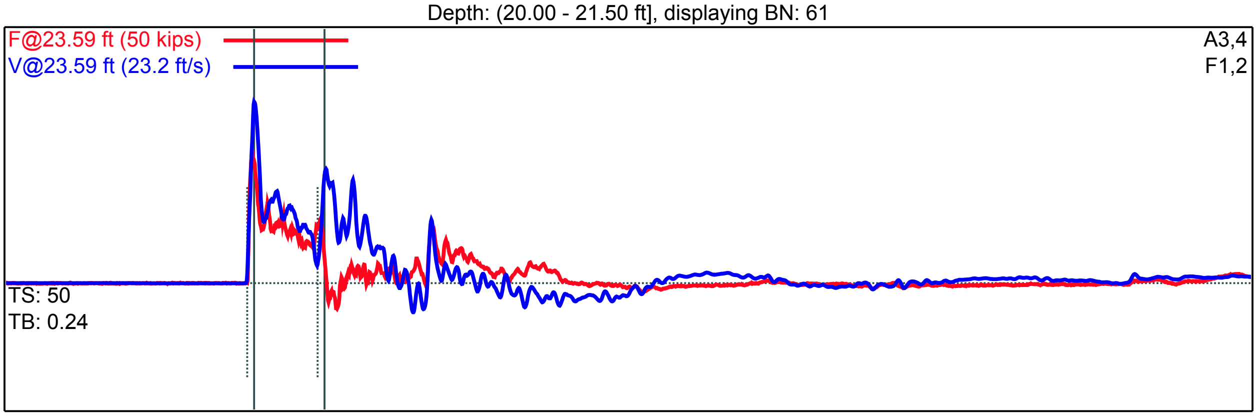
MOORE TWINING-TRUCK 170-SPT SOIL SAMPLER
pc
BH1

18.0- 19.5
Interval start: 6/2/2022

AR: 1.21 in^2
LE: 23.59 ft
WS: 16807.9 ft/s

SP: 0.492 k/ft3
EM: 30000 ksi

Depth: (20.00 - 21.50 ft], displaying BN: 61



F1 : [652AWJ1] 223.3 PDICAL (1) FF1
F2 : [652AWJ2] 222.94 PDICAL (1) FF1

A3 (PR): [K10815] 418.274 mv/6.4v/5000g (1) VF1
A4 (PR): [K10814] 390 mv/6.4v/5000g (1) VF1

BL#	BC	FMX	VMX	BPM	EFV	ETR
	/6"	kips	ft/s	bpm	ft-lb	%
27	7	28	16.1	1.9	299	85.5
28	7	26	16.5	50.2	309	88.2
29	7	27	16.6	50.0	311	88.9
30	7	27	16.7	50.1	305	87.3
31	7	27	16.7	50.1	308	88.0
32	7	27	16.4	50.1	312	89.0
33	7	26	16.7	50.2	313	89.3
34	14	27	16.7	50.2	302	86.4
35	14	27	16.6	50.2	307	87.9
36	14	27	16.9	50.1	309	88.2
37	14	27	16.6	50.0	300	85.7
38	14	27	16.5	50.2	302	86.2
39	14	27	16.2	50.0	304	86.9
40	14	27	16.4	50.2	301	86.1
41	14	27	16.4	50.1	305	87.0
42	14	27	16.4	50.3	303	86.6
43	14	27	16.4	50.0	305	87.1
44	14	27	16.2	50.1	301	86.0
45	14	27	16.3	50.1	302	86.3
46	14	27	16.3	50.1	296	84.5
47	14	27	16.3	50.1	297	84.9
48	16	27	16.3	50.0	304	86.9
49	16	27	16.2	50.2	298	85.0
50	16	28	16.4	49.9	299	85.4
51	16	27	16.4	50.1	299	85.5
52	16	27	16.3	49.9	298	85.2
53	16	27	16.4	50.0	300	85.8
54	16	27	16.5	50.0	297	84.9
55	16	27	16.6	50.0	300	85.7
56	16	27	16.3	50.0	301	86.1
57	16	27	16.5	50.0	306	87.5

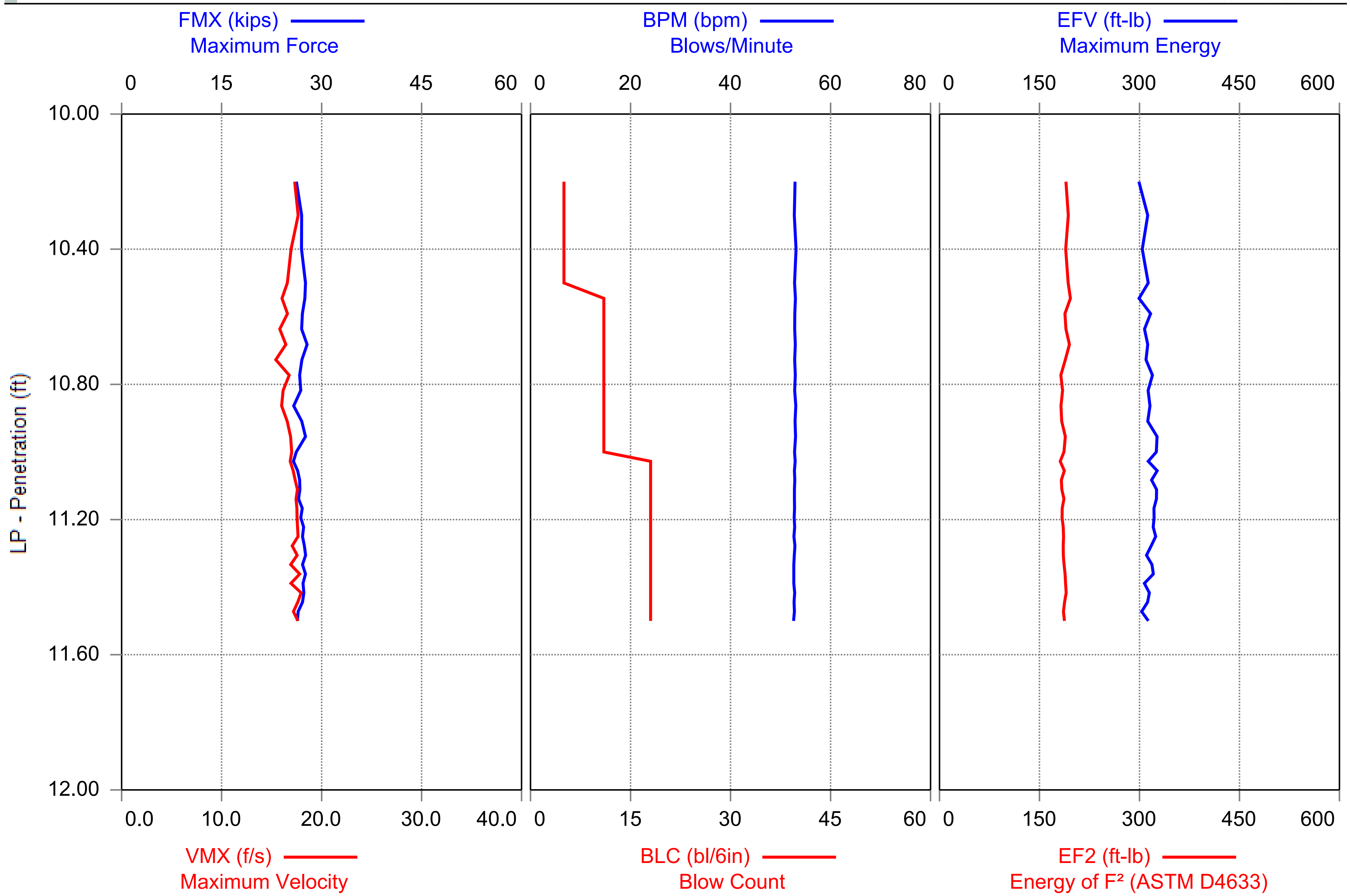
58	16	27	16.5	50.1	303	86.6
59	16	27	16.4	49.9	306	87.4
60	16	26	16.5	50.1	302	86.3
61	16	26	16.4	50.0	298	85.2
62	16	26	16.4	50.0	300	85.7
63	16	26	16.5	50.0	303	86.5
	Average	27	16.4	50.1	302	86.2
	Std Dev	0	0.1	0.1	3	0.9
	Maximum	28	16.9	50.3	309	88.2
	Minimum	26	16.2	49.9	296	84.5

N-value: 30

BN: 63 7 / 14 / 16

Sample Interval Time: 43.20 seconds.

GRL Engineers, Inc. - PDIPLOT2 Ver 2021.1.61.0 - Case Method & iCAP® Results
Printed: 03-June-2022 Test started: 02-June-2022
MOORE TWINING-TRUCK 170-SPT SOIL SAMPLER - 10-11.5



Case Method & iCAP® Results

MOORE TWINING-TRUCK 170-SPT SOIL SAMPLER - 10-11.5 BH1
OP: pc Date: 02-June-2022

AR: 1.21 in² SP: 0.492 k/ft³
LE: 13.59 ft EM: 30,000 ksi
WS: 16,807.9 f/s JC: 0.00

FMX: Maximum Force
VMX: Maximum Velocity
BPM: Blows/Minute
EFV: Maximum Energy
EF2: Energy of F² (ASTM D4633)
ETR: Energy Transfer Ratio - Rated
RAT: Length Ratio for SPT

BL#	Depth ft	BLC bl/6in	TYPE	FMX kips	VMX f/s	BPM bpm	EFV ft-lb	EF2 ft-lb	ETR (%)	RAT
5	10.50	5	AV4	27	17.1	52.9	307.2	191.3	87.8	1.2
			STD	0	0.4	0.1	5.8	1.8	1.7	0.0
			MAX	28	17.7	53.1	313.1	193.2	89.5	1.2
			MIN	26	16.6	52.7	299.1	189.2	85.5	1.2
16	11.00	11	AV11	27	16.3	52.9	314.3	187.8	89.8	1.2
			STD	1	0.5	0.1	7.4	4.5	2.1	0.0
			MAX	28	17.0	53.1	326.4	196.3	93.2	1.2
			MIN	26	15.4	52.8	299.3	182.1	85.5	1.2
34	11.50	18	AV18	27	17.4	52.8	317.5	186.0	90.7	1.2
			STD	0	0.3	0.1	6.4	2.2	1.8	0.0
			MAX	28	18.0	52.9	326.5	189.9	93.3	1.3
			MIN	26	16.9	52.6	303.3	181.1	86.7	1.2
			Average	27	17.0	52.8	315.2	187.2	90.1	1.2
			Std. Dev.	1	0.6	0.1	7.5	3.6	2.1	0.0
			Maximum	28	18.0	53.1	326.5	196.3	93.3	1.3
			Minimum	26	15.4	52.6	299.1	181.1	85.5	1.2

Total number of blows analyzed: 33

BL# Sensors

2-34 F1: [652AWJ1] 223.3 (1.00); F2: [652AWJ2] 222.9 (1.00); A3: [K10815] 418.3 (1.00);
A4: [K10814] 390.0 (1.00)

BL# Comments

34 5/11/18

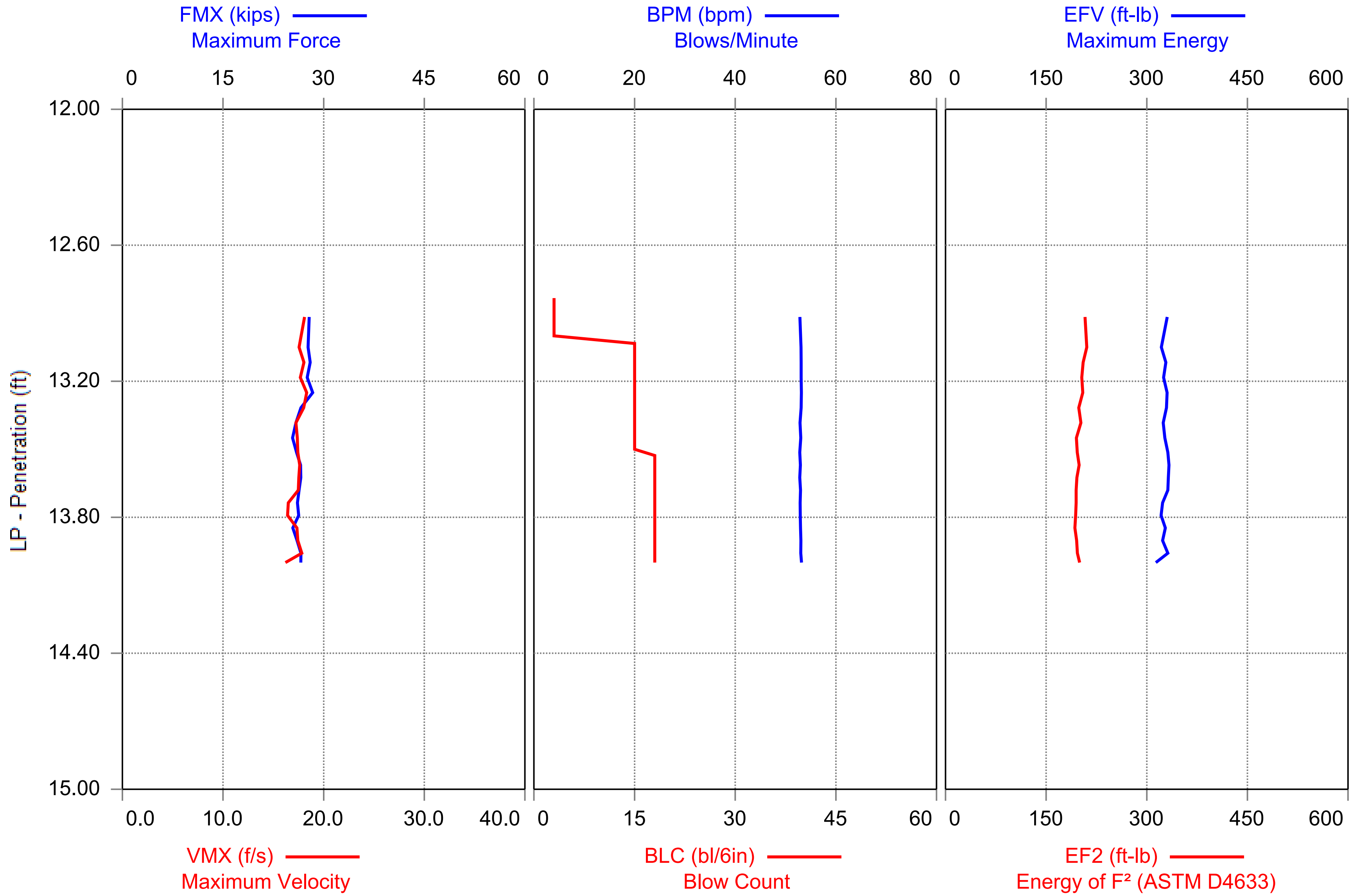
Time Summary

Drive 37 seconds 12:48 PM - 12:48 PM BN 1 - 34

GRL Engineers, Inc. - PDIPLOT2 Ver 2021.1.61.0 - Case Method & iCAP® Results

Printed: 03-June-2022 Test started: 02-June-2022

MOORE TWINING-TRUCK 170-SPT SOIL SAMPLER - 12.5 - 14.0



Case Method & iCAP® Results

MOORE TWINING-TRUCK 170-SPT SOIL SAMPLER - 12.5 - 14.0 BH1
OP: pc Date: 02-June-2022

AR: 1.21 in² SP: 0.492 k/ft³
LE: 18.59 ft EM: 30,000 ksi
WS: 16,807.9 f/s JC: 0.00

FMX: Maximum Force
VMX: Maximum Velocity
BPM: Blows/Minute
EFV: Maximum Energy
EF2: Energy of F² (ASTM D4633)
ETR: Energy Transfer Ratio - Rated
RAT: Length Ratio for SPT

BL#	Depth	BLC	TYPE	FMX	VMX	BPM	EFV	EF2	ETR	RAT
	ft	bl/6in		kips	f/s	bpm	ft-lb	ft-lb	(%)	
3	13.00	3	AV2	28	18.1	52.8	330.4	207.9	94.4	1.2
			STD	1	0.7	0.1	21.2	3.3	6.1	0.0
			MAX	28	18.9	52.9	351.6	211.2	100.4	1.2
			MIN	27	17.4	52.7	309.2	204.6	88.3	1.2
18	13.50	15	AV15	27	17.6	53.0	325.9	202.5	93.1	1.2
			STD	1	0.8	0.1	8.7	5.7	2.5	0.0
			MAX	28	18.9	53.2	339.0	211.7	96.9	1.2
			MIN	25	16.0	52.8	311.8	190.3	89.1	1.2
36	14.00	18	AV18	26	17.3	52.9	328.2	195.5	93.8	1.2
			STD	1	1.8	0.1	20.4	2.6	5.8	0.0
			MAX	27	20.0	53.2	354.2	199.8	101.2	1.2
			MIN	25	14.2	52.7	294.0	189.7	84.0	1.2
			Average	27	17.5	53.0	327.3	199.2	93.5	1.2
			Std. Dev.	1	1.4	0.1	16.6	5.8	4.7	0.0
			Maximum	28	20.0	53.2	354.2	211.7	101.2	1.2
			Minimum	25	14.2	52.7	294.0	189.7	84.0	1.2

Total number of blows analyzed: 35

BL# Sensors

2-36 F1: [652AWJ1] 223.3 (1.00); F2: [652AWJ2] 222.9 (1.00); A3: [K10815] 418.3 (1.00);
A4: [K10814] 390.0 (1.00)

BL# Comments

36 3 / 15 / 18

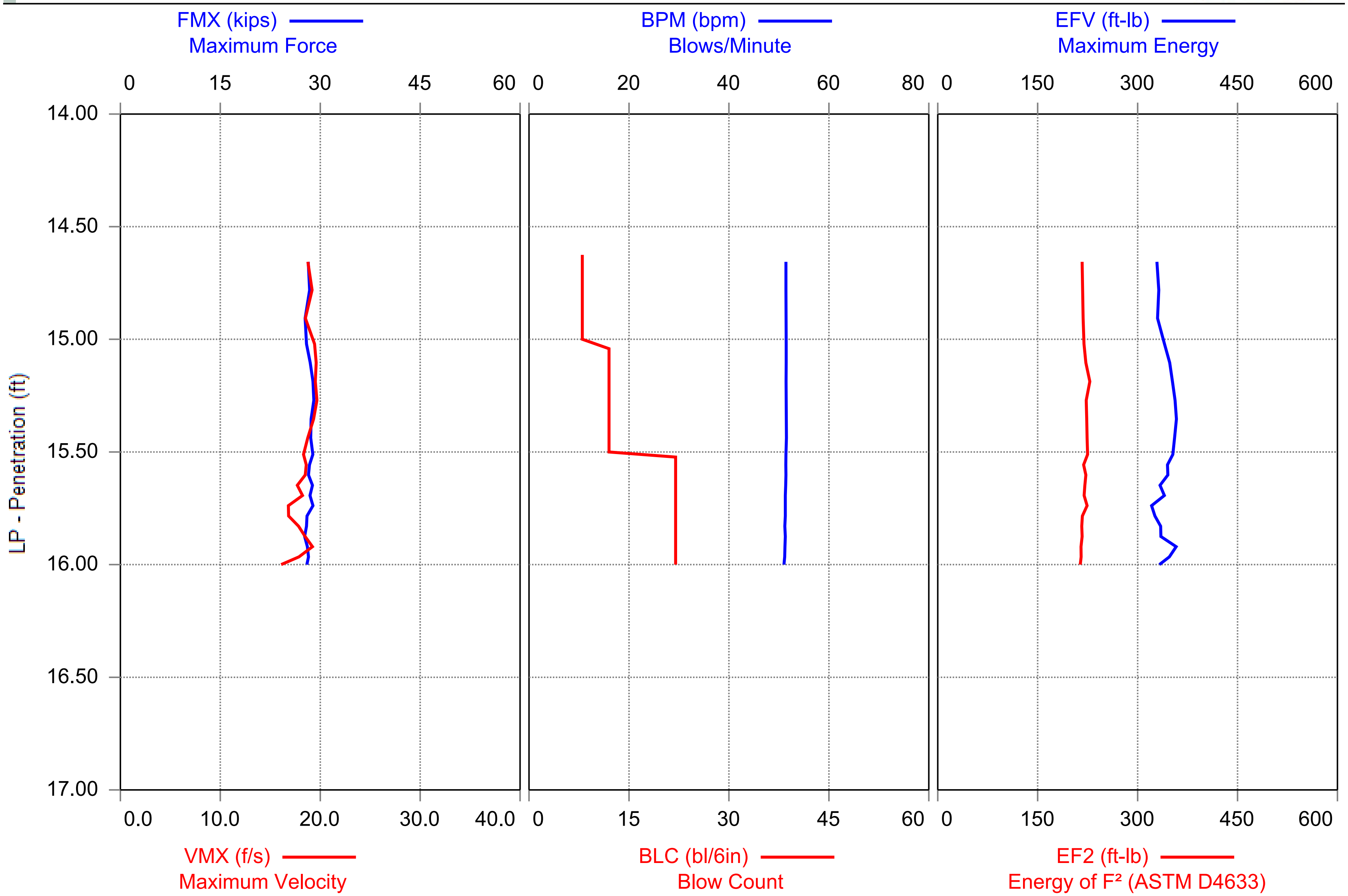
Time Summary

Drive 39 seconds 12:58 PM - 12:58 PM BN 1 - 36

GRL Engineers, Inc. - PDIPLOT2 Ver 2021.1.61.0 - Case Method & iCAP® Results

Printed: 03-June-2022 Test started: 02-June-2022

MOORE TWINING-TRUCK 170-SPT SOIL SAMPLER - 14.5 - 16.0



Case Method & iCAP® Results

MOORE TWINING-TRUCK 170-SPT SOIL SAMPLER - 14.5 - 16.0 BH1

OP: pc Date: 02-June-2022

AR: 1.21 in² SP: 0.492 k/ft³
LE: 18.59 ft EM: 30,000 ksi
WS: 16,807.9 f/s JC: 0.00

FMX: Maximum Force
VMX: Maximum Velocity
BPM: Blows/Minute
EFV: Maximum Energy
EF2: Energy of F² (ASTM D4633)
ETR: Energy Transfer Ratio - Rated
RAT: Length Ratio for SPT

BL#	Depth ft	BLC bl/6in	TYPE	FMX kips	VMX f/s	BPM bpm	EFV ft-lb	EF2 ft-lb	ETR (%)	RAT
8	15.00	8	AV7	28	18.8	51.4	331.3	217.1	94.7	1.1
			STD	1	0.7	0.1	8.9	2.1	2.5	0.0
			MAX	28	19.5	51.6	344.8	220.1	98.5	1.2
			MIN	27	17.6	51.2	315.2	213.5	90.0	1.1
20	15.50	12	AV12	29	19.3	51.4	352.9	224.1	100.8	1.1
			STD	0	0.5	0.1	7.0	4.0	2.0	0.0
			MAX	29	20.0	51.5	361.6	232.4	103.3	1.1
			MIN	28	18.3	51.3	340.2	218.2	97.2	1.1
42	16.00	22	AV22	28	17.9	51.3	339.0	218.7	96.9	1.1
			STD	1	1.0	0.1	14.8	5.5	4.2	0.0
			MAX	30	19.8	51.5	365.8	228.4	104.5	1.1
			MIN	27	15.7	51.0	307.4	210.1	87.8	1.1
			Average	28	18.5	51.4	341.8	220.0	97.6	1.1
			Std. Dev.	1	1.0	0.1	14.3	5.3	4.1	0.0
			Maximum	30	20.0	51.6	365.8	232.4	104.5	1.2
			Minimum	27	15.7	51.0	307.4	210.1	87.8	1.1

Total number of blows analyzed: 41

BL# Sensors

2-42 F1: [652AWJ1] 223.3 (1.00); F2: [652AWJ2] 222.9 (1.00); A3: [K10815] 418.3 (1.00);
A4: [K10814] 390.0 (1.00)

BL# Comments

42 8 / 12/ 22

Time Summary

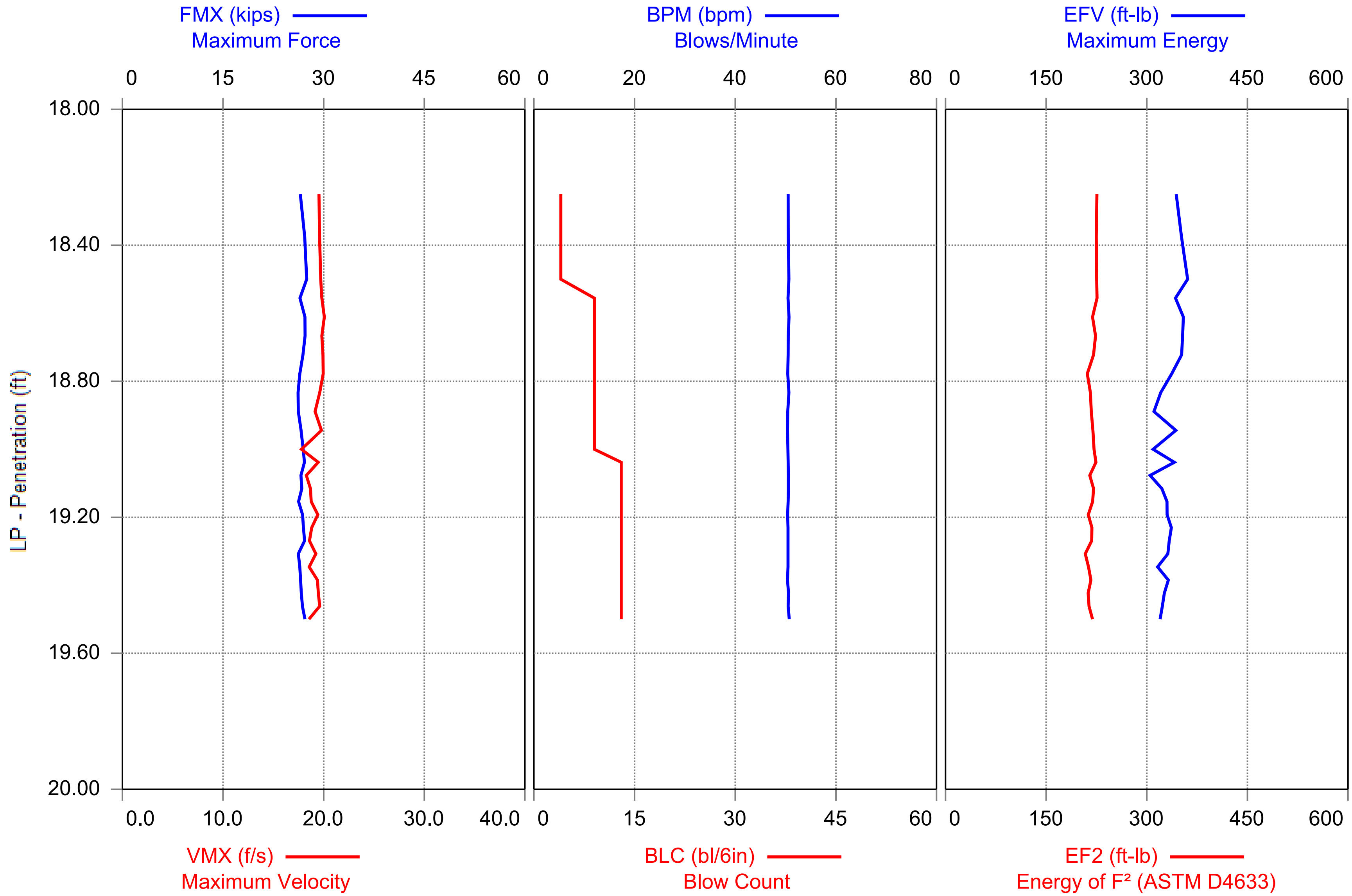
Drive 47 seconds 1:07 PM - 1:08 PM BN 1 - 42

GRL Engineers, Inc. - PDIPLOT2 Ver 2021.1.61.0 - Case Method & iCAP® Results

Printed: 03-June-2022

Test started: 02-June-2022

MOORE TWINING-TRUCK 170-SPT SOIL SAMPLER - 18.0- 19.5



Case Method & iCAP® Results

MOORE TWINING-TRUCK 170-SPT SOIL SAMPLER - 18.0- 19.5 BH1
OP: pc Date: 02-June-2022

AR: 1.21 in² SP: 0.492 k/ft³
LE: 23.59 ft EM: 30,000 ksi
WS: 16,807.9 f/s JC: 0.00

FMX: Maximum Force
VMX: Maximum Velocity
BPM: Blows/Minute
EFV: Maximum Energy
EF2: Energy of F² (ASTM D4633)
ETR: Energy Transfer Ratio - Rated
RAT: Length Ratio for SPT

BL#	Depth	BLC	TYPE	FMX	VMX	BPM	EFV	EF2	ETR	RAT
	ft	bl/6in		kips	f/s	bpm	ft-lb	ft-lb	(%)	
4	18.50	4	AV3	27	19.6	50.6	352.2	225.3	100.6	1.1
			STD	0	0.1	0.1	7.0	0.4	2.0	0.0
			MAX	27	19.7	50.7	360.9	225.7	103.1	1.1
			MIN	27	19.5	50.5	343.9	224.8	98.2	1.1
13	19.00	9	AV9	27	19.5	50.5	336.0	219.4	96.0	1.1
			STD	0	0.7	0.1	16.9	4.0	4.8	0.0
			MAX	27	20.1	50.7	354.5	225.8	101.3	1.1
			MIN	26	17.8	50.4	309.4	211.3	88.4	1.1
26	19.50	13	AV13	27	19.0	50.5	326.7	216.3	93.4	1.1
			STD	0	0.4	0.1	9.2	4.0	2.6	0.0
			MAX	27	19.6	50.7	341.1	224.2	97.5	1.1
			MIN	26	18.3	50.4	304.9	208.2	87.1	1.1
			Average	27	19.3	50.5	333.1	218.5	95.2	1.1
			Std. Dev.	0	0.6	0.1	14.8	4.8	4.2	0.0
			Maximum	27	20.1	50.7	360.9	225.8	103.1	1.1
			Minimum	26	17.8	50.4	304.9	208.2	87.1	1.1

Total number of blows analyzed: 25

BL# Sensors

2-26 F1: [652AWJ1] 223.3 (1.00); F2: [652AWJ2] 222.9 (1.00); A3: [K10815] 418.3 (1.00);
A4: [K10814] 390.0 (1.00)

BL# Comments

26 4 / 9 /13

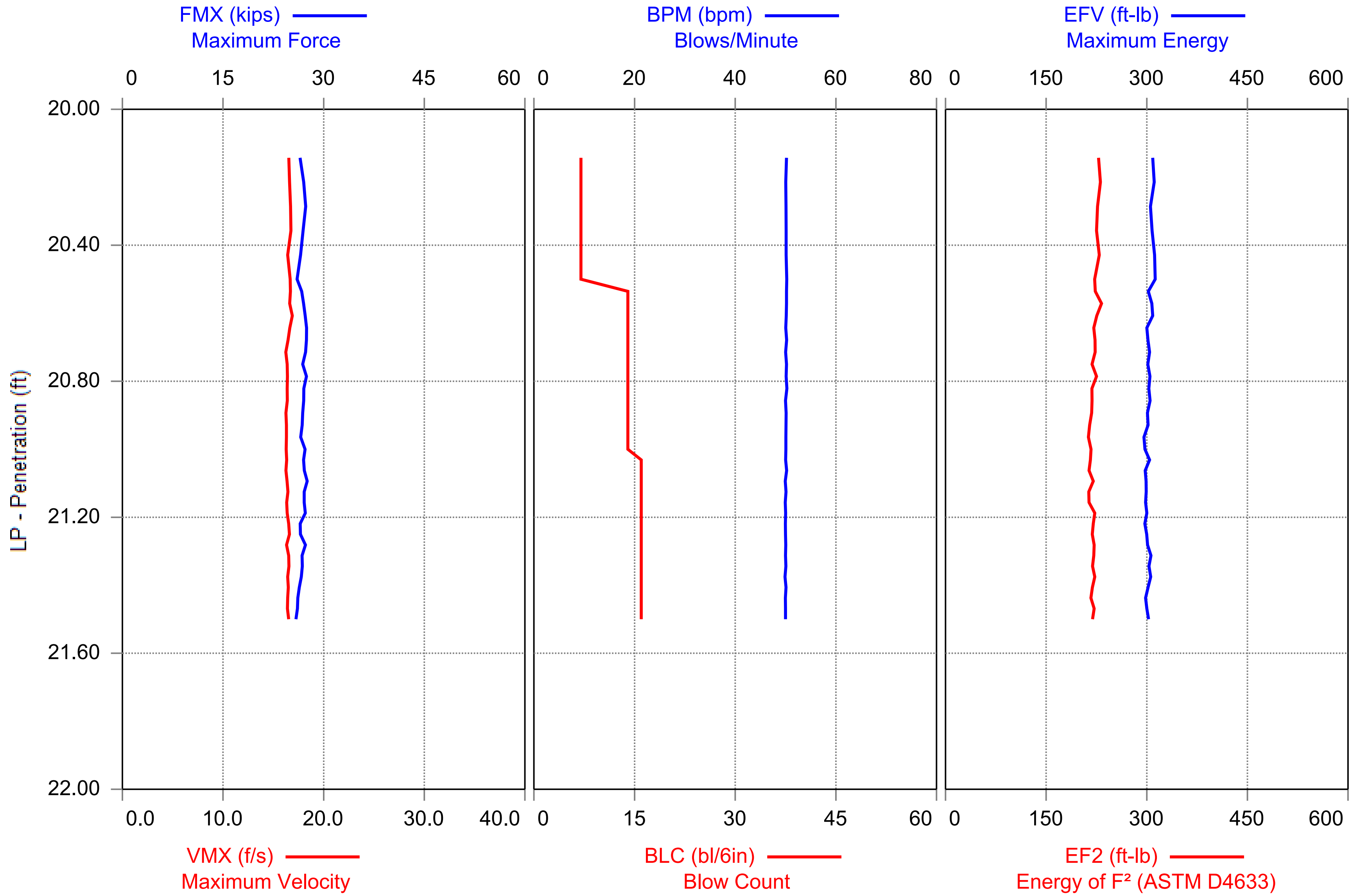
Time Summary

Drive 28 seconds 10:17 AM - 10:17 AM BN 2 - 26

GRL Engineers, Inc. - PDIPLOT2 Ver 2021.1.61.0 - Case Method & iCAP® Results

Printed: 03-June-2022 Test started: 02-June-2022

MOORE TWINING-TRUCK 170-SPT SOIL SAMPLER - 20-21.5



Case Method & iCAP® Results

MOORE TWINING-TRUCK 170-SPT SOIL SAMPLER - 20-21.5 BH1

OP: pc Date: 02-June-2022

AR: 1.21 in² SP: 0.492 k/ft³
LE: 23.59 ft EM: 30,000 ksi
WS: 16,807.9 f/s JC: 0.00

FMX: Maximum Force
VMX: Maximum Velocity
BPM: Blows/Minute
EFV: Maximum Energy
EF2: Energy of F² (ASTM D4633)
ETR: Energy Transfer Ratio - Rated
RAT: Length Ratio for SPT

BL#	Depth	BLC	TYPE	FMX	VMX	BPM	EFV	EF2	ETR	RAT
	ft	bl/6in		kips	f/s	bpm	ft-lb	ft-lb	(%)	
7	20.5	7	AV6	27	16.6	50.1	309.6	227.0	88.4	1.1
			STD	0	0.1	0.1	2.5	2.8	0.7	0.0
			MAX	27	16.7	50.2	312.6	230.9	89.3	1.1
			MIN	26	16.4	50.0	305.5	222.3	87.3	1.1
21	21.0	14	AV14	27	16.4	50.1	302.5	220.8	86.4	1.1
			STD	0	0.2	0.1	3.4	4.9	1.0	0.0
			MAX	27	16.9	50.3	308.8	232.6	88.2	1.1
			MIN	27	16.2	50.0	295.7	213.0	84.5	1.1
37	21.5	16	AV16	27	16.4	50.0	300.9	218.7	86.0	1.1
			STD	0	0.1	0.1	2.8	3.0	0.8	0.0
			MAX	28	16.6	50.2	306.2	222.6	87.5	1.1
			MIN	26	16.2	49.9	297.0	213.5	84.9	1.1
			Average	27	16.5	50.1	302.9	220.9	86.6	1.1
			Std. Dev.	0	0.2	0.1	4.3	4.8	1.2	0.0
			Maximum	28	16.9	50.3	312.6	232.6	89.3	1.1
			Minimum	26	16.2	49.9	295.7	213.0	84.5	1.1

Total number of blows analyzed: 36

BL# Sensors

2-37 F1: [652AWJ1] 223.3 (1.00); F2: [652AWJ2] 222.9 (1.00); A3: [K10815] 418.3 (1.00);
A4: [K10814] 390.0 (1.00)

BL# Comments

37 7 / 14 / 16

Time Summary

Drive 43 seconds 10:23 AM - 10:24 AM BN 1 - 37