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Geotechnical Investigation Report California Conservations Corp Multipurpose Building 1500 Alamar Way, Fortuna, California

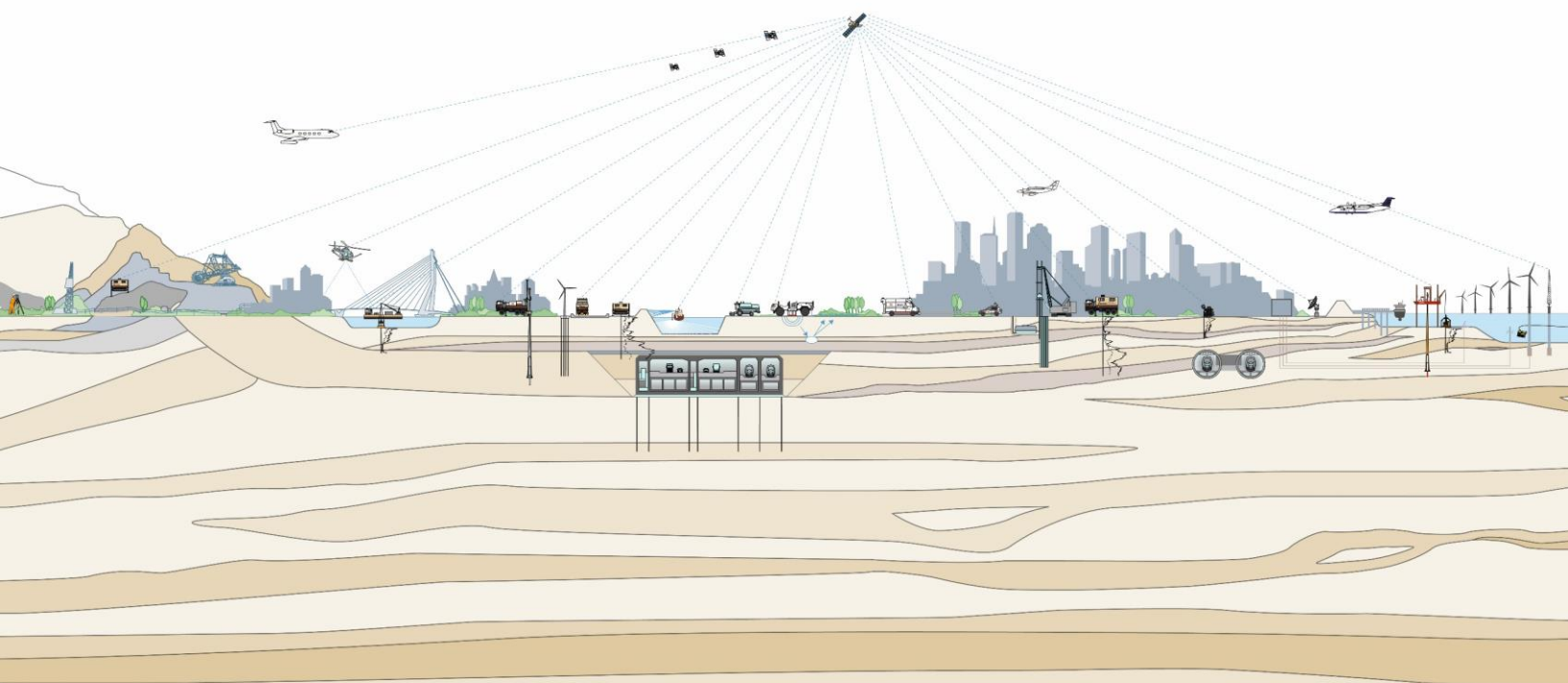
August 2019

Fugro Project No. 04.72190034

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State of California Department of General Services

Final



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Prepared for: State of California Department of General Services
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Attention: Mr. Jeffery A. Tsuruoka

**Final Geotechnical Investigation Report
California Conservation Corp, Multipurpose Building
1500 Alamar Way, Fortuna, California**

Dear Mr. Tsuruoka,

Fugro is pleased to submit this final geotechnical investigation report for the proposed new multipurpose building project at the California Conservation Corp Campus in Fortuna, California. Our work was authorized by Task Orders 2, 5, and 6 under our retainer agreement 5901-B with the State of California dated December 8, 2017 and was executed in general accordance with the scopes listed in the Task Orders.

This report was prepared to identify the key geologic and geotechnical aspects of the site and provide geotechnical recommendations for design and construction of the project. This report also summarizes the results of our geotechnical and geologic site data review, field exploration, and laboratory testing for the project site. We appreciate this opportunity to be of service to Department of General Services. Should you have any questions or require additional information, please contact us.

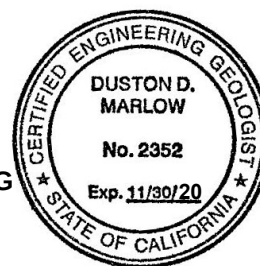
Sincerely,

Fugro USA Land, Inc.


Ronald L. Bajuniemi, PE, GE
Principal Consultant




Duston D. Marlow, PG, CEG
Associate Geologist



Distribution: One electronic copy to the addressee

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1. INTRODUCTION

This report presents the results of the geotechnical investigation conducted by Fugro USA Land, Inc. (Fugro) for the multipurpose building project. The project site is located on the California Conservation Corp (CCC) Campus at 1500 Alamar Way in the City of Fortuna, Humboldt County, California, as shown on the Site and Vicinity Map (**Plate 1**). Coordinates for the site are Latitude 40.5809897°N and Longitude 124.1530046°W.

1.1 Project Description

As proposed, the multipurpose building site is in an undeveloped area of the CCC campus bounded on the west by Riverwalk Drive, on the south by Alamar Way, on the east by paved parking, west of the existing Administration Building, and on the north by the CCC campus property boundary as shown on the Site Map (**Plate 2**).

The proposed multipurpose building is a one-story building with an estimated foot print of approximately 9,500 square feet. Building loads are estimated at approximately 8 kips for dead load and 8 kips for live load.

1.2 Scope of Services

The purpose of our geotechnical investigations was to identify key geotechnical and geologic hazards at the site and provide geotechnical recommendations for design and construction of the project. The scope of our services performed under Task Orders 2, 5, and 6 included the following:

- Compile and review available published geotechnical and geologic data and information provided by the Department of General Services (DGS) pertaining to the site and vicinity.
- Per Fugro standards and industry safety protocols, prepare Health and Safety Plan to cover workers involved with the site investigation activities.
- Locate and mark investigation locations in the field, notify Underground Service Alert (USA) a minimum of 48 hours prior to performing field investigations, and obtain drilling permits from the Humboldt County Department of Human and Health Services, Division of Environmental Health. All exploration locations were screened by a private utility locating firm for the existing utilities or obstructions.
- Subsurface explorations as proposed and authorized included:
 - Task Order 2 – six (6) borings (1 boring to 50 feet and 5 borings to 5 feet in depth), one (1) SCPT to 100 feet in depth, and one (1) CPT to 75 feet in depth
 - Task Order 5 – three (3) infiltration tests (three 12-inch diameter holes to a between 2 to 3 feet in depth.
 - Task Order 6 – two (2) borings, one boring to 100 feet in depth and one boring to 75 feet in depth.
- Drum investigation waste derived from the borings, perform analytical tests for characterization and disposal of the drums.
- Perform laboratory testing that included classification and index tests, Atterberg limits, compaction, R-value, and corrosivity testing on selected soil samples from the site.

- Identify the geotechnical and geologic conditions (e.g., stratigraphy, subsurface soil characteristic and engineering properties, depths to groundwater, and assessment geologic hazards) that could impact the project design and construction.
- Perform engineering analysis using the field and laboratory data, including detailed liquefaction triggering, post-liquefaction deformation, dynamic densification (seismic compression), and lateral spreading evaluations.
- Prepare this report to summarize the results of our geotechnical and geologic data review, field exploration, laboratory testing, applicable geologic hazards evaluations, and engineering analyses, and to provide geotechnical conclusions and recommendations for design and construction of the project in accordance with scopes of work provided in our executed task orders.



2. DATA REVIEW, EXPLORATION AND LABORATORY TESTING

2.1 Review of Existing Data

As part of our study, Fugro reviewed relevant geotechnical, geologic, and seismic data, as well as results of previous explorations and laboratory testing performed within the vicinity of the project site, including the following reports, literature, and maps. The conclusions from our review of the existing data are presented in subsequent sections of this report.

2.1.1 Existing Data and Reports

- SHN Consulting Engineers and Geologists, July 5, 1991, *California Conservation Corps, Fortuna California Site Plan*, Site Utilities "Red Lined" As-Built.
- SHN Consulting Engineers and Geologists, 2016, *Eel River Valley Groundwater Basin, Humboldt County, California, Groundwater Sustainability Plan Alternative*.
- Van Houten Consultants, Inc., January 24, 1990, *Geologic Hazards Study, South 12th Street Area, Fortuna, California*, Van Houten No. 89-260.01.
- *Humboldt Low Impact Development Stormwater Manual*, v2.0, June 30, 2016.
- R.J. McLaughlin, S.D. Ellen, M.C. Blake, Jr., Angelo S. Jayko, W.P. Irwin, K.R. Aalto, G.A. Garver, and S.H. Clarke Jr., 2000, *Geology of the Cape Mendocino, Eureka, Garberville, and Southwestern Part of the Hayfork 30 x 60 Minute Quadrangles and Adjacent Offshore Area, Northern California*, USGS Miscellaneous Field Studies Map, MF-23366.
- *City of Fortuna General Plan 2030*.
- Humboldt County General Plan Update, Humboldt 21st Century, 2015, *Central Humboldt Seismic Safety Map*.
- Federal Emergency Management Agency, *Flood Insurance Rate Map (FIRM), Panel 06023C1209F (11/4/16)*.
- Von Dohlen, Jerry, 2015, *Liquefaction Hazard Zones: Humboldt County, California, 2015*, Stanford University Earthworks Library, <https://earthworks.stanford.edu/catalog/stanford-nk595pg0743>.

2.2 Field Investigations

The field investigations were completed in general accordance with Task Orders 2, 5, and 6 for the project with some modifications due to subsurface conditions encountered. The field investigations were performed between April 30 and June 12, 2019, requiring three separate mobilizations to the site. In addition, a hand auger was also used to clear the top 5 to 6 feet of soils for utilities below existing ground surface at the boring and CPT locations.

Prior to the start of the investigations, the locations were marked in the field, Underground Service Alert (USA) was notified, and a private utility locating company; Del Ray GPR, was retained to clear the locations. Drilling permits were obtained from the Humboldt County Department of Human and Health Services, Division of Environmental Health. The locations of the completed field explorations are shown on the Investigation Locations Map (**Plate 3**).



2.2.1 Task Order 2 – Field Investigations

The planned investigations under Task Order 2 included six Borings (B-1 through B-6), one cone penetrometer test (CPT), and one seismic cone penetrometer test (SCPT). The purpose of Boring B-1 and the CPTs was to investigate the subsurface conditions at the site of the proposed new multipurpose building. The purpose of Borings B-2 through B-6 was to assess the possible presence of an improved building pad under the existing campus buildings. The Task Order 2 borings were performed April 30, 2019. The Task Order 2 CPTs were performed on May 8, 2019. Locations of explorations are shown on Investigation Locations Map (**Plate 3**).

2.2.1.1 Borings

Borings B-1 was drilled with a truck mounted CME 45 drill rig and advanced to a depth of 51 1/2 feet below ground surface (bgs). Boring B-1 was drilled using 4 1/4 inch I.D. hollow stem augers. Sampling was performed using standard penetration test (SPT) and modified California sampler with 6-inch liners driven with an auto trip hammer (140-pound weight, 30-inch drop). Sampling was performed at 5 feet intervals. Upon completion of the drilling, B-1 was backfilled with grout via tremie method per Humboldt County Department of Human and Health Services, Division of Environmental Health drilling permit requirements.

The planned depth of Borings B-2 through B-6 was 5 feet bgs. Fugro had planned to drill all these borings with the drill rig; however, site conditions, and existing utilities at the time of our investigation restricted use of the drill rig to only two (B-2 and B-3) of the five shallow borings. Borings B-2 and B-3 were advanced to 5 feet bgs with 4 1/4 -inch I.D. hollow stem augers. The remaining three shallow Borings (B-4 through B-6) were hand augered because of limited rig access. Depths of hand augered borings were limited to 6 inches to 14 inches bgs upon encountering refusal with a hand auger in a gravelly sand layer containing trace to some, cobbles and asphalt debris. Representative grab samples of the materials encountered in the auger borings were collected.

The soils encountered in our borings were classified by our field engineer in general accordance with the Unified Soil Classification System (ASTM D2487 and D2488). The logs of the borings as well as a key for the classification of the soils are included in **Appendix A**.

Cuttings from the borings were drummed and sampled for analytical testing. Drummed cuttings are stored on site. Upon receipt of analytical results and evaluation of results, then arrangements for drum pick up and disposal will be made.

2.2.1.2 CPT Soundings

The planned CPT soundings were to extend to depths up to 100 feet; however, the CPTs attempted met refusal in dense gravelly sand/sandy gravel layers with measured tip resistance exceeding 450 tons per square feet (tsf). At each CPT location, three attempts were made to get to the planned depth. At the CPT-1 locations, refusal depths ranged from about 11.9 feet to the maximum depth achieved of about 24.5 feet. At the CPT-2 locations (proposed SCPT) refusal depths ranged from about 14.7 feet to the maximum depth achieved of about 26.2 feet bgs. Because of the relatively shallow refusal depth, a SCPT was not performed.



CPT soundings were performed by Fugro, in general accordance with ASTM D5778, using a 25-ton truck-mounted CPT rig with an electronic piezocone penetrometer that has a tip area of 15 cm², a friction sleeve area of 225 cm², and a tip end area ratio of 0.59. The cone was advanced at a standard rate of 2 cm/sec into the ground to measure tip resistance, sleeve friction, and excess pore pressure. The CPT holes were backfilled with neat cement grout in accordance Humboldt County Department of Human and Health Services, Division of Environmental Health drilling permit requirements. The CPT logs and interpretations are presented in **Appendix A**.

2.2.2 Task Order 5 – Field Investigations

The planned investigations under Task Order 5 included the performance of 3 infiltration (or percolation) tests. These locations (PT-1 through PT-3) were selected by the DGS design team and are shown on the Investigation Locations Map (**Plate 3**).

Each infiltration test site consisted of drilling a 12-inch diameter hole between 2 to 3 feet in depth, dependent on the subsurface conditions encountered. These holes were drilled with truck mounted CME 45 drill rig with 12-inch diameter solid flight auger. PT-1, PT-2 and PT-3 were drilled to 28.5 inches, 34.5 inches, and 34.5 inches in depth bgs., respectively. Each hole was logged by our field engineer. The logs of the infiltration test holes are presented in **Appendix A**. Holes were drilled and presoaked on April 30, 2019.

The infiltration tests were performed on May 1, 2019, in general accordance with Humboldt County guidance documents and results of the infiltration tests were documented in the field by our field engineer. Results of testing were previously provided to DGS and are provided in a later section of this report.

Cuttings from the infiltration test holes were drummed and sampled for analytical testing. Drummed cuttings are stored on site. Upon receipt of analytical results and evaluation of results, arrangements for drum pick up and disposal will be made.

2.2.3 Task Order 6 – Field Investigation

The planned investigation under Task Order 6 include 2 borings up to 100 feet in depth to further evaluate the potential for liquefaction at the propose multipurpose building site below 50 feet in depth because of the relatively shallow refusal of CPTs and the presence of medium dense sands and gravels at 50 feet in Boring B-1. The proposed boring locations were adjacent to the CPT-1 and CPT-2 locations. Fugro budgeted three days for drilling the two borings based on estimates provided by our drilling subcontractor.

During the allotted time for drilling, we were able to complete only 1 boring; Boring B-7 (near CPT-1 location), to a depth of 77 feet from May 10 to May 12, 2019, due to the difficult drilling conditions within the encountered sands and gravels layers. B-7 was drilled using a truck mounted Mobile B-59 drill rig using auger and rotary wash methods. Standard penetration test (SPT) sampling was performed at 5 feet intervals to depth of 65 feet and then a 10 feet interval to 75 feet. Temporary NW size casing was extended to 32 feet bgs during the progression of drilling. Boring was terminated when circulation was lost at 77 feet. Driller lost 500 gallons of drilling fluid (bentonite/water mixture) without getting any fluid returns to surface. Loss of circulation within gravel deposits of high permeability is not an unusual occurrence when drilling.

The soils encountered in our borings were classified by our field engineer in general accordance with the Unified Soil Classification System (ASTM D2487 and D2488). The logs of the borings as well as a key for the classification of the soils are included in **Appendix A**.

Cuttings from the boring were drummed and sampled for analytical testing. Drummed cuttings are stored on site. Upon receipt of analytical results and evaluation of results, then arrangements for drum pick up and disposal will be made.

Upon completion of the drilling, Boring B-7 was backfilled with grout via tremie method per Humboldt County Department of Human and Health Services, Division of Environmental Health drilling permit requirements.

2.3 Geotechnical Laboratory Testing

Our geotechnical laboratory testing program was directed toward a quantitative and qualitative evaluation of the physical and mechanical properties of the soils underlying the site. The testing program under Task Orders 2 and 6 included:

- One (1) R-value per ASTM D2844,
- One (1) moisture content and dry unit weight per ASTM D2937,
- Twenty (20) sieve analyses per ASTM D422,
- One (1) sieve analyses and hydrometer per ASTM D422,
- Three (3) Atterberg Limits per ASTM D4318,
- One (1) soil chemistry for corrosion (pH, resistivity, chlorides, and sulfates),
- One (1) compaction test per ASTM D1557, and
- One (1) percent passing #200 sieve analyses per ASTM D1140.

All tests were performed by Moore Twining's geotechnical laboratory in Fresno, California. The laboratory testing results are included in **Appendix B**. Some of the test results are also presented on the boring logs (**Appendix A**) at the appropriate sample depths.

3. GEOLOGIC SETTING

3.1 Regional Geology

The project site is located within Humboldt County in the Coast Ranges geomorphic province of California. The Coast Ranges geomorphic province is comprised mainly of Franciscan complex inland, and sand and other alluvial deposits located closer to the coast. This area is located within a complex geologic setting, characterized by very high rates of active tectonic deformation and seismicity. This area is located north of the Mendocino Triple Junction where three crustal plates (the North American, Pacific, and Gorda plates) intersect. North of Cape Mendocino, the Gorda plate is being actively subducted beneath North America, forming what is referred to as the Cascadia subduction zone. In much of Humboldt County, secondary deformation related to the subduction zone is expressed on-land as a series of northwest trending thrust faults and folds. The geomorphic landscape of the region is largely a result of the active tectonic processes and the setting in this dynamic coastal environment.

The project is in the lower Eel River Valley within the Eel River basin. The Eel River basin is a mountainous area uplifted in the post-Miocene era and underlain by deformed, faulted, locally sheared, and in part, metamorphosed accumulation of subducted continental margin deposits.

Basement Rock, beneath the lower Eel River Valley area, is the Paleocene-Eocene Yager terrane, a part of the Coastal belt of the Franciscan Complex (Blake, et al., 1985; Clarke, 1992). The Franciscan Complex is a regional bedrock unit that consists of a series of "terranes", that are discrete blocks of highly deformed oceanic crust welded to the western margin of the North American plate over the past 140 million years. The Yager terrane consists of 9,800 feet of well indurated marine mudstone and thin bedded siltstone. The Yager terrane bedrock is at least 8,000 feet below the ground surface in the vicinity of the City of Fortuna, based on geologic cross sections presented in Ogle (1953).

The basement rock in the region is overlain by a late Miocene, to middle Pleistocene, age sequence of marine and terrestrial deposits, referred to as the Wildcat Group (Ogle, 1953). The marine portion of the Wildcat Group includes some 6,000 to 8,000 feet of mudstone and lesser amounts of sandstone that were deposited in the Eel River Basin. Gradationally overlying the marine portion of the Wildcat Group are 2,500 to 3,200 feet of nonmarine sandstone and conglomerate, which represent the uppermost part of the Wildcat depositional sequence. The Wildcat Group is truncated at its top by an unconformity of middle Pleistocene age, and is overlain by coastal plain and fluvial deposits of middle to late Pleistocene age. In the Eureka and Eel River valley areas, these middle and late Pleistocene age deposits are referred to as the Hookton Formation (Ogle, 1953). The Hookton Formation sediments are described as gravel, sand, silt, and clay, which have a characteristically yellow-orange color (Ogle, 1953).

3.2 Site Geology

The geologic setting for the City of Fortuna is characterized by the proximity of two of the most significant geomorphic influences in Humboldt County. These are the Little Salmon fault and the Eel River. The project site is about 750 feet east of the Eel River and is underlain by alluvium derived from the Eel and Van Duzen rivers and from streams draining the hills east of Fortuna (Rohner, Strongs, and Jameson Creeks). The

Little Salmon fault is possibly the most active on land fault in the region. The fault is associated with a large amount of cumulative displacement, and therefore defines a distinct geologic boundary between materials on either side of the fault.

The area around the confluence of the Van Duzen and Eel Rivers is underlain by a series of alluvial terraces that date back to the late Pleistocene. Uplift and folding associated with the Eel River syncline (a roughly east-west trending, active crustal fold) have resulted in broad deformation of these low relief surfaces. The most prominent alluvial surface in the vicinity is the broad Rohnerville terrace (sometime referred to as the Rohnerville "Formation"). The elevation of the terrace beneath the airport is approximately 400 feet and it descends to as low as 60 feet in the northern part of downtown Fortuna. The terrace sediments are defined by Kilbourne (1985) as "unconsolidated and gently folded, older Eel River flood plain deposits consisting predominantly of gravel with smaller amounts of sand, silt, and clay."

As the Rohnerville descends beneath Fortuna, it is mapped by Kilbourne (1985 as being buried by younger alluvial deposits derived from a series of streams that drain from the hills east of Fortuna (Rohner, Strongs, and Jameson Creeks). This alluvium is mapped as Holocene in age by Kilbourne, and is defined as "unconsolidated sand, silt, gravel, and clay deposited by streams in canyon bottoms." These younger alluvial deposits extend up the stream canyons bordering the City of Fortuna as recent stream channel deposits. The geology of the area is shown on the Regional Geologic Map (**Pate 4**).

3.3 Regional Seismicity

Humboldt county is located within a seismically active region of California. The county is located within the two highest seismic risk zones of the California Uniform Building Code. Cape Mendocino (offshore of the County) experiences the highest concentration of earthquake events in the continental United States. More than 60 earthquakes have produced discernible damage in the region since the mid-1800s (Dengler et al., 1992). Historic seismicity and paleo seismic studies in the area suggest there are six distinct sources of damaging earthquakes in the region: 1) the Gorda Plate, 2) the Mendocino fault, 3) the Mendocino Triple Junction, 4) the northern end of the San Andreas fault zone, and the Cascadia Subduction Zone (Dengler et al., 1992). Faulting in the region is shown on Regional Fault Map (**Plate 5**).

Earthquakes originating within the Gorda Plate account for the majority of historic seismicity. These earthquakes occur primarily offshore, along left-lateral faults, and are generated by the internal deformation within the plate as it moves toward the subduction zone. Significant historic Gorda Plate earthquakes have ranged from magnitude 5 to 7.5. The November 8, 1980, earthquake (magnitude 7.2) was generated 30 miles off the coast of Trinidad on a left-lateral fault within the Gorda Plate.

The Mendocino Fault is the second most frequent source of earthquakes in the region. The fault represents the plate boundary between the Gorda and Pacific plates, and typically generates right lateral strike-slip displacement. Significant, historic Mendocino fault earthquakes have ranged from magnitude 5 to 7.5. The September 1, 1994, magnitude 7.2 event originating west of Petrolia was generated along the Mendocino fault. The Mendocino triple junction was identified as a separate seismic source, only after the magnitude 6.0 August 17, 1991, earthquake. Significant seismic events associated with the triple junction are shallow

onshore earthquakes that appear to range from magnitude 5 to 6. Raised Holocene age marine terraces near Cape Mendocino suggest larger events are possible in this region.

Earthquakes originating on the northern San Andreas Fault are extremely rare but can be very large. The northern San Andreas Fault is right lateral strike-slip fault that represents the plate boundary between the Pacific and North American plates. The fault extends through the point Delgada region and terminates at the Mendocino triple junction. The 1906 San Francisco earthquake (magnitude 8.3) caused the most significant damage in the north coast region, with the possible exception of the April 1992 Petrolia earthquake (Dengler et al., 1992).

Earthquakes originating within the North American plate can be anticipated from a number of intraplate sources, including the Mad River Fault zone and the Little Salmon Fault. There have not been large magnitude earthquakes associated with faults within the North American plate; however, the December 21, 1954, magnitude 6.5 event may have occurred in the Mad River Fault zone. Damaging North American plate earthquakes are expected to range from magnitude 6.5 to 8. The Little Salmon Fault appears to be the most active fault in the region and is capable of generating very large earthquakes.

3.3.1 Little Salmon Fault

The Little Salmon Fault is the closest fault to the site that is known to be active. The Little Salmon Fault is a northwest trending, northeast dipping thrust fault located just east of the City of Fortuna. Offset relations within the upper Wildcat Group suggest that vertical separation exceeds 5,900 feet, representing 4.4 miles of dip-slip motion on the Little Salmon Fault since the Quaternary (in the past 700,000 to 1 million years). Estimates of the amount of fault slip for individual earthquakes along the fault range from 15 to 23 feet. Radiocarbon dating suggests that earthquakes occurred on the Little Salmon Fault about 300, 800 and 1,600 years ago. The average slip rate for the Little Salmon Fault for the past 6,000 years is between 6 and 10 millimetres per year. Based on the currently (2005) available fault parameters, the maximum magnitude earthquake for the Little Salmon Fault is thought to be between 7.0 (CDMG/USGS, 1996) and 7.3 (Geomatrix Consultants, 1994).

3.3.2 Cascadia Subduction Zone (CSZ)

The CSZ represents the most significant potential earthquake source in the north coast region. The CSZ is the location where the oceanic crust of the Gorda and Juan de Fuca plates are being subducted beneath the continental crust of the North American Plate. A great subduction event may rupture along 200 km or more of the coast, from Cape Mendocino to British Columbia, resulting in tsunami inundation in low-lying coastal areas. The April 25, 1992, Petrolia earthquake (magnitude 7.1) appears to be the only historic earthquake involving slip along the subduction zone, but this event was confined to the southernmost portion of the fault. It is estimated that there have been 6 significant subduction zone events along the CSZ in the last 3,000 years (Darienzo and Peterson, 1995). Paleoseismic studies along the subduction zone suggest that great earthquakes are generated along the zone every 300 to 500 years. Historic records from Japan, describing a tsunami thought to have originated along the CSZ, suggest the most recent great subduction event occurred on January 27, 1700. A great subduction earthquake would generate long duration, very

strong ground shaking throughout the north coast. The CSZ is located offshore, west of the north coast region.

4. SITE CONDITIONS

4.1 Surface Conditions

At the time of our study and as shown on the attached Site Plan (**Plate 3**), the proposed multipurpose building site is located in southwest corner of the CCC campus property and is bounded by Alamar Way on the south, paved parking on the east (west of Administration building), a fenced and tree lined property boundary on the north, and Riverwalk Drive on the west.

The site is undeveloped except for a concrete slab and picnic table. The proposed building site is grass covered and relatively flat. Trees and bushes are present along the northern property and this area topographic undulations or berms higher than the site. Along Alamar Way the site slopes south to street level. There is a utility vault adjacent to Alamar Way along the toe of this slope. This slope scatters areas of bushes and trees. The building site elevation is between 46 and 47 feet mean sea level datum (NGVD 1929). In general, the elevation at the proposed site of the multipurpose building is consistent with rest of the CCC campus property and the existing structures. As-built site plan for the CCC campus shows floor elevations of existing building at 48 feet.

4.2 Subsurface Conditions

4.2.1 Proposed Multipurpose Building

The subsurface soil conditions encountered by our Borings (B-1 and B-7), CPTs (CPT-1 and CPT-2) and infiltration test holes (PT-1, PT-2, and PT-3) for the proposed multipurpose building site are consistent with each other and with geologic mapping of the project site vicinity. In general, we encountered between 7 to 11 1/2 feet of alluvium consisting of predominately sandy clay and sandy silt underlain by terrace deposits consisting of gravelly sand and sandy gravels to the maximum depth investigated, 76 feet bgs.

The alluvium at the potential building area generally consists of predominately medium stiff, sandy clay of low plasticity. Within the alluvium interlayers; ranging the thickness from about 1 to 2 feet of very loose to medium dense, sand and gravel and very loose sandy silt. The underlying terrace deposits consist of medium dense to very dense, gravelly sands and sandy gravels. The terrace deposits were found to be very dense below 51 feet. The boring logs and CPTs are included in **Appendix A**. Our laboratory test results for the onsite soil samples are included in **Appendix B**.

4.2.2 Existing Campus Buildings

Previously, investigations (Van Houten, 1990) in the northeast corner of the property in the vicinity of the existing dormitories and west of parking area on north side of the existing administration building, indicated up about 5 feet of fill material exists at surface. The fill in general, was characterized as medium stiff, sandy silt, containing organics material (grasses), pieces of wood, and wire.

Fugro performed five shallow Borings (B-2 through B-6) up to 5 feet in depth adjacent to existing building on the CCC campus and tried to assess if building pads had been improved (overexcavation and replacement and/or recompacted). The following provides a summary of the observed materials encountered in our Borings B-2 through B-6.



Boring B-2 was located near the southeast corner of the Administration Building. At this location we encountered medium dense, silt, with some interlayers of sand and gravel to a depth of 5 1/2 feet bgs.

Boring B-3 was located on the west side of the shop building. At this location we encountered 2 inches of asphalt and 4 inches of aggregate base, underlain by 3 feet of medium dense, silt with sand, followed by 1 1/2 feet of loose, sand with gravel and wood debris to a total depth of 5 1/2 feet. Wood debris (chips) content was estimated at 20% of the sample.

Boring B-4 was located along to the northeast side of the South Dormitory building. This location was hand augered to a depth of 14 inches. This boring encountered poorly graded sand with silt and gravel to total depth of 14 inches bgs. At this depth hand auger could not be advanced.

Boring B-5 was located along the southeast side of the East Dormitory building. This location was hand augered to a depth of 5 inches. This boring had 1 1/2 inches of topsoil with organic matter and roots, underlain by 4 1/2 inches of silty sand with gravel and traces of asphalt fragments. Auger could not be advanced past this depth.

Boring B-6 was located along the northwest side of the East Dormitory building. This location was hand augered to a depth of 14 inches. This boring encountered 6 inches of topsoil with organic matter, underlain by 6 inches of silty sand with gravel and traces of asphalt fragments.

Based on the comparison of materials encountered in our borings versus previous borings; it would appear that in the vicinity of the existing buildings, that up to about 3 feet of the 5 feet of the fill layer (sandy silt with organics material, wood and wire) have been replaced sand with gravel. This is supported Borings B-4, B-5 and B-6 where hand auger was unable to penetrate silty sand with gravel at 4.5 to 14 inches in depth, and in our Boring B-3, where below 3 feet (bottom of the aggregate base) we encountered 1.5 feet of sand and gravel with wood debris (estimated at 20 percent). The logs of borings are included in **Appendix A**.

4.3 Groundwater

Groundwater was encountered in Boring B-1 and B-7 at the proposed multipurpose building site. As observed in our Borings B-1 and B-7, groundwater, the site groundwater table is estimated to be at a depth of 28 feet (Elevation of about +18). It should be noted that these borings might not have been left open for a sufficient period of time to establish equilibrium groundwater conditions. Fluctuations in the groundwater table could occur due to change in seasons, variations in rainfall, changes in water level in the nearby Eel River, and other factors.

Other groundwater measurements in the immediate area include Boring 3 and MW-1 from the Von Houten report (1990). Groundwater was encountered in Boring 3 at a depth of about 27 feet (Elevation of about +19) on November 28, 1989, located approximately 0.1 mile south of the proposed multipurpose building site. Groundwater was reported from a monitoring well (MW-1) located at the Pacific Lumber Maki Woodwaste Site (Von Houten,1990), located about 0.25 of mile south and east of the project site at a depth of about 22 feet (Elevation of about + 29) on February 19, 1988.

The principal aquifers within the Eel River Basin are defined based on the primary stratigraphic units of the Basin; the shallow alluvial aquifer and the underlying aquifer associated with the Carlotta Formation. The shallow alluvial aquifer underlies the site is on the order of 100 to 200 feet in thickness. Review of hydrographs of wells within the shallow alluvial aquifer near the site indicates historic highs of about Elevation +31 to +32. It should be noted that the shallow alluvial aquifer is in direct connection with the Eel River. Water levels in monitoring wells track the changes in water level in the Eel River. It is generally observed that the aquifer level is generally 10 feet lower than river level when the river and aquifer levels are plotted against each other. So as an example; if you took a 100-year flood level at Fortuna of Elevation +46 feet (Fortuna General Plan 2030), then it would be reasonable to assume that water level in the shallow aquifer at the site could reach elevation +36 feet.

From the data reviewed, the historic high groundwater level within the shallow alluvial aquifer at the site would be in the range of Elevation +31 to Elevation +36. We recommend a design groundwater Elevation of +36 feet be used for the project designs, which generally corresponds to the top elevation of terrace deposits.

5. GEOLOGIC HAZARDS

The opinions, conclusion, and recommendations in the following sections were based on the results of our review of available information relating to geotechnical, geologic, and seismic data within the vicinity of the site, project field exploration and laboratory programs, and site-specific engineering analyses.

5.1 Surface Fault Offset

Based on existing geologic maps and literature, there are no known active fault traces within, adjacent to, or trending towards proposed multipurpose building project site. The closest known active fault is the Little Salmon Fault, located approximately 3 miles to the northeast. The site is not located within a Fault-Rupture Hazard Zone, as determined by the Alquist-Priolo Earthquake Fault Zoning Act.

Other active faults in the area include the Mendocino and San Andreas faults, located 30 and 40 miles to the southwest of the site respectively. While we cannot preclude the possibility of new fault breaks developing during future earthquakes, historic occurrences of surface fault rupture have generally followed pre-existing active fault traces.

5.2 Shaking Hazards

The project site is located within an active seismic region with multiple, nearby, seismic sources. Due to the proximity of the project site to the Little Salmon fault, Mendocino triple junction, and the Cascadia subduction zone, the potential exists for long sustained periods of intense ground shaking. Strong ground shaking can cause the structures to shake; and has the potential capability of inducing other phenomena that can indirectly cause damage to structures. These phenomena include soil liquefaction, seismic compression of dry soils, lateral spreading, and ground cracking, seismically induced waves, such as tsunamis and seiches, inundation due to dam or embankment failure, and landsliding. Detailed discussions of these other phenomena relevant to the site are presented in the subsequent paragraphs.

5.3 Liquefaction and Seismic Compression

Soil liquefaction is a phenomenon primarily associated with saturated cohesionless soil layers. These soils can dramatically lose strength due to increased pore water pressure during cyclic loading, such as imposed by earthquakes. During the loss of strength, the soils acquire mobility sufficient to permit both horizontal and vertical movements. Soils that are most susceptible to liquefaction are clean, loose, uniformly graded, saturated sands that lie close to the ground surface; although, liquefaction can also occur in fine-grained soils, such as low-plasticity silts. In addition, seismic compression may also occur within loose to medium dense, dry sand layers located above groundwater level.

The City of Fortuna General Plan 2030 states that the liquefaction potential of the alluvial deposits is unknown. There are ample historic accounts of extensive liquefaction in the Eel River flood sediments, but none within the City of Fortuna planning area. However, it concludes that there appears to be areas near the Eel River may have the potential for liquefaction to occur.

The project site is located near the Eel River and within the Eel River flood plain. The project site is underlain by young alluvium and terrace deposits of Pleistocene to Holocene age. The Northern Humboldt Seismic Safety Map (**Plate 6**) shows the project site is within an area of potential liquefaction.

Our site liquefaction evaluations, which included liquefaction history review and liquefaction triggering and post-liquefaction deformation analyses, are presented in the following sections. In addition, potential for dry sand dynamic densification (seismic compression) was also evaluated.

5.3.1 Liquefaction Triggering Hazard Assessment Methodology

A liquefaction triggering hazard assessment was performed using two empirical CPT-based and SPT-based liquefaction triggering methods: 1) NCEER (Youd et al., 2001), and 2) Boulanger and Idriss (2014). This liquefaction evaluation was conducted for CPT-01, CPT-01A, CPT-01B, CPT-02, CPT-02A, and CPT-02B; and Borings B-1 and B-7. The design ground water table was assumed to be at a depth of 10 feet.

The factor of safety (FS) against liquefaction is defined as:

$$FS = \frac{CRR}{CSR} \quad \text{Equation 1}$$

where CSR is the cyclic stress ratio, which is a measure of the earthquake loading induced in the soil, and the CRR is the cyclic resistance ratio, which is a measure of the soil resistance.

5.3.1.1 Seismically Induced Cyclic Demand

In simplified liquefaction triggering evaluations, the seismic demand is expressed by means of the earthquake induced cyclic stress ratio (CSR):

$$CSR = 0.65 \times \frac{PGA}{g} \times \frac{\sigma_{vo}}{\sigma_{vo}'} \times r_d \quad \text{Equation 2}$$

where PGA is peak ground acceleration at the mudline, g is the acceleration of gravity, r_d is a depth-varying factor that accounts for the non-rigid response of the soil column, and σ_{vo} and σ_{vo}' are the vertical total and effective stress.

The estimated earthquake and depth-specific CSR is converted to a reference earthquake magnitude of 7.5 and vertical effective stress of 1 atm by means of the magnitude scaling factor (MSF) and overburden correction factor (K_σ) as shown in the following equation:

$$CSR_{M=7.5, \sigma_v'=1atm} = CSR \times \frac{1}{MSF} \times \frac{1}{K_\sigma} \times r_d \quad \text{Equation 3}$$

For example, if the controlling magnitude event is less than the reference magnitude of 7.5, a MSF greater than 1 is applied to reduce the equivalent cyclic stress ratio due to the decreased number of cycles imparted by the lesser magnitude event.

The earthquake scenario used in the liquefaction evaluations corresponds to a PGA of 1.08 g and an earthquake magnitude of 8.2 (see **Appendix D**).

5.3.1.2 Cyclic Resistance to Liquefaction Based on CPT NCEER (Youd et al., 2001)

The expression to compute CRR by the NCEER method (Youd et al., 2001) is:

$$CRR_{M=7.5, \sigma_v'=1atm} = \begin{cases} 0.833[(q_{c1N})_{cs}/1000] + 0.05 & \text{if } (q_{c1N})_{cs} < 50 \\ 93[(q_{c1N})_{cs}/1000]^3 + 0.08 & \text{if } (q_{c1N})_{cs} \geq 50 \end{cases} \quad \text{Equation 4}$$

where $(q_{c1N})_{cs}$ is the clean-sand cone penetration resistance normalised to approximately 1 atm, calculated as:

$$(q_{c1N})_{cs} = K_c q_{c1N} \quad \text{Equation 5}$$

where K_c is the correction factor for grain characteristics as a function of the soil behaviour type index I_c , and q_{c1N} is the overburden stress normalized cone tip resistance calculated as:

$$q_{c1N} = \left(\frac{P_a}{\sigma_{vo}'} \right)^n \frac{q_c}{P_a} \quad \text{Equation 6}$$

where n is a stress exponent that varies with soil type; q_c is the measured tip cone penetration resistance; P_a is the atmospheric pressure in the same units as q_c or σ_{vo}' ; and σ_{vo}' is the effective overburden stress.

Boulanger and Idriss (2014)

The expression to compute CRR by the Boulanger and Idriss (2014) method is:

$$CRR_{M=7.5, \sigma_v'=1atm} = \exp \left(\frac{q_{c1Ncs}}{113} + \left(\frac{q_{c1Ncs}}{1000} \right)^2 - \left(\frac{q_{c1Ncs}}{140} \right)^3 + \left(\frac{q_{c1Ncs}}{137} \right)^4 - 2.80 \right) \quad \text{Equation 7}$$

where q_{c1Ncs} is the cone tip resistance corrected for the influence of fines content calculated as:

$$q_{c1Ncs} = q_{c1N} + \Delta q_{c1N} \quad \text{Equation 8}$$

where Δq_{c1N} is a fines content correction factor and q_{c1N} is the overburden stress normalized cone tip resistance calculated as:

$$q_{c1N} = \left(\frac{P_a}{\sigma_{vo}'} \right)^m \frac{q_c}{P_a} \quad \text{Equation 9}$$

where m is a stress exponent; q_c is the measured tip cone penetration resistance; P_a is the atmospheric pressure in the same units as q_c or σ_{vo}' ; and σ_{vo}' is the effective overburden stress.

The CPT tip resistance in a highly stratified soil column is affected by softer soil layers above or below the cone tip. This results in smaller measured CPT tip resistance in thin layers of granular soils located between softer soil layers compared to what would be measured in thicker layers of the same granular soil. The analyses presented in this report include a correction of the CPT tip resistance for thin soil layers based on the recommendations in Youd et al. (2001).

5.3.1.3 Cyclic Resistance to Liquefaction Based on SPT

The cyclic resistance ratio can be estimated based on the energy and overburden corrected blow count value of the SPT adjusted for clean sands $[(N_1)_{60cs}]$ by means of empirical relationships.

NCEER (Youd et al., 2001)

$$CRR_{M=7.5} = \frac{1}{34 - (N_1)_{60cs}} + \frac{(N_1)_{60cs}}{135} + \frac{50}{(10 \cdot (N_1)_{60cs} + 45)^2} - \frac{1}{200} \quad \text{Equation 10}$$

Boulanger and Idriss (2014)

$$CRR_{M=7.5, \sigma'v=1atm} = \exp \left(\frac{(N_1)_{60cs}}{14.1} + \left(\frac{(N_1)_{60cs}}{126} \right)^2 - \left(\frac{(N_1)_{60cs}}{23.6} \right)^3 + \left(\frac{(N_1)_{60cs}}{25.4} \right)^4 - 2.8 \right) \quad \text{Equation 11}$$

5.3.2 **Free-Field Liquefaction-Induced Reconsolidation Settlements**

Liquefaction-induced settlements are vertical movements of the ground surface caused by earthquake loading after soil liquefaction occurs. These settlements can be attributed to different mechanisms, including:

1. Reconsolidation of the liquefied soils (volumetric compression),
2. Ground loss due to sand boils,
3. Lateral spreading or other forms of lateral instability, and
4. Soil-structure interaction ratcheting and bearing capacity failure.

Free-field liquefaction-induced reconsolidation settlements were calculated herein using empirical approaches that provide an estimate of the volumetric strain, which is integrated over the thickness of the one-dimensional soil column to obtain a settlement estimate at the ground surface. The settlements calculated by the current empirical approaches can vary significantly, and therefore, a combination of approaches is recommended to obtain more consistent and stable estimates of liquefaction-induced reconsolidation settlements (Kramer, 2008). The approaches used in this study include Ishihara and Yoshimine (1992) and Idriss and Boulanger (2008). These approaches provide estimates of volumetric strains based on soil penetration resistance, CSR, and factor of safety against liquefaction.

5.3.3 Liquefaction Evaluation Results and Conclusions

The results of the liquefaction triggering hazard assessment are presented in **Appendix E. Figures E-1 and E-2** present the results based on CPT data, and **Figures E-4 and E-5** present the results based on SPT-data. **Figures E-1 and E-2** are presented as a series, so for example, **Figures E-1a to E-1f** represent results for one assessment method (NCEER) but with each individual figure presenting the results of a specific CPT location. Each figure includes profiles of tip resistance, friction ratio, volumetric strain, estimated cumulative settlement, factor of safety against liquefaction, and soil behaviour classification. Refer to **Figure E-3** for the legend applicable to **Figures E-1 and E-2**.

Table 5.1 summarizes liquefaction triggering hazard assessment results. In general, these results indicate that the sands between depths of 10 and 22 feet are potentially liquefiable under the design ground motions. Furthermore, there are additional deeper sand layers between depths of about 30 and 50 feet that could also be potentially liquefiable under the design ground motions. The sands below a depth of about 50 feet are estimated to be not liquefiable under the design ground motions.

CPTs are often preferable to SPTs for use in liquefaction triggering and settlement assessment because they provide continuous data with depth and are generally less subject to equipment or operator variability. However, the CPT data available for the project does not extend below a depth of about 25 feet. Therefore, the free-field liquefaction-induced reconsolidation settlements were estimated by combining the settlements calculated using CPT data above a depth of 25 feet and the settlements calculated using SPT data below a depth of 25 feet. Thus, the estimated free-field liquefaction-induced reconsolidation settlements range from about 1.4 (Combination of CPT-02B and B-1) to 3.3 inches (Combination of CPT-01 and B-7). It is important to note that the actual settlements may differ from our estimates due to the great amount of uncertainties in the current liquefaction triggering and settlement analysis methodologies. Additionally, it is generally accepted that the contribution of liquefiable soil layers to surface settlement diminishes as the depths of these layers increase.

Table 5.1: Summary of CPT- and SPT-Based Liquefaction Analysis Results

Location	Estimated Depth of Potentially Liquefiable Soils (ft)	Free-Field Liquefaction-Induced Reconsolidation Settlements (inches)	
		Above depth of 25 ft ¹	Below depth of 25 ft ¹
CPT-01	14 to 15 16 to 18	0.7	-
CPT-01A	-	-	-
CPT-01B	-	-	-
CPT-02	17 to 21	0.6	-
CPT-02A	-	-	-
CPT-02B	10 to 10.5 13 to 14 16 to 20 21 to 22	0.9	-
B-1	15 to 23.5 33.5 to 38 47 to 51.5	3.5	0.5
B-7	17.5 to 22.5 27.5 to 32.5 37.5 to 42.5 47.5 to 52.5	0.5	2.6
Notes: 1. 25 feet is the CPT maximum exploration depth			

Based on our review of available maps and literature, and the results of our site evaluations, it is our opinion, when the site is subjected to the design ground motions, the likelihood of liquefaction occurring in the upper 25 feet at the site is high. Additionally, based on boring logs presented in **Appendix A**, and the results presented in **Table 5.1** and in **Appendix E**, we believe that the layers of potentially liquefiable soils below a depth of 25 feet are not continuous across the project site and therefore the potential for liquefaction in these layers is low.

Additionally, it would be reasonable to assume that existing campus buildings may be also subjected to liquefaction settlements similar to or greater than settlements estimated for the site of the new multipurpose building.

5.3.4 Seismic Compression Evaluations

We performed a CPT-based seismic compression evaluation for the project site based on the procedure proposed by Robertson and Shao (2010). The SPT data was not used for the seismic compression evaluation as there is only one SPT test within the top 10 feet (design groundwater depth) at both Borings B-1 and B-7. Therefore, the results from an SPT-based procedure would not be representative.

Similar to the liquefaction triggering hazard assessment, a PGA of 1.08 g, a mean earthquake magnitude of 8.2, and the project design groundwater level at a depth of 10 feet were used in our analyses. The

potential seismic compression settlements of the near-surface unsaturated sandy soils (predominately sandy lean clay and lean clay with minor sand and gravel interbeds) at the site are estimated to be on the order of 0.1 inch for the design ground motions. It is our opinion that the potential for seismic compression to impact the site is low.

5.4 Lateral Spreading

Lateral spreading occurs when soils liquefy during an earthquake event and the liquefied soils with the overlying soils move laterally to unconfined spaces (for example, channel banks), which causes significant horizontal ground displacements.

As mentioned in **Section 5.3.3**, we believe that the layers of potentially liquefiable soils below a depth of 25 feet are not continuous across the project site, and therefore, these deeper layers are not expected to contribute to the lateral spreading potential of the site. Additionally, CPTs are often preferable to SPTs for use in liquefaction assessments because they provide continuous data with depth and are generally less subject to equipment or operator variability. Therefore, the lateral spreading potential of the project site was estimated using only CPT data, which extends to a maximum depth of 25 feet.

Our site lateral spreading evaluations generally followed the methodologies proposed by Youd et al. (2002), Zhang et al. (2004), and Idriss and Boulanger (2008) to estimate the maximum shear strain of each liquefiable soil layer and calculate the Lateral Displacement Index (LDI) and corresponding potential ground lateral displacement at each CPT location.

The calculated LDIs and order of ground lateral displacements at the site for each CPT location are summarized in the **Table 5.2**. It should be noted these values should be considered as an index due to the limitations of the current engineering knowledge and analysis methodology of a very complex phenomenon.

Table 5.2: Summary of CPT- and SPT-Based Lateral Spreading Analysis Results

Location	Calculated Lateral Displacement Index - LDI (inches)	Estimated Average Ground Lateral Displacement (inches)
CPT-01	6	2
CPT-01A	0	0
CPT-01B	0	0
CPT-02	5	2
CPT-02A	0	0
CPT-02B	7	2.5

Based on our review of available maps and literature, and the results of our site evaluations (LDI and displacement), it is our opinion, when the site is subjected to the design ground motions, the likelihood of lateral spreading occurring at the site is low. Additionally, there have been numerous large magnitude earthquakes in Northern California and there has been no documented evidence of lateral spreading along the Eel River in Fortuna.

5.5 Flooding

According to FEMA Flood Insurance Rate Map panel 06023C1209F, effective on November 4, 2016, the project area is located within a 100-year flood zone. Base flood elevations are shown on FEMA Flood Insurance Rate Map (**Plate 7**).

5.6 Corrosive Soils

Corrosivity tests, that include pH, chlorides, sulfates, and resistivity were performed by Moore Twining, Inc. on one representative onsite near-surface soil samples (from Boring B-1, bulk sample from 0 to 5 feet depth). The test results are also included in **Appendix B**. The following table presents a summary of test results.

Table 5.3: Summary of Corrosion Test Results for Buried Concrete

Boring	Depth (ft)	Sulfate (%)	Chloride (%)	pH
B1 Bulk	0-5	0.0037	ND (<0.00060)	5.8

Sulfate tests indicated a sulfate content of 0.0037 percent. Therefore, as shown in **Table 5.4** below, these results indicate that the soils underlying the site have negligible potential for sulfate attack on concrete. Sulfate concentrations would be considered to be insufficient to damage reinforced concrete structures and cement mortar-coated steel at the locations tested. Corrosion potential to the reinforcing steel in concrete evaluated with consideration of chloride content, would indicate that chloride content is insufficient to attack steel embedded in a concrete mortar coating.

Table 5.4: Potential for Sulfate Attack on Concrete

Relative Degree of Sulfate Attack	Water-Soluble Sulfate, as SO ₄ (%)
Negligible	0.00-0.10
Positive +	0.10-0.20
Considerable ++	0.20-0.50
Severe	Over 0.50
Notes: + Use Type II Cement ++Use Type V Cement	

The pH of the soils tested is 5.8. The soils would be considered to be slightly corrosive to buried iron, steel, mortar-coated steel, and reinforced concrete structures.

In addition to pH tests, resistivity tests have been performed on soil samples at the site to provide an indication of corrosion potential of the soil environment on buried ferrous metals such as steel or cast-iron pipes. The approximate resistivity of surficial soils at the site 3,402 ohm-cm. Corrosivity ratings based on soil resistivity according to American Society for Testing and Materials (ASTM G187-12a: Standard Test Method for Measurement of Soil Resistivity Using the Two-Electrode Soil Box Method) are provided in **Table 5.5**.

Table 5.5: Corrosion Potential Based on Resistivity

Soil Resistivity (ohm-cm)	ASTM
>10,000	Very mildly corrosive
5001-10,000	Mildly corrosive
2,001-5,000	Moderately corrosive
1,001-2,000	Severely corrosive
501-1,000	Extremely corrosive
0-500	Extremely corrosive

Based on **Table 5.5**, the onsite soils are moderately corrosive. All buried iron, steel, cast iron, ductile iron, galvanized steel and dielectric coated steel or iron should be properly protected against corrosion depending upon the critical nature of the structure. All buried metallic pressure piping such as ductile iron pipeline should be protected against corrosion as well.

5.7 Compressible Soils

The near-surface soils encountered at the site we predominately alluvium that consist generally of medium stiff low plasticity sandy clays with silt to a depth of about 10 feet bgs. The alluvium is underlain by terrace deposits consisting of loose to very dense sands and gravels to the total depth investigated. Since the proposed new multipurpose building is lightly loaded, the near surface soils would not be considered compressible under the anticipated new loading.

5.8 Expansive Soils

The near-surface soils encountered at the site were predominately alluvium that consist of sandy lean clays with plasticity index of 9. The expansion potential of the near-surface soils at this site is considered low.

6. DISCUSSION AND CONCLUSIONS

It is our opinion that the project is feasible from a geotechnical and engineering geologic standpoint, provided that the conclusions and recommendations presented in this report are incorporated into the project design and specifications. The principal geotechnical considerations are discussed in the following sections.

6.1 Seismicity and Geologic Hazards

The site is in a seismically active region of California. Significant earthquakes in the project area have been associated with movements within the fault zones. Earthquakes occurring along faults in the area have the potential to produce strong ground shaking at the site. Structures within the project area will experience similar shaking effects during a moderate to strong earthquake. Details discussions regarding the site geologic hazards are presented in **Section 5.0**.

Based on the results of our review and evaluation, geologic hazards at the project site consist of the potential for strong ground shaking, liquefaction, lateral spreading, flooding, and corrosive soils. Detailed measures to mitigate these geologic hazards are incorporated in our recommendations presented in **Section 7.0**.

However, the potential for surface fault offset, dynamic densification, seismically induced waves, expansive soil, compressible soils, and slope stability at the project potential building area appeared to be low, in our opinion.

6.2 Liquefaction and Lateral Spreading

Based on our review of available maps and literature, and the results of our site evaluations, it is our opinion, when the site is subjected to the design ground motions, the likelihood of liquefaction occurring in the upper 25 feet at the site is high. Additionally, based on boring logs presented in **Appendix A**, and the results presented in **Table 5.1** and in **Appendix E**, we believe that the layers of potentially liquefiable soils below a depth of 25 feet are not continuous across the project site and therefore the potential for liquefaction in these layers is low.

CPTs are often preferable to SPTs for use in liquefaction triggering and settlement assessment because they provide continuous data with depth and are generally less subject to equipment or operator variability. However, the CPT data available for the project does not extend below a depth of about 25 feet. Therefore, the free-field liquefaction-induced reconsolidation settlements were estimated by combining the settlements calculated using CPT data above a depth of 25 feet and the settlements calculated using SPT data below a depth of 25 feet. Thus, we estimate free-field liquefaction-induced reconsolidation settlements range from about 1.4 to 3.3 inches. It is important to note that the actual settlements may differ from our estimates due to the great amount of uncertainties in the current liquefaction triggering and settlement analysis methodologies. Additionally, it is a generally accepted that the contribution of liquefiable soil layers to surface settlement diminishes as the depths of these layers increase.

The layers of potentially liquefiable soils below a depth of 25 feet are not continuous across the project site, and therefore, these deeper layers are not expected to contribute to the lateral spreading potential of the

site. Based on our review of available maps and literature, and the results of our site evaluations, it is our opinion, when the site is subjected to the design ground motions, the likelihood of lateral spreading occurring at the site is low.

We recommend the proposed new building be supported on a deep foundation system that provides proper bearing supports during the potential soil liquefaction events. The deep foundation should be designed to resist downdrag loads that would be imposed upon the foundations due to soil liquefaction.

Consideration should also be given to using flexible pipe connections to tolerate the estimated potential liquefaction-induced settlement of about 3 inches at locations where the pipes are connected to pile-supported structures. Underground pipelines (gas lines, sanitary sewers, water services, etc.) should be properly designed to compensate for the settlement caused by the liquefaction of the underlying supporting soils. It should also be noted that after a major liquefaction event, phenomena such as sand boils, ground cracking, and differential movement of overlying improvements such as roadways and utilities may be observed and may require repair.

6.3 Preliminary Corrosion Evaluation

Corrosivity tests that include redox, pH, chlorides, sulfates, and resistivity were performed by Moore Twining, Inc. in Fresno, California, on one representative onsite near-surface soil samples (from Boring B-1 from 0 to 5 feet). A discussion of corrosivity of onsite soils is provided in **Section 5.0**. The test results of the onsite included in **Appendix B**. We recommend these test results and the report be forwarded to your underground contractors, pipeline designers, and foundation designers and contractors, so that they can design and install corrosion protection measures for buried concrete structures and ferrous metal. We also recommend additional testing be performed if the test results in **Appendix B** are deemed insufficient by the designers of the corrosion protection.

6.4 Compressible Soils

The near-surface soils encountered at the site we predominately alluvium that consist generally of medium stiff low plasticity sandy clays with silt to a depth of about 10 feet bgs. The alluvium is underlain by terrace deposits consisting of loose to very dense sands and gravels to total depth investigated. Since the proposed new multipurpose building is lightly loaded, the near surface soils would not be considered compressible under the anticipated new loading..

6.5 Infiltration Tests

Three infiltration tests were performed at the proposed multipurpose building site. The infiltration tests were performed in 12-inch diameter holes. Each hole was presoaked prior to performing the test with 8 gallons of water. The observed soil type encountered at each location and depth of each hole provided in the **Table 6.1**.



Table 6.1: Summary of Infiltration Test Holes

Location	Depth (Inches)	Material Type
PT-1	28.5	Well graded Sand with silt (SW)
PT-2	34.5	Lean Clay with sand and gravel (CL)
PT-3	34.5	Sandy Silt with clay (ML)

Observed depth to groundwater from our geotechnical borings is about 28 feet bgs. The results of our infiltration tests for location PT-1, PT-2, and PT-3 are provided in **Table 6.2**, **Table 6.3**, and **Table 6.4**, respectively.

Table 6.2: PT-1 Infiltration Test Data

Time (hr:min)	Interval (min.)	Measurement (in.)	Drop in Water Level (in.)	Percolation Rate (MPI)
09:26		17.5	Add water	
09:28	2	10.0	7.5	0.27
09:29		12.0	Add Water	
09:31	1	10.5	2.5	0.40
09:32		26.0	Add water	
09:34	2	22.5	3.5	0.57
09:39	5	16.0	6.5	0.77
09:44	5	12.5	3.5	1.42
09:49	5	9.0	3.5	1.42
09:50	5	25.5	Add Water	
09:55	5	21.0	4.5	1.11
10:00	5	17.0	4.0	1.25
10:05	5	12.5	4.5	1.11
10:10	5	8.5	4.0	1.25

Table 6.3: PT-2 Infiltration Test Data

Time (hr:min)	Interval (min.)	Measurement (in.)	Drop in Water Level (in.)	Percolation Rate (MPI)
10:20		26.0	Add water	
10:26	6	13.5	2.5	2.40
10:46	20	12.0	1.5	13.33
11:16	20	11.25	0.75	26.67
11:17		27.0	Add water	
11:37	20	15.0	12.0	1.67
11:47	10	12.5	2.5	4.0
11:57	10	12.5	0	No Infiltration
12:07	10	12.5	0	No infiltration



Table 6.4: PT-3 Infiltration Test Data

Time (hr:min)	Interval (min.)	Measurement (in.)	Drop in Water Level (in.)	Percolation Rate (MPI)
10:25		27..5	Add water	
10:30	5	19.0	8.5	0.59
10:35	5	15.5	3.5	1.43
10:40	5	13.0	2.5	2.0
10:45	5	13.0	0	No infiltration
10:50	5	13.0	0	No infiltration
11:21	21	13.0	0	No infiltration
11:42	21	13.0	0	No infiltration

In summary, PT-1 has permeable material within the full depth of the test interval; whereas, PT-2 and PT3 have some permeable material in the near surface, the remainder are fine grained and have limited infiltration capacity. These results were communicated to the design team in our Infiltration Test Preliminary Results Memorandum, dated May 8, 2019.

6.6 Construction Considerations

Excavations will be required to construct building foundations, install utilities, and to remove locally weak or unsuitable soils. All excavations that will be deeper than 5 feet and will be entered by workers should be shored or sloped for safety in accordance with Occupational Safety and Health Administration (OSHA) standards.

Existing weak fills and soils at the site surface will require removal and re-compaction within the proposed improvement areas. The removal should extend to depths where competent soils are encountered. If earthwork is performed during the dry season, moisture conditioning will be required to raise the onsite soil moisture contents to the engineered fill placement and compaction recommendation presented in this report. If earthwork is performed during or shortly after wet weather conditions, the moisture content of the soils could be appreciably above optimum. Consequently, subgrade preparation and fill placement may be difficult. Additional recommendations for wet weather construction can be provided at the time of construction, if required.

During our explorations around the existing structures on campus, we encountered fill with wood debris from 3.5 to 5 feet in depth adjacent to the shop building. Although our investigations at the multipurpose building site did not encounter wood debris, weak fill with heterogenous debris potentially exist throughout the CCC campus. We recommend any weak fill encountered at the building site be removed and replaced with properly engineered fills.

7. RECOMMENDATIONS

7.1 Seismic Design

The proposed new building should be designed to resist the lateral forces generated by earthquake shaking in accordance with Chapter 16 of the 2016 California Building Code (CBC). This section presents seismic design criteria for use with the 2016 CBC and its companion document, ASCE 7-10, *Minimum Design Loads for Buildings and Other Structures*.

The 2016 CBC assigns a Seismic Design Category E to structures classified as Risk Category I, II or III and located at sites with mapped spectral response acceleration for a period of 1 second, S_1 , larger or equal than 0.75 g. Additionally, for structures assigned to a Seismic Design Category E, 2016 CBC requires that the design ground motions be determined according to the site-specific procedures described in Chapter 21 of ASCE 7-10. Based on the United States Geological Survey (USGS) 2008 National Seismic Hazard Map Project (NSHMP) (Petersen et al., 2008), the S_1 for the project site is 0.913 g. Therefore, the design ground motions for the project were calculated using the site-specific procedures in Chapter 21 of ASCE 7-10.

Since the site soils are vulnerable to potential failure or collapse under seismic loading, such as liquefiable soils, ASCE 7-10 requires that the site soils be assigned as Site Class "F" and a site response analysis, in accordance with Section 21.1 of ASCE 7-10, be performed. However, if the fundamental period of vibration of the structure is equal or less than 0.5 seconds, a site response analysis is not required per the exception provided in Section 20.3.1 of ASCE 7-10 and a Site Class can be determined based on non-liquefiable conditions. In this report, we have assumed that the at grade 1-story high building will have a fundamental period of vibration less than 0.5 seconds, and therefore, a site response analysis was not conducted, and the Site Class and seismic design parameters for the project were determined assuming non-liquefiable conditions. Based on the site geology and subsurface soil conditions encountered at the site, we recommend the site be characterized as Site Class "D", described as a "stiff soil" profile, with a time-weighted average shear wave velocity (V_s) in the top 100 feet (30 meters [m]) (V_{s30}) of 230 m/s. A site response analysis according to Section 21.1 of ASCE 7-10 may be needed if the finalized project building design differs from our assumptions or when required by the governing agencies.

The design ground motions for the project were calculated following the site-specific ground motion procedures defined in Chapter 21 (Sections 21.2, 21.3 and 21.4) of ASCE 7-10 as required by 2016 CBC for structures assigned to Seismic Design Category E. The site-specific seismic design response spectra are presented in **Table 7.1**, and design acceleration parameters are presented in **Table 7.2**. Detailed discussions of the development of the site-specific seismic design criteria are included in **Appendix D**.

Table 7.1: MCE_R and Design Response Spectra per ASCE 7-10 for a V_{s30} of 230 m/sec, 5% Damping

Period (sec)	Horizontal Spectral Acceleration (g)	
	Site-Specific MCE_R	Design Response Spectrum
0.01 (PGA)	1.13	0.751
0.03	1.22	0.815
0.05	1.42	0.949
0.075	1.61	1.07
0.1	1.83	1.22
0.12	1.95	1.30
0.128	1.99	1.32
0.15	2.09	1.40
0.2	2.28	1.52
0.25	2.49	1.66
0.3	2.67	1.78
0.4	2.84	1.89
0.5	2.76	1.84
0.6	2.58	1.72
0.638	2.52	1.68
0.75	2.37	1.58
1	1.99	1.33
1.5	1.41	0.941
2	1.08	0.722
3	0.676	0.451
4	0.440	0.293
5	0.309	0.206
7.5	0.155	0.103
8	0.139	0.0927
10	0.0965	0.0643

Table 7.2: Design Acceleration Parameters per ASCE 7-10 for a V_{s30} of 230 m/sec, 5% Damping

Parameter	Value
S_{MS}	2.56 g
S_{M1}	2.17 g
S_{DS}	1.70 g
S_{D1}	1.44 g

7.2 Earthwork

7.2.1 Site Clearing and Preparation

The site should be cleared of all obstructions, including any existing structures and their entire foundation systems, concrete slabs-on grade, existing utilities and pipelines and their associated backfill, designated trees and their associated entire root systems, landscaping, and debris. Concrete/asphalt concrete, baserock, and trench backfill materials can be reused as new fills provided debris is removed and concrete/asphalt concrete are broken up to meet the engineered fill size requirements presented in this report. Additionally, any undocumented fill encountered at the site should be removed.

Holes resulting from the removal of underground obstructions extending below the proposed finish grade should be cleared and backfilled with engineered fills and compacted to the requirements presented in this report. We recommend backfilling operations for any excavations to remove underground obstructions be performed under observations and testing of the project Geotechnical Engineer. After clearing, areas containing heavy surface vegetation should be stripped to an appropriate depth to remove these materials. We estimate the stripping depth to be about 6 inches. The amount of actual stripping should be determined in the field at the time of construction. Stripped materials should be removed from the site or stockpiled for later use in landscaping, if desired.

7.2.2 Subgrade Preparation

Following the site clearing and preparation, soil subgrades in areas to receive engineered fill, slabs-on-grade, or pavements be scarified to a depth of at least 12 inches, moisture conditioned to approximately 3 percent above optimum water content and compacted to the requirements for engineered fills. Locally weak fills and soils, if encountered, should also be excavated and replaced, or otherwise stabilized as recommended by the project Geotechnical Engineer at the time of earthwork operations.

The prepared subgrade surface should be firm, unyielding, and kept moist during construction. The subgrades should be protected from damage caused by weather and construction traffic. If the subgrades are left exposed to weather for extended periods of time or are disturbed by construction traffic, the project Geotechnical Engineer should be consulted on the need for subgrade moisture reconditioning and/or scarifying and recompacting to eliminate shrinkage cracks and disturbances.

7.2.3 Engineered Fill Materials

Any new fills placed at the site should consist of engineered fills that meet the requirements presented in this report, except for landscaping materials which are placed on level ground. All engineered fills should have an organic content of less than 3 percent by volume and should not contain rocks or lumps larger than 4 inches in greatest dimension with not more than 15 percent larger than 2.5 inches.

Onsite soils and fills can be used as new fills. Imported, non-expansive fills should consist of sub-angular to angular particles, have a plasticity index not exceeding 12, and have a significant fine content. All imported fills should not contain environmental contaminants or debris and should be non-corrosive.

7.2.4 Fill Placement and Compaction

Within the upper 5 feet of the finished ground surface, we recommend engineered fills be compacted to at least 90 percent relative compaction, as determined by ASTM D1557. Engineered fills below a depth of 5 feet should be compacted to at least 95 percent relative compaction. The upper 6 inches of subgrade soils beneath pavements should be compacted to at least 95 percent relative compaction. Fill material should be spread and compacted in lifts not exceeding approximately 8 inches in uncompacted thickness.

We recommend engineered fills be moisture conditioned to approximately 3 percent above optimum water content. In order to achieve satisfactory compaction of fill materials, it may be necessary to adjust the water content at the time of earthwork operations. This may require that water be added to soils that are too dry, or that aeration be performed in any soils that are too wet. In order to achieve satisfactory compaction of onsite excavated soils from near or below the existing groundwater level will require drying at the time of construction.

7.2.5 Trench Backfill and Pipe Bedding

Pipeline trenches should be backfilled with engineered fills placed in lifts of approximately 8 inches in uncompacted thickness. Thicker lifts can be used provided the method of compaction is approved by the project Geotechnical Engineer and the required minimum degree of compaction is achieved. Backfill should be placed by mechanical means only; jetting is not permitted. Onsite soils, and onsite and imported fills when used for trench backfill should be compacted to at least 90 percent relative compaction. Imported sands and aggregate bases when used for trench backfill should be compacted to at least 95 percent relative compaction and sufficient water is added during backfilling operations to prevent the soil from "bulking" during compaction. The upper 3 feet of trench backfill in foundation, slab, and pavement areas should be entirely compacted to at least 95 percent relative compaction.

Sand or gravel backfilled trench laterals that extend from irrigated landscaped areas, such as lawns or planting strips, toward pavements, exterior slabs, and building foundations, should be plugged with onsite or imported clayey soils, low strength concrete, or sand-cement slurry mixture below the edges of pavements and exterior slabs, and under perimeters of the foundations. The plugs for the trench laterals should be at least 24 inches thick, extend at least 24 inches beyond the trench walls, and extend from the bottom of the trench to the top of the sand or gravel backfills.

Bedding material should consist of Caltrans Class 2 Aggregate Base or Aggregate Base Course (ABC) meeting the requirements of Section 26 of Caltrans Standard Specifications. All bedding material shall have 3/4-inch maximum aggregate size and be free from organic or vegetable matter, lumps, or balls of silt/clay, or any other deleterious matter. ABC material shall conform to the following gradations when tested in accordance with ASTM C136 or California Test 202.

Table 7.3: Aggregate Base Course Gradation Requirements

Sieve Size (Square Openings)	Percentage by Weight Passing Sieves
1 inch Screen	100
3/4 inch Screen	90 to 100
No. 4 Sieve	35 to 60
No. 30 Sieve	10 to 30
No. 200 Sieve	2 to 9

In addition to the above requirements, all material used shall conform to the following quality requirements:

- Resistance (R-Value) with the minimum test results of 78;
- Sand Equivalent with the minimum test result of 22; and
- Durability Index with the minimum test result of 35.

7.2.6 Exterior Flatwork

We recommend that exterior slabs, such as sidewalks and patios, be placed directly on the properly prepared subgrades in accordance with the recommendations presented in this report. Eliminating aggregate base, gravel, or crushed rock base beneath exterior slabs will reduce the potential for landscape irrigation water to seep through the granular materials and cause the underlying soil subgrades to saturate or pipe. Prior to placing concrete, subgrade soils should be moisture conditioned to increase their moisture content to approximately 3 percent above laboratory optimum moisture (ASTM D-1557).

The clayey soils and fills at the site could be subjected to volume changes during fluctuations in moisture content. As a result of these volume changes, some vertical movement of exterior slabs should be anticipated. This movement could result in damage to the exterior slabs and might require periodic maintenance or replacement. Adequate clearance should be provided between the exterior slabs and building elements that overhang these slabs, such as doors that open outward. We recommend reinforcing exterior slabs with steel bars in lieu of wire mesh. To reduce potential crack formation, considerations should be given to installing of #4 bars spaced at approximately 18 inches on center in both directions. Both score joints and expansion joints can be used to control cracking and allow for expansion and contraction of the concrete slabs.

We recommend appropriate flexible, relatively impermeable fillers be used at all expansion and cold joints. The installation of dowels at all expansion and cold joints will reduce differential slab movements; if used, the dowels should be at least 30 inches long and should be spaced at a maximum lateral spacing of 18 inches. Although exterior slabs that are adequately reinforced will still crack, trip hazards requiring replacement of the slabs will be reduced.

It should be noted, movements or failures of the exterior slabs should be anticipated after major liquefaction events. Repair of the exterior slabs, as well as site regrading, may be needed after the events.

7.2.7 Surface Drainage and Landscaping

We recommend positive surface gradients of at least two percent be provided adjacent to structures and for pavements to direct surface water toward suitable discharge facilities. Roof downspouts and landscaping drainage inlets should be connected to solid pipes that discharge into appropriate facilities. Ponding of surface water must not be allowed adjacent to structure foundations and exterior slabs, adjacent to pavements.

In order to reduce moisture changes in the soils below and adjacent to structure foundations and exterior slabs, landscaping and irrigation systems should be designed and installed in a uniform and systematic manner as equally as possible on all sides of the foundations and adjacent to exterior slabs. If landscaping plans include trees, they should be planted a minimum distance of one-half the anticipated mature height of the trees from improvements to reduce the adverse effects from the tree roots. We recommend that drought resistant plants and low flow/drip irrigation watering systems be used. All irrigation systems should be regularly maintained and inspected for leakage. Over-watering must be avoided.

For bio-retention swales and basins (if planned), where they are located within 10 feet of infrastructure improvements (such as structure foundations, exterior flatwork, and pavements), we recommend they be lined with a relatively impermeable membrane to reduce water seepage and the potential for damage to other infrastructure improvements (such as foundations, exterior slabs, and pavements). The membrane can consist of a layer of STEGO Wrap 15-mil or equivalent installing below and along the sides of these facilities to direct the collected water into subdrain pipes. The membrane should be lapped and sealed in accordance with the manufacturer's requirements, including sealing joints where pipes penetrate the membrane.

The bio-treatment soil mix materials within swales and basins should be considered as having no lateral load resistant. We recommend the sidewall slopes of the swales and basins not to exceed 2:1 (horizontal to vertical) to reduce potential vertical and lateral movements of surrounding ground surface. In addition, we recommend either improvements (foundations, exterior slabs, and pavements) be setback beyond an imaginary 1:1 (horizontal to vertical) plane projected upward from the bottom edges of the swales and basins or the affected areas of the improvements be supported by deepening foundations or edges. Alternatively, properly designed below-grade enclosure structures can be used to build the swales and basins and to retain surrounding ground and improvements.

7.2.8 Construction During Wet Weather Conditions

If construction proceeds during or shortly after wet weather conditions, the moisture content of the onsite soils could be appreciably above optimum. Consequently, subgrade preparation, placement of onsite soil as structural fill might not be possible. A geotechnical engineer can provide alternative wet weather construction recommendations in the field at the time of construction, if appropriate.

7.3 Building Foundation

Due to the site liquefaction hazard potential during MCE events, we recommend the proposed building be supported on a deep foundation system, such as Case-in-Drilled-Hole (CIDH) or Auger Cast Piles (ACP).

The deep foundation should also be used to support any exterior equipment that is considered essential parts of the building. Building interior ground floor slabs should be designed to span between grade beams that are supported by piers or piles.

The deep foundation should be designed to resist downdrag loads that would be potentially imposed upon the foundations due to site soil liquefaction during MCE events. Consideration should also be given to using flexible pipe connections to tolerate the estimated potential liquefaction-induced settlement of 3 inches at locations where the pipes are connected to pile-supported structures.

7.3.1 Pile Axial Load Capacity

The proposed building can be supported by Case-in-Drilled-Hole (CIDH) piers that develop their load carrying capacity from soil frictions/adhesions. We recommend the piers be at least 18 inches in diameter and have a center-to-center spacing of at least 3 times the shaft diameter. The design lengths of the piers can be determined using the recommended ultimate skin frictions provided in the **Table 7.4**. In calculating the axial vertical capacity, ignore friction above 25 feet.

Table 7.4: Recommended Pier Ultimate Skin Friction

Pier Embedment Length (ft)	Skin Friction (psf)
25 to 60	2,000
Below 60	3,000

The allowable axial capacity can be calculated by dividing the ultimate axial capacity by the factors of safety provided in **Table 7.5**. Eighty (80) percent of the skin friction capacity may be used in determining uplift resistance. In addition, the weight of the pier can be ignored in the calculations of the axial capacity.

Table 7.5: Factors of Safety for Axial Loading of Pier Foundation

Load Condition	Factor of Safety
Dead Load	3
Dead plus Live Loads	2
Total Loads (including wind or seismic)	1.5

The piers should also be designed to resist downdrag loads that would be imposed upon the foundations due to potential liquefaction of the isolated sand lenses between the upper 10 and 25 feet of the pier shaft. We recommend an average negative skin friction of 1,300 psf be included along the upper 25 feet of the pier shaft to account for the potential liquefaction-induced downdrag forces.

Static total and differential settlements of the pier supported structure are estimated to be insignificant (i.e., less than 0.5 inch) and within tolerable limits for the proposed structure. Seismic settlement of the pier is estimated to be less than 1 inch assuming the pier is designed to resist the downdrag force only using pier shaft skin friction.

In order to provide pier “fixity” to resist lateral loading, we recommend a minimum pile length of 30 feet based on our L-pile analysis results.

7.3.2 Pier Lateral Load Capacity

We evaluated pier lateral load capacities using a computer program LPILE Version 2019.11.01 (Ensoft, 2019) to model subsurface soils as a series of discrete resistances with nonlinear behavior. Our preliminary analyses assumed a 60-foot long, 18-inch diameter pier with a design concrete strength (f'_c) of 5,000 pounds per square inch (psi). An elastic pier section with a bending stiffness of 1.04×10^{10} pound square inches (lb-in^2) was modeled, which represents an assumed cracked upper pier section with a 50 percent reduction in bending stiffness. The top of the pier is assumed to be located at grade. The pier axial loads and cap passive resistances were not included in our analyses.

Our estimated lateral loads for 1/4-inch, 1/2-inch, and 1-inch of lateral displacements at pier head for each pier and loading case are presented in the **Table 7.6**. The LPILE printouts of calculated pier deflection, bending moment, and shear force versus embedment depth are presented on **Plate 8**. We recommend the piers be at least 30 feet long in order to provide appropriate pier “fixity” to resist lateral loadings.

Table 7.6: Estimated 60' Long 18" Dia. Pile Lateral Load Capacities

Load Case	Pier Head Condition	Pier Head Displacement (inches)	Lateral Load at Pier Head (kips)	Max. Moment in Pier (kip-ft)
1	Free	0.25	10.8	33.2
2	Free	0.5	15.8	52.4
3	Free	1.0	23.4	84.1
4	Fixed	0.25	22.7	85.7
5	Fixed	0.5	33.7	141.7
6	Fixed	1.0	51.1	239.8

A soil passive resistance equal to an equivalent fluid weighing 350 pcf (pounds per cubic foot) can also be used against the vertical face of the pier caps and grade beams embedded within the onsite fills. The upper 12 inches of soils should be neglected in passive resistance design unless they are confined by a pavement or slab. This passive resistance should be considered as an ultimate value (a factor of safety of 1.0) and assumes a deflection of approximately 1/2 inch in order to fully mobilize the passive resistance. Movement compatibility evaluations should be performed for both the pier cap and pier shaft in order to develop the desired passive resistance.

For closely spaced piers, the shear planes in the soil overlap and the lateral resistance for a pier within the group is less than that of a single pier. We note that the leading piles are generally less impacted by group effects and tend to draw higher loads. To account for the reduction of soil resistance because of group effects, we recommend multiplying the lateral loads by the reduction factors provided in the **Table 7.7**. Reduction factors, or p-multipliers, are a function of center-to-center spacing where D is the pile diameter. P-multipliers should be applied to trailing piles in the direction of loading.

As an example, a 1 by 6 pier row with a center-to-center spacing of 6 diameters and loaded in the direction parallel to the pile row would use a p-multiplier of 1.0 for the lead pile and 0.7 for all trailing piles. The same group loaded perpendicular to the pier row would use a p-multiplier of 1.0 for all pier. Linear interpolation may be used for other pier spacing.

Table 7.7: Reduction Factors for Pier Lateral Load Capacity

Center-to-Center Spacing	p-Multiplier
8D	1.0
6D	0.7
4D	0.4
3D	0.3

7.3.3 Pier Construction

We recommend that the installation or excavation of all piers be performed under the direct observation of the project Geotechnical Engineer to confirm that the pier foundations are founded in suitable materials and constructed in accordance with the recommendations presented herein. All piers should be installed or constructed vertically to their design tip elevations at the specified locations in order to develop adequate vertical pier capacities.

Underground obstructions may be encountered during pier hole excavation. Where caving/sloughing due to seepage or high groundwater conditions exist, additional measures such as using casing, polymer drilling slurry, and pouring concrete by tremie methods immediately after excavating may be necessary. We recommend concrete pours of pier excavations be performed on the same day as drilling. We should be consulted on the need for additional measures for pier construction as needed during drilling of the initial piers.

Alternatively, Auger Cast Piles (ACP) may be used in lieu of drilled piers. The alternative foundation system, if proposed, should be designed and installed by a specialty contractor who is familiar with the site subsurface conditions. Fugro should review the designs of the alternative foundation system.

7.3.4 Building Interior Ground Floor Slab

The interior ground floor slab should consist structural slabs that are designed to span between grade beams that are supported by piers or piles. The slab should be underlain by an at least 12 inches thick of properly compacted non-expansive engineered fills that extend at least 3 feet beyond the foundation footprints.

If migration of water vapor through interior slab is undesirable, we recommend a vapor retarder and an underlying 4-inch layer of $\frac{3}{4}$ -inch, clean, crushed, uniformly graded gravel/drain rock be placed between the bottom of the slab and the recommended imported non-expansive engineered fill layer. The gravel/drain rock layer can be considered as part of the non-expansive engineered fill layer. We recommend the vapor retarder consist of a single layer of Stego Wrap Vapor Barrier 15 mil or equivalent provided the equivalent

satisfies the following criteria: a permeance less than 0.01 perms as guided by ACI 302.2R, Class A strength as determined by ASTM E1745, and a thickness of at least 15 mils. Installation of the vapor retarder, including protrusions where pipes or conduit penetrate the membrane, should conform to ASTM E1643 and the manufacturer's requirements. Care must be taken to protect the membrane from tears and punctures during construction. We do not recommend placing sand or gravel over the membrane. The subgrade below the slab should be properly prepared, firm, and non-yielding. All foundation excavations should be kept moist and free of loose soils and standing water prior to concrete placement.

Concrete slabs retain moisture and often take many months to dry; construction water added during the concrete pour further increases the curing time. If the slabs are not allowed to completely cure prior to constructing the super-structure, the concrete slabs will expel water vapor and the vapor will be trapped under impermeable flooring. A proper water/cement ratio should be determined by the foundation designers for the slabs to reduce vapor transmitting if need. We recommend the foundation designer determine if corrosion protection is needed for the foundation concrete and reinforcing steel. The corrosivity test results of onsite soil samples are included in **Appendix B**; the foundation designer should determine if additional testing is needed. In addition, the foundation designers should provide recommendations to reduce the potential for differential concrete curing if necessary.

7.4 Additional Geotechnical Services

Fugro should review geotechnical aspects of the plans and specifications to check for conformance with the intent of our recommendations. We recommend that Fugro be also retained to provide geotechnical services during earthwork operation and foundation installation to observe compliance with the design concepts, specifications, and recommendations presented in this report. Our presence will also allow us to modify design if unanticipated subsurface conditions are encountered.

8. LIMITATIONS

The opinions, conclusions, and recommendations presented in this report are based on our reviews of available geologic and geotechnical data, maps, reports, our site subsurface exploration and laboratory testing results, our engineering analysis results, and information provided by others. Our opinions, conclusions, and recommendations are solely professional opinions and were made in accordance with generally accepted local and current geotechnical engineering principles and practices. We make no warranty, either express or implied.

Site exploration and testing characterizes subsurface conditions only at the locations where the explorations or tests are performed and at the time when services were conducted; actual subsurface conditions between explorations or tests may be different than those described in this report. Variations of subsurface conditions from those analyzed or characterized in this report are not uncommon and may become evident during construction. In addition, changes in the condition of the site can occur over time as a result of either natural processes (such as earthquakes, flooding, or changes in ground water levels) or human activity (such as construction adjacent to the site, dumping of fill, or excavating). If changes to the site's surface or subsurface conditions occur since the performance of the field work described in this report, or if differing subsurface conditions are encountered, we should be contacted immediately to evaluate the differing conditions to assess if the opinions, conclusions, and recommendations provided in this report are still applicable or should be amended.

This report has been prepared for the exclusive use of the Department of General Services and their consultants for specific application to the proposed California Conservation Corp Multipurpose Building Project in Fortuna, California as described herein. In the event that there are any changes in the nature, design or location of the project, as described in this report, or if any future additions are planned, the conclusions and recommendations contained in this report shall not be considered valid unless the project changes are reviewed by us, and the conclusions and recommendations presented in this report are modified or verified in writing.

Reliance on this report by others must be at their risk unless we are consulted on the use or limitations. We cannot be responsible for the impacts of any changes in geotechnical standards, practices, or regulations subsequent to performance of services without our further consultation. We can neither vouch for the accuracy of information supplied by others, nor accept consequences for unconsulted use of segregated portions of this report.

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PLATES



Base map from Esri



Imagery from NAIP, 2018.



Imagery from NAIP, 2018.

Legend

Proposed Explorations

-  Boring
-  CPT
-  Percolation test

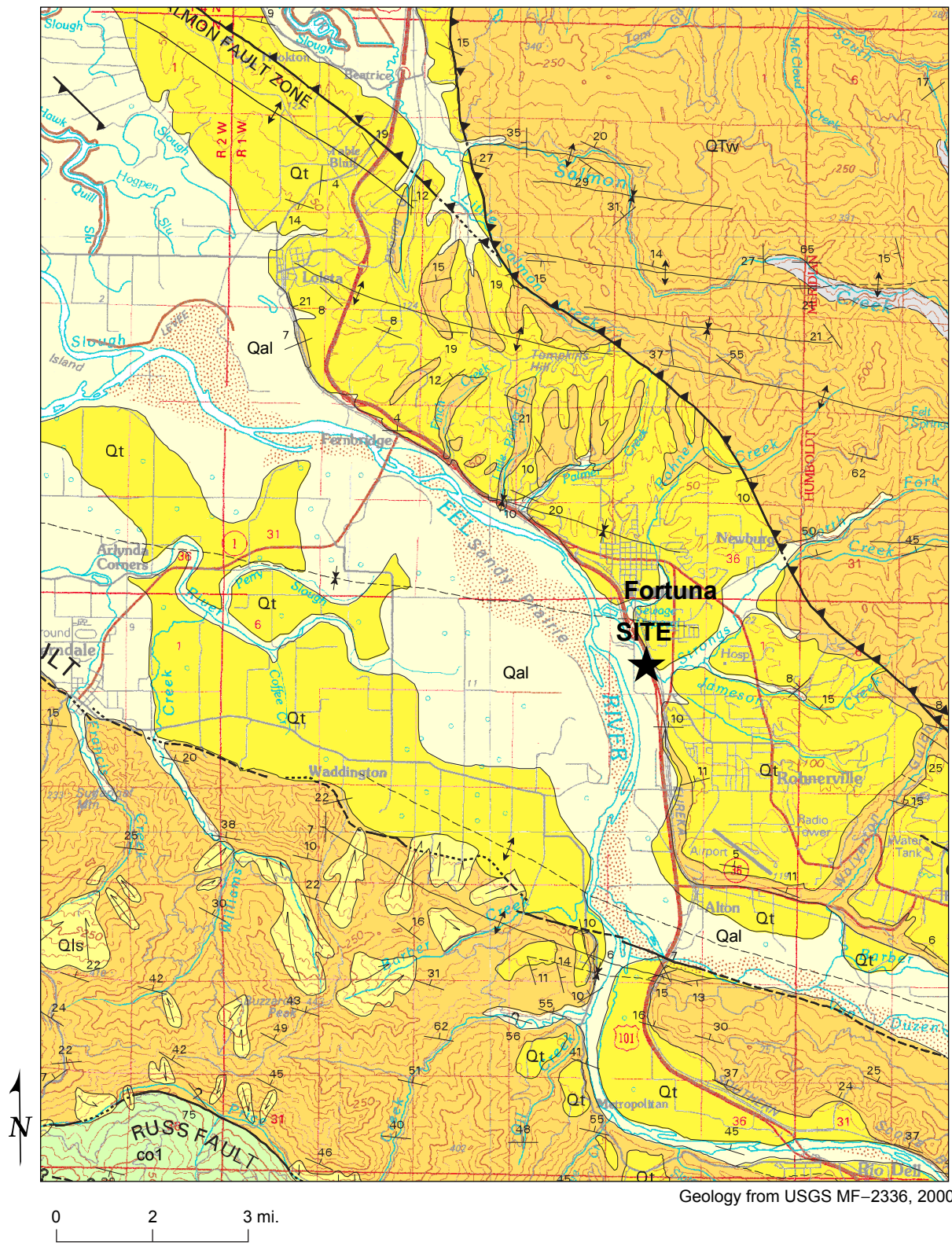
Previous Explorations

-  Boring
-  Test pit

Note: CPT locations include 1A, 1B, 2A, and 2B.

INVESTIGATION LOCATIONS MAP

PLATE 3



See Figure 4B for explanation of geologic units

GEOLOGIC MAP WITH FAULTS

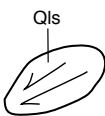
PLATE 4A

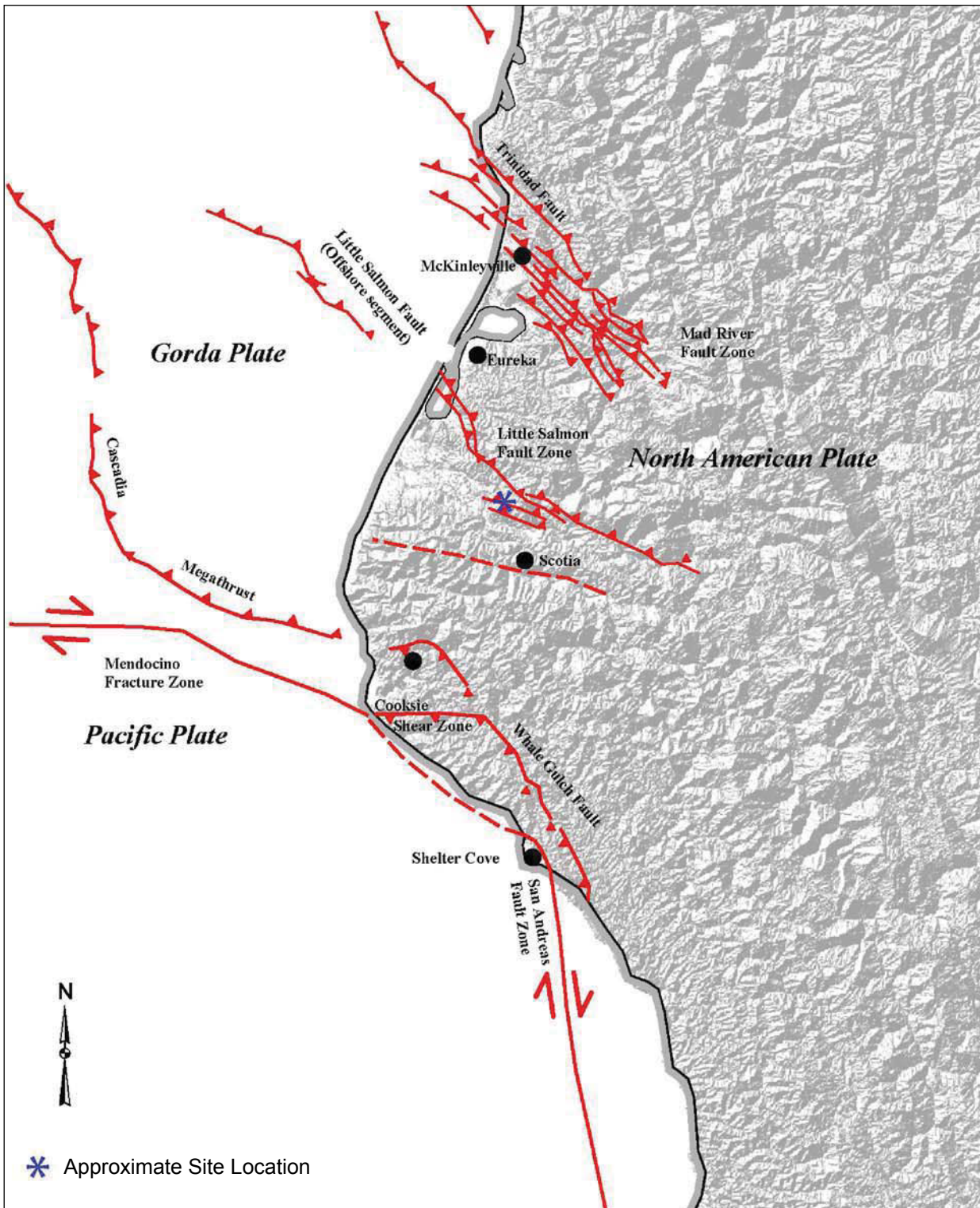
Geologic Units

Qal	Alluvial deposits (Holocene and late Pleistocene?) -Clay, silt, sand, gravel, and boulders, deposited in stream beds, alluvial fans, terraces, flood plains and ponds; and soils formed on these deposits. Includes largely Holocene deposits in modern stream channels and on flood plains
Qt	Undifferentiated nonmarine terrace deposits (Holocene and Pleistocene) - Dissected and (or) uplifted gravel, sand, silt, and clay, deposited in fluvial settings. In western Eureka quadrangle (Sheet 1) unit includes minor shallow marine intertongues and warped and tilted beds of late Pleistocene Hookton and Rohnerville Formations of Ogle (1953), in addition to younger late Pleistocene and Holocene fluvial terrace units a few feet to a few tens of feet higher than normal modern high-water level
Qls	Landslide deposits (Holocene and Pleistocene) -Unsorted clay- to boulder-size debris and broken rock masses that have moved downslope in debris flows, earth flows, and as more-or-less intact rotational or translational blocks, largely from Pleistocene to present. Only large landslides, occupying tens to hundreds of acres, are depicted here.
QTW	Marine and nonmarine overlap deposits (late Pleistocene to middle Miocene) -Thin-bedded to massive, weakly lithified siltstone, fine- to medium-grained sandstone, silty to diatomaceous mudstone and locally soft, scaly mudstone. Locally includes lenses of pebble to boulder conglomerate, carbonate concretions, abundant molluscan fossils, woody debris, and horizons of rhyolitic volcanic ash that are greater than 1 meter thick in some areas. Includes the Wildcat Group (Ogle, 1953), the Bear River beds (Haller, 1980), and related outlier Neogene deposits isolated along faults near Briceland, Garberville, Benbow, Piercy, Bridgeville and northeast of Weott. Unit also includes minor fault-bounded blocks along or near the coast between Bear River and the Mattole River that are incorporated into melange of the Coastal terrane; the Neogene Falor Formation northeast of Eureka (Manning and Ogle, 1950); and equivalent deposits in the offshore area deposited in shelf, slope, and slope basin settings. A few poorly exposed erosional remnants of shallow marine to brackish water strata mapped along high ridge crests overlying the Franciscan Complex in the 1:24,000 Zenia quadrangle are tentatively assigned to this unit. South of this map, unit correlates with valley-fill, perched gravel and shallow marine to nonmarine coal-bearing sedimentary rocks of Quaternary and Tertiary age in the Round Valley area of Covelo 1:100,000 quadrangle (Jayko and others, 1989)

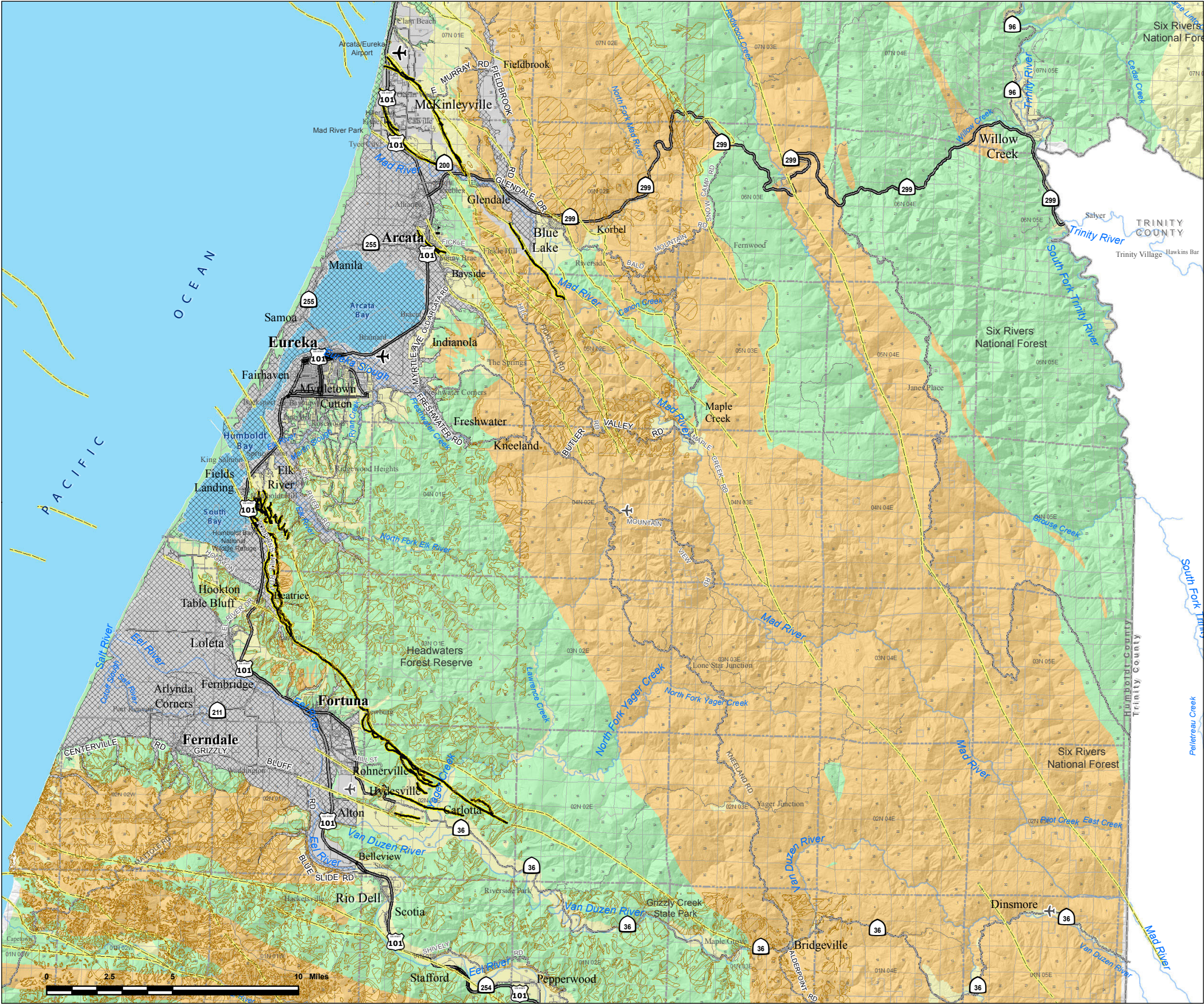
co1	Melange -Dominantly of highly folded argillite and abundant clayey, penetratively sheared rock that exhibits rounded, lumpy, and irregular, poorly incised topography
-----	--

Symbols

	Landslide -Arrows indicate direction of movement
	Inclined -Ball denotes top of beds is known from sedimentary features
	Thrust fault -Barbs on upper plate, dashed where approximate, dotted where concealed, queried where uncertain
	Synclinal or synformal axis -showing direction of plunge
	Anticlinal or antiformal axis -showing direction of plunge
	Contact -Dashed where approximate, dotted where concealed, queried where uncertain
	Fault -Dashed where approximate, dotted where concealed, queried where uncertain



REGIONAL FAULT MAP



Legend

- Alquist Priolo Fault
- Historic Quaternary Faults
- Historic Landslides
- Area of Potential Liquefaction

Seismic Safety

- 3 High Instability
- 2 Moderate Instability
- 1 Low Instability
- 0 Relatively Stable

Humboldt County General Plan Update
HUMBOLDT21st Century

Sources:

Alquist-Priolo Earthquake Fault Zones and fault traces as shown on Official Alquist-Priolo Earthquake Fault Zones Maps produced by the California Geologic Survey.

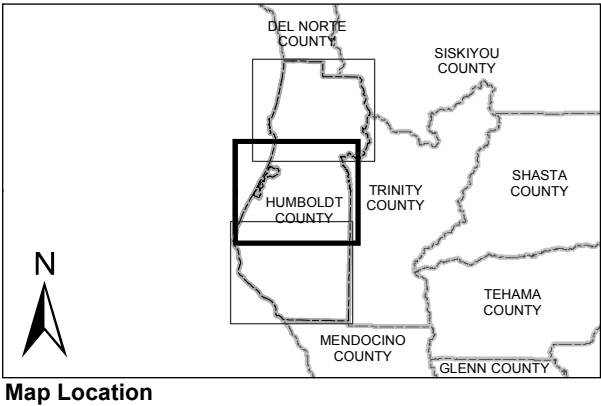
Historic Quaternary Faults, USGS - This data set contains locations and information on faults and associated folds in the United States that are believed to be sources of M>6 earthquakes during the Quaternary (the past 1,600,000 years). <http://earthquake.usgs.gov/regional/qfaults/>

Historic Landslides Geologic units and geomorphic features were mapped by the California Department of Conservation Division of Mines and Geology (1999)

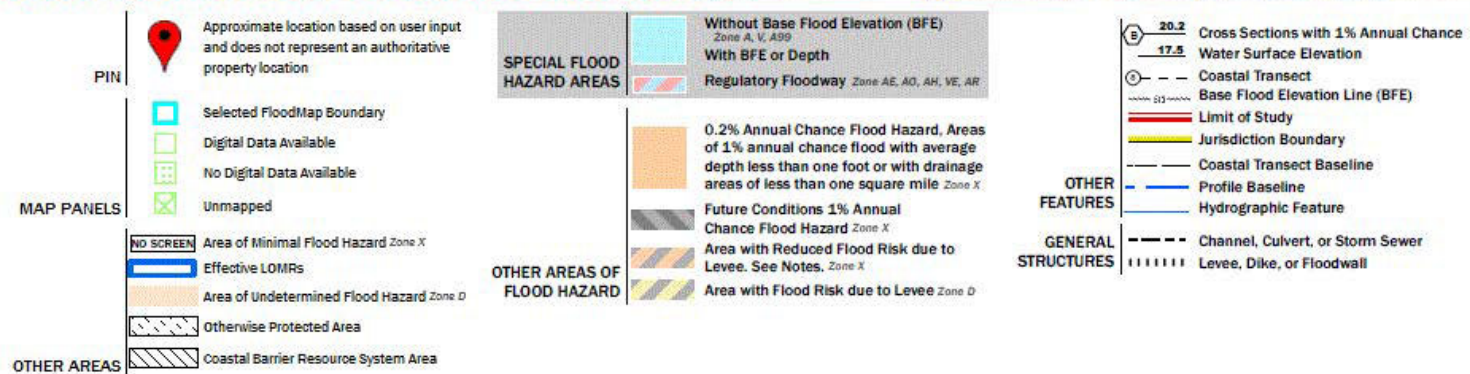
Slope Stability and Liquefaction Zones for Humboldt County. Digitized using Humboldt County General Plan Geologic Map (1984) and Seismic Safety Maps (1979) - Humboldt Bay and Vicinity, North and South Sheets.

This map is intended for planning purposes only and should not be used for precise measurement or legal boundary delineation. Original map size is 17"X11". Map produced by Humboldt County Planning and Building Department, Feb 2015.

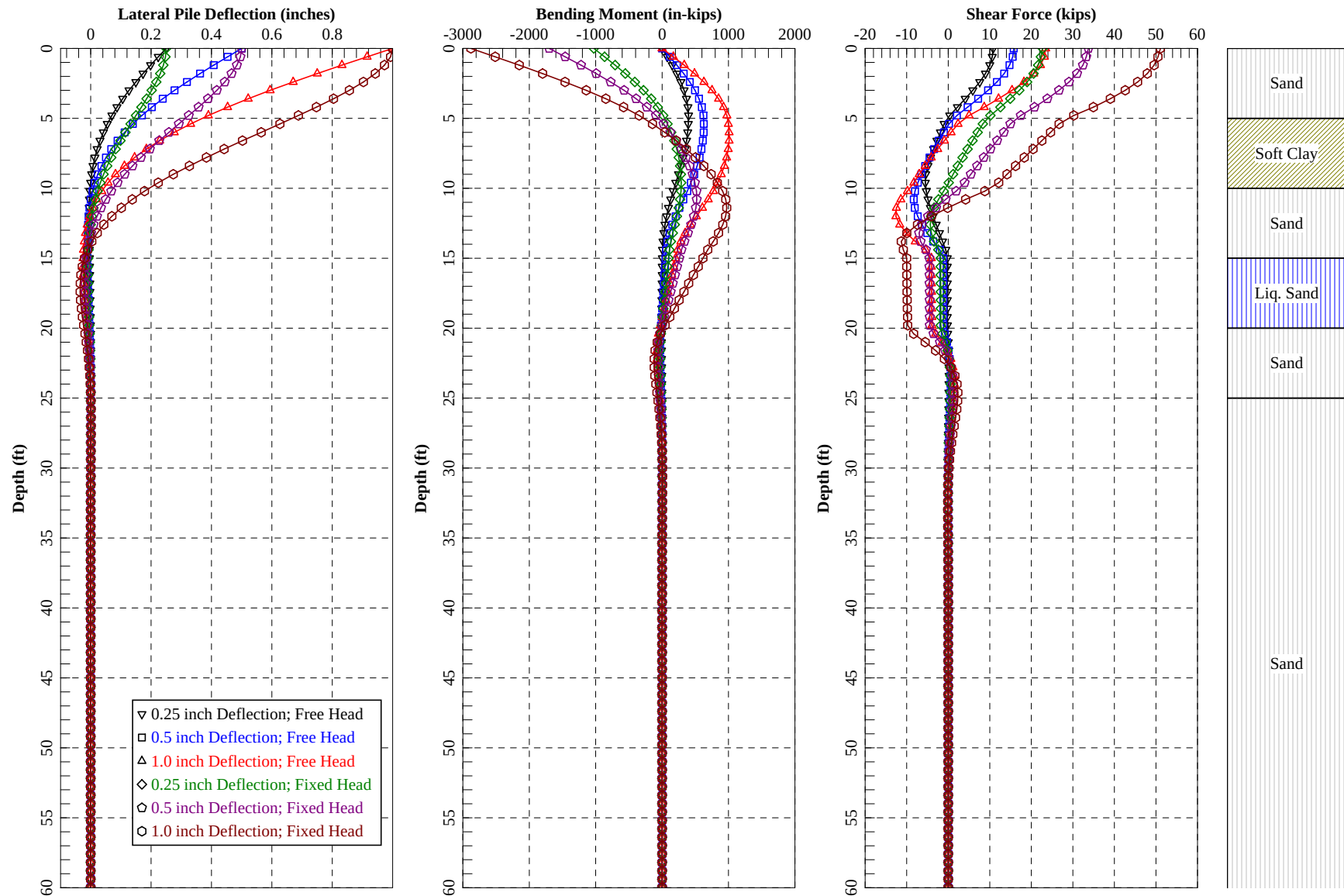
Map Document: H:\aps\3sheets2015_SeismicSafety.mxd Plots Directory: H:\plots\GPU2015draft\



LIQUEFACTION POTENTIAL MAP
(CENTRAL HUMBOLDT SEISMIC SAFETY)



FEMA FLOOD INSURANCE RATE MAP



Estimated 60' Long 18" Dia. Pile Lateral Load Capacities, Elastic Cracked Section (50% EI)

LPile Lateral Load Analysis Results



APPENDIX A FIELD EXPLORATIONS

CLASSIFICATION AND MATERIAL SYMBOLS

MAJOR DIVISIONS PER ASTM D2488-06			MAJOR GROUP NAMES AND MATERIAL SYMBOLS	
COARSE-GRAINED SOILS More than 50% retained on the No. 200 sieve	GRAVELS MORE THAN 50% OF COARSE FRACTION RETAINED ON NO. 4 SIEVE	Clean gravels less than 5% fines	GW	Well-Graded GRAVEL
			GP	Poorly Graded GRAVEL
		Gravels with more than 12% fines	GM	SILTY GRAVEL
			GC	CLAYEY GRAVEL
	SANDS MORE THAN 50% OF COARSE FRACTION PASSING NO. 4 SIEVE	Clean sand less than 5% fines	SW	Well-Graded SAND
			SP	Poorly Graded SAND
		Sands with more than 12% fines	SM	SILTY SAND
			SC	CLAYEY SAND
FINE-GRAINED SOILS 50% or more passes the No. 200 sieve	SILTS AND CLAYS Liquid Limit Less than 50%		ML	SILT
			CL	Lean CLAY
			OL	ORGANIC SILT
	SILTS AND CLAYS Liquid Limit Greater than 50%		MH	Elastic SILT
			CH	Fat CLAY
			OH	ORGANIC CLAY
	HIGHLY ORGANIC SOILS		PT	Peat or Highly Organic Soils
	Notes: Classification of soils on the boring logs is in general accordance with ASTM D2488, or D2487 if appropriate laboratory data are available. The geologic formation is noted in bold font at the top of interpreted interval on the boring logs.		OTHER MATERIAL SYMBOLS	
			Debris or Mixed Fill	
			Pavement with Aggregate Base	

SAMPLER TYPE

SPT (Driven) 1-3/8" ID 2" OD	Modified California (Driven) 2-3/8" ID 3" OD	Modified California (Driven) 1-7/8" ID 2-1/2" OD
Shelby Tube (Pushed) 2-7/8" ID 3" OD	Pitcher Barrel (Rotary-cut) 2-7/8" ID	Osterberg (Piston) 2-7/8" ID
101 Geobarrel (Rotary-cut) 2-7/8" ID	Rock Core (Rotary-cut) See log for size	Vibracore (Vibrated) See log for size
Push-core (Pushed) See log for size	Collected from Auger	Other See log for details

Note: Refer to text of report for additional details or other sampler types.

BLOW COUNT

Number of blows required to drive sampler each of three 6-in. intervals, as measured in the field (uncorrected). An SPT hammer (140 lb., falling 30-in.) was used unless otherwise noted on the boring log. For example:

Blow Count	Description
5 7 8	5, 7, and 8 blows for first, second, and third interval, respectively.
35 50/3"	35 blows for the first interval. 50 blows for the first 3 inches of the second interval. Lack of third value implies that driving was stopped 3 inches into the second interval.
WOH WOH 5	"WOH" indicates that the weight of the hammer was sufficient to advance the sampler over the first two intervals. 5 blows were required to advance the sampler over the third interval.

N-VALUE

The N-Value represents the blowcount for the last 12 inches of the sample drive if three 6-inch intervals were driven. N-value presented is independent of impact energy. If 50 hammer blows were insufficient to drive through either the second or the third interval, the total number of blows and total length driven are reported (excluding the first interval). "ref" (refusal) indicates that 50 blows were insufficient to drive through the first 6-inch interval.

Parenthesis indicate that an approximate correction has been applied for non-SPT drive samplers. For example, a factor of 0.63 is commonly used to adjust blow counts obtained using a 3-inch outside diameter modified California sampler to correspond to Standard Penetration Test.

UNDRAINED SHEAR STRENGTH

A value of undrained shear strength is reported. The value is followed by a letter code indicating the type of test that was performed, as follows:

U - Unconfined Compression
Q - Unconsolidated Undrained Triaxial
T - Torvane
P - Pocket Penetrometer
M - Miniature Vane
F - Field Vane
R - R-value

OTHER TESTS

Field or laboratory tests without a dedicated column on the boring log are reported in the Other Tests column. A letter code is used to indicate the type of test. For certain tests, a value representing the test result is also provided. Typical letter codes are as follows. Additional codes may be used. Refer to the report text and the laboratory testing results for additional information.

k - Permeability (cm/s)
Consol - Consolidation
Gs - Specific Gravity
MA - Particle Size Analysis
EI - Expansion Index
OVM - Organic Vapor Meter

WATER LEVEL SYMBOLS

▽ Initial water level
▼ Final water level
~ Seepage encountered

INCREASING MOISTURE CONTENT

↓ Dry
Moist
Wet

CONSISTENCY OF COHESIVE SOIL

CONSISTENCY	UNDRAINED SHEAR STRENGTH (KIPS PER SQUARE FOOT)
Very Soft	< 0.25
Soft	0.25 to 0.50
Medium Stiff	0.50 to 1.0
Stiff	1.0 to 2.0
Very Stiff	2.0 to 4.0
Hard	> 4.0

Note: In absence of test data, consistency has been estimated based on manual observation.

APPARENT DENSITY OF COHESIONLESS SOIL

APPARENT DENSITY	N-VALUE
Very Loose	0 to 4
Loose	5 to 9
Medium Dense	10 to 29
Dense	30 to 49
Very Dense	> 49

DEPTH, ft	MATERIAL SYMBOL	SAMPLER TYPE	BLOW COUNT OR PRESSURE, psi	N VALUE OR RQD%	RECOVERY	LOCATION:	DRY UNIT WEIGHT, pcf	WATER CONTENT, %	% PASSING #200 SIEVE	LIQUID LIMIT, %	PLASTICITY INDEX	UNDRAINED SHEAR STRENGTH, S_u , ksf	OTHER TESTS
MATERIAL DESCRIPTION													
						ALLUVIUM (Qal) SANDY Lean CLAY (CL): medium stiff, brown, moist, low plasticity, 43.8% sand, 56.2% lean clay.							
						ALLUVIUM (Qal) Poorly-graded GRAVEL with SAND (GP): cobble sized gravel.							
						ALLUVIUM (Qal) SANDY Lean CLAY (CL): 43.8% sand, 56.2% lean clay.			56	26	9		
5						ALLUVIUM (Qal) Poorly-graded SAND (SP): dense, dark brown, moist, 61.5% medium to coarse sand, 22.4% silt, 16.1% subangular to subrounded gravel.	114	12	22				
						ALLUVIUM (Qal) SANDY SILT (ML): medium dense to dense, brown, moist, medium to coarse grained sand, low plasticity, trace subangular to subrounded gravel.							
10						ALLUVIUM (Qal) SANDY SILT (ML): very loose, brown, moist, yellow mottling, non-plastic, no dilatency, 5% sub-angular gravel, trace 3" clasts.			69				
						TERRACE DEPOSITS (Qt) Well-graded SAND (SW): medium dense, grayish brown, moist, 46.7% fine to coarse gravel, 48.7% fine to coarse sand.							
15						TERRACE DEPOSITS (Qt) SAND with GRAVEL (SP-SW): loose, grayish brown, moist, fine to medium sand, fine gravel - subangular to subrounded.			5				
						- Cuttings sample: 22-24 feet							
25						TERRACE DEPOSITS (Qt) SANDY GRAVEL (GP-GW): dense to very dense, grayish brown, moist, 29.7% sand, 64.5% subangular to subrounded gravel, low plasticity fines, 0-5% 3" clasts.			6				
						TERRACE DEPOSITS (Qt) SAND with GRAVEL (SP-SW): dense, grayish brown, moist, fine to medium sand, fine gravel- subangular to subrounded.							
						TERRACE DEPOSITS (Qt)							

Continued

BORING DEPTH: 51.5 ft
BACKFILL: Grout
DEPTH TO WATER: 28.3 ft
FIELDWORK DATE: April 30, 2019
DRILLING METHOD: 6-in. dia. Hollow Stem Auger

HAMMER TYPE: Automatic Trip
RIG TYPE: Truck Mounted CME 45
DRILLED BY: MTA, J. Sheddon
LOGGED BY: J. Wilson
CHECKED BY: D. Marlow

LOG OF BORING NO. B-1
CCC Fortuna
Fortuna, California

PLATE A-2



DEPTH, ft	MATERIAL SYMBOL	SAMPLER TYPE	BLOW COUNT OR PRESSURE, psi	N VALUE OR RQD% RECOVERY	LOCATION:	DRY UNIT WEIGHT, pcf	WATER CONTENT, %	% PASSING #200 SIEVE	LIQUID LIMIT, %	PLASTICITY INDEX	UNDRAINED SHEAR STRENGTH, S_u ksf	OTHER TESTS
					MATERIAL DESCRIPTION							
		S-7	18 25 23	48	7.5 18"	SANDY GRAVEL (GP-GW): dense, brown, wet, 38.6% sand, 55.2% subangular to subrounded gravel (5% 3" clasts), low plasticity fines.		6				
35		S-8	8 13 11	24	10 18"	TERRACE DEPOSITS (Qt) SANDY GRAVEL (GP-GW): medium dense, grayish brown, wet, subangular to subrounded gravel, low plasticity fines, 5-10% 3" clasts. 36.4% Sand, 60.9% Gravel.		3				
40		S-9	4 10 19	29	14.5 18"	- increasing sand content						
45		S-10	15 22 22	44	16 18"	- dense Sandy (37.5%) GRAVEL (57.3%)		5				
50		S-11	6 11 16	27	11 18"	- Sandy (32.2%), GRAVEL (64.3%).		4				
					TD Boring at 51.5 feet bgs. Initial groundwater encountered at 30 feet bgs. NOTES: 1. Terms and symbols defined on Plate A-1.							

FCLP STANDARD LOG C:\USERS\PUBLIC\DOCUMENTS\BENTLEY\GINT\PROJECTS\CCC FORTUNA.GPJ GINT ENVIRONMENTAL LIBRARY\GLB 7/24/19 10:12 a



DEPTH, ft	MATERIAL SYMBOL	SAMPLER TYPE	BLOW COUNT OR PRESSURE, psi	N VALUE OR RQD%	RECOVERY	LOCATION:	DRY UNIT WEIGHT, pcf	WATER CONTENT, %	% PASSING #200 SIEVE	LIQUID LIMIT, %	PLASTICITY INDEX	UNDRAINED SHEAR STRENGTH, S_u ksf	OTHER TESTS
5		S-1B S-1A S-2	4 6 4	7	18 18 18	ALLUVIUM (Qal) SILT (ML): medium dense, brown, dry, roots, 10% gravel.							
						- increased clay content before 4 feet.							
						TD Boring at 5.5 feet bgs. No groundwater encountered. NOTES: 1. Terms and symbols defined on Plate A-1.							

BORING DEPTH: 5.5 ft
 BACKFILL: Cuttings
 DEPTH TO WATER: Not Encountered
 FIELDWORK DATE: April 30, 2019
 DRILLING METHOD: 6-in. dia. Hollow Stem Auger

HAMMER TYPE: Automatic Trip
 RIG TYPE: Truck Mounted CME 45
 DRILLED BY: MTA, J. Sheddon
 LOGGED BY: A. Leven
 CHECKED BY: D. Marlow

LOG OF BORING NO. B-2
 CCC Fortuna
 Fortuna, California

PLATE A-4

DEPTH, ft	MATERIAL SYMBOL	SAMPLER TYPE	BLOW COUNT OR PRESSURE, psi	N VALUE OR RQD%	RECOVERY	LOCATION: MATERIAL DESCRIPTION	DRY UNIT WEIGHT, pcf	WATER CONTENT, %	% PASSING #200 SIEVE	LIQUID LIMIT, %	PLASTICITY INDEX	UNDRAINED SHEAR STRENGTH, S _u , ksf	OTHER TESTS
						6 Inches of Asphalt and Base.							
						Possible Engineered Fill SILT with SAND (ML): medium dense, light brown, moist, 15-20% sand, <5% gravel, medium plasticity fines, subangular to angular gravel.							
						- increase in gravel content to 5-10%.							
						ARTIFICIAL FILL (af) Poorly-graded SAND (SP): medium dense to dense, light brown, moist, low plasticity fines, 5-10% subangular to subrounded gravel, 20-50% wood debris at 4-5 feet bgs, at 4 feet ~50% wood debris							
						TD Boring at 5.5 feet bgs. No groundwater encountered. NOTES: 1. Terms and symbols defined on Plate A-1.							

BORING DEPTH: 5.5 ft
BACKFILL: Grout
DEPTH TO WATER: Not Encountered
FIELDWORK DATE: April 30, 2019
DRILLING METHOD: 6-in. dia. Hollow Stem Auger

HAMMER TYPE: Automatic Trip
RIG TYPE: Truck Mounted CME 45
DRILLED BY: MTA, J. Sheddon
LOGGED BY: J. Wilson
CHECKED BY: D. Marlow



DEPTH, ft	MATERIAL SYMBOL	SAMPLER TYPE	BLOW COUNT OR PRESSURE, psi	N VALUE OR RQD%	RECOVERY	LOCATION:	DRY UNIT WEIGHT, pcf	WATER CONTENT, %	% PASSING #200 SIEVE	LIQUID LIMIT, %	PLASTICITY INDEX	UNDRAINED SHEAR STRENGTH, S_u ksf	OTHER TESTS
1						Possible Engineered Fill SILTY SAND with GRAVEL (SM): loose to medium dense, light brown to brown, moist, 15-20% gravel, 5-10% cobble.							
						TD Boring, encounter refusal at 14 Inches bgs. No groundwater encountered. NOTES: 1. Terms and symbols defined on Plate A-1.							

BORING DEPTH: 1.2 ft
 BACKFILL: Cuttings
 DEPTH TO WATER: Not Encountered
 FIELDWORK DATE: April 30, 2019
 DRILLING METHOD: 6-in. dia. Hand Auger

HAMMER TYPE: N/A
 RIG TYPE: N/A
 DRILLED BY: MTA, J. Sheddon
 LOGGED BY: J. Wilson
 CHECKED BY: D. Marlow

LOG OF BORING NO. B-4
 CCC Fortuna
 Fortuna, California

PLATE A-6



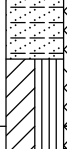
DEPTH, ft	MATERIAL SYMBOL	SAMPLER TYPE	BLOW COUNT OR PRESSURE, psi	N VALUE OR RQD%	RECOVERY	LOCATION: MATERIAL DESCRIPTION	DRY UNIT WEIGHT, pcf	WATER CONTENT, %	% PASSING #200 SIEVE	LIQUID LIMIT, %	PLASTICITY INDEX	UNDRAINED SHEAR STRENGTH, S_u ksf	OTHER TESTS
						2 Inches of grass/organic matter Possible Engineered Fill SILTY SAND with GRAVEL (SM): dense, brown, moist, 20-20% gravel, <5% cobbles, subangular to subrounded trace asphalt fragments, low plasticity fines. TD Boring, encounter refusal at 5 Inches bgs. No groundwater encountered. NOTES: 1. Terms and symbols defined on Plate A-1.							

BORING DEPTH: 0.5 ft
BACKFILL: Cuttings
DEPTH TO WATER: Not Encountered
FIELDWORK DATE: April 30, 2019
DRILLING METHOD: 6-in. dia. Hand Auger

HAMMER TYPE: N/A
RIG TYPE: N/A
DRILLED BY: MTA, J. Sheddon
LOGGED BY: J. Wilson
CHECKED BY: D. Marlow

LOG OF BORING NO. B-5
CCC Fortuna
Fortuna, California

PLATE A-7

DEPTH, ft	MATERIAL SYMBOL	SAMPLER TYPE	BLOW COUNT OR PRESSURE, psi	N VALUE OR RQD%	RECOVERY	LOCATION:	DRY UNIT WEIGHT, pcf	WATER CONTENT, %	% PASSING #200 SIEVE	LIQUID LIMIT, %	PLASTICITY INDEX	UNDRAINED SHEAR STRENGTH, S_u ksf	OTHER TESTS
1						6 Inches of grass and organic matter. Possible Engineered Fill SILT with CLAY and GRAVEL (CL-ML): soft to medium stiff, light brown to brown, moist, 15-20% gravel, <5% cobbles, subangular to subrounded fragments, low plasticity fines, trace sand and asphalt. TD Boring, encounter refusal at 1.2 feet bgs. No groundwater encountered. NOTES: 1. Terms and symbols defined on Plate A-1.							

BORING DEPTH: 1.2 ft
BACKFILL: Cuttings
DEPTH TO WATER: Not Encountered
FIELDWORK DATE: April 30, 2019
DRILLING METHOD: 6-in. dia. Hand Auger

HAMMER TYPE: N/A
RIG TYPE: N/A
DRILLED BY: MTA, J. Sheddon
LOGGED BY: J. Wilson
CHECKED BY: D. Marlow

DEPTH, ft	MATERIAL SYMBOL	SAMPLER TYPE	BLOW COUNT OR PRESSURE, psi	N VALUE OR RQD% RECOVERY	LOCATION:	MATERIAL DESCRIPTION	DRY UNIT WEIGHT, pcf	WATER CONTENT, %	% PASSING #200 SIEVE	LIQUID LIMIT, %	PLASTICITY INDEX	UNDRAINED SHEAR STRENGTH, S_u ksf	OTHER TESTS
						ALLUVIUM (Qal) Lean CLAY with SAND (CL): medium stiff, brown, moist, 5-15% subrounded gravel <2"			53	25	8		
5		S-1	5	8		ALLUVIUM (Qal) CLAYEY SAND (SC):			44				
10		S-2A S-2B	18 32	65		TERRACE DEPOSITS (Qt) Well-graded GRAVEL (GW): very dense, brown, moist, trace sand and clay, gravel subrounded and <0.5"			5				
15		S-3	7 10 12	22		TERRACE DEPOSITS (Qt) Poorly-graded GRAVEL with SILT and SAND (GP): medium dense, as above.			3				
20		S-4	8 8 14	22		medium dense, as above							
25		S-5	31 28 38	66		very dense, as above, increased sand content			4				

Continued

BORING DEPTH: 76.5 ft
BACKFILL: Grout
DEPTH TO WATER: 29.7 ft Not Encountered
FIELDWORK DATES: June 10, 2019 to June 12, 2019
DRILLING METHOD: 4.25-in. dia. Rotary Wash

HAMMER TYPE: Rope and Cat Head
RIG TYPE: Truck Mounted Mobile B-59
DRILLED BY: DCD, T. Oglivie
LOGGED BY: A. Leven
CHECKED BY: D. Marlow

LOG OF BORING NO. B-7
CCC Fortuna
Fortuna, California

PLATE A-9

DEPTH, ft	MATERIAL SYMBOL	SAMPLER TYPE	BLOW COUNT OR PRESSURE, psi	N VALUE OR RQD%	RECOVERY	LOCATION:	MATERIAL DESCRIPTION	DRY UNIT WEIGHT, pcf	WATER CONTENT, %	% PASSING #200 SIEVE	LIQUID LIMIT, %	PLASTICITY INDEX	UNDRAINED SHEAR STRENGTH, S _u ksf	OTHER TESTS
35		S-6	13 12 9	21	10.5 18"		medium dense, as above, coarse gravel							
35		S-7	12 21 13	34	7 18"		dense, wet, as above, increased gravel content, gravel <1.5".							
40		S-8	13 13 7	19	10.5 18"		TERRACE DEPOSITS (Qt) Poorly-graded SAND with GRAVEL (SP): medium dense, brown, wet, fine to coarse sand, gravel <2.5"			1				
45		S-9	28 23 19	42	12 18"		medium dense			7				
50		S-10	6 12 11	23	3 18"		TERRACE DEPOSITS (Qt) Well-graded GRAVEL with SILT and SAND (GW): medium dense, brown, wet, gravel <2.5", few fines							
55		S-11	8 11 28	39	10 18"					5				
60		S-12	8 25 34	59	6.5 18"		dense, iron staining present on gravel grains			6				

Continued

PLATE A-10



DEPTH, ft	MATERIAL SYMBOL	SAMPLER TYPE	BLOW COUNT OR PRESSURE, psi	N VALUE OR RQD% RECOVERY	LOCATION:	MATERIAL DESCRIPTION	DRY UNIT WEIGHT, pcf	WATER CONTENT, %	% PASSING #200 SIEVE	LIQUID LIMIT, %	PLASTICITY INDEX	UNDRAINED SHEAR STRENGTH, S_u ksf	OTHER TESTS
65		S-13A S-13B	12 27 50	77 3.5 18"		TERRACE DEPOSITS (Qt) Poorly graded SAND with SILT and GRAVEL (SP): very dense, brown, wet, subangular to subrounded gravel <6.5"			7				
75		S-14	12 23 50	73 7.5 18"		TERRACE DEPOSITS (Qt) Well-graded SAND with CLAY and GRAVEL (SW-SC): very dense, brown, wet, subangular to subrounded gravel <0.5" TD Boring at 76.5 feet bgs. 6/12/2019 NOTES: 1. Terms and symbols defined on Plate A-1.			7				



DEPTH, ft	MATERIAL SYMBOL	SAMPLER TYPE	BLOW COUNT OR PRESSURE, psi	N VALUE OR RQD%	RECOVERY	LOCATION: MATERIAL DESCRIPTION	DRY UNIT WEIGHT, pcf	WATER CONTENT, %	% PASSING #200 SIEVE	LIQUID LIMIT, %	PLASTICITY INDEX	UNDRAINED SHEAR STRENGTH, S_u ksf	OTHER TESTS
						ALLUVIUM (Qal) Well-graded SAND with SILT (SW):							
						Infiltration Test location. TD boring at 28.5 inches bgs. NOTES: 1. Terms and symbols defined on Plate A-1.							

BORING DEPTH: 2.4 ft
BACKFILL: Bentonite Chips and Grout
DEPTH TO WATER: Not Encountered
FIELDWORK DATE: April 30, 2019
DRILLING METHOD: 13-in. dia. Flight Auger

HAMMER TYPE: Automatic Trip
RIG TYPE: Truck Mounted CME 45
DRILLED BY: MTA, J. Sheddon
LOGGED BY: J. Wilson
CHECKED BY: D. Marlow



DEPTH, ft	MATERIAL SYMBOL	SAMPLER TYPE	BLOW COUNT OR PRESSURE, psi	N VALUE OR RQD%	RECOVERY	LOCATION: MATERIAL DESCRIPTION	DRY UNIT WEIGHT, pcf	WATER CONTENT, %	% PASSING #200 SIEVE	LIQUID LIMIT, %	PLASTICITY INDEX	UNDRAINED SHEAR STRENGTH, S_u ksf	OTHER TESTS
						ALLUVIUM (Qal) Lean CLAY with GRAVEL (CL):							
						Infiltration Test location. TD Boring at 34.5 inches bgs. NOTES: 1. Terms and symbols defined on Plate A-1.							

BORING DEPTH: 2.9 ft
BACKFILL: Bentonite Chips and Grout
DEPTH TO WATER: Not Encountered
FIELDWORK DATE: April 30, 2019
DRILLING METHOD: 13-in. dia. Flight Auger

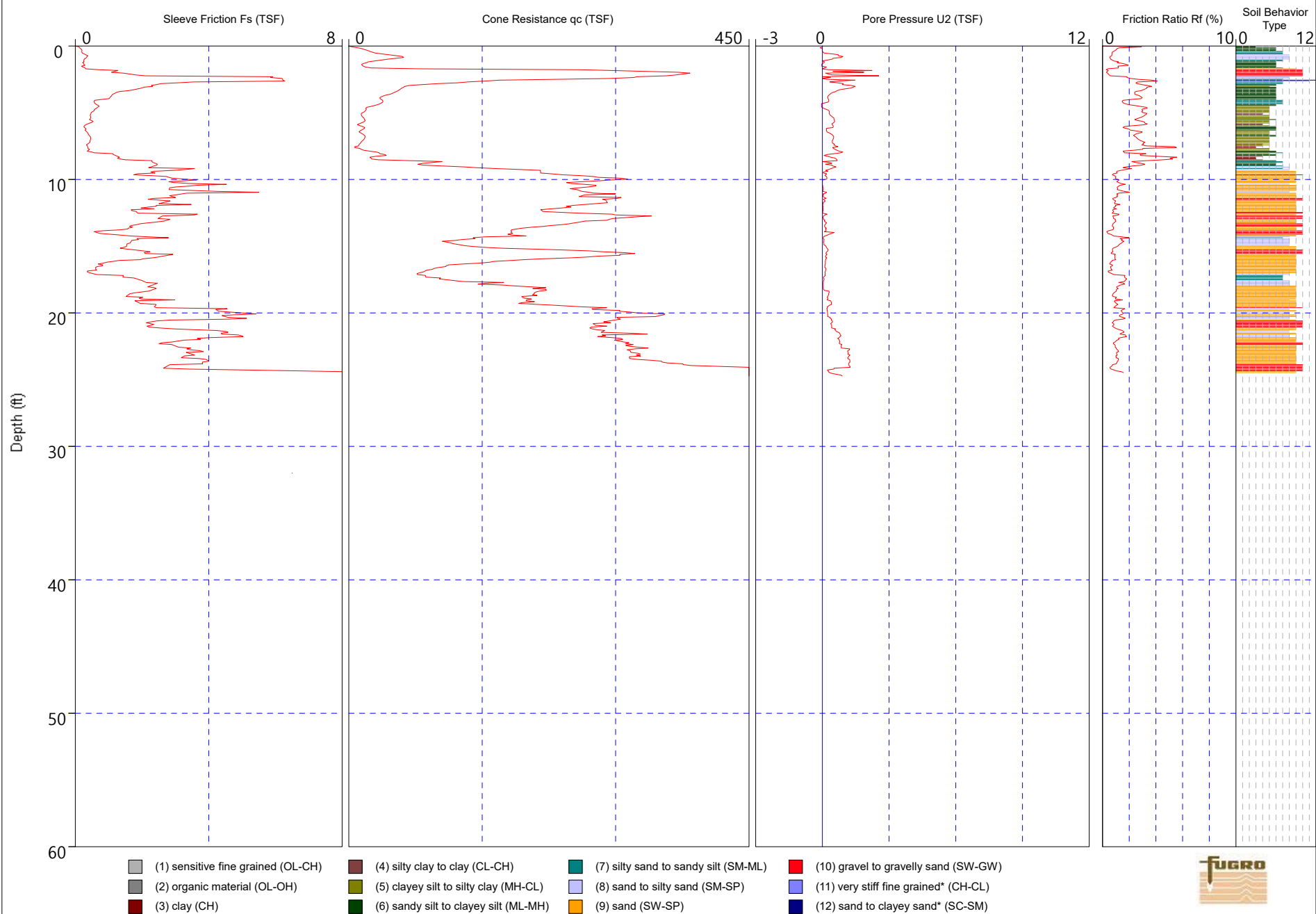
HAMMER TYPE: Automatic Trip
RIG TYPE: Truck Mounted CME 45
DRILLED BY: MTA, J. Sheddon
LOGGED BY: J. Wilson
CHECKED BY: D. Marlow

HAMMER TYPE: Automatic Trip
RIG TYPE: Truck Mounted CME 45
DRILLED BY: MTA, J. Sheddon
LOGGED BY: J. Wilson
CHECKED BY: D. Marlow

Job Number: 04.72190034
Operator: D. Garza
Location: Fortuna, CA

CPT Number: CPT-01
Date: 08-May-2019
Elevation: 0.00

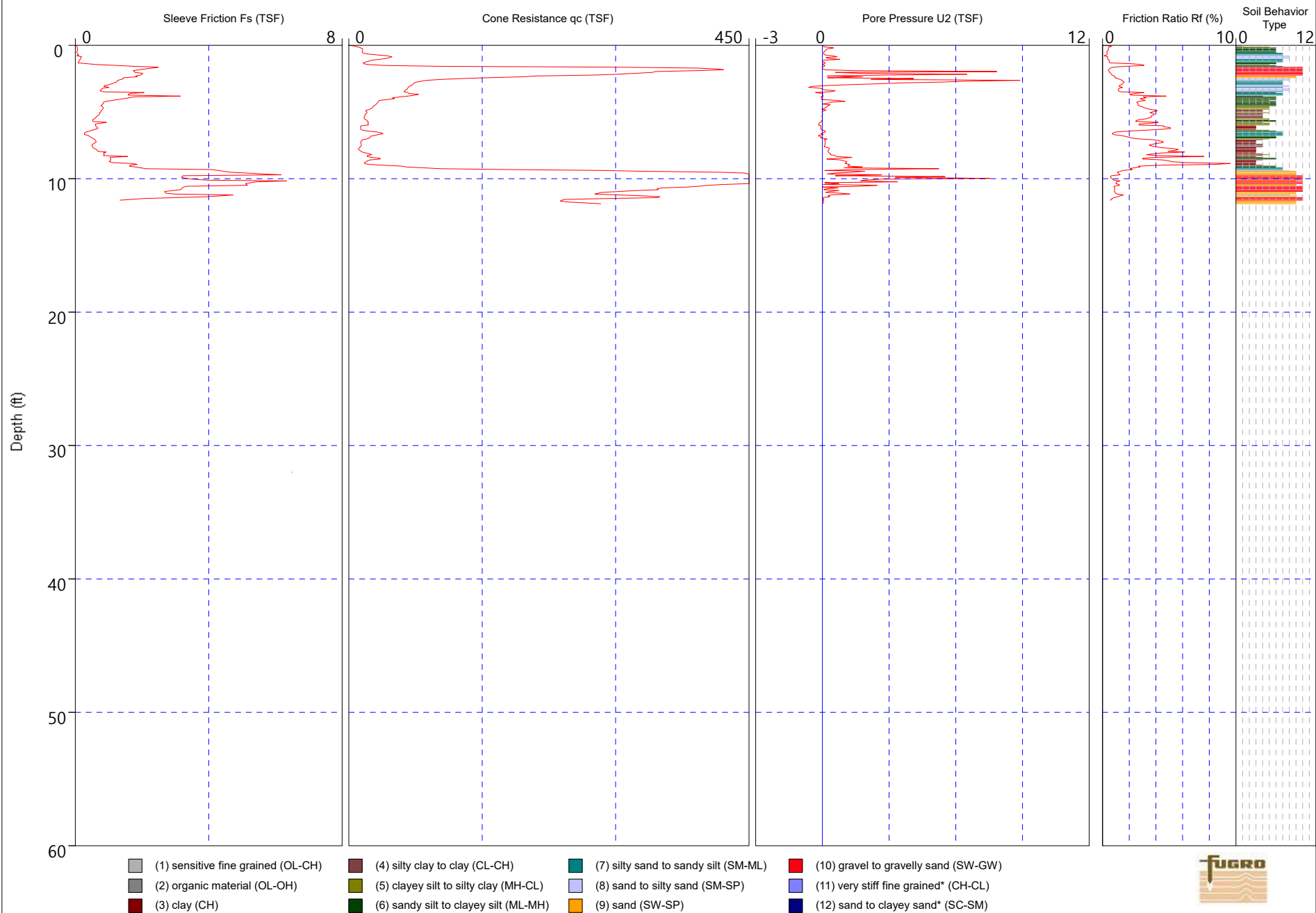
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Job Number: 04.72190034
Operator: D. Garza
Location: Fortuna, CA

CPT Number: CPT-01A
Date: 08-May-2019
Elevation: 0.00

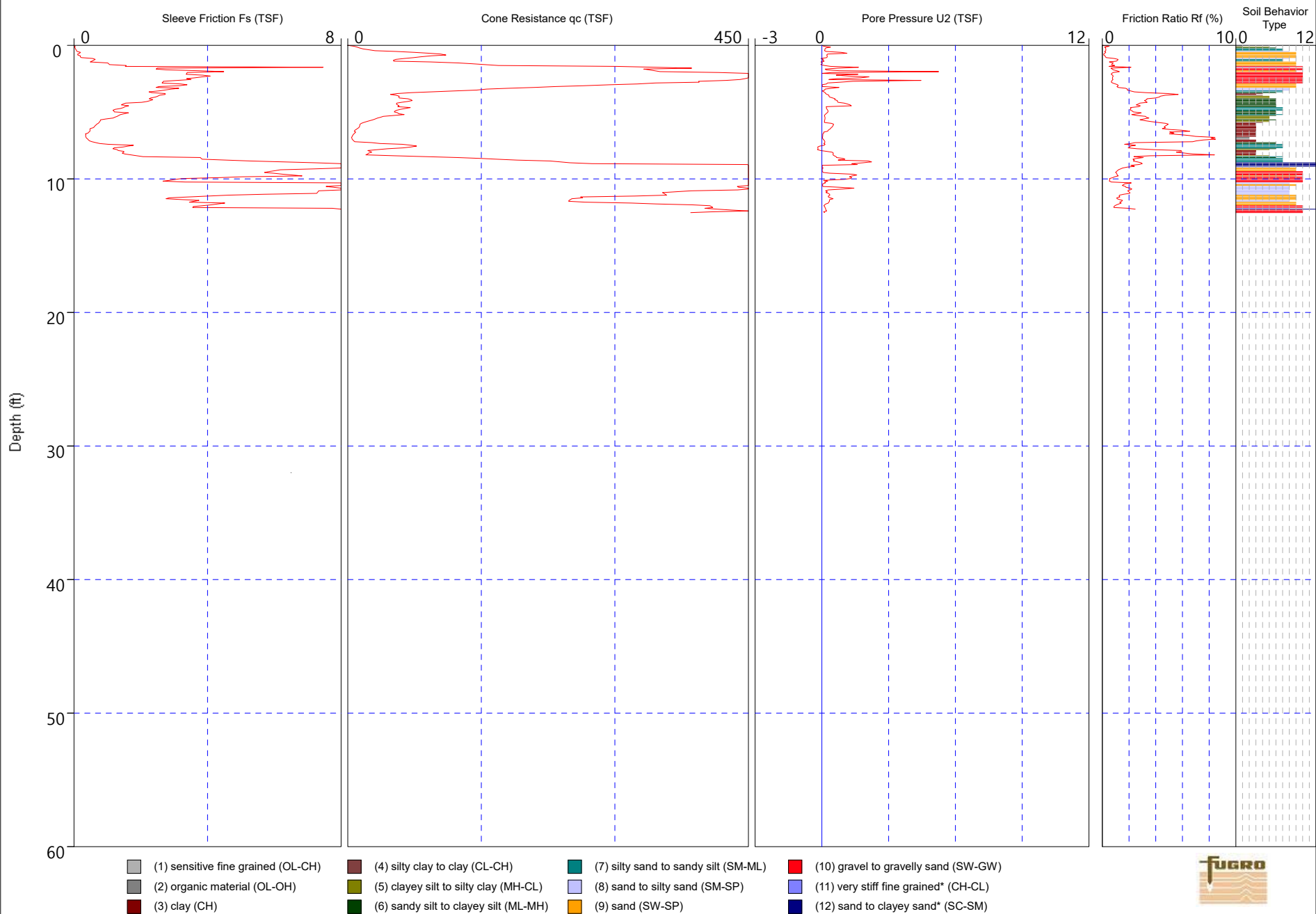
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Cone Number: CP15-CF75PB7SN2-P1E1 2598



Job Number: 04.72190034
Operator: D. Garza
Location: Fortuna, CA

CPT Number: CPT-01B
Date: 08-May-2019
Elevation: 0.00

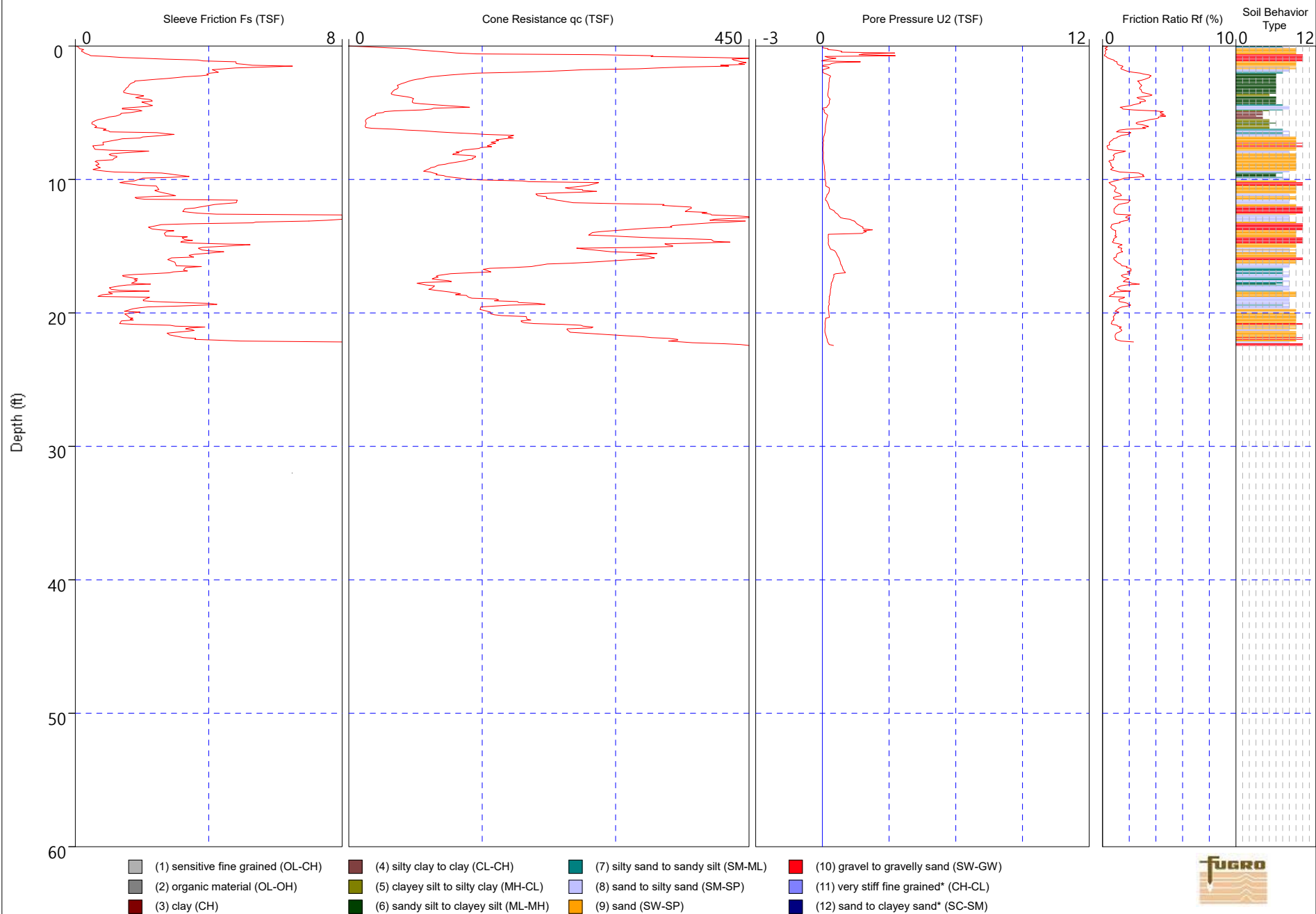
Coordinates: 40.5808793 -124.1538455
Cone Number: CP15-CF75PB7SN2-P1E1 2598



Job Number: 04.72190034
Operator: D. Garza
Location: Fortuna, CA

CPT Number: CPT-02
Date: 08-May-2019
Elevation: 0.00

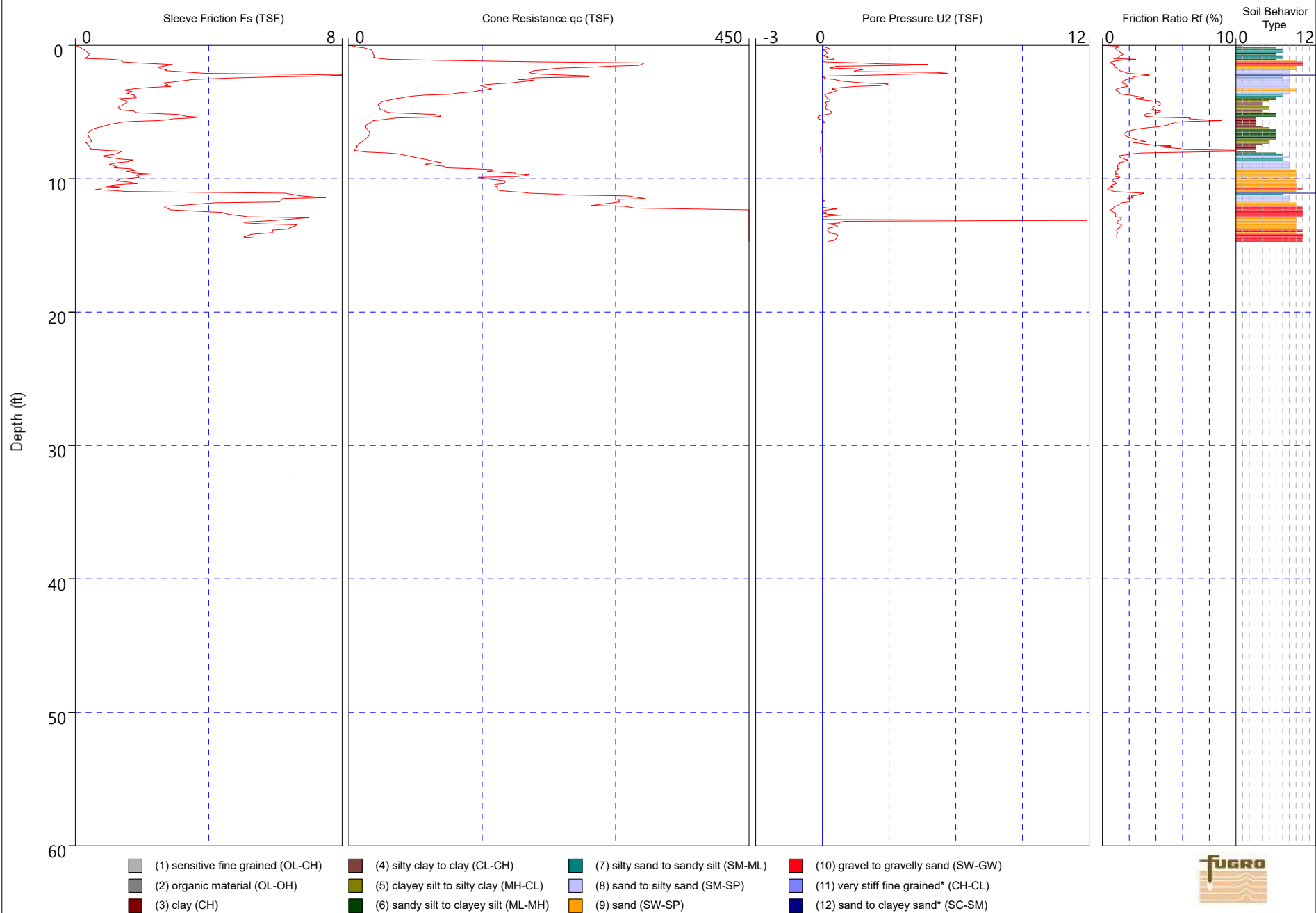
Coordinates: 40.5809625 -124.1534941
Cone Number: CP15-CF75PB7SN2-P1E1 2598



Job Number: 04.72190034
Operator: D. Garza
Location: Fortuna, CA

CPT Number: CPT-02A
Date: 08-May-2019
Elevation: 0.00

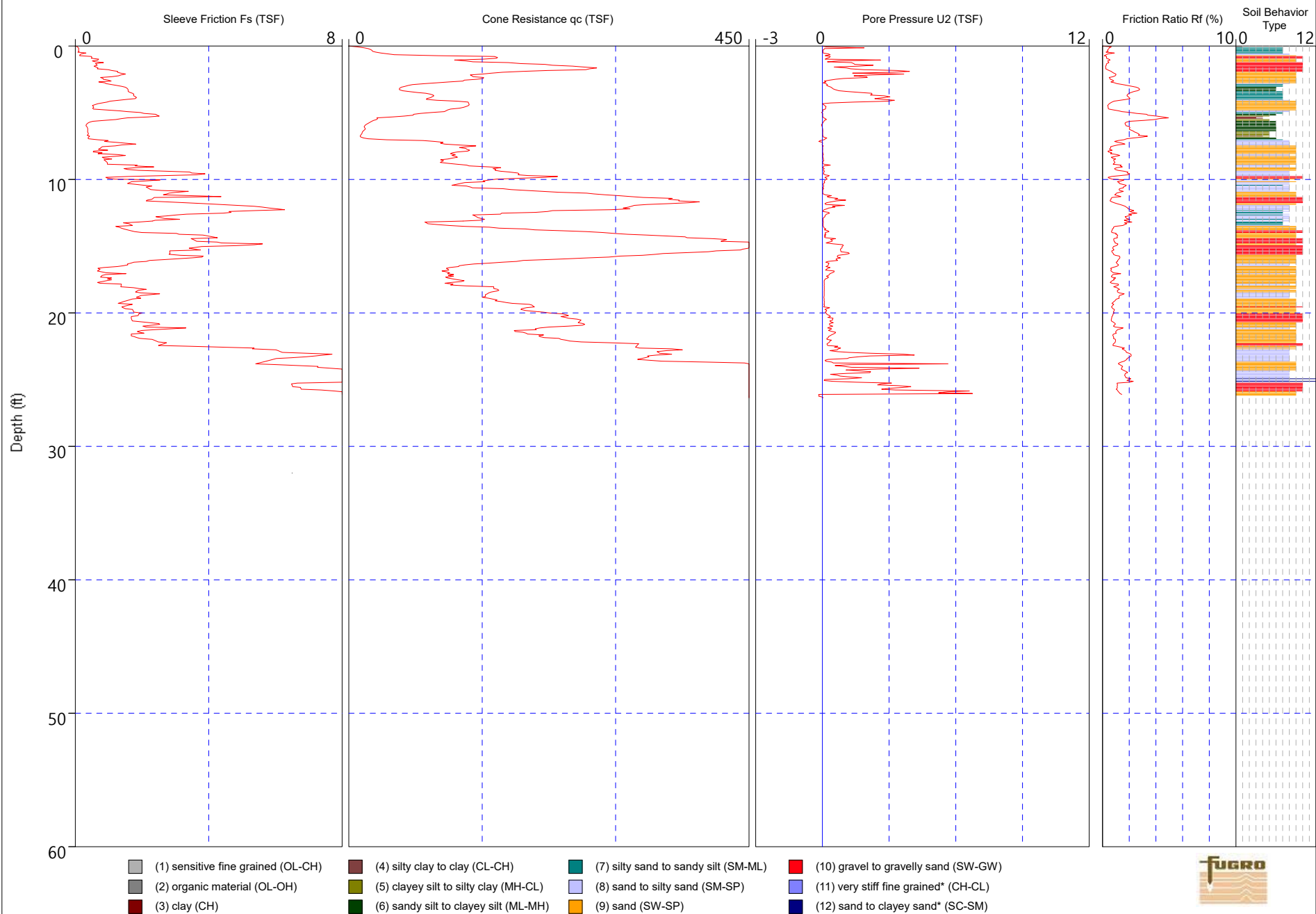
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Cone Number: CP15-CF75PB7SN2-P1E1 2598



Job Number: 04.72190034
Operator: D. Garza
Location: Fortuna, CA

CPT Number: CPT-02B
Date: 08-May-2019
Elevation: 0.00

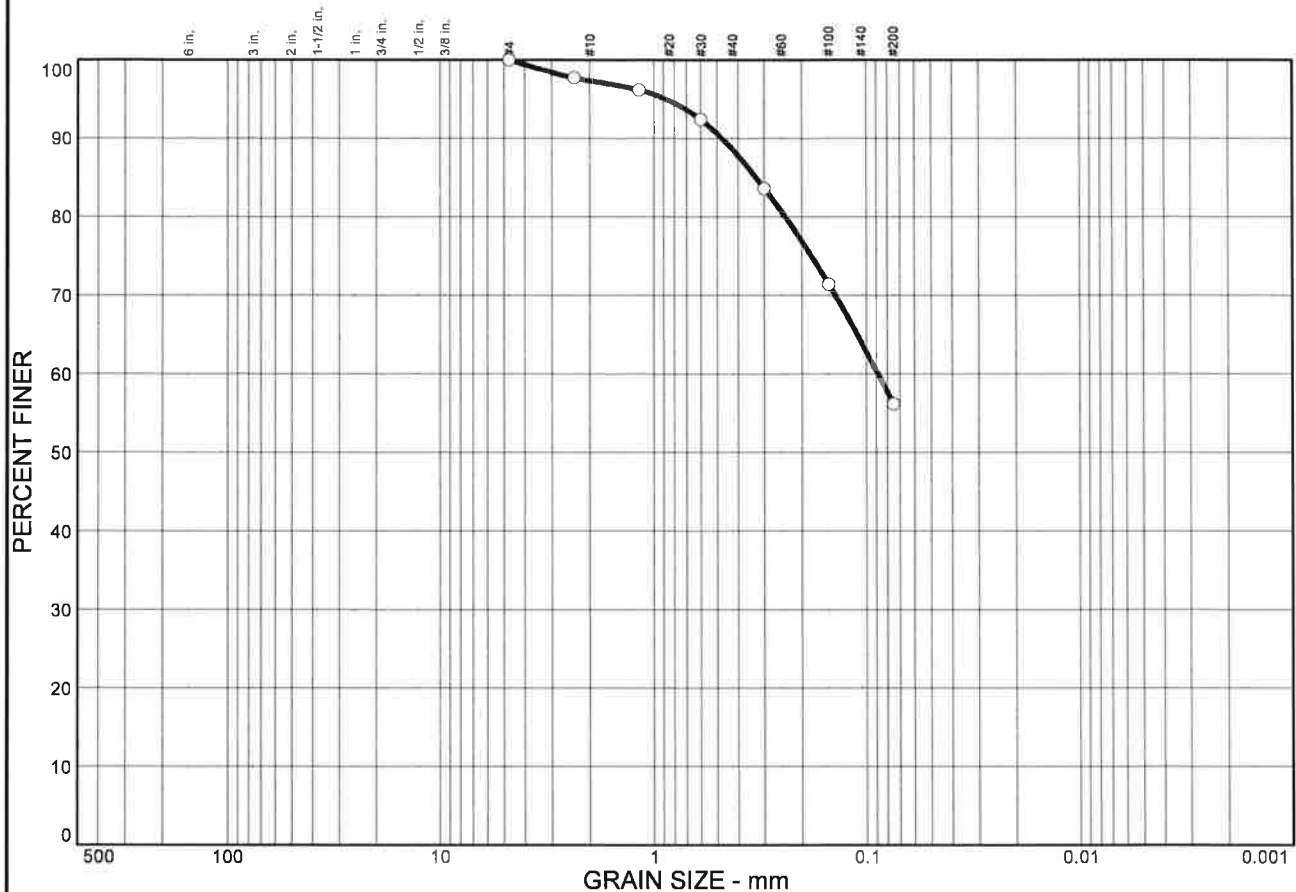
Coordinates: 40.5809269 -124.1535606
Cone Number: CP15-CF75PB7SN2-P1E1 2598





APPENDIX B LABORATORY TESTING PROGRAM

Particle Size Distribution Report



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	0.0	43.8	56.2	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#4	100.0		
#8	97.7		
#16	96.1		
#30	92.4		
#50	83.6		
#100	71.4		
#200	56.2		

Material Description

Sandy lean clay

Atterberg Limits

PL= 17 LL= 26 PI= 9

Coefficients

D₈₅= 0.329 D₆₀= 0.0886 D₅₀=
D₃₀= D₁₅= D₁₀=
C_u= C_c=

Classification

USCS= CL AASHTO=

Remarks

* (no specification provided)

Sample No.: B-1 (B-01)

Location:

Source of Sample: DGS CCC Multipurpose Bldg., Fortuna

Date: 5/16/19

Elev./Depth: 0-5'

Moore Twining Associates, Inc.

Fresno, CA

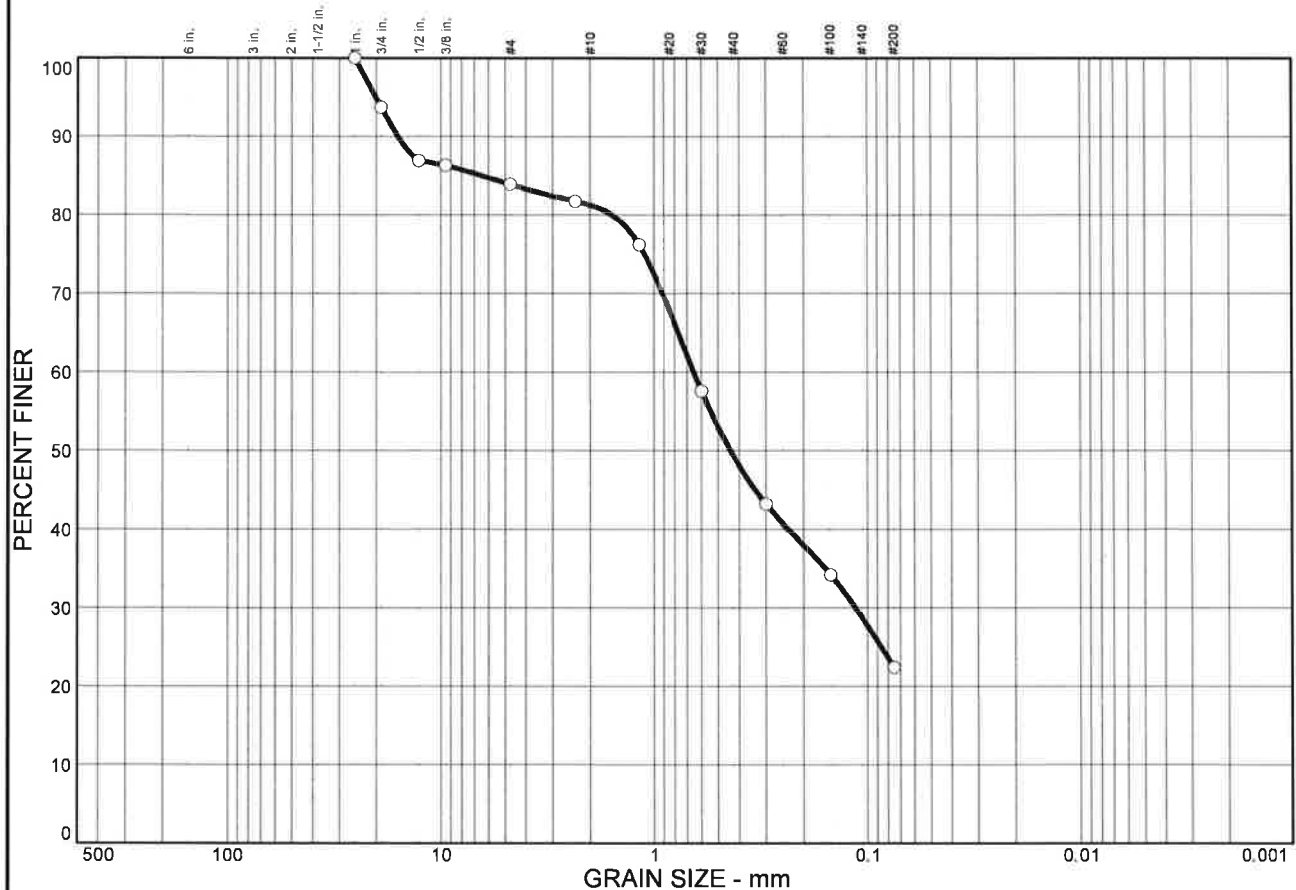
Client: Fugro USA Land

Project: DGS CCC Multi purpose Bldg., Fortuna, 4.72190034

Project No: Pending

PLATE B-1

Particle Size Distribution Report



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	16.1	61.5		22.4

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1 in.	100.0		
3/4 in.	93.7		
1/2 in.	86.9		
3/8 in.	86.3		
#4	83.9		
#8	81.7		
#16	76.2		
#30	57.6		
#50	43.2		
#100	34.2		
#200	22.4		

Material Description

Atterberg Limits
 PL= LL= PI=

Coefficients
 D₈₅= 6.44 D₆₀= 0.652 D₅₀= 0.439
 D₃₀= 0.115 D₁₅= D₁₀=
 C_u= C_c=

Classification
 USCS= AASHTO=

Remarks

* (no specification provided)

Sample No.: B-1 (S-01B)
Location:

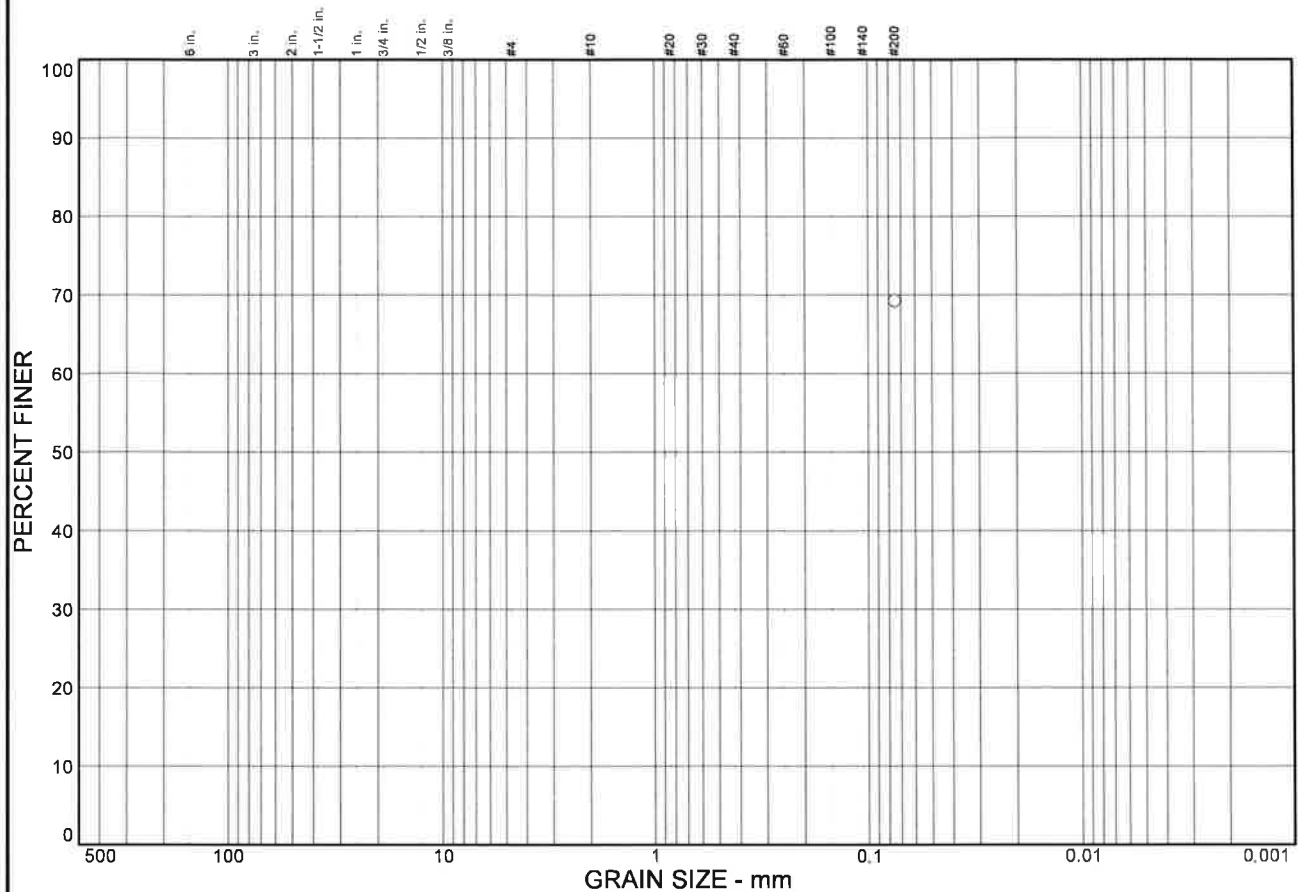
Source of Sample: DGS CCC Multipurpose Bldg., Fortuna **Date:** 5/16/19
Elev./Depth: 5.5-6'

Moore Twining Associates, Inc.
Fresno, CA

Client: Fugro USA Land
Project: DGS CCC Multi purpose Bldg., Fortuna, 4.72190034
Project No: Pending

PLATE B-2

Particle Size Distribution Report



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
			69.3	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	69.3		

* (no specification provided)

Material Description

PL= 20
 Atterberg Limits
 LL= 27
 PI= 7

Coefficients

D₈₅=
 D₆₀=
 D₅₀=
 D₃₀=
 D₁₅=
 D₁₀=
 C_u=
 C_c=

Classification

USCS=
 AASHTO=

Remarks

Sample No.: B-1 (S-02)
Location:

Source of Sample: DGS CCC Multipurpose Bldg., Fortuna

Date: 5/16/19
Elev./Depth: 10-11.5'

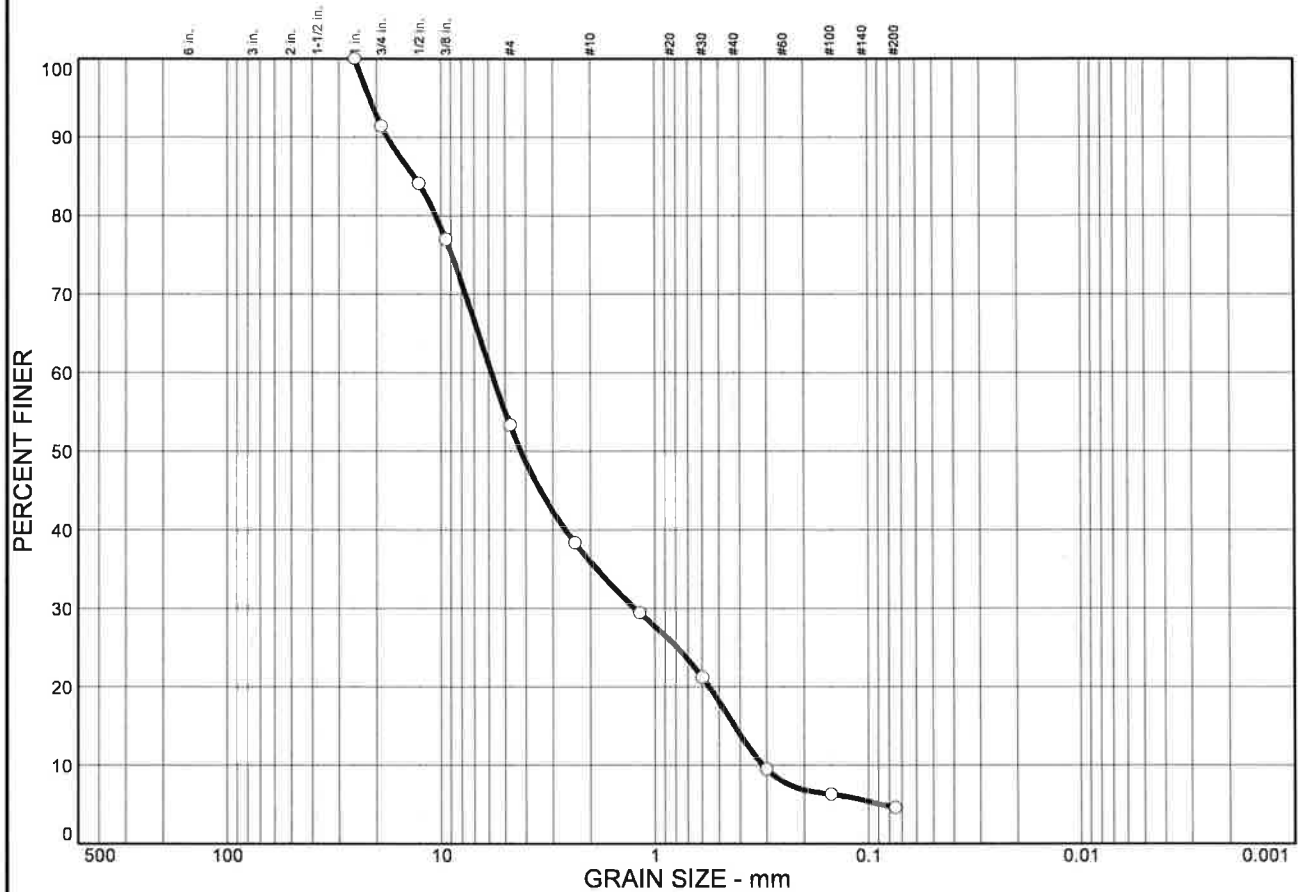
Moore Twining Associates, Inc.
Fresno, CA

Client: Fugro USA Land
Project: DGS CCC Multi purpose Bldg., Fortuna, 4.72190034

Project No: Pending

PLATE B-3

Particle Size Distribution Report



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	46.7	48.7		4.6

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1 in.	100.0		
3/4 in.	91.4		
1/2 in.	84.0		
3/8 in.	77.0		
#4	53.3		
#8	38.3		
#16	29.4		
#30	21.2		
#50	9.5		
#100	6.3		
#200	4.6		

* (no specification provided)

Material Description

Atterberg Limits
 PL= LL= PI=

Coefficients
 D₈₅= 13.4 D₆₀= 5.81 D₅₀= 4.23
 D₃₀= 1.25 D₁₅= 0.425 D₁₀= 0.312
 C_u= 18.61 C_c= 0.86

Classification
 USCS= AASHTO=

Remarks

Sample No.: B-1 (S-04)
Location:

Source of Sample: DGS CCC Multipurpose Bldg., Fortuna

Date: 5/16/19

Elev./Depth: 16.5-17.5'

Moore Twining Associates, Inc.

Fresno, CA

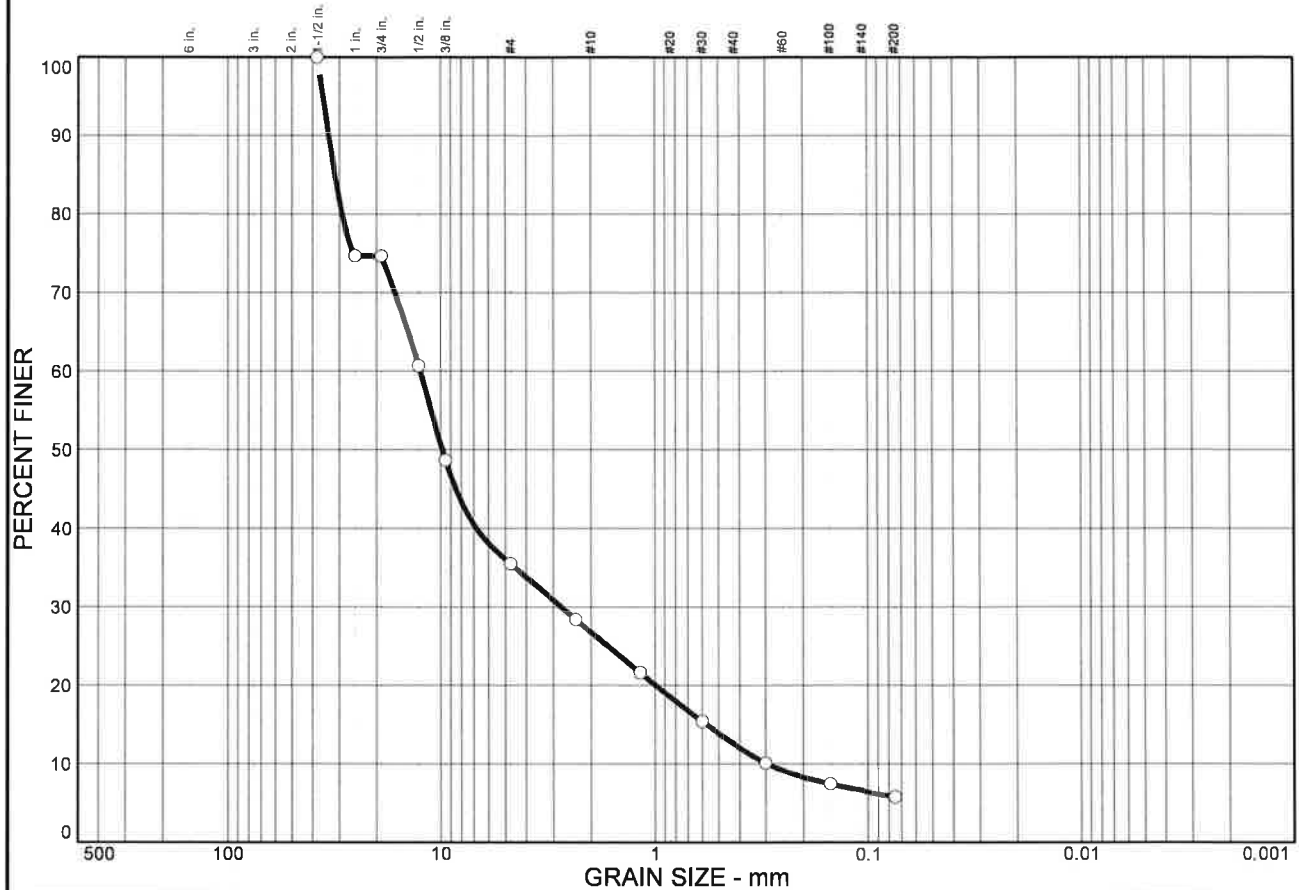
Client: Fugro USA Land

Project: DGS CCC Multi purpose Bldg., Fortuna, 4.72190034

Project No: Pending

PLATE B-4

Particle Size Distribution Report



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	64.5	29.7	5.8	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1-1/2 in.	100.0		
1 in.	74.6		
3/4 in.	74.6		
1/2 in.	60.7		
3/8 in.	48.7		
#4	35.5		
#8	28.4		
#16	21.6		
#30	15.4		
#50	10.1		
#100	7.4		
#200	5.8		

Material Description

Atterberg Limits
 PL= LL= PI=

Coefficients
 D₈₅= 31.5 D₆₀= 12.5 D₅₀= 9.86
 D₃₀= 2.77 D₁₅= 0.573 D₁₀= 0.295
 C_u= 42.35 C_c= 2.08

Classification
 USCS= AASHTO=

Remarks

* (no specification provided)

Sample No.: B-1 (S-06)
Location:

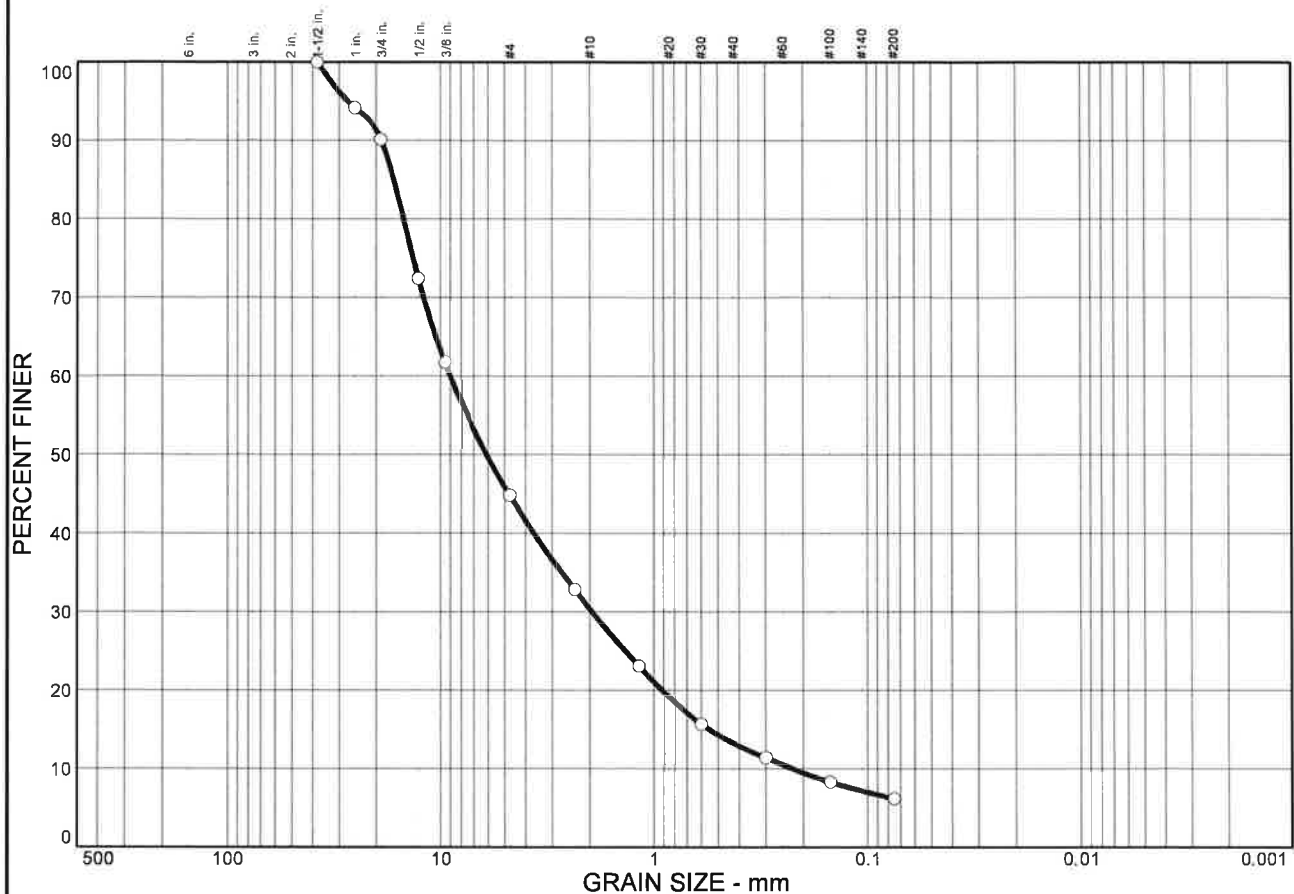
Source of Sample: DGS CCC Multipurpose Bldg., Fortuna **Date:** 5/16/19
Elev./Depth: 25.0-25.8'

Moore Twining Associates, Inc.
Fresno, CA

Client: Fugro USA Land
Project: DGS CCC Multi purpose Bldg., Fortuna, 4.72190034
Project No: Pending

PLATE B-5

Particle Size Distribution Report



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	55.2	38.6	6.2	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1-1/2 in.	100.0		
1 in.	94.1		
3/4 in.	90.1		
1/2 in.	72.4		
3/8 in.	61.7		
#4	44.8		
#8	32.8		
#16	23.1		
#30	15.7		
#50	11.4		
#100	8.3		
#200	6.2		

Material Description

Atterberg Limits
 PL= LL= PI=

Coefficients
 D₈₅= 16.6 D₆₀= 9.01 D₅₀= 6.11
 D₃₀= 1.96 D₁₅= 0.550 D₁₀= 0.224
 C_u= 40.15 C_c= 1.89

Classification
 USCS= AASHTO=

Remarks

* (no specification provided)

Sample No.: B-1 (S-07)
Location:

Source of Sample: DGS CCC Multipurpose Bldg., Fortuna

Date: 5/16/19

Elev./Depth: 30.0-30.62'

Moore Twining Associates, Inc.

Fresno, CA

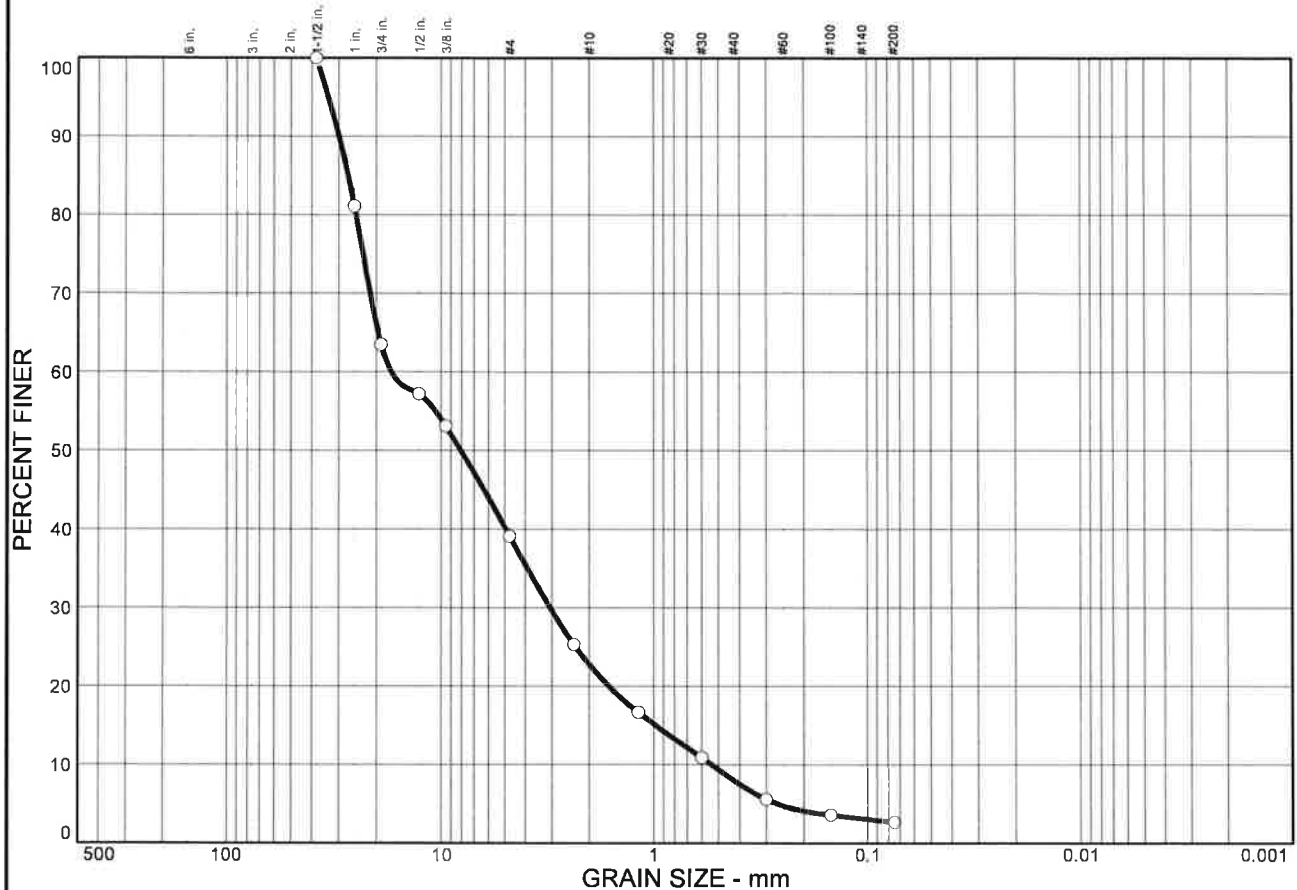
Client: Fugro USA Land

Project: DGS CCC Multi purpose Bldg., Fortuna, 4.72190034

Project No: Pending

PLATE B-6

Particle Size Distribution Report



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	60.9	36.4	2.7	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1-1/2 in.	100.0		
1 in.	81.1		
3/4 in.	63.5		
1/2 in.	57.2		
3/8 in.	53.1		
#4	39.1		
#8	25.3		
#16	16.7		
#30	10.9		
#50	5.6		
#100	3.6		
#200	2.7		

Material Description

Atterberg Limits
 PL= LL= PI=

Coefficients
 D₈₅= 27.2 D₆₀= 17.1 D₅₀= 8.07
 D₃₀= 3.06 D₁₅= 0.979 D₁₀= 0.540
 C_u= 31.65 C_c= 1.02

Classification
 USCS= AASHTO=

Remarks

* (no specification provided)

Sample No.: B-1 (S-08)

Source of Sample: DGS CCC Multipurpose Bldg., Fortuna

Date: 5/16/19

Location:

Elev./Depth: 35.0-35.8'

Moore Twining Associates, Inc.

Fresno, CA

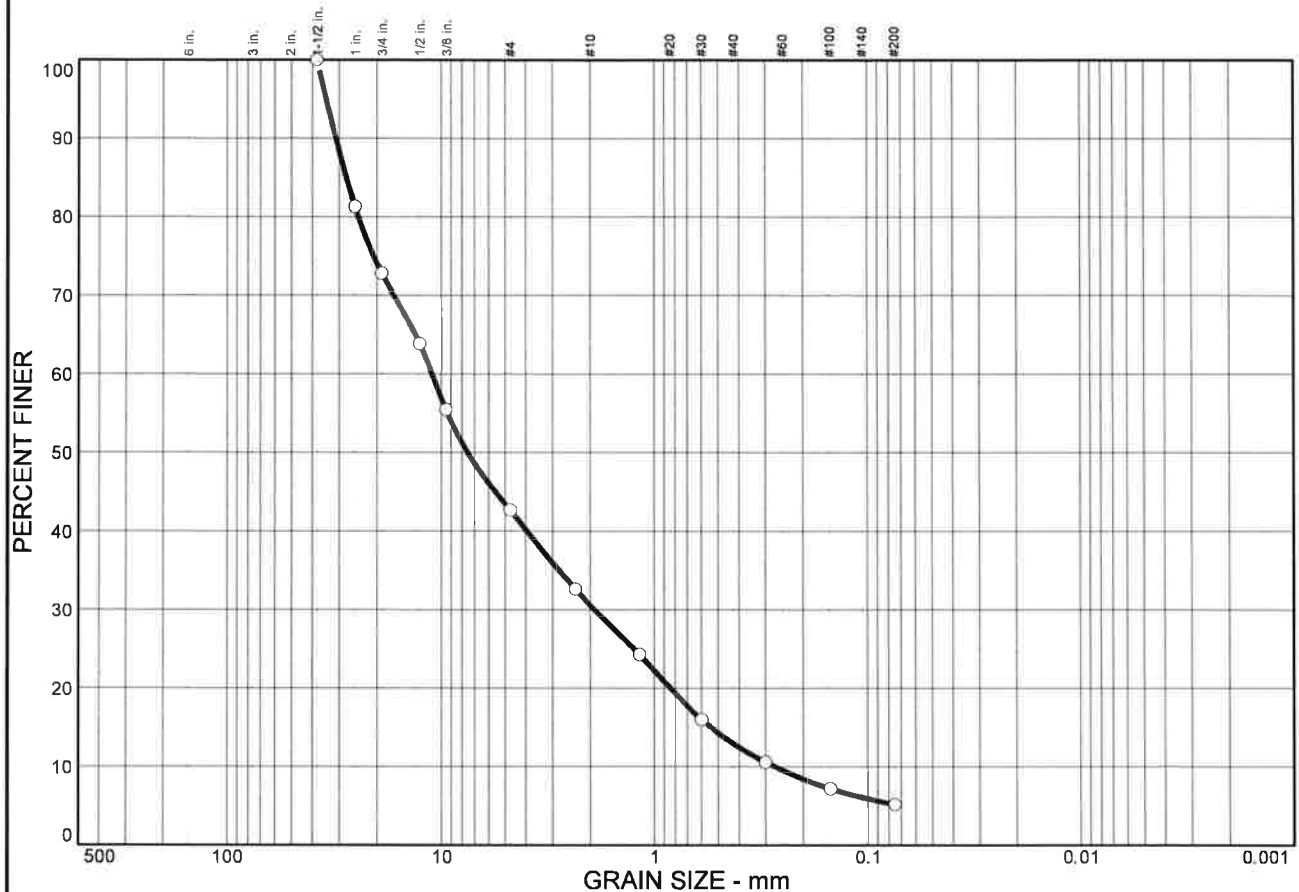
Client: Fugro USA Land

Project: DGS CCC Multi purpose Bldg., Fortuna, 4.72190034

Project No: Pending

PLATE B-7

Particle Size Distribution Report



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	57.3	37.5	5.2	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1-1/2 in.	100.0		
1 in.	81.3		
3/4 in.	72.8		
1/2 in.	63.9		
3/8 in.	55.5		
#4	42.7		
#8	32.6		
#16	24.3		
#30	16.0		
#50	10.6		
#100	7.2		
#200	5.2		

Material Description

Atterberg Limits
 PL= LL= PI=

Coefficients
 D₈₅= 27.9 D₆₀= 11.1 D₅₀= 7.52
 D₃₀= 1.92 D₁₅= 0.543 D₁₀= 0.271
 C_u= 40.99 C_c= 1.22

Classification
 USCS= AASHTO=

Remarks

* (no specification provided)

Sample No.: B-1 (S-10)
Location:

Source of Sample: DGS CCC Multipurpose Bldg., Fortuna **Date:** 5/16/19
Elev./Depth: 45.0-46.3'

Moore Twining Associates, Inc.
Fresno, CA

Client: Fugro USA Land
Project: DGS CCC Multi purpose Bldg., Fortuna, 4.72190034
Project No: Pending

PLATE B-8

The graph shows a grain size distribution curve for a soil sample. The Y-axis represents the Percent Finer, ranging from 0 to 100. The X-axis represents the Grain Size in mm, ranging from 500 to 0.075. The curve is a smooth, continuous line connecting data points that show the percentage of soil finer than a given grain size.

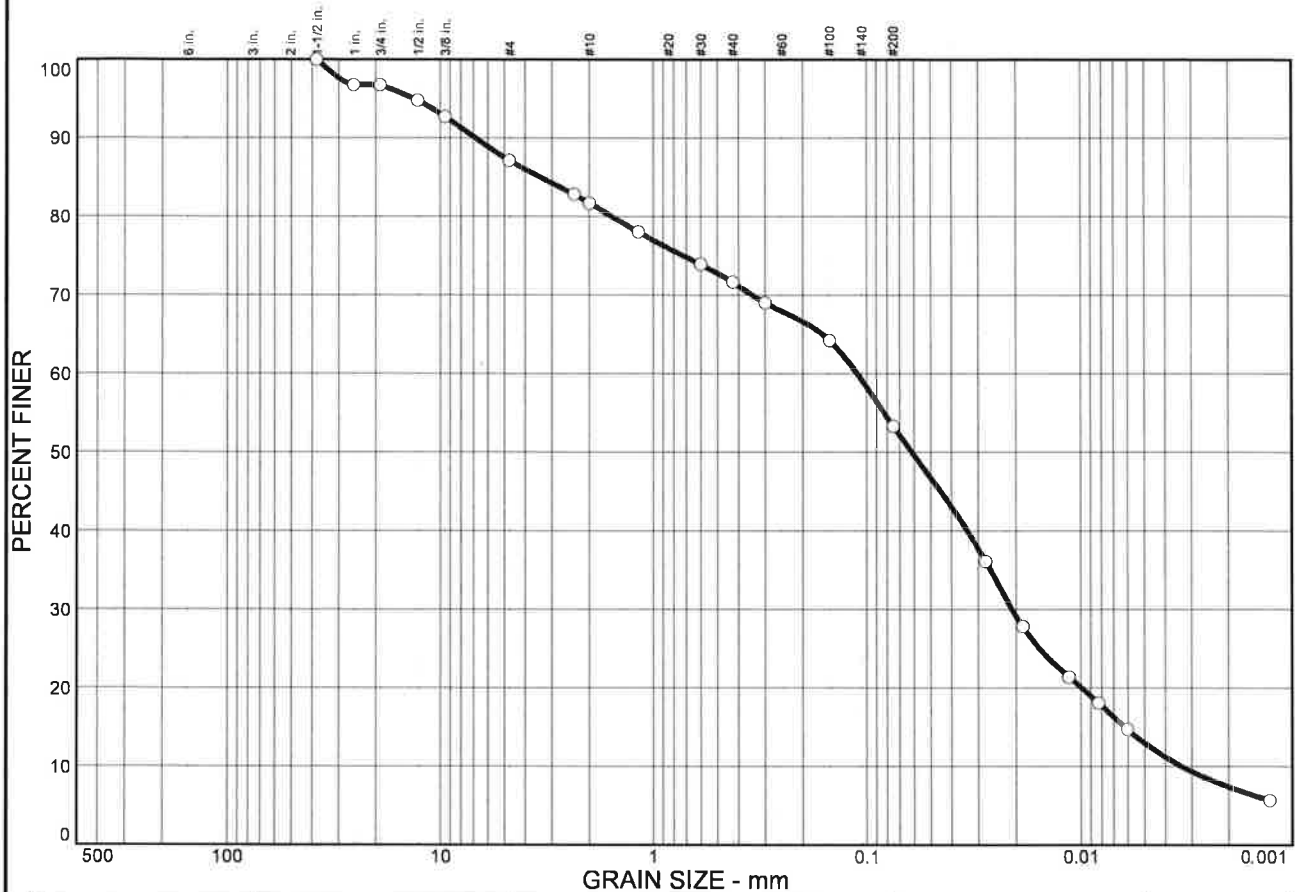
Grain Size (mm)	Percent Finer (%)
4.75	100
2.0	78
0.85	75
0.425	63
0.25	52
0.15	35
0.075	22
0.0425	15
0.025	10
0.015	6
0.0075	4
0.00425	2

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1-1/2 in.	100.0		
1 in.	78.7		
3/4 in.	75.7		
1/2 in.	63.1		
3/8 in.	52.4		
#4	35.7		
#8	22.7		
#16	15.0		
#30	10.3		
#50	7.0		
#100	4.9		
#200	3.5		

<u>Atterberg Limits</u>		
PL=	LL=	PI=
<u>Coefficients</u>		
D ₈₅ = 29.9	D ₆₀ = 11.7	D ₅₀ = 8.83
D ₃₀ = 3.56	D ₁₅ = 1.18	D ₁₀ = 0.569
C _u = 20.62	C _c = 1.90	
<u>Classification</u>		
USCS=	AASHTO=	
<u>Remarks</u>		

Client: Fugro USA Land
Project: DGS CCC Multi purpose Bldg., Fortuna, 4.72190034
Project No: Pending **PLATE**

Particle Size Distribution Report



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	12.9	33.8	40.4	12.9

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1-1/2 in.	100.0		
1 in.	96.7		
3/4 in.	96.7		
1/2 in.	94.7		
3/8 in.	92.7		
#4	87.1		
#8	82.8		
#10	81.7		
#16	78.0		
#30	73.9		
#40	71.6		
#50	69.0		
#100	64.2		
#200	53.3		

* (no specification provided)

Material Description

Sandy lean clay

Atterberg Limits

PL= 17

LL= 25

PI= 8

Coefficients

D₈₅= 3.42

D₆₀= 0.111

D₅₀= 0.0614

D₃₀= 0.0209

D₁₅= 0.0062

D₁₀= 0.0033

C_u= 33.16

C_c= 1.18

Classification

USCS= CL

AASHTO=

Remarks

Sample No.: B-7 (B-01)

Source of Sample: DGS CCC Multipurpose Bldg., Fortuna,

Date: 6/17/19

Location:

Elev./Depth: 0-5'

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Fresno, CA

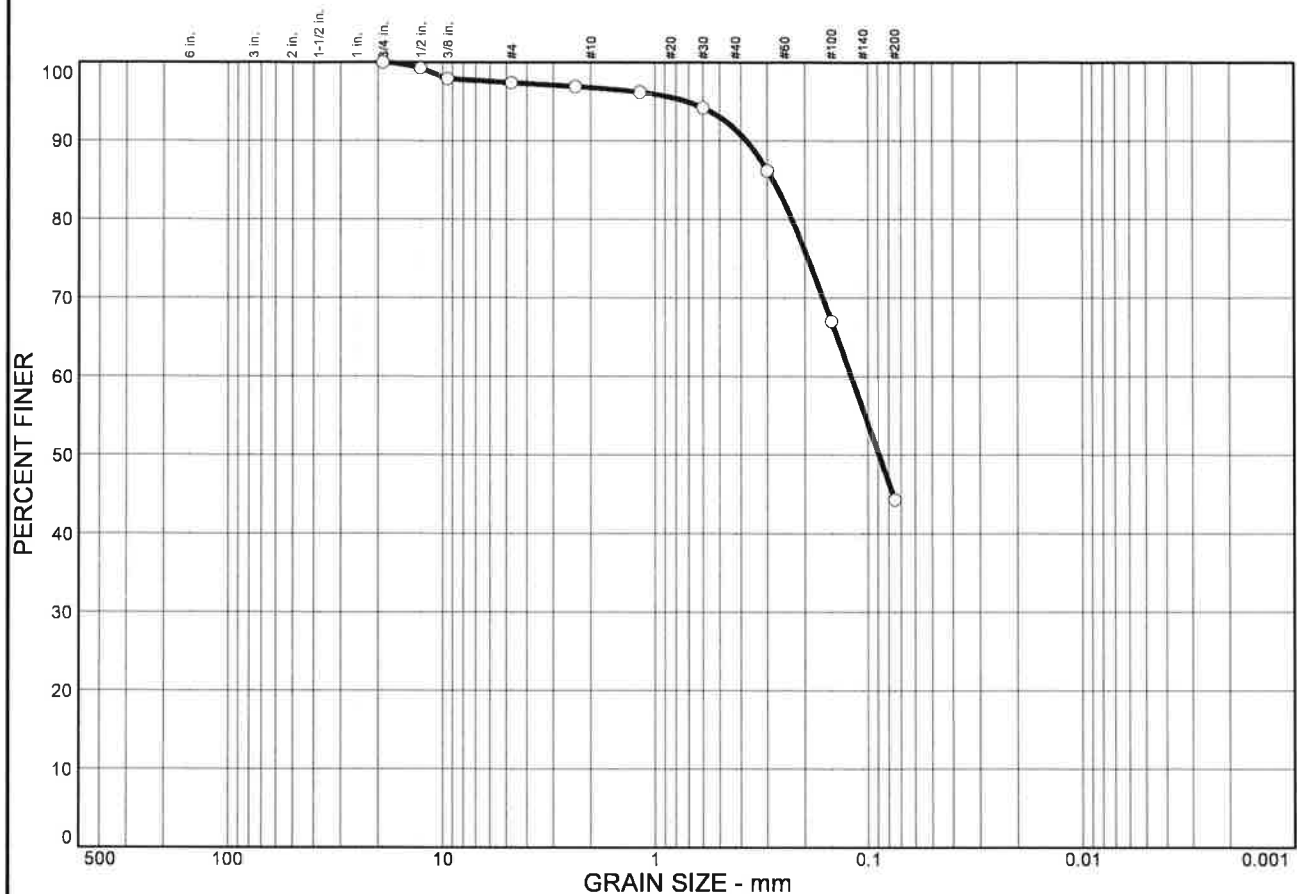
Client: Fugro USA Land

Project: DGS CCC Multi purpose Bldg., Fortuna, 4.72190034

Project No: Pending

PLATE B-10

Particle Size Distribution Report



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	2.6	53.1		44.3

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3/4 in.	100.0		
1/2 in.	99.2		
3/8 in.	97.9		
#4	97.4		
#8	96.9		
#16	96.2		
#30	94.2		
#50	86.2		
#100	67.0		
#200	44.3		

Material Description

Atterberg Limits
 PL= LL= PI=

Coefficients
 D₈₅= 0.283 D₆₀= 0.121 D₅₀= 0.0891
 D₃₀= D₁₅= D₁₀=
 C_u= C_c=

Classification
 USCS= AASHTO=

Remarks

* (no specification provided)

Sample No.: B-7 (S-01)
Location:

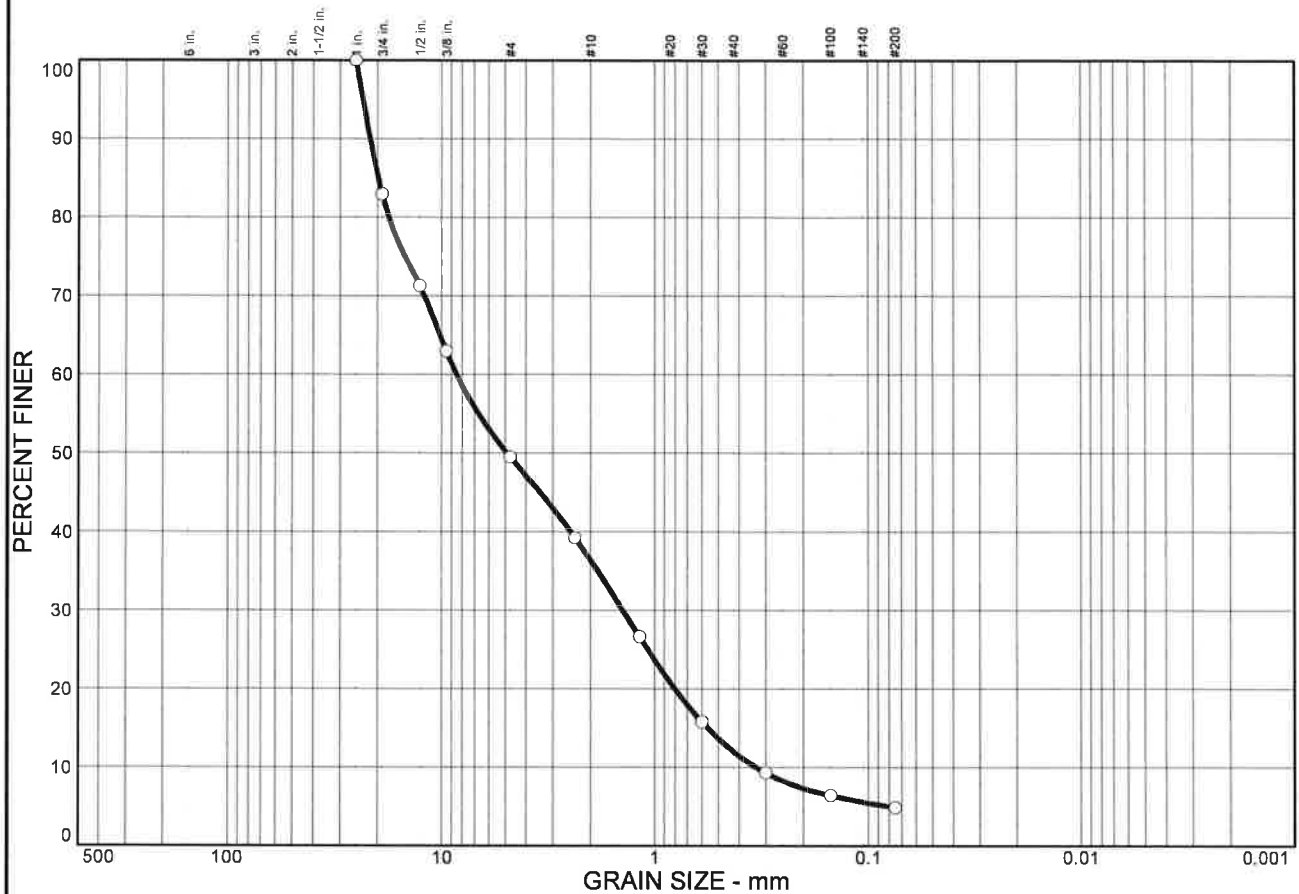
Source of Sample: DGS CCC Multipurpose Bldg., Fortuna, **Date:** 6/17/19
Elev./Depth: 4.5-6'

Moore Twining Associates, Inc.
Fresno, CA

Client: Fugro USA Land
Project: DGS CCC Multi purpose Bldg., Fortuna, 4.72190034
Project No: Pending

PLATE B-11

Particle Size Distribution Report



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	50.5	44.6		4.9

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1 in.	100.0		
3/4 in.	82.9		
1/2 in.	71.3		
3/8 in.	63.0		
#4	49.5		
#8	39.2		
#16	26.6		
#30	15.8		
#50	9.3		
#100	6.4		
#200	4.9		

* (no specification provided)

Material Description

Poorly graded gravel with sand

Atterberg Limits

PL=

LL=

PI=

Coefficients

D₈₅= 19.9

D₆₀= 8.49

D₅₀= 4.92

D₃₀= 1.42

D₁₅= 0.563

D₁₀= 0.333

C_u= 25.48

C_c= 0.71

Classification

USCS= GP

AASHTO=

Remarks

Sample No.: B-7 (S-02B & S-02A) Source of Sample: DGS CCC Multipurpose Bldg., Fortuna, Date: 6/17/19
Location: Elev./Depth: 9.5-10' & 10-

Moore Twining Associates, Inc.

Fresno, CA

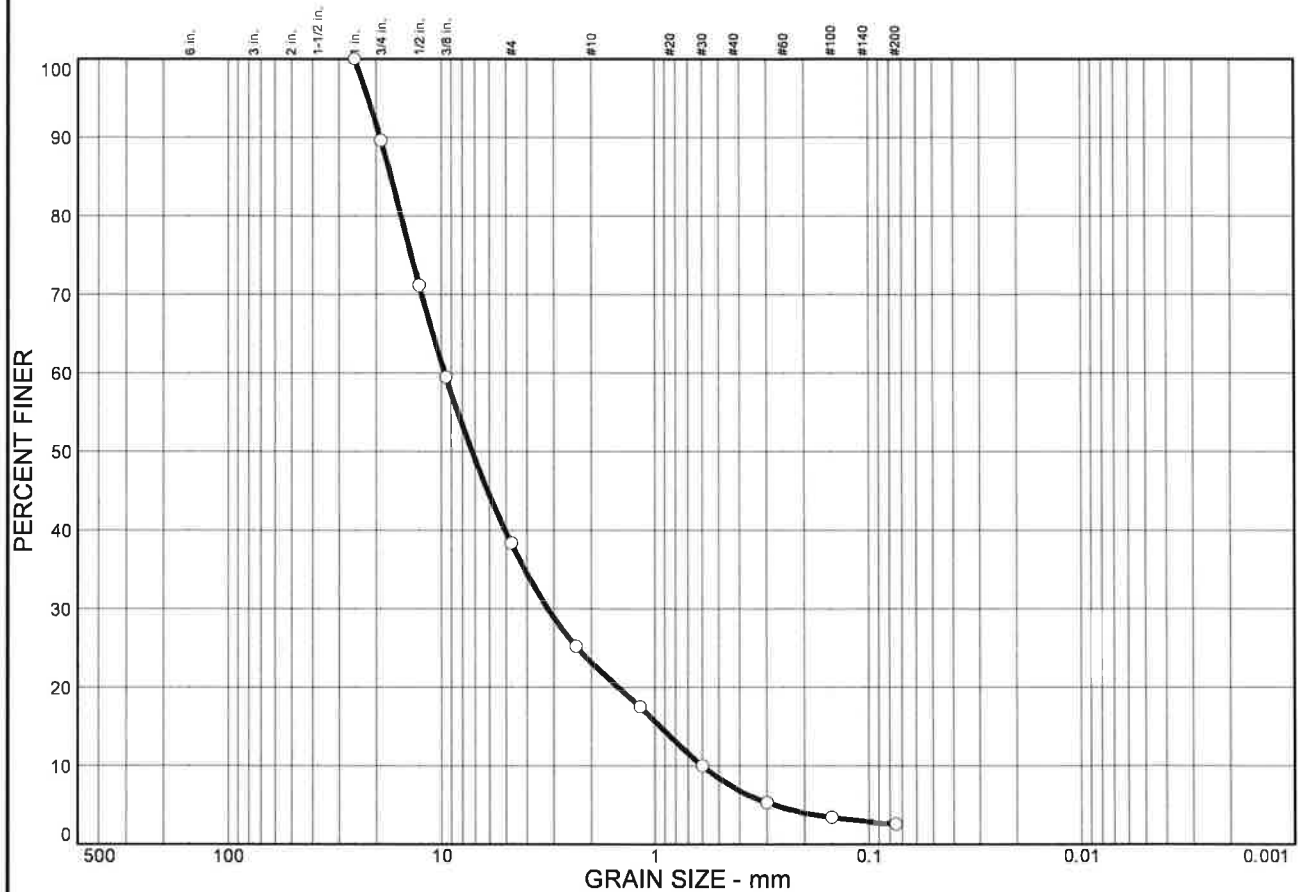
Client: Fugro USA Land

Project: DGS CCC Multi purpose Bldg., Fortuna, 4.72190034

Project No: Pending

PLATE B-12

Particle Size Distribution Report



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	61.7	35.7		2.6

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1 in.	100.0		
3/4 in.	89.6		
1/2 in.	71.1		
3/8 in.	59.5		
#4	38.3		
#8	25.2		
#16	17.5		
#30	10.0		
#50	5.3		
#100	3.4		
#200	2.6		

* (no specification provided)

Material Description

Well-graded gravel with sand

Atterberg Limits

PL=

LL=

PI=

Coefficients

D₈₅= 17.1

D₆₀= 9.65

D₅₀= 7.22

D₃₀= 3.20

D₁₅= 0.946

D₁₀= 0.600

C_u= 16.09

C_c= 1.77

Classification

USCS= GW

AASHTO=

Remarks

Sample No.: B-7 (S-03 & S04)

Source of Sample: DGS CCC Multipurpose Bldg., Fortuna,

Date: 6/17/19

Location:

Elev./Depth: 14.5-15.1 /

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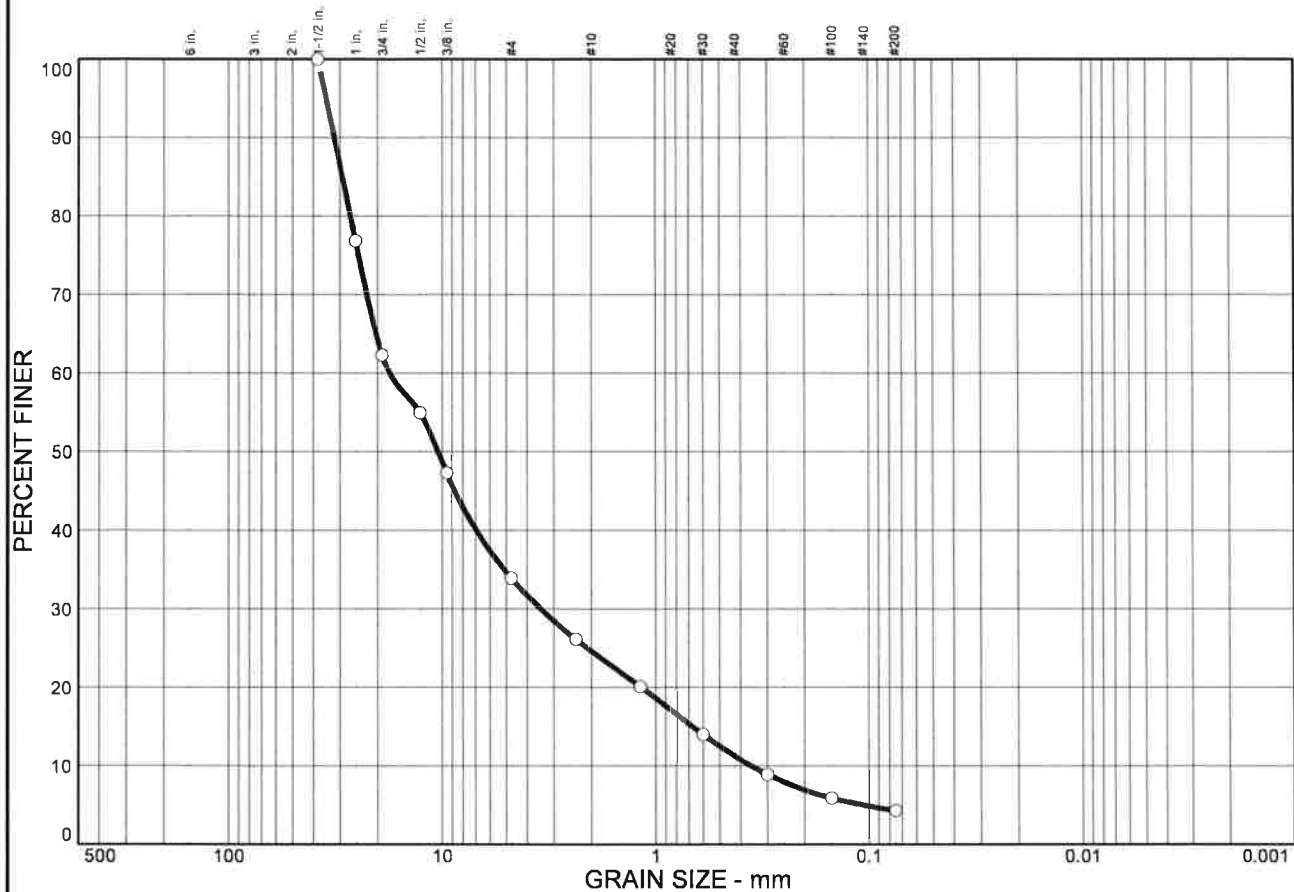
Client: Fugro USA Land

Project: DGS CCC Multi purpose Bldg., Fortuna, 4.72190034

Project No: Pending

PLATE B-13

Particle Size Distribution Report



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	66.1	29.6		4.3

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1-1/2 in.	100.0		
1 in.	76.8		
3/4 in.	62.3		
1/2 in.	54.9		
3/8 in.	47.3		
#4	33.9		
#8	26.1		
#16	20.1		
#30	14.0		
#50	8.9		
#100	5.9		
#200	4.3		

* (no specification provided)

Material Description

Well-graded gravel with sand

Atterberg Limits

PL=

LL=

PI=

Coefficients

D₈₅= 29.3

D₆₀= 17.6

D₅₀= 10.5

D₃₀= 3.46

D₁₅= 0.672

D₁₀= 0.357

C_u= 49.18

C_c= 1.91

Classification

USCS= GW

AASHTO=

Remarks

Sample No.: B-7 (S-05)

Location:

Source of Sample: DGS CCC Multipurpose Bldg., Fortuna,

Date: 6/17/19

Elev./Depth: 24.5'

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Fresno, CA

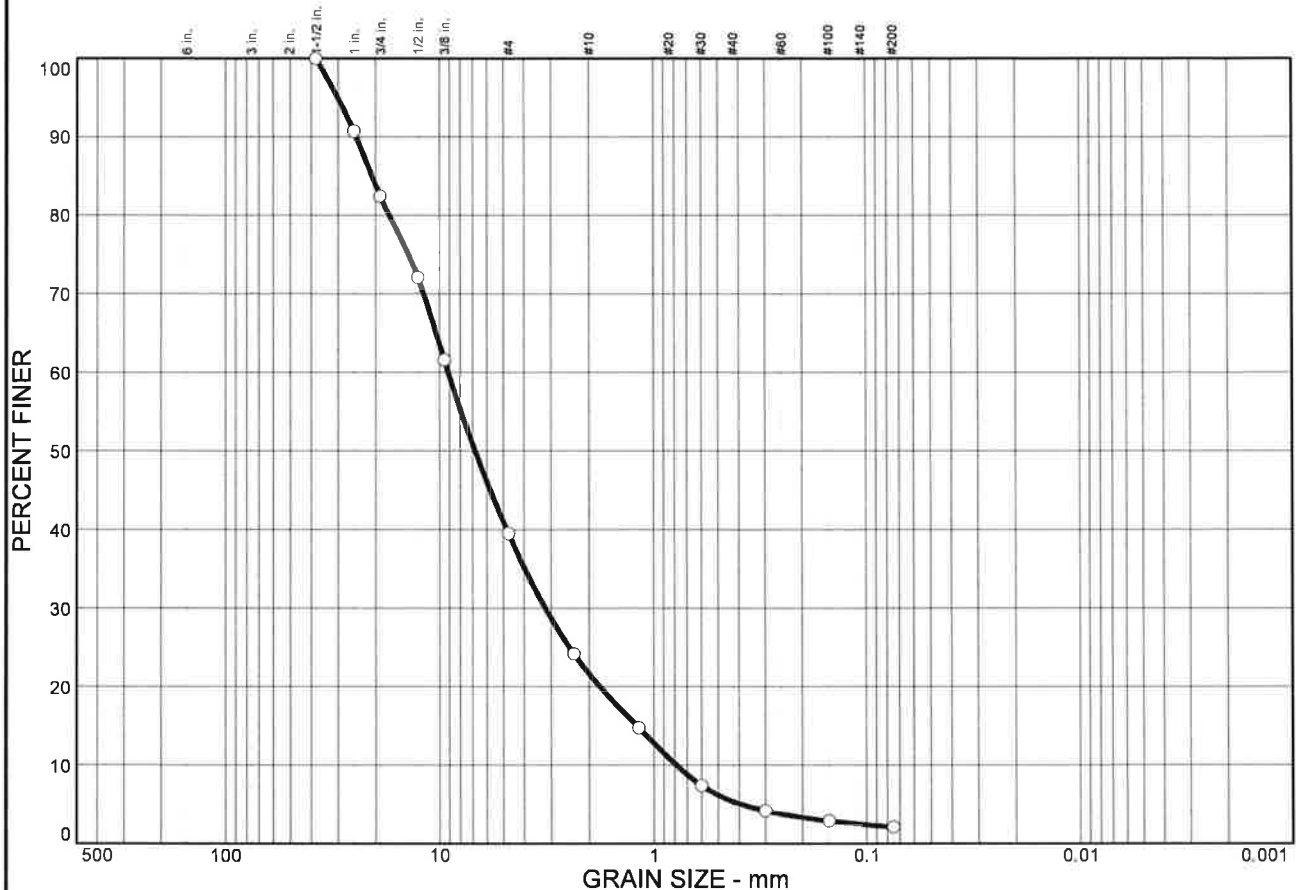
Client: Fugro USA Land

Project: DGS CCC Multi purpose Bldg., Fortuna, 4.72190034

Project No: Pending

PLATE B-14

Particle Size Distribution Report



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	60.5	37.4	2.1	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1-1/2 in.	100.0		
1 in.	90.7		
3/4 in.	82.4		
1/2 in.	72.1		
3/8 in.	61.6		
#4	39.5		
#8	24.2		
#16	14.8		
#30	7.4		
#50	4.2		
#100	2.9		
#200	2.1		

Material Description
Well-graded gravel with sand

Atterberg Limits
 PL= LL= PI=

Coefficients
 D₈₅= 20.9 D₆₀= 9.13 D₅₀= 6.82
 D₃₀= 3.20 D₁₅= 1.20 D₁₀= 0.787
 C_u= 11.60 C_c= 1.42

Classification
 USCS= GW AASHTO=

Remarks

* (no specification provided)

Sample No.: B-7 (S-06)
Location:

Source of Sample: DGS CCC Multipurpose Bldg., Fortuna,

Date: 6/17/19

Elev./Depth: 29.5-31.0'

Moore Twining Associates, Inc.

Fresno, CA

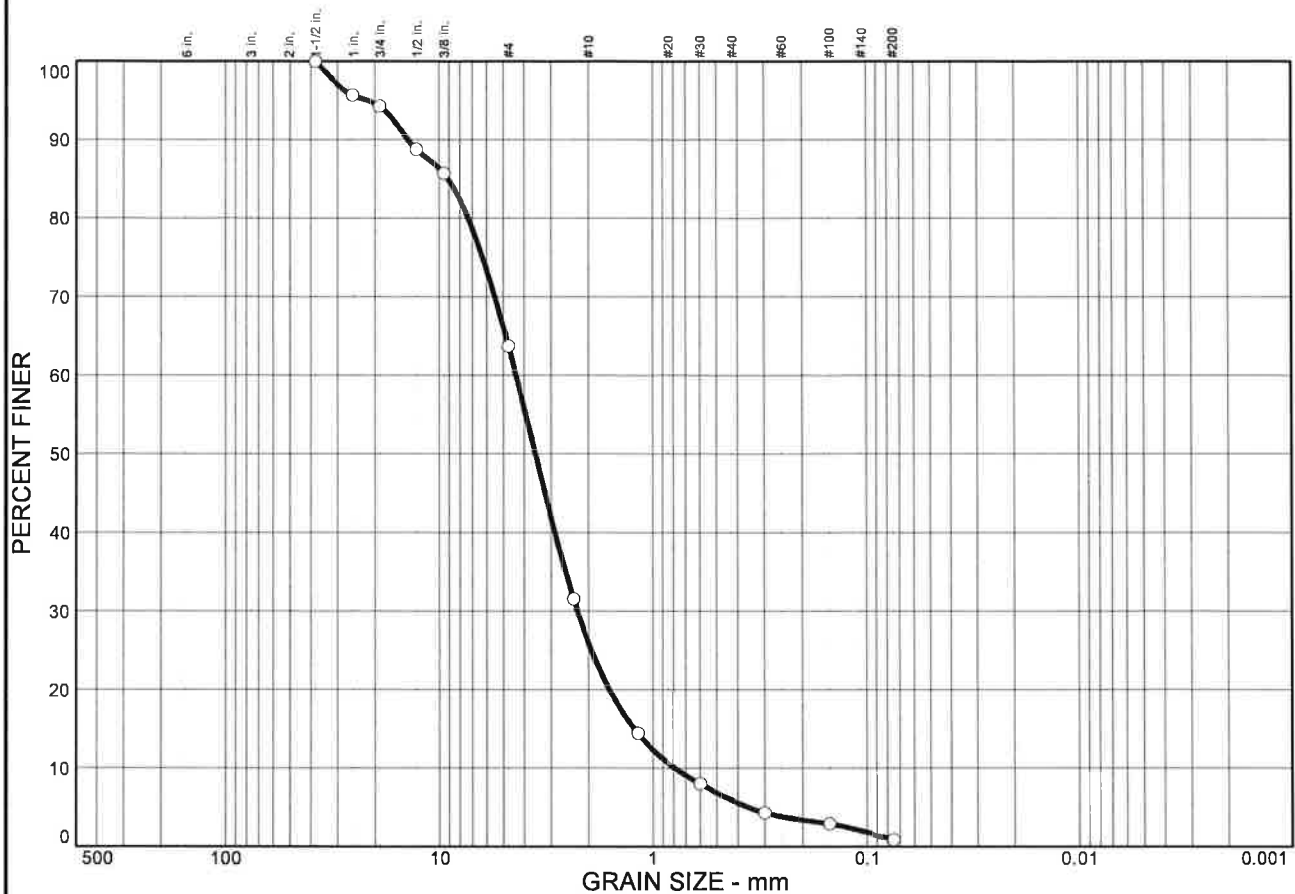
Client: Fugro USA Land

Project: DGS CCC Multi purpose Bldg., Fortuna, 4.72190034

Project No: Pending

PLATE B-15

Particle Size Distribution Report



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	36.3	62.8		0.9

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1-1/2 in.	100.0		
1 in.	95.6		
3/4 in.	94.2		
1/2 in.	88.7		
3/8 in.	85.7		
#4	63.7		
#8	31.5		
#16	14.4		
#30	8.0		
#50	4.3		
#100	2.9		
#200	0.9		

* (no specification provided)

Material Description

Poorly graded sand with gravel

Atterberg Limits

PL= LL= PI=

Coefficients

D₈₅= 9.11 D₆₀= 4.38 D₅₀= 3.56
D₃₀= 2.27 D₁₅= 1.23 D₁₀= 0.789
C_u= 5.56 C_c= 1.49

Classification

USCS= SP AASHTO=

Remarks

Sample No.: B-7 (S-08)

Location:

Source of Sample: DGS CCC Multipurpose Bldg., Fortuna,

Date: 5/17/19

Elev./Depth: 39.5-41'

Moore Twining Associates, Inc.

Fresno, CA

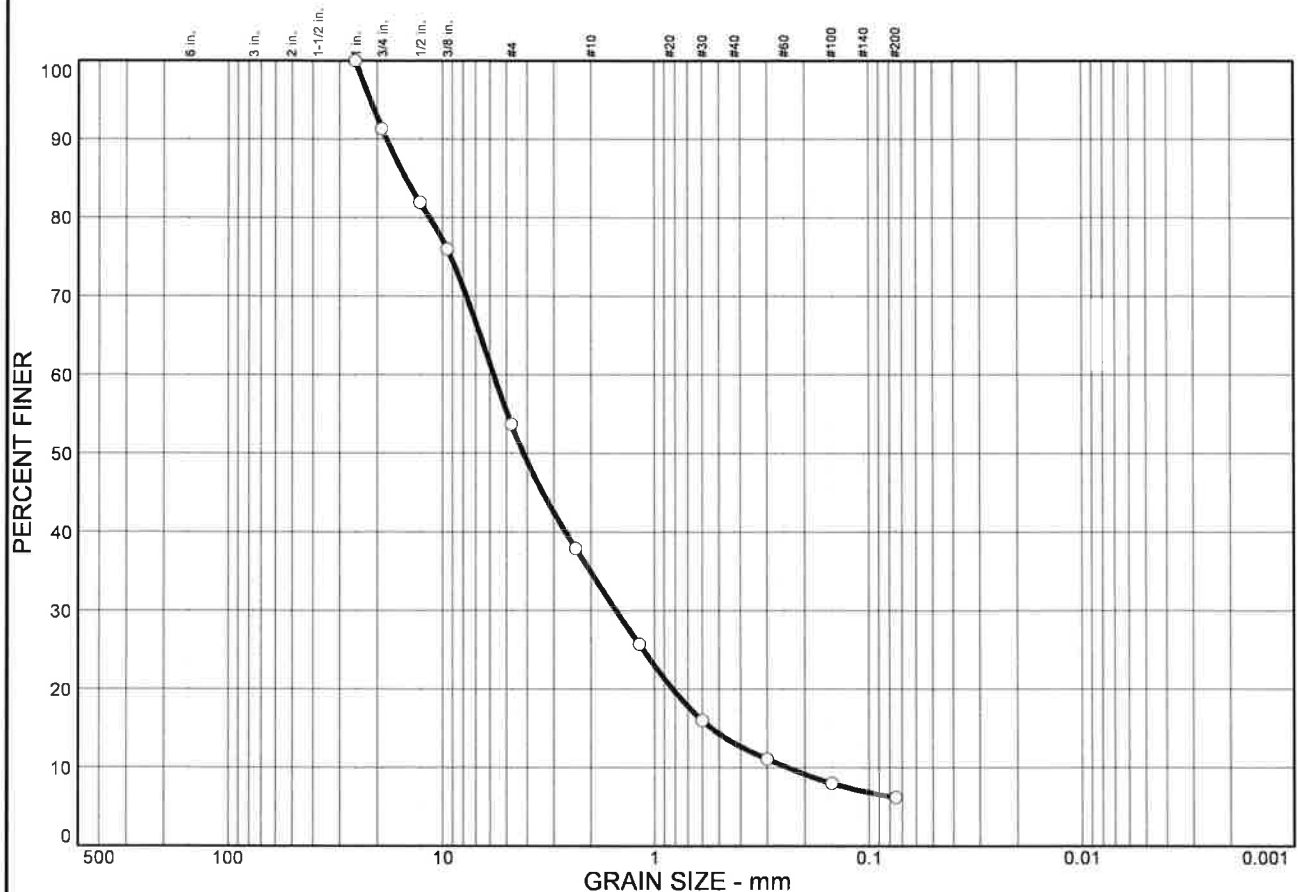
Client: Fugro USA Land

Project: DGS CCC Multi purpose Bldg., Fortuna, 4.72190034

Project No: Pending

PLATE B-16

Particle Size Distribution Report



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	46.3	47.5	6.2	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1 in.	100.0		
3/4 in.	91.3		
1/2 in.	81.9		
3/8 in.	76.0		
#4	53.7		
#8	37.9		
#16	25.7		
#30	16.0		
#50	11.1		
#100	8.0		
#200	6.2		

* (no specification provided)

Material Description

Well-graded sand with silt and gravel

Atterberg Limits

PL= LL= PI=

Coefficients

D₈₅= 14.8 D₆₀= 5.76 D₅₀= 4.17
D₃₀= 1.52 D₁₅= 0.542 D₁₀= 0.241
C_u= 23.94 C_c= 1.66

Classification

USCS= SW-SM AASHTO=

Remarks

Sample No.: B-7 (S-09)

Location:

Source of Sample: DGS CCC Multipurpose Bldg., Fortuna,

Date: 6/17/19

Elev./Depth: 44.5-46.0'

Moore Twining Associates, Inc.

Fresno, CA

Client: Fugro USA Land

Project: DGS CCC Multi purpose Bldg., Fortuna, 4.72190034

Project No: Pending

PLATE B-17

Grain size distribution curve showing Percent Finer versus Grain Size (mm). The curve starts at 100% finer for 60 mm and decreases to approximately 5% finer for 0.075 mm.

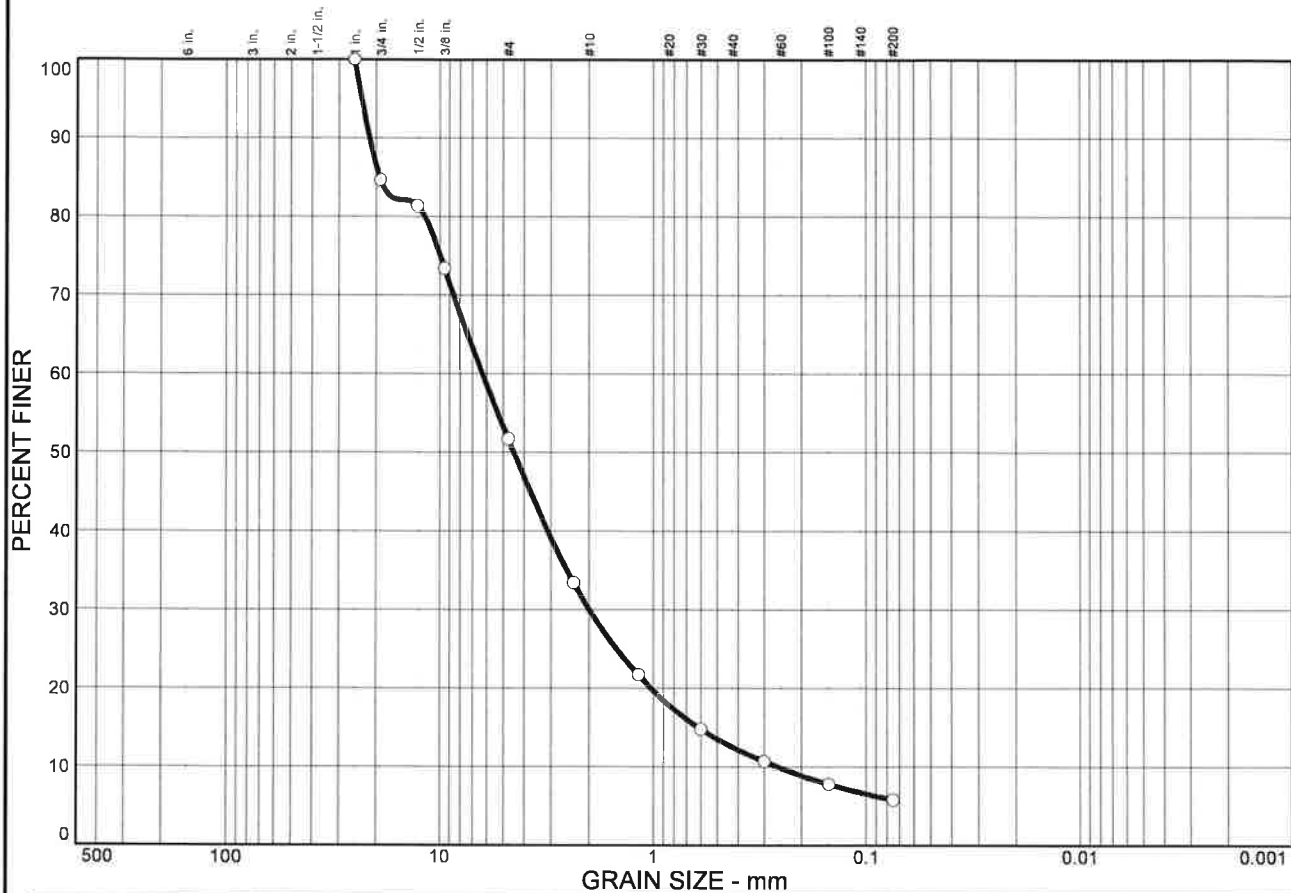
Grain Size (mm)	Percent Finer (%)
60	100
4.75	86
2.5	67
1.18	57
0.85	39
0.6	29
0.425	25
0.3	23
0.25	12
0.15	7
0.075	5

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1 in.	100.0		
3/4 in.	85.6		
1/2 in.	67.3		
3/8 in.	56.8		
#4	38.6		
#8	28.9		
#16	25.3		
#30	22.6		
#50	11.7		
#100	7.0		
#200	5.2		

Remarks

PLATE B-18

Particle Size Distribution Report



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	48.3	45.9		5.8

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1 in.	100.0		
3/4 in.	84.6		
1/2 in.	81.3		
3/8 in.	73.4		
#4	51.7		
#8	33.4		
#16	21.7		
#30	14.8		
#50	10.7		
#100	7.8		
#200	5.8		

* (no specification provided)

Material Description

Well-graded gravel with silt and sand

Atterberg Limits

PL=

LL=

PI=

Coefficients

D₈₅= 19.3

D₆₀= 6.28

D₅₀= 4.48

D₃₀= 2.00

D₁₅= 0.616

D₁₀= 0.258

C_u= 24.30

C_c= 2.46

Classification

USCS= GW-GM

AASHTO=

Remarks

Sample No.: B-7 (S-12)

Source of Sample: DGS CCC Multipurpose Bldg., Fortuna,

Date: 6/17/19

Location:

Elev./Depth: 59.5-61.0'

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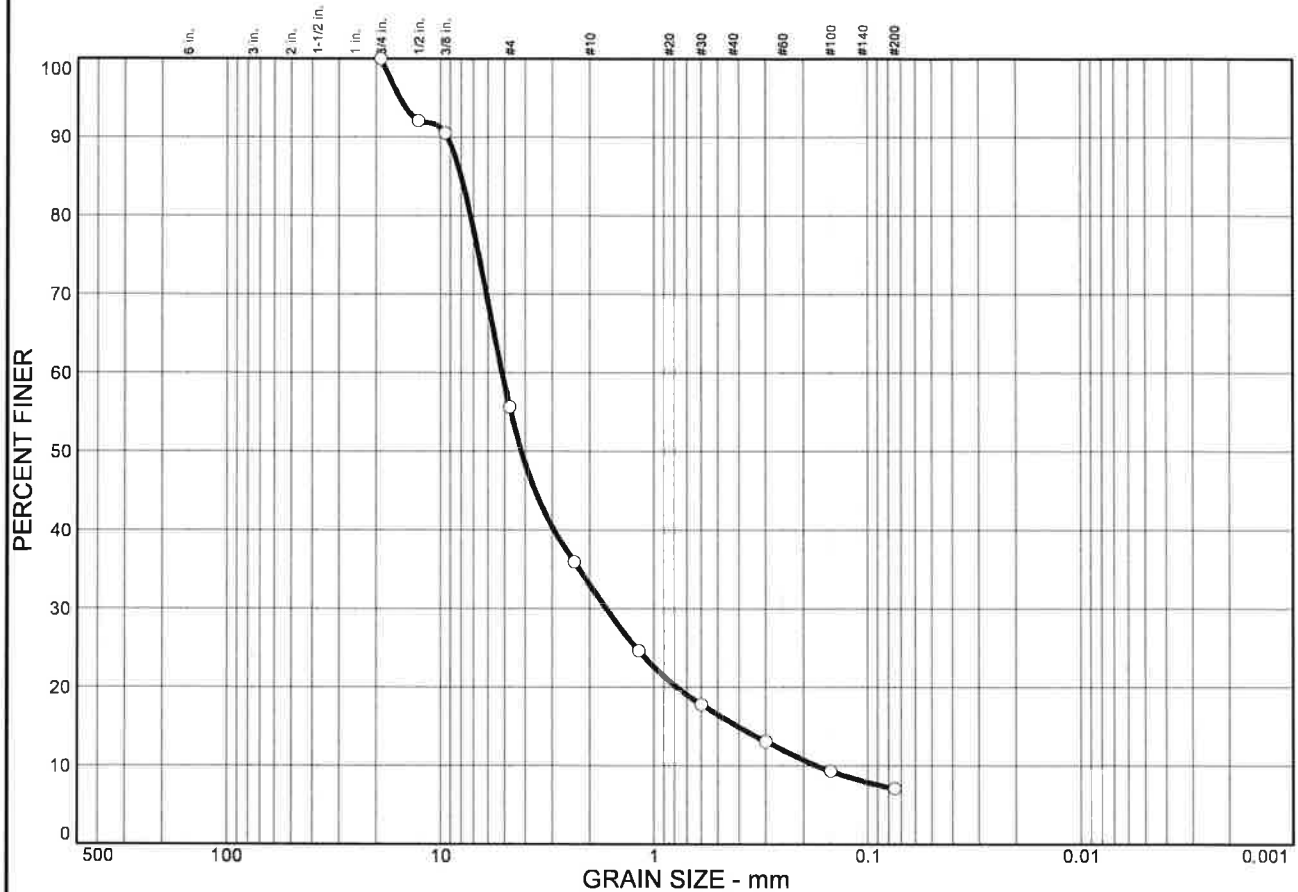
Client: Fugro USA Land

Project: DGS CCC Multi purpose Bldg., Fortuna, 4.72190034

Project No: Pending

PLATE B-19

Particle Size Distribution Report



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	44.4	48.5		7.1

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3/4 in.	100.0		
1/2 in.	92.0		
3/8 in.	90.5		
#4	55.6		
#8	35.9		
#16	24.6		
#30	17.8		
#50	13.1		
#100	9.3		
#200	7.1		

* (no specification provided)

Material Description

Poorly graded sand with silt and gravel

Atterberg Limits

PL=

LL=

PI=

Coefficients

D₈₅= 8.02

D₆₀= 5.16

D₅₀= 4.19

D₃₀= 1.68

D₁₅= 0.404

D₁₀= 0.174

C_u= 29.57

C_c= 3.13

Classification

USCS= SP-SM

AASHTO=

Remarks

Sample No.: B-7 (S-13B & S13A) Source of Sample: DGS CCC Multipurpose Bldg., Fortuna, Date: 6/17/19
Location: Elev./Depth: 64.5- 65.5'

Moore Twining Associates, Inc.

Fresno, CA

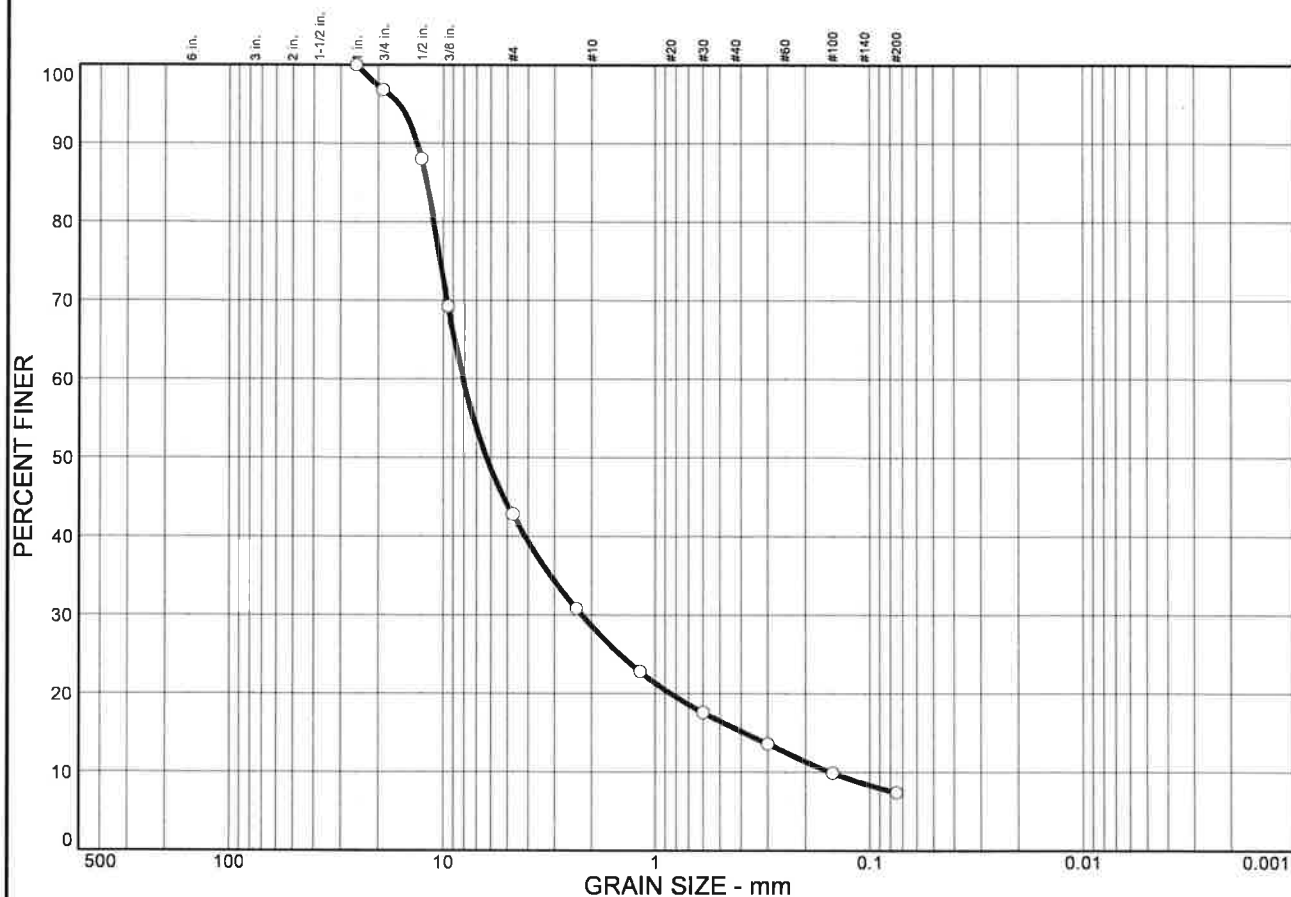
Client: Fugro USA Land

Project: DGS CCC Multi purpose Bldg., Fortuna, 4.72190034

Project No: Pending

PLATE B-20

Particle Size Distribution Report



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	57.2	35.4	7.4	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1 in.	100.0		
3/4 in.	96.8		
1/2 in.	88.0		
3/8 in.	69.3		
#4	42.8		
#8	30.8		
#16	22.8		
#30	17.6		
#50	13.6		
#100	9.9		
#200	7.4		

* (no specification provided)

<u>Material Description</u>		
Poorly graded gravel with silt and sand		
<u>Atterberg Limits</u>		
PL=	LL=	PI=
<u>Coefficients</u>		
D ₈₅ = 12.0	D ₆₀ = 8.08	D ₅₀ = 6.28
D ₃₀ = 2.22	D ₁₅ = 0.386	D ₁₀ = 0.153
C _u = 52.73	C _c = 3.99	
<u>Classification</u>		
USCS= GP-GM	AASHTO=	
<u>Remarks</u>		

Sample No.: B-7 (S-14)
Location:

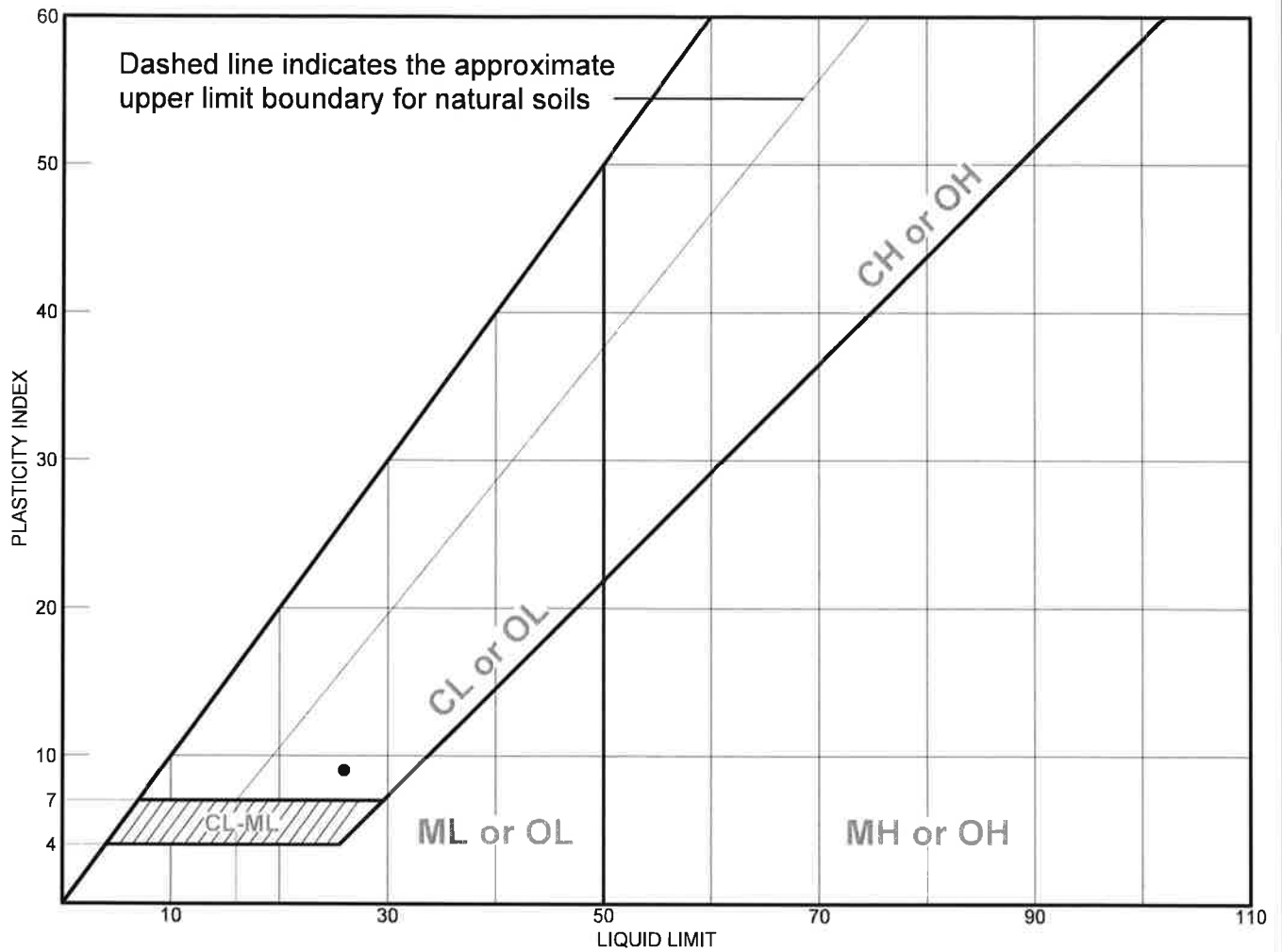
Source of Sample: DGS CCC Multipurpose Bldg., Fortuna, Date: 6/17/19
Elev./Depth: 74.5-76.0'

Moore Twining Associates, Inc.
Fresno, CA

Client: Fugro USA Land
Project: DGS CCC Multi purpose Bldg., Fortuna, 4.72190034
Project No: Pending

PLATE B-21

LIQUID AND PLASTIC LIMITS TEST REPORT



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	Sandy lean clay	26	17	9	88.6	56.2	CL

Project No. Pending

Client: Fugro USA Land

Project: DGS CCC Multi purpose Bldg., Fortuna, 4.72190034

● Source: DGS CCC

Sample No.: B-1 (B-01) Elev./Depth: 0-5'

Remarks:

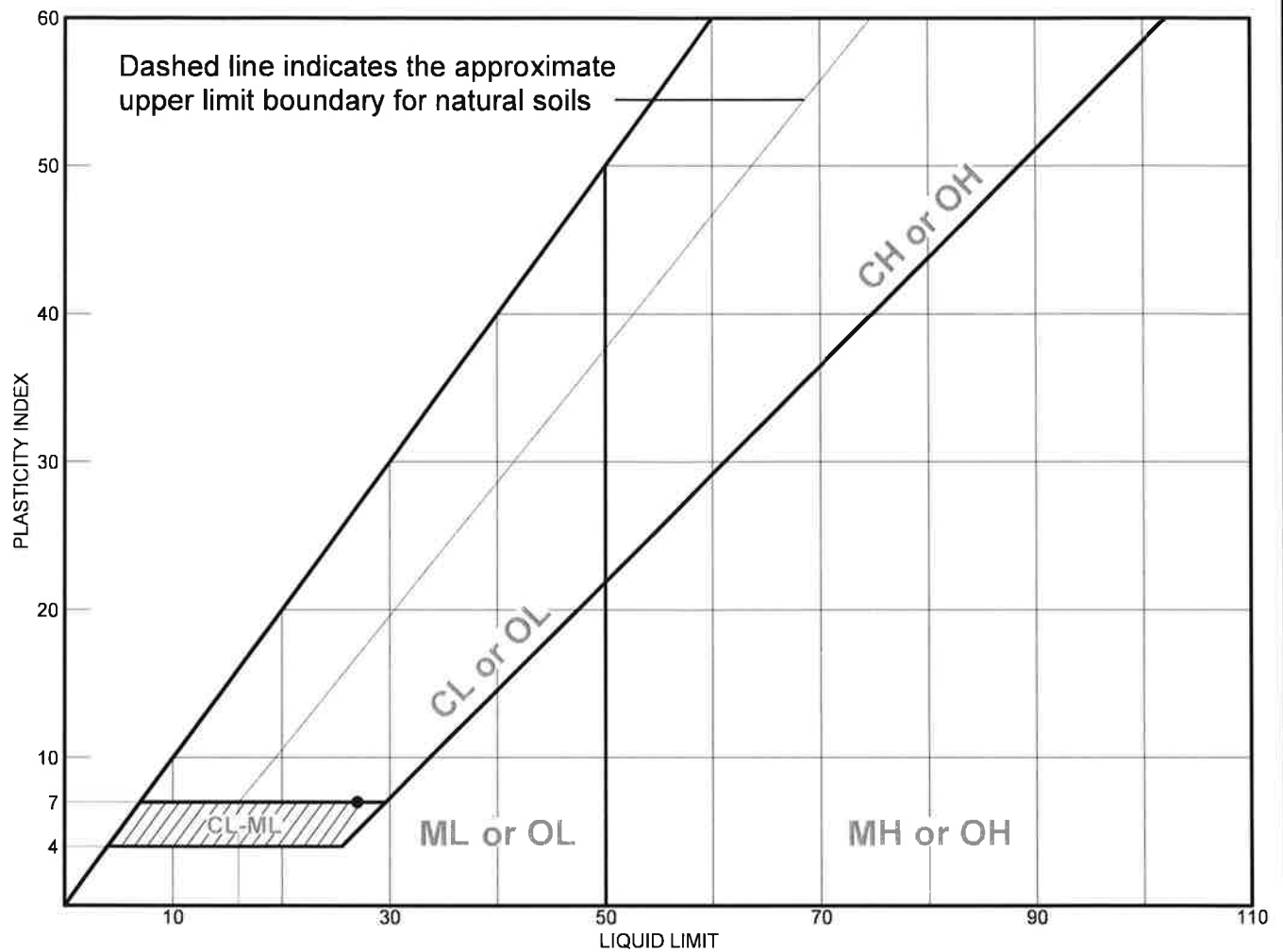
●

Moore Twining Associates, Inc.

Fresno, CA

PLATE B-22

LIQUID AND PLASTIC LIMITS TEST REPORT



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●		27	20	7		69.3	

Project No. Pending

Client: Fugro USA Land

Project: DGS CCC Multi purpose Bldg., Fortuna, 4.72190034

● Source: DGS CCC

Sample No.: B-1 (S-02)

Elev./Depth: 10-11.5'

Remarks:

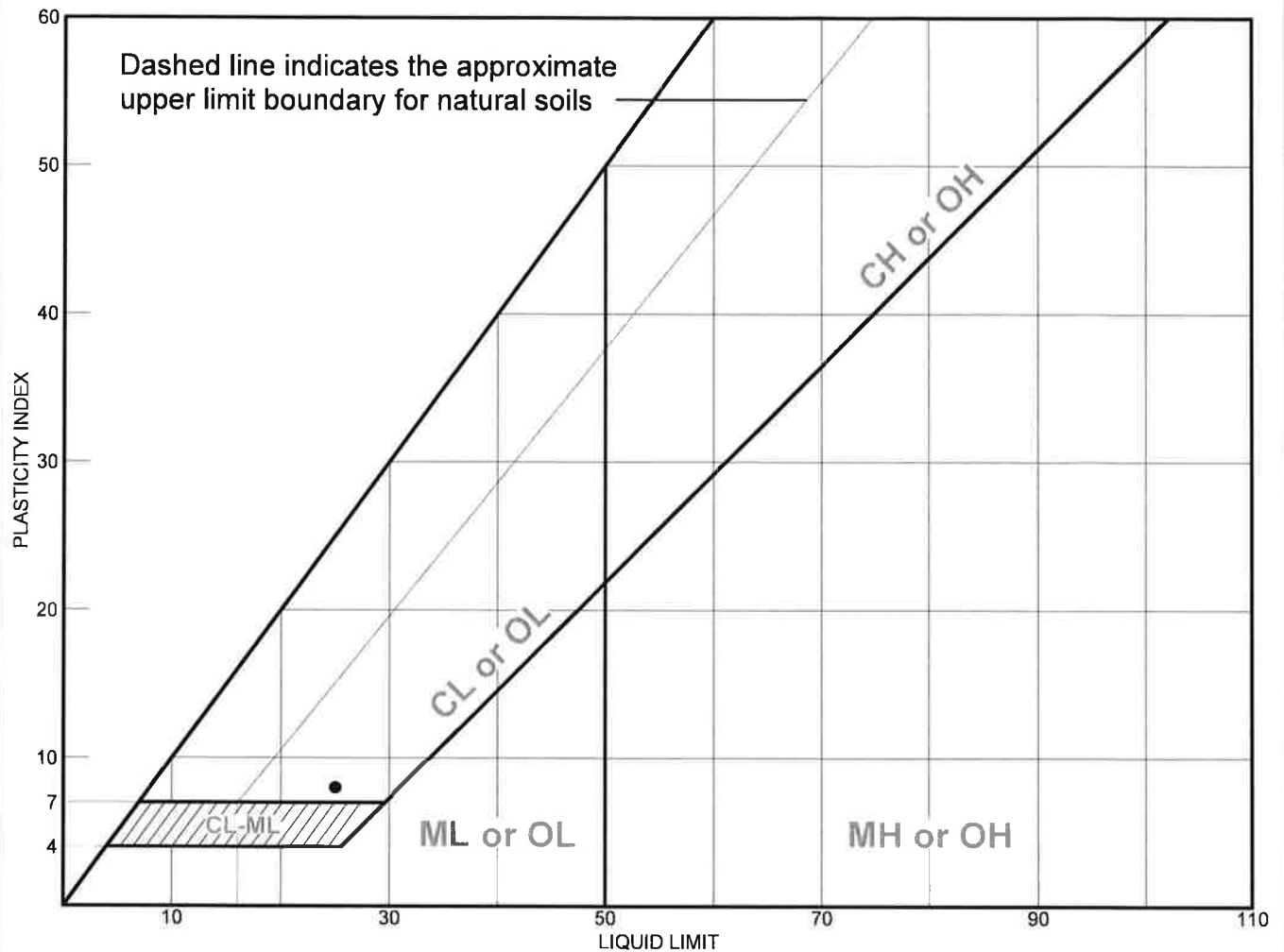
●

Moore Twining Associates, Inc.

Fresno, CA

PLATE B-23

LIQUID AND PLASTIC LIMITS TEST REPORT



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	Sandy lean clay	25	17	8	71.6	53.3	CL

Project No. Pending

Client: Fugro USA Land

Project: DGS CCC Multi purpose Bldg., Fortuna, 4.72190034

● Source: DGS CCC

Sample No.: B-7 (B-01) Elev./Depth: 0-5'

Remarks:

●

Moore Twining Associates, Inc.

Fresno, CA

PLATE B-24

A graph showing the relationship between Dry density (pcf) and Water content (%) for a soil sample. The x-axis represents Water content (%) from 4 to 16, and the y-axis represents Dry density (pcf) from 113.5 to 126.0. The curve is a smooth, downward-sloping parabola-like shape, indicating the soil's compaction characteristics.

Water content, %	Dry density, pcf
6.2	120.9
8.8	122.5
10.5	123.5
13.2	116.2

Elev/ Depth	Classification		Nat. Moist.	Sp.G.	LL	PI	% > 3/8 in.	% < No.200
	USCS	AASHTO						
0-5'	CL				25	8	7.3	53.3

TEST RESULTS		MATERIAL DESCRIPTION
Maximum dry density = 123.4 pcf Optimum moisture = 10.1 %		Sandy lean clay
Project No. Pending Client: Fugro USA Land Project: DGS CCC Multi purpose Bldg., Fortuna, 4.72190034 ● Source: DGS CCC Sample No.: B-7 (B-01) Elev./Depth: 0-5'		Remarks:
Moore Twining Associates, Inc. Fresno, CA		
		PLATE B-25



MOORE TWINING
ASSOCIATES, INC.

DENSITY MOISTURE

PROJECT Fugro USALand - DGS CCC Multipurpose Bldg., Fortuna - 4.72190034 DATE 5/16/2019

PROJECT NUMBER A28019.12 TECHNICIAN 997

BORING NO.	B-1									
DEPTH	6-6.5'									
SAMPLE NO.	S-01A									
LENGTH (IN.)	5.1									
TOTAL WT. (g)	1043.8									
WET WT. (g)	263.4									
DRY WT. (g)	236.1									
WET DENSITY	127.8									
% MOISTURE	11.6									
DRY DENSITY	114.5									

BORING NO.										
DEPTH										
SAMPLE NO.										
LENGTH (IN.)										
TOTAL WT. (g)										
WET WT. (g)										
DRY WT. (g)										
WET DENSITY										
% MOISTURE										
DRY DENSITY										

PLATE B-26



Project Name:	Fugro USA Land - DGS CCC Multipurpose Bldg., Fortuna - 4.72190034	Report Date:	6/3/2019
Project Number:	A28019.12	Sample Date:	5/16/2019
Subject:	Minimum Resistivity, ASTM G187	Sampled By:	Client
Material Description:	Sandy lean clay	Tested By:	MA
Location:	B-1 (B-01A) @ 0-5'	Test Date:	5/29/2019

Laboratory Test Results, Minimum Resistivity - ASTM G187

Total Water Added, mls	Resistivity, Ohm-cm
50 mls	64,699
100 mls	58,696
150 mls	52,026
200 mls	46,023
250 mls	38,019
300 mls	29,347
350 mls	17,342
400 mls	10,339
450 mls	6,203
500 mls	3,402
550 mls	3,468

Remarks: Min. Resistivity is 3,402 Ohm-cm

PLATE B-27



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

May 29, 2019

Work Order #: **FE20010**

Michael Shwiyhat
MTA Geotechnical Division
2527 Fresno Street
Fresno, CA 93721

RE: DGSCCC Multipurpose Bldg. Fortuna

Enclosed are the analytical results for samples received by our laboratory on **05/20/19**. For your reference, these analyses have been assigned laboratory work order number **FE20010**.

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



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

MTA Geotechnical Division
2527 Fresno Street
Fresno CA, 93721

Project: DGSCCC Multipurpose Bldg. Fortuna
Project Number: 4.72190034
Project Manager: Michael Shwiyhat

Reported:
05/29/2019

Analytical Report for the Following Samples

Sample ID	Notes	Laboratory ID	Matrix	Date Sampled	Date Received
B1@0-5 B01A		FE20010-01	Soil	05/20/19 00:00	05/20/19 09:45

MTA Geotechnical Division
2527 Fresno Street
Fresno CA, 93721

Project: DGSCC Multipurpose Bldg. Fortuna
Project Number: 4.72190034
Project Manager: Michael Shwiyhat

Reported:
05/29/2019

B1@0-5 B01A

FE20010-01 (Soil)

Sampled: 05/20/19 00:00

Analyte	Flag	Result	Reporting Limit	Units	Dilution	Batch	Prepared	Analyzed	Method
Inorganics									
Chloride		ND	6.0	mg/kg	3	B9E2129	05/21/19	05/23/19	ASTM D4327-84
Chloride		ND	0.00060	% by Weight	3	[CALC]	05/23/19	05/23/19	ASTM D4327-84
Sulfate as SO ₄		0.0037	0.00060	% by Weight	3	[CALC]	05/23/19	05/23/19	ASTM D4327-84
pH		5.8	0.10	pH Units	1	B9E2129	05/21/19	05/24/19	ASTM D4972-89 Mod
Sulfate as SO ₄		37	6.0	mg/kg	3	B9E2129	05/21/19	05/23/19	ASTM D4327

Notes and Definitions

DUP1 A high RPD was observed between a sample and this sample's duplicate.

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)**

ANALYTICAL CHEMISTRY DIVISION
CALIFORNIA ELAP CERTIFICATION # 1371

WORK ORDER #: FEZ0010
PAGE 1 OF 2

REPORT TO:		INVOICE TO:		REPORT COPY TO:		REPORTING:								
ATTENTION: Michael Shwighat		ATTENTION:				☒ STANDARD FORMAT ☐ PDF ☐ EDT (SWRCB) ☐ EXCEL ☐ GEOTRACKER/COELT (LUFT) GLOBAL ID: ☐ COUNTY ENVIRONMENTAL HEALTH: ☐ STATE WATER RESOURCES CONTROL BOARD: ☐ OTHER:								
COMPANY NAME:		COMPANY NAME:												
ADDRESS:		ADDRESS:												
PHONE:		PHONE:												
EMAIL / FAX:		EMAIL / FAX:												
SAMPLE INFORMATION				SAMPLE TYPES		PROJECT INFORMATION								
SAMPLED BY (PRINT):				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: DGS CCC Multipurpose Bldg., Fortuna								
☐ PUBLIC SYSTEM ☒ ROUTINE ☐ PRIVATE WELL ☐ REPEAT ☐ OTHER ☐ REPLACEMENT						PROJECT NUMBER: 472190034								
TURN AROUND TIME ☐ STANDARD ☐ RUSH, DUE ON: 5-days						PROJECT MANAGER: Michael Shwighat								
						ANALYSIS REQUESTED								
LAB USE	NOTES ON RECEIVED CONDITION:				Corrosion									STATION CODE
	☐ CUSTODY SEAL(S) BROKEN ☐ SAMPLE(S) DAMAGED													
	☐ ON ICE ☐ AMBIENT TEMP. ☐ INCORRECT PRESERVATION													
	CLIENT SAMPLE ID DATE TIME TYPE													
	1 BIC-0-5 BOIA 5-20-19 N/A SL ✓													
COMMENTS / ADDITIONAL INSTRUCTIONS:														
RELINQUISHED BY		COMPANY		DATE		TIME		RECEIVED BY		COMPANY				
Miguel Alcaraz		MTA/MAT		5-20-19		0945		[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.

PLATE B-31

FL-SC-0005-04 A

COC Info		Filter or Split		Container		Preservative		Date/Time/Initials	
Was temperature within range?	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~~	Chemistry $\leq 6^{\circ}\text{C}$ Micro $< 10^{\circ}\text{C}$ Temp $^{\circ}\text{C}$	Yes No ~~N/A~~	Did all bottles arrive unbroken and intact?	Yes No ~~N/A~~
If samples were taken today, is there evidence that chilling has begun?	Yes ~~No~~ N/A	Was a sufficient amount of sample received?	~~Yes~~ No N/A	Were PM notified of discrepancies?	Yes No ~~N/A~~	Do samples have a hold time < 72 hours?	Yes ~~No~~ N/A	125ml (A) 250ml (B) 1Liter (C) 40ml VOA (V)	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			Bacti $\text{Na}_2\text{S}_2\text{O}_3$			
						None (P)			
						Cr6 Buffer (P) Borate Carbonate Buffer			
						HNO_3 (P)			
						H_2SO_4 (P)			
						NaOH (P)			
						$\text{NaOH} + \text{ZnAc}$ (P)			
						Dissolved Oxygen 300ml (P)			
						None (AG)			
						None (CG) 500ml			
						$\text{Na}_2\text{S}_2\text{O}_3$ 250ml (Brown P) 549			
						$\text{Na}_2\text{S}_2\text{O}_3$ (AG)			
						$\text{Na}_2\text{S}_2\text{O}_3$ (AG)			
						Thio/K Citrate			
						NH_4Cl (AG) 552			
						HCl (AG)			
						None (CG) 500ml			
						H_3PO_4 (AG)			
						Other:			
						Plastic Bag			
						Low Level Hg/Metals Double Bag			
						Client Own			
						Glass Jar: 125/ 250/ 500			
						Soil Tube: Brass/ Steel/ Plastic			
						5 g Encore			
						Ascorbic Acid (AG) Voa			

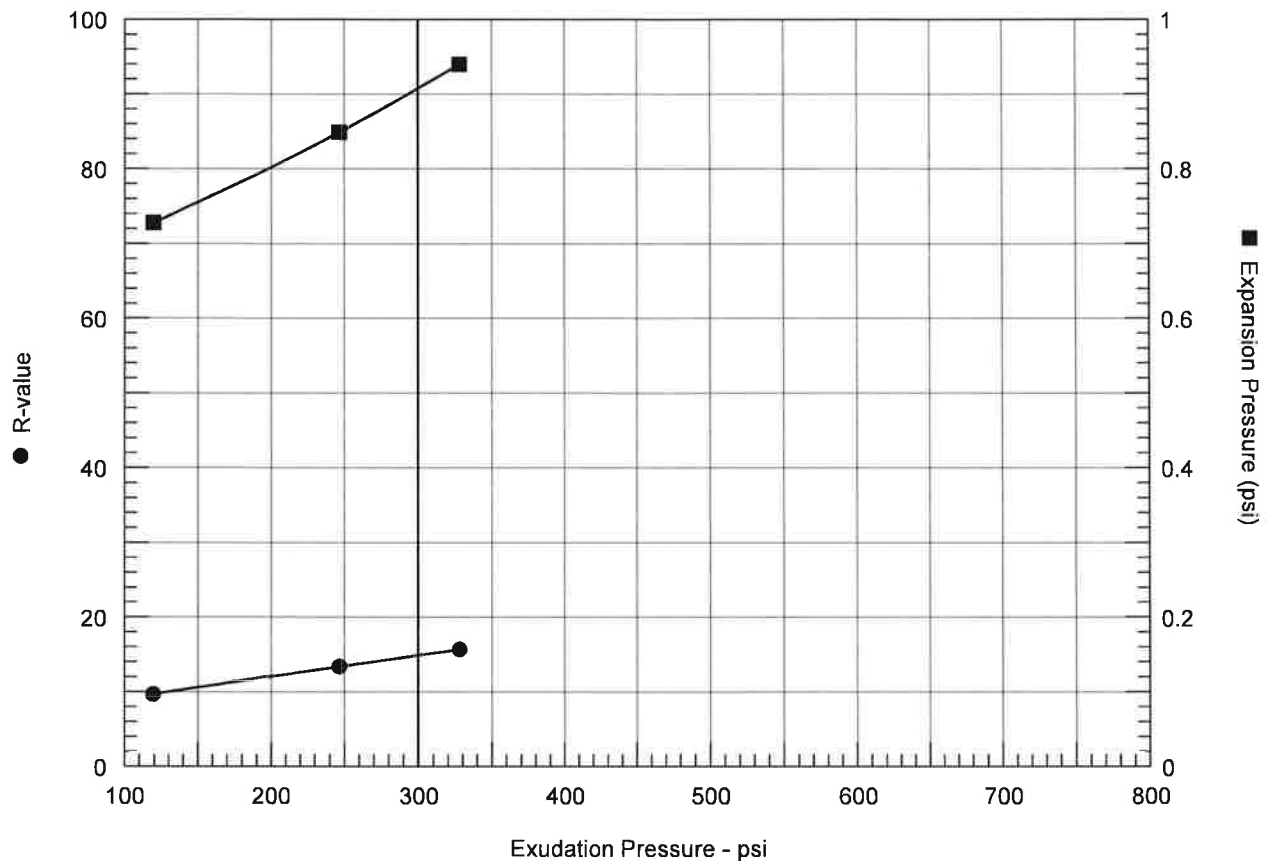
PLATE B-32

Labeled by:- AL @ 15/4/20

Labels checked by: h @ /04/

FL-SC-0003-06

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	50	114.8	15.7	0.73	129	2.54	119	10	10
2	100	119.0	13.7	0.94	115	2.48	328	16	16
3	100	116.8	14.7	0.85	120	2.51	247	13	13

Test Results	Material Description
R-value at 300 psi exudation pressure = 15 Exp. pressure at 300 psi exudation pressure = 0.91 psi	Sandy lean clay
Project No.: Pending Project: DGS CCC Multi purpose Bldg., Fortuna, 4.72190034 Source of Sample: DGS CCC Multipurpose Bldg., Fortuna Sample Number: B-1 (B-01) Date: 6/3/2019	Tested by: Checked by: Remarks:
R-VALUE TEST REPORT Moore Twining Associates, Inc.	PLATE B-33



APPENDIX C
PREVIOUS FIELD EXPLORATION LOGS AND LABORATORY TEST RESULTS

LOG OF BORING 1

Equipment 4.5" Ø FLIGHT AUGER

Elevation 48' ** Date 11/27/89

Laboratory Tests

Penetrometer (tsf)	Blows/foot *	Moisture Content(%)	Dry Density (pcf)
44.5	5	12.9	95
1.0	6	45.7	61
3.0	18		
	10	6.6	108



DARK BROWN SANDY SILT (ML)

medium stiff, moist to wet with pebbles to 1/4" & (Fill)

DARK GRAY-BROWN SANDY SILT (ML)

medium stiff, moist to wet with organic (grasses) material and pieces of wood (Fill)

GREEN-BROWN SANDY SILT (ML)

medium stiff, wet, with fine-grained sand

GRAY SANDY GRAVEL (GP)

medium dense, moist to wet with cobbles to 2.5" & and medium to coarse sand

No Free Ground Water Encountered

* Converted to Standard Penetration counts (N-values)

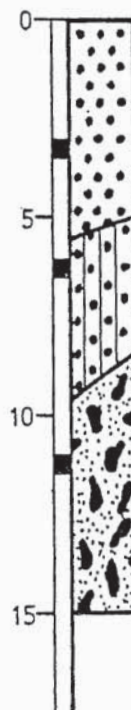
**Reference: NGVD Datum (Corps of Engineers Benchmark) Topographic Survey Map - A.P. No. 200-363-17 and 17 by Weale B. Penfold, dated 9/21/89

LOG OF BORING 2

Equipment 4.5" Ø FLIGHT AUGER

Elevation 48' Date 11/27/89

44.5	42	7.7	113
3.0	12	12.0	94
	31	1.8	112



DARK GRAY GRAVELLY SAND (SP)

dense, moist, fine- to coarse-grained with occasional gravel subrounded (FILL)

BROWN SILTY GRAY SAND (SM)

medium dense, moist, fine-grained

GRAY SANDY GRAVEL (GP)

dense, moist, gravel to 3.0" & with medium to coarse-grained sand

(HOLE COLLAPSED, UNABLE TO SAMPLE)

No Free Ground Water Encountered



Van Houten Consultants

Geologists and
Geotechnical
Engineers

LOG OF BORINGS 1 & 2
Geologic Hazards Study
South 12th Street Area
Fortuna, California

PLATE

2

Job No: 89-260.01 Appr: *[Signature]* Date: 1/24/90

Plate C-1

LOG OF TEST PIT 5

Laboratory Tests

Penetrometer
(tsf)

>4.5

1.0

Blows/foot

Moisture
Content(%)

Dry
Density (pcf)

Depth (ft)
Sample

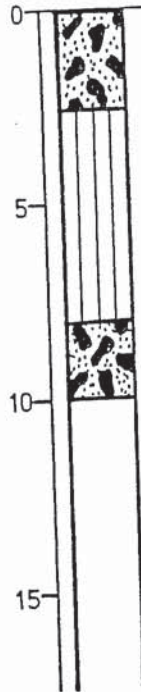
Equipment

BACKHOE

Elevation

46'

Date 11/27/89



GRAY SANDY GRAVEL (GP)

dense, wet to saturated

Water seepage @ 2.5'

BROWN SANDY CLAY SILT (ML)

soft to medium stiff, wet to saturated

GRAY SANDY GRAVEL (GP)

Dense, wet, with cobbles to 6" & with silt and fine-grained sand

LOG OF TEST PIT 6

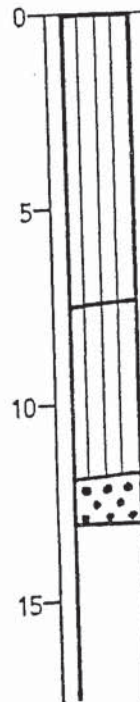
Equipment

BACKHOE

Elevation

48'

Date 11/27/89



BROWN SANDY SILT (ML)

stiff, moist to wet with rounded gravel to 2" & with debris (asphalt and wood) from 0.5' to 4.0'

(FILL)

Wire @ 5'

GREEN-BROWN SANDY SILT (ML)

medium stiff, wet

GRAY GRAVELLY SAND (SP)

medium dense, moist to wet with gravel to 3" &

No Free Ground Water Encountered



Van Houten Consultants

Geologists and
Geotechnical
Engineers

Job No: 89-260.01

Appr: *gva*

Date: 1/24/90

Plate C-2

LOG OF TEST PITS 5 & 6
Geologic Hazards Study
South 12th Street Area
Fortuna, California

PLATE

9



APPENDIX D

SITE SPECIFIC SEISMIC HAZARD ANALYSES

This appendix summarizes the analyses and results of a site-specific Probabilistic Seismic Hazard Analysis (PSHA) conducted to estimate the severity of ground motions that may affect the project site for specific design levels of hazard. The PSHA was conducted using the seismic source model adopted by the United States Geological Survey (USGS) to develop the 2008 National Seismic Hazard Map Project (NSHMP) (Petersen et al., 2008), and the Ground Motions Models (GMM) adopted by the USGS to develop the 2014 NSHMP (Petersen et al., 2014). The design ground motion parameters were calculated following the site-specific ground motion procedures defined in Chapter 21 of ASCE 7-10 (ASCE, 2010) as required by CBC (CBSC, 2016) for structures assigned to Seismic Design Category E.

D.1 Subsurface Conditions for the Seismic Hazard Assessment

The time-weighted average shear wave velocity (V_s) in the top 100 feet (30 meters [m]) (V_{s30}) is an important input parameter to include the local site conditions in the PSHA. In-situ V_s measurements are not available at the project site, and therefore, V_s profiles were estimated using the empirical correlations between Standard Penetration Test (SPT) data and V_s recommended for sandy soils by Wair et al. (2012) and Brandenberg et al. (2010). A V_s profile was calculated for each boring B-1 and B-7 using the average of the V_s estimates from the two empirical correlations. These V_{ss} profiles are presented on **Figure D-1**. These V_s profiles extend to the maximum exploration depth of each boring.

Because the V_s profiles do not extend to 100 ft, a time-weighted average V_s was first calculated only to 50 ft (15 m) (V_{s14}) and 75 ft (23 m) (V_{s23}) for borings B-1 and B-7, respectively, and then, these V_{s14} and V_{s23} values were extended using the empirical correlation by Boore (2004) to calculate a V_{s30} value for each boring. The V_{s30} for the site was calculated as the average of the V_{s30} value for each boring. The calculated V_{s30} value for the site is 755 ft/sec (230 m/sec). This V_{s30} value corresponds to a Site Class D according to CBC (CBSC, 2016).

D.2 Probabilistic Seismic Hazard Analysis

D.2.1 Project Location

A PSHA was conducted for one representative location of the project site. The geographical coordinates of the location used for the seismic hazard analyses are tabulated in **Table D.1**.

Table D.1: Representative Project Location Coordinates used in the PSHA

Latitude	Longitude
40.5810°N	124.1530°W

D.2.2 Methodology

D.2.2.1 PSHA Framework

The methodology for a PSHA includes the following components:

1. Seismic Source Model. This includes defining the location, style, and rates of earthquake occurrence in the model area. The characterization includes developing values for the following seismic source parameters:

- a. Source location and geometry. All major active faults and seismotectonic provinces are defined within the model area. This includes the geographical extent at the surface as well as the orientation and depth of the source zones.
 - b. Source type (e.g., shallow crustal area source zones, fault sources, subduction zones, etc.) and style of faulting (e.g., normal, strike-slip, reverse, etc.).
 - c. Magnitude potential (i.e., range of earthquake sizes possible on each source) and magnitude distribution (i.e., characterized using a magnitude probability density function).
 - d. Earthquake magnitude recurrence, which is a characterization of the annual rate at which earthquakes of a specified magnitude or greater occur in each source.
2. Ground Motion Model. Characterization of ground motion attenuation characteristics of each source are based on the geologic and tectonic environment. These characteristics are described by a series of ground motion models, or GMM (also known as “attenuation relationships”, “attenuation models”, or “ground motion prediction equations”).
 3. Probabilistic Seismic Hazard Analysis. A PSHA uses inputs from the seismic source model and GMMs selected for the specific environment, to estimate the ground motion hazard at the site. The hazard is expressed in terms of the annual frequency of exceeding a given spectral acceleration at the project site (i.e., annual hazard curves). This information also can be shown in the form of uniform hazard response spectra (UHRS), which correspond to spectral acceleration having the same probability of exceedance across all structural periods. The UHRS are typically used by different design codes to define the design response spectra.
 4. Deaggregation of seismic hazard to evaluate the relative contribution of different earthquake scenarios, including magnitude, source-to-site distance, epsilon (number of standard deviations above or below the median), etc.

D.2.2.2 PSHA Calculation

Computation of the seismic hazard involves the combination of uncertainties in earthquake size, location, frequency, and resulting ground motions. The estimated annual rate at which the ground motion, A , will exceed a particular value, a , is computed by (Cornell, 1968):

$$\lambda[A > a] = \sum_{i=1}^{N_{source}} N(M_{min}) \iint P[A > a|m, r] f_M(m) f_R(r) dm dr$$

where N_{source} is the total number of seismic sources; $N(M_{min})$ is the annual rate of earthquake with magnitude greater than or equal to M_{min} ; $P[A > a|m, r]$ is the probability of the ground motion, A , exceeding the threshold value, a , given the earthquake magnitude and distance from the seismic source; and $f_M(m)$ and $f_R(r)$ are probability density functions describing magnitude and distance.

The computation of this integral is carried out numerically. By assuming that earthquake occurrence can be modeled as a Poisson process, the probability of exceedance in a specified exposure period (typically corresponding to the useful life of a project) may be estimated as follows:

$$P[A > a, t] = 1 - e^{-[\lambda(a)t]}$$

where $P[A > a, t]$ is the conditional probability of the spectral acceleration (A) exceeding a specified acceleration (a) during a time interval (t) given that an earthquake will occur, and $\lambda(a)$ is the mean annual rate of exceedance of the specified acceleration level.

D.2.2.3 Seismic Source Model

The PSHA was conducted using the seismic source model adopted by the USGS to develop the 2008 NSHMP (Petersen et al., 2008). The details of this seismic source model can be found in Petersen et al. (2008).

D.2.2.4 Empirical Ground Motion Models

The attenuation of seismic waves from a seismic source were modeled using empirical ground motion models (GMM). These empirical GMM should model the type of rupture mechanism as well as the regional geology to properly estimate site-specific strong ground motion parameters.

For the shallow crustal sources, four of the Next Generation Attenuation (NGA) West 2 GMMs (Bozorgnia et al., 2014) were used. These four NGA West 2 GMM are: Abrahamson et al. (2014), Boore et al. (2014), Campbell and Bozorgnia (2014), and Chiou and Youngs (2014). The four NGA West 2 GMMs were equally weighted, following the weighting scheme used in the development of the 2014 USGS NSHMP (Petersen et al., 2014).

For the subduction sources, the GMMs adopted by the USGS in the development of the 2014 NSHMP (Petersen et al., 2014) were used. For the subduction interface sources the following GMMs were used: Atkinson and Boore (2003) (Global model), Zhao et al. (2006), Atkinson and Macias (2009), and Abrahamson et al. (2016). For the subduction intraslab sources the following GMMs were used: Atkinson and Boore (2003) (Global and Cascadia models), Zhao et al. (2006), and Abrahamson et al. (2016). The weighting scheme of the GMMs for the subduction sources is the same weighting scheme as used in the development of the 2014 USGS NSHMP (Petersen et al., 2014).

D.2.2.5 Implementation

The PSHA was performed using the computer program EZ-FRISK™ (Fugro, 2015). The implementation of the USGS source model in EZ-FRISK™ was validated through comparison of ground motion estimates from the computer program against the 2008 USGS NSHMP values for 148 test sites showing agreement within 5 percent for the majority of the test sites.

D.2.3 **Results from the PSHA**

Figure D-2 shows the mean annual hazard curves for selected structural periods ranging from peak ground acceleration (PGA) to 10 sec. A structural period of 0.01 sec is used to represent the PGA. These hazard curves represent the total mean hazard from combining all seismic sources and ground motion models (GMMs). This figure also indicates the annual frequency of exceedance corresponding to a return period of 2,475 years.

Figures D-3 presents the seismic hazard deaggregation with respect to earthquake magnitude, distance, and epsilon for PGA and spectral acceleration (Sa) at 1 second for a return period of 2,475 years. **Table D.2** tabulates the calculated mean magnitude, distance, and epsilon for PGA and Sa at 1 second for a return period of 2,475 years. Epsilon is the number of standard deviations that the estimated ground motion amplitude deviates from the estimated median ground motion amplitude. Thus, an epsilon of 1 indicates that the probabilistic value of the ground motion corresponds to a median plus one-standard-deviation value.

Table D.2: Mean Seismic Hazard Deaggregation for a Return Period of 2,475 years and V_{S30} of 230 m/sec

	PGA	Sa at 1 sec.
Mean Magnitude (Mw)	8.2	8.0
Mean Distance (km)	12.9	11.4
Mean Epsilon	1.2	1.3

Figure D-4 presents the 5 percent-damped mean horizontal UHRS for a return period of 2,475 years and a V_{S30} of 230 m/sec. **Table D.3** tabulates the mean horizontal UHRS for periods ranging from 0.01 (i.e., PGA) to 10 sec for a return period of 2,475 years.

Table D.3: Mean Horizontal UHRS for Return Period of 2,475 Years and a V_{s30} of 230 m/sec, 5% Damping

Period (sec)	Horizontal Spectral Acceleration (g)
0.01 (PGA)	1.08
0.03	1.17
0.05	1.36
0.075	1.54
0.1	1.76
0.15	2.00
0.2	2.18
0.25	2.36
0.3	2.52
0.4	2.63
0.5	2.51
0.6	2.32
0.75	2.08
1	1.69
1.5	1.18
2	0.884
3	0.532
4	0.335
5	0.227
7.5	0.114
10	0.0709

D.3 Design Response Spectra and Peak Ground Acceleration for Liquefaction Evaluations

D.3.1 Design Response Spectra

According to CBC (CBSC, 2016), the proposed structure is assigned to Seismic Design Category E, and therefore, the design response spectrum and design acceleration parameters should be developed following the site-specific ground motion procedures defined in Chapter 21 of ASCE 7-10.

ASCE 7-10 defines a site-specific Risk-Targeted Maximum Considered Earthquake (MCE_R) as the lesser of probabilistic (MCE_R) and deterministic (MCE_R) ground motions. The probabilistic MCE_R ground motion is calculated as the ground motion in the direction of maximum horizontal response that is expected to achieve 1 percent probability of collapse within a 50-year period. The deterministic MCE_R ground motion is defined as the 84th percentile ground motion in the direction of maximum horizontal response of the largest acceleration from deterministic seismic hazard analysis (DSHA) of the characteristic earthquakes on all known active faults within the project region. Additionally, ASCE 7-10 specifies a lower limit to the deterministic MCE_R ground motion using a code-based spectrum. The site-specific design response spectrum is calculated as 2/3 of the MCE_R . The site-specific design response spectrum should be greater

than or equal to 80 percent of the spectral acceleration as determined by using the general response spectrum of Section 11.4.5 of ASCE 7-10.

The PSHA results described in the previous section were used to calculate the probabilistic MCE_R spectrum. As specified in ASCE 7-10, to obtain ground motions with a uniform 1 percent probability of collapse within a 50-year period, the UHRS for a return period of 2,475 was scaled by a risk coefficient, C_R . The C_R values were calculated using Method 1 described in Chapter 21 of ASCE 7-10. The mapped risk coefficients at spectral periods of 0.2 and 1.0 sec, C_{RS} and C_{R1} , respectively, were determined using USGS data accessed through the SEAOC (Structural Engineers Association of California) and OSHPD (California's Office of Statewide Health Planning and Development) online application (<https://seismicmaps.org>, last accessed July 12, 2019). The value of these risk coefficients C_{RS} and C_{R1} are 0.950 and 0.907, respectively. The ground motions in the direction of maximum horizontal response were calculated by applying the scaling factors recommended in ASCE 7-10. **Figure D-5** shows the UHRS along with the probabilistic MCE_R spectrum.

The deterministic MCE_R spectrum was calculated by performing a DSHA in EZ-FRISK™ (Fugro, 2015) using the same seismic source model and GMM's and weights used in the PSHA. The ground motions in the direction of maximum horizontal response were calculated by applying the scaling factors recommended in ASCE 7-10. **Figure D-5** illustrates the calculation of the deterministic MCE_R spectrum. The deterministic MCE_R spectrum was calculated as the maximum of the 84th DSHA spectrum and the lower limit specified by ASCE 7-10 calculated for a Site Class D. The transition period from constant velocity to constant displacement, T_L , required to calculate the lower limit, was estimated as 8 seconds using USGS data accessed through the SEAOC and OSHPD online application (<https://seismicmaps.org>, last accessed July 12, 2019).

Figure D-6 presents the development of the design response spectrum for the site. In this case, the probabilistic MCE_R spectrum is lower than the deterministic MCE_R spectrum for all the spectral periods, and therefore, the site-specific MCE_R spectrum is the same as the probabilistic MCE_R . Following ASCE 7-10, the recommended design response spectrum for the site was calculated as the maximum of 2/3 of the site-specific MCE_R and the lower limit specified by ASCE 7-10 (80 percent of general spectrum for Site Class D). **Table D.4** tabulates the spectral ordinates of the recommended design response spectrum and MCE_R spectrum per ASCE 7-10. The corresponding design acceleration parameters S_{MS} , S_{M1} , S_{DS} , and S_{D1} are tabulated in **Table D.5**.

Table D.4: MCE_R and Design Response Spectra per ASCE 7-10 for a Vs30 of 230 m/sec, 5% Damping

Period (sec)	Horizontal Spectral Acceleration (g)							
	UHS for Return Period of 2,475 Years	Probabilistic MCE _R	84 th Deterministic Spectrum	Deterministic Lower Limit	Deterministic MCE _R	Site- Specific MCE _R	80% General Response Spectrum	Design Response Spectrum
0.01 (PGA)	1.08	1.13	1.12	0.675	1.24	1.13	0.512	0.751
0.03	1.17	1.22	1.24	0.825	1.36	1.22	0.620	0.815
0.05	1.36	1.42	1.40	0.975	1.54	1.42	0.728	0.949
0.075	1.54	1.61	1.55	1.16	1.70	1.61	0.863	1.07
0.1	1.76	1.83	1.75	1.35	1.92	1.83	0.997	1.22
0.12	1.86	1.95	1.87	1.50	2.06	1.95	1.11	1.30
0.128	1.90	1.99	1.91	1.50	2.10	1.99	1.15	1.32
0.15	2.00	2.09	2.03	1.50	2.24	2.09	1.15	1.40
0.2	2.18	2.28	2.20	1.50	2.42	2.28	1.15	1.52
0.25	2.36	2.49	2.38	1.50	2.65	2.49	1.15	1.66
0.3	2.52	2.67	2.53	1.50	2.84	2.67	1.15	1.78
0.4	2.63	2.84	2.64	1.50	3.04	2.84	1.15	1.89
0.5	2.51	2.76	2.41	1.50	2.83	2.76	1.15	1.84
0.6	2.32	2.58	2.20	1.50	2.64	2.58	1.15	1.72
0.638	2.25	2.52	2.12	1.41	2.57	2.52	1.15	1.68
0.75	2.08	2.37	1.94	1.20	2.40	2.37	0.974	1.58
1	1.69	1.99	1.54	0.900	2.01	1.99	0.730	1.33
1.5	1.18	1.41	1.15	0.600	1.52	1.41	0.487	0.941
2	0.884	1.08	0.880	0.450	1.19	1.08	0.365	0.722
3	0.532	0.676	0.520	0.300	0.728	0.676	0.243	0.451
4	0.335	0.440	0.324	0.225	0.470	0.440	0.183	0.293
5	0.227	0.309	0.212	0.180	0.318	0.309	0.146	0.206
7.5	0.114	0.155	0.113	0.120	0.169	0.155	0.0974	0.103
8	0.102	0.139	0.102	0.113	0.153	0.139	0.0913	0.0927
10	0.0709	0.0965	0.0720	0.0720	0.108	0.0965	0.0584	0.0643

Table D.5: Design Acceleration Parameters per ASCE 7-10 for a Vs30 of 230 m/sec, 5% Damping

Parameter	Value
S _{MS}	2.56 g
S _{M1}	2.17 g
S _{DS}	1.70 g
S _{D1}	1.44 g

D.3.2 Peak Ground Acceleration for Liquefaction Evaluations

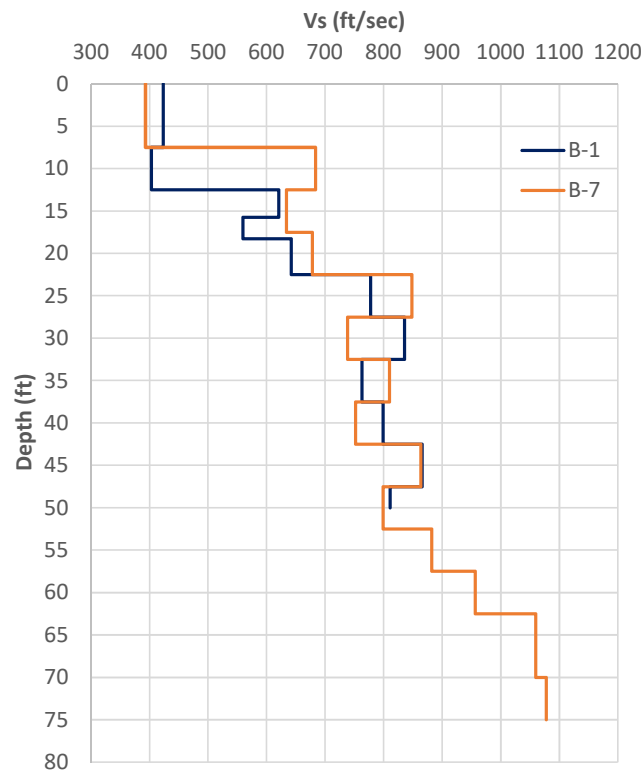
The peak ground acceleration (PGA) to evaluate the liquefaction potential of the soils at the site was calculated following the requirements of the California Building Code (CBC) (CBSC, 2016). For a site-specific study, the CBC requires that the PGA for liquefaction evaluations correspond to the Maximum Considered Earthquake Geometric Mean (MCEG) PGA determined following the site-specific ground motion procedures defined in Chapter 21 of ASCE 7-10.

ASCE 7-10 defines the site-specific MCEG PGA as the lesser of the probabilistic MCEG PGA and the deterministic MCEG PGA. Additionally, the site-specific MCEG PGA should not be less than 80% of the PGA determined using a code-based general procedure. The probabilistic MCEG PGA corresponds to the geometric mean PGA for a return period of 2,475 years. The deterministic MCEG PGA is calculated as the largest 84th percentile geometric mean PGA for the characteristic earthquakes on all known active faults within the project region. Additionally, ASCE 7-10 specifies a lower limit to the deterministic MCEG PGA using a code-based value.

The probabilistic and deterministic MCEG PGA were calculated using the results from previous sections and are tabulated in **Table D.6**. As presented in this table, in this case, the probabilistic MCEG PGA is lower than the deterministic MCEG PGA, and therefore, the site-specific MCEG PGA is the same as the probabilistic MCEG PGA.

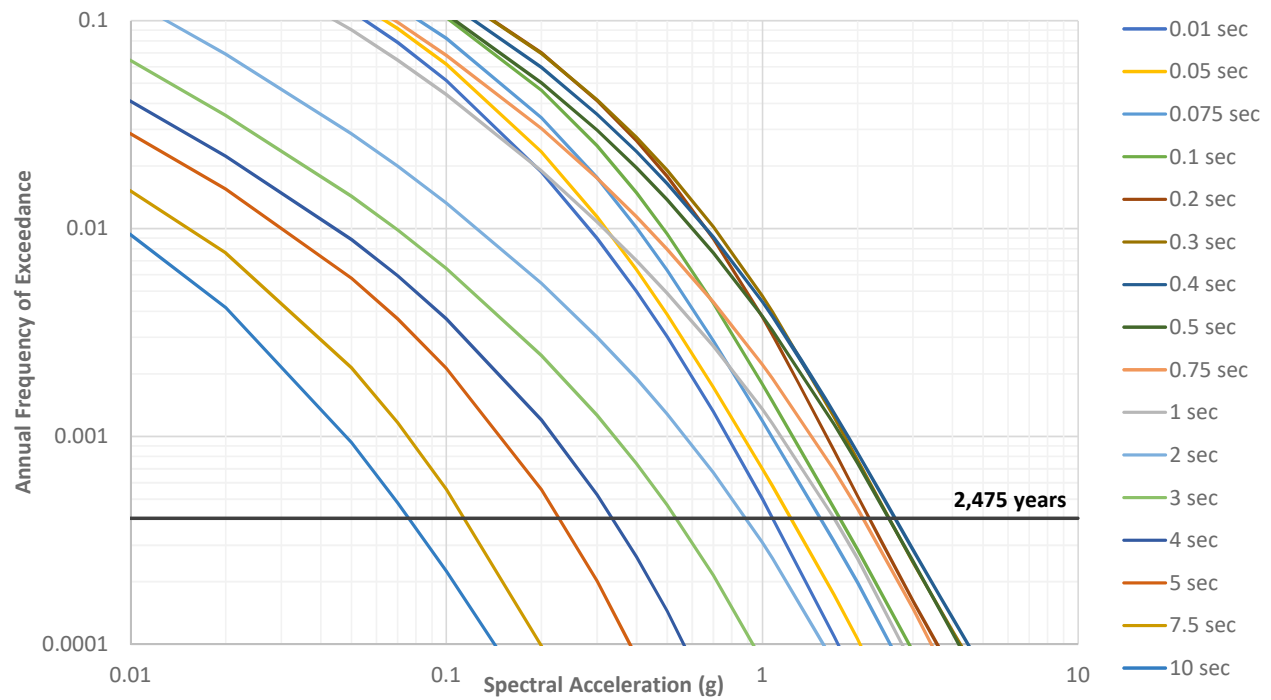
Table D.6: Calculation of Site-Specific MCEG PGA per ASCE 7-10 for a V_{s30} of 250 m/sec, 5% Damping

	MCEG PGA (g)
Probabilistic	1.08
Deterministic	1.12
Site-Specific	1.08



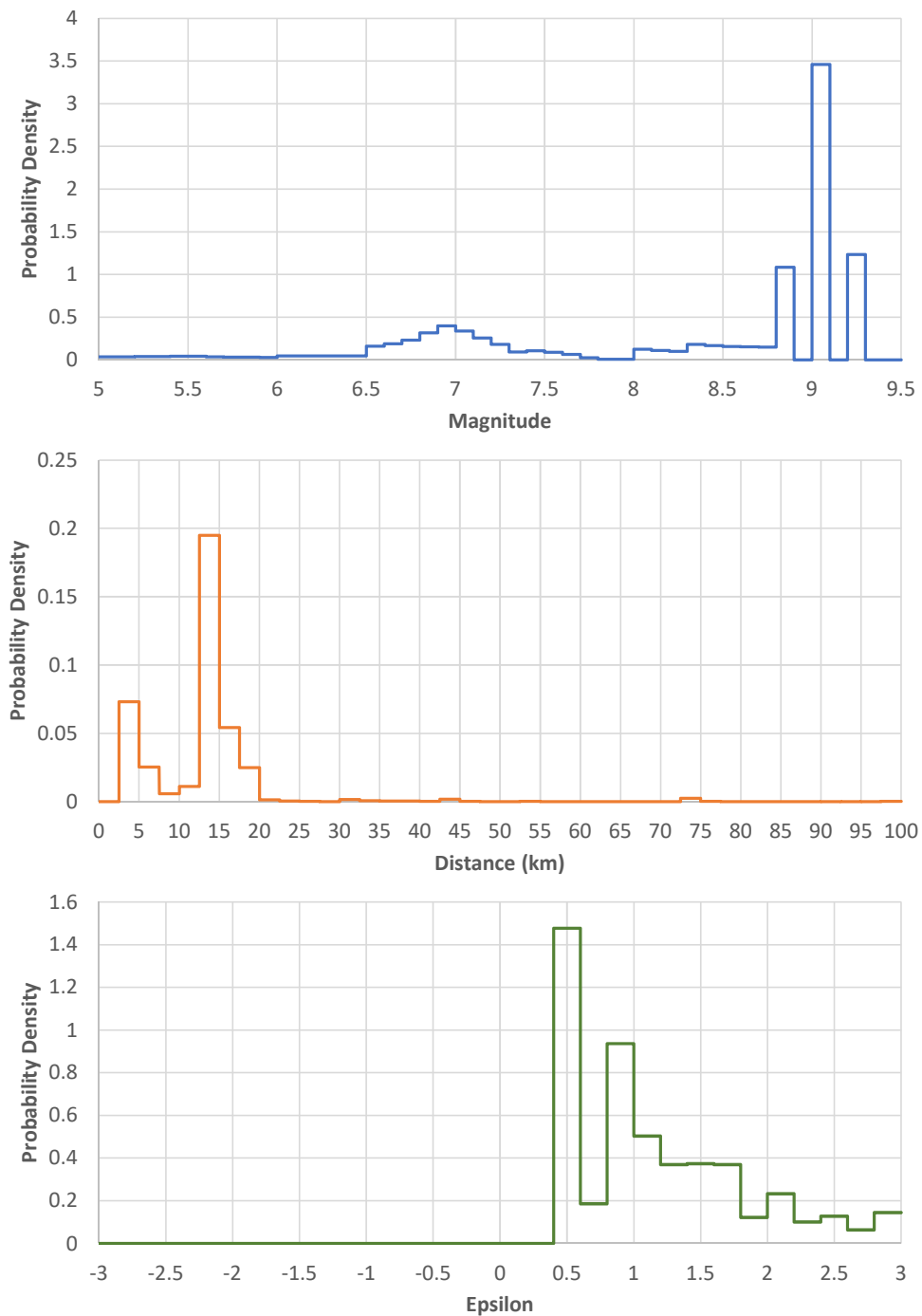
**CALCULATED SHEAR WAVE VELOCITY PROFILES
LOCATIONS B-1 AND B-7**

FIGURE D-1



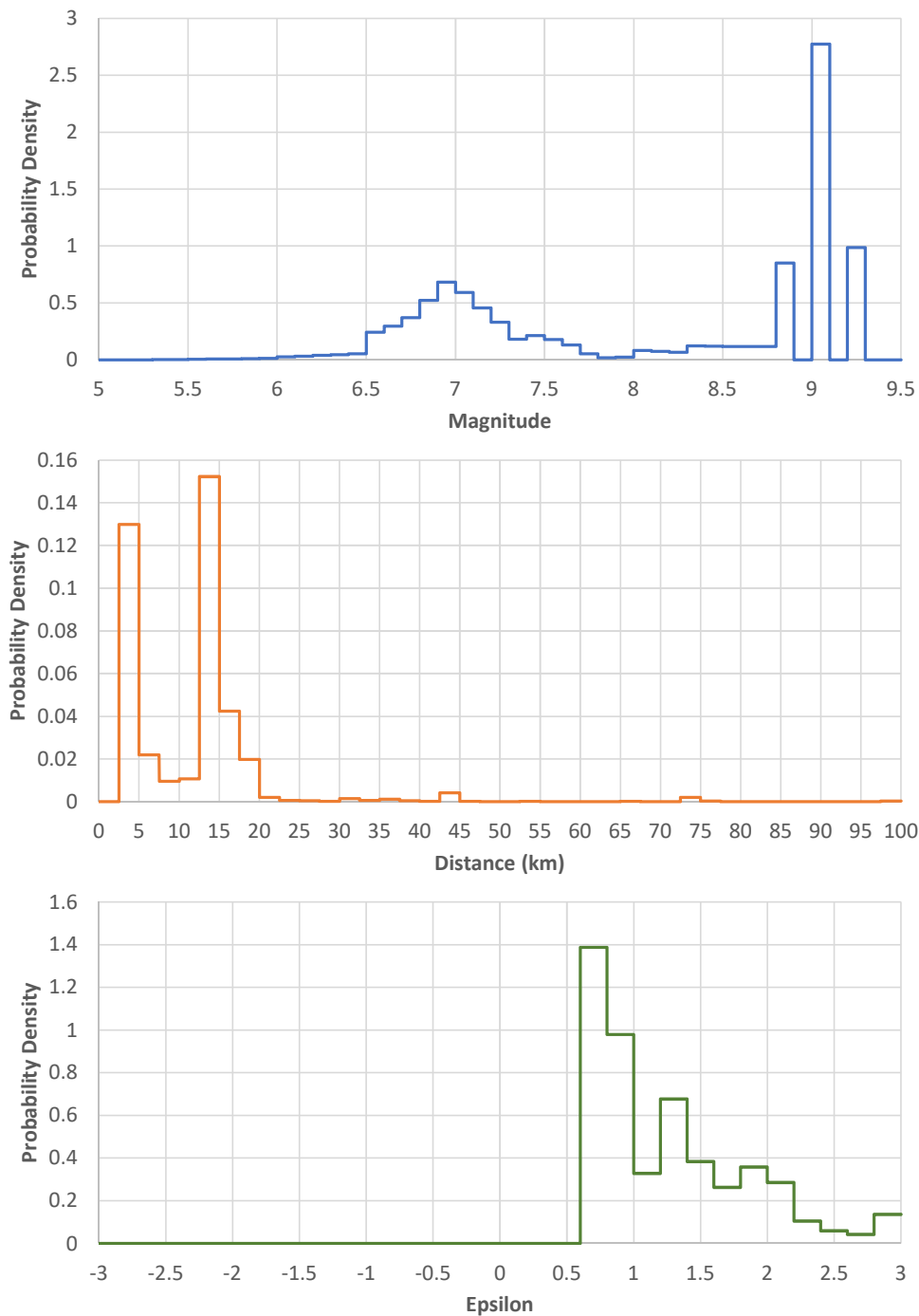
MEAN ANNUAL HAZARD CURVES FOR SELECTED STRUCTURAL PERIODS
Vs30 of 230 M/SEC

FIGURE D-2



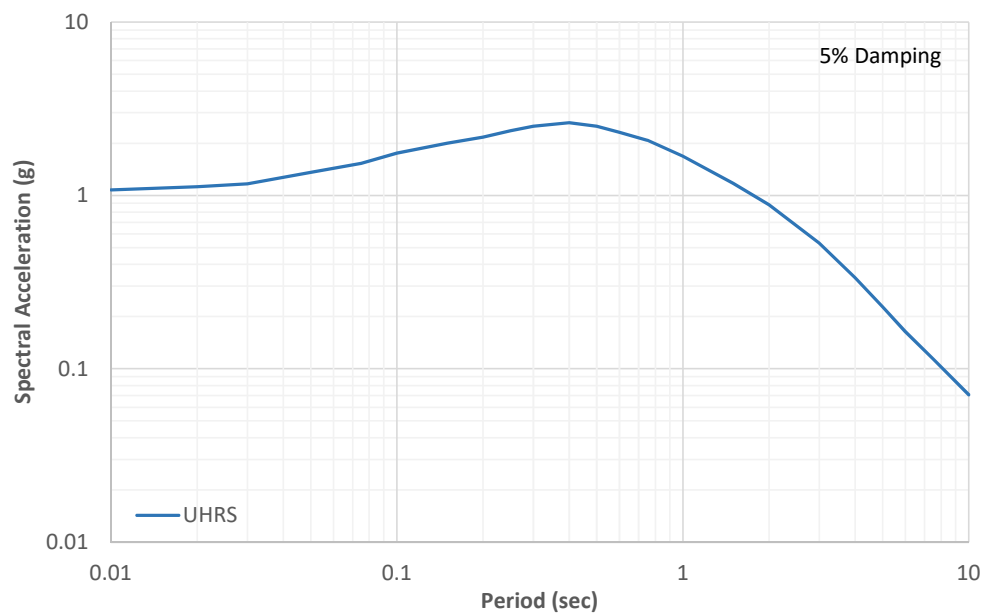
**SEISMIC HAZARD DEAGGREGATION PER MAGNITUDE, DISTANCE, AND EPSILON
FOR PGA AND RETURN PERIOD OF 2,475 YEARS
Vs30 of 230 M/SEC**

FIGURE D-3a



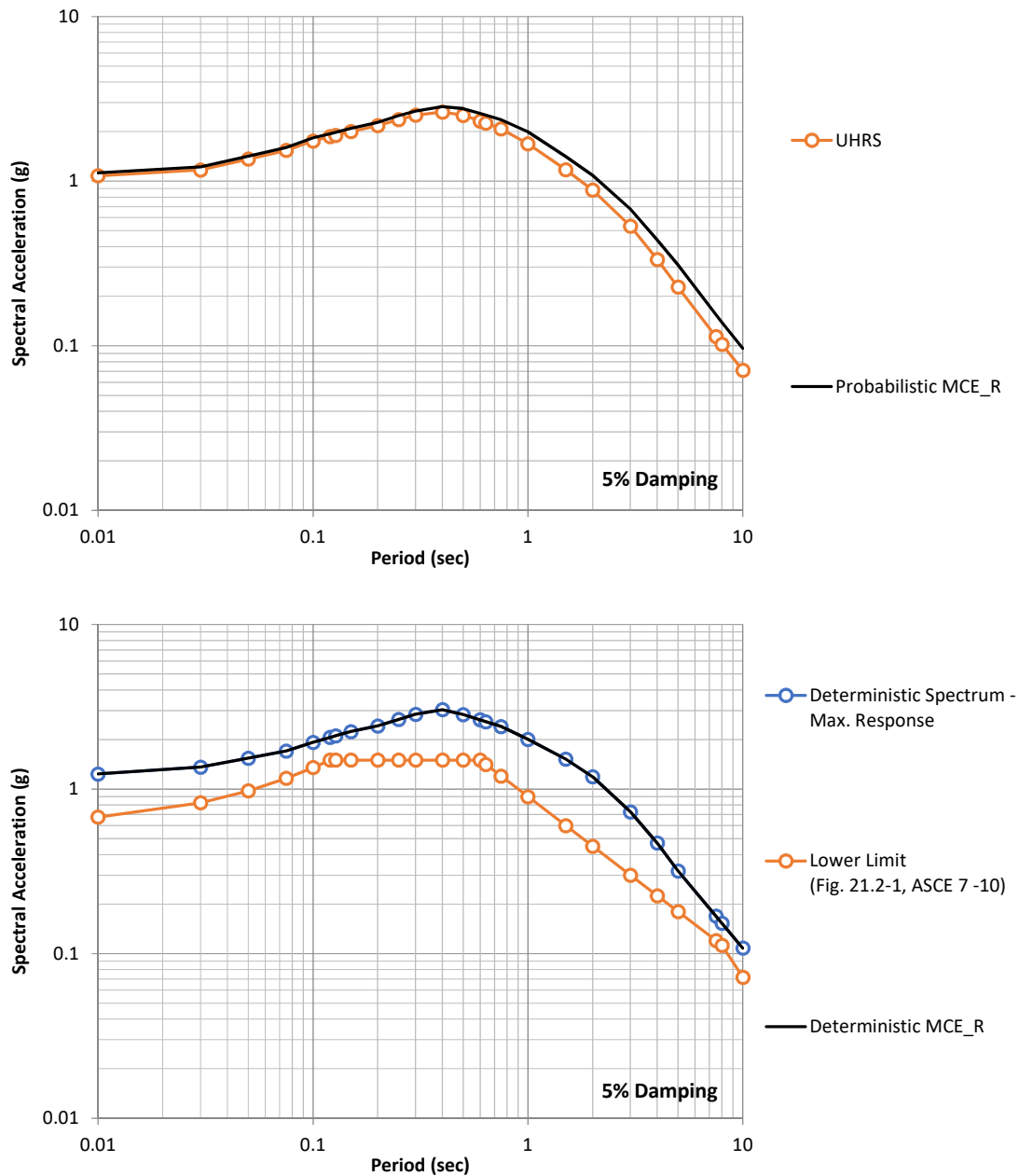
**SEISMIC HAZARD DEAGGREGATION PER MAGNITUDE, DISTANCE, AND EPSILON
 FOR SA AT 1 SEC AND RETURN PERIOD OF 2,475 YEARS
 Vs30 of 230 M/SEC**

FIGURE D-3b



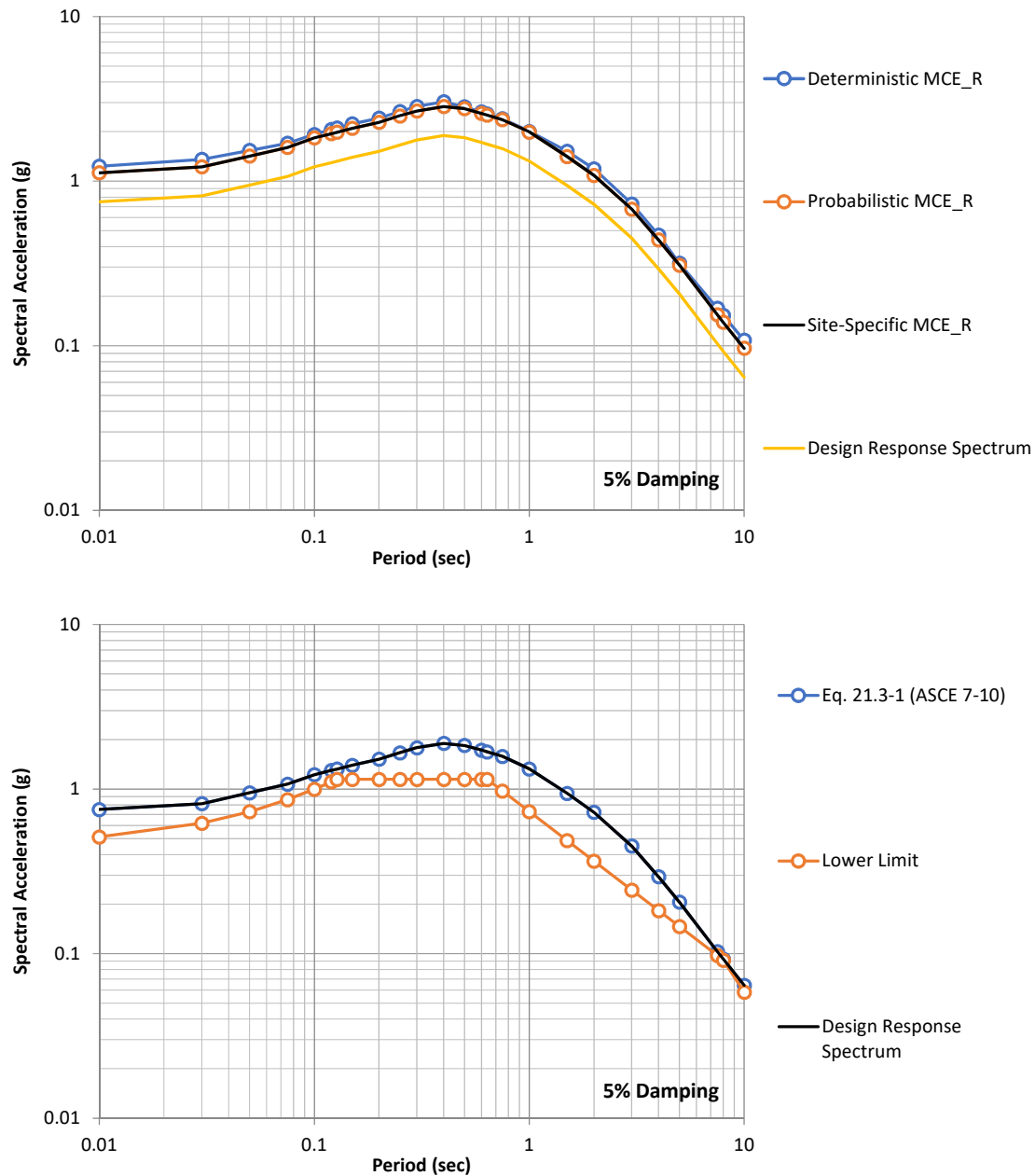
**MEAN HORIZONTAL UNIFORM HAZARD RESPONSE SPECTRUM
RETURN PERIOD OF 2,475 YEARS
Vs30 of 230 M/SEC**

FIGURE D-4



**CALCULATION OF THE PROBABILISTIC AND DETERMINISTIC
HORIZONTAL MCE_R RESPONSE SPECTRA PER ASCE 7-10
Vs30 OF 230 M/SEC**

FIGURE D-5



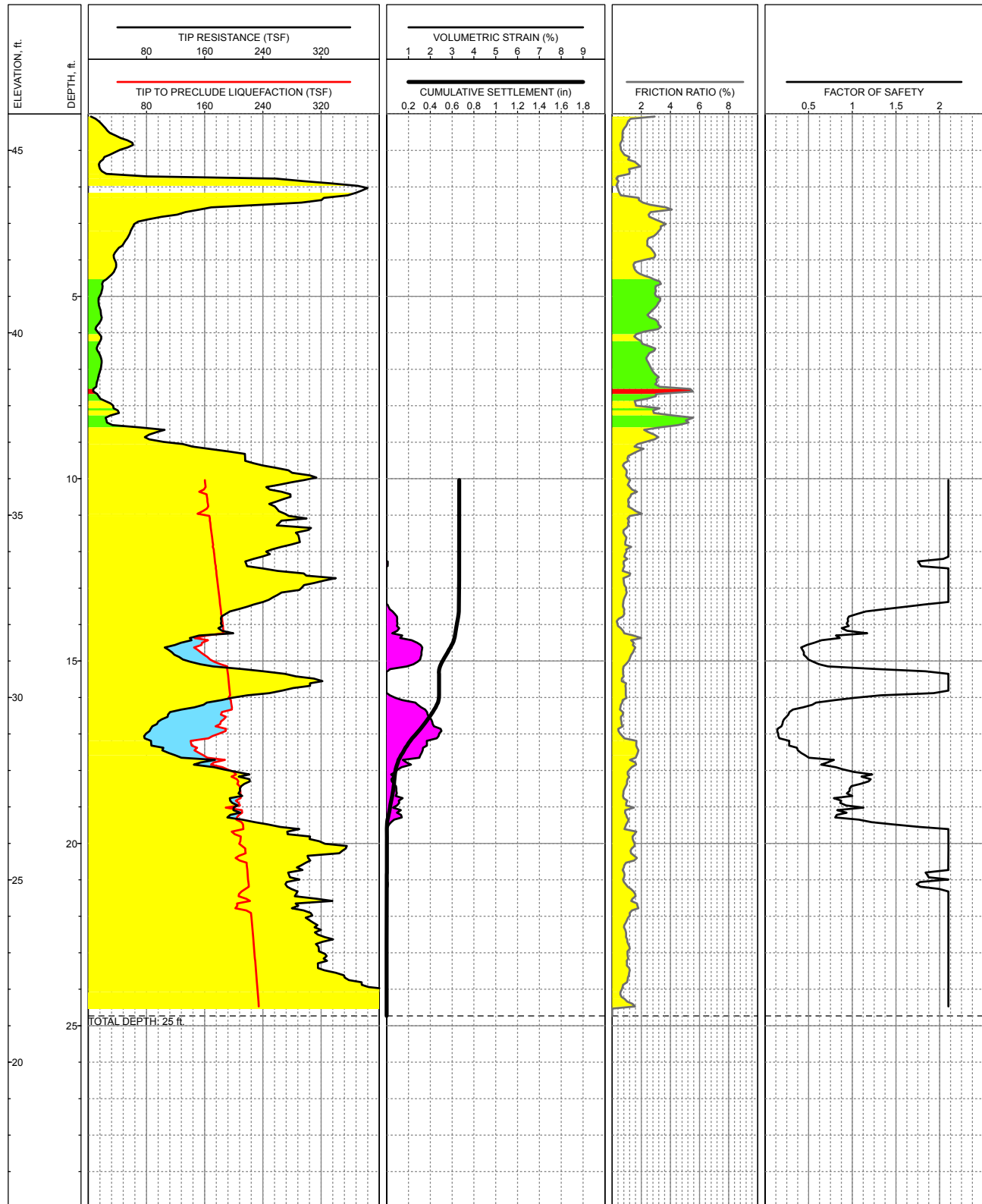
**DEVELOPMENT OF SITE SPECIFIC HORIZONTAL MCE_R
AND DESIGN RESPONSE SPECTRA PER ASCE 7-10
Vs30 OF 230 M/SEC**

FIGURE D-6



APPENDIX E

LIQUEFACTION SETTLEMENT ANALYSES

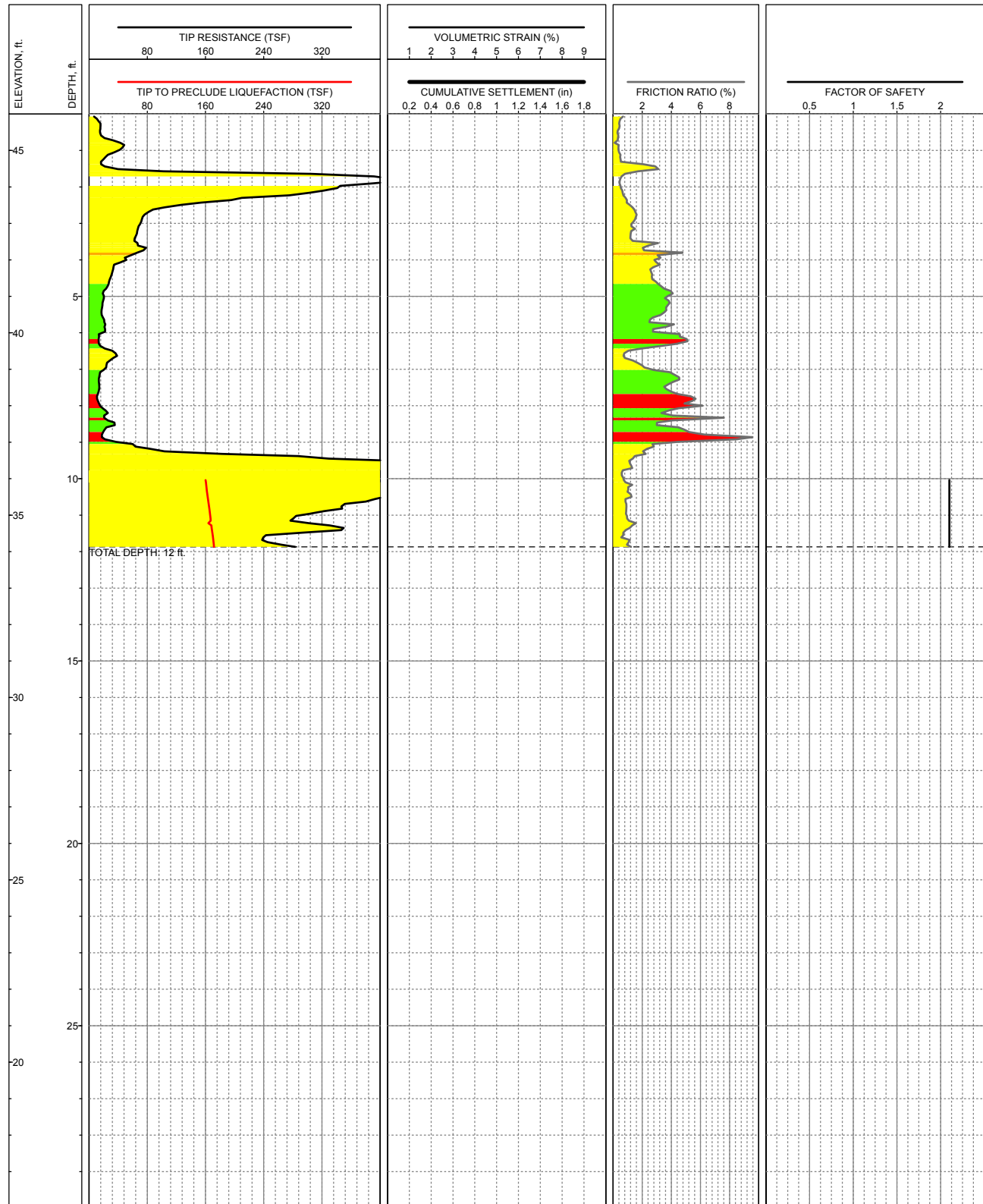


LOCATION: E5,963,282, N2,708,981, CA State Plane, Zone 2, NAD83, Ft
SURFACE EL: 46.0ft +/- (-)
COMPLETION DEPTH: 25ft
TESTDATE: 5/8/2019

EXPLORATION METHOD: CPT
WATER DEPTH: 10 ft
CONE AREA RATIO: 0.59

LOG OF CPT-01
M= 8.2, PGA = 1.08 g, NCEER

PLATE E-1a

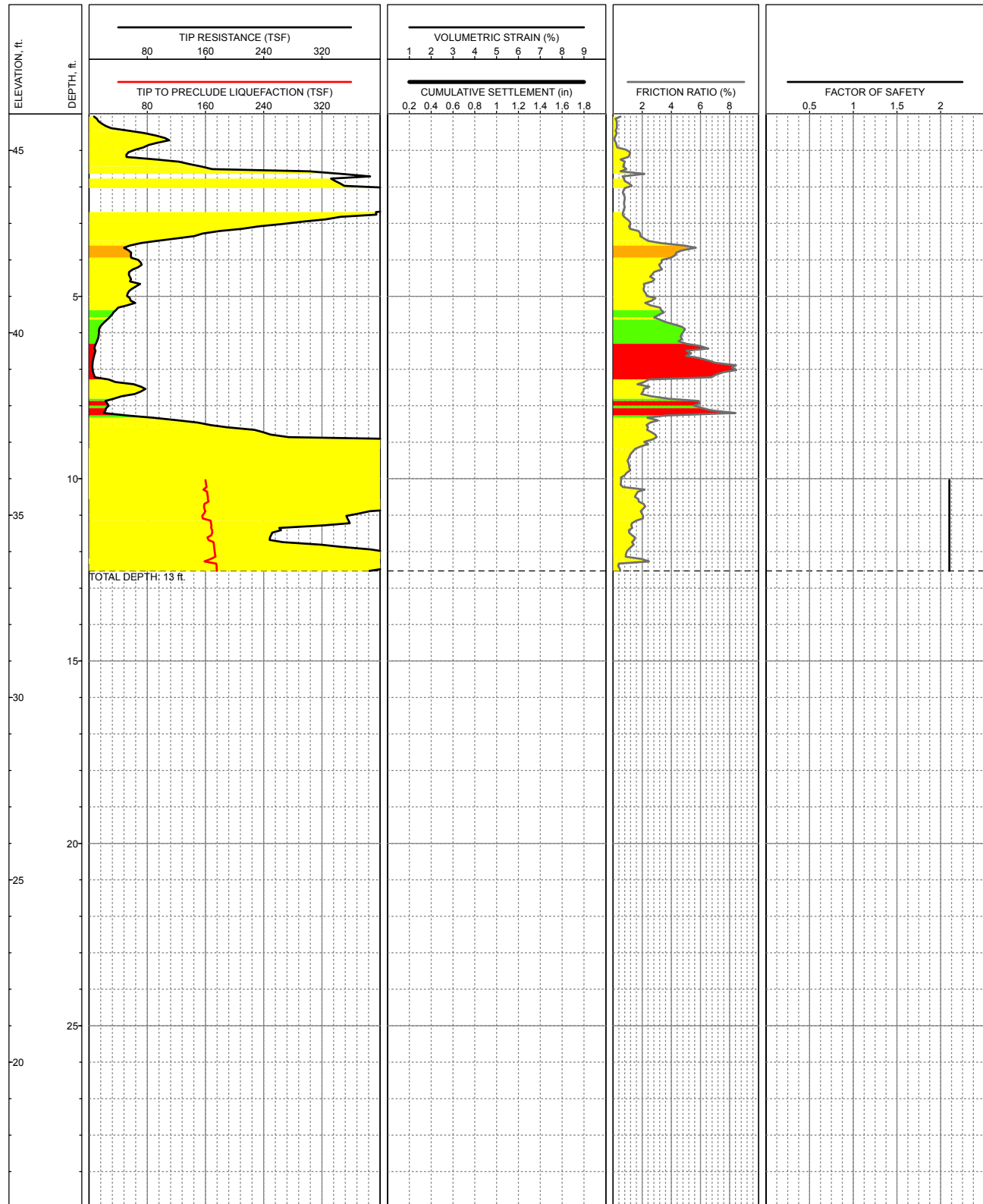


LOCATION: E5,963,282, N2,708,981, CA State Plane, Zone 2, NAD83, Ft
SURFACE EL: 46.0ft +/- (-)
COMPLETION DEPTH: 12ft
TESTDATE: 5/8/2019

EXPLORATION METHOD: CPT
WATER DEPTH: 10 ft
CONE AREA RATIO: 0.59

LOG OF CPT-01A
M= 8.2, PGA = 1.08 g, NCEER

PLATE E-1b

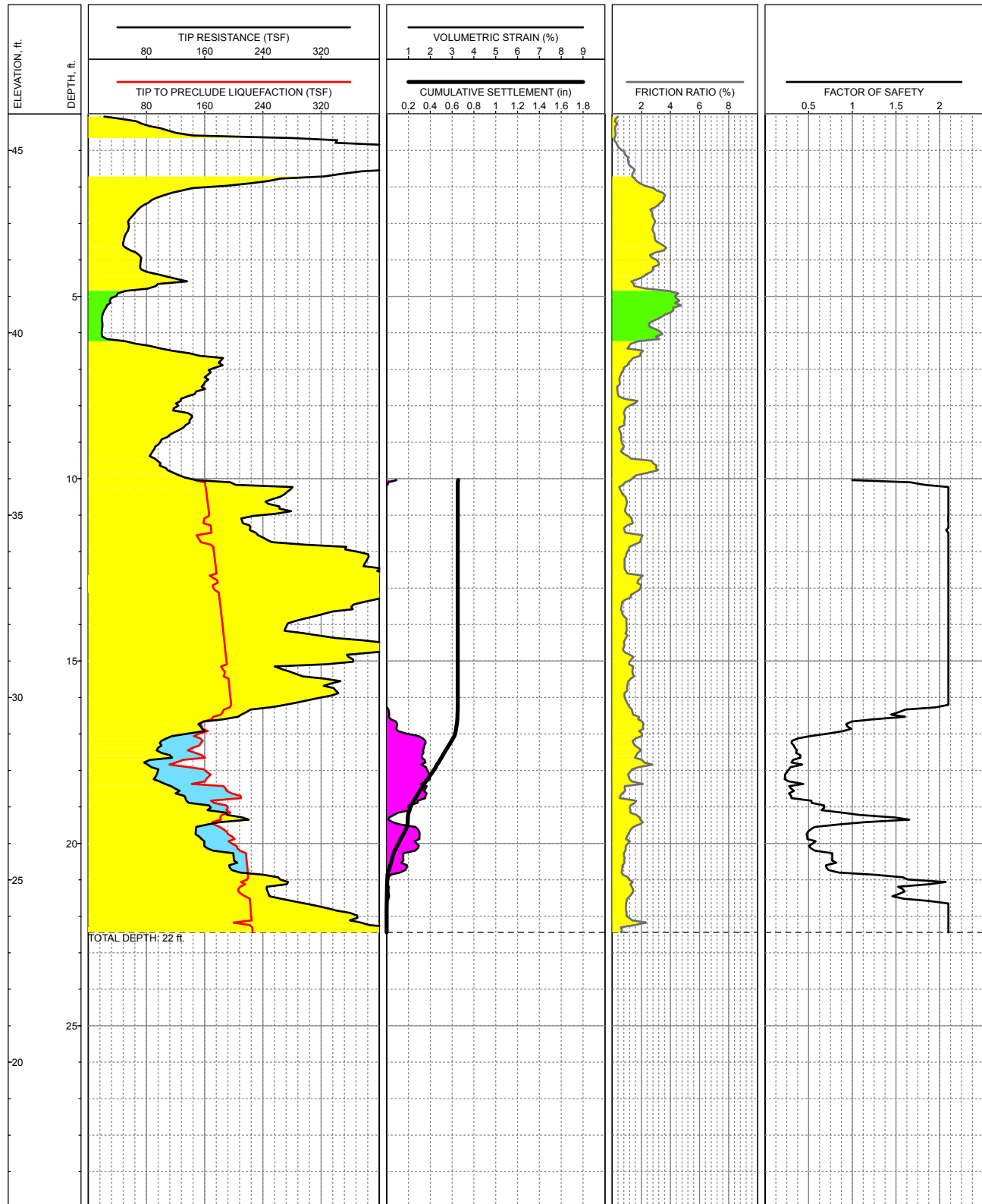


LOCATION: E5,963,282, N2,708,981, CA State Plane, Zone 2, NAD83, Ft
SURFACE EL: 46.0ft +/- (
COMPLETION DEPTH: 13ft
TESTDATE: 5/8/2019

EXPLORATION METHOD: CPT
WATER DEPTH: 10 ft
CONE AREA RATIO: 0.59

LOG OF CPT-01B
M= 8.2, PGA = 1.08 g, NCEER

PLATE E-1c



LOCATION: E5,963,372, N2,708,990, CA State Plane, Zone 2, NAD83, Ft
 SURFACE EL: 46.0ft +/- (-)
 COMPLETION DEPTH: 22ft
 TESTDATE: 5/8/2019

EXPLORATION METHOD: CPT
 WATER DEPTH: 10 ft
 CONE AREA RATIO: 0.59

LOG OF CPT-02
 M= 8.2, PGA = 1.08 g, NCEER

PLATE E-1d

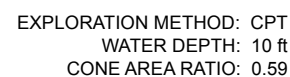
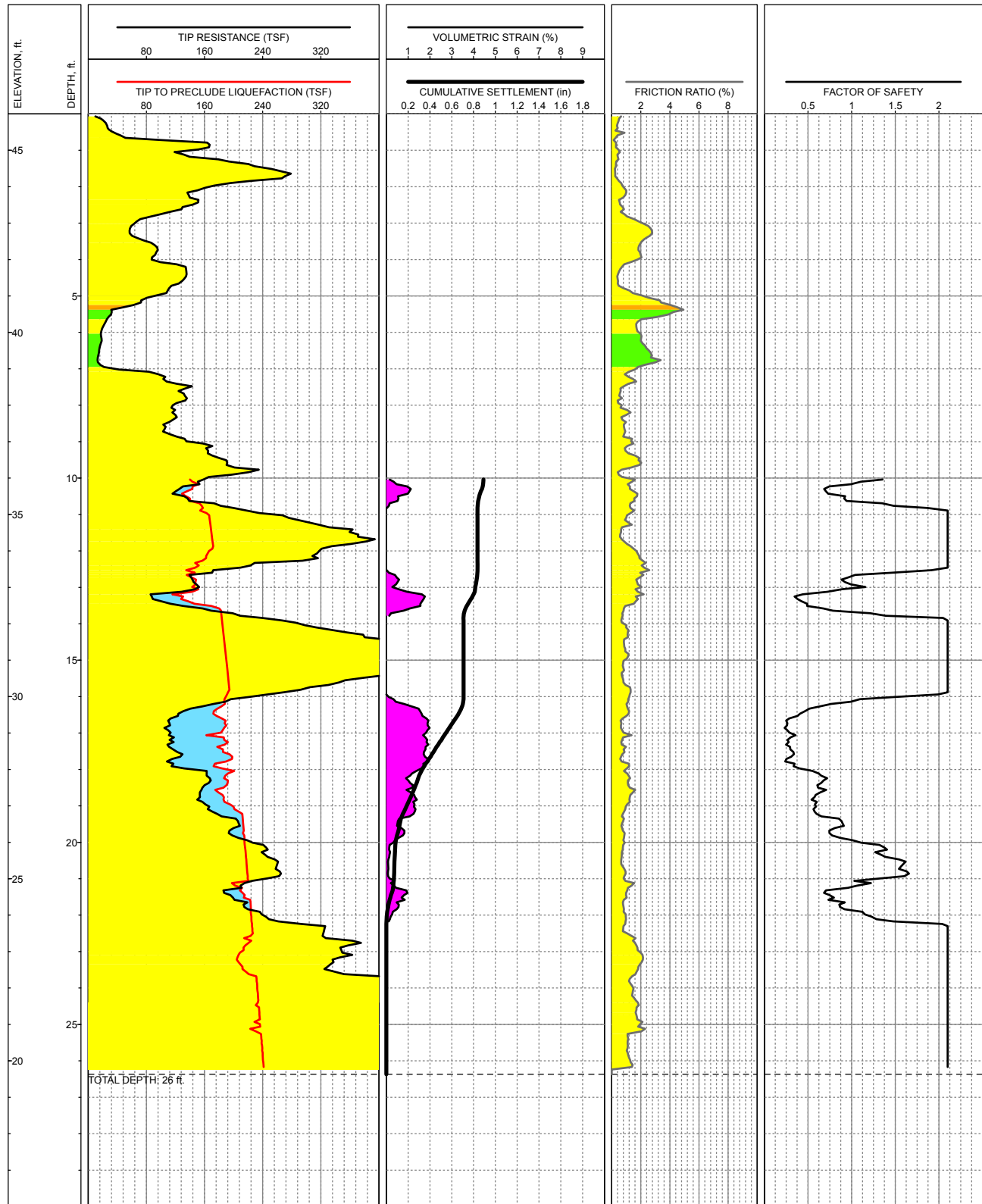


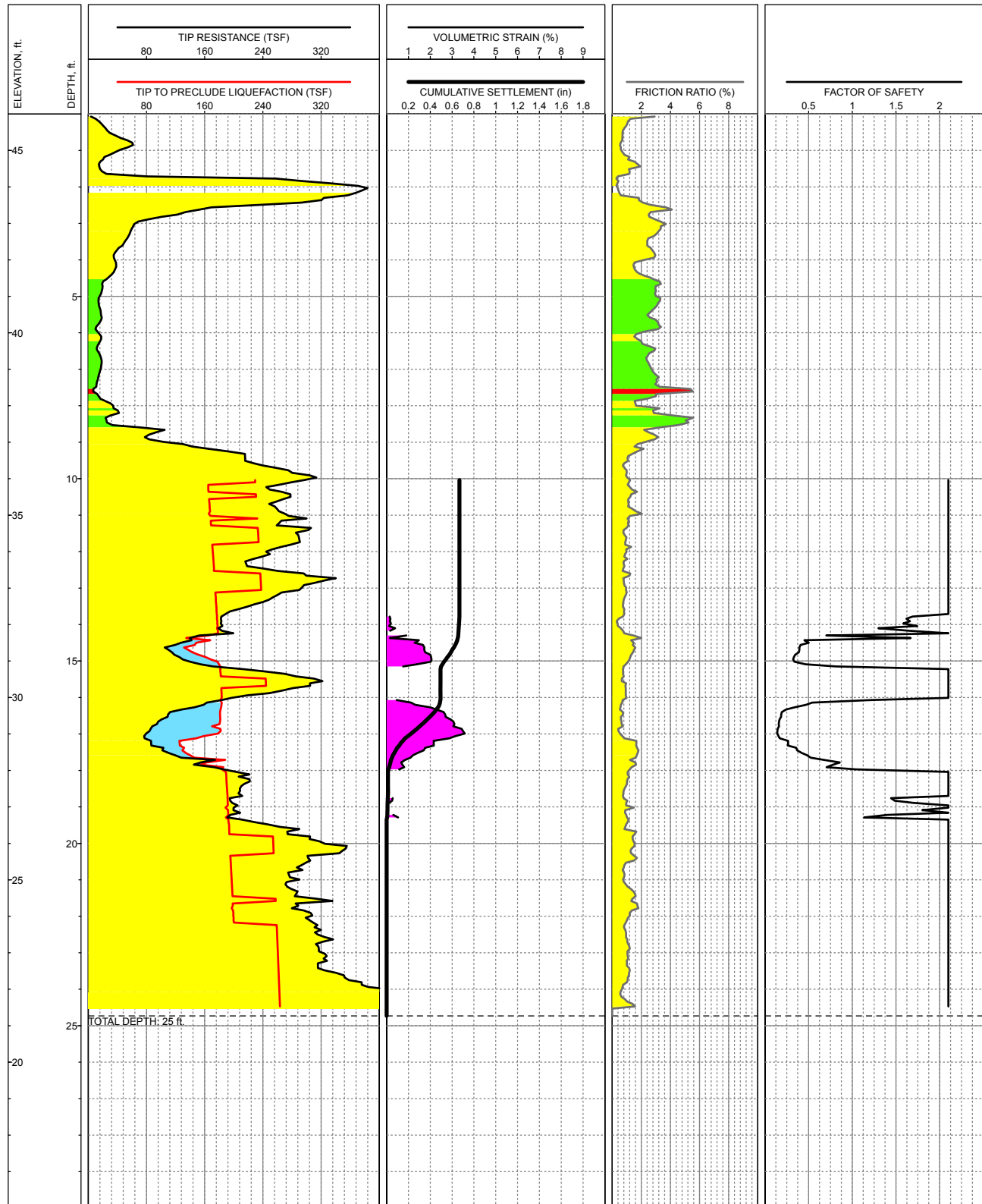
PLATE E-1e



LOCATION: E5,963,372, N2,708,990, CA State Plane, Zone 2, NAD83, Ft
 SURFACE EL: 46.0ft +/- (-)
 COMPLETION DEPTH: 26ft
 TESTDATE: 5/8/2019

EXPLORATION METHOD: CPT
 WATER DEPTH: 10 ft
 CONE AREA RATIO: 0.59

LOG OF CPT-02B
 M= 8.2, PGA = 1.08 g, NCEER



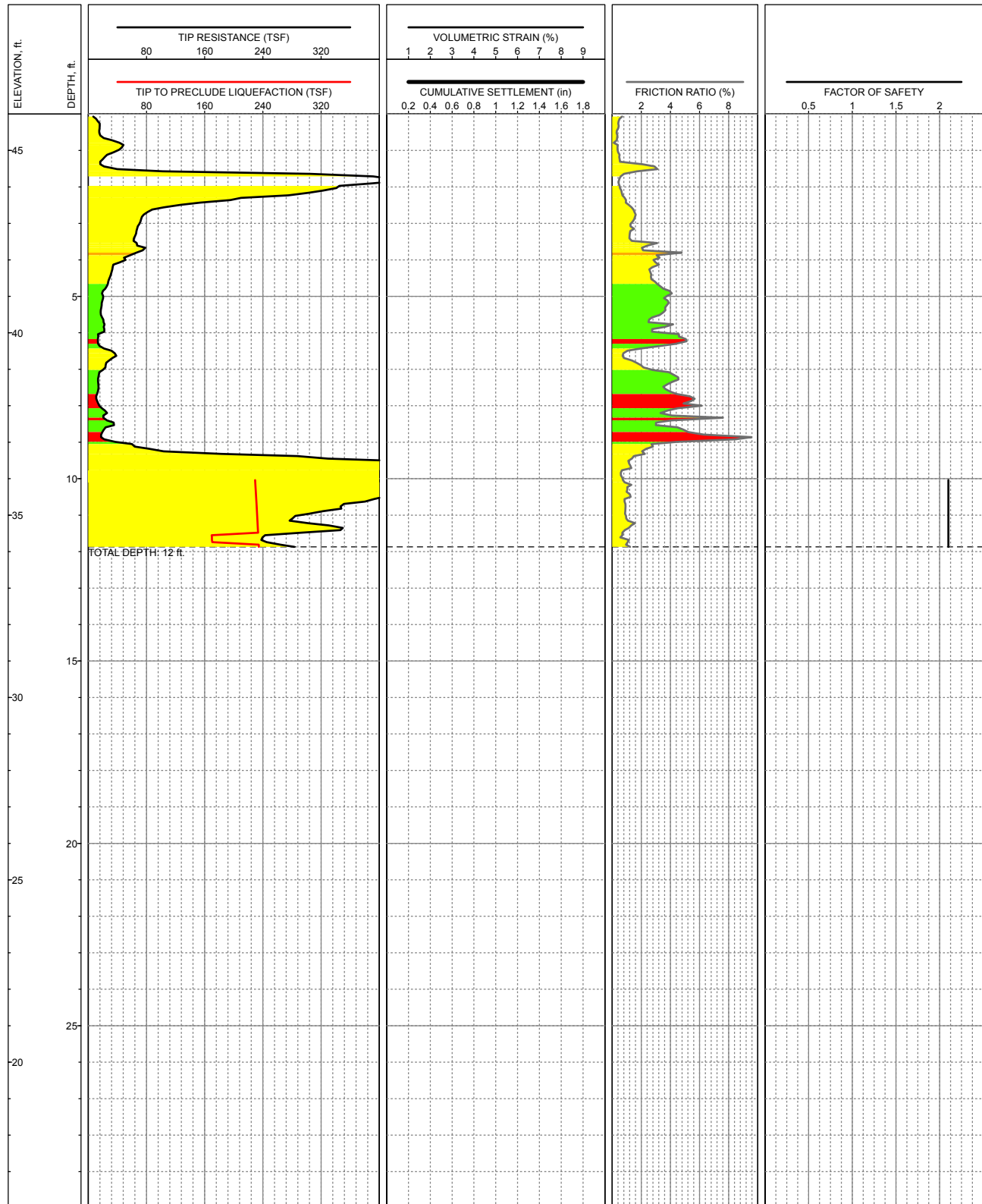
LOCATION: E5,963,282, N2,708,981, CA State Plane, Zone 2, NAD83, Ft
SURFACE EL: 46.0ft +/- (-)
COMPLETION DEPTH: 25ft
TESTDATE: 5/8/2019

EXPLORATION METHOD: CPT
WATER DEPTH: 10 ft
CONE AREA RATIO: 0.59

LOG OF CPT-01

M= 8.2, PGA = 1.08 g, Boulanger and Idriss (2014)

PLATE E-2a



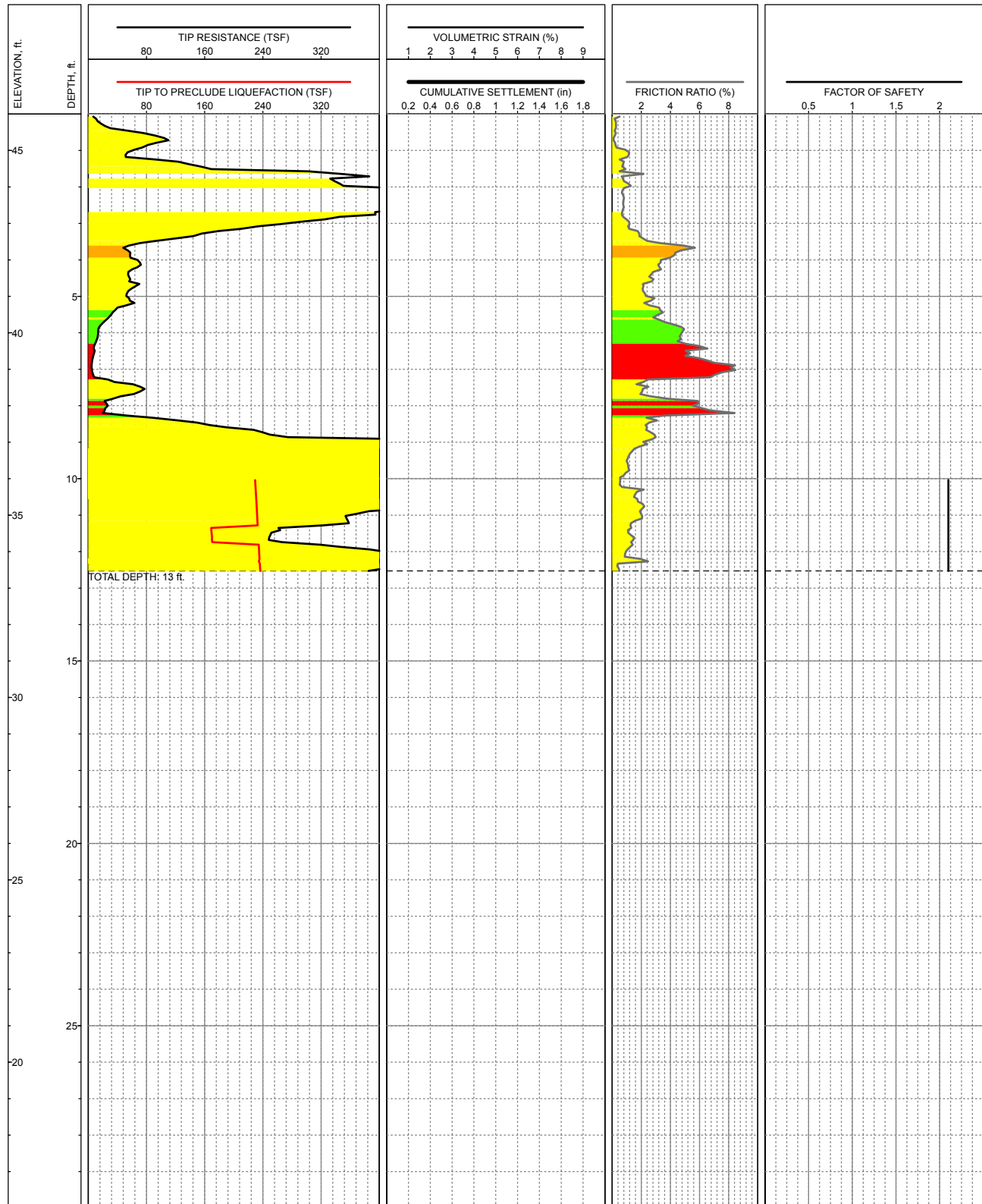
LOCATION: E5,963,282, N2,708,981, CA State Plane, Zone 2, NAD83, Ft
SURFACE EL: 46.0ft +/- (-)
COMPLETION DEPTH: 12ft
TESTDATE: 5/8/2019

EXPLORATION METHOD: CPT
WATER DEPTH: 10 ft
CONE AREA RATIO: 0.59

LOG OF CPT-01A

M= 8.2, PGA = 1.08 g, Boulanger and Idriss (2014)

PLATE E-2b



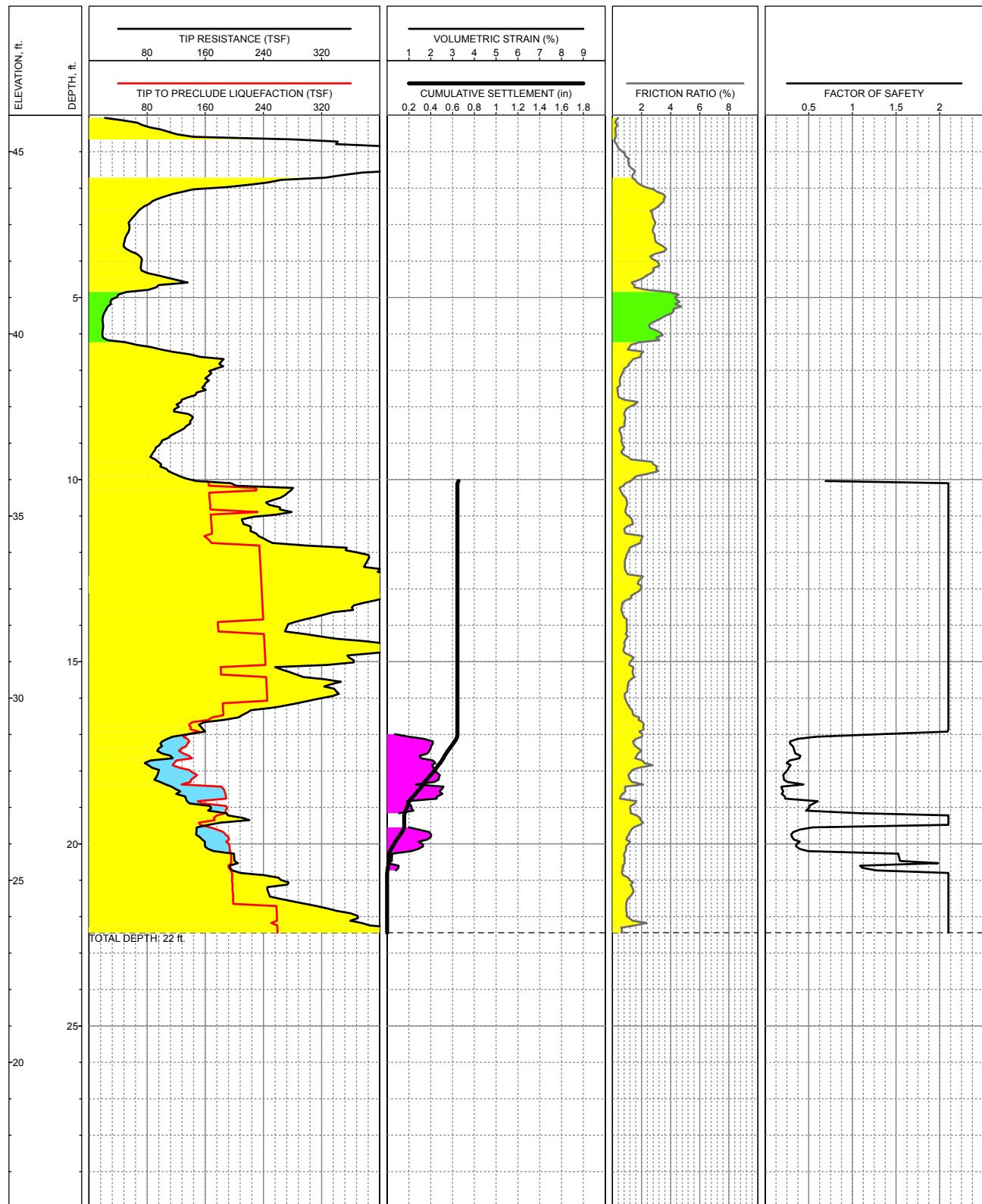
LOCATION: E5,963,282, N2,708,981, CA State Plane, Zone 2, NAD83, Ft
SURFACE EL: 46.0ft +/- ()
COMPLETION DEPTH: 13ft
TESTDATE: 5/8/2019

EXPLORATION METHOD: CPT
WATER DEPTH: 10 ft
CONE AREA RATIO: 0.59

LOG OF CPT-01B

M= 8.2, PGA = 1.08 g, Boulanger and Idriss (2014)

PLATE E-2c



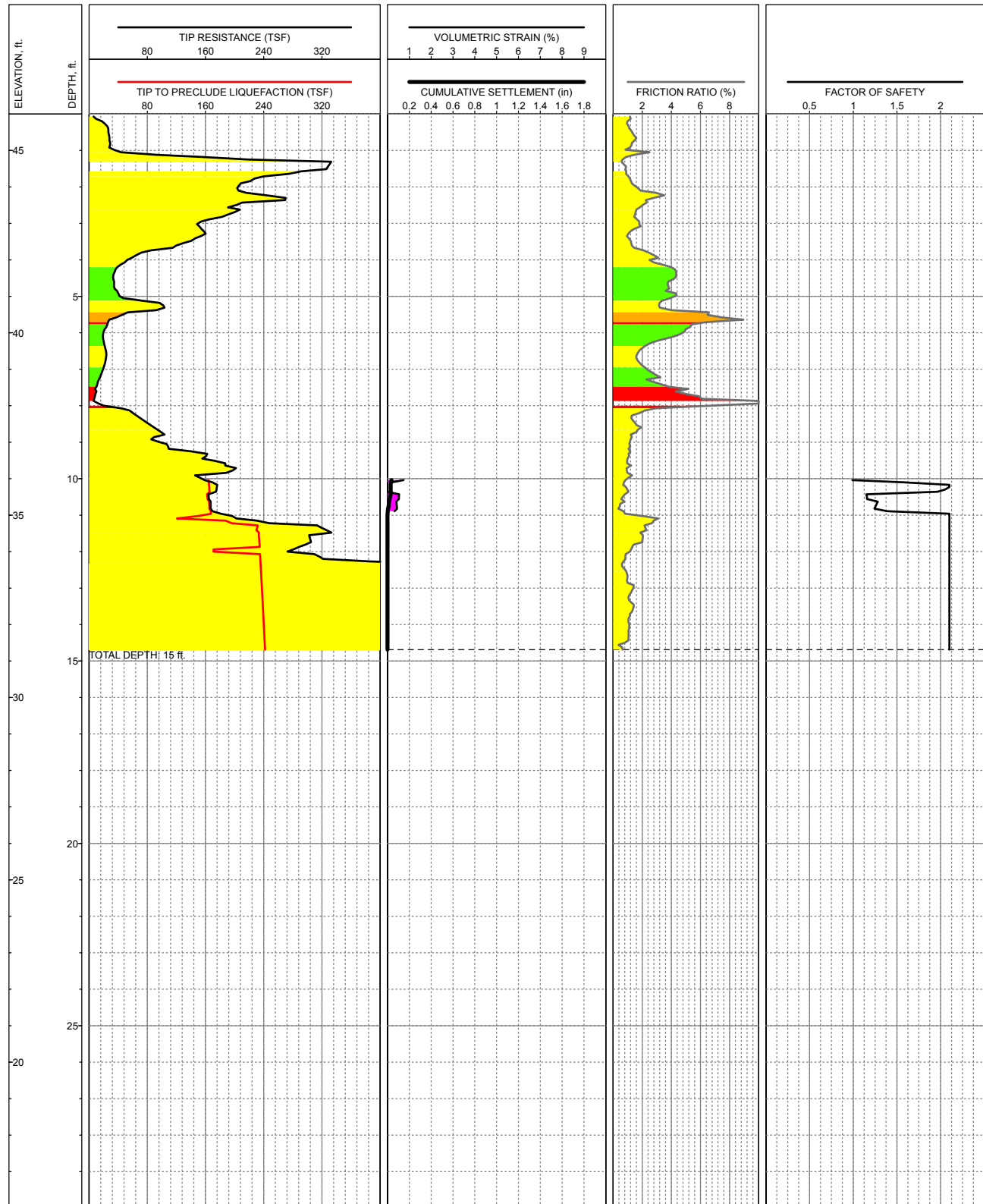
LOCATION: E5,963,372, N2,708,990, CA State Plane, Zone 2, NAD83, Ft
SURFACE EL: 46.0ft +/- (-)
COMPLETION DEPTH: 22ft
TESTDATE: 5/8/2019

EXPLORATION METHOD: CPT
WATER DEPTH: 10 ft
CONE AREA RATIO: 0.59

LOG OF CPT-02

M= 8.2, PGA = 1.08 g, Boulanger and Idriss (2014)

PLATE E-2d



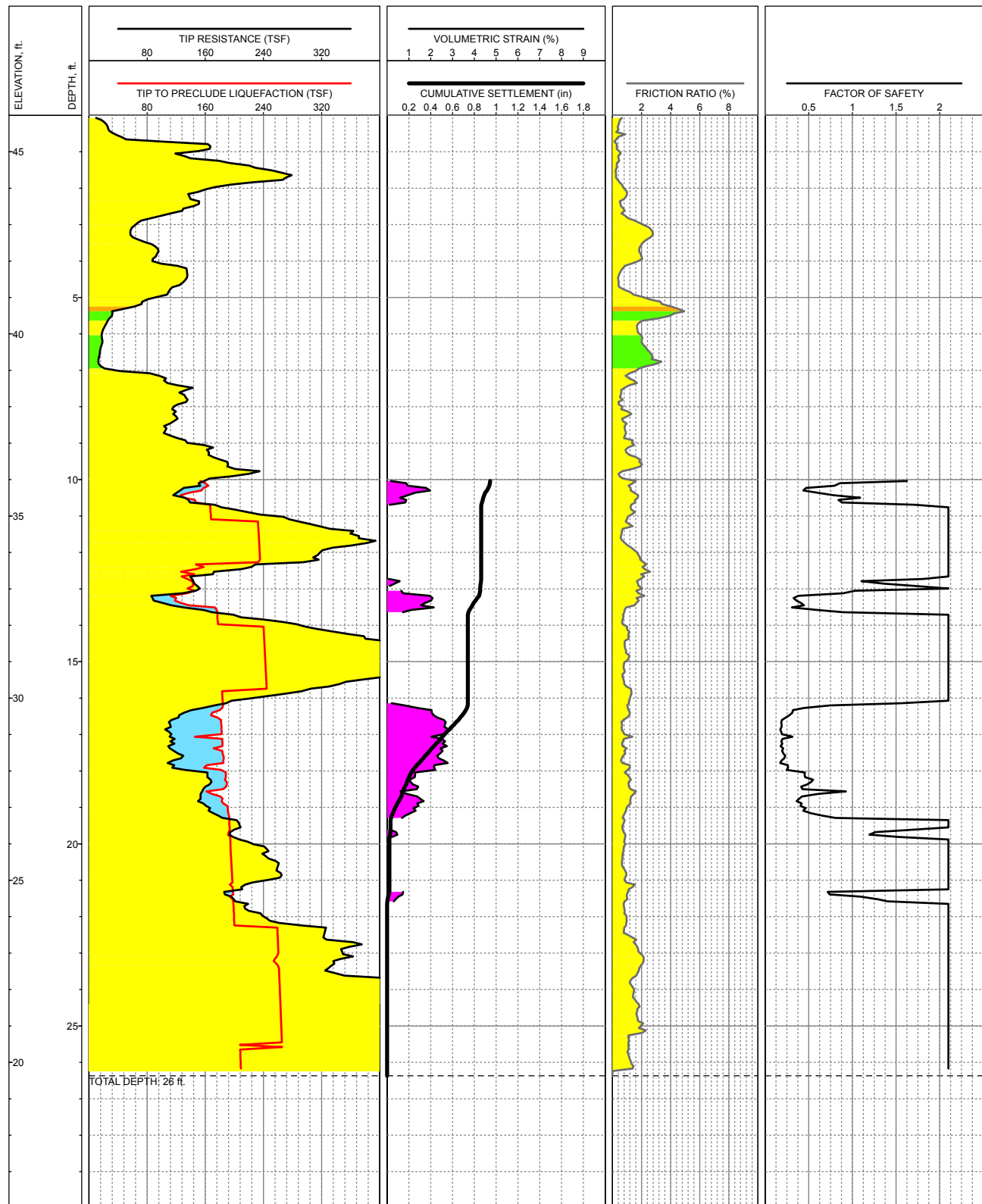
LOCATION: E5,963,372, N2,708,990, CA State Plane, Zone 2, NAD83, Ft
SURFACE EL: 46.0ft +/- ()
COMPLETION DEPTH: 15ft
TESTDATE: 5/8/2019

EXPLORATION METHOD: CPT
WATER DEPTH: 10 ft
CONE AREA RATIO: 0.59

LOG OF CPT-02A

M= 8.2, PGA = 1.08 g, Boulanger and Idriss (2014)

PLATE E-2e



LOCATION: E5,963,372, N2,708,990, CA State Plane, Zone 2, NAD83, Ft
SURFACE EL: 46.0ft +/- (-)
COMPLETION DEPTH: 26ft
TESTDATE: 5/8/2019

EXPLORATION METHOD: CPT
WATER DEPTH: 10 ft
CONE AREA RATIO: 0.59

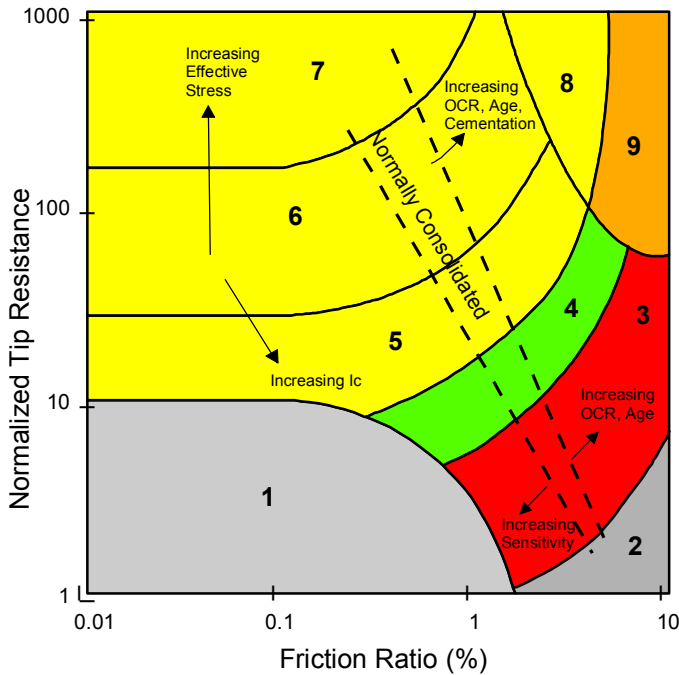
LOG OF CPT-02B

M= 8.2, PGA = 1.08 g, Boulanger and Idriss (2014)

PLATE E-2f



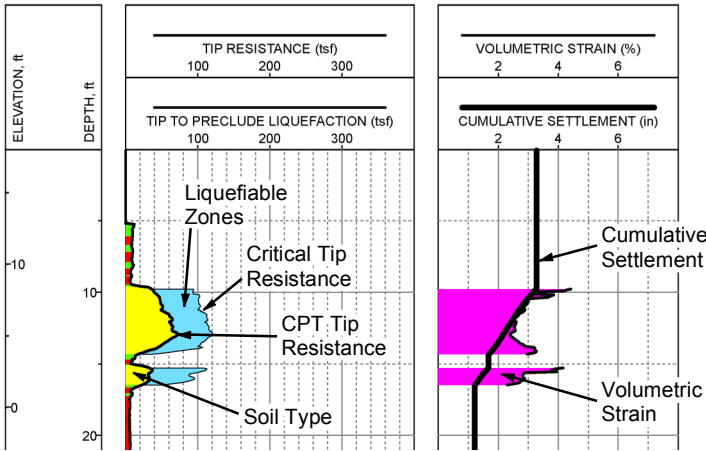
CPT CORRELATION CHART
(Robertson 1990)



Zone	Soil Behavior Type
1	Sensitive Fine-grained
2	Peats
3	Silty Clay to Clay
4	Clayey Silt to Silty Clay
5	Silty Sand to Sandy Silt
6	Clean Sand to Silty Sand
7	Gravelly Sand to Dense Sand
8	Very Stiff Sand to Clayey Sand*
9	Very Stiff Fine-Grained*

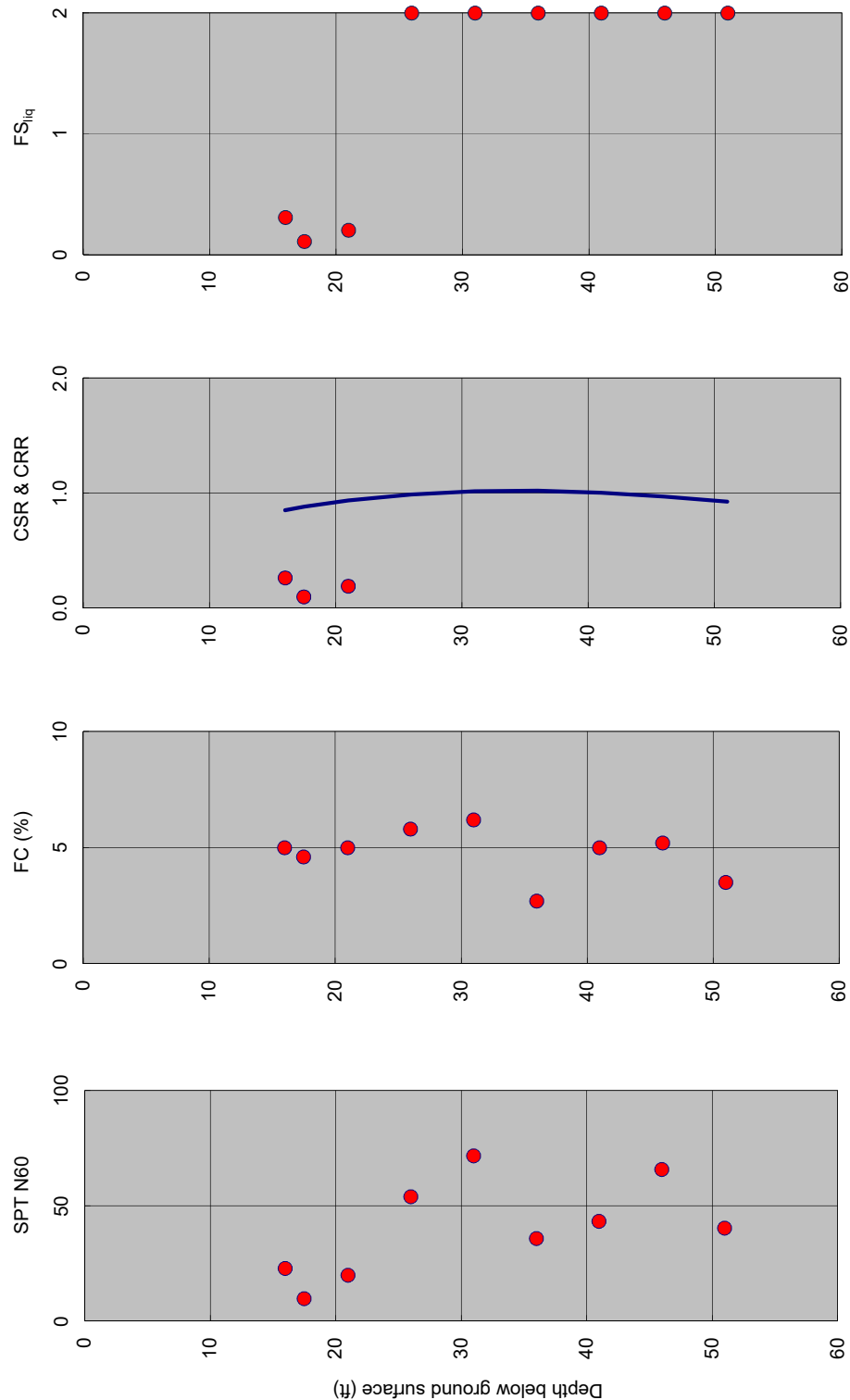
*heavily overconsolidated or cemented

CPT LOG COMPONENTS



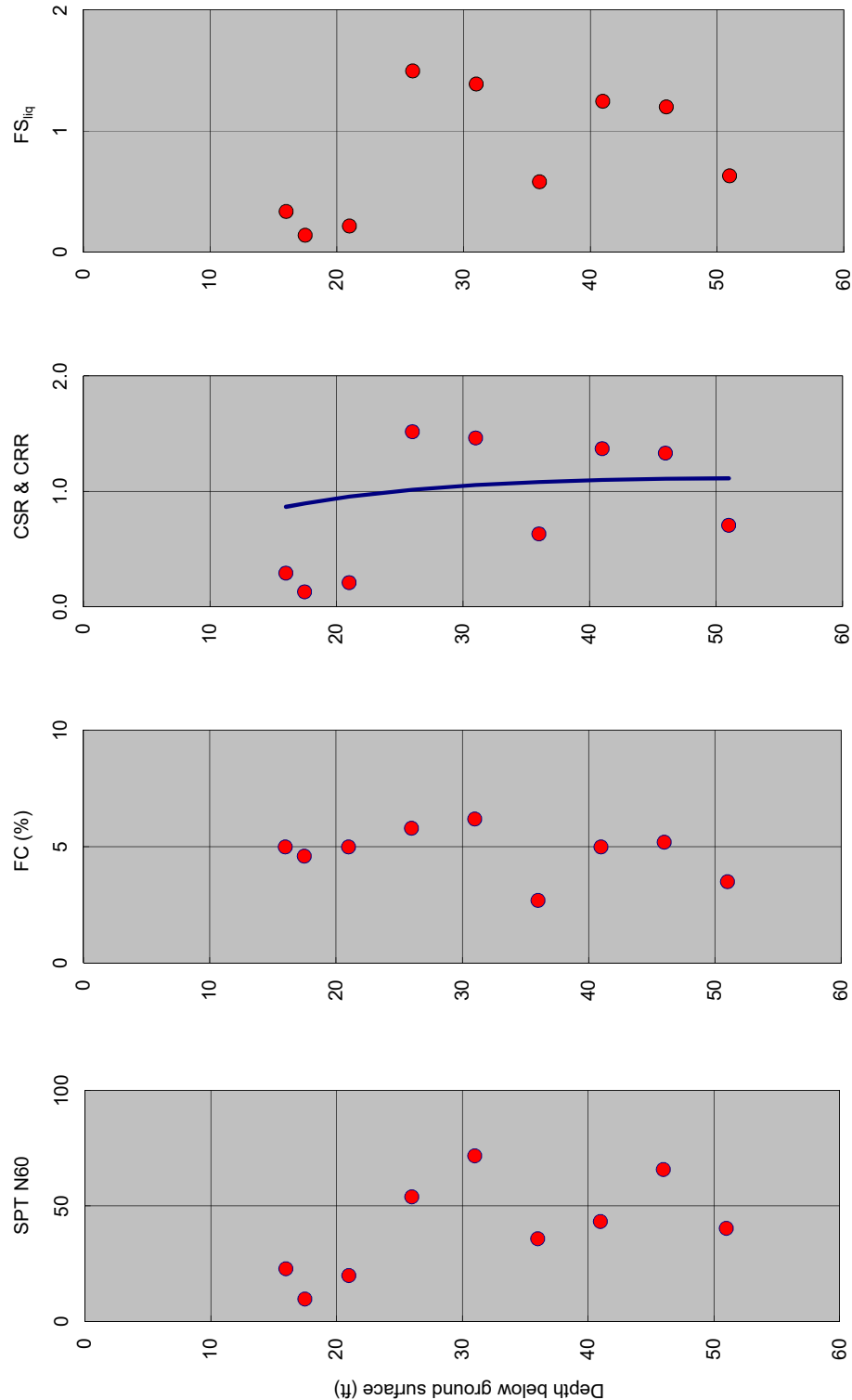
KEY TO LIQUEFACTION LOGS

FIGURE E-3



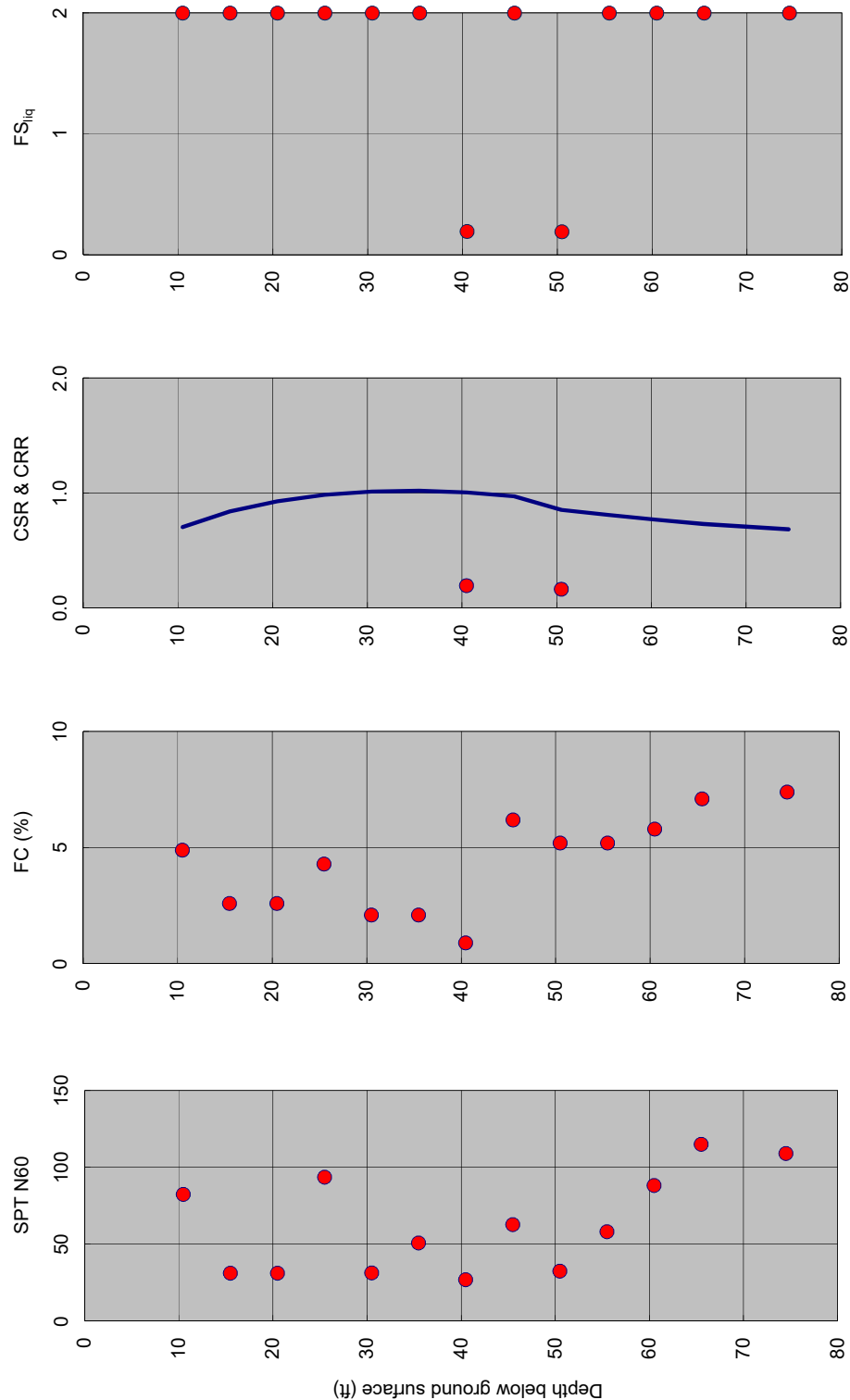
**SPT-BASED LIQUEFACTION TRIGGERING ANALYSIS
 BORING B-1
 NCEER (YOUDE ET AL., 2001)**

FIGURE E-4a



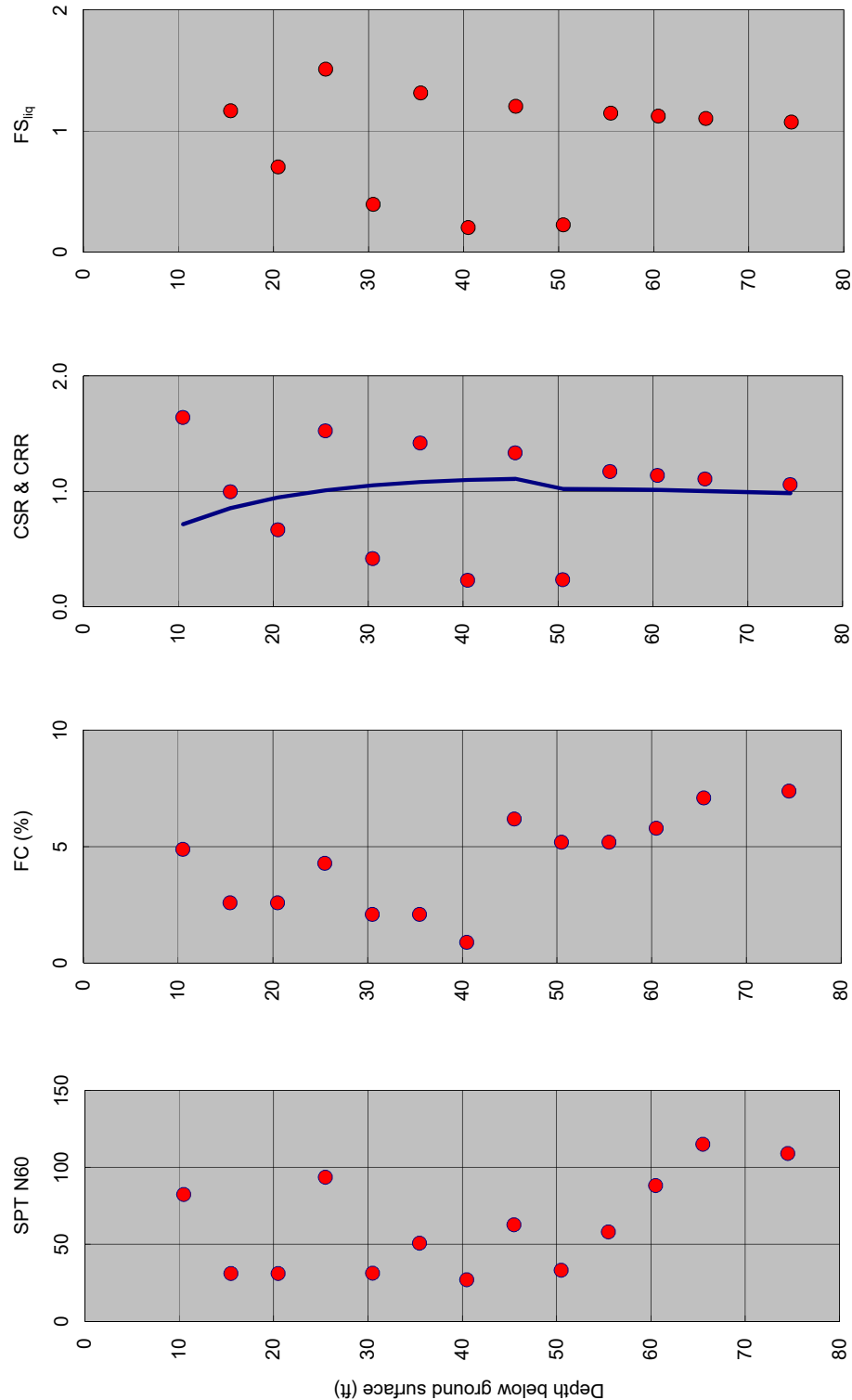
**SPT-BASED LIQUEFACTION TRIGGERING ANALYSIS
 BORING B-1
 BOULANGER AND IDRIS (2014)**

FIGURE E-4b



**SPT-BASED LIQUEFACTION TRIGGERING ANALYSIS
 BORING B-7
 NCEER (YOUDE ET AL., 2001)**

FIGURE E-5a



**SPT-BASED LIQUEFACTION TRIGGERING ANALYSIS
BORING B-7
BOULANGER AND IDRIS (2014)**

FIGURE E-5b