Appendix C

Geotechnical Analysis

Geotechnical Engineering Report
Southern California Gas Company
New Office Building
8101 Rosemead Boulevard
Pico Rivera, California

Campos EPC
2400 Katella Suite 600
Anaheim, CA 92806
1-855-CAMPOS1 (226-7671)

Prepared for: Southern California Gas Company
8101 Rosemead Blvd
Pico Rivera, CA 90660

Campos EPC Project Number: 00037.0000.0023

Date: January 24, 2022

CAMPOS | EPC

Prepared for:
Mr. Chris Parsons
Southern California Gas Company
8101 Rosemead Blvd
Pico Rivera, CA 90660

Geotechnical Engineering Report
Southern California Gas Company
New Office Building
8101 Rosemead Boulevard
Pico Rivera, California

Eric Backlund, PE (PA)
Senior Geotechnical Engineer

Romeo Shiplee, PE
Engineering Manager

REGISTERED PROFESSIONAL ENGINEER
ROMEO R. SHIPLEE
No. 81127
Exp. 9/30/2023
CIVIL
STATE OF CALIFORNIA

Campos EPC
2400 Katella Suite 600
Anaheim, CA 92806
1-855-CAMPOS1 (226-7671)

Campos Project Number: 00037.0000.0023

Date: January 24, 2022

Table of Contents

Figures

Tables

Appendices

1. Introduction

1.1 General

Campos EPC (Campos) is pleased to provide this report for geotechnical engineering services performed on the proposed New Office Building in Pico Rivera, California. The approximate location of the site is shown in Figure 1, Site Location Plan. This report should be read in its entirety and the limitations of the report are provided in Section 6.

1.2 Project Description

Campos understands that Southern California Gas Company (SoCal Gas) is planning to design, build, and commission a new, two-story, office building with an approximate footprint of 44,000 square feet. The site is located at 8101 Rosemead Boulevard in Pico Rivera, California. The proposed structure is planned within the main parking lot area in the southeastern portion of the facility. Column loads were estimated to be about 450 kips.

The proposed site is an existing parking lot that is relatively level. Therefore, it is assumed that proposed cuts and fills will be less than about two feet.

Our recommendations within this report are based on our current understanding of the proposed project. If details of the project change, Campos should be notified to review our recommendations and evaluate if they need to be modified.

1.3 Scope of Services

Campos's scope of services for geotechnical engineering on this project were provided in our proposal dated October 21, 2021. Our scope of services consisted of reviewing the previously performed subsurface explorations activities, performing engineering analysis, and preparing this report.

2. Site Exploration

2.1 Borings

Moore Twining Associates, Inc. (Moore Twining) performed 5 borings in the vicinity of the proposed building from June 2 to 3, 2021. Gregg Drilling & Testing advanced the borings using 8-inch outer diameter hollow stem auger drilling techniques to depths of 25 to 50 feet below ground surface (bgs). Soils were sampled with a Standard Penetration Test (SPT) sampler. The SPT was conducted in general accordance with ASTM D1586 and consists of driving a 2-inch outer diameter, 1-3/8-inch inner diameter spoon through the soil with a 140 lb hammer dropping 30 inches. The number of blows per 6 inches is recorded. The number of blows to drive the spoon from 6 to 12 inches is known as the "N-value". Modified California spoon sampling was performed in general accordance with ASTM D3550. The Modified California spoon samples are driven in the same manner as the SPT but consist of a 3-inch outer diameter, 2.4 inch inner diameter split spoon. 6-inch brass liners were utilized within the spoon to obtain samples. To obtain an equivalent "N-value" from the modified California sampler blow counts, the value should be multiplied by 0.65. An automatic hammer was used to perform the sampling. To obtain the N-value at 60% efficiency (known as N_{60}), an energy correction factor of 1.3, may be assumed.

The borings were backfilled with cement grout with 5 percent bentonite. After the grout settled, the borings were capped with bentonite chips and topped with cold patch asphalt that was tamped.

The boring logs are included in Appendix A.

2.2 Lab Testing

Moore Twining requested laboratory testing be performed on select soil and rock samples to evaluate the physical and engineering properties. Moore Twining performed the following laboratory tests in general accordance with the referenced standard:

- Moisture content (ASTM D2216)
- Dry Density (ASTM D2937)
- Grain Size Distribution (ASTM D422)
- Atterberg Limits (ASTM D4318)
- Direct Shear (ASTM D3080)
- Consolidation (ASTM D2435)
- Expansion Index Test (ASTM D4829)
- R-Value (ASTM D2844)

One sample was selected by Moore Twining to perform chemical analysis testing associated with the corrosion potential of the near surface soils. Moore Twining performed the corrosion analysis tests in general accordance with the following standards:

- pH (Cal Test 643)
- Minimum Soil Resistivity (ASTM G187)
- Sulfate Content (Cal Test 417)
- Chloride Content (Cal Test 422)

Laboratory test results are provided in Appendix B and are also summarized on the boring logs in Appendix A.

2.3 Cone Penetration Tests

Moore Twining performed two cone penetrometer test (CPT) soundings on June 3, 2021. Gregg Drilling & Testing advanced the CPT soundings hydraulicly using a 25-ton rig in accordance with ASTM D5778 to depths from 50 to 75 feet bgs. The CPTs utilized an electronic piezocone with a 60-degree apex and a diameter of 44.5 mm (about 1.75 inches). The CPT holes were backfilled with neat cement and topped with asphalt cold patch.

The CPT logs are presented in Appendix C.

2.4 Percolation Tests

Moore Twining performed 2 percolation tests at the site. Percolation test holes were advanced by Gregg Drilling & Testing on June 3, 2021 using 8 inch outer diameter hollow stem augers to depths from 3.5 to 5 feet bgs. The depth of the percolation test borings P-1 and P-2 measured to be 56 inches and 43 inches, respectively.

Percolation tests were performed in each of the percolation test borings. The preparation of the test hole and the percolation testing were conducted in accordance with County of Los Angeles Administration Manual GS200.2 "Guidelines for Geotechnical Investigation and Reporting Low Impact Development Stormwater Infiltration," dated June 30, 2017, prepared by the County of Los Angeles, Department of Public Works, Geotechnical and Materials Engineering Division. The percolation tests were performed by adding water to the test holes and measuring the decline in the water level over time. The test holes were presoaked with about 5 gallons of water so that the water level was at least 5 times the hole's radius. On the day of the percolation testing, the test hole was presoaked multiple times with a head of at least 12 inches of water for at least 1 hour. The holes were then filled with water to about 25.8 inches from the bottom of the percolation pipe in P-1 and to about 18.6 inches from the bottom of the percolation pipe in P-2, and the time it took for the water to fall 12 inches was recorded to determine the time interval for testing. 10 minute readings were performed at P-1 and 30 minute readings were performed at P-2. The amount of time for the water level to drop 12 inches or the amount of drop over the measurement time (whichever was faster) was recorded for each interval. Water was refilled after each interval and the process repeated. At least 8 intervals were measured until a stabilized rate of drop was achieved (defined as less than 10% difference between 3 consecutive readings). Including the 1 hour pre-soak, P-1 was conducted for about 2 hours and P-2 was conducted for about 6 hours.

The percolation boring logs and test results are provided in Appendix D.

3. Site Description

3.1 Site Description

The proposed building location is located at the SoCalGas Pico Rivera Base at 8101 Rosemead Boulevard in Pico Rivera, California. The proposed building is located within a parking lot in the southern portion of the base just north of the intersection of Maxine Street and Manzanar Avenue. The site location is shown in Figure 1 and recent site conditions are shown by aerial photography in Figure 2.

The elevation at the site ranges from around elevation 144 to 149 feet and generally slopes from east to west. Elevations in this report reference NAVD88.

Based upon a description of a site visit by Moore Twining, the existing parking lot is relatively flat and generally drains by sheet flow toward concrete swales in the drive lanes which flow to drain inlet(s) tied to an underground storm drain system. The existing pavement surface was in good condition, relatively free of cracking. Existing underground utilities are present within the site. The locations of underground electric lines and other anomalies identified by a utility locator were painted on the ground surface in the area of the field exploration locations.

Based on aerial photography, as recently as 1952 the site was used for agricultural purposes. Between 1952 and 1968, a parking lot was installed over a portion of the site in the location of the proposed building. By 1973, additional parking area was added. Current site conditions are similar to how they were in 1994 with the exception of the trees in planters in the parking lot were removed around 2017.

3.2 Geology

3.2.1 Physiographic Region

The project is located within the Peninsular Ranges geomorphic province (California Geological Survey 2002). The Peninsular Ranges is a series of ranges separated by northwest trending valleys approximately parallel to faults branching from the San Andreas Fault. The geology generally consists of granitic rock intruding older metamorphic rocks. It extends into lower California and is bound on the east by the Colorado Desert.

3.2.2 Surficial Geology

A custom report was generated through the Web Soil Survey. Based on this reference, the site is underlain by Urban land. The report describes the site as being derived from discontinuous human transported material over mixed alluvium derived from granite and/or sedimentary rock and being within a landform of alluvial fans.

Based upon the "Geologic Compilation of Quaternary Surficial Deposits in Southern California" (Bedrossian 2012), the site is underlain by young alluvial fan deposits (Qyf). Based on the "Map database for surficial materials in the conterminous United States" (Soller 2009), the site is underlain by alluvial sediments of the Holocene to Pliocene age that are greater than 100 feet thick.

The soil conditions observed in the borings indicate soils that are consistent with alluvial deposits.

3.2.3 Bedrock Geology

Based on the "Geologic Map of California" (Jennings, 2010), the site is underlain by marine and nonmarine (continental) sedimentary rocks of Pleistocene to Holocene age. The unit consists of unconsolidated to semi-consolidated alluvium, lake, playa, and terrace deposits. The deposits are primarily non-marine but include marine deposits near the coast.

The soil conditions observed in the borings did not encounter bedrock.

3.3 Subsurface Description

A summary of the geologic conditions is provided in the following Table. A detailed description of the soil and rock conditions encountered can be observed in the Boring Logs in Appendix A. Geologic cross-sections of the subsurface conditions are provided in Appendix E.

Table 3-1. Summary of Subsurface Conditions

Origin	USCS	Description
Asphalt and aggregate base	n/a	3 to 4.5 inches of asphalt concrete was observed at the ground surface in each of the borings performed at the site. 4 to 9 inches of aggregate base was observed beneath the asphalt.
Fill		Fill material was observed in the 3 of the 7 borings performed at the site and extended to depths ranging from approximately 1 to 3.5 feet. The fill soils were classified as silty sand. The relative density of the soils was medium dense.
Alluvial	SP SP-SM SW-SM	Alluvial soils were encountered beneath the pavement and fill soils to termination depth. These soils were classified as silty sand, poorly graded sand, and silt with varying amount of sand, silt, and gravel. The relative density of the sandy soils was loose to dense with the loose soils primarily being located in the upper 10-15 feet. The silt soils had a consistency ranging from soft to very stiff with the soft layers in the upper 10 feet.

3.3.1 Groundwater

The borings were drilled with hollow-stem auger drilling methods which allow for observation of the moisture of soil samples to evaluate groundwater levels. Groundwater was not observed within the borings at the time they were performed.

The USGS National Water Information System which contains groundwater measurements from wells identified several wells within about 3,000 feet of the site. In addition, the Los Angeles Department of Public Works has a database of well readings in the area. The well data is summarized in the following table. The closest wells indicate groundwater could be encountered as shallow as about 40 feet below grade or about elevation 105-110.

Table 3-2. Summary of Nearby Well Data

Source	Well ID	Location	Dates	Depth to GW (ft)	GW Elevation (ft)
LA DPW	1583X	2,100 ft NW	1950-2021	44-140	5-101
LA DPW	1583U	2,000 ft SW	1952-2018	56-135	7-86
LA DPW	1593S	2,400 ft SE	1950-2018	47-134	7-94
LA DPW	1582R	3,000 ft NW	1952-2008	46-122	28-104
USGS	335829118065202	2,800 ft NW	1998-2003	43-85	65-105
USGS	335829118065201	2,800 ft NW	1998-2003	45-80	68-103
USGS	335829118065203	2,800 ft NW	1998-2003	44-85	63-103
USGS	335829118065204	2,800 ft NW	1998-2003	52-90	58-95
USGS	335829118065205	2,800 ft NW	1998-2003	38-77	72-108
USGS	335829118065206	2,800 ft NW	1998-2003	36-72	75-110

Groundwater levels will fluctuate both seasonally and annually due to various factors including climatic conditions, site development, changes in runoff conditions, well pumping, etc.

3.4 Geo-Hazard Assessment

Campos has reviewed the geology at the site along with the results from the subsurface exploration to evaluate potential geo-hazards. A summary of the potential geo-hazards and risks they pose at the site follow:

Table 3-3. Summary of Potential Geo-Hazards

Geo-Hazard	Risk	Narrative
Karst	Low	Karst refers to a geologic setting formed from the dissolution of rocks such as limestone, dolomite, and gypsum which can result in sinkholes at the ground surface or voids within the rock. Based on a review of "Karst in the United States" (Weary 2014), the site is not mapped as being underlain by rock typical to karst. Therefore, the risk is low.
Pyritic shale	Low	Pyrite can be present within carbonaceous shales and is prone to producing sulfuric acid and gypsum growth. It can also cause expansion. Pyrite or carbonaceous shales were not observed.
Mining	Low	Historical surficial mining or deep mining may cause impacts at the surface if present. Based on a review of the California DCR Mines Online map, mines were not identified as being located within the project area.
Seismic	Moderate	Based on our evaluation of the borings the seismic site class based on ASCE 7-16 is Site Class D. Using the USGS 2014 earthquake data set, the mean design earthquake with a 2% chance of exceedance in 50 years is a magnitude 6.~~91~~ 88 approximately ~~9.9~~10.24 km away. Based on the USGS 2014 earthquake dataset, the design peak ground acceleration (PGA) is 0.805.

Geo-Hazard	Risk	Narrative
Liquefaction	Low	Liquefaction typically occurs in wet, very loose sands and silts when ground motions cause them to lose their strength. The soils onsite consist of medium dense to dense sands and gravels above the groundwater table so the potential for liquefaction is low.
Lateral Spreading	Low	Lateral spreading is the lateral movement of sloping saturated deposits. The soils observed onsite are not saturated and are generally medium dense to dense and the site is relatively flat. The risk of lateral spreading is low.
Expansive soils	Low	Expansive soils can expand and contract with moisture changes. Typically these are high plasticity fat clays. Fat clays were not observed within the borings. The risk of expansive soils to impact the pipe is low.
Tsunami	Low	Based on the CGS Tsunami Hazard Map (CGS 2009), the site is not located within a Tsunami hazard area and the risk of tsunami impacts is low.
Landslides	Low	The California Geological Survey Information Warehouse or the USGS landslide inventory map does not have landslides mapped within 10 miles of the project area. Steep slopes were not observed in the immediate vicinity of the project area.
Flooding	Low	FEMA maps the project site as being within Zone X, "Area with Reduced Flood Risk due to Levee".
Scour	Low	Due to the low risk of flooding and no streams onsite, scour impacts to the site will be minimal.

4. Earthwork Recommendations

Campos has reviewed the boring logs and laboratory testing from the site and developed the following recommendations for the various phases of site development. Earthwork and site preparation activities are expected to include demolition of existing site features, excavations for foundations and utilities, backfilling of trenches, and final grading of the site.

Earthwork should be performed under the full-time observation of a representative of the Geotechnical Engineer. Activities requiring observation include:

- Site preparation
- Proof-rolling and subgrade evaluation
- Subgrade improvement procedures
- Fill placement and compaction

4.1 Site Preparation

4.1.1 Demolition

The proposed building is located within an existing parking lot. If roadways, structures, or foundations are present in the proposed work area and are not planned to be reused, they should be demolished and removed from the site. The borings performed by Moore Twining identify the asphalt concrete to range in thickness from 3-4.5 inches and the aggregate base to range in thickness from 4-9 inches. The aggregate base may be left in place.

If desired, the asphaltic concrete may be processed into an acceptable gradation and reused as fill on site. However, it is anticipated that fill soils will not be required.

4.1.2 Existing Utilities

Existing utilities were identified in the vicinity of the proposed building during the subsurface exploration program. During design, these utilities should be identified and determined whether they should remain in service, be relocated, removed, or abandoned in place.

During site preparation, the contractor should take care to identify the location of existing utilities and utility related structures in the development area. Existing utilities to remain in service should be protected during construction. Other utilities should be relocated, removed or abandoned in place, in accordance with the project specifications.

4.1.3 Existing Fill Soils

Existing fill soils were encountered in the upper 3.5 feet in 3 of the 7 borings performed at the site. Whether the fill soils were placed under engineered controls is unknown at this time. Uncontrolled fills can pose a risk of being loose and result in excessive total or differential settlements at a site. However, since the fill soils are less than 4 feet in thickness, the pavement in the proposed site has been performing well, and the consideration that a proof-roll should provide insight as to whether there is any loose deposits, the risk of uncontrolled fill deposits posing a risk to site development is low. If large areas of loose deposits are observed during construction, then Campos should be contacted to provide an evaluation and supplemental recommendations.

4.1.4 Subgrade Preparation and Proof Rolling

Within the areas of site development where cuts, fills, structures, parking lots, or roadways are proposed, the site should be prepared prior to starting work. Because the site is an existing parking lot, topsoil or vegetation are not anticipated; however, if topsoil, roots, or vegetation are observed during excavation, they should be removed.

The site should be proof-rolled prior to placing of fill soils. Proof-rolling should be performed with a loaded tri-axle dump truck, loaded water truck, or with a 10-ton vibratory roller. Proof-rolling should be performed uniformly over the entire area and in perpendicular passes. In areas where large equipment cannot be utilized, lighter walk behind compaction equipment may be utilized to perform the proof-roll.

Proof-rolling is performed to identify zones of weakness in the subgrade where further evaluation and possible stabilization may be required. A visual inspection of the proof-roll should identify a firm and stable subgrade. The subgrade may also be evaluated with a hand probe to explore for potential zones of weakness.

If soft, unstable areas that exhibit rutting, pumping, or other instability are identified, that location should be remediated at the direction of the onsite representative of the geotechnical engineer. The area may need to be explored further by methods including test pits or laboratory testing. Stabilization techniques may include, but are not limited to:

- Scarifying, moisture conditioning, and recompacting soft/loose soils
- Removing soft/loose soils and replacing them with approved, compacted fill (see Section 4.2)
- Over-excavating to firm, stable soils and backfilling to grade with approved, compacted fill (see Section 4.2)

4.2 Fill Recommendations

Fill soils should consist of non-organic soils. Fill soils should generally be classified as or be a combination of SC, SM, SP, SW, GC, GM, GP, or GW soils as identified by ASTM D2487. Fill soils should have a maximum particle size of less than 4 inches. Frozen soils or soils containing frost should not be used as fill soils.

For each unique fill source (on-site borrow or import location) and if a change in material type occurs, laboratory testing including moisture content (ASTM D2216), grain size distribution (ASTM D6193), Atterberg limits (ASTM D4318), and Modified Proctor (ASTM D1557) should be performed. The results should be evaluated to confirm suitability of the fill source prior to being used onsite.

The onsite soils identified in the upper 5 feet were primarily sandy and are anticipated to be suitable for reuse as fill. Moisture contents of the upper 5 feet ranged from 2.2 to 8.4 percent. There is a possibility that moisture may need to be added to some of the soils during construction if they are too dry to facilitate compaction.

Fill soils shall be placed on stable subgrades that have passed a proof-roll and do not contain frost, ponding, or muddy soils. If stable subgrades are not present, the subgrade should be excavated to stable soils prior to placing Fill. Fill shall be placed in approximately level lifts. Lifts should not exceed a loose thickness of 12 inches if using large compaction equipment or 8 inches if using walk behind compaction equipment.

Fill soils should be moisture conditioned to within about 3% of the optimum moisture content and compacted to at least 95% of the maximum dry density as determined by the Modified Proctor Test (ASTM D1557) beneath proposed structures and roadways. In-situ density testing of the soils should be tested with either the sand cone (ASTM D1556) or nuclear density test (ASTM D6938). Density testing should be performed at a rate of 1 test per 5,000 ft^2 per lift for aerial fills and 1 test per 150 feet per lift of trench fills. Fill soils should also be judged to be firm and stable without significant movement under the weight of construction equipment passing over it.

4.2.1 Drainage Recommendations

Water should not be allowed to pool or pond onsite. Both temporary and permanent site grading should be planned to direct groundwater away from excavations, structures, and foundations. Sloping the ground surface at about a 2% slope away from structures for about 10 feet is recommended for permanent structures.

If water accumulates within excavations, it should be pumped out and the subgrade evaluated to confirm it is stable.

4.2.2 Temporary Excavations

Temporary excavations may be required during construction for trenching, foundation installations, or other reasons. Temporary excavations should comply with OSHA 29 CFR, Part 1926, Subpart P, "Excavations and Trenches". OSHA requires the contractor to designate a competent person to be responsible for the excavations who is capable of identifying existing and predictable hazards in the surroundings or working conditions and who has authorization to take prompt corrective measures to eliminate them. Complying with OSHA regulations and the stability of temporary trenches is the responsibility of the contractor.

For planning purposes, the soils encountered within our borings are classified by OSHA as Type C and requires a maximum allowable slope of 1.5H:1V side slope for excavations of 20 feet or less. In the event sidewall seepage or local instabilities are observed, a shallower slope may be required to maintain safety onsite.

Surface loads and stockpiles should be kept a minimum of 5 feet or the depth of the excavation, whichever is greater, from the edge of the top of slope.

Shoring may be used to facilitate construction. Use of these systems should keep ground displacement and vibrations within acceptable limits. Shoring system designs should be sealed by a licensed professional engineer within the state of the project. The system should be evaluated to consider slopes and appropriate surcharges including structures, live loads, and construction loads.

5. Design Recommendations

Campos has reviewed the boring logs and laboratory testing from the site and developed the following recommendations for the various phases of site development.

5.1 Foundations

Without ground improvement, the allowable bearing pressures of shallow foundations would be low (less than about 2,000 psf) to keep estimated settlement less than about 1 inch. To consider shallow foundations either removal and replacement of the upper loose soils would need to be performed or ground improvement of the soils beneath the foundations will need to be performed. We are recommending ground improvement with aggregate piers to support shallow foundations. Alternatively, driven piles can be considered to support the foundations.

5.1.1 Shallow Foundations supported on Aggregate Piers

Based on the results of our exploration, the building foundations can be supported by shallow spread footings provided ground improvement is implemented. While consideration could be given to a variety of ground improvement methodologies, we recommend considering aggregate piers to support the foundations. Aggregate piers go by a variety of terms including stone columns, rammed aggregate piers, vibratory stone columns, etc. Aggregate piers consist of stone columns that are typically 20-36 inches in diameter and are extended to a target depth.

Foundations supported on aggregate piers can typically support an allowable bearing pressure of 4,000 to 6,000 psf depending on the spacing of the aggregate piers. Due to a variety or proprietary installation techniques, final design of the aggregate pier layout is typically performed by the contractor and submitted to the engineer for approval. If desired, Campos EPC can perform the design of the aggregate piers. A load test of a test aggregate pier at the site should be completed prior to construction to confirm design assumptions.

Lateral foundation loads can be resisted by the friction of the bottom of the foundation. A friction factor of 0.35 may be used to calculate the resisting force. A factor of safety of 1.5 should be applied when using frictional resistance. If passive resistance of the foundation is to be used in conjunction with frictional resistance a factor of safety of 2 should be applied to the total resistance. A passive earth pressure coefficient of 2.8 and a unit weight of 115 pcf may be used (equivalent fluid pressure of 322 psf). Passive pressure should begin 1 foot below grade to account for potential future disturbance.

Any excessively loose, soft, or wet soils encountered in the footing excavations should be removed from below all footings. In areas where soft or unsuitable material is undercut, the footing could be lowered, or the excavation may be backfilled to re-establish the desired footing elevations.

Provided the foundation design and construction recommendations discussed herein are employed, the total settlement for the proposed foundations is estimated to be less than about 1 inch with differential settlement between similarly sized and loaded foundations being about half of the total settlement.

5.1.2 Driven Piles

Driven piles could also be considered for supporting the foundations. H-piles or pipe piles are feasible alternatives. Piles should be driven a minimum depth of 25 feet to support the foundations within the medium dense soils below the surficial loose zone. A summary of anticipated allowable skin friction and end bearing for driven piles is provided in the following table for preliminary pile design. If driven piles are selected, the capacity should be confirmed prior to finalizing pile design. These allowable values include a factor of safety of 2 for skin friction and 3 for end bearing.

Table 5-1. Summary of Axial Capacity for Driven Piles

Depth (feet)	Allowable Skin Friction (psf)	Allowable End Bearing (psf)
0-20	150	n/a
20-40	500	20,000
40-50	600	30,000

Pile cap design should consider a center-to-center pile spacing of at least 3 pile diameters. If closer spacing is required, ground effects should be considered.

Lateral resistance can be achieved through the lateral resistance of the piles by running LPILE. The following parameters are the ultimate strength values recommended for use in LPILE analysis of deep foundations.

Table 5-2. LPILE Parameter Recommendations

Soil Layer	Depth (feet)	Effective Unit Weight (pcf)	Angle of Friction (degrees)
Sand (Reese)	0-20	110	29
Sand (Reese)	20-40	115	32
Sand (Reese)	40-50	60	32

A test pile program consisting of 2 control piles should be performed to provide data for confirming pile design. Control piles should be installed utilizing a pile driving analyzer (PDA).

5.2 Slabs-on-Grade

Slabs-on-grade may be supported on a subgrade consisting of properly prepared onsite or fill soils. Slabs-on-grade should be designed and constructed in accordance with the recommendations of the most recent versions of ACI Committee Reports 360R and 302.10R.

Based on the subgrade preparation procedures recommended in this report, a subgrade modulus (k) of 150 pci is recommended for use in slab design. We recommend supporting the slab on a minimum of 4 inches of CalTrans Class 2 aggregate base to serve as a capillary break and provide uniform support of the slab.

A vapor retarder should be considered and located immediately below the slab. A vapor retarder often consists of visqueen or polyvinyl plastic sheeting at least 10 mil in thickness.

It is typical for construction activities to disturb the building pad between the time the building pad is prepared, and the new floor slab is constructed. We recommend that just prior to vapor retarder installation and slab construction, the building area subgrade be proof-rolled, and any unstable zones be stabilized. The moisture content of the subgrade soils should be maintained within the recommended range until floor slabs are completed.

5.3 Pavement Recommendations

The pavement sections depend on the proposed wheel loads. While wheel loads have not been provided, we have estimated the wheel loads from the following assumptions. The pavement sections have been designed assuming the asphalt will be placed at the end of construction. If construction loads will be on the asphalt, the design traffic loading may need to be modified. For the pavement designs, we assumed a 20-year design life for the pavement.

- Parking Lot (8,000 ESALs or TI of 5.0)
  - 1,000 passenger vehicles per day (one trip in and one trip out for 500 cars)
- Heavy Duty Drive Lanes (375,000 ESALs or TI of 8.0)
  - Four H-20 truck loads of up to 90,000 lbs per day (one trip in and out for two trucks).
  - 2,000 passenger vehicles per day (one trip in and one trip out for 1,000 cars)

Moore Twining performed one R-value (ASTM D2844) test on a sample for this project from boring HS-3 at a depth of 1/5 to 5 feet. The lab test results are included in Appendix A. The lab testing resulted in an R-values of 62. Due to potential variability in soil types used onsite we have used an R-value of 50 in our analysis. We have assumed the pavement subgrade will be prepared in accordance with the recommendations in Section 3.4.

Parking areas should be sloped with drainage gradients of at least 2% to carry surface water to the storm drains. Surface water ponding should not occur on site during or after construction.

5.3.1 Asphalt Concrete

The asphalt concrete pavement design sections were based on Caltrans Highway Design Manual Section 610. Our recommendations for the asphalt pavement sections are summarized in the following table:

Table 3. Recommended AC Pavement Sections

Section Type	Traffic Index	Asphalt Concrete (in.)	Aggregate Base (in)
Parking Lot	5.0	3.0	4.0
Heavy Duty Drive Lane	8.0	5.0	5.0

The asphalt concrete should consist of a 1.5-inch-thick course of Superpave 9.5 mm hot mix asphalt for the surface course. The wearing coarse may be Superpave 12.5 mm hot mix asphalt and should be placed in lifts not exceeding 4 inches.

The aggregate base should consist of Class 2 aggregate.

Pavements can undergo seasonal movements due to changes in temperature and subgrade moisture. In addition, movements may occur during typical loading conditions. Movements can accelerate pavement deterioration. As joints and cracking develop, surface water can infiltrate into the pavement and exacerbate cracking. The design life assumes that standard maintenance will be performed. Standard maintenance includes a crack sealing program and slurry seal coating as cracking becomes more pronounced.

5.3.2 Portland Cement Concrete

Concrete pavements may be desired in heavily trafficked areas or where added durability is required such as loading docks. Our design of PCC pavement sections follows the procedure outlined in "Guide for Design of Pavement Structures" (AASHTO 1993). The following assumptions were made:

- Reliability of 90%
- Standard Deviation of 0.35
- Initial Serviceability of 4.5
- Terminal Serviceability of 2.5

Table 4. Recommended PCC Pavement Sections

Section Type	Traffic Index	Portland Concrete (in.)	Aggregate Base (in)
Heavy Duty Drive Lane	8.0	6.0	4.0

Concrete should have a minimum compressive strength of 4,000 psi. The pavement should be designed as jointed concrete with a load transfer device between the joints.

The aggregate base should consist of Class 2 aggregate.

Construction joints in the pavement should be sealed with a flexible sealer to prevent infiltration of water.

5.4 Seismic Considerations

The seismic site recommendations were evaluated using ASCE7-16. Recommendations for seismic design of the site are included in Table 5-5.

Table 5-5. Seismic Design Recommendations

Parameter	Variable	Value
Seismic Site Class		"D"
Peak Ground Acceleration of MCE	PGA	0.768
Site Modified Peak Ground Acceleration	PGA_M	0.845
Ground Motion for MCE (0.2 sec period)	S_s	1.786
Ground Motion for MCE (1.0 sec period)	S_1	0.64
Site Amplification Factor for 0.2 second	F_a	1.0
Site Amplification Factor for 1.0 second	F_v	1.7 (See Note)
Site Modified Spectral Acceleration Value	S_{MS}	2.143
Site Modified Spectral Acceleration Value	S_{M1}	1.7 (See Note)
Seismic Design Value at 0.2 Second	S_{DS}	1.429
Seismic Design Valuer at 1.0 Second	S_{D1}	1.7 (See Note)

Note: ASCE7-16 states that a site-specific response analysis be performed except if conditions in Section11.4.8 are met.

Fugro performed a site-specific seismic hazard assessment which is attached as Appendix F.

The risk of liquefaction or lateral spreading due to seismic activity at the site is low due to the depth of groundwater.

5.5 Corrosion Considerations

Select soil samples were tested by Moore Twining Associates, Inc. for properties that can be used to evaluate corrosion potential. A summary of the results of these tests are provided in Table 5-6.

Table 5-6. Summary of Corrosion Potential Test Results

Boring	Depth (ft)	Sulfates (mg/kg)	Chlorides (mg/kg)	Min. Resistivity (ohm-cm)	pH
HS-5	1-3.5	19	6.0	2,700	8.0

The results of the corrosion potential testing were compared to ACI 318. Based upon the sulfate levels, the soils are classified as having a class S0 risk of sulfate exposure. Based on anticipated sulfate exposure, there is not a requirement for specific cement type, no required water-cement ratio and a minimum unconfined compressive strength of 2,500 psi is recommended by ACI 318.

6. Limitations

Campos performed our services in a manner consistent with that level of care and skill ordinarily exercised by members of the profession currently practicing in the same locality and under similar conditions as this project.

This report was prepared for the exclusive use by our Client and specifically for use on the referenced project. Campos assumes no responsibility if this report is used by other parties or for other projects. Any third-party use of this information is for information only and is done at their own risk. No warranties, either express or implied, are intended or made.

Campos is not responsible for the misinterpretation of our recommendations presented within this report.

Our recommendations in this report are based upon our understanding of the proposed project at the time of this report. If changes are made in the design, nature, or location of the proposed project, our conclusions and recommendations shall not be considered valid unless we are retained to review such changes and confirm or modify our conclusions and recommendations in writing.

This report is based solely on the data acquired at the locations of exploration noted in this report. It is possible that the subsurface conditions (including but not limited to soil or rock types, depths and thickness of layers, groundwater depths, etc.) between the exploration locations may vary. If during the course of project construction, the subsurface conditions vary from those noted in the report, Campos should be notified to review and make any necessary changes to our recommendations and conclusions.

This report has not considered hazardous material classifications nor environmental impacts. If there is concern about potential environmental impacts, additional studies should be performed.

This report should be considered valid for a period of two-years after issuance. After that time, we should be engaged to review site conditions and plans to evaluate if conditions may have changed that may influence our recommendations.

A greater level of understanding of the site can be obtained with additional explorations, testing, and analysis. Additional information also has a cost associated with it. As such, our Clients share in determining the level of investigation to be performed and the amount of risk to take on. If our Client would like to limit risks further, we can perform additional testing.

7. References

Bedrossian et. Al. 2012. Geologic Compilation of Quaternary Surficial Deposits in Southern California. California Geological Survey. Special Report 217.

California Division of Mine Reclamation, 2021. Mines Online Interactive Map. https://maps.conservation.ca.gov/mol/index.html

California Geological Survey, 2002. California Geomorphic Provinces. Note 36.

California Geological Survey, 2009. CGS Information Warehouse: Tsunami Hazard Area Map. Accessed at https://maps.conservation.ca.gov/cgs/informationwarehouse/index.html.

Horton, J.D., San Juan, C.A., and Stoeser, D.B., 2017, The State Geologic Map Compilation (SGMC) geodatabase of the conterminous United States (ver. 1.1, August 2017): U.S. Geological Survey Data Series 1052, 46 p., https://doi.org/10.3133/ds1052.

Jennings et. Al. 2010. Geologic Map of California. California Geological Survey. GDM No. 2.

Los Angeles County Public Works. 2021. Well Map. https://dpw.lacounty.gov/general/wells/#

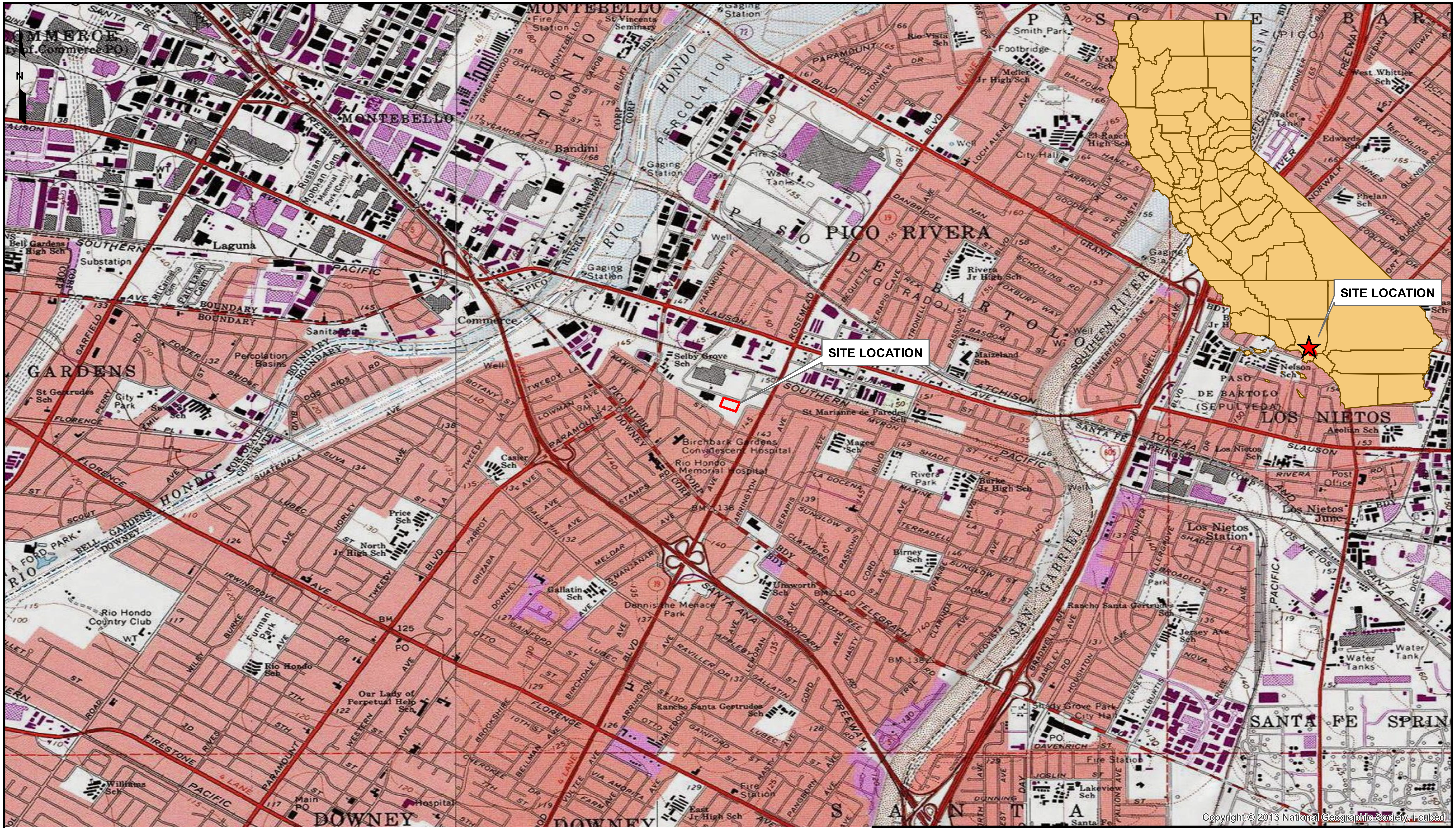
Soil Survey Staff, Natural Resources Conservation Service, United States Department of Agriculture. Web Soil Survey. Available online at the following link: http://websoilsurvey.sc.egov.usda.gov/.

Soller, D.R., Reheis, M.C., Garrity, C.P., and Van Sistine, D.R., 2009, Map database for surficial materials in the conterminous United States: U.S. Geological Survey Data Series 425, scale 1:5,000,000 [https://pubs.usgs.gov/ds/425/]

United Stated Geological Survey. 2021. National Water Information System: Mapper. https://maps.waterdata.usgs.gov/mapper

Weary, D.J., and Doctor, D.H., 2014, Karst in the United States: A digital map compilation and database: U.S. Geological Survey Open-File Report 2014–1156

FIGURES



0 2,000 4,000

Feet

CAMPOS | EPC

Project No.	00037.0000.0023
Drawn By	ESB
Checked	RRS
Date	10/28/21
Revision	0

Site Location Plan

Southern California Gas
New Office Building
8101 Rosemead Boulevard
Pico Rivera, California

FIGURE
1

0 50 100
Feet

Notes
1. Boring locations are approximate.

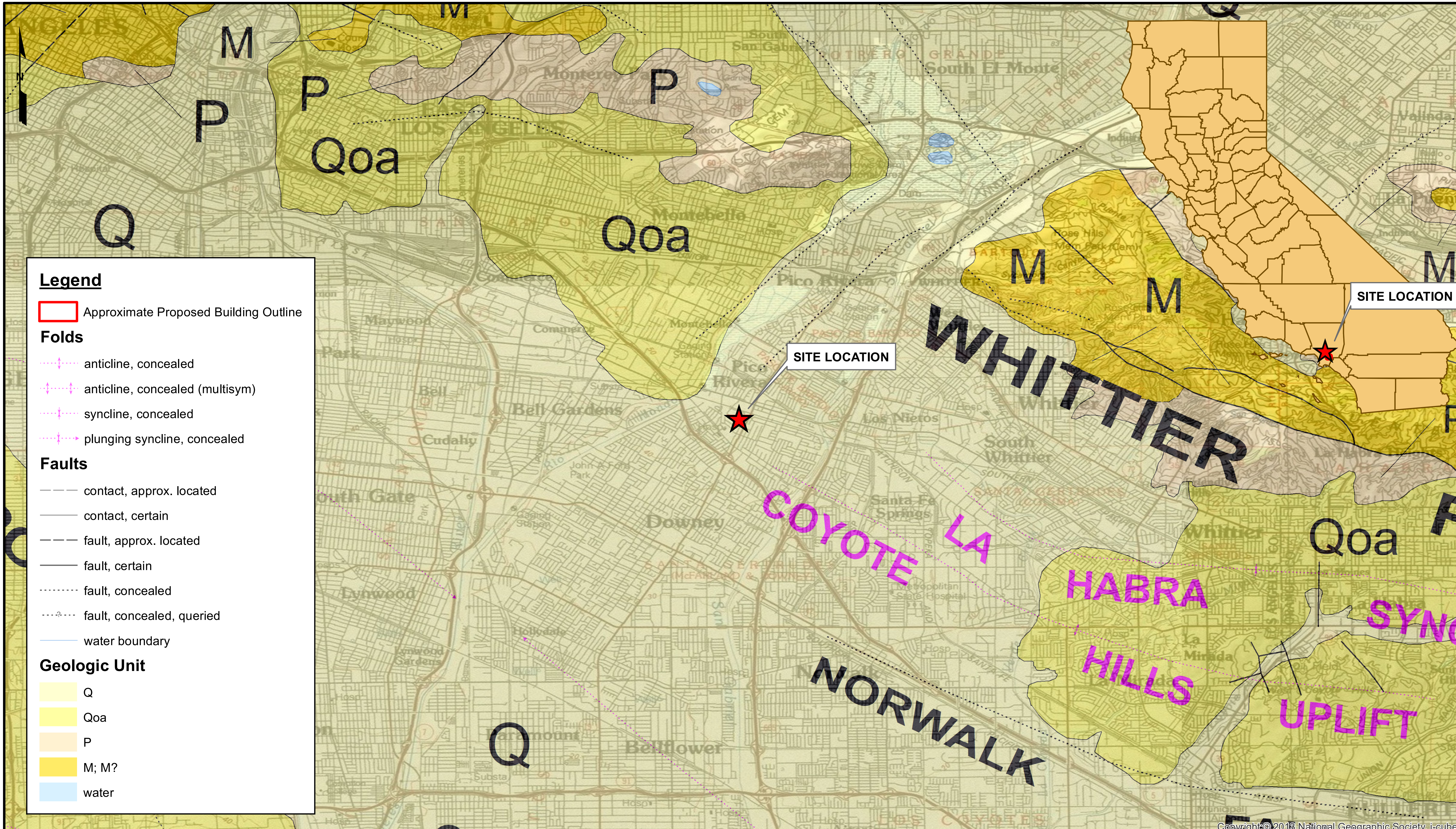
CAMPOS | EPC

Project No.	00037.0000.0023
Drawn By	ESB
Checked	RRS
Date	10/28/21
Revision	0

Exploration Location Plan

Southern California Gas
New Office Building
8101 Rosemead Boulevard
Pico Rivera, California

FIGURE
2



0 8,000 16,000
Feet

References
Jennings et. Al. 2010. Geologic Map of California. California Geological Survey. GDM No. 2.

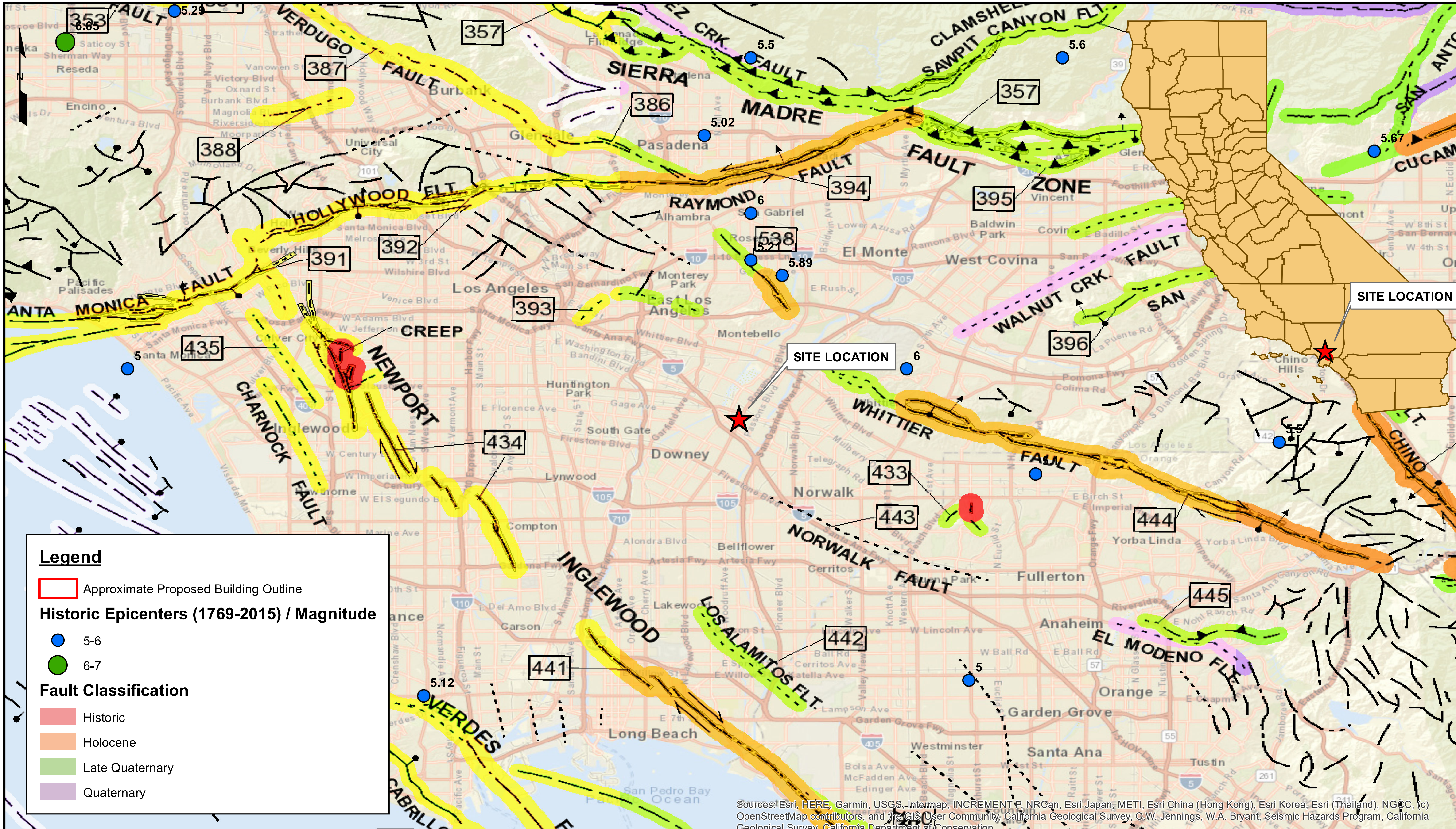
CAMPOS | EPC

Project No.	00037.0000.0023
Drawn By	ESB
Checked	RRS
Date	10/28/21
Revision	0

Bedrock Geology Map

Southern California Gas
New Office Building
8101 Rosemead Boulevard
Pico Rivera, California

FIGURE
3



0 4 8
Miles

References

Jennings et. Al. 2010. Geologic Map of California. California Geological Survey. GDM No. 2.

CAMPOS | EPC

Project No.	00037.0000.0023
Drawn By	ESB
Checked	RRS
Date	10/28/21
Revision	0

Faults and Epicenter Map

Southern California Gas
New Office Building
8101 Rosemead Boulevard
Pico Rivera, California

FIGURE
4

APPENDIX A

Geotechnical Borings Logs

MOORE TWINING ASSOCIATES, INC.

Since 1898

Test Boring: HS-1

Project: Proposed Gas Operations Control Building, Pico Rivera

Project Number: C73111.01

Drilled By: Gregg Drilling & Testing

Drill Type: MARL M-11

Auger Type: 8" O.D. Hollow Stem Augers

Hammer Type: 140 Pound Auto Trip

Logged By: A.H.

Date: June 2, 2021

Elevation: N/A

Depth to Groundwater First Encountered During Drilling: N/E

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
0		AC	Asphalt Concrete = 4 inches			
	9/6 8/6 9/6	AB	Aggregate Base = 6 inches		17	6.5
		FILL SM	FILL - SILTY SAND; medium dense, moist, fine grained, brown, with fine gravel			
	6/6 8/6 11/6	SP-SM		DD = 91.2 pcf	19	2.3
5	3/6 5/6 6/6	SP	AT 1.5 FEET - NATIVE - SILTY SAND; medium dense, moist, fine grained, brown		11	
			AT 2 FEET - POORLY GRADED SAND WITH SILT; medium dense, moist, fine grained, grayish brown			
10	2/6 2/6 3/6		AT 3.5 FEET - POORLY GRADED SAND; medium dense, damp, fine to medium grained, grayish brown AT 5 FEET - Fine grained		5	
15	3/6 2/6 3/6		Loose		5	
20	5/6 6/6 5/6		Medium dense, fine to medium grained		11	
	7/6 10/6 13/6	SP-SM	POORLY GRADED SAND WITH SILT; medium dense, damp, fine to medium grained, gray AT 24 FEET - Fine grained		23	
25			Bottom of Boring HS-1 at 25 feet			

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.

Test Boring: HS-2

Project: Proposed Gas Operations Control Building, Pico Rivera

Project Number: C73111.01

Drilled By: Gregg Drilling & Testing

Drill Type: MARL M-11

Auger Type: 8" O.D. Hollow Stem Augers

Hammer Type: 140 Pound Auto Trip

Logged By: A.H.

Date: June 3, 2021

Elevation: N/A

Depth to Groundwater First Encountered During Drilling: N/E

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
0		AC	Asphalt Concrete = 4 inches			
		AB	Aggregate Base = 5.5 inches			
	7/6 8/6 9/6	SM	SILTY SAND; medium dense, moist, fine grained, brown	From 1.5-3': DD = 93.5 pcf Sand = 70.8% -200 = 29.2%	17	6.0
	2/6 1/6 2/6		Very loose, with some interbedded sandy silt		3	
5	4/6 4/6 4/6		Loose, increase in fines content	From 1.5-5': EI = 0 From 5-6.5': DD = 86.2 pcf Sand = 58.7% -200 = 41.3% ø = 29° c = 140 psf	8	8.0
10	2/6 1/6 3/6	ML	SILT WITH SAND; soft, moist, non-plastic, brown		4	17.7
15	3/6 4/6 6/6	SP-SM	POORLY GRADED SAND WITH SILT; loose, damp, fine to medium grained, light brown	From 15-16.5': Sand = 91.1% -200 = 8.9%	10	
20	7/6 9/6 8/6		Medium dense		17	
25	5/6 8/6 11/6		Gray		19	

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.

Test Boring: HS-2

Project: Proposed Gas Operations Control Building, Pico Rivera

Project Number: C73111.01

Drilled By: Gregg Drilling & Testing

Drill Type: MARL M-11

Auger Type: 8" O.D. Hollow Stem Augers

Hammer Type: 140 Pound Auto Trip

Logged By: A.H.

Date: June 3, 2021

Elevation: N/A

Depth to Groundwater First Encountered During Drilling: N/E

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
30	6/6 8/6 9/6	ML	SANDY SILT; very stiff, damp, non-plastic, brown	From 30-31.5': Sand = 32.1% -200 = 67.9% LL = Non-Viscous PI = Non-Plastic	17	
35	5/6 8/6 11/6		Non-plastic	From 35-35.75': Sand = 38.0% -200 = 62.0%	19	
		SM	SILTY SAND; medium dense, moist, very fine grained, brown	From 35.75-36.5': Sand = 86.4% -200 = 13.6%		
40	10/6 13/6 14/6	SP-SM	POORLY GRADED SAND WITH SILT; medium dense, damp, fine grained, gray	From 40-41.5': Sand = 89.0% -200 = 11.0%	27	
45	13/6 15/6 12/6		Increase in grain size, fine to medium grained, with some coarse sand, and a little fine gravel		27	
		ML	SILT WITH SAND; very stiff, moist, non-plastic, gray	From 46.25-46.5': LL = Non-Viscous PI = Non-Plastic		
	6/6 12/6 14/6	SM	SILTY SAND; medium dense, moist, fine grained, gray, high fines content	From 48.5-49.25': Sand = 50.7% -200 = 49.3%	26	
50		SP-SM	POORLY GRADED SAND WITH SILT; medium dense, damp, fine grained, gray, a little fine to coarse gravel	From 49.25-50': Gravel = 8.2% Sand = 85.0% -200 = 6.8%		
			Bottom of Boring HS-2 at 50 feet			
55						

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.

Since 1898

Test Boring: HS-3

Project: Proposed Gas Operations Control Building, Pico Rivera

Project Number: C73111.01

Drilled By: Gregg Drilling & Testing

Drill Type: MARL M-11

Auger Type: 8" O.D. Hollow Stem Augers

Hammer Type: 140 Pound Auto Trip

Logged By: A.H.

Date: June 2, 2021

Elevation: N/A

Depth to Groundwater First Encountered During Drilling: N/E

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
0		AC	Asphalt Concrete = 4.5 inches			
		AB	Aggregate Base = 5 inches			
	9/6 11/6 11/6	SM	SILTY SAND; medium dense, moist, fine grained, brown	From 1.25-2.75': DD = 92.0 pcf	22	4.9
						2.2
	3/6 5/6 5/6		AT 1.75 FEET - Damp, gray, decrease in fines content	From 1.5-5': Sand = 86.3% -200 = 13.7% R-value = 62	10	
5	7/6 10/6 12/6		AT 2.75 FEET - Loose, fine to medium grained		22	1.7
		SP	POOLY GRADED SAND; medium dense, damp, fine grained, gray	From 5-6.5': DD = 94.3 pcf		
10	4/6 3/6 5/6		Loose		8	
15	2/6 4/6 5/6	SM	SILTY SAND; loose, moist, fine grained, brown		9	
		ML	AT 16.25 FEET - SANDY SILT; stiff, moist, non-plastic, brown			
20	4/6 4/6 5/6	SM	SILTY SAND; loose, moist, fine grained, brown, laminated, with iron-oxide staining, high fines content	From 20-21.5': Sand = 75.4% -200 = 24.6%	9	
	10/6 14/6 15/6	SP	POOLY GRADED SAND; medium dense, moist, fine to medium grained, gray		29	
25			Bottom of Boring HS-3 at 25 feet			

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.

Test Boring: HS-4

Project: Proposed Gas Operations Control Building, Pico Rivera

Project Number: C73111.01

Drilled By: Gregg Drilling & Testing

Drill Type: MARL M-11

Auger Type: 8" O.D. Hollow Stem Augers

Hammer Type: 140 Pound Auto Trip

Logged By: A.H.

Date: June 2, 2021

Elevation: N/A

Depth to Groundwater First Encountered During Drilling: N/E

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
0		AC	Asphalt Concrete = 3.1 inches			
	12/6 14/6 12/6	AB	Aggregate Base = 4 inches		26	6.9
	6/6 8/6 9/6	SM	SILTY SAND; medium dense, moist, fine grained, brown At 3.25': Light brown, decrease in fines content and moisture content	From 2.5-4': DD = 87.9 pcf Sand = 84.8% -200 = 15.2%	17	6.6 3.0
5	2/6 1/6 2/6		Very loose		3	13.7
		ML	AT 5.25 FEET - SILT WITH SAND; soft, damp, non-plastic, brown	From 5.25-6.5': Sand = 19.9% -200 = 80.1%		
10	3/6 4/6 4/6	SP	POORLY GRADED SAND; loose, damp, fine grained, light brown		8	
15	5/6 6/6 6/6		Medium dense		12	
20	7/6 8/6 12/6				20	
	8/6 8/6 10/6				18	
25			Bottom of Boring HS-4 at 25 feet			

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.

Test Boring: HS-5

Project: Proposed Gas Operations Control Building, Pico Rivera

Project Number: C73111.01

Drilled By: Gregg Drilling & Testing

Drill Type: MARL M-11

Auger Type: 8" O.D. Hollow Stem Augers

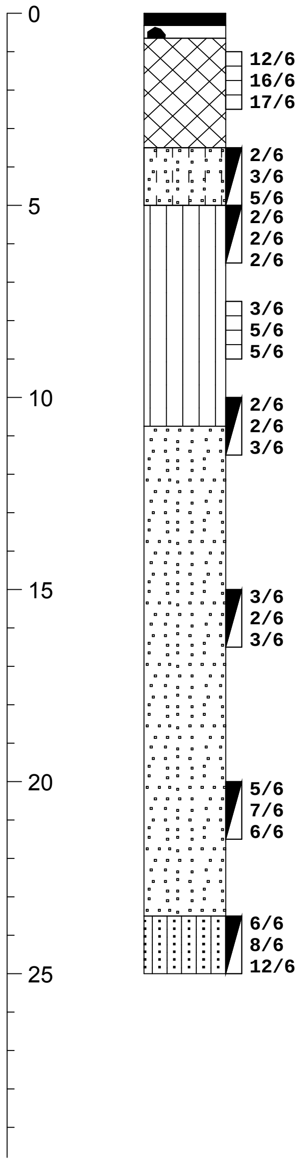
Hammer Type: 140 Pound Auto Trip

Logged By: A.H.

Date: June 2, 2021

Elevation: N/A

Depth to Groundwater First Encountered During Drilling: N/E



ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
0		AC	Asphalt Concrete = 3.75 inches			
	12/6 16/6 17/6	AB	Aggregate Base = 4 inches	From 1-2.5': DD = 117.6 pcf	33	8.4
		FILL	FILL - SILTY SAND; medium dense, moist, fine grained, brown, with some weakly cemented clods	From 1-3.5': pH = 8.0 SR = 2,700 ohm-cm Cl < 0.00060% SS = 0.0019%		
	2/6 3/6 5/6	SP-SM	NATIVE - POORLY GRADED SAND WITH SILT; loose, damp, fine grained, brown		8	4.1
5	2/6 2/6 2/6	ML	SILT WITH SAND; soft, moist, non-plastic, brown		4	16.0
	3/6 5/6 5/6		At 7.5 feet - Medium stiff, increase in moisture content	From 7.5-9': DD = 93.2 pcf Sand = 29.1% -200 = 70.9% LL = Non-Viscous PI = Non-Plastic	10	23.0
10	2/6 2/6 3/6				5	
		SP	POORLY GRADED SAND; loose, damp, fine grained, brown			
15	3/6 2/6 3/6		Light brown		5	
20	5/6 7/6 6/6		Medium dense		13	
			Fine to coarse grained, trace fine gravel, gray			
	6/6 8/6 12/6	SM	SILTY SAND; medium dense, moist, very fine grained, grayish brown, high fines content		20	
25			Bottom of Boring HS-5 at 25 feet			

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.

Test Boring: P-1

Project: Proposed Gas Operations Control Building, Pico Rivera

Project Number: C73111.01

Drilled By: Gregg Drilling & Testing

Drill Type: MARL M-11

Auger Type: 8" O.D. Hollow Stem Augers

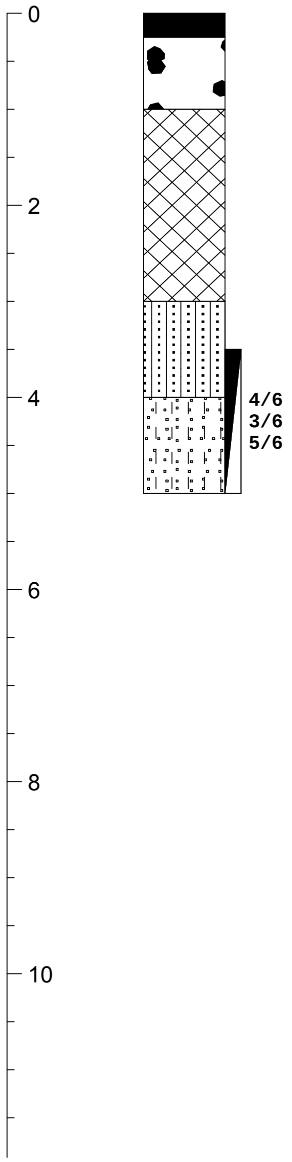
Hammer Type: 140 Pound Auto Trip

Logged By: A.H.

Date: June 2, 2021

Elevation: N/A

Depth to Groundwater First Encountered During Drilling: N/E

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
0		AC	ASPHALT CONCRETE - 3 inches			
		AB	Aggregate Base = 9 inches			
		FILL	FILL - SILTY SAND WITH GRAVEL; moist, fine to medium grained, brown			
		SM	NATIVE - SILTY SAND; moist, fine grained, brown AT 3.5 FEET - Loose		8	
4	4/6 3/6 5/6	SP-SM	AT 4 FEET - POORLY GRADED SAND WITH SILT; loose, damp, fine grained, gray	From 4-5': Sand = 92.3% -200 = 7.7%		2.5
			Bottom of Percolation Test Boring P-1 at 5 feet (Hole measured to be 56 inches deep after pulling augers and setting up percolation test)			

Notes:

Figure Number

Test Boring: P-2

Project: Proposed Gas Operations Control Building, Pico Rivera

Project Number: C73111.01

Drilled By: Gregg Drilling & Testing

Drill Type: MARL M-11

Auger Type: 8" O.D. Hollow Stem Augers

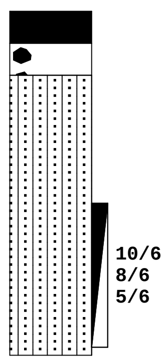
Hammer Type: 140 Pound Auto Trip

Logged By: A.H.

Date: June 2, 2021

Elevation: N/A

Depth to Groundwater First Encountered During Drilling: N/E



ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
0		AC	Asphalt Concrete = 4 inches			
		AB	Aggregate Base = 4 inches			
		SM	SILTY SAND; moist, fine grained, brown			
2	10/6 8/6 5/6		Medium dense	From 2-3.5': Sand = 67.8% -200 = 32.2%	13	6.7
			Bottom of Percolation Test Boring P-2 at 3.6 feet			
4						
6						
8						
10						

Notes:

Figure Number

KEY TO SYMBOLS

Symbol	Description
Strata symbols	
	Asphalt concrete
	Aggregate base
	Fill
	Silty sand
	Poorly graded sand with silt
	Poorly graded sand
	Silt

Symbol	Description
Misc. Symbols	
	Boring continues
Soil Samplers	
	Standard penetration test
	California Modified split barrel ring sampler

Notes:

1. Exploratory borings were drilled on 6/3/21 using a MARL M-11 drill rig equipped with 8" outside diameter hollow stem augers.

2. Groundwater was not encountered in any of the borings.

3. Boring locations were measured or paced from existing features.

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

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

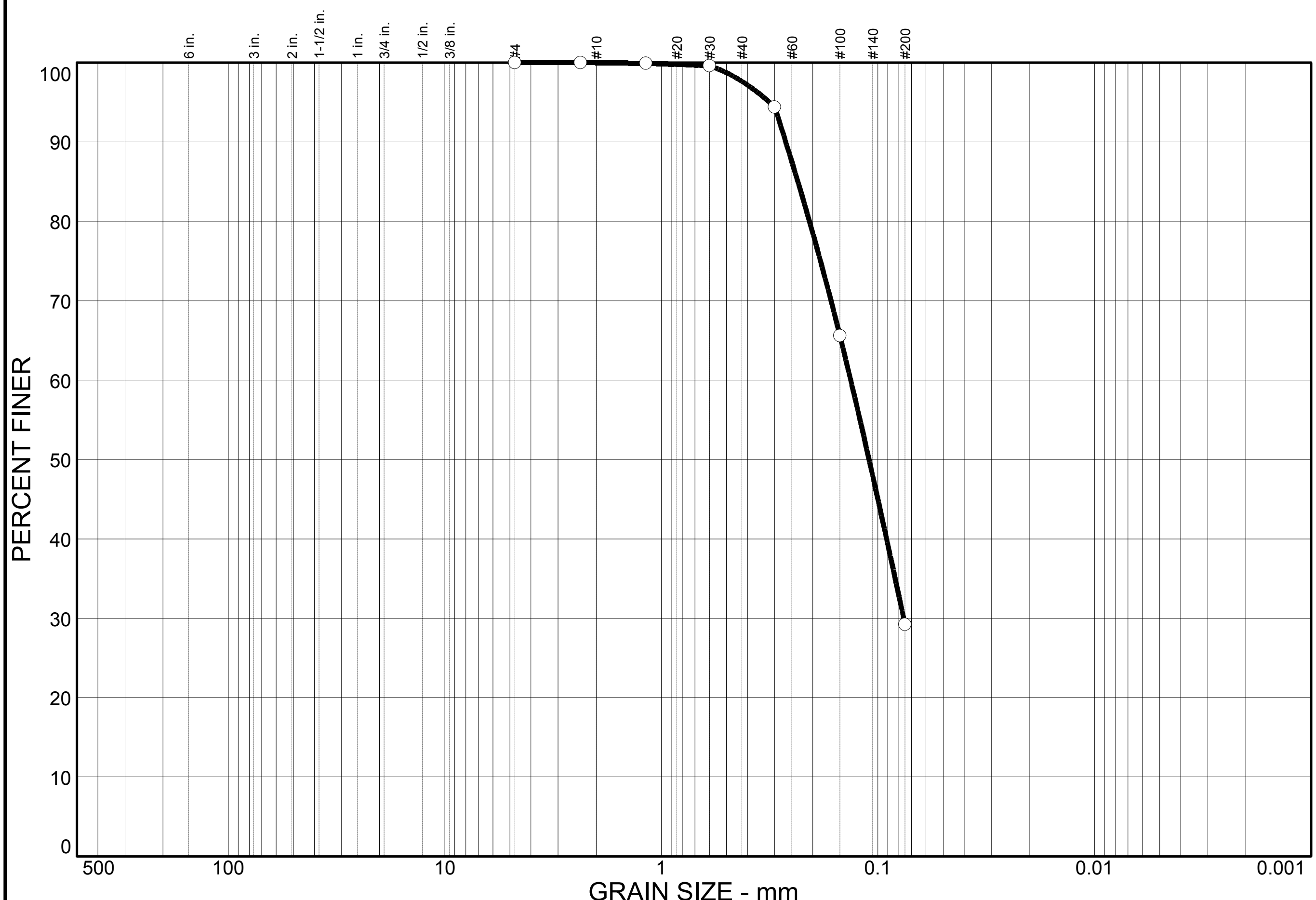
6. Results of tests conducted on samples recovered are reported on the logs.

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

APPENDIX B

Laboratory Test Results

Particle Size Distribution Report



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	0.0	70.8	29.2	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#4	100.0		
#8	100.0		
#16	99.9		
#30	99.6		
#50	94.4		
#100	65.6		
#200	29.2		

* (no specification provided)

Material Description

Silty sand

Atterberg Limits

PL= LL= PI=

Coefficients

D_{85}= 0.234 D_{60}= 0.134 D_{50}=
D_{30}= 0.0761 D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= SM AASHTO=

Remarks

Sample No.: HS-2 **Source of Sample:** **Date:** 6/2/21
Location: **Elev./Depth:** 1.5-3'

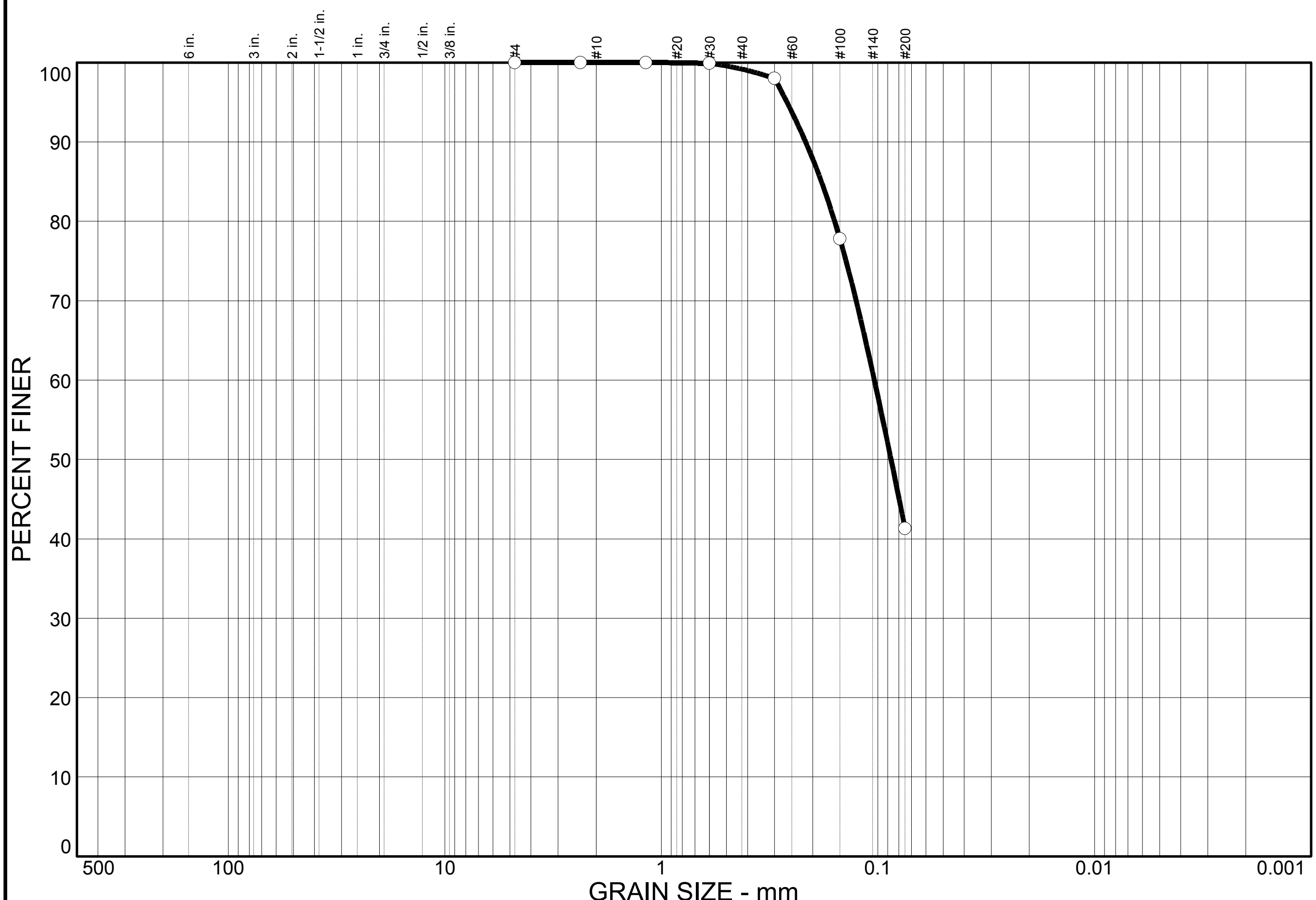
Moore Twining Associates, Inc.

Fresno, CA

Client: Geosyntec Consultans
Project: Proposed Gas Operations Control Building, Pico Rivera

Project No: C73111.01 **Figure**

Particle Size Distribution Report



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	0.0	58.7	41.3	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#4	100.0		
#8	100.0		
#16	100.0		
#30	99.9		
#50	98.0		
#100	77.8		
#200	41.3		

* (no specification provided)

Material Description

Silty sand

Atterberg Limits

PL= LL= PI=

Coefficients

D_{85}= 0.183 D_{60}= 0.104 D_{50}=
D_{30}= D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= SM AASHTO=

Remarks

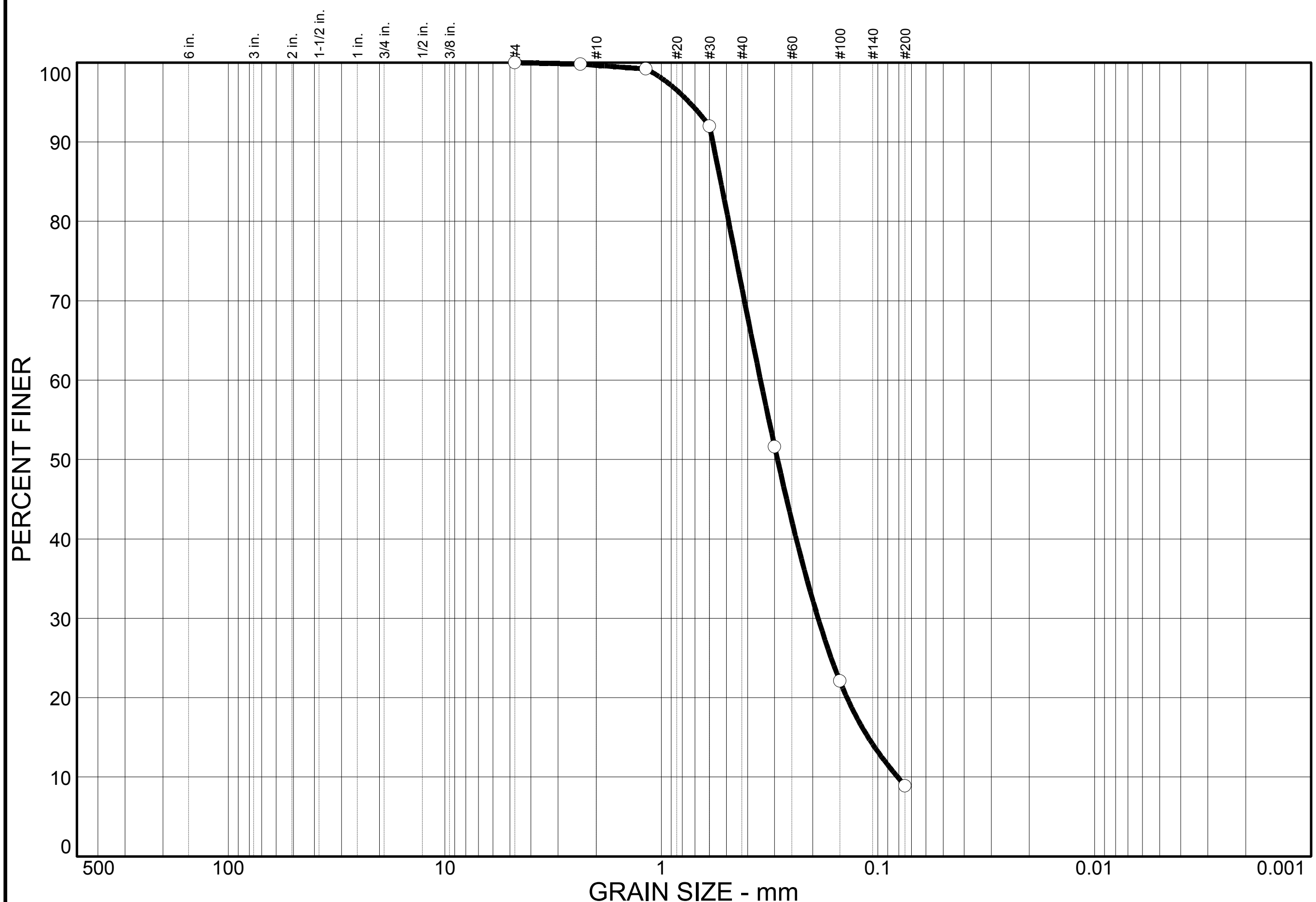
Sample No.: HS-2 **Source of Sample:** **Date:** 6/2/21
Location: **Elev./Depth:** 5-6.5'

Moore Twining Associates, Inc.

Fresno, CA

Client: Geosyntec Consultans
Project: Proposed Gas Operations Control Building, Pico Rivera
Project No: C73111.01 **Figure**

Particle Size Distribution Report



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	0.0	91.1	8.9	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#4	100.0		
#8	99.8		
#16	99.2		
#30	92.0		
#50	51.6		
#100	22.1		
#200	8.9		

* (no specification provided)

Material Description

Poorly graded sand with silt

Atterberg Limits

PL= LL= PI=

Coefficients

D_{85}= 0.533 D_{60}= 0.349 D_{50}= 0.291
D_{30}= 0.189 D_{15}= 0.111 D_{10}= 0.0811
C_u= 4.30 C_c= 1.26

Classification

USCS= SP-SM AASHTO=

Remarks

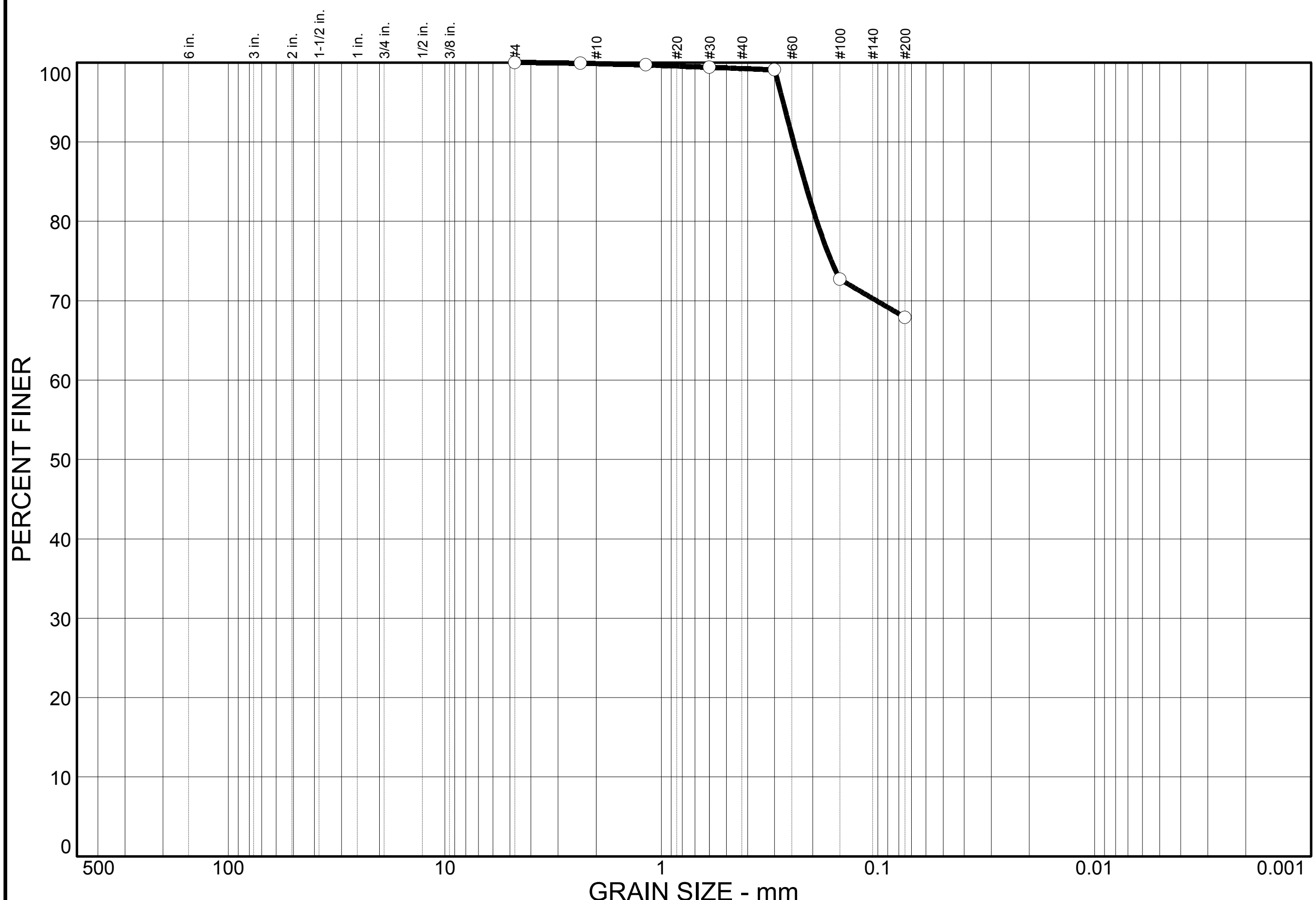
Sample No.: HS-2 **Source of Sample:** **Date:** 6/2/21
Location: **Elev./Depth:** 15-16.5'

Moore Twining Associates, Inc.

Fresno, CA

Client: Geosyntec Consultans
Project: Proposed Gas Operations Control Building, Pico Rivera
Project No: C73111.01 **Figure**

Particle Size Distribution Report



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	0.0	32.1	67.9	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#4	100.0		
#8	99.9		
#16	99.7		
#30	99.4		
#50	99.1		
#100	72.7		
#200	67.9		

* (no specification provided)

Material Description

Sandy silt

Atterberg Limits

PL= NP LL= NV PI= NP

Coefficients

D_{85}= 0.218 D_{60}= D_{50}=
D_{30}= D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= ML AASHTO=

Remarks

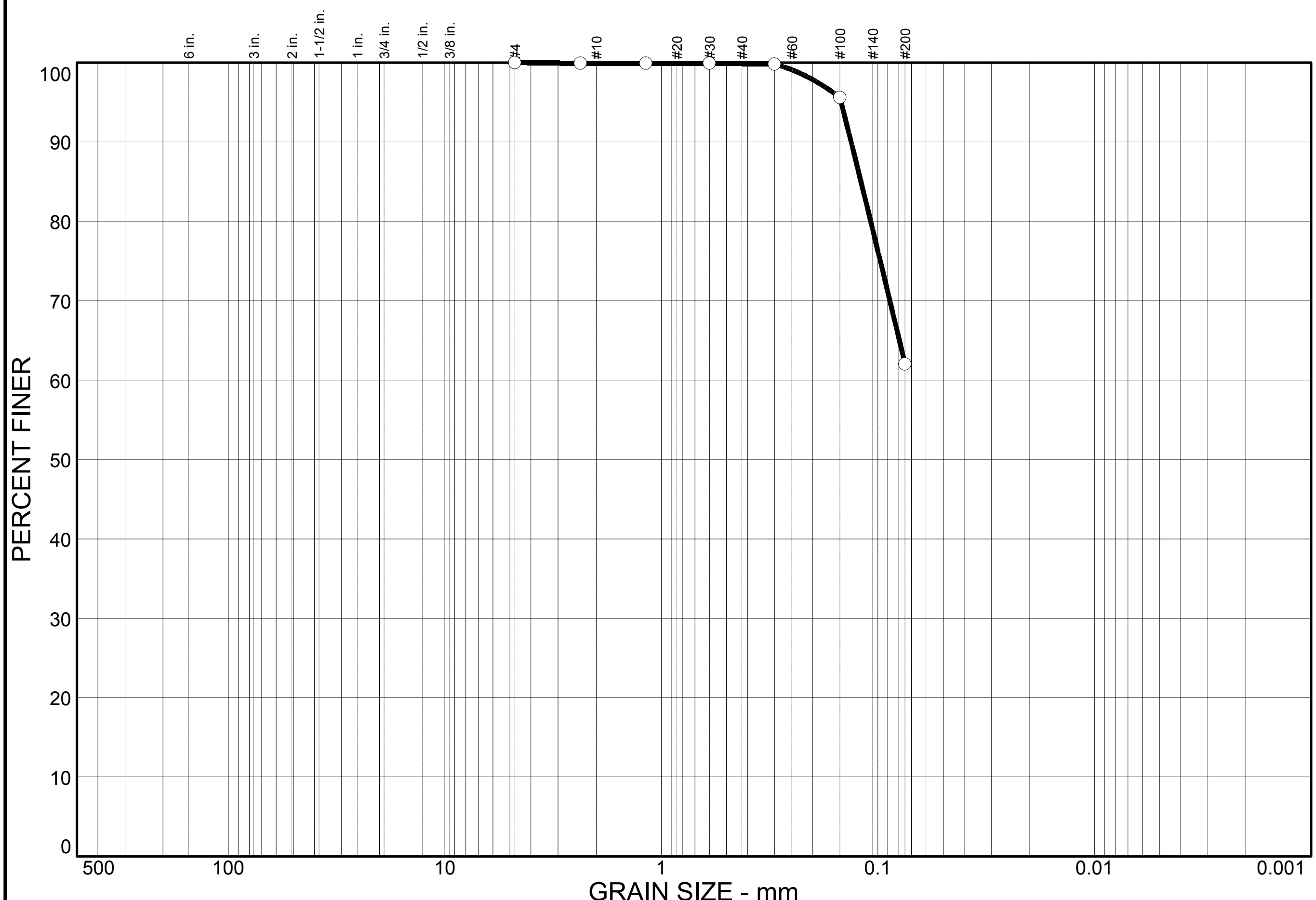
Sample No.: HS-2 **Source of Sample:** **Date:** 6/2/21
Location: **Elev./Depth:** 30-31.5'

Moore Twining Associates, Inc.

Fresno, CA

Client: Geosyntec Consultans
Project: Proposed Gas Operations Control Building, Pico Rivera
Project No: C73111.01 **Figure**

Particle Size Distribution Report



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	0.0	38.0	62.0	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#4	100.0		
#8	99.9		
#16	99.9		
#30	99.9		
#50	99.8		
#100	95.6		
#200	62.0		

* (no specification provided)

Material Description

Sandy Silt

Atterberg Limits

PL= LL= PI=

Coefficients

D_{85}= 0.120 D_{60}= D_{50}=
D_{30}= D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= ML AASHTO=

Remarks

Sample No.: HS-2 **Source of Sample:** **Date:** 6/2/21
Location: **Elev./Depth:** 35-35.75'

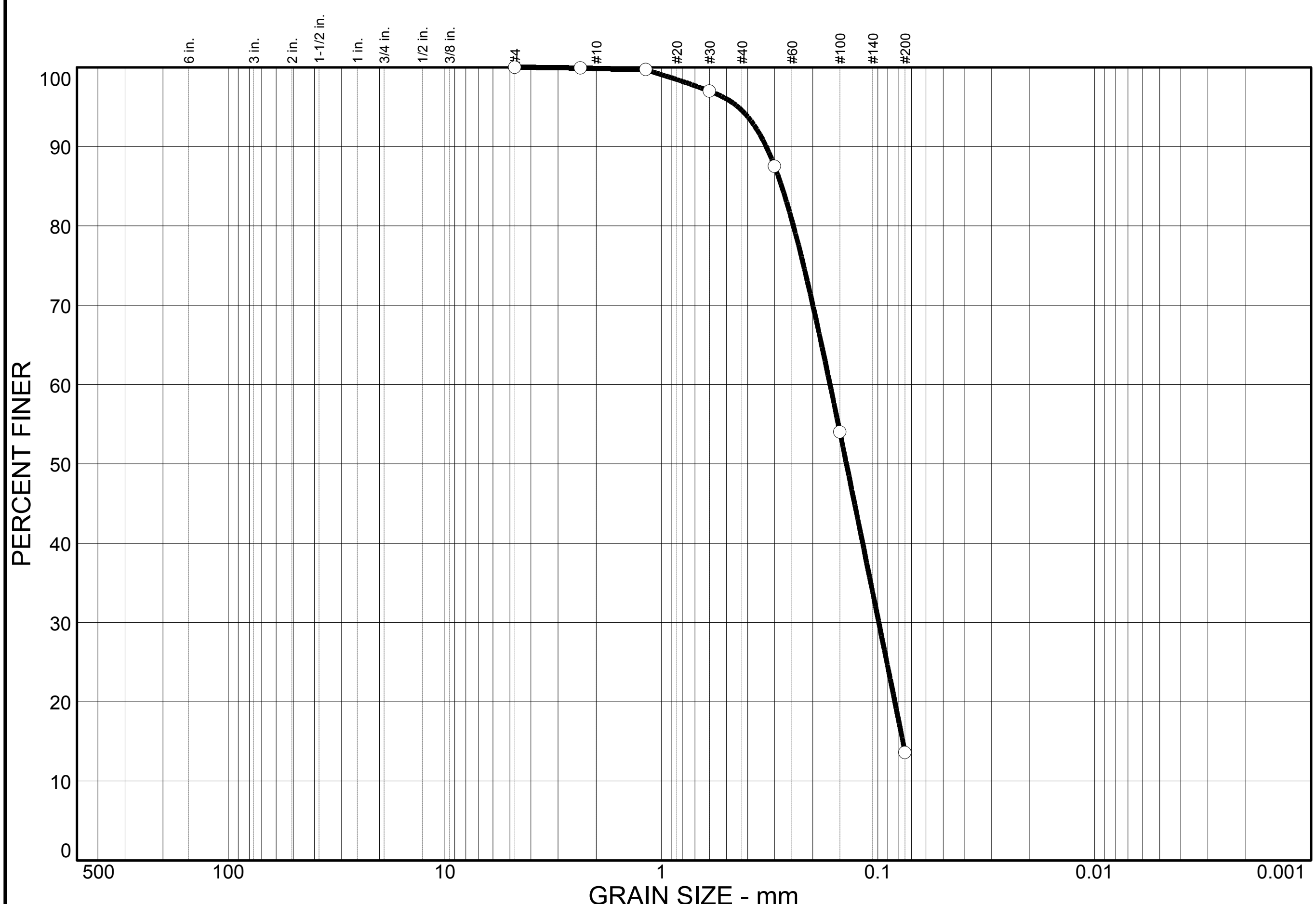
Moore Twining Associates, Inc.

Fresno, CA

Client: Geosyntec Consultans
Project: Proposed Gas Operations Control Building, Pico Rivera

Project No: C73111.01 **Figure**

Particle Size Distribution Report



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	0.0	86.4	13.6	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#4	100.0		
#8	99.9		
#16	99.7		
#30	97.0		
#50	87.5		
#100	54.0		
#200	13.6		

* (no specification provided)

Material Description

Silty sand

Atterberg Limits

PL= LL= PI=

Coefficients

D_{85}= 0.279 D_{60}= 0.167 D_{50}= 0.140
D_{30}= 0.0992 D_{15}= 0.0768 D_{10}=
C_u= C_c=

Classification

USCS= SM AASHTO=

Remarks

Sample No.: HS-2 **Source of Sample:** **Date:** 6/2/21
Location: **Elev./Depth:** 35.75-36.5'

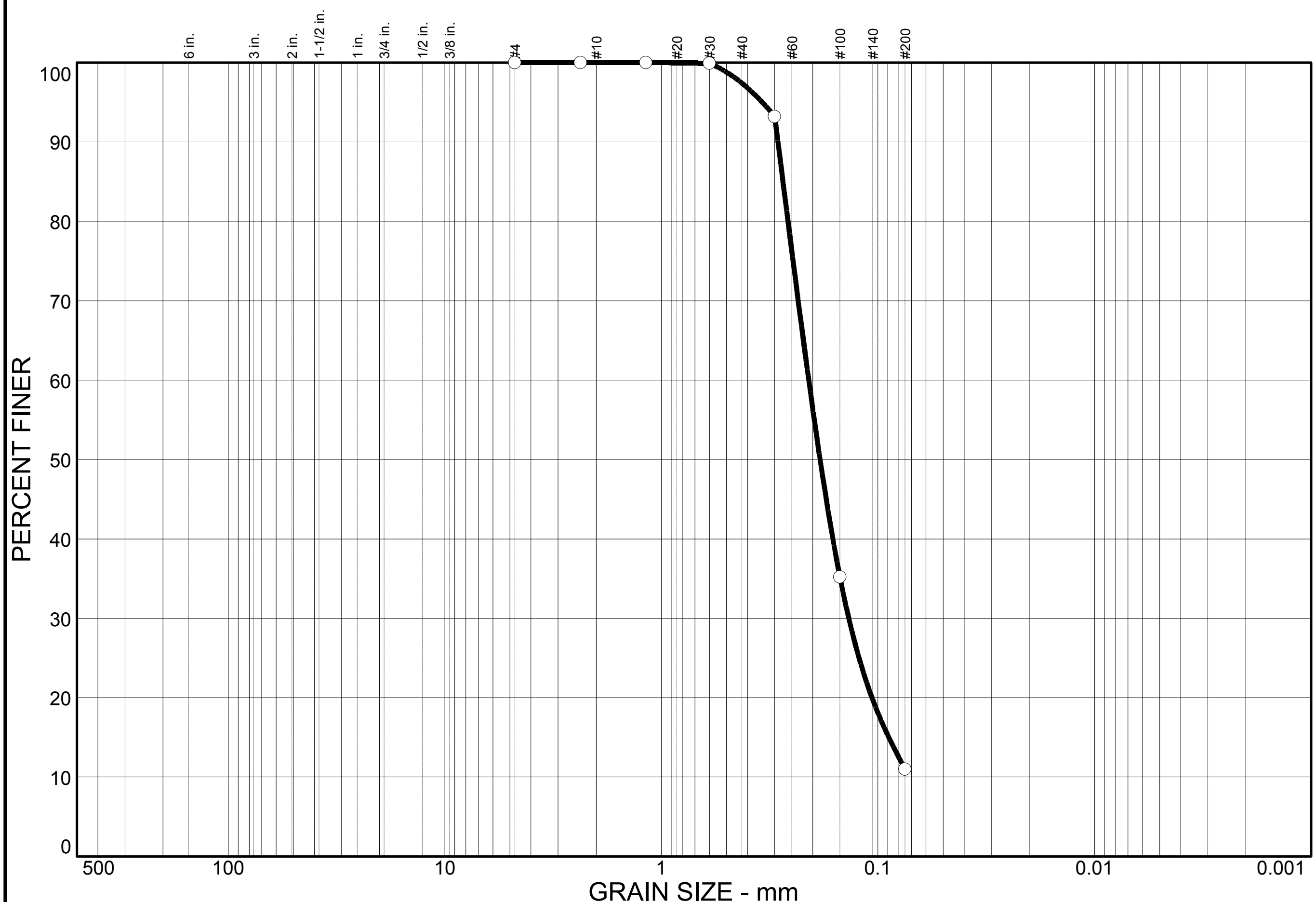
Moore Twining Associates, Inc.

Fresno, CA

Client: Geosyntec Consultans
Project: Proposed Gas Operations Control Building, Pico Rivera

Project No: C73111.01 **Figure**

Particle Size Distribution Report



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	0.0	89.0	11.0	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#4	100.0		
#8	100.0		
#16	100.0		
#30	99.9		
#50	93.2		
#100	35.2		
#200	11.0		

* (no specification provided)

Material Description

Poorly graded sand with silt

Atterberg Limits

PL= LL= PI=

Coefficients

D_{85}= 0.275 D_{60}= 0.209 D_{50}= 0.185
D_{30}= 0.137 D_{15}= 0.0892 D_{10}=
C_u= C_c=

Classification

USCS= SP-SM AASHTO=

Remarks

Sample No.: HS-2 **Source of Sample:** **Date:** 6/2/21
Location: **Elev./Depth:** 40-41.5'

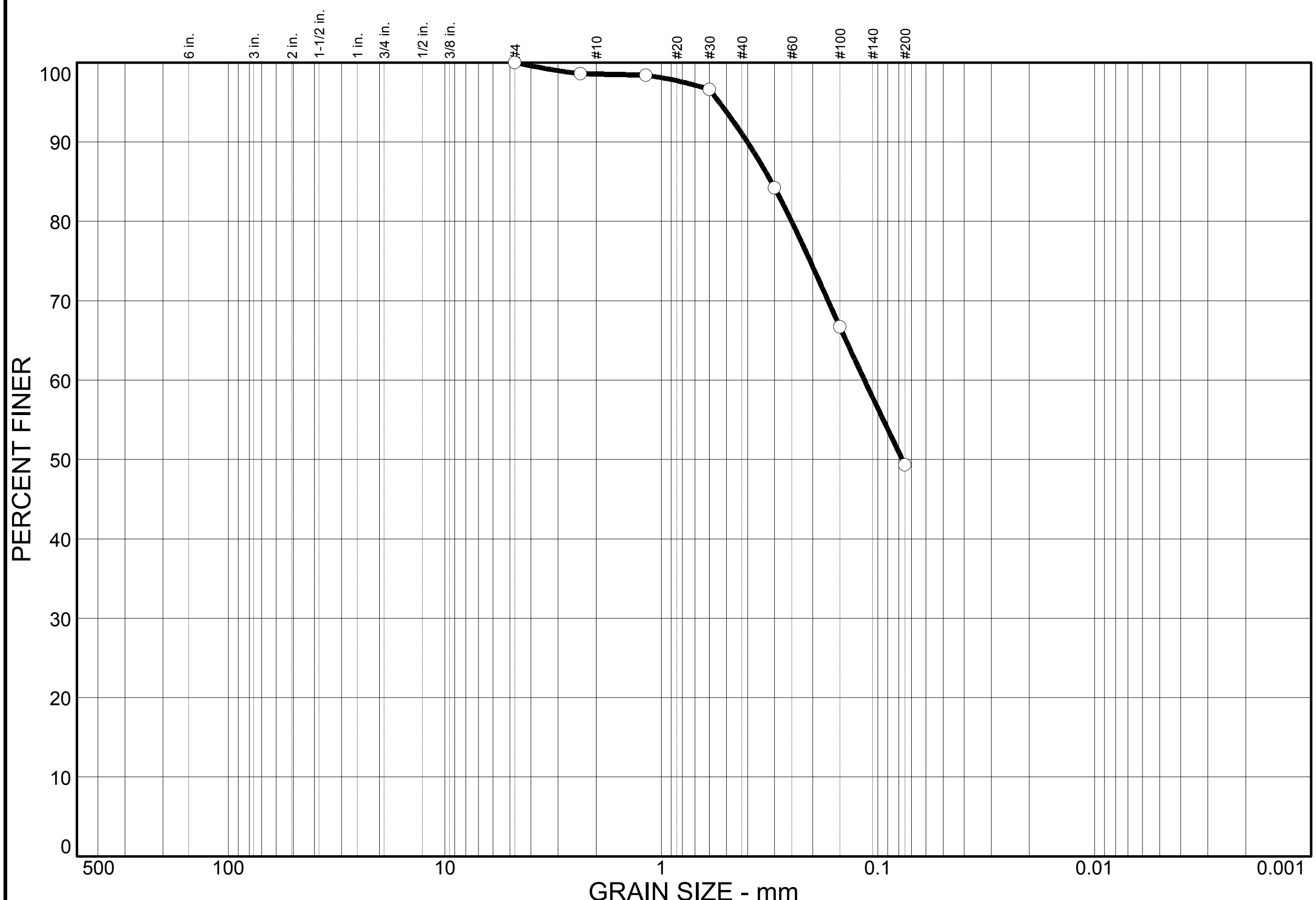
Moore Twining Associates, Inc.

Fresno, CA

Client: Geosyntec Consultans
Project: Proposed Gas Operations Control Building, Pico Rivera

Project No: C73111.01 **Figure**

Particle Size Distribution Report



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	0.0	50.7	49.3	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#4	100.0		
#8	98.6		
#16	98.4		
#30	96.6		
#50	84.2		
#100	66.7		
#200	49.3		

* (no specification provided)

Material Description

Silty sand

Atterberg Limits

PL= LL= PI=

Coefficients

D_{85}= 0.311 D_{60}= 0.115 D_{50}= 0.0772
D_{30}= D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= SM AASHTO=

Remarks

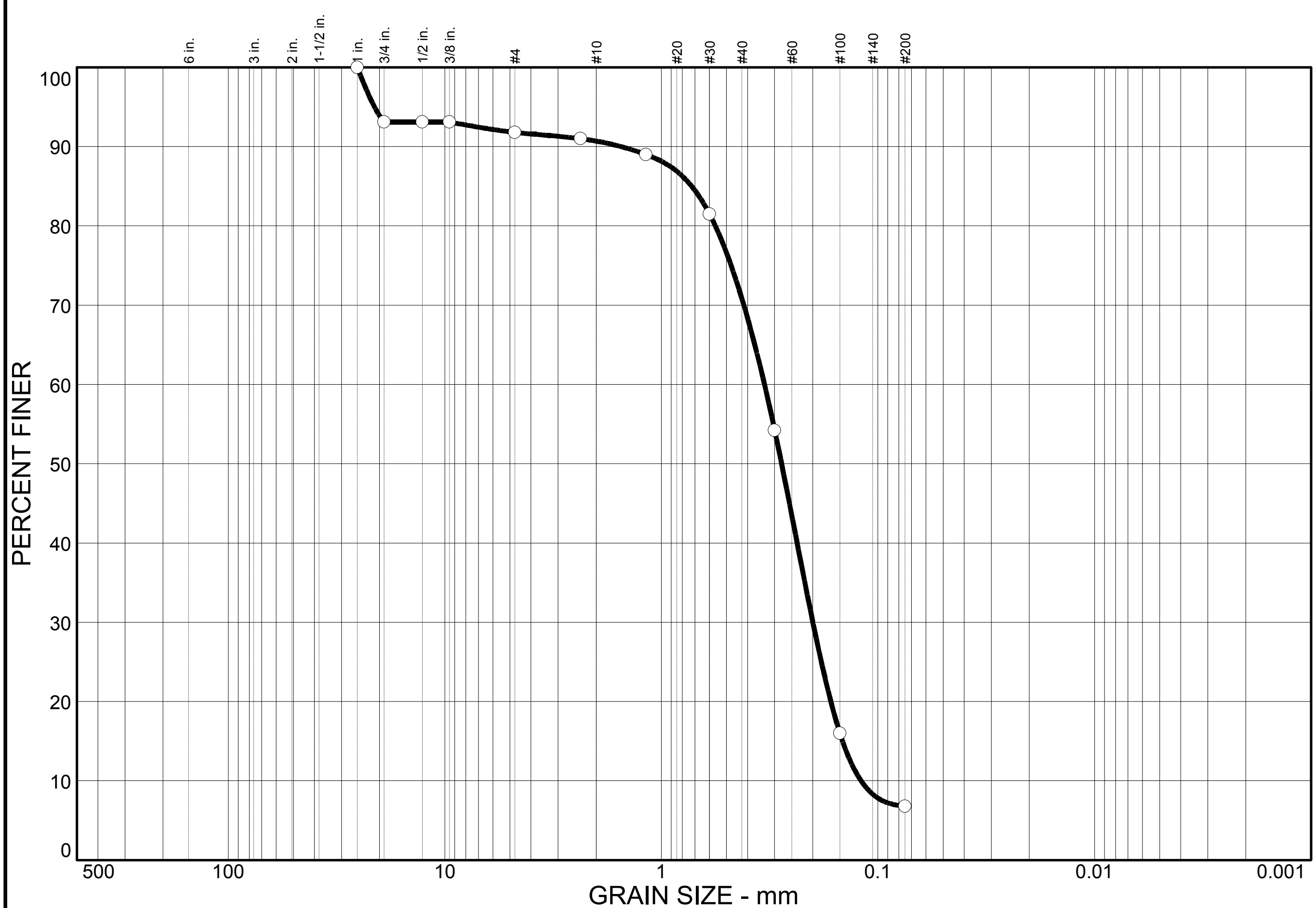
Sample No.: HS-2 **Source of Sample:** **Date:** 6/2/21
Location: **Elev./Depth:** 48.5-49.25'

Moore Twining Associates, Inc.

Fresno, CA

Client: Geosyntec Consultans
Project: Proposed Gas Operations Control Building, Pico Rivera
Project No: C73111.01 **Figure**

Particle Size Distribution Report



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	8.2	85.0	6.8	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1 in.	100.0		
3/4 in.	93.1		
1/2 in.	93.1		
3/8 in.	93.1		
#4	91.8		
#8	91.0		
#16	89.0		
#30	81.5		
#50	54.2		
#100	16.0		
#200	6.8		

* (no specification provided)

Material Description

Poorly graded sand with silt

Atterberg Limits

PL= LL= PI=

Coefficients

D_{85}= 0.726 D_{60}= 0.334 D_{50}= 0.279
D_{30}= 0.200 D_{15}= 0.146 D_{10}= 0.120
C_u= 2.79 C_c= 1.00

Classification

USCS= SP-SM AASHTO=

Remarks

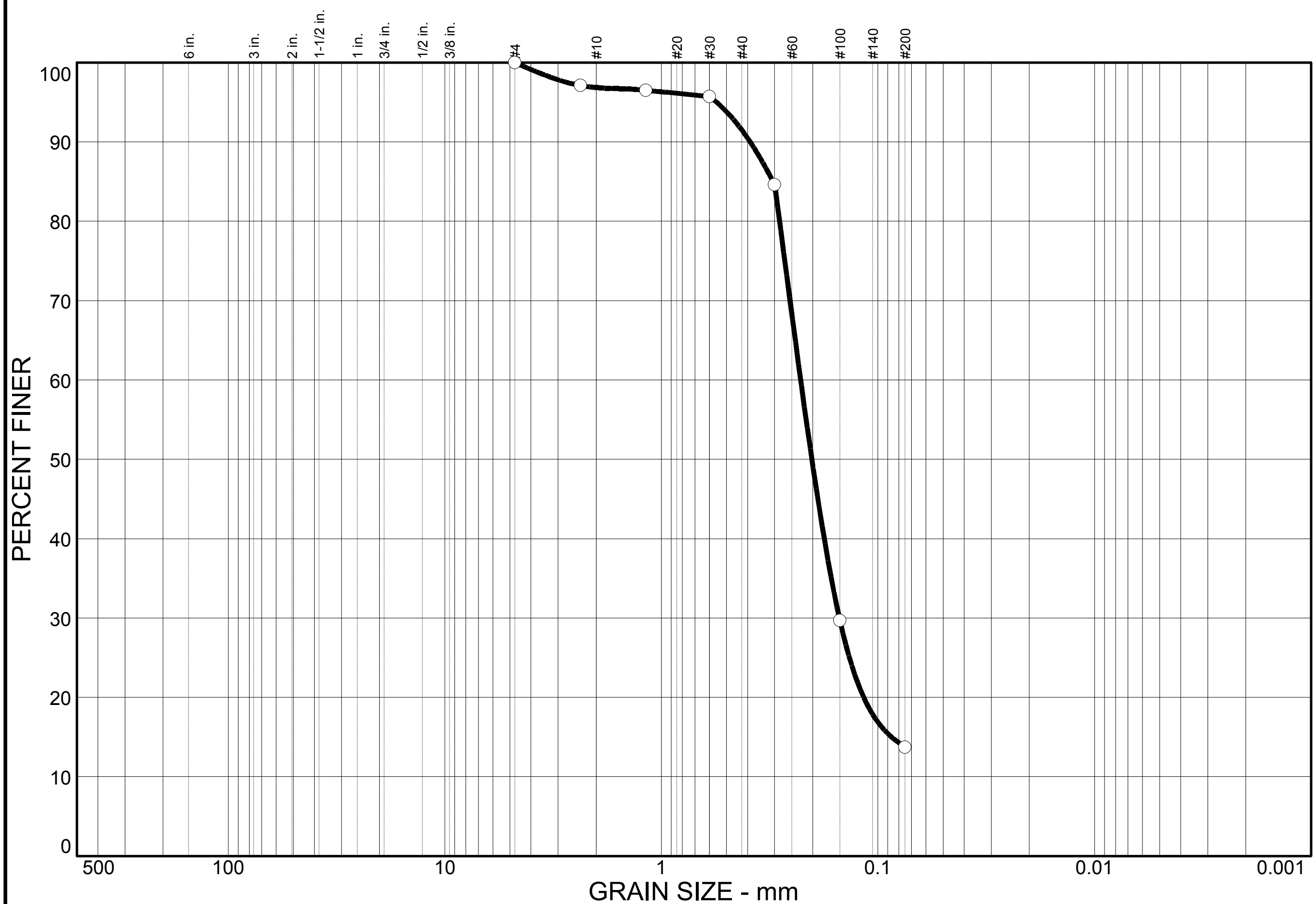
Sample No.: HS-2 **Source of Sample:** **Date:** 6/2/21
Location: **Elev./Depth:** 49.25-50'

Moore Twining Associates, Inc.

Fresno, CA

Client: Geosyntec Consultans
Project: Proposed Gas Operations Control Building, Pico Rivera
Project No: C73111.01 **Figure**

Particle Size Distribution Report



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	0.0	86.3	13.7	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#4	100.0		
#8	97.1		
#16	96.5		
#30	95.7		
#50	84.6		
#100	29.7		
#200	13.7		

* (no specification provided)

Material Description

Silty sand

Atterberg Limits

PL= LL= PI=

Coefficients

D_{85}= 0.305 D_{60}= 0.228 D_{50}= 0.202
D_{30}= 0.151 D_{15}= 0.0866 D_{10}=
C_u= C_c=

Classification

USCS= SM AASHTO=

Remarks

Sample No.: HS-3 **Source of Sample:** **Date:** 6/2/21
Location: **Elev./Depth:** 1.5-5'

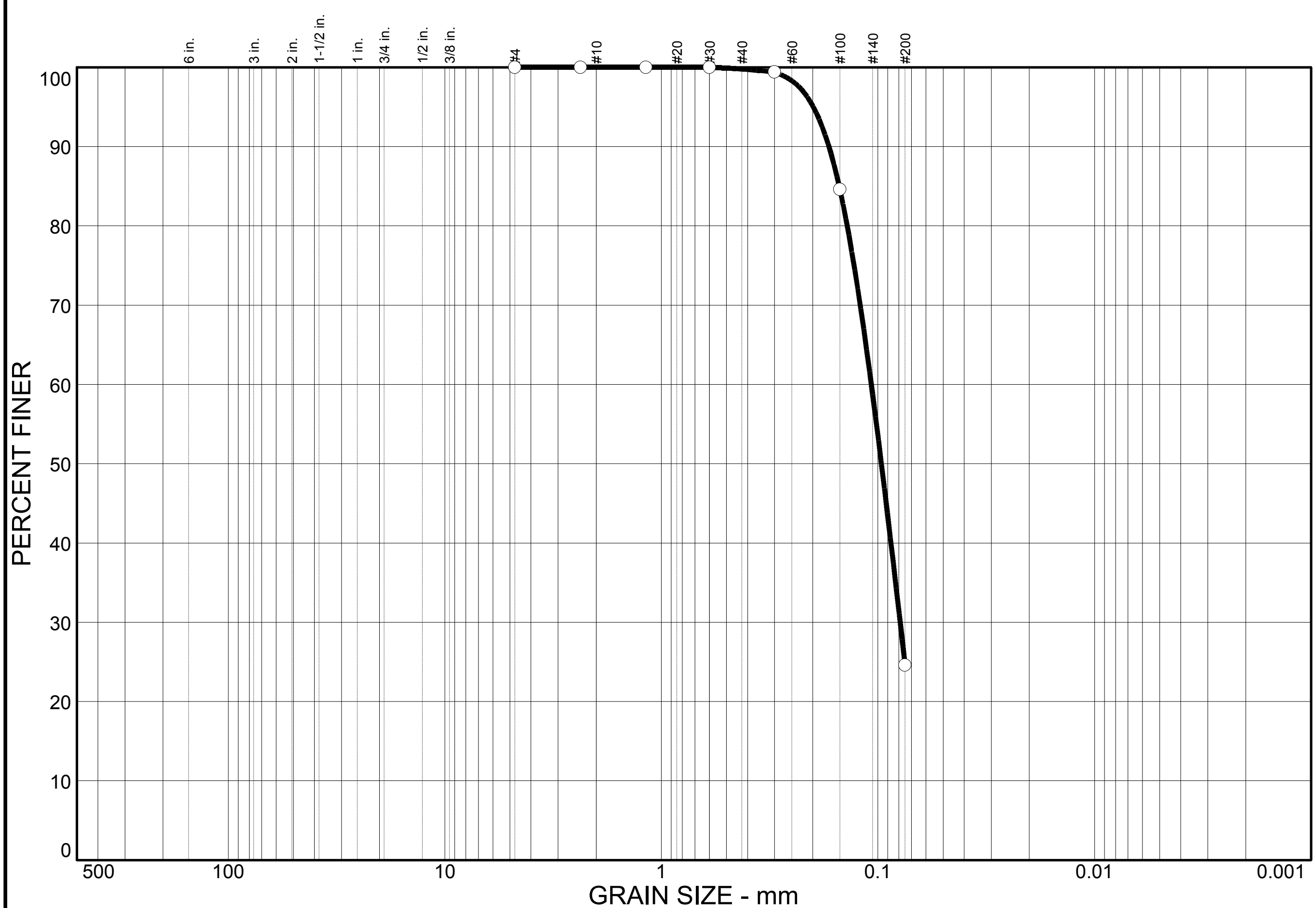
Moore Twining Associates, Inc.

Fresno, CA

Client: Geosyntec Consultans
Project: Proposed Gas Operations Control Building, Pico Rivera

Project No: C73111.01 **Figure**

Particle Size Distribution Report



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	0.0	75.4	24.6	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#4	100.0		
#8	100.0		
#16	100.0		
#30	100.0		
#50	99.4		
#100	84.6		
#200	24.6		

* (no specification provided)

Material Description

Silty sand

Atterberg Limits

PL= LL= PI=

Coefficients

D_{85}= 0.151 D_{60}= 0.107 D_{50}= 0.0965
D_{30}= 0.0790 D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= SM AASHTO=

Remarks

Sample No.: HS-3 **Source of Sample:** **Date:** 6/2/21
Location: **Elev./Depth:** 20-21.5'

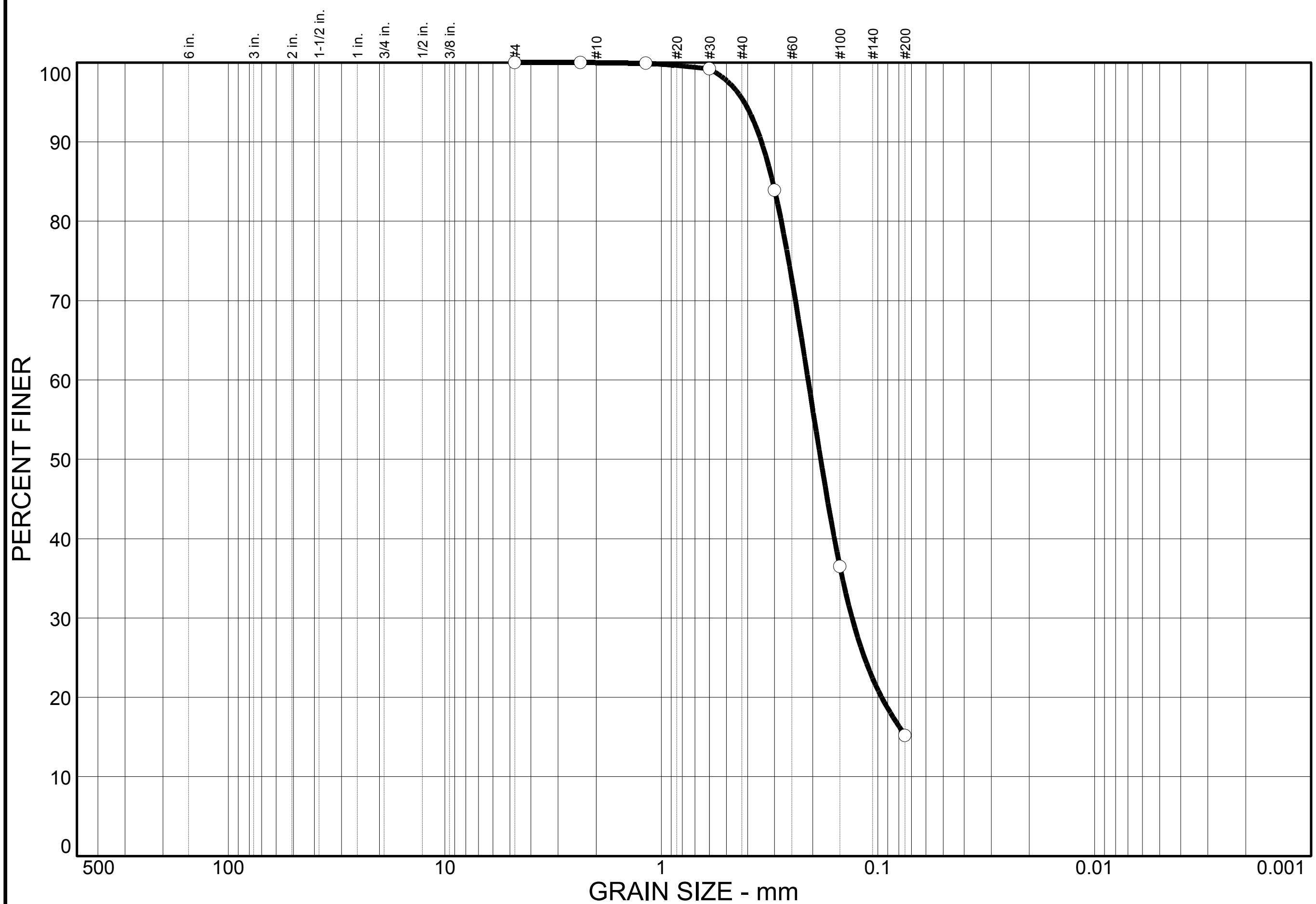
Moore Twining Associates, Inc.

Fresno, CA

Client: Geosyntec Consultans
Project: Proposed Gas Operations Control Building, Pico Rivera

Project No: C73111.01 **Figure**

Particle Size Distribution Report



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	0.0	84.8	15.2	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#4	100.0		
#8	100.0		
#16	99.9		
#30	99.2		
#50	83.9		
#100	36.5		
#200	15.2		

* (no specification provided)

Material Description

Silty sand

Atterberg Limits

PL= LL= PI=

Coefficients

D_{85}= 0.307 D_{60}= 0.210 D_{50}= 0.184
D_{30}= 0.132 D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= SM AASHTO=

Remarks

Sample No.: HS-4 **Source of Sample:** **Date:** 6/2/21
Location: **Elev./Depth:** 2.5-4'

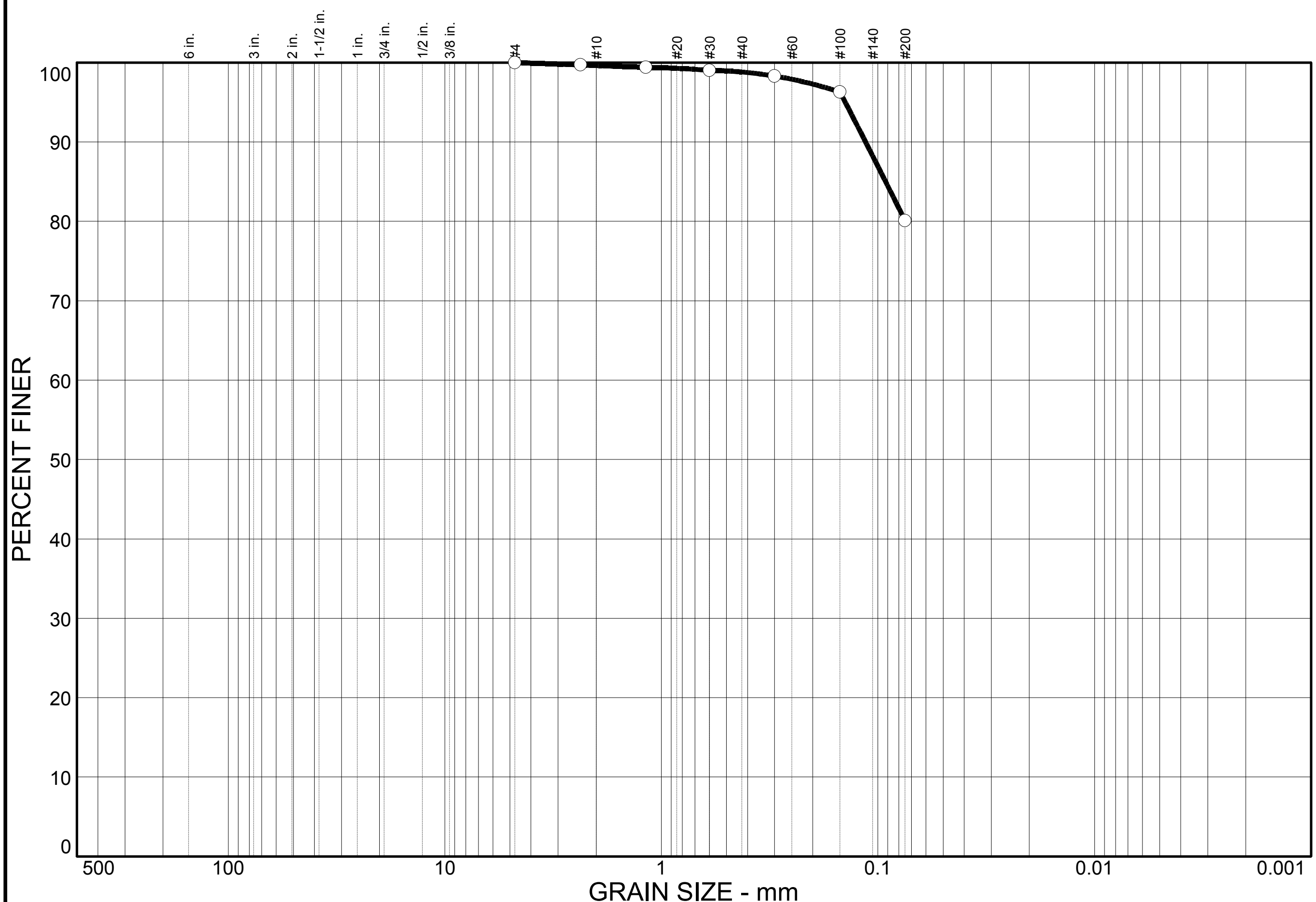
Moore Twining Associates, Inc.

Fresno, CA

Client: Geosyntec Consultans
Project: Proposed Gas Operations Control Building, Pico Rivera

Project No: C73111.01 **Figure**

Particle Size Distribution Report



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	0.0	19.9	80.1	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#4	100.0		
#8	99.7		
#16	99.4		
#30	99.0		
#50	98.3		
#100	96.3		
#200	80.1		

* (no specification provided)

Material Description

Silt with sand

Atterberg Limits

PL= LL= PI=

Coefficients

D_{85}= 0.0923 D_{60}= D_{50}=
D_{30}= D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= ML AASHTO=

Remarks

Sample No.: HS-4 **Source of Sample:** **Date:** 6/2/21
Location: **Elev./Depth:** 5.25-6.5'

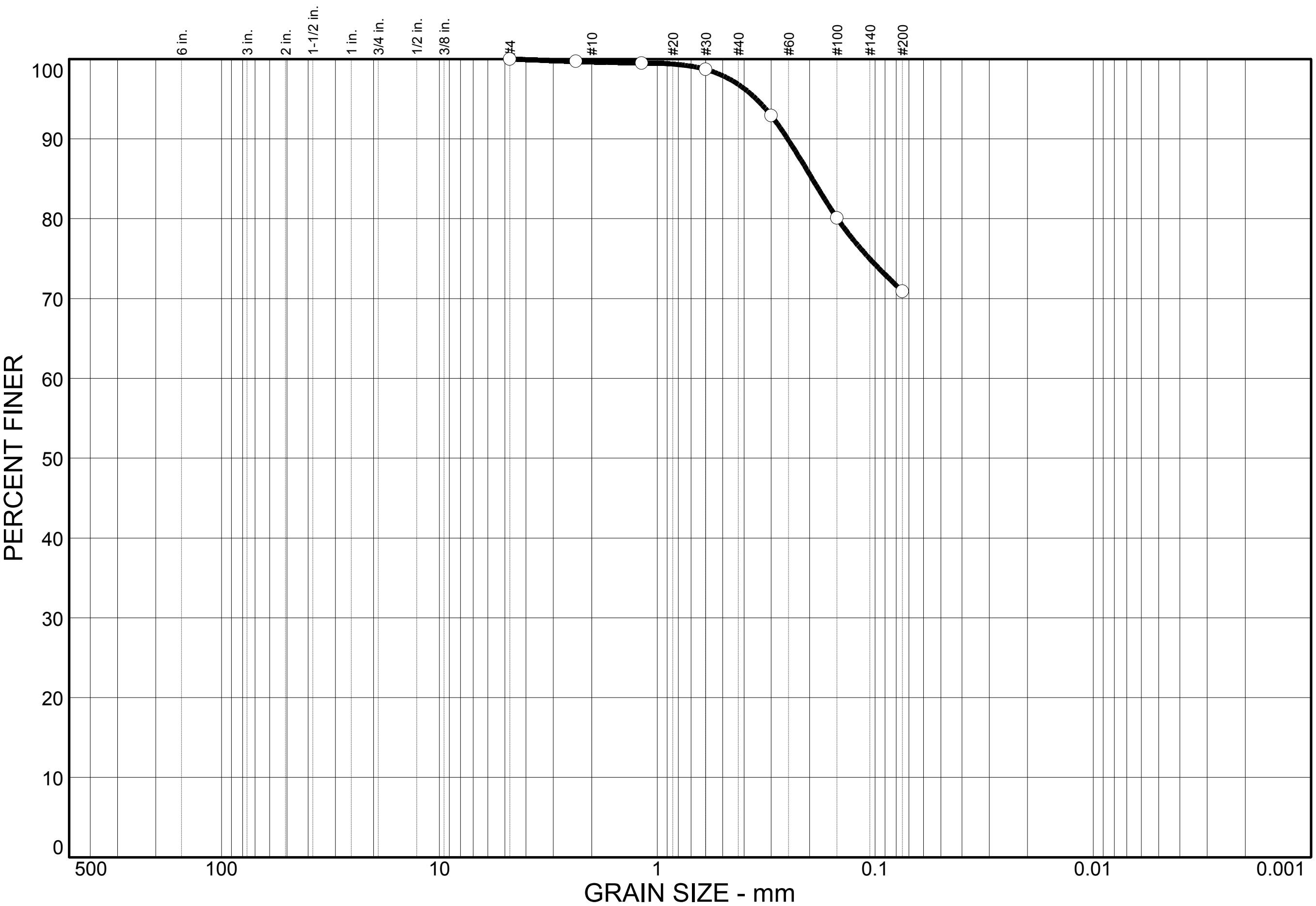
Moore Twining Associates, Inc.

Fresno, CA

Client: Geosyntec Consultans
Project: Proposed Gas Operations Control Building, Pico Rivera

Project No: C73111.01 **Figure**

Particle Size Distribution Report



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	0.0	29.1	70.9	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#4	100.0		
#8	99.7		
#16	99.5		
#30	98.7		
#50	92.9		
#100	80.1		
#200	70.9		

* (no specification provided)

Material Description

Silt with sand

Atterberg Limits

PL= NP LL= NV PI= NP

Coefficients

D_{85}= 0.195 D_{60}= D_{50}=
D_{30}= D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= ML AASHTO=

Remarks

Sample No.: HS-5 **Source of Sample:** **Date:** 6/2/21
Location: **Elev./Depth:** 7.5-9'

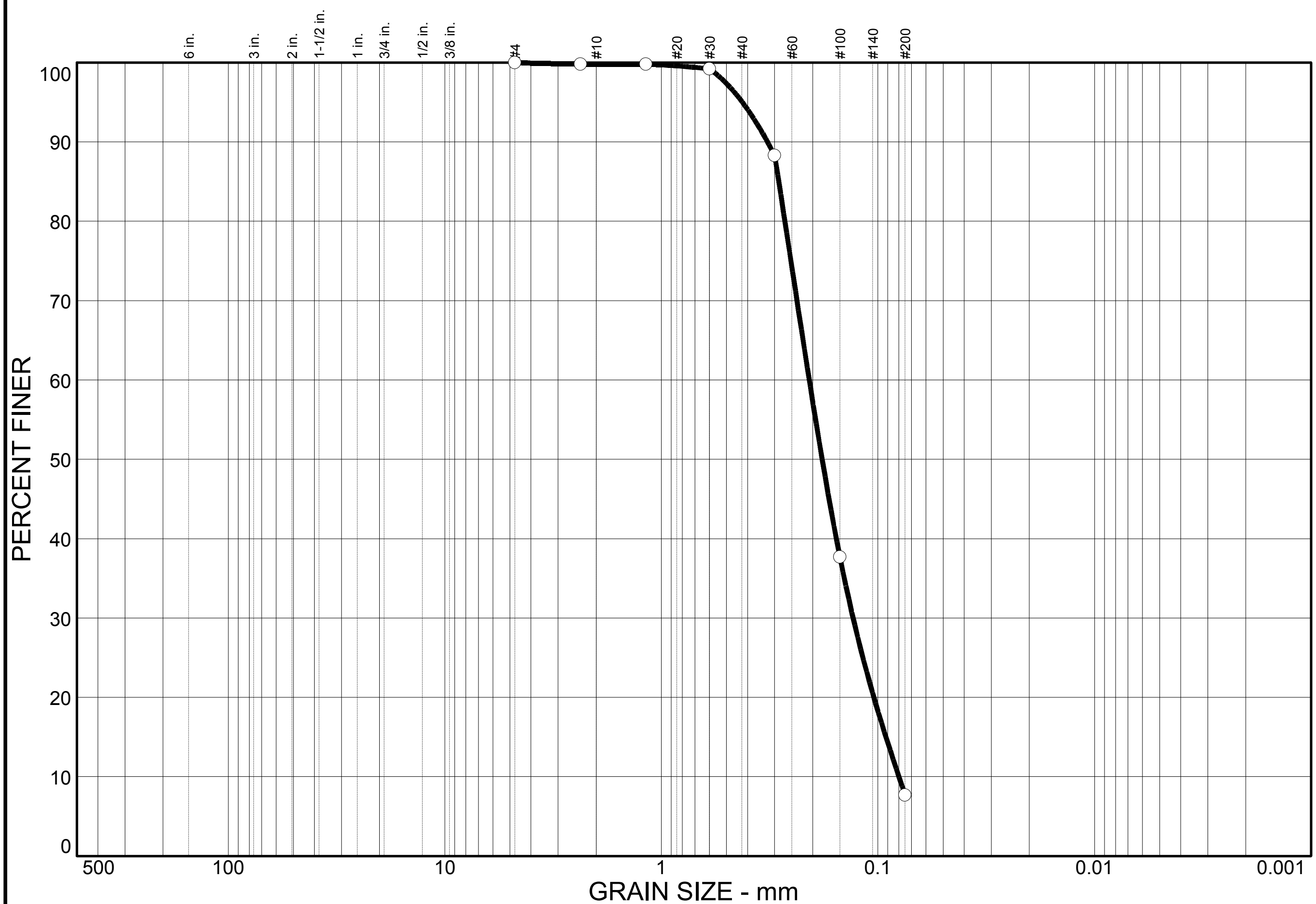
Moore Twining Associates, Inc.

Fresno, CA

Client: Geosyntec Consultans
Project: Proposed Gas Operations Control Building, Pico Rivera

Project No: C73111.01 **Figure**

Particle Size Distribution Report



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	0.0	92.3	7.7	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#4	100.0		
#8	99.8		
#16	99.8		
#30	99.2		
#50	88.3		
#100	37.7		
#200	7.7		

* (no specification provided)

Material Description

Poorly graded sand with silt

Atterberg Limits

PL= LL= PI=

Coefficients

D_{85}= 0.287 D_{60}= 0.208 D_{50}= 0.181
D_{30}= 0.130 D_{15}= 0.0919 D_{10}= 0.0800
C_u= 2.60 C_c= 1.02

Classification

USCS= SP-SM AASHTO=

Remarks

Sample No.: P-1 **Source of Sample:** **Date:** 6/2/21
Location: **Elev./Depth:** 4-5'

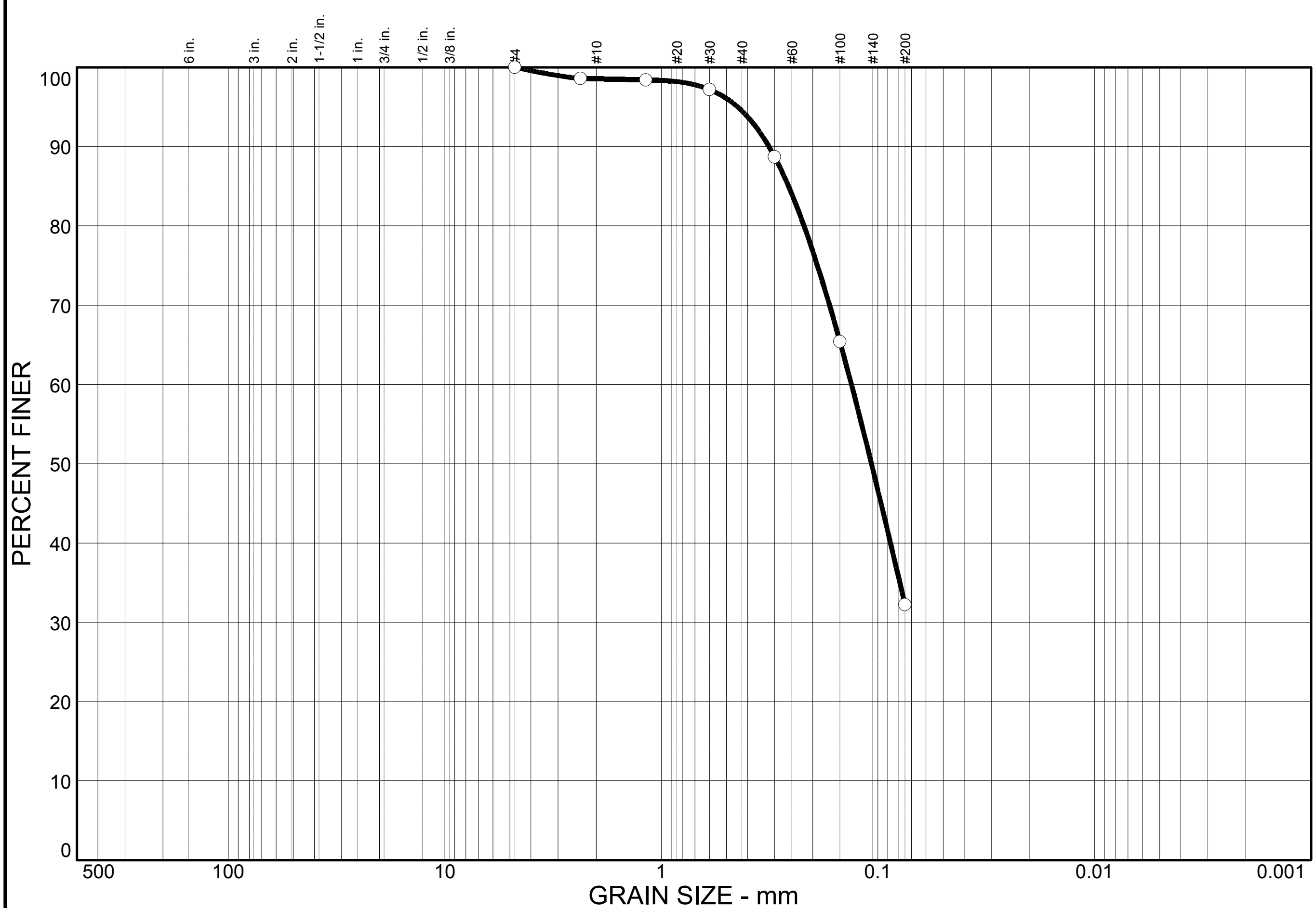
Moore Twining Associates, Inc.

Fresno, CA

Client: Geosyntec Consultans
Project: Proposed Gas Operations Control Building, Pico Rivera

Project No: C73111.01 **Figure**

Particle Size Distribution Report



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	0.0	67.8	32.2	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#4	100.0		
#8	98.6		
#16	98.4		
#30	97.2		
#50	88.7		
#100	65.4		
#200	32.2		

* (no specification provided)

Material Description

Silty sand

Atterberg Limits

PL= LL= PI=

Coefficients

D_{85}= 0.259 D_{60}= 0.133 D_{50}= 0.107
D_{30}= D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= SM AASHTO=

Remarks

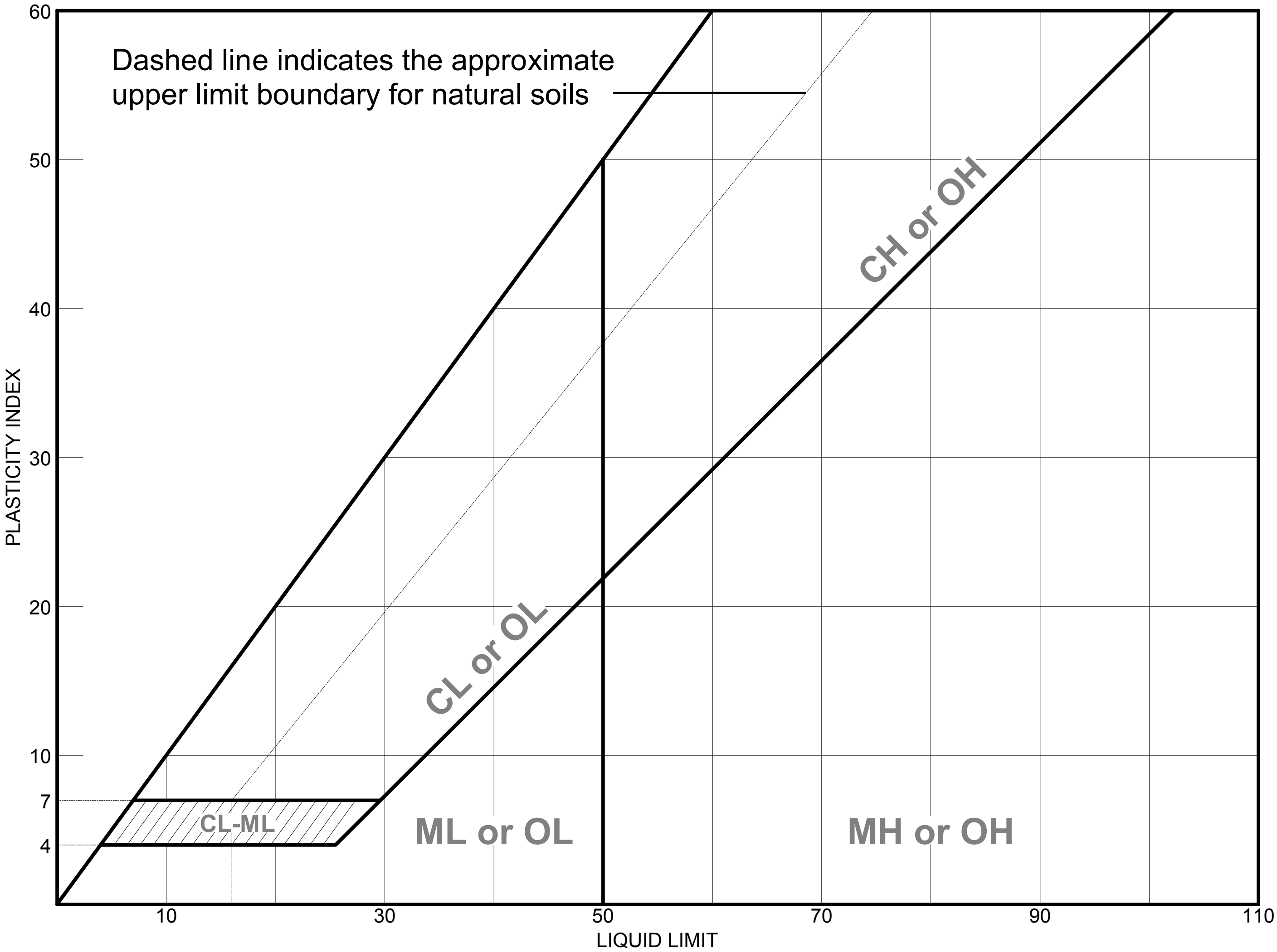
Sample No.: P-2 **Source of Sample:** **Date:** 6/2/21
Location: **Elev./Depth:** 2-3.5'

Moore Twining Associates, Inc.

Fresno, CA

Client: Geosyntec Consultans
Project: Proposed Gas Operations Control Building, Pico Rivera
Project No: C73111.01 **Figure**

LIQUID AND PLASTIC LIMITS TEST REPORT



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	Sandy silt	NV	NP	NP	99.2	67.9	ML

Project No. C73111.01 **Client:** Geosyntec Consultans

Project: Proposed Gas Operations Control Building, Pico Rivera

● **Source:** **Sample No.:** HS-2 **Elev./Depth:** 30-31.5'

Remarks:

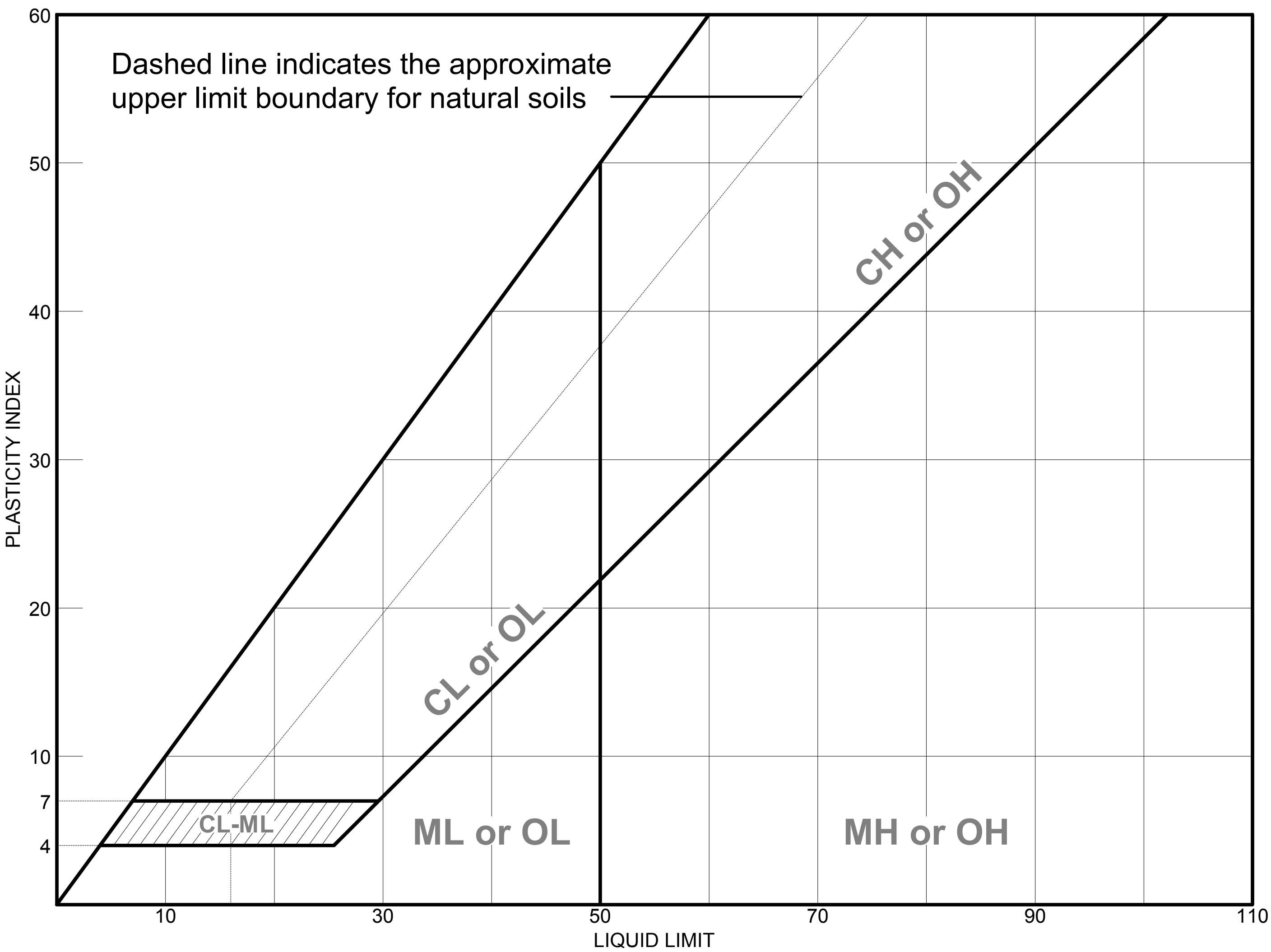
●

Moore Twining Associates, Inc.

Fresno, CA

Figure

LIQUID AND PLASTIC LIMITS TEST REPORT



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	Silt with Sand	NV	NP	NP			ML

Project No. C73111.01 **Client:** Geosyntec Consultans

Project: Proposed Gas Operations Control Building, Pico Rivera

● **Source:** **Sample No.:** HS-2 **Elev./Depth:** 46.25-46.5'

Moore Twining Associates, Inc.

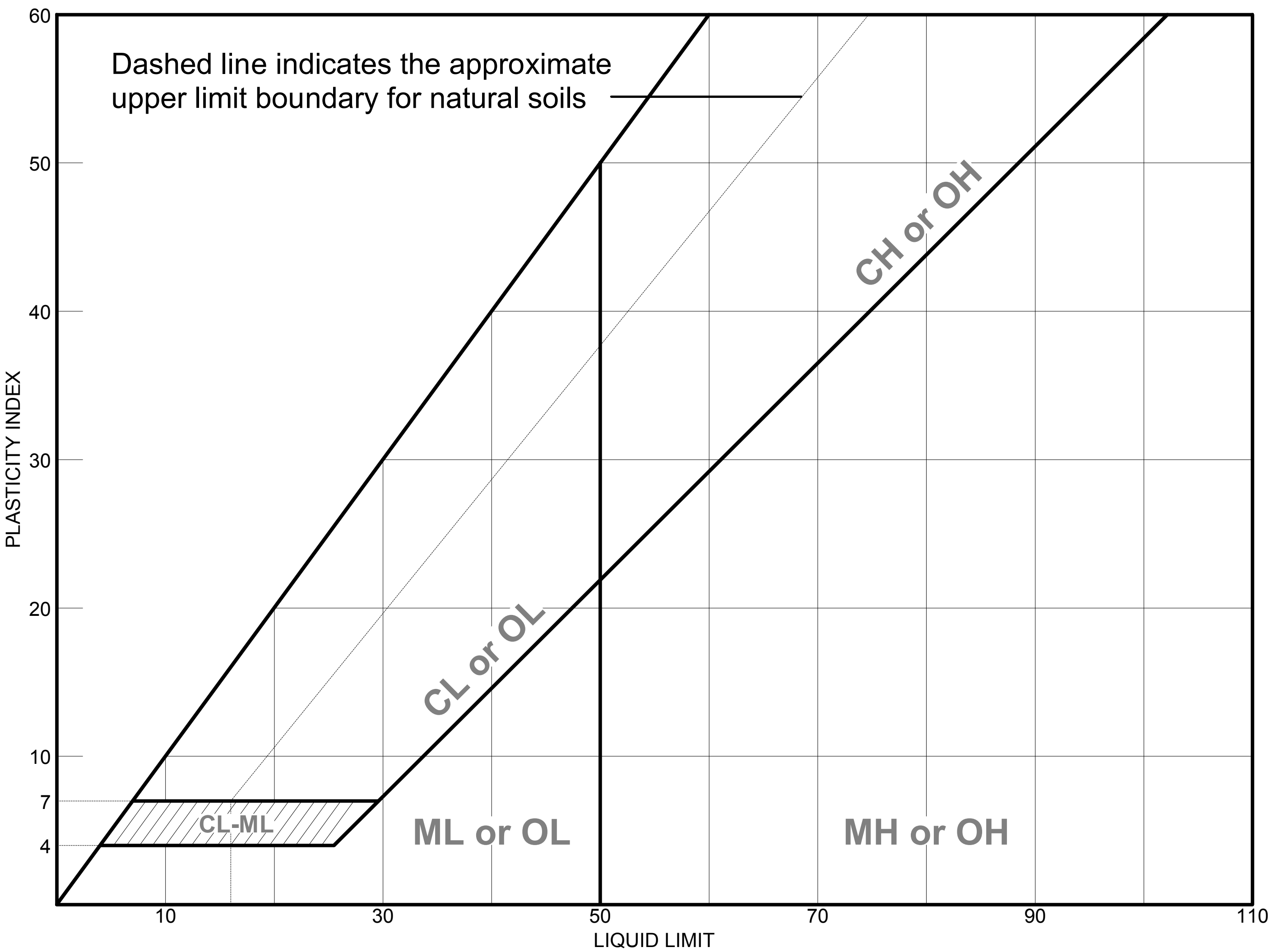
Fresno, CA

Remarks:

●

Figure

LIQUID AND PLASTIC LIMITS TEST REPORT



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	Silt with sand	NV	NP	NP			ML

Project No. C73111.01 **Client:** Geosyntec Consultans

Project: Proposed Gas Operations Control Building, Pico Rivera

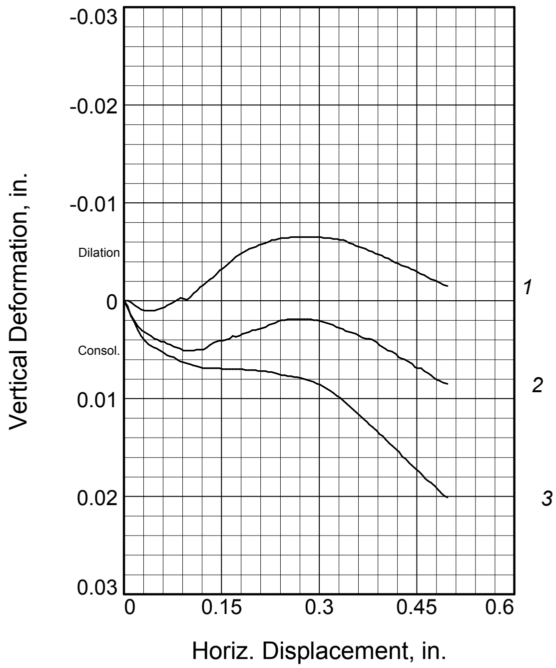
● **Source:** **Sample No.:** HS-5 **Elev./Depth:** 7.5-9'

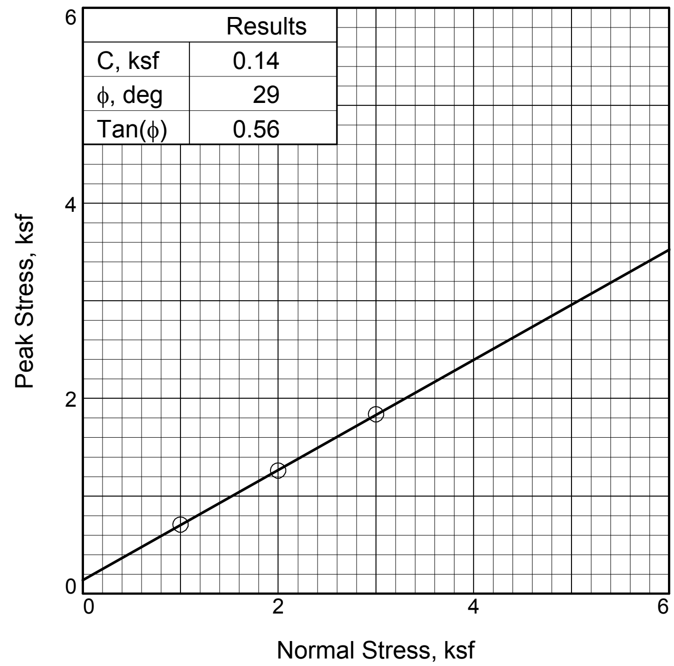
Remarks:
●

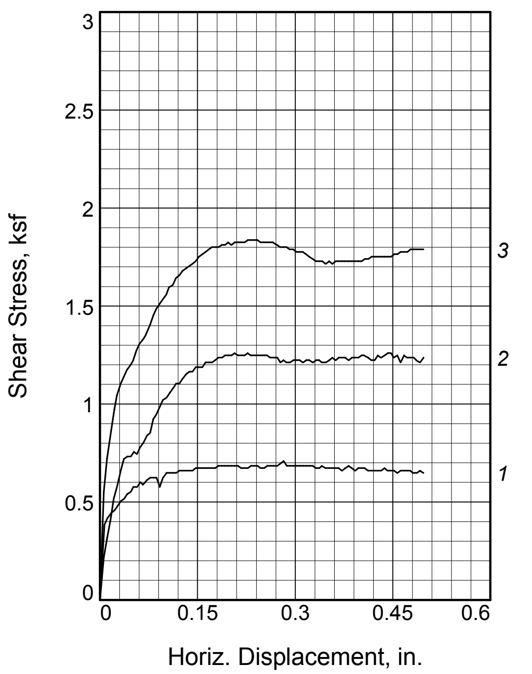
Moore Twining Associates, Inc.

Fresno, CA

Figure







	Sample No.	1	2	3
Initial	Water Content, %	9.8	20.1	14.1
	Dry Density, pcf	83.9	77.4	80.8
	Saturation, %	26.7	46.8	35.6
	Void Ratio	0.9711	1.1361	1.0482
	Diameter, in.	2.42	2.42	2.42
	Height, in.	1.00	1.00	1.00
At Test	Water Content, %	34.7	39.1	33.6
	Dry Density, pcf	85.0	80.0	85.8
	Saturation, %	97.0	97.0	96.1
	Void Ratio	0.9469	1.0666	0.9273
	Diameter, in.	2.42	2.42	2.42
	Height, in.	0.99	0.97	0.94
	Normal Stress, ksf	1.00	2.00	3.00
	Peak Stress, ksf	0.71	1.26	1.84
	Displacement, in.	0.28	0.21	0.23
	Ultimate Stress, ksf			
	Displacement, in.			
	Strain at peak, %	11.7	8.6	9.4

Sample Type:

Description: Silty sand

Specific Gravity= 2.65

Remarks:

Figure ________

Client: Geosyntec Consultans

Project: Proposed Gas Operations Control Building, Pico Rivera

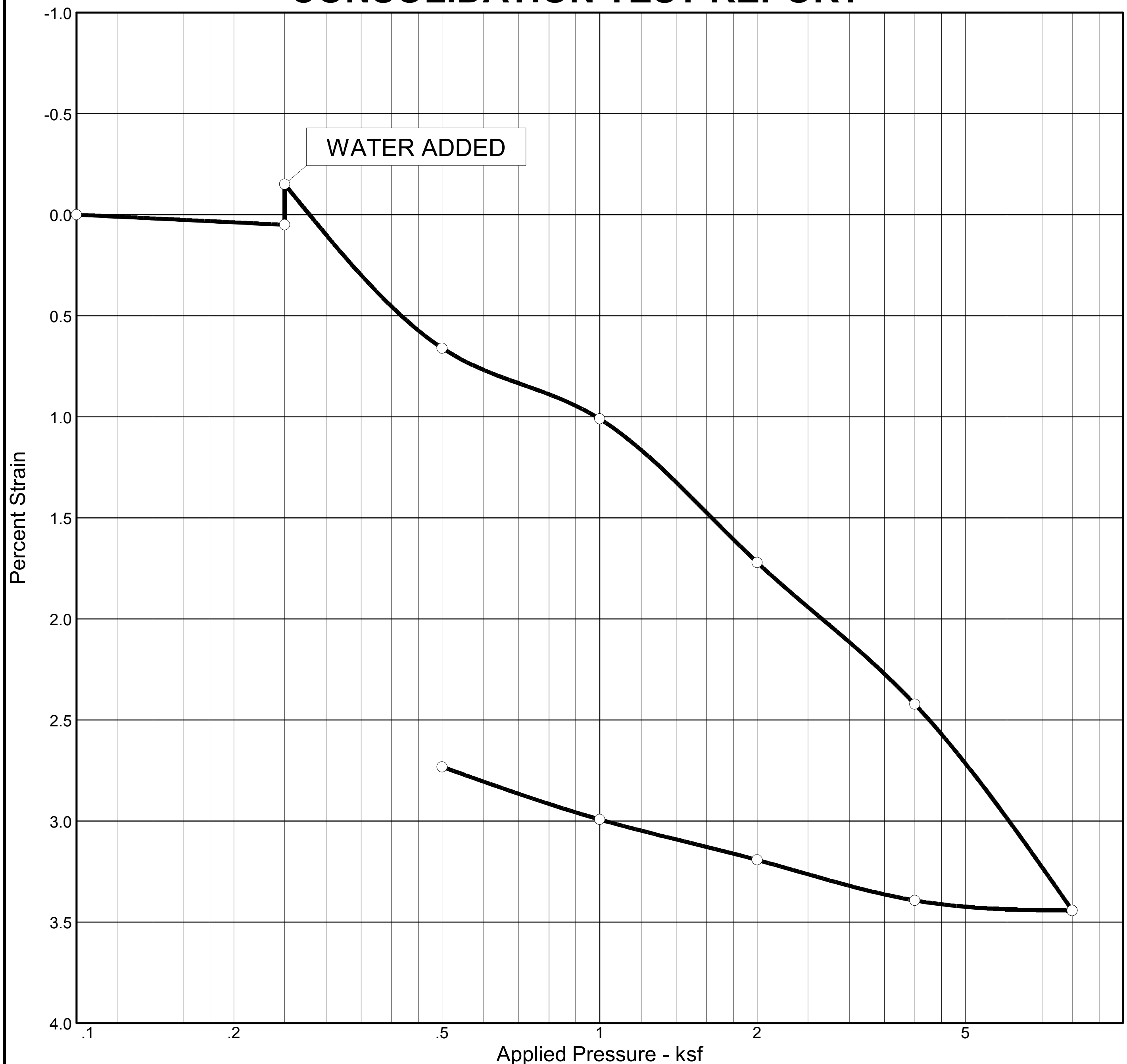
Sample Number: HS-2 **Depth:** 5-6.5'

Proj. No.: C73111.01 **Date Sampled:** 6/2/21

DIRECT SHEAR TEST REPORT

Moore Twining Associates, Inc.
Fresno, CA

CONSOLIDATION TEST REPORT



Natural Sat.	Natural Moist.	Dry Dens. (pcf)	LL	PI	Sp. Gr.	Overburden (ksf)	P_c (ksf)	C_c	C_s	Swell Press. (ksf)	Swell %	e_o
24.1 %	7.7 %	89.6			2.65		1.74	0.06	0.01	0.29	0.3	0.847

MATERIAL DESCRIPTION	USCS	AASHTO
Silty sand	SM	

Project No. C73111.01 **Client:** Geosyntec Consultans

Project: Proposed Gas Operations Control Building, Pico Rivera

Source: **Sample No.:** HS-2 **Elev./Depth:** 1.5-3'

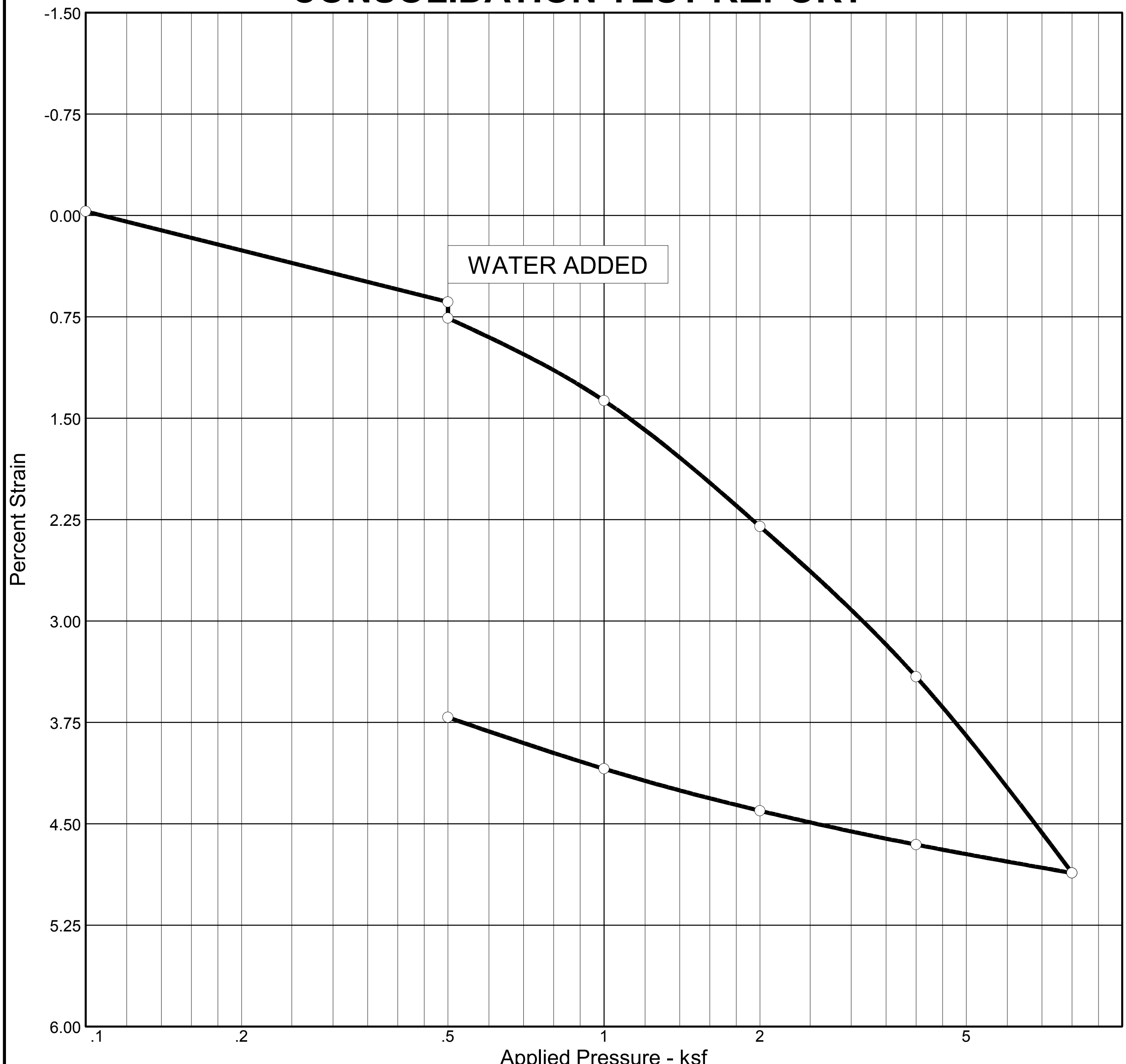
Moore Twining Associates, Inc.

Fresno, CA

Remarks:

Figure

CONSOLIDATION TEST REPORT



Natural		Dry Dens. (pcf)	LL	PI	Sp. Gr.	Overburden (ksf)	P_c (ksf)	C_c	C_s	Swell Press. (ksf)	Clpse. %	e_o
Sat.	Moist.											
30.7 %	10.8 %	85.7			2.65		1.83	0.09	0.02		0.2	0.931

MATERIAL DESCRIPTION	USCS	AASHTO
Silty sand	SM	

Project No. C73111.01 **Client:** Geosyntec Consultans

Project: Proposed Gas Operations Control Building, Pico Rivera

Source: **Sample No.:** HS-2 **Elev./Depth:** 5-6.5'

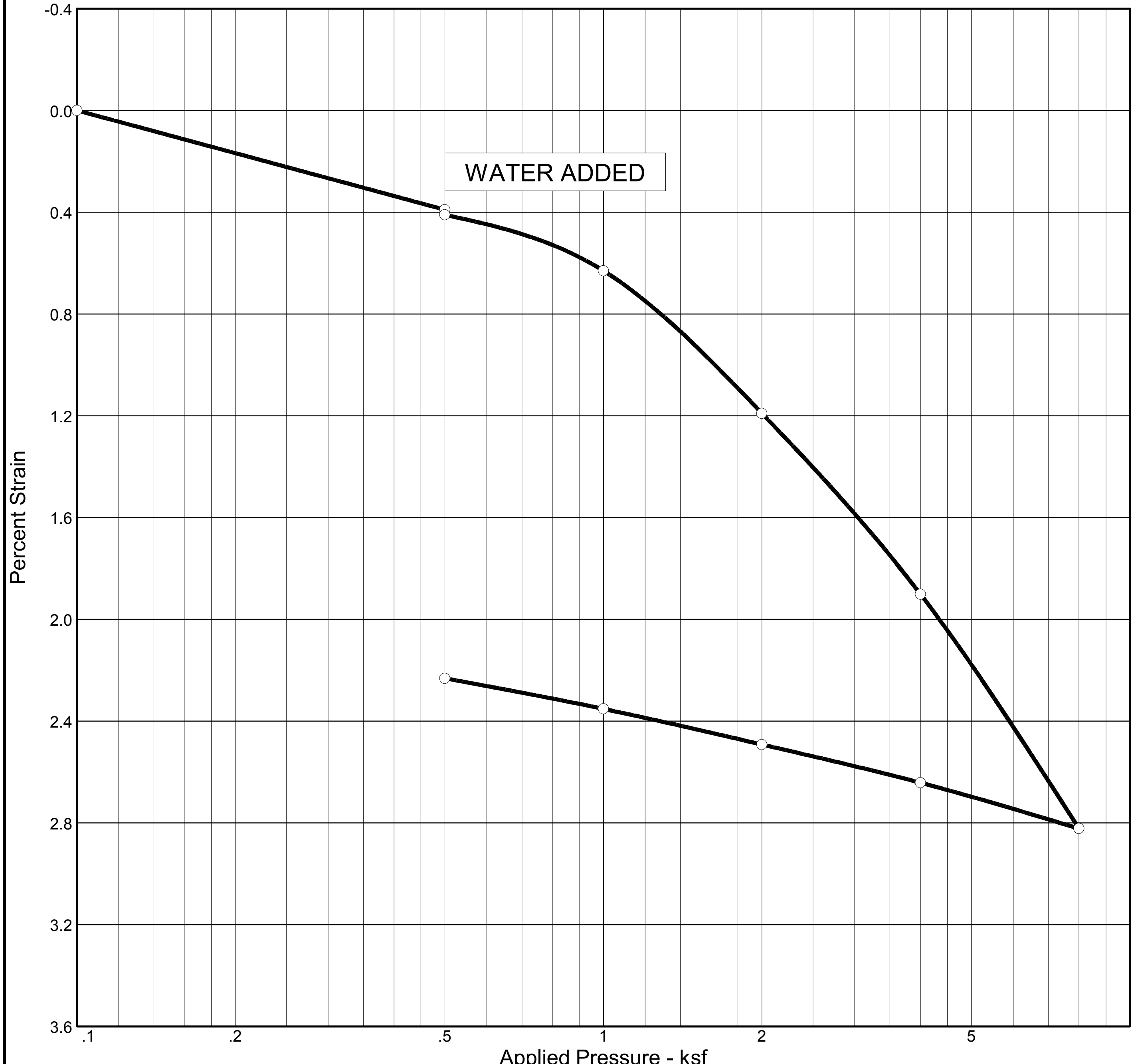
Moore Twining Associates, Inc.

Fresno, CA

Remarks:

Figure

CONSOLIDATION TEST REPORT



Natural		Dry Dens. (pcf)	LL	PI	Sp. Gr.	Overburden (ksf)	P_c (ksf)	C_c	C_s	Swell Press. (ksf)	Swell %	e_o
Sat.	Moist.											
18.2 %	5.9 %	89.2			2.65		1.79	0.06	0.01			0.855

MATERIAL DESCRIPTION	USCS	AASHTO
Silty sand	SM	

Project No. C73111.01 **Client:** Geosyntec Consultans

Project: Proposed Gas Operations Control Building, Pico Rivera

Source: **Sample No.:** HS-4 **Elev./Depth:** 2.5-4'

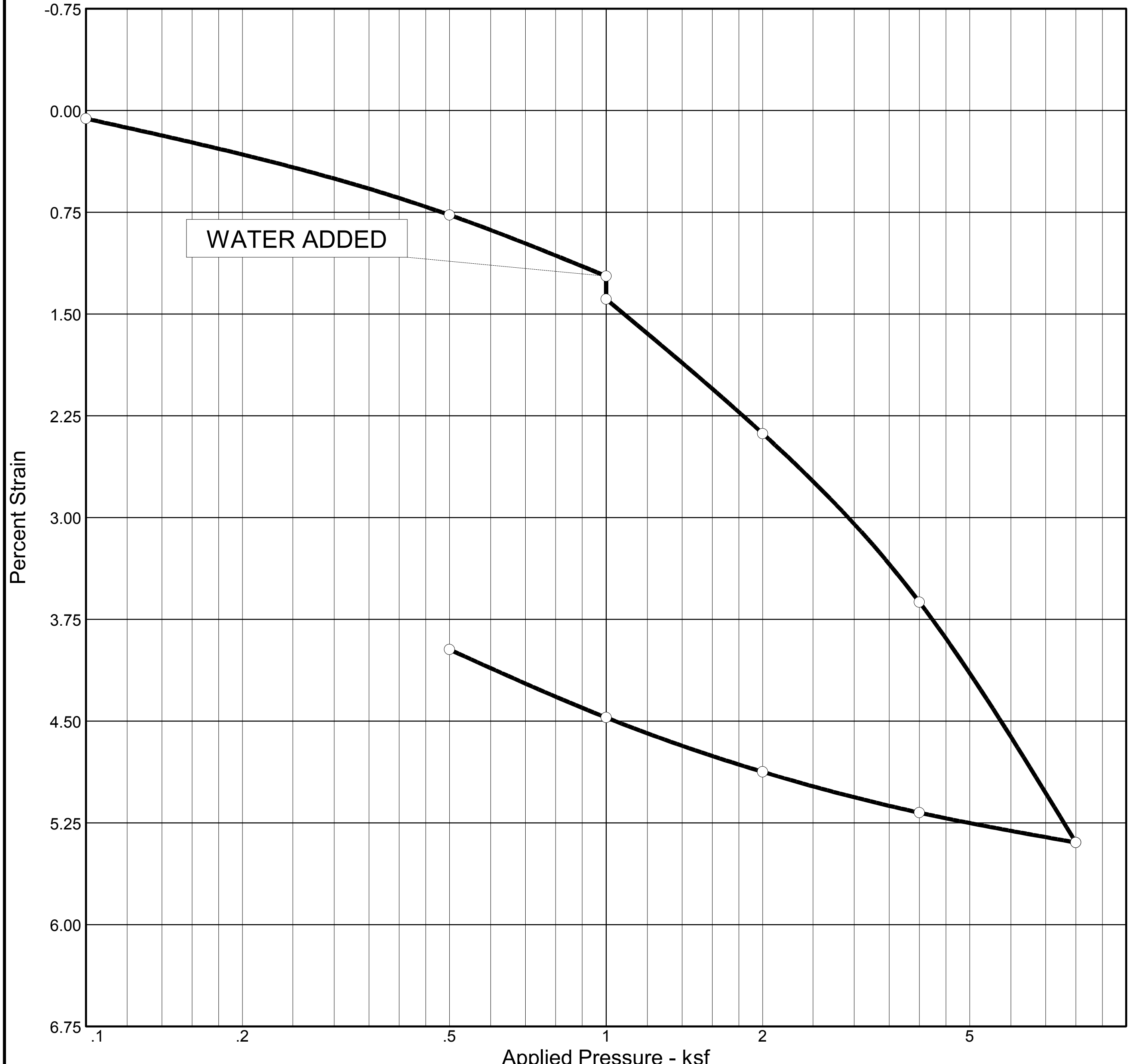
Remarks:

Moore Twining Associates, Inc.

Fresno, CA

Figure

CONSOLIDATION TEST REPORT



Natural		Dry Dens. (pcf)	LL	PI	Sp. Gr.	Overburden (ksf)	P_c (ksf)	C_c	C_s	Swell Press. (ksf)	Clpse. %	e_o
Sat.	Moist.											
78.9 %	21.6 %	95.8	NP	NP	2.65		4.17	0.10	0.02		0.2	0.726

MATERIAL DESCRIPTION	USCS	AASHTO
Silt with sand	ML	

Project No. C73111.01 **Client:** Geosyntec Consultans

Project: Proposed Gas Operations Control Building, Pico Rivera

Source: **Sample No.:** HS-5 **Elev./Depth:** 7.5-9'

Moore Twining Associates, Inc.

Fresno, CA

Remarks:

Figure

MOORE TWINING ASSOCIATES, INC.

Since 1898

EXPANSION INDEX TEST, ASTM D4829

MTA PROJECT NAME:	Proposed Gas Operations Control Building, Pico Rivera	REPORT DATE:	6/21/2021
		TEST DATE:	6/7/2021
MTA PROJECT NO.:	C73111.01		
SAMPLE I.D.:	HS-2-1.5-5'		
SAMPLED BY:	AH		
SAMPLE DATE:	6/2/2021	TESTED BY:	MA

MATERIALS DESCRIPTION: Silty sand with some sandy silt

% PASSING # 4 SIEVE: 100

Initial Moisture Determination:		Final Moisture Determination:	
Pan + Wet Soil Wt., gm	250.0	Wet Soil Wt., lbs	1.0020
Pan + Dry Soil Wt., gm	231.2	Dry Soil Wt., lbs	0.8620
Pan Wt., gm	0.0		
Initial % Moisture Content	8.1	Final % Moisture Content	16.2

Initial Expansion Data:		**Final Expansion Data:**	
Ring + Sample Wt., lbs	0.9321	Ring + Sample Wt., lbs	1.0020
Ring Wt., lbs	0.0000	Ring Wt., lbs	0.0000
Remolded Wt., lbs	0.9321	Remolded Wt., lbs	1.0020
Remolded Wet Density, pcf	128.2	Remolded Wet Density, pcf	137.9
Remolded Dry Density, pcf	118.5	Remolded Dry Density, pcf	118.7

Initial Volume	Final Volume
0.00727222	0.007265

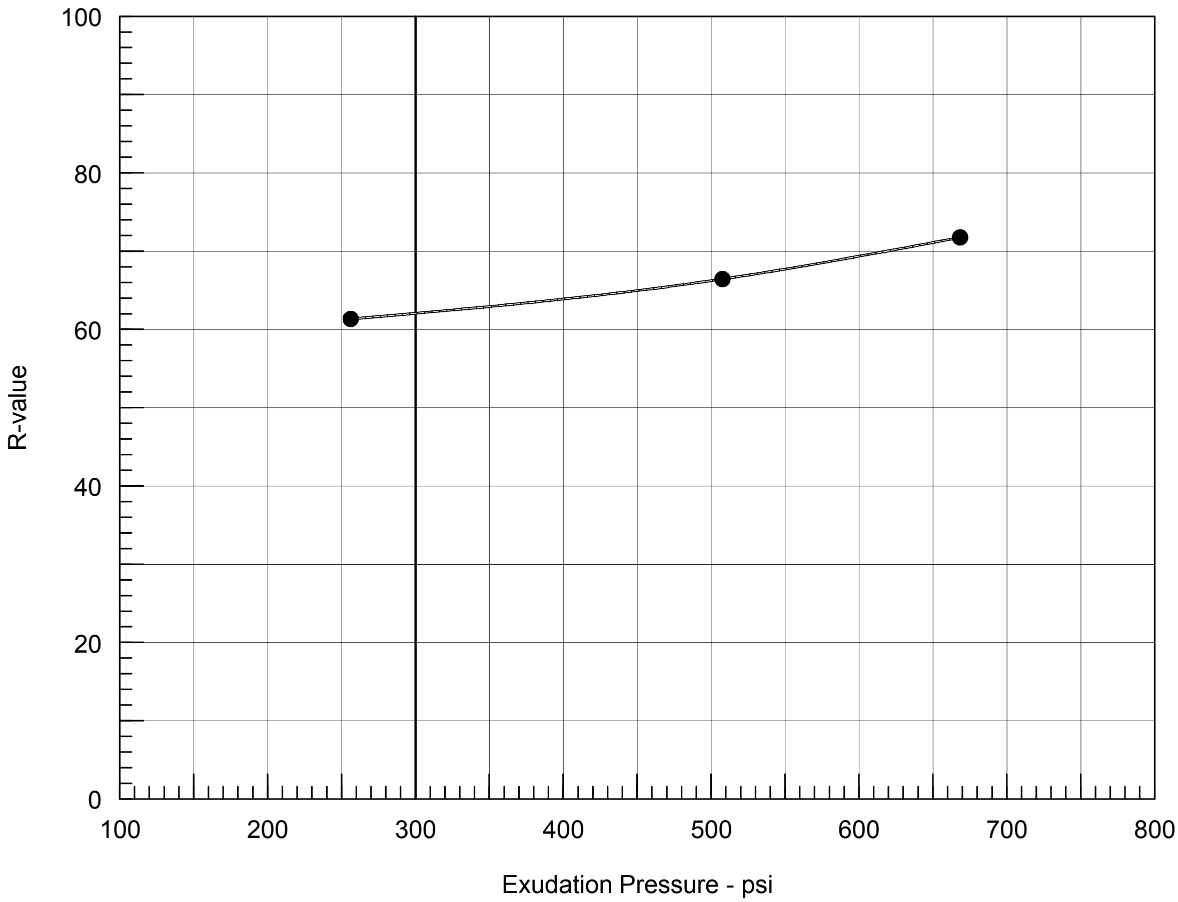
Expansion Data:	
Initial Gage Reading, in:	0.0500
Final Gage Reading, in:	0.0490
Expansion, in:	-0.0010
Expansion Index	0

Comments: **Very Low Expansion Potential**

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

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

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	270	110.8	8.8	0.00	20	2.36	668	74	72
2	350	110.9	11.1	0.00	24	2.36	508	69	66
3	290	111.4	12.2	0.00	30	2.35	256	65	61

Test Results	Material Description
R-value at 300 psi exudation pressure = 62	Silty sand

Project No.: C73111.01

Project: Proposed Gas Operations Control Building, Pico Rivera

Sample Number: HS-3 **Depth:** 1.5-5'

Date: 6/21/2021

Tested by: MP

Checked by: MS

Remarks:

R-VALUE TEST REPORT

Moore Twining Associates, Inc.

Figure NA

MOORE TWINING ASSOCIATES, INC.
Since 1898
California ELAP Certificate #1371

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

MTA Geotechnical Division 2527 Fresno Street Fresno CA, 93721	**Project:** Proposed Gas Operations Control Building Pico R. **Project Number:** Pending **Project Manager:** Allen Harker	**Reported:** 06/21/2021

Analytical Report for the Following Samples

Sample ID	Notes	Laboratory ID	Matrix	Date Sampled	Date Received
HS-5 @1-3.5		HF07018-01	Soil	06/03/21 00:00	06/07/21 09:51

MOORE TWINING ASSOCIATES, INC.
California ELAP Certificate #1371

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

MTA Geotechnical Division	**Project:** Proposed Gas Operations Control Building Pico R.	**Reported:**
2527 Fresno Street	**Project Number:** Pending	06/21/2021
Fresno CA, 93721	**Project Manager:** Allen Harker	

HS-5 @1-3.5

HF07018-01 (Soil) Sampled: 06/03/21 00:00

Analyte	Flag	Result	Reporting Limit	Units	Dilution	Batch	Prepared	Analyzed	Method
Inorganics									
Chloride		ND	6.0	mg/kg	3	B1F1416	06/14/21	06/14/21	Cal Test 422
Chloride		ND	0.00060	% by Weight	3	[CALC]	06/14/21	06/14/21	[CALC]
Sulfate as SO4		**0.0019**	0.00060	% by Weight	3	[CALC]	06/14/21	06/14/21	[CALC]
pH		**8.0**	0.10	pH Units	1	B1F1416	06/14/21	06/14/21	Cal Test 643
Sulfate as SO4		**19**	6.0	mg/kg	3	B1F1416	06/14/21	06/14/21	Cal Test 417

Notes and Definitions

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

MOORE TWINING ASSOCIATES, INC.

Project Name:	Proposed Gas Operations Control Building, Pico Rivera	Report Date:	6/21/2021
Project Number:	C73111.01	Sample Date:	6/2/2021
		Sampled By:	AH
Subject:	Minimum Resistivity, ASTM G187	Tested By:	MA
Material Description:	Silty sand	Test Date:	6/9/2021
Location:	HS-5 @ 1-3.5'		

Laboratory Test Results, Minimum Resistivity - ASTM G187

Total Water Added, mls	Resistivity, Ohm-cm
100 mls	3,100
125 mls	2,700
150 mls	2,700
175 mls	2,900

Remarks: Min. Resistivity is 2,700 Ohm-cm

www.mooretwining.com
PH: 559.268.7021
FX: 559.268.7126
2527 Fresno Street
Fresno, CA 93721

APPENDIX C

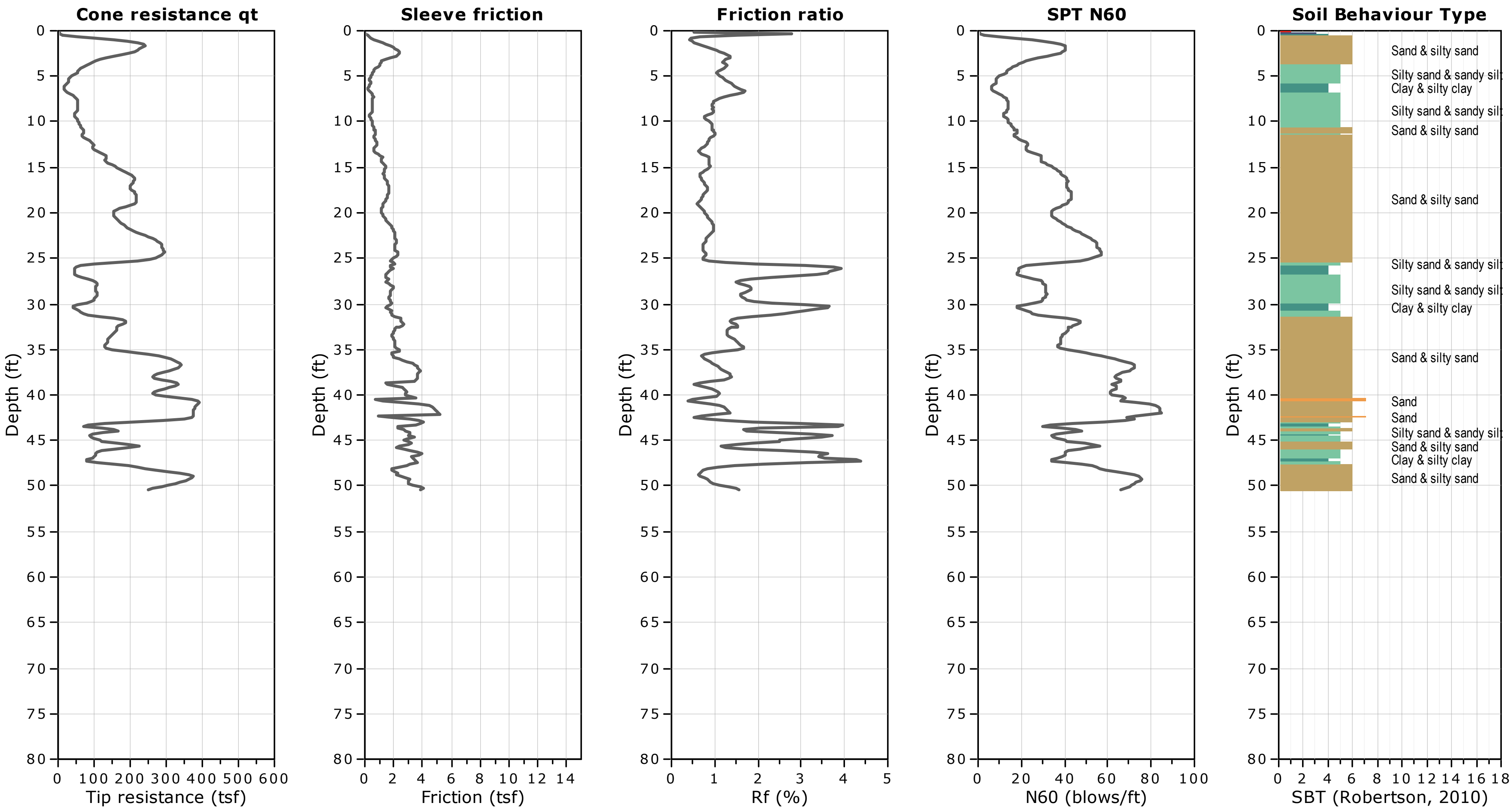
CPT Results

GREGG DRILLING, LLC
WWW.GREGGDRILLING.COM

CPT: CPT-02

CLIENT: GEOSYNTEC
SITE: GOCC, PICO RIVERA, CA

FIELD REP: KLYNT O.
Total depth: 50.36 ft, Date: 6/3/2021



1. Sensitive fine grained
2. Organic material
3. Clay to silty clay
4. Clayey silt to silty clay
5. Silty sand to sandy silt
6. Clean sand to silty sand
7. Gravely sand to sand
8. Very stiff sand to clayey
9. Very stiff fine grained

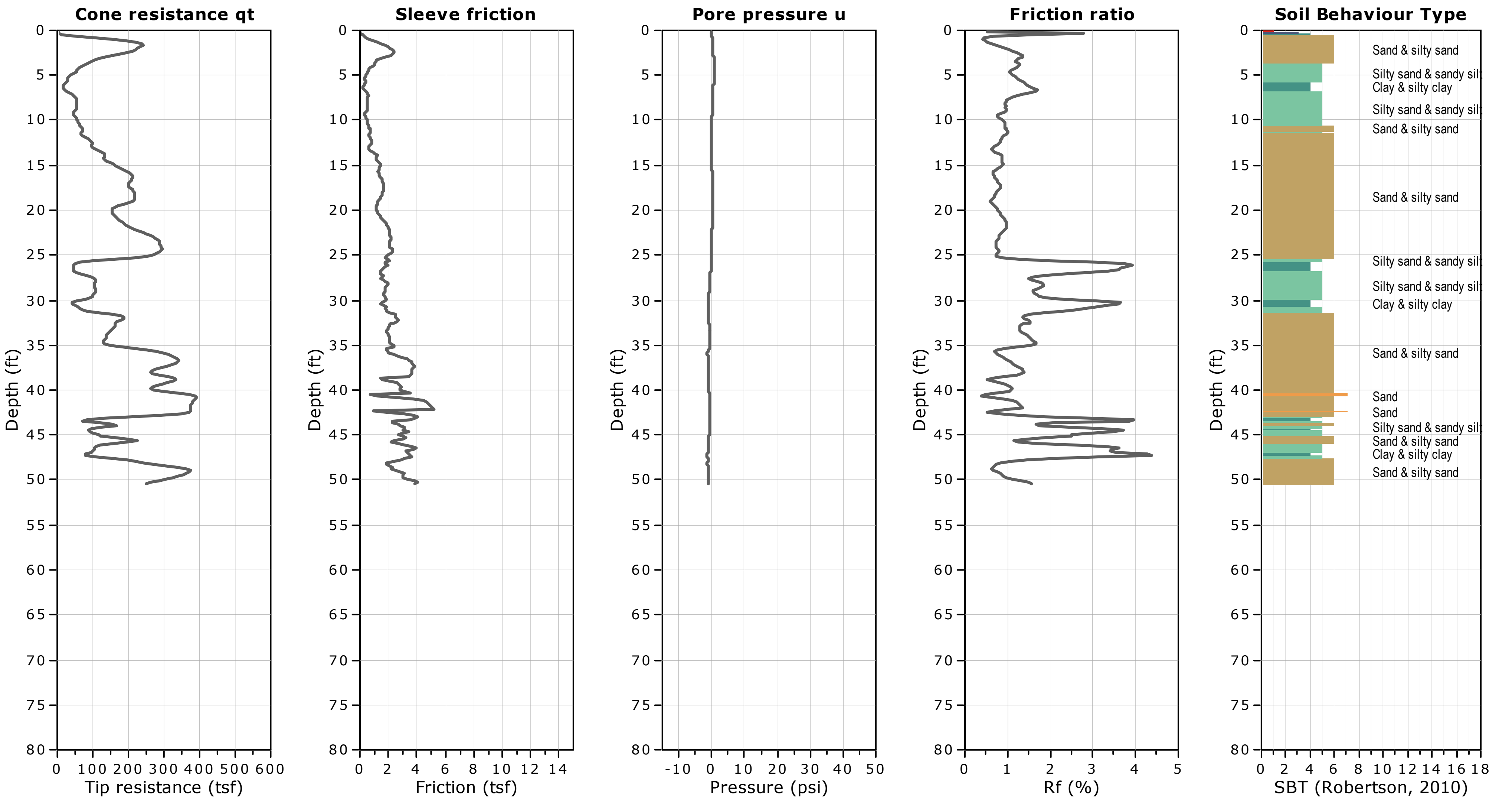
CPT: CPT-02

CLIENT: GEOSYNTEC

SITE: GOCC, PICO RIVERA, CA

FIELD REP: KLYNT O.

Total depth: 50.36 ft, Date: 6/3/2021



1. Sensitive fine grained	4. Clayey silt to silty clay	7. Gravely sand to sand
2. Organic material	5. Silty sand to sandy silt	8. Very stiff sand to clayey
3. Clay to silty clay	6. Clean sand to silty sand	9. Very stiff fine grained

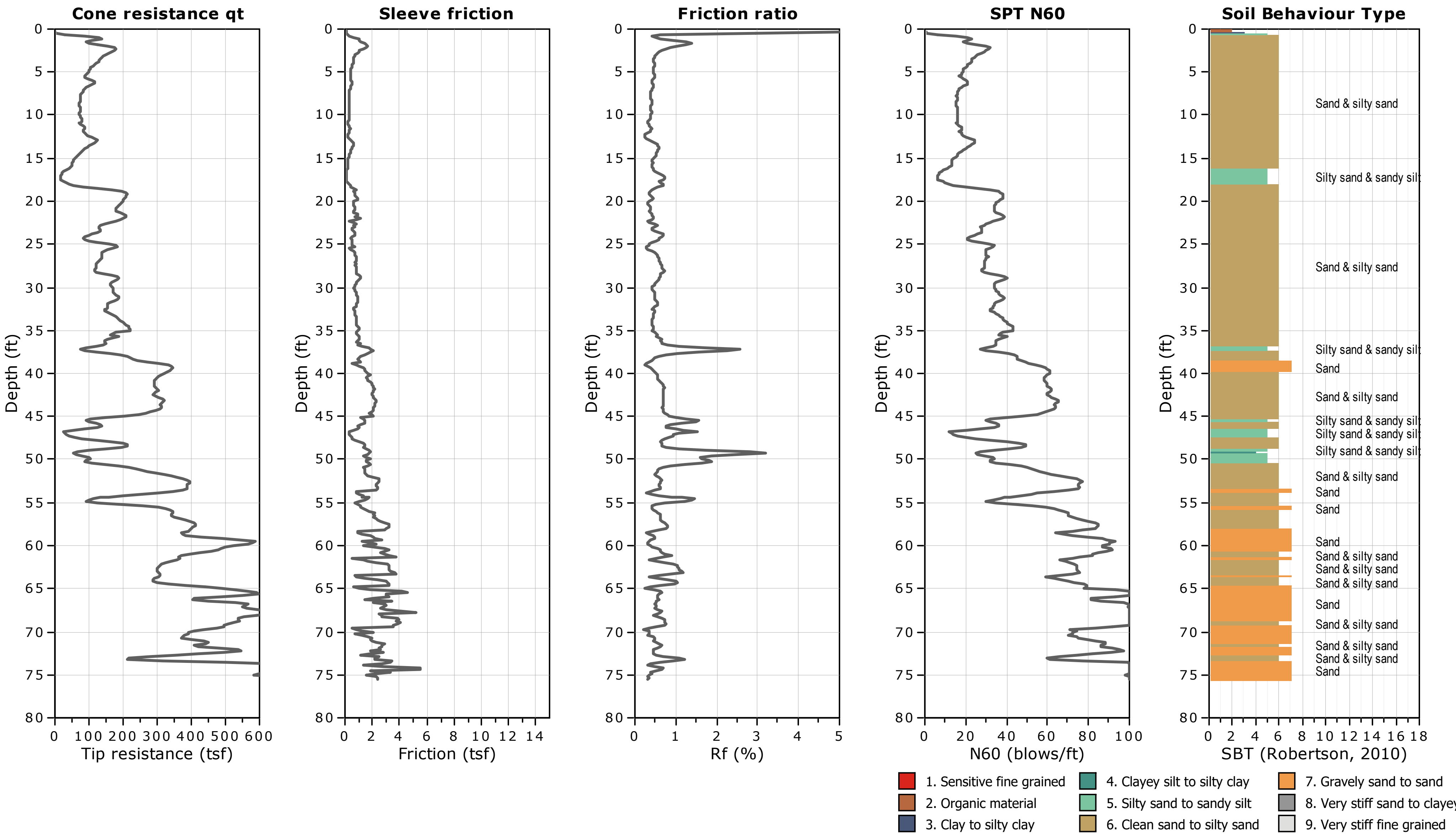
WATER TABLE FOR ESTIMATING PURPOSES ONLY

GREGG DRILLING, LLC
WWW.GREGGDRILLING.COM

CPT: SCPT-01

CLIENT: GEOSYNTEC
SITE: GOCC, PICO RIVERA, CA

FIELD REP: KLYNT O.
Total depth: 75.46 ft, Date: 6/3/2021

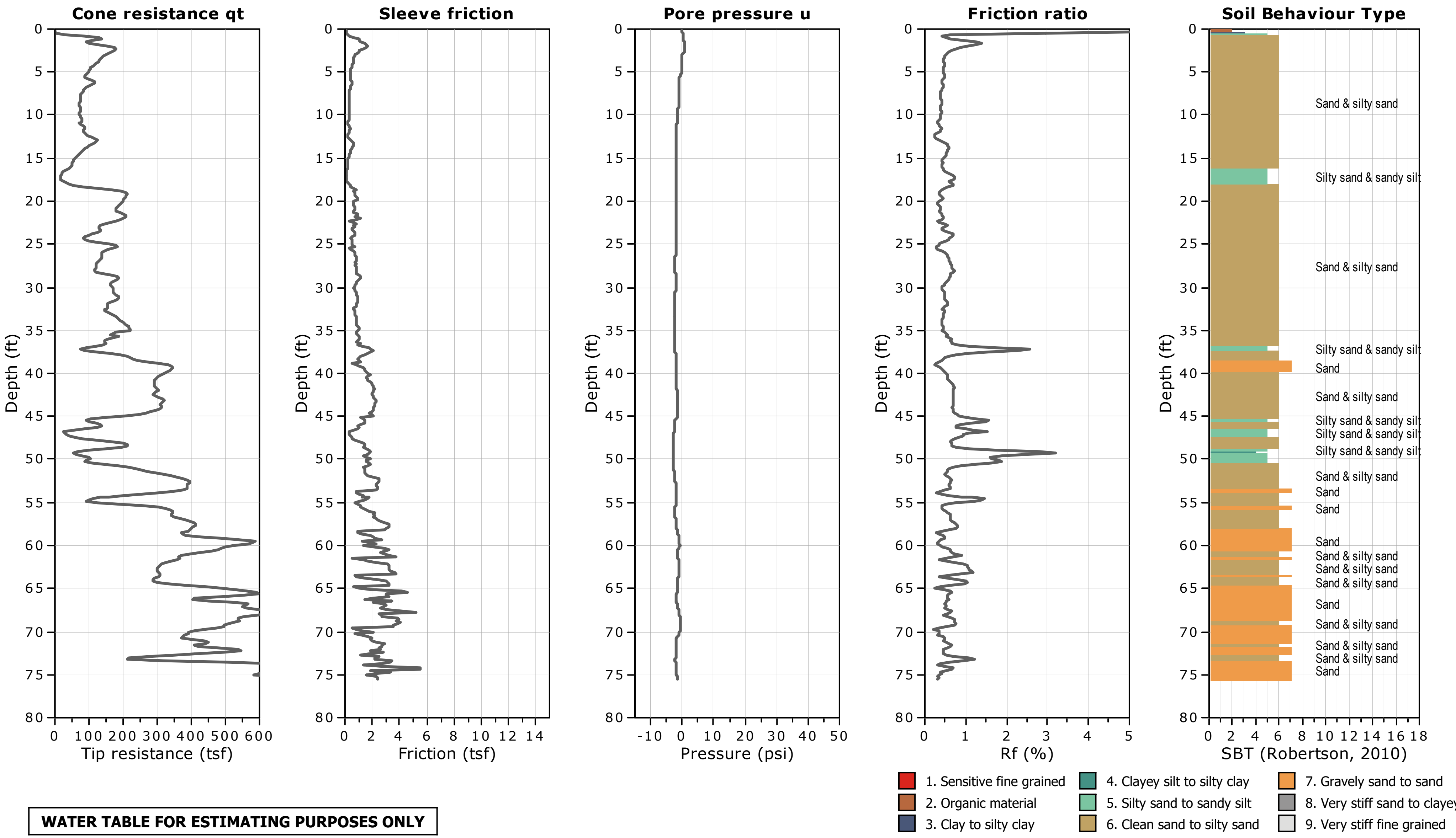


GREGG DRILLING, LLC
WWW.GREGGDRILLING.COM

CPT: SCPT-01

CLIENT: GEOSYNTEC
SITE: GOCC, PICO RIVERA, CA

FIELD REP: KLYNT O.
Total depth: 75.46 ft, Date: 6/3/2021



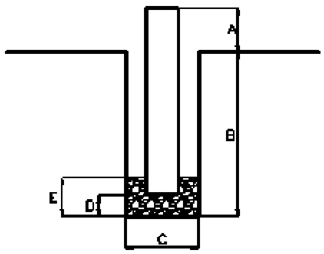
WATER TABLE FOR ESTIMATING PURPOSES ONLY

APPENDIX D

Percolation Test Results

PERCOLATION TEST
No. P-1

Project:	Proposed Gas Operations Control Center Building	**Project No.**	C73111.01
Location:	8101 Rosemead Boulevard, Pico Rivera, California	**Test Date:**	6/3/2021
Coordinates:			



A. Top of Pipe Above Ground	10	Inches
B. Depth of Hole	56	Inches
C. Diameter of Hole	8	Inches
D. Depth of Gravel Below Pipe	4	Inches
E. Total Gravel Layer Depth	22	Inches
F. Pipe Length	62	Inches
G. Pipe Diameter	2	Inches

Pre-saturated: 6/3/2021 Filled with water to 17 to 18 inches from bottom
Refilled after hole was dry and repeated for 1 hour

Gravel Correction Factor: 2.6

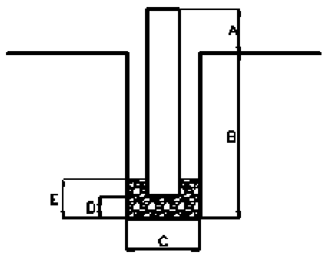
Trial 1 was Test Method Time Interval Determination after 1 hour presoak

Trial	Date	Time	Depth To Water* (feet)	Time Interval (min)	Water Drop (inches)	Uncorrected, Unfactored Percolation Rate, (minutes per inch)	Unfactored Infiltration Rate, (Inches per hour)
1	6/3/2021	11:45:00	3.35				
	6/3/2021	11:48:40	4.35	3.67	12	0.8	6.9
Begin 2	6/3/2021	11:51:10	3.25				
Test	6/3/2021	11:55:20	4.25	4.17	12	0.9	5.8
3	6/3/2021	11:59:25	3.38				
	6/3/2021	12:03:45	4.38	4.33	12	0.9	6.0
4	6/3/2021	12:05:00	3.36				
	6/3/2021	12:09:10	4.36	4.17	12	0.9	6.1
5	6/3/2021	12:11:00	3.34				
	6/3/2021	12:15:40	4.34	4.67	12	1.0	5.4
6	6/3/2021	12:16:50	3.37				
	6/3/2021	12:21:40	4.37	4.83	12	1.0	5.3
7	6/3/2021	12:22:50	3.35				
	6/3/2021	12:27:45	4.35	4.92	12	1.0	5.2
8	6/3/2021	12:29:00	3.37				
	6/3/2021	12:33:50	4.37	4.83	12	1.0	5.4
9	6/3/2021	12:34:55	3.35				
	6/3/2021	12:40:00	4.35	5.08	12	1.1	5.1

* Depth to water measured from top of pipe

PERCOLATION TEST
No. P-2

Project:	Proposed Gas Operations Control Center Building	**Project No.**	C73111.01
Location:	8101 Rosemead Boulevard, Pico Rivera, California	**Test Date:**	6/3/2021
Coordinates:			



A. Top of Pipe Above Ground	19 Inches
B. Depth of Hole	43 Inches
C. Diameter of Hole	8 Inches
D. Depth of Gravel Below Pipe	2 Inches
E. Total Gravel Layer Depth	17 Inches
F. Pipe Length	60 Inches
G. Pipe Diameter	2 Inches

Pre-saturated: 6/3/2021 Filled with water to 14 to 17 inches from bottom
Refilled after 1/2 hour and continued presoak for 1 hour total

Gravel Correction Factor: 2.6

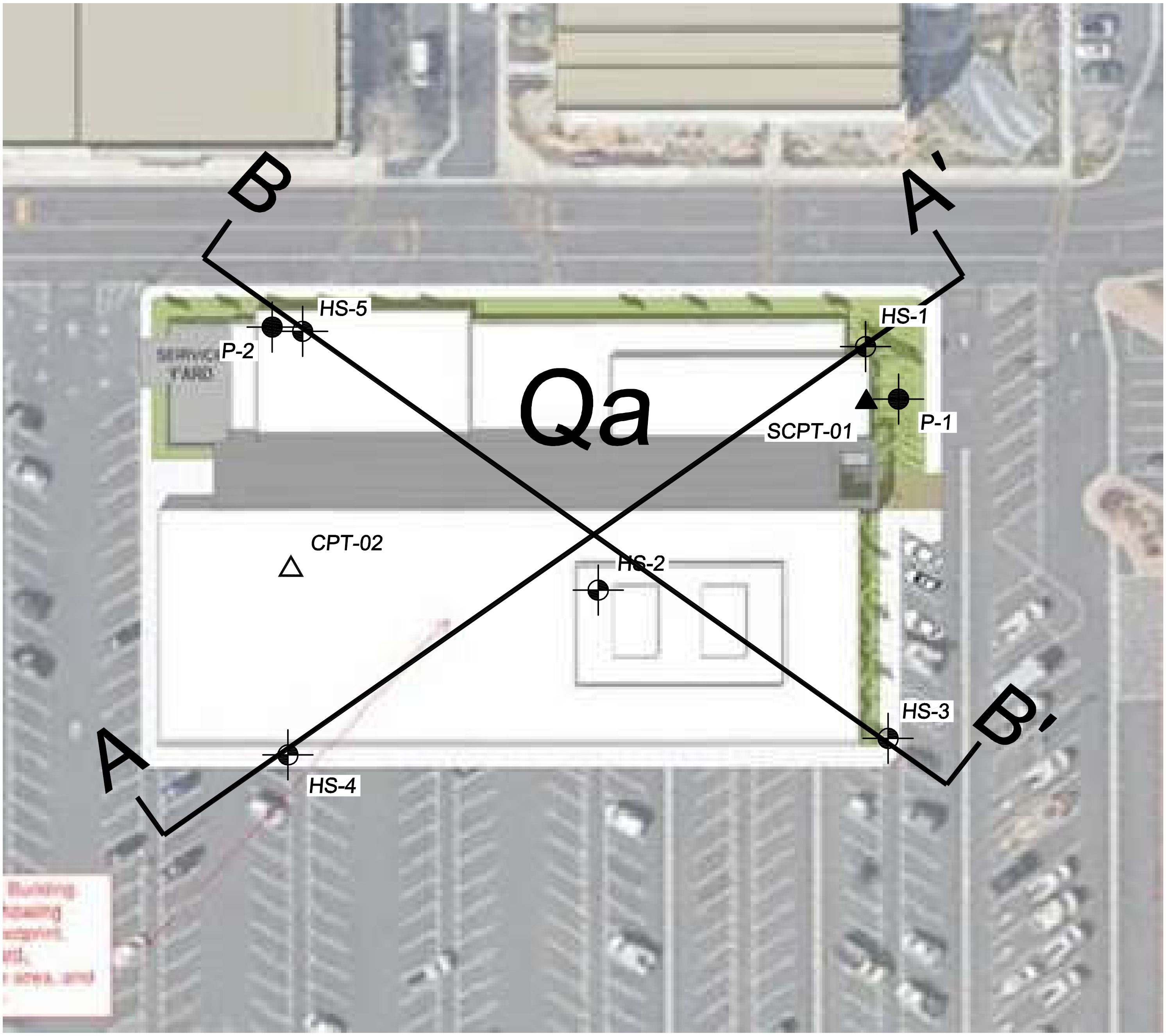
Trials 1. 2 and 3 were Test Method Time Interval Determination after 1 hour presoak

Trial	Date	Time	Depth To Water* (feet)	Time Interval (min)	Water Drop (inches)	Uncorrected, Unfactored Percolation Rate, (minutes per inch)	Unfactored Infiltration Rate, (Inches per hour)
1	6/3/2021	11:32:00	3.62				
	6/3/2021	11:42:00	4.07	10.0	5.4	4.7	1.4
2	6/3/2021	11:42:00	4.07				
	6/3/2021	11:56:30	4.42	14.5	4.2	8.8	1.0
3	6/3/2021	11:56:30	4.42				
	6/3/2021	12:06:20	4.68	9.83	3.12	8.1	1.6
Begin 4	6/3/2021	12:08:00	3.55				
Test	6/3/2021	12:43:00	4.37	35.0	9.84	9.1	0.8
5	6/3/2021	12:45:00	3.58				
	6/3/2021	13:15:00	4.39	30.0	9.72	7.9	0.9
6	6/3/2021	13:16:30	3.58				
	6/3/2021	13:46:30	4.38	30.0	9.6	8.0	0.9
7	6/3/2021	13:48:00	3.58				
	6/3/2021	14:18:00	4.37	30.0	9.48	8.1	0.9
8	6/3/2021	14:19:30	3.58				
	6/3/2021	14:49:30	4.36	30.0	9.36	8.2	0.9
9	6/3/2021	14:51:00	3.58				
	6/3/2021	15:21:00	4.37	30.0	9.48	8.1	0.9
10	6/3/2021	15:22:15	3.58				
	6/3/2021	15:52:15	4.35	30.0	9.24	8.3	0.9
11	6/3/2021	15:53:15	3.58				
	6/3/2021	16:23:15	4.35	30.0	9.24	8.3	0.9

* Depth to water measured from top of pipe

APPENDIX E

Geologic Cross-Sections



HS-1 — APPROXIMATE LOCATION OF HOLLOW STEM AUGER BORING

P-1 — APPROXIMATE LOCATION OF PERCOLATION TEST BORING

SCPT-01 — APPROXIMATE LOCATION OF CONE PENETRATION TEST WITH SHEAR WAVE VELOCITIES

CPT-02 — APPROXIMATE LOCATION OF CONE PENETRATION TEST

A A' — CROSS SECTION

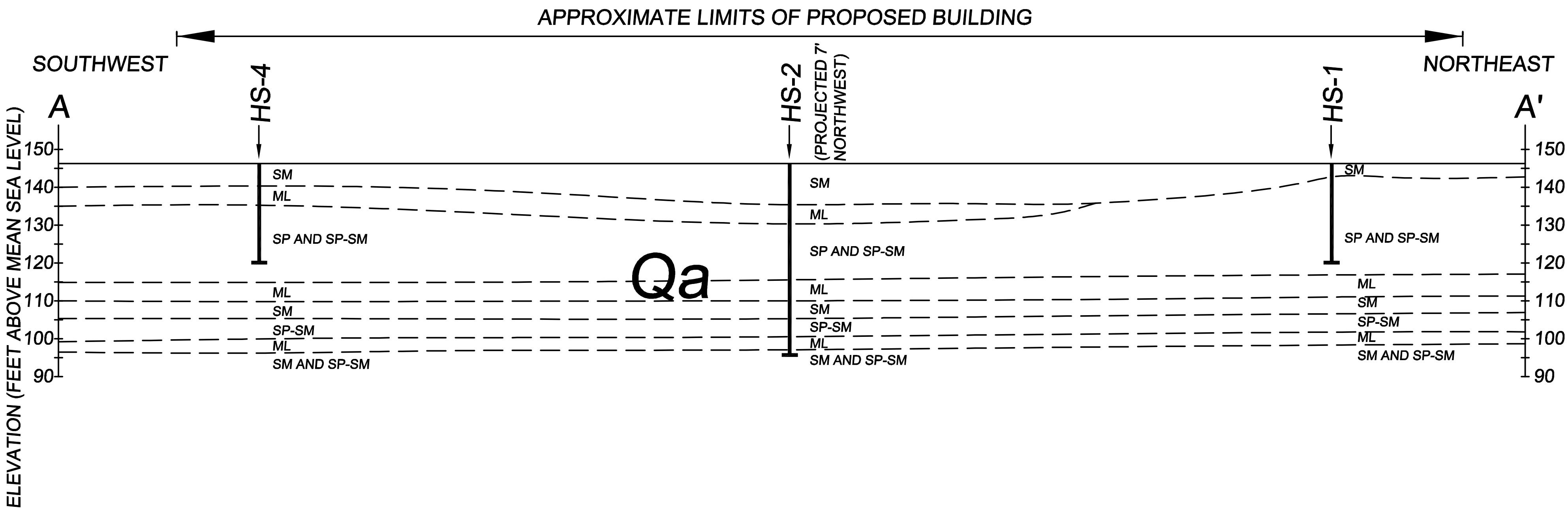
Qa: ALLUVIAL GRAVEL, SAND, AND SILT OF VALLEYS AND FLOODPLAINS

N

0 60

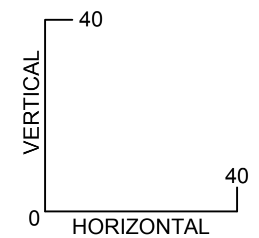
APPROXIMATE SCALE IN FEET

SITE GEOLOGIC MAP PROPOSED GAS OPERATIONS CONTROL CENTER BUILDING SO CAL GAS FACILITY 8101 ROSEMEAD BOULEVARD PICO RIVERA, CALIFORNIA	FILE NO. 73111-01-02	DATE DRAWN: 07/06/21	MOORE TWINING ASSOCIATES, INC.
	DRAWN BY: RM	APPROVED BY:	
	PROJECT NO. C73111.01	DRAWING NO. 6	



Qa: ALLUVIAL GRAVEL, SAND, AND SILT OF VALLEYS AND FLOODPLAINS

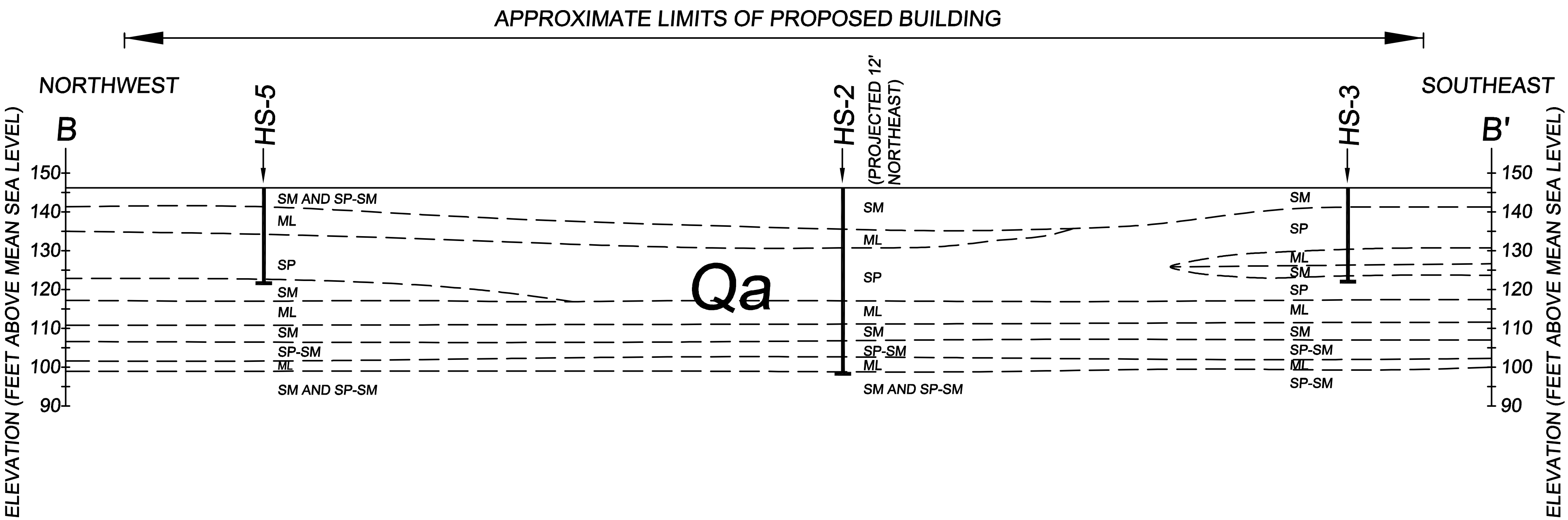
SM: SILTY SAND
ML: SILT WITH SAND OR SANDY SILT
SP: POORLY GRADED SAND
SP-SM: POORLY GRADED SAND WITH SILT



APPROXIMATE SCALE
IN FEET

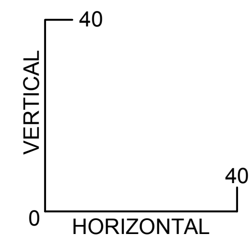
CROSS SECTION A-A' PROPOSED GAS OPERATIONS CONTROL CENTER BUILDING SO CAL GAS FACILITY 8101 ROSEMEAD BOULEVARD PICO RIVERA, CALIFORNIA	FILE NO. 73111-01-02	DATE DRAWN: 07/06/21
	DRAWN BY: RM	APPROVED BY:
	PROJECT NO. C73111.01	DRAWING NO. 7

MOORE TWINING ASSOCIATES, INC.



Qa: ALLUVIAL GRAVEL, SAND, AND SILT OF VALLEYS AND FLOODPLAINS

SM: SILTY SAND
ML: SILT WITH SAND OR SANDY SILT
SP: POORLY GRADED SAND
SP-SM: POORLY GRADED SAND WITH SILT



APPROXIMATE SCALE
IN FEET

CROSS SECTION B-B' PROPOSED GAS OPERATIONS CONTROL CENTER BUILDING SO CAL GAS FACILITY 8101 ROSEMEAD BOULEVARD PICO RIVERA, CALIFORNIA	FILE NO. 73111-01-02	DATE DRAWN: 07/06/21
	DRAWN BY: RM	APPROVED BY:
	PROJECT NO. C73111.01	DRAWING NO. 8

MOORE TWINING
ASSOCIATES, INC.

APPENDIX F

Site-Specific Hazard Analysis

Gas Operations Control Center Building

Site-Specific Seismic Hazard Assessment | Pico Rivera, California

04.00184103-PR-001 02 | September 14, 2021

Final

Moore Twining Associates, Inc.

Document Control

Document Information

Project Title	Site-Specific Seismic Hazard Assessment for the Gas Operations Control Center Building, Pico Rivera, California
Document Title	Gas Operations Control Center Building
Fugro Project No.	04.00184103
Fugro Document No.	04.00184103-PR-001
Issue Number	02
Issue Status	Final

Client Information

Client	Moore Twining Associates, Inc.
Client Address	2527 Fresno St., Fresno, California 93721
Client Contact	Allen H. Harker

Revision History

Issue	Date	Status	Comments on Content	Prepared By	Checked By	Approved By
01	July 16, 2021	For Review	Awaiting client comments	JL	AF	AF
02	September 14, 2021	Final	Final report	JL	AF	AF

Project Team

Initials	Name	Role
JL	Jinchi Lu, PhD, PE	Senior Engineer
AF	Alfredo Fernandez, PhD, PE	Principal Engineer

FUGRO
Fugro USA Land, Inc.
1777 Botelho Drive, Suite 262
Walnut Creek, California 94596
T +1 925 949-7100

Moore Twining Associates, Inc.
2527 Fresno Street
Fresno, California 93721

September 14, 2021

Dear Mr. Harker,

We are pleased to submit this report summarizing the results and recommendations from the site-specific seismic hazard assessment conducted for the proposed gas operations control center building located in Pico Rivera, California. Our services were performed in general accordance with our Proposal No. 04.P0184103, Document No. 04.P0184103-P-001(01), dated January 13, 2021, and authorized on July 1, 2021.

Introduction

We understand that the proposed project will consist of a gas operations control center building within an existing gas facility located at 8101 Rosemead Boulevard in Pico Rivera, California. This report 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 2014 National Seismic Hazard Map Project (NSHMP) (Petersen et al., 2014), and the NGA West 2 Ground Motion Models (Bozorgnia et al., 2014). The design ground motion parameters were calculated following the site-specific ground motion procedures defined in Chapter 21 of ASCE 7-16 (ASCE, 2016, 2018) as required by the 2019 California Building Code (CBC) (CBSC, 2019).

Subsurface Conditions

The site-specific geotechnical subsurface information available for review consisted of five geotechnical borings conducted by Gregg Drilling & Testing and provided by Moore Twining Associates, Inc. (2021), and one Cone Penetration Test (CPT) and one Seismic CPT (SCPT) conducted by Gregg Drilling, LLC. (2021). According to the explorations available, the subsurface conditions comprise primarily of sands and silty sands.

The time-weighted average shear wave velocity (Vs) in the top 100 feet (ft) (30 meters [m]) (Vs30) is an important input parameter to include the local site conditions in the PSHA. The conducted SCPT provides in-situ Vs measurements for the site; and therefore, the design shear wave velocity profile for the site was calculated mainly using the shear wave velocity data from the SCPT (SCPT-01), which extended to a depth of 74.5 ft. The Vs profile for SCPT-01 is presented on Figure 1. Additionally, we understand from communication with Moore Twining Associates, that ground modification is being planned for the upper 40 ft. Therefore, as directed by Moore Twining Associates, the Vs for the upper 40 feet depth from SCPT-01 was increased by 15 percent to account for the densification from the ground modification. This modified Vs profile is presented on Figure 1.

The Vs30 value for the site was calculated directly from the Vs measurements provided by SCPT-01 modified to account for the densification from ground modification. Because the Vs profile does not extend to 100 ft, a time-weighted average Vs was first calculated only to the maximum exploration depth of 74.5 ft (23 m) (Vs23), then this Vs23 value was extended using the empirical correlation proposed by Boore (2004) to calculate a Vs30 value. The calculated Vs30 value is 1015 ft/sec (310 m/sec). This Vs30 value corresponds to a Site Class D according to CBC (CBSC, 2019).

Probabilistic Seismic Hazard Analysis

Project Location

A Probabilistic Seismic Hazard Analysis (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 1.

Table 1: Representative Project Location Coordinates used in the PSHA

Latitude	Longitude
33.9673°N	118.1091°W

Methodology

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 is 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.

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):

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

Equation 1

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:

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

Equation 2

where $P[A > a, \mathrm{t}]$ 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.

Seismic Source Model

The PSHA was conducted using the seismic source model adopted by the UGSG to develop the 2014 NSHMP (Petersen et al., 2014) for California which corresponds to the Uniform California Earthquake Rupture Forecast, Version 3 (UCERF3). The details of this seismic source model can be found in Field et al. (2013).

Empirical Ground Motion Models

The attenuation of seismic waves from a seismic source was modeled using empirical ground motion models (GMM's). These empirical GMM's should model the type of rupture mechanism as well as the regional geology to properly estimate site-specific strong ground motion parameters. Four of the Next Generation Attenuation (NGA) West 2 GMM's (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 & Youngs (2014). The four NGA West 2 GMM's were equally weighted, following the weighting scheme used in the development of the 2014 USGS NSHMP (Petersen et al., 2014).

Implementation

The PSHA was performed using the USGS computer code *nshmp-haz*, which has been used by the USGS to develop the US national seismic hazard maps.

Results from the PSHA

Figure 2 shows the mean annual seismic hazard curves for selected spectral periods ranging from 0.01 to 10 seconds for a Vs30 of 310 m/sec. A spectral period of 0.01 seconds is used to represent the peak ground acceleration (PGA). These hazard curves represent the total mean hazard from combining all seismic sources and ground motion models. This figure also indicates the annual frequency of exceedance corresponding to a return period of 2,475 years.

Table 2 tabulates the calculated mean magnitude, distance, and epsilon from the seismic hazard deaggregation 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 2: Mean Seismic Hazard Deaggregation for a Return Period of 2,475 Years and Vs30 of 310 m/sec

	PGA	Sa at 1 second
Mean Magnitude (Mw)	6.91	7.23
Mean Distance (km)	9.9	12.2
Mean Epsilon	1.3	1.2

Figure 3 presents the 5 percent-damped mean horizontal UHRS for a return period of 2,475 years and a Vs30 of 310 m/sec. Table 3 tabulates the mean horizontal UHRS for periods ranging from 0.01 (i.e., PGA) to 10 seconds for a return period of 2,475 years.

Table 3: Mean Horizontal UHRS for Return Period of 2,475 Years and a Vs30 of 310 m/sec, 5% Damping

Period (sec)	Horizontal Spectral Acceleration (g)
0.01	0.849
0.03	0.881
0.05	1.01
0.075	1.25
0.1	1.48
0.15	1.75
0.2	1.95
0.25	2.08
0.3	2.16
0.4	2.11
0.5	1.98
0.75	1.56
1	1.24
1.5	0.833
2	0.605
3	0.373
4	0.256
5	0.192
7.5	0.111
10	0.0723

Design Response Spectrum

According to ASCE 7-16, for Site Class D sites with S1 (mapped 5 percent damped spectral response acceleration parameter at a period of 1 second) greater than or equal to 0.2 g, the design response spectrum and design acceleration parameters should be developed following the site-specific ground motion procedures defined in Section 21.2 of ASCE 7-16. The S1 for the project site was calculated as 0.639 g using the USGS web service (https://earthquake.usgs.gov/ws/designmaps/asce7-16.html). Therefore, the design ground motions for the site should be calculated using the site-specific procedures from ASCE 7-16.

ASCE 7-16 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 7-16 specifies a lower limit to the deterministic MCE_R ground motion. The site-specific MCE_R should not be less than 150 percent of the site-specific design response spectrum. The site-specific design response spectrum is calculated as 2/3 of the site-specific 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 11.4.6 of ASCE 7-16, using modified Fa and Fv values provided in Section 21.3 of ASCE 7-16.

The PSHA results described in the previous section were used to calculate the probabilistic MCE_R spectrum. As specified in ASCE 7-16, 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 7-16. The mapped risk coefficients at spectral periods of 0.2 and 1.0 sec, C_{RS} and C_{R1}, respectively, were determined using the USGS web service (https://earthquake.usgs.gov/ws/designmaps/asce7-16.html). The values of these risk coefficients C_{RS} and C_{R1} are 0.902 and 0.9, respectively. The ground motions in the direction of maximum horizontal response were calculated by applying the scaling factors recommended in ASCE 7-16. Figure 4 shows the UHRS for a return period of 2,475 years along with the probabilistic MCE_R response spectrum.

The deterministic MCE_R spectrum was calculated by performing a DSHA in EZ-FRISK^{TM} (Fugro, 2019) using the same seismic sources and GMM's used in the PSHA. The UCERF3 source model includes magnitude frequency distributions (MFD's) which relate frequency of occurrence to earthquake magnitude; however, these MFD's include multi-fault ruptures scenarios with large magnitudes but with a low probability of occurrence. Therefore, following the current USGS approach to calculate deterministic ground motions from the UCERF3 source model, to estimate the characteristic magnitude for the seismic sources, we used the empirical relationships proposed by Wells and Coppersmith (1994) that relates rupture geometry to earthquake magnitude. The calculated characteristic magnitude values were checked for consistency with the values provided in the catalog of deterministic ruptures from the 2014 NSHMP provided by the Building Seismic Safety Council (BSSC) (https://earthquake.usgs.gov/scenarios/catalog/bssc2014). The ground motions in the direction of maximum horizontal response were calculated by applying the scaling factors recommended in ASCE 7-16. Figure 4 illustrates the calculation of the deterministic MCE_R response spectrum. The deterministic MCE_R response spectrum was calculated as the maximum of the 84th DSHA response spectrum and the lower limit specified by ASCE 7-16 Supplement 1 calculated for a Site Class D.

Figure 5 presents the development of the site-specific MCE_R and design response spectra for the site. In this case, the probabilistic MCE_R spectrum is lower than the deterministic MCE_R spectrum for all spectral periods. The site-specific MCE_R spectrum is the maximum of: 1) the minimum of the probabilistic and deterministic MCE_R, and 2) 150 percent of the design response spectrum. Following ASCE 7-16, 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 7-16 (80 percent of the general spectrum for Site Class D, using modified Fa and Fv values provided in Section 21.3 of ASCE 7-16). The transition period from constant velocity to constant displacement, T_L, required to calculate the lower limit, was estimated as 8 seconds using the USGS web service (https://earthquake.usgs.gov/ws/designmaps/asce7-16.html).

Table 4 tabulates the spectral ordinates of the recommended site-specific MCE_R and design response spectra per ASCE 7-16. The corresponding design acceleration parameters S_{MS}, S_{M1}, S_{DS}, and S_{D1} are tabulated in Table 5.

Table 4: MCE_R and Design Response Spectra per ASCE 7-16 for a Vs30 of 310 m/sec, 5% Damping

	Horizontal Spectral Acceleration (g)							
Period (sec)	UHRS for Return Period of 2,475 Years	Probabilistic MCE_R	84th Deterministic Spectrum	Deterministic Lower Limit	Deterministic MCE_R	Site-Specific MCE_R	80% General Response Spectrum	Design Response Spectrum
0.01 (PGA)	0.849	0.842	0.994	0.554	1.09	0.842	0.412	0.561
0.03	0.881	0.874	1.01	0.565	1.12	0.874	0.476	0.583
0.05	1.01	1.00	1.13	0.628	1.24	1.00	0.540	0.669
0.075	1.25	1.24	1.35	0.750	1.48	1.24	0.619	0.827
0.1	1.48	1.46	1.55	0.863	1.70	1.46	0.699	0.976
0.15	1.75	1.73	1.85	1.03	2.03	1.73	0.858	1.16
0.179	1.87	1.85	1.98	1.11	2.18	1.85	0.951	1.24
0.2	1.95	1.93	2.07	1.15	2.28	1.93	0.951	1.29
0.25	2.08	2.11	2.24	1.28	2.52	2.11	0.951	1.41
0.3	2.16	2.24	2.39	1.39	2.75	2.24	0.951	1.49
0.4	2.11	2.25	2.50	1.50	2.96	2.25	0.951	1.50
0.5	1.98	2.16	2.40	1.47	2.91	2.16	0.951	1.44
0.75	1.56	1.78	1.93	1.24	2.44	1.78	0.951	1.19
0.896	1.36	1.57	1.68	1.09	2.16	1.57	0.951	1.05
1	1.24	1.45	1.54	1.01	2.00	1.45	0.852	0.969
1.5	0.833	1.01	1.01	0.694	1.37	1.01	0.568	0.675
2	0.605	0.755	0.732	0.514	1.01	0.755	0.426	0.503
3	0.373	0.482	0.509	0.370	0.731	0.482	0.284	0.321
4	0.256	0.339	0.376	0.280	0.553	0.339	0.213	0.226
5	0.192	0.259	0.287	0.218	0.430	0.259	0.170	0.173
7.5	0.111	0.150	0.151	0.114	0.226	0.170	0.114	0.114
8	0.101	0.136	0.134	0.102	0.200	0.160	0.107	0.107
10	0.0723	0.0976	0.0885	0.0672	0.133	0.102	0.0682	0.0682

Table 5: Design Acceleration Parameters per ASCE 7-16 for a Vs30 of 310 m/sec, 5% Damping

Parameter	Value
S_{MS}	2.03 g
S_{M1}	1.52 g
S_{DS}	1.35 g
S_{D1}	1.01 g

Limitations of this Study

This report has been prepared solely to assist Moore Twining Associates, Inc. with the seismic design of the Gas Operations Control Center Building located in Pico Rivera, California. The results herein apply to the specific location mentioned and are not applicable to other locations.

Seismic hazard analysis is a dynamic, rapidly evolving field of earthquake engineering. It is likely that the standard of practice in the project region for these services will evolve over the next few years. Additionally, the analyses were conducted using the geotechnical information available to the date of issue of this report. Consequently, the results presented in this study should be reviewed if new standards of practice or geotechnical data are available during the design of the project.

This report has been prepared for the exclusive use of Moore Twining Associates, Inc. and their agents for the specific application of the Gas Operations Control Center Building located in Pico Rivera, California. In our opinion, the findings, conclusions, professional opinions, and recommendations presented herein were prepared in accordance with the generally accepted geotechnical earthquake engineering practice of the project region.

Although information contained in this report may be of some use for other purposes, it may not contain sufficient information for other parties or uses. If any changes are made to the project as described in this report, the conclusions and recommendations in this report shall not be considered valid unless the changes are reviewed, and the conclusions and recommendations of this report are modified or validated in writing by Fugro.

Sincerely,

Jinchi Lu, PhD, PE
Senior Engineer

Alfredo Fernandez, PhD, PE
Principal Engineer

References

Abrahamson N.A., Silva W., Kamai R. (2014). Summary of the ASK14 Ground-Motion Relation for Active Crustal Regions, *Earthquake Spectra*, *30*(3) 1025–1055.

American Society of Civil Engineers (ASCE), (2016). “ASCE Standard 7-16 – Minimum Design Loads for Buildings and Other Structures,” ASCE 7-16.

American Society of Civil Engineers (ASCE). (2018). “ASCE Standard 7-16 – Minimum Design Loads for Buildings and Other Structures, Supplement 1”.

Boore, D.M. (2004). Estimating Vs(30) (or NEHRP Site Classes) from Shallow Velocity Models (Depths < 30 m), *Bulletin of the Seismological Society of America*, *94*(2), 591–597.

Boore D.M., Stewart J.P., Seyhan E., & Atkinson G.M. (2014). NGA-West 2 Equations for Predicting PGA, PGV, and 5%-Damped PSA for Shallow Crustal Earthquakes, *Earthquake Spectra*, *30*(3), 1057–1085.

Bozorgnia Y., Abrahamson N.A., Al Atik L., Ancheta T.D., Atkinson G.M., Baker J.W., Baltay A., Boore D.M., Campbell K.W., Chiou B.S.J., Darragh R., Day S., Donahue J., Graves R.W., Gregor N., Hanks T., Idriss I.M., Kamai R., Kishida T., Kottke A., Mahin S.A., Rezaeian S., Rowshandel B., Seyhan E., Shahi S., Shantz T., Silva W., Spudich P., Stewart J.P., Watson-Lamprey J., Wooddell K., & Youngs R. (2014). NGA-West2 Research Project, *Earthquake Spectra*, *30*(3), 973-987.

California Building Standards Commission (CBSC). (2019). 2019 California Building Code.

Campbell K.W. & Bozorgnia Y. (2014). NGA-West2 Ground Motion Model for the Average Horizontal Components of PGA, PGV, and 5% Damped Linear Acceleration Response Spectra, *Earthquake Spectra* *30*(3), 1087–1115.

Chiou, B.S.J. & Youngs, R.R. (2014). Update of the Chiou and Youngs NGA Model for the Average Horizontal Component of Peak Ground Motion and Response Spectra, *Earthquake Spectra,* 30(3), 1117–1153.

Cornell, C.A. (1968). Engineering Seismic Risk Analysis: *Seismological Society of America Bulletin*, *58*(5).

Field, E. L., Biasi, G.P., Bird, P., Dawson, T.E., Felzer, K.R., Jackson, D.D., Johnson, K.M., Jordan, T.H., Madden, C., Michael, A.J., Milner, K.R., Page, M.T., Parsons, T., Powers, P.M., Shaw, B.E., Thatcher, W.R., Weldon, R.J., II, & Zeng, Y. (2013). The uniform California earthquake rupture forecast, version 3 (UCERF3)—The time-independent model (US Geological Survey Open-File Report 2013–1165, California Geological Survey Special Report 228, and Southern California Earthquake Center Publication 1792, US Geological Survey, 2013). *California Geological Survey Special Report 228, and Southern California Earthquake Center Publication 1792*, 97. http://pubs.usgs.gov/of/2013/1165.

Fugro. (2019). “EZ-FRISK, Software for Earthquake Ground Motion Estimation, Version 8.06,” Fugro USA Land, Inc. http://www.ez-frisk.com/.

Gregg Drilling, LLC. (2021). "CPT Site Investigation, GOCC, Pico Rivera, California", Gregg Drilling, LLC, GREGG Project Number D1215052, June 2021.

Moore Twining Associates, Inc. (2021). "Building Boring Logs with Lab Test Data, Pico Rivera, California", drilling conducted by Gregg Drilling & Testing, Project No. C73111.01, June 2021.

Petersen M. D., Moschetti, M. P., Powers, P. M., Mueller, C. S., Haller, K. M., Frankel, A. D., Zeng, Y, Rezaelian, S., Harmsen, S. C., Boyd, O. S., Field, N, Chen, R., Rukstales, K. S., Luco, N, Wheeler, R. L., Williams, R. A., & Olsen, A. H. (2014). *Documentation for the 2014 Update of the United States National Seismic Hazard Maps*, (No. 2014-1091). U.S. Geological Survey.

Wells, D.L. & Coppersmith, K.J. (1994). "New Empirical Relationships among Magnitude, Rupture Length, Rupture Width, Rupture Area, and Surface Displacement," Bulletin of the Seismological Society of America, *84*(4), 974-1002, August 1994.

List of Figures

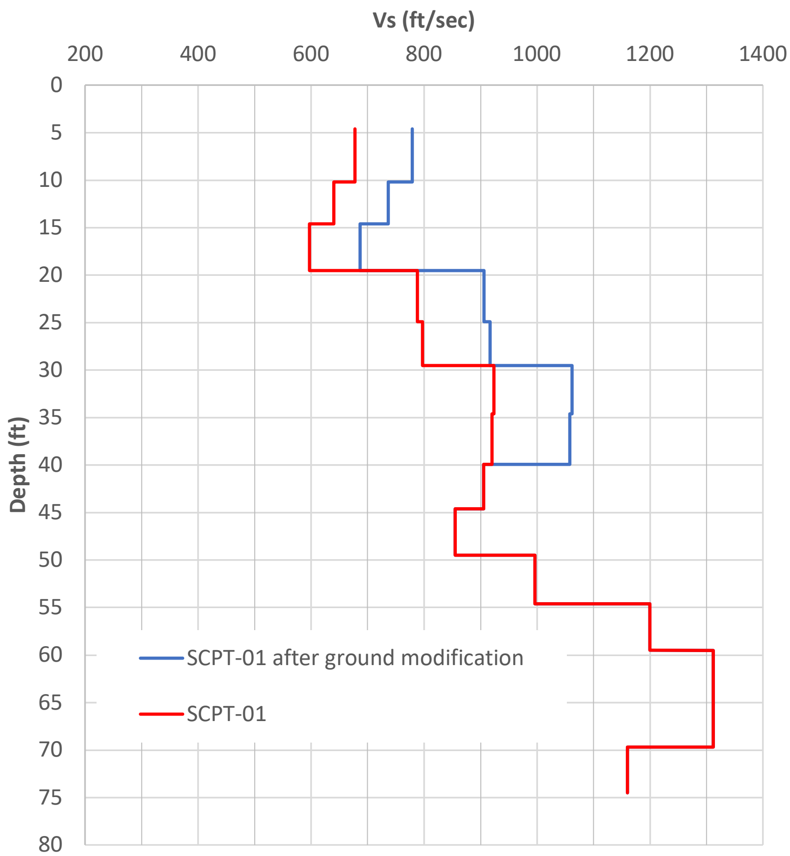


Figure 1: Shear Wave Velocity Profiles

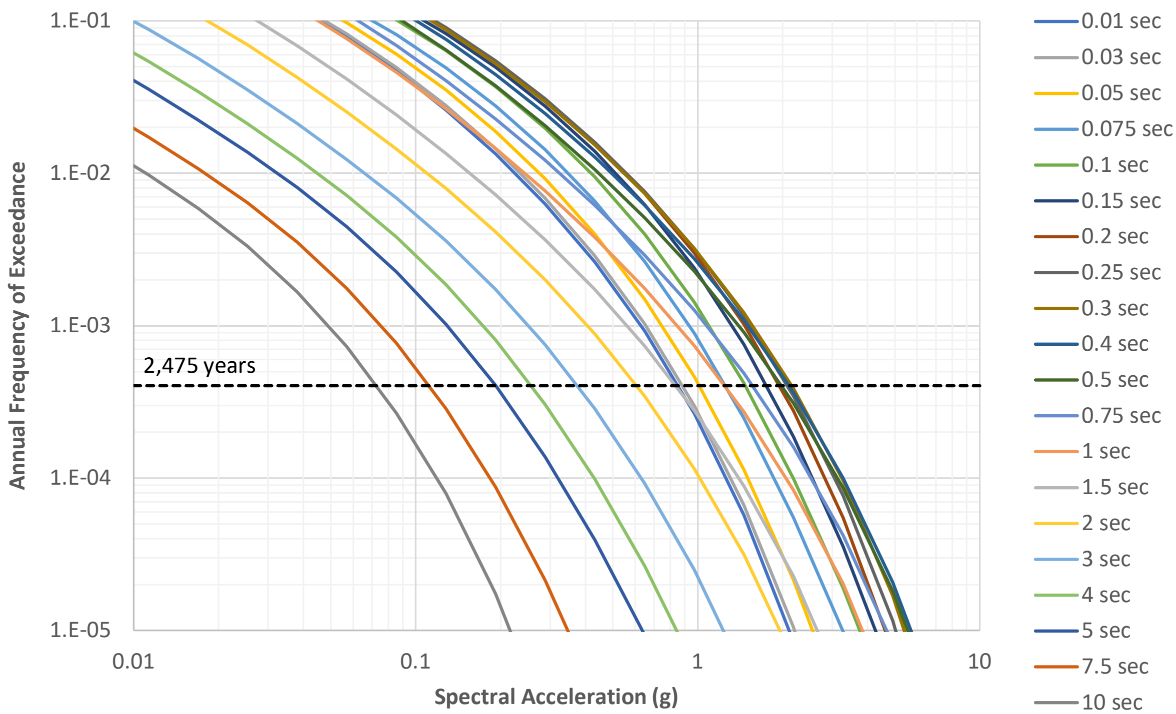


Figure 2: Mean Annual Seismic Hazard Curves for Vs30 of 310 m/s

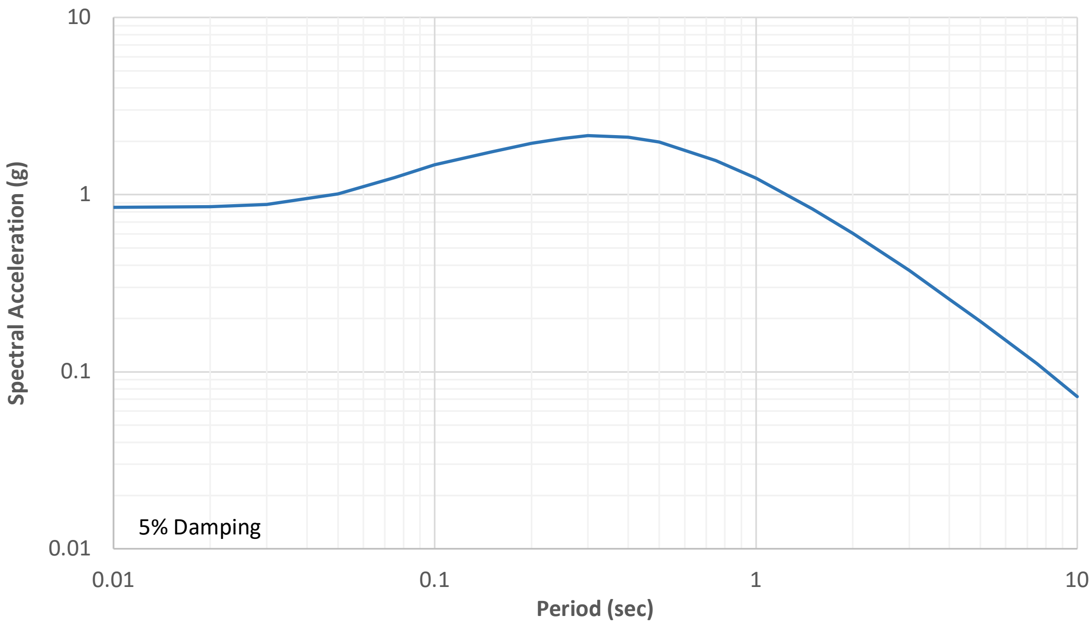
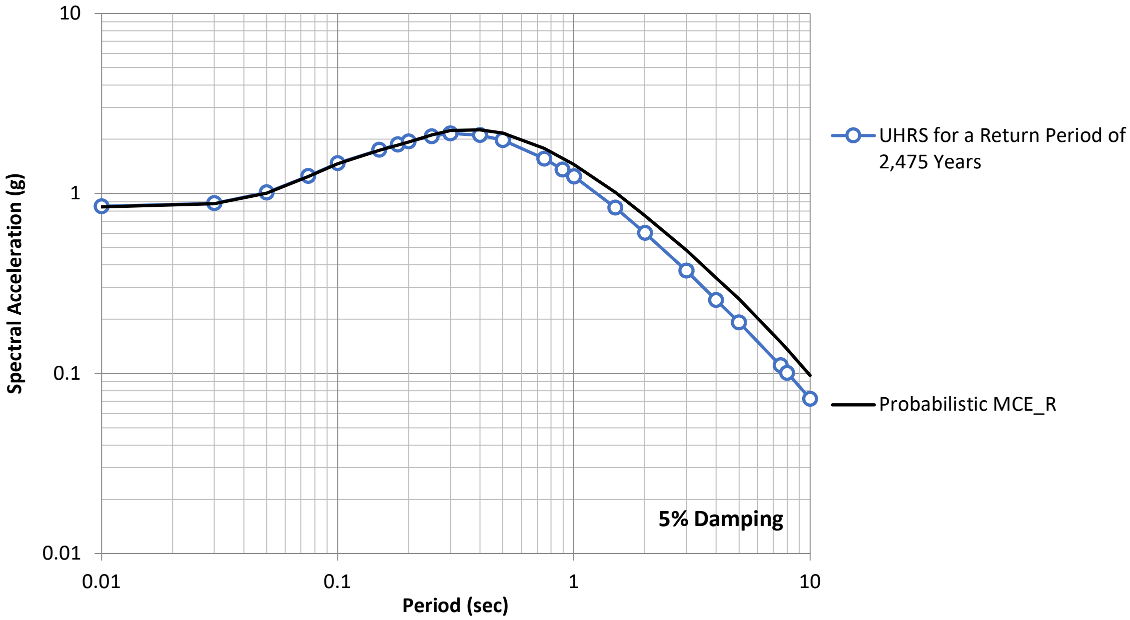


Figure 3: Mean Horizontal Uniform Hazard Response Spectrum for a Return Period of 2,475 Years and Vs30 of 310 m/s



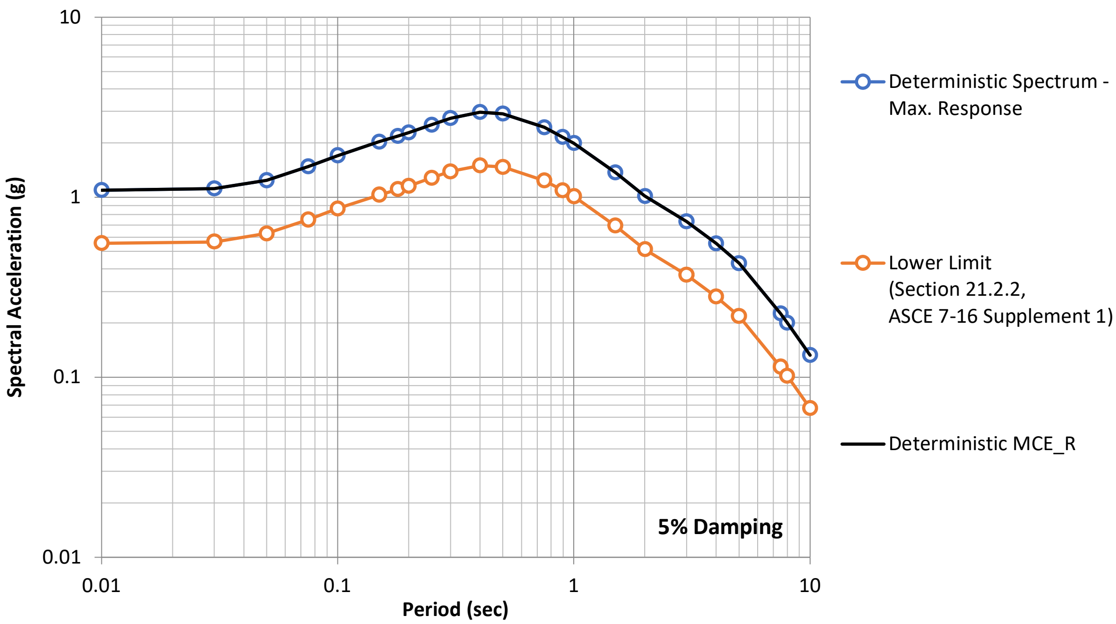
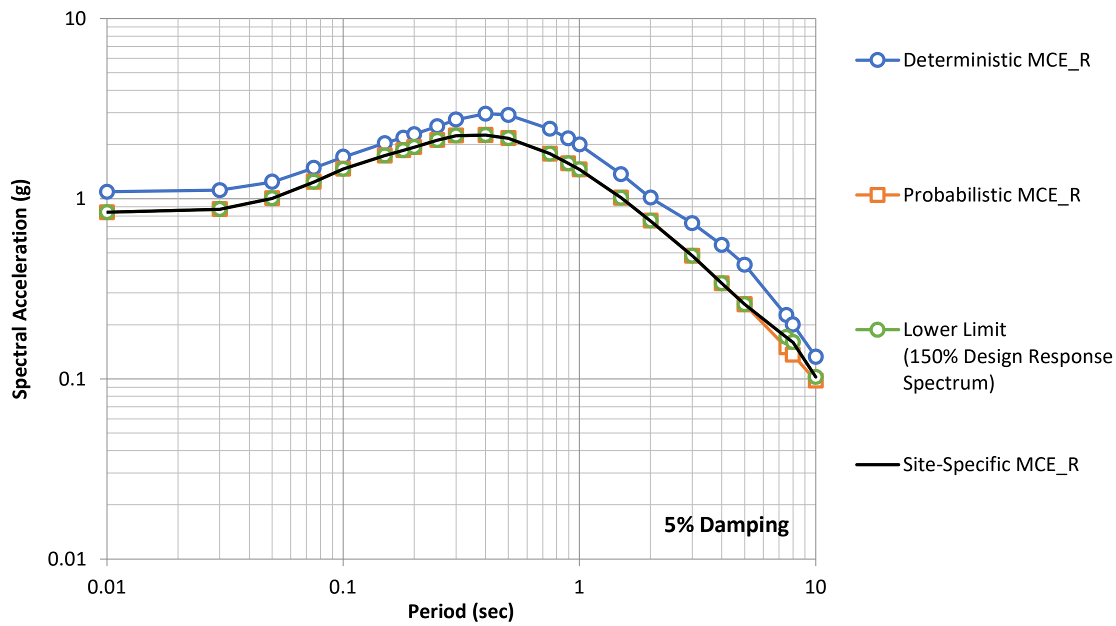


Figure 4: Calculation of the Probabilistic and Deterministic Horizontal MCE_R Response Spectra per ASCE 7-16 for Vs30 of 310 m/s



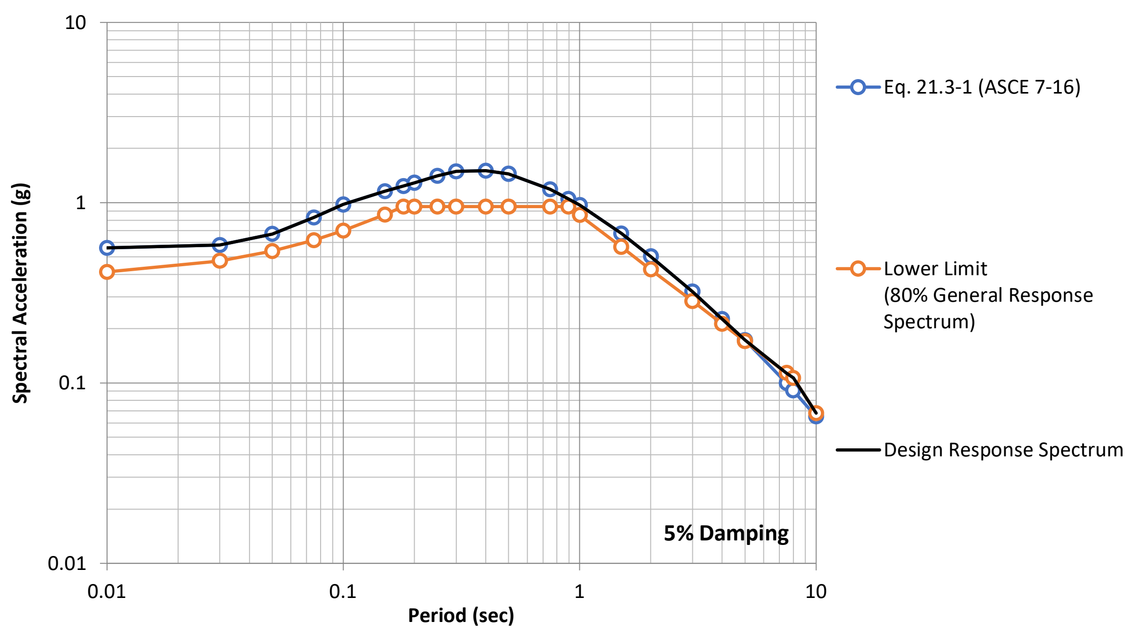


Figure 5: Calculation of the Site-Specific Horizontal MCE_R and Design Response Spectra per ASCE 7-16 for Vs30 of 310 m/s