GOLETA TRAIN DEPOT PROJECT
GOLETA, CALIFORNIA

GEOTECHNICAL EXPLORATION

SUBMITTED TO
Jim Keenan
Anil Verma Associates, Inc.
444 South Flower Street
Suite 1688
Los Angeles, CA 90071

PREPARED BY
ENGEO Incorporated

March 23, 2020

PROJECT NO.
16370.000.000

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Project No.
16370.000.000

March 23, 2020

Mr. Jim Keenan
Anil Verma Associates, Inc.
444 South Flower Street, Suite 1688
Los Angeles, CA 90071

Subject: Goleta Train Depot Project
La Patera Lane
Goleta, California

GEOTECHNICAL EXPLORATION

Dear Mr. Keenan:

ENGEO prepared this geotechnical report for the Goleta Train Depot Project as outlined in our agreement dated June 19, 2019. We characterized the subsurface conditions at the site to provide the enclosed geotechnical recommendations for design.

Our experience and that of our profession clearly indicate that the risk of costly design, construction, and maintenance problems can be significantly lowered by retaining the design geotechnical engineering firm to review the project plans and specifications and provide geotechnical observation and testing services during construction. Please let us know when working drawings are nearing completion, and we will be glad to discuss these additional services with you.

If you have any questions or comments regarding this report, please call and we will be glad to discuss them with you.

Sincerely,

ENGEO Incorporated

Randy Hildebrant

REGISTERED PROFESSIONAL ENGINEER
RANDY HILDEBRANT
No. 3090
GEOTECHNICAL
STATE OF CALIFORNIA

Randy Hildebrant, GE
rh/rhb/dt

Robert H. Boeche, CEG

CERTIFIED ENGINEERING GEOLOGIST
ROBERT BOECHE
No. 2318
STATE OF CALIFORNIA

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1.0 INTRODUCTION

1.1 PURPOSE AND SCOPE

ENGEO prepared this geotechnical report for design of a train depot in Goleta, California. We prepared this report as outlined in our agreement dated June 19, 2019. Anil Verma Associates, Inc. authorized ENGEO to conduct the following scope of services.

- Site reconnaissance, review of available geologic maps, and review of available on-line or in-house aerial photographs and historical topographs.
- Drilling five auger borings at accessible areas of the site to a maximum depth of 50 feet and four percolation test holes.
- Sampling and laboratory testing of select samples.
- Data analysis and conclusions.
- Report preparation.

For our use, we received the Request for Proposal for Professional Design Services for The Goleta Train Depot Project dated January 17, 2019, by the City of Goleta.

This report was prepared for the exclusive use of our client and their consultants for design of this project. In the event that any changes are made in the character, design, or layout of the development, we must be contacted to review the conclusions and recommendations contained in this report to evaluate whether modifications are recommended. This document may not be reproduced in whole or in part by any means whatsoever, nor may it be quoted or excerpted without our express written consent.

EXHIBIT 1.1-1

1.2 PROJECT LOCATION

Figure 1 displays a Vicinity Map. The proposed Train Depot site is located at 27 South La Patera Lane, in Goleta, California. The project also consists of improvement for South La Patera Lane from the train station and extending south to Hollister Avenue.

Figure 2 shows site boundaries and our exploratory locations. The site is bordered on the north by land owned by Union Pacific Railroad, which includes the existing train station platform. The lot south of the proposed train station depot site includes multiple buildings and their associated parking lots. The train depot site is bonded by an existing warehouse to the west and South La Patera Lane to the east. Improvements are also proposed for South La Patera Lane from the existing train station extending to Hollister Avenue.

Currently, a warehouse, loading platforms, and parking lots occupy the property. The warehouse occupies roughly half the train depot project area and is located in the northern middle of the project area. There is an approximately 4-foot grade change from exterior grades to the top of the loading platforms. Existing fuel tanks associated with an onsite power generator are located adjacent to the southwest corner of the existing warehouse and the approximate location is noted on Figure 2.

EXHIBIT 1.2-1

1.3 PROJECT DESCRIPTION

Based on our discussions with you and review of the information provided, we understand that the following site improvements are proposed:

1. Earthwork is assumed to be composed only of minor grading.

2. Demolition of the existing warehouse and construction of an 8,000-square-foot single-story train depot of light-framed construction.

3. Paved access ways and parking.

4. Utilities and other infrastructure improvements such as improvements to the north end of South La Patera Lane.

5. Concrete flatwork.

6. Post-construction stormwater treatment.

The depot building and parking will be located on land owned by the City of Goleta, located immediately adjacent to the existing platform. The train depot building will include a lobby, ticketing area, waiting room, café, community room, restrooms/shower/changing facilities, bike storage, and baggage lockers. The proposed project will not be modifying the existing platform and it is assumed new improvements will be outside of Railroad Right-of-Way. The project will also include access improvements along South La Patera Lane between Hollister Avenue and the proposed depot.

2.0 FINDINGS

2.1 SITE HISTORY

We reviewed available historical aerial photographs on www.historicaerials.com. The 1947 photograph shows the project site covered with orchards. The 1953 photograph shows the project site cleared of the orchards with the existing warehouse structure shown in the 1967 photograph.

2.2 GEOLOGY AND SEISMICITY

2.2.1 Geology

According to the United States Geologic Survey (USGS) (Minor et al. 2007, Figure 3), the train depot project area and the majority of South La Patera Lane is mapped as an area with upper Pleistocene-aged intermediate alluvial deposits consisting of weakly consolidated, stratified silt, sand, and gravel that form low, rounded, moderately dissected terraces and piedmont alluvial fans that rest at higher elevations compared to the younger than the younger Holocene- and upper Pleistocene-aged coastal piedmont alluvium and colluvium at lower elevations. The area near the intersection of South La Patera Lane with Hollister Avenue is mapped as Holocene- and upper Pleistocene-aged alluvium and colluvium consisting of poorly consolidated silt, sand, and gravel deposits of modern drainages and piedmont alluvial fans and floodplains.

2.2.2 Seismicity

The Santa Barbara County area contains numerous active earthquake faults. An active fault is defined by the State Mining and Geology Board as one that has had surface displacement within Holocene time (about the last 11,000 years) (Bryant and Hart, 2007).

The site is not located within a currently designated Alquist-Priolo Earthquake Fault Zone and no known surface expression of active faults is believed to exist within the site. Fault rupture through the site, therefore, is not anticipated.

The site does lie within a seismically active region. According to a search using the 2008 National Seismic Hazard Maps spatial search feature, the nearest active fault is the Mission Ridge-Arroyo Parida-Santa Ana fault, which is mapped approximately 0.6 mile from the site. This fault is considered capable of a moment magnitude earthquake of 6.9. Other active faults in the region are summarized in the table below.

TABLE 2.2.2-1: Active Faults Capable of Producing Significant Ground Shaking at the Site

FAULT NAME	DISTANCE FROM SITE (MILES)	MAXIMUM MOMENT MAGNITUDE
Mission Ridge-Arroyo Parida-Santa Ana	0.6	6.9
Red Mountain	4.0	7.4
North Channel	6.2	6.8
Pitas Point Connected	7.0	7.3
Santa Ynez Connected	7.8	7.4
Oak Ridge Connected	15.8	7.4

The regional seismicity of the Central California Coast was recently evaluated by the Working Group on Southern California / Los Angeles Region. Their UCERF3 model estimates a greater increase in the likelihood of larger earthquakes in the region compared to most of California, because the region has more faults that can host multi-fault ruptures. The UCERF3 model concurs with previous studies that consider the Southern San Andreas Fault, located approximately 43 miles northeast of the site, the most likely to host a large earthquake.

According to UCERF2 & 3, the 30-year probability for a Magnitude 6.7 or greater earthquake along the Mission Ridge-Arroyo Parida-Santa Ana, Subsection 1, nearest to the site, is 0.30%. The 30-year probability for a Magnitude 6.7 or greater earthquake on the Red Mountain Fault, Subsection 6, is 2.84% and 3.20% on Subsection 5 of that same fault. Estimates for the Pitas Point fault is about 1.1%. Santa Ynez Fault Zone Subsection 13 estimates are about 1.76 % and 2.34% for the Oak Ridge fault (Onshore), Subsection 0. UCERF3 shows the Channel Islands Western Deep Ramp fault, Subsection 0, located 11.4 miles south of the site, with a 0.47% probability of a >6.7M earthquake within the next 30 years.

Based on the historic seismicity, the proximity of known active faults, and the estimated earthquake probabilities for the Central California area as a whole, it should be expected that the site will experience strong seismic ground shaking during the lifetime of the proposed improvements. The ground shaking hazard levels at the site are similar to those for most of the Central Coast.

2.3 FIELD EXPLORATION

Our field exploration included an initial hand auger exploration and placement of a shallow piezometer, drilling five borings, and performing four percolation tests. We performed our field exploration between August 12 and August 14, 2019, and completed the preliminary hand auger on July 1, 2019. The deepest boring terminated at 51½ feet below the ground surface.

The location and elevations of our explorations are approximate and were estimated by pacing from features shown on Figure 2; they should be considered accurate only to the degree implied by the method used.

2.3.1 Hand Auger

We performed a hand auger boring near the intersection of South La Patera Lane and Hollister Avenue to a maximum depth of 13½ feet below the ground surface. Following the boring, we placed a PVC pipe with perforations in the lower approximately 2 feet and backfilled the annulus with pea gravel. The location of the hand auger boring is shown on Figure 2 and the boring log is included in Appendix A.

2.3.2 Borings

We observed drilling of five borings at the locations shown on the Site Plan, Figure 2. An ENGEO Engineer observed the drilling and logged the subsurface conditions at each location. We retained a CME 75 – Rubber Track Mounted Drill Rig and crew to advance the borings using 8-inch-diameter hollow-stem auger methods. The borings were advanced to depths ranging from 11½ to 51½ feet below existing grade.

We obtained bulk soil samples from drill cuttings and retrieved disturbed soil samples at various intervals in the borings using both a Standard Penetration Test split spoon sampler and 3-inch outer diameter (O.D.) split-spoon sampler outfitter with 2.5-inch diameter stainless steel liners.

The blow counts were obtained by using a 140-pound auto-hammer with a 30-inch free fall. The samplers were driven 18 inches and the number of blows were recorded for each 6 inches of penetration. Unless otherwise indicated, the blows per foot recorded on the boring log represent the accumulated number of blows to drive the last 1 foot of penetration; the blow counts have not been converted using any correction factors. When sampler driving was difficult, penetration was recorded only as inches penetrated for 50 hammer blows. We used the field logs to develop the boring logs presented in Appendix A.

The boring logs graphically depict the subsurface conditions encountered at the time of exploration, and describe the soil type, color, consistency, and visual classification in general accordance with the United Soil Classification System (USCS). Subsurface conditions at other locations may differ from conditions occurring at these boring locations, and the passage of time may result in altered subsurface conditions. In addition, stratification lines represent the approximate boundaries between soil types, and the transitions may be gradual.

PHOTO 2.3.2-1: Boring Inside Existing Warehouse

2.3.3 Percolation Tests

Percolation testing was performed using the borehole method as generally described in The Guidelines for Geotechnical Investigation and Reporting Low Impact Development Stormwater Infiltration dated June 30, 2017, by the County of Los Angeles Department of Public Works Geotechnical and Materials Engineering Division. Percolation Test Holes P1 through P3 were drilled with a 6-inch solid-stem auger while Percolation Test Hole P4 was drilled with an 8-inch hollow-stem auger. All locations were generally performed to an approximate depth of 4½ feet. The bottom 1 to 2 inches was covered with pea gravel, a 4-inch perforated pipe was inserted, and annulus filled with 1-inch minus river rock. All test locations were filled with water to the ground surface a minimum of the day prior to running the test to obtain a near saturated condition. Prior to running the test procedure, water was either added or removed to provide the initial 12 inches of water, measured from the top of the pea gravel. The water level was measured in frequent intervals over the course of 8 hours with a final measurement taken the following day prior to backfilling of the test holes. A bulk soil sample was collected from the upper 1 to 3 feet of Test P2. Other percolation test locations were adjacent to boring locations.

2.4 SURFACE CONDITIONS

The train depot project site is generally level with a loading ramp located in the northeastern portion. The existing warehouse covers roughly half the project site with either asphalt pavement or concrete covering the remaining surface with small landscape areas near South La Patera Lane. The existing warehouse floor elevation is up to about 4 feet higher than surrounding grade to accommodate loading without a ramp in the northwestern portion. There is about a 20-foot elevation differential between the train depot location and the intersection of South La Patera Lane and Hollister Avenue. As noted previously, underground fuel tanks are located adjacent to the southwest corner of the warehouse. Area drains are also located throughout the hardscape area.

PHOTO 2.4-1: East Side of Train Station Depot Site

PHOTO 2.4-2: West Side of Train Station Depot Site

PHOTO 2.4-3: South La Patera Lane at Hollister Avenue

2.5 SUBSURFACE CONDITIONS

The borings generally encountered an upper layer of stiff to hard sandy lean clay, which ranged between 8 and 14 feet in thickness. The Plasticity Index ranged between 2 and 21, indicating a low to medium shrink/swell potential. Underlying the clay, the borings encountered varying layers of clayey sand, silty sand, silt, and lean clay. Sandy layers ranged from medium dense to very dense and clayey layers were stiff to hard. Borings 1-B2 and 1-B3 encountered a hard lean clay layer with marine shells at depths of approximately 38 feet and 35 feet respectively. Underlying the existing warehouse, Boring 1-B3 encountered approximately 5 feet of hard lean clay fill with Plasticity Indices ranging between 8 and 31, indicating high variability of the fill and a low to high shrink/swell potential.

Consult the Site Plan and exploration logs for specific subsurface conditions at each location. We include our exploration logs in Appendix A. The logs contain the soil type, color, consistency, and visual classification in general accordance with the Unified Soil Classification System. The logs graphically depict the subsurface conditions encountered at the time of the exploration.

2.6 GROUNDWATER CONDITIONS

We observed static groundwater in two of our subsurface explorations. Groundwater was encountered at 20 feet below the ground surface at boring 1-B5 and 30 feet below the ground surface at Boring 1-B2. Boring 1-B5 is located nearly 1,500 feet from the train depot project site

Fluctuations in the level of groundwater may occur due to variations in rainfall, irrigation practice, and other factors not evident at the time measurements were made.

2.7 LABORATORY TESTING

Select samples recovered during drilling activities were tested to determine various soil characteristics:

TABLE 2.7-1: Laboratory Testing

CHARACTERISTIC	TEST METHOD
Natural Moisture Content	ASTM D2216
Plasticity Index	ASTM D4318
Hydrometer	ASTM D422
Particle Size Distribution	ASTM D1140
Unconfined Compression	ASTM D2166
R-Value	CTM-301

Moisture contents, dry densities, plasticity indices, and fines contents are recorded on the boring logs in Appendix A; other laboratory data and individual test results are included in Appendix B.

3.0 CONCLUSIONS

From a geotechnical engineering viewpoint, in our opinion, the site is suitable for the proposed development, provided the geotechnical recommendations in this report are properly incorporated into the design plans, specifications, and construction. The primary geotechnical concerns that could affect development on the site is expansive soils and strong ground shaking. We summarize our conclusions below.

3.1 EXPANSIVE SOILS

We observed potentially expansive lean clay near the surface of the site in Borings 1-B2, 1-B3, and 1-B4, which may exhibit low to high shrink/swell potential with variations in moisture content.

Expansive soils change in volume with changes in moisture. They can shrink or swell and cause heaving and cracking of slabs-on-grade, pavements, and structures founded on shallow foundations. Building damage due to volume changes associated with expansive soils can be reduced by: (1) using a rigid mat foundation that is designed to resist the settlement and heave of expansive soil, (2) deepening the foundations to below the zone of moisture fluctuation, i.e. by using deep footings or drilled piers, and/or (3) using footings at normal shallow depths but bottomed on a layer of select fill having a low expansion potential.

Successful performance of structures on expansive soils requires special attention during construction. It is imperative that exposed soils be kept moist prior to placement of concrete for foundation construction. It can be difficult to remoisturize clayey soils without excavation, moisture conditioning, and recompaction.

We have also provided specific grading recommendations for compaction of clay soil at the site. The purpose of these recommendations is to reduce the swell potential of the clay by compacting the soil at a high moisture content and controlling the amount of compaction. The effects of expansive soil may be reduced with proper foundation design and construction.

3.2 EXISTING FILL

Our borings indicate that portions of the site are underlain by existing fill. It is unclear what level of moisture conditioning or compaction was performed on the fill without proper documentation.

Without proper documentation of existing fill placed on the site, we recommend complete removal and recompaction of the existing fill. We present fill removal recommendations in Section 5.1.

3.3 SEISMIC HAZARDS

Potential seismic hazards resulting from a nearby moderate to major earthquake can generally be classified as primary and secondary. The primary effect is ground rupture, also called surface faulting. The common secondary seismic hazards include ground shaking, and ground lurching. The following sections present a discussion of these hazards as they apply to the site. Based on topographic and lithologic data, the risk of regional subsidence or uplift, landslides, and seiches is considered low to negligible at the site.

3.3.1 Ground Rupture

Since there are no known active faults crossing the property and the site is not located within an Earthquake Fault Special Study Zone, it is our opinion that ground rupture is unlikely at the subject property.

3.3.2 Ground Shaking

An earthquake of moderate to high magnitude generated within the Santa Barbara region could cause considerable ground shaking at the site, similar to that which has occurred in the past. To mitigate the shaking effects, structures should be designed using sound engineering judgment and the most recent California Building Code (CBC) requirements, as a minimum. Seismic design provisions of current building codes generally prescribe minimum lateral forces, applied statically to the structure, combined with the gravity forces of dead-and-live loads. The code-prescribed lateral forces are generally considered to be substantially smaller than the comparable forces that would be associated with a major earthquake. Therefore, structures should be able to: (1) resist minor earthquakes without damage, (2) resist moderate earthquakes without structural damage but with some nonstructural damage, and (3) resist major earthquakes without collapse but with some structural as well as nonstructural damage. Conformance to the current building code recommendations does not constitute any kind of guarantee that significant structural damage would not occur in the event of a maximum magnitude earthquake; however, it is reasonable to expect that a well-designed and well-constructed structure will not collapse or cause loss of life in a major earthquake (SEAOC, 1996).

3.3.3 Liquefaction

Seismically induced soil liquefaction is a process by which soil undergoes a significant loss of strength due to cyclic loading and corresponding increase in pore water pressure. The effects of liquefaction can be a drastic decrease in soil shear strength, vertical settlement, lateral spreading and ground surface disruptions. Soils most susceptible to liquefaction are clean, loose, saturated, uniformly graded fine sands below the groundwater table. Empirical evidence and laboratory testing indicates that loose to medium dense gravels, silty sands, low-plasticity silts, and some low-plasticity clays are also potentially liquefiable.

We performed a liquefaction potential analysis of blow count to estimate liquefaction potential using the procedure introduced by the 1996 National Center for Earthquake Engineering Research (NCEER) workshop and the 1998 NCEER/National Science Foundation (NSF) workshop. The workshops are summarized by Youd et al. (2001). The Cyclic Stress Ratio (CSR) was estimated from the PGA_M of 1.11g. The Magnitude Scaling Factor (MSF) was estimated for a mean Moment Magnitude of 7.4. The results indicate that a silty sand layer located below the groundwater level in Boring 1-B3 is potentially liquefiable.

3.3.4 Seismically Induced Settlement Analyses

Seismically induced settlement can be generally subdivided into two categories for granular soils, settlement as a result of liquefaction of saturated or nearly saturated soils and dynamic densification of non-saturated soils. We have included recommendations for mitigation of seismic settlement in our Foundation Recommendations.

3.3.4.1 Liquefaction-Induced Settlement

Deformation of the ground surface is a common result of liquefaction. Vertical settlement may result from densification of the deposit or volume loss from venting to the ground surface. Densification occurs as excess pore pressures dissipate, resulting as vertical settlement at the ground surface. In addition to the above analysis, we also evaluated the capping effect of any overlying non-liquefiable soils. In order for liquefaction-induced ground failure to occur, the pore water pressure generated within the liquefied strata must exert a sufficient enough force to break through the overlying soil and vent to the surface resulting in sand boils or fissures.

In 1985, Ishihara presented preliminary empirical criteria to assess the potential for ground surface disruption at liquefiable sites based on the relationship between thickness of liquefiable sediments and thickness of overlying non-liquefiable soil. A more recent study by Youd and Garris (1995) expanded on the work of Ishihara to include data from over 308 exploratory borings, 15 different earthquakes, and several ranges of recorded peak ground acceleration.

Based on the above studies and thickness of liquefiable material, it appears there is a sufficient cap of non-liquefiable material to reduce the risk of surface venting.

We calculated potential liquefaction-induced settlement estimate using Ishihara and Yoshimine (1992). We estimate the total liquefaction-induced settlement based on Boring 1-B3 to be less that 1 inch.

3.3.4.2 Dynamic Densification

Densification of loose granular soil above the water table can cause settlement of the ground surface due to earthquake-induced vibrations. Sands encountered above the assumed groundwater level at the site medium dense to dense. We estimate that these deposits may settle up to about ⅓ inch in Boring 1-B2 using the procedure by Tokimatsu and Seed (1984/1987).

3.3.5 Lateral Spreading

Lateral spreading is a failure within weaker soil material, such as lurching or liquefaction, which causes the soil to move toward a free face or down a slope. Due to relatively level topography and distance to free faces, it is our opinion that the risk of lateral spreading is low.

3.3.6 Ground Lurching

Ground lurching is a result of the rolling motion imparted to the ground surface during energy released by an earthquake. Such rolling motion can cause ground cracks to form in weaker soils. The potential for the formation of these cracks is considered greater at contacts between deep alluvium and bedrock. Such an occurrence is possible at the site as in other locations in the Santa Barbara region, but based on the site location, it is our opinion that the offset is expected to be minor. We provide recommendations for foundation and pavement design in this report that are intended to reduce the potential for adverse impacts from lurch cracking.

3.3.7 Flooding and Tsunamis

The Civil Engineer should review pertinent information relating to possible flood levels for the subject site based on final pad elevations and provide appropriate design measures for development of the project, if recommended.

3.4 STATIC AND PERCHED GROUNDWATER

It does not appear that the static groundwater level beneath the site is likely to affect the proposed development. However, perched water can:

1. Impede grading activities.

2. Cause moisture damage to sensitive floor coverings.

3. Transmit moisture vapor through slabs causing excessive mold/mildew build-up, fogging of windows, and damage to computers and other sensitive equipment.

4. Cause premature pavement failure if hydrostatic pressures build up beneath the section.

We provide recommendations to reduce the effects of perched water in subsequent sections including the use of vapor retarders and cut-off curbs.

3.5 SOIL CORROSION POTENTIAL

As part of this study, we obtained representative soil samples from both the project site and the borrow site and submitted to a qualified analytical lab for determination of pH, resistivity, sulfate, and chloride. The results are included in Appendix B and summarized in the table below.

TABLE 3.5-1: Corrosion Potential Test Results

SAMPLE NUMBER AND DEPTH	REDOX POTENTIAL (mV)	pH	RESISTIVITY (ohms-cm)	CHLORIDE CONCENTRATION (mg/kg)	SULFATE CONCENTRATION (mg/kg)
1-B2 @ 1-3’	180	8.85	1,900	N.D.	30
1-B3 @ 1-3’	200	7.87	1,400	N.D.	43
1-B4 @ 1-3’	210	7.71	4,400	N.D.	25

According to Cerco Analytical, based upon the resistivity measurements, 1-B2 @ 1-3’ and 1-B3 @ 1-3’ are classified as “corrosive” and 1-B4 @ 1-3’ is classified as “moderately corrosive.” All buried iron, steel, cast iron, ductile iron, galvanized steel, and dielectric coated steel or iron should be properly protected against corrosion.

The chloride ion concentrations were reported as none detected with a reporting limit of 15 mg/kg.

The sulfate ion concentrations are determined to be insufficient to damage reinforced concrete structures and cement mortar-coated steel at these locations.

The pH of the soils ranged from 7.71 and 8.85 and does not present corrosion problems for buried iron, steel, mortar-coated steel, and reinforced concrete structures.

The redox potential of 1-B2 @ 1-3' is indicative of potentially "moderately corrosive" soils and the remaining samples are indicative of potentially "slightly corrosive" soils resulting from anaerobic soil conditions.

Considering a 'Not Applicable' sulfate exposure according to ACI 318, a minimum concrete compressive strength of 2,500 psi is specified by the building code. It should be noted, however, that the structural engineering design requirements for concrete may result in more stringent concrete specifications. We recommend using a maximum water-to-cement ratio of 0.50 to reduce vapor intrusion. If desired to investigate further, we recommend consultation with a corrosion engineer.

3.6 NATURALLY OCCURRING RADON GAS

Radon is a radioactive gas formed by the decay of small amounts of uranium and thorium naturally present in rock and soil. Sometimes radon gas can move from underlying soil and rock into houses and become concentrated in indoor air. According to research performed by the California Geological Survey (CGS) (Churchill, 2008) high radon potential areas relate to a group of Monterey Formation geologic units and portions of adjacent alluvial units that have a Monterey Formation component. In Santa Barbara and Ventura counties, Rincon Shale was identified as a radon prone geologic unit (Churchill, 1997). The CGS has mapped the project area as an area overlain by soil and rock was not encountered in any of our exploration locations, therefore, the potential for naturally occurring radon gas is low.

4.0 CONSTRUCTION MONITORING

Our experience and that of our profession clearly indicate that the risk of costly design, construction, and maintenance problems can be significantly lowered by retaining the design geotechnical engineering firm to:

1. Review the final grading, improvement, and foundation plans and specifications prior to construction to evaluate whether our recommendations have been implemented, and to provide additional or modified recommendations, as needed. This also allows us to check if any changes have occurred in the nature, design or location of the proposed improvements and provides the opportunity to prepare a written response with updated recommendations.

2. Perform construction monitoring to check the validity of the assumptions we made to prepare this report. Earthwork operations should be performed under the observation of our representative to check that the site is properly prepared, the selected fill materials are satisfactory, and that placement and compaction of the fills has been performed in accordance with our recommendations and the project specifications. Sufficient notification to us prior to earthwork is important.

If we are not retained to perform the services described above, then we are not responsible for any party's interpretation of our report (and subsequent addenda, letters, and verbal discussions).

5.0 EARTHWORK RECOMMENDATIONS

As used in this report, relative compaction refers to the in-place dry unit weight of soil expressed as a percentage of the maximum dry unit weight of the same soil, as determined by the ASTM D1557 laboratory compaction test procedure, latest edition. Compacted soil is not acceptable if it is unstable; it should exhibit only minimal flexing or pumping, as observed by an ENGEO representative. The term "moisture condition" refers to adjusting the moisture content of the soil by either drying if too wet or adding water if too dry.

We define "structural areas" as any area sensitive to settlement of compacted soil. These areas include, but are not limited to building pads, sidewalks, pavement areas, and retaining walls.

5.1 EXISTING FILL REMOVAL

In the area of the proposed building structure, remove existing fill to competent native soil, as evaluated by an ENGEO representative. The lateral extent and depth of fill is expected to vary. Fill should be more prominent underlying the existing warehouse structure and is estimated to be up to about 5 feet in thickness. Removed material may be reused as engineered fill if it meets the recommendations of Section 5.4; however, due to the relative shrink/swell potential compared to the native site material, we do not recommend the existing fill be placed within the envelope of the proposed train station depot building if conventional footings with slab-on-grade is utilized for the foundation type. Fill may remain in place in areas outside the proposed building if the fill, as observed by ENGEO, appears firm and meets the recommendations of Section 5.4.

5.2 GENERAL SITE CLEARING

Areas to be developed should be cleared of surface and subsurface deleterious materials, including existing building foundations, slabs, buried tanks, utility and irrigation lines, pavements, debris, and designated trees, shrubs, and associated roots. Clean and backfill excavations extending below the planned finished site grades with suitable material compacted to the recommendations presented in Section 5.6. Retain ENGEO to observe and test backfilling.

5.3 OVER-OPTIMUM SOIL MOISTURE CONDITIONS

The contractor should anticipate encountering excessively over-optimum (wet) soil moisture conditions during winter or spring grading, or during or following periods of rain. Wet soil can make proper compaction difficult or impossible. Wet soil conditions can be mitigated by:

1. Frequent spreading and mixing during warm dry weather.
2. Mixing with drier materials.
3. Mixing with a lime, lime-flash, or cement product; or
4. Stabilizing with aggregate, geotextile stabilization fabric, or both.

Options 3 and 4 should be evaluated by ENGEO prior to implementation.

5.4 ACCEPTABLE FILL

Onsite soil material is suitable as fill material provided it is processed to remove concentrations of organic material, debris, and particles greater than 4 inches in maximum dimension.

Imported fill materials should meet the above requirements and have a plasticity index less than 12. Allow ENGEO to sample and test proposed imported fill materials at least 5 days prior to delivery to the site.

5.5 REUSE OF ONSITE RECYCLED MATERIALS

If desired to reuse asphaltic or Portland Cement concrete as engineered fill, we recommend that it be ground up and thoroughly mixed with onsite or import soil. In general, recycled asphalt or concrete should be ground down to less than 4 inches in greatest dimension, with no more than 25 percent larger than 2½ inches. Recycled material should be thoroughly mixed with a sufficient amount of soil, such that there is no more than 40 percent by weight of recycled material in the final mix.

We recommend that fill containing recycled asphalt and concrete be placed near the bottom of the proposed basement fills and/or spread out evenly across the site. Recycled fill should not be used within 2 feet of finished grade in building or roadway areas.

If proper equipment is used and quality control standards implemented, recycled material may be used as Class 2 Aggregate Subbase or Base if laboratory testing shows it meets Caltrans specifications for the material.

5.6 FILL COMPACTION

5.6.1 Grading in Structural Areas

The exposed non-yielding surface to receive fill or improvements should be scarified to a depth of 8 inches, moisture conditioned, and recompacted to provide adequate bonding with the initial lift of fill. Fill should be placed in loose lifts lift thickness not exceeding 8 inches.

We provide the following compaction recommendations:

TABLE 5.6.1-1: Compaction Recommendations

FILL DEPTH FROM PROPOSED FINISH GRADE	MINIMUM PERCENTAGE POINTS OVER OPTIMUM MOISTURE CONTENT	RELATIVE COMPACTION
Onsite soil	3	90% min.
Non-expansive building pad fill	0	95% min.
Pavement Subgrade (upper 12 inches)	2	95% min.
Non-expansive trench backfill	0	90% min.
Caltrans Class 2 AB (sidewalk, pavement, curb, and gutter)	0	95% min

Relative compaction refers to in-place dry density of the fill material expressed as a percentage of the maximum dry density (as determined by ASTM D-1557). Optimum moisture is the moisture content corresponding to the maximum dry density.

5.6.2 Underground Utility Backfill

The contractor is responsible for conducting all trenching and shoring in accordance with CALOSHA requirements. Project consultants involved in utility design should specify pipe-bedding materials. Trench backfill should be compacted in accordance with the recommendations provided in Section 5.6.1. In general, we do not recommend the use of rock backfill with little to no fines. ENGEO should be consulted prior to use.

Where utility trenches cross underneath buildings, we recommend that a plug be placed within the trench backfill to help prevent the normally granular bedding materials from acting as a conduit for water to enter beneath the building. The plug should be constructed using a sand cement slurry (minimum 28-day compressive strength of 500 psi) or relatively impermeable native soil for pipe bedding and backfill. We recommend that the plug extend for a distance of at least 3 feet in each direction from the point where the utility enters the building perimeter.

Jetting of backfill is not an acceptable means of compaction. We may allow thicker loose lift thicknesses based on acceptable density test results, where increased effort is applied to rocky fill or for the first lift of fill over pipe bedding.

5.7 SITE DRAINAGE

5.7.1 Surface Drainage

The project civil engineer is responsible for designing surface drainage improvements. With regard to geotechnical engineering issues, we recommend that finish grades be sloped away from buildings and pavements to the maximum extent practical to reduce the potentially damaging effects of expansive soil. The latest California Building Code Section 1804.4 specifies minimum slopes of 5 percent away from foundations. Where property boundaries or surface improvements restrict meeting this slope requirement, we recommend that specific drainage requirements be developed. As a minimum, we recommend the following:

1. Discharge roof downspouts into closed conduits and direct away from foundations to appropriate drainage devices.

2. Do not allow water to pond near foundations, pavements, or exterior flatwork.

5.8 STORMWATER INFILTRATION

We performed percolation testing on August 14, 2019. Generally, percolation rates are very low, less than 2¼ inches over an 8-hour period. A final measurement was taken the morning of August 15, 2019 prior to backfilling the holes with cement grout. This is further supported by the density and stiffness of the site soils and fines content (percentage passing the No. 200 sieve) generally exceeding 30 percent. Percolation test results are included in Appendix C. In some of the test locations, the readings show an increase in water level within the borehole with time. We speculate that due to the very low percolation rate and removing water to establish 12 inches of water for a starting point, water seeped into the borehole from the wetted sidewalls. Therefore, the following percolation rates were calculated from the final reading a day after the initial start of the test. No correction factors have been applied to the below percolation rates.

TABLE 5.8-1: Percolation Rates

TEST LOCATION	SOIL	PERCOLATION RATE
P-1	Sandy Lean Clay	1,490 min/in
P-2	Sandy Lean Clay	945 min/in
P-3	Sandy Silt	446 min/in
P-4	Sandy Lean Clay	390 min/in

In accordance with the Stormwater Technical Guide for Low Impact Development, Compliance with Stormwater Post-Construction Requirements in Santa Barbara County dated February 18, 2014, onsite testing information is used to generally justify using an infiltration rate of 0.5 in/hr (120 min/in) or greater. Therefore, Best Management Practices should assume that negligible stormwater infiltration will occur at the site.

6.0 FOUNDATION RECOMMENDATIONS

We developed foundation recommendations using data obtained from our field exploration, laboratory test results, and engineering analysis. As previously mentioned, foundations should be appropriate to reduce the effects of expansive soil. We recommend three foundation types.

- Post-tensioned mat slab.
- Conventionally reinforced mat slab.
- Conventional footings with interconnected grade-beams, slab-on-grade, and non-expansive pad cap.

6.1 POST-TENSIONED MAT SLAB

The proposed train station depot building may be supported on post-tensioned (PT) mat foundations bearing on prepared native soil or engineered fill.

The Structural Engineer should determine the actual PT mat thickness using the geotechnical recommendations in this report; we defer to the professional judgment of the Structural Engineer on the necessary mat thickness. ENGEO should be retained to review the PT mat foundation design. We recommend that the thickened edge be at least 12 inches wide.

The PT mat may be designed for an average allowable bearing pressure of up to 1,000 pounds per square foot (psf) for dead-plus-live loads with maximum localized bearing pressures of 1,500 psf at column or wall loads. Allowable bearing pressures can be increased by one-third for wind or seismic loads. Design PT mats using the criteria presented in Table 6.1-1.

TABLE 6.1-1: Post-Tensioned Mat Design Recommendations

CONDITION	CENTER LIFT	EDGE LIFT
Edge Moisture Variation Distance, e_m (feet)	**7.7**	**4.1**
Differential Soil Movement, y_m (inches)	**0.5**	**1.2**

The above values are based on the procedure presented by the Post-Tensioning Institute "Design of Post-Tensioned Slabs-on-Ground" Third Edition, including appropriate addenda (2004) or "Standard Requirements for Design and Analysis of Shallow Post-Tensioned Concrete Foundations on Expansive Soils" (PTI DC 10.5-12).

Underlay PT mats with a moisture reduction system as recommended below. In addition, moisture conditioning of the building foundation subgrade should be to a moisture content at least five percentage points above optimum immediately prior to foundation construction. The subgrade should not be allowed to dry prior to concrete placement. We also recommend that ENGEO be retained to observe the pre-pour moisture conditions to check that our report recommendations have been followed.

6.1.1 Additional Settlement Requirements

We recommend that PT mats designed in accordance with the above recommendations be checked for a differential settlement of ½ inch over a distance of 30 feet for the non-collapse seismic case.

6.1.2 Slab Moisture Vapor Reduction

When buildings are constructed with concrete slab-on-grade, such as post-tensioned mats, water vapor from beneath the slab will migrate through the slab and into the building. This water vapor can be reduced but not stopped. Vapor transmission can negatively affect floor coverings and lead to increased moisture within a building. When water vapor migrating through the slab would be undesirable, we recommend the following to reduce, but not stop, water vapor transmission upward through the slab-on-grade.

1. Install a vapor retarder membrane directly beneath the slab. Seal the vapor retarder at all seams and pipe penetrations. Vapor retarders shall conform to Class A vapor retarder in accordance with ASTM E 1745, latest edition, “Standard Specification for Plastic Water Vapor Retarders used in Contact with Soil or Granular Fill under Concrete Slabs.”

2. Concrete shall have a concrete water-cement ratio of no more than 0.50.

3. Provide inspection and testing during concrete placement to check that the proper concrete and water cement ratio are used.

4. Moist cure slabs for a minimum of 3 days or use other equivalent curing specific by the structural engineer.

The structural engineer should be consulted as to the use of a layer of clean sand or pea gravel (less than 5 percent passing the U.S. Standard No. 200 Sieve) placed on top of the vapor retarder membrane to assist in concrete curing.

6.2 CONVENTIONALLY REINFORCED MAT SLAB

The structure may, alternatively, be supported on conventionally reinforced mat foundation. We recommend the mat be designed to cantilever 6 feet at the perimeter and free span interior areas for a distance of 20 feet. The PT mat may be designed for an average allowable bearing pressure of up to 1,000 pounds per square foot (psf) for dead-plus-live loads with maximum localized bearing pressures of 1,500 psf at column or wall loads. These values may be increased by one-third when considering transient loads, such as wind or seismic. Provided the site earthwork is conducted in accordance with the recommendations of this report, a subgrade modulus of 100 psi/in can be used for structural slab design.

The foundation system should be designed to accommodate the settlement recommended in Section 6.1.1.

Vapor transmission through the mat should be reduced by implementing the recommendations in Section 6.1.2.

6.3 CONVENTIONAL FOOTINGS WITH SLAB-ON-GRADE

The proposed train depot can also be supported on continuous or isolated spread footings bearing in competent native soil or compacted fill in combination with non-expansive material supporting the slab-on-grade. Isolated footings should be structurally connected with grade-beams in at least two orthogonal horizontal directions to increase rigidity of the foundation system.

Due to the expansion potential of the near-surface soil, we recommend that interior floor slabs be supported on non-expansive fill to reduce the likelihood of slab damage from heave or shrinkage. For a conventional interior slab, we recommend a minimum 24 inches of non-expansive fill. The non-expansive fill should extend a minimum of five feet beyond the building envelope. The non-expansive fill should have a PI of 12 or less, have sufficient fines, and low corrosion potential for the foundation concrete. A sample of non-expansive fill should be provided to ENGEO a minimum of 5 days prior to delivering to the project site.

6.3.1 Footing Dimensions and Allowable Bearing Capacity

Provide minimum footing dimensions as follows in the Table 6.3.1-1 below.

TABLE 6.3.1-1: Minimum Footing Dimensions

FOOTING TYPE	*MINIMUM DEPTH (INCHES)	MINIMUM WIDTH (INCHES)
Continuous	**24**	**12**
Isolated	**24**	**24**

Minimum footing depths shown above are taken from lowest adjacent pad grade. The cold joint between the exterior footing and slab-on-grade should be located at least 4 inches above adjacent exterior grade.

Design foundations recommended above for a maximum allowable bearing pressure of 2,500 pounds per square foot (psf) for dead-plus-live loads. Increase this bearing capacity by one-third for the short-term effects of wind or seismic loading.

The maximum allowable bearing pressure is a net value; the weight of the footing may be neglected for design purposes. Footings located adjacent to utility trenches should have their bearing surfaces below an imaginary 1:1 (horizontal:vertical) plane projected upward from the bottom edge of the trench to the footing.

6.3.2 Waterstop

If a two-pour system is used for footings and slab, the cold joint between the exterior footing and slab-on-grade should be located at least 4 inches above adjacent finish exterior grade. If this is not done, then we recommend the addition of a waterstop between the two pours to reduce moisture penetration through the cold joint and migration under the slab. Use of a monolithic pour would eliminate the need for the waterstop.

6.3.3 Reinforcement

The structural engineer should design footing reinforcement to support the intended structural loads without excessive settlement. Reinforce continuous footings with top and bottom steel to provide structural continuity and to permit spanning of local irregularities. At a minimum, continuous footings should be designed to structurally span a clear distance of 5 feet.

To help resist expansive soil movement, reinforce continuous footings with at least four No. 4 steel reinforcement bars, two top and two bottom.

6.3.4 Foundation Lateral Resistance

Lateral loads may be resisted by friction along the base and by passive pressure along the sides of foundations. The passive pressure is based on an equivalent fluid pressure in pounds per cubic foot (pcf). We recommend the following allowable values for design:

- Passive Lateral Pressure: 300 pcf
- Coefficient of Friction: 0.30

The above allowable values include a factor of safety of 1.5. Increase the above values by one-third for the short-term effects of wind or seismic loading.

Passive lateral pressure should not be used for footings on or above slopes.

6.3.5 Settlement

Provided our report recommendations are followed and given the proposed construction (Section 1.3), we estimate total and differential static foundation settlements to be less than approximately ¾ and ½ inch acting over a distance of 30 feet, respectively. The foundation system should be designed to accommodate the seismic settlement recommended in Section 6.1.1.

6.3.6 Interior Concrete Floor Slabs

To reduce the effects of expansive soil on interior slabs, in addition to the non-expansive pad cap, we recommend the following:

1. Provide a minimum concrete thickness of 5 inches.

2. Reinforce slabs with No. 4 rebar on 16-inch centers, each way, placed within the middle third of the slab.

The structural engineer should provide final design thickness and additional reinforcement, if necessary, for the intended structural loads.

6.3.7 Slab Moisture Vapor Reduction

When buildings are constructed with concrete slab-on-grade, water vapor from beneath the slab will migrate through the slab and into the building. This water vapor can be reduced but not stopped. Vapor transmission can negatively affect floor coverings and lead to increased moisture within a building. When water vapor migrating through the slab would be undesirable, we recommend the following to reduce, but not stop, water vapor transmission upward through the slab-on-grade.

1. Construct a moisture retarder system directly beneath the slab on-grade that consists of the following:
   a. Vapor retarder membrane sealed at all seams and pipe penetrations and connected to all footings. Vapor retarders shall conform to Class A vapor retarder in accordance with ASTM E 1745, latest edition, “Standard Specification for Plastic Water Vapor Retarders used in Contact with Soil or Granular Fill under Concrete Slabs”. The vapor retarder should be **underlain by**
   b. 4 inches of clean crushed rock. Crushed rock should have 100 percent passing the ¾-inch sieve and less than 5 percent passing the No. 4 Sieve.

2. Use a concrete water-cement ratio for slabs-on-grade of no more than 0.50.

3. Provide inspection and testing during concrete placement to check that the proper concrete and water cement ratio are used.

4. Moist cure slabs for a minimum of 3 days or use other equivalent curing specified by the structural engineer.

The structural engineer should be consulted as to the use of a layer of clean sand or pea gravel (less than 5 percent passing the U.S. Standard No. 200 Sieve) placed on top of the vapor retarder membrane to assist in concrete curing.

6.4 DRILLED PIERS

Other improvements such as overheard canopies and lights may be supported on drilled, cast-in-place, straight-shaft friction piers.

The piers should have a minimum diameter of 12 inches and extend to a depth of at least 8 feet below the existing ground surface. Design piers for an allowable downward skin friction of 500 pounds per square foot for combined dead-plus-live loads with a one-third increase allowed for either transient wind or seismic loading. For pier load capacity computations, exclude the upper 3 feet.

Piers should be spaced a minimum of three pier diameters, center-to-center. Where closer spacing is unavoidable, the piers should be designed with a reduced skin friction of 330 psf. Resistance to uplift loads is developed in friction along the pier shafts. We recommend that an allowable uplift frictional resistance of 330 pounds per square foot be used.

Lateral loads exerted on drilled piers and may be resisted by a passive resistance based on an equivalent fluid pressure of 300 pounds per cubic foot acting against the 1.5 times individual pier diameter. The passive earth pressure should start at a depth of 12 inches or where there is 8 feet horizontal distance to daylight in sloping areas.

The bottoms of pier excavations should be dry, reasonably clean, and free of loose soil before reinforcing steel is installed and concrete is placed. We recommend that the excavation of piers be performed under our direct observation to establish that the piers are founded in suitable materials and constructed in accordance with the recommendations presented in this letter.

If caving is observed, each shaft may need to be cased. If groundwater is encountered, remove it from excavations prior to concrete placement. If groundwater cannot be removed from excavations prior to concrete placement, then we recommend that concrete be placed by tremie pipe. The concrete should be tremied to the bottom of the hole keeping the tremie pipe below the surface of the concrete to avoid entrapment of water in the concrete. As concrete is poured, water is displaced out of the hole.

In addition, the expansive soils may exert upward pressure on the base of grade beams. This force can be neglected if a 2-inch void form of degradable material is utilized at the base of the beams/panels. Under no circumstance should grade beams be cast upon dry, desiccated soil.

Pier holes should be drilled with straight shafts and special care during construction to not allow concrete to “mushroom” out at the top of the pier. If needed, a sonotube concrete form may be used. If the provided recommendations are incorporated into the construction practices, the uplift pressure on the drilled piers may be neglected.

The pier reinforcement should be designed by the Structural Engineer, but as a minimum, at least two No. 4 rebars should extend the full length of each pier. Where applicable, the pier reinforcement should be tied to the grade beam as recommended by the Structural Engineer.

While structural loads were not provided, we anticipate that total vertical settlement for the recommended pier foundation should not exceed approximately ½ inch.

6.5 CBC PARAMETERS

It is our understanding that structures will be designed under the upcoming 2019 California Building Code (CBC). Based on the subsurface conditions encountered in the borings, we characterized the site as Site Class D in accordance with the 2019 CBC. We provide the 2019 CBC seismic design parameters in Table 6.5-1 below, which include design spectral response acceleration parameters based on the mapped Risk-Targeted Maximum Considered Earthquake (MCE_R) spectral response acceleration parameters.

TABLE 6.5-1: 2019 CBC Seismic Design Parameters

PARAMETER	VALUE
Site Class	D
Mapped MCE_R Spectral Response Acceleration at Short Periods, S_S (g)	2.291
Mapped MCE_R Spectral Response Acceleration at 1-second Period, S_1 (g)	0.808
Site Coefficient, F_A	1.0
Site Coefficient, F_V	Null*
MCE_R Spectral Response Acceleration at Short Periods, S_{MS} (g)	2.291
MCE_R Spectral Response Acceleration at 1-second Period, S_{M1} (g)	Null*
Design Spectral Response Acceleration at Short Periods, S_{DS} (g)	1.527
Design Spectral Response Acceleration at 1-second Period, S_{D1} (g)	Null*
MCE_G Peak Ground Acceleration adjusted for Site Class effects, PGA_M (g)	1.11

*Requires site-specific ground motion hazard analysis per ASCE 7-16 Section 11.4.8

Considering the proposed single-level train depot building, we estimate the fundamental periods of the proposed structure to be less than 1.5T_s (where T_s is 0.60 seconds for this project). Therefore, the structural engineer may consider exception of Section 11.4.8 of ASCE 7-16 as follows:

> "A ground motion hazard analysis is not required for structures… where, structures on Site Class D sites with S_1 greater than or equal to 0.2, provided the value of the seismic response coefficient C_s is determined by Eq. (12.8-2) of ASCE 7-16 for values of $T \leq 1.5T_S$ and taken as equal to 1.5 times the value computed in accordance with either Eq. (12.8-3) of ASCE 7-16 for $1.5T_s < T \leq T_L$. or Eq. (12.8-4) of ASCE 7-16 for T> T_L.""

If the noted exception is not used, a ground motion hazard analysis should be performed and can be provided in a separate letter.

7.0 EXTERIOR FLATWORK

Exterior flatwork includes items such as concrete sidewalks, steps, and outdoor courtyards exposed to foot traffic only. Provide a minimum section of 4 inches of concrete over 4 inches of aggregate base. Thicken flatwork edges to at least 8 inches to help control moisture variations in the subgrade and place rebar within the middle third of the slab to help control the width and offset of cracks. Reinforcement consisting of No. 3 bars spaced 18 inches on-center each way can be placed to help reduce cracks. The turndown may be omitted if the thickness of the flatwork is increased to 6 inches. As is common with concrete construction, minor cracking should be expected. Construct control and construction joints in accordance with current Portland Cement Association Guidelines.

8.0 PAVEMENT DESIGN

8.1 FLEXIBLE PAVEMENTS

We obtained three near-surface samples for Resistance Value (R-Value) testing. The tests resulted in R-Values of 8, 7, and less than 5. The following pavement sections have been determined for Traffic Indices of 4.5 through 7, an assumed R-Value of 5, and in accordance to the design methods contained in chapter 630 of Caltrans Highway Design Manual.

We have also provided an alternative section based on the Caltrans Subgrade Enhancement Geosynthetic Design and Construction Guide (latest revision September 21, 2013). Based on this guideline, the subgrade enhancement should consist of a Class B1 Woven Geotextile. The geotextile should be placed between the subgrade and Class 2 aggregate base layer. The requirements of Class B1 Woven Geotextile are included in the following Table 8.1-1.

TABLE 8.1-1: Caltrans Class B1 Woven Geotextile Requirements

Elongation at break, % ASTM D 4632	Grab tensile strength (min), lb ASTM D 4632	Wide width tensile strength (min) at 5% strain, lb/ft ASTM D 4595	Wide width tensile strength (min) at ultimate strain, lb/ft ASTM D 4595	Tear strength (min), lb ASTM D 4533	Puncture strength (min), lb ASTM D 6241	Permittivity (min), Sec^{-1} ASTM D 4491	Apparent opening size (max), inch ASTM D 4751	Ultraviolet stability (retained strength after 500 hrs exposure) (min), % ASTM D 4355
<50	--	2,000	4,800	--	620	0.20	0.024	70

When using the Subgrade Enhancement Geotextile (SEG_T), the Caltrans Subgrade Enhancement Geosynthetic Design and Construction Guide allows the design R-value of the subgrade soil to be 20. With the SEG_T, the thickness of the hot mix asphalt remains the same but the thickness of Class 2 aggregate base is reduced.

TABLE 8.1-2: Pavement Sections

TRAFFIC INDEX (TI)	R-VALUE OF 5		
	HMA (INCHES)	AB (INCHES)	AB (INCHES) With SEG_T
4.5	2½	10	7
5.0	2¾	11	8
5.5	3¾	11	8
6.0	3¾	13	9
6.5	3¾	14	11
7.0	4	16	12

*Notes: HMA – Hot Mix Asphalt
AB – Caltrans Class 2 aggregate base (R-Value of 78 or greater)
Per City of Goleta General Street Specifications: when the traffic index is less than 5.5, the minimum thickness of HMA shall be 0.20' (~2½"). When the traffic index is 5.5 or greater, the minimum thickness of HMA shall be 0.30' (~3¾").

The Traffic Indices and minimum pavement section(s) should be confirmed by the Civil Engineer and the City of Goleta.

8.1.1 Pavement Construction

Pavement construction and all materials should conform to the specifications and requirements of the latest edition of the Standard Specifications by the Division of Highways, Department of Public Works, State of California, and City of Goleta requirements and the following minimum requirements.

- All pavement subgrades should be scarified to a depth of 12 inches below finished subgrade elevation. The subgrade soil should be moisture conditioned to at least 2 percentage points above optimum and compacted to at least 95 percent relative compaction in accordance with city requirements.
- Subgrade soil should be in a stable, non-pumping condition at the time aggregate base materials are placed and compacted.
- Adequate provisions must be made such that the subgrade soils and aggregate base materials are not allowed to become saturated.
- Aggregate base materials should meet current Caltrans specifications for Class 2 aggregate base and should be compacted to at least 95 percent of maximum dry density.
- Asphalt paving materials should meet current Caltrans specifications for asphalt concrete.

8.2 RIGID PAVEMENTS

Use concrete pavement sections to resist heavy loads and turning forces in areas such as fire lanes or trash enclosures. Final design of rigid pavement sections, and accompanying reinforcement, should be performed based on estimated traffic loads and frequencies. We recommend the following minimum design sections for rigid pavements using ACI 330R-08 Design Guide for Concrete Parking Lots:

TABLE 8.2-1: Rigid Pavement Sections

AVERAGE DAILY TRUCK TRAFFIC (ADTT*)	R-VALUE OF 5	
	CONCRETE (IN)	AB (IN)
10	6	6
25	6½	9
100	7½	9
300	7½	12
700	8	12

*Notes: ADTT – average daily truck traffic. Trucks are defined as vehicles with at least six wheels; excludes panel trucks, pickup trucks, and other four-wheel vehicles.
AB – Caltrans Class 2 aggregate base (R-Value of 78 or greater)

TABLE 8.2-2: Spacing Between Joints

PAVEMENT THICKNESS (IN)	MAXIMUM SPACING (FT)
4, 4½	10
5, 5½	12½
6 or greater	15

- Jointed Plane Concrete Pavement (JPCP) should have a minimum 28-day compressive strength (f'c) of 4,000 psi for a 20-year design life.

- Design assumes there is edge support provided by a curb or paving.

8.3 SUBGRADE AND AGGREGATE BASE COMPACTION

Compact finish subgrade and aggregate base in accordance with Section 5.6. Aggregate Base should meet the requirements for ¾-inch maximum Class 2 AB in accordance with Section 26 of the latest Caltrans Standard Specifications.

8.4 CUT-OFF CURBS

Saturated pavement subgrade or aggregate base can cause premature failure or increased maintenance of asphalt concrete pavements. This condition often occurs where landscape areas directly abut and drain toward pavements. If desired to install pavement cutoff barriers, they should be considered where pavement areas lie downslope of any landscape areas that are to be sprinklered or irrigated, and should extend to a depth of at least 4 inches below the aggregate base layer. Cutoff barriers may consist of deepened concrete curbs or deep-root moisture barriers.

If reduced pavement life and greater than normal pavement maintenance are acceptable to the owner, then the cutoff barrier may be eliminated.

9.0 LIMITATIONS AND UNIFORMITY OF CONDITIONS

This report presents geotechnical recommendations for design of the improvements discussed in Section 1.3 for the Goleta Train Depot project. If changes occur in the nature or design of the project, we should be allowed to review this report and provide additional recommendations, if any. It is the responsibility of the client to transmit the information and recommendations of this report to the appropriate organizations or people involved in design of the project, including but not limited to owners, architects, engineers, and designers. The conclusions and recommendations contained in this report are solely professional opinions and are valid for a period of no more than 2 years from the date of report issuance.

We strived to perform our professional services in accordance with generally accepted geotechnical engineering principles and practices currently employed in the area; no warranty is expressed or implied. There are risks of earth movement and property damages inherent in building on or with earth materials. We are unable to eliminate all risks; therefore, we are unable to guarantee or warrant the results of our services.

This report is based upon field and other conditions discovered at the time of report preparation. We developed this report with limited subsurface exploration data. We assumed that our subsurface exploration data are representative of the actual subsurface conditions across the site. Considering possible underground variability of soil and groundwater, additional costs may be required to complete the project. We recommend that the owner establish a contingency fund to cover such costs. If unexpected conditions are encountered, ENGEO must be notified immediately to review these conditions and provide additional and/or modified recommendations, as necessary.

Our services did not include excavation sloping or shoring, soil volume change factors, flood potential, or a geohazard exploration. In addition, our geotechnical exploration did not include work to determine the existence of possible hazardous materials. If any hazardous materials are encountered during construction, the proper regulatory officials must be notified immediately.

This document must not be subject to unauthorized reuse, that is, reusing without written authorization of ENGEO. Such authorization is essential because it requires ENGEO to evaluate the document's applicability given new circumstances, not the least of which is passage of time.

Actual field or other conditions will necessitate clarifications, adjustments, modifications or other changes to ENGEO's documents. Therefore, ENGEO must be engaged to prepare the necessary clarifications, adjustments, modifications or other changes before construction activities commence or further activity proceeds. If ENGEO's scope of services does not include onsite construction observation, or if other persons or entities are retained to provide such services, ENGEO cannot be held responsible for any or all claims arising from or resulting from the performance of such services by other persons or entities, and from any or all claims arising from or resulting from clarifications, adjustments, modifications, discrepancies or other changes necessary to reflect changed field or other conditions.

We determined the lines designating the interface between layers on the exploration logs using visual observations. The transition between the materials may be abrupt or gradual. The exploration logs contain information concerning samples recovered, indications of the presence of various materials such as clay, sand, silt, rock, existing fill, etc., and observations of groundwater encountered. The field logs also contain our interpretation of the subsurface conditions between sample locations. Therefore, the logs contain both factual and interpretative information. Our recommendations are based on the contents of the final logs, which represent our interpretation of the field logs.

SELECTED REFERENCES

American Concrete Institute, 2014, Building Code Requirements for Structural Concrete (ACI 318-14) and Commentary (ACI 318R-14).

American Concrete Institute, 2008, Guide for the Design and Construction of Concrete Parking Lots (ACI 318R-08).

American Society of Civil Engineers. Minimum Design Loads for Buildings and other Structures ASCE 7-16. Reston: American Society of Civil Engineers. 2016.

Bryant, W. and Hart, E., 2007, Special Publication 42, "Fault-Rupture Hazard Zones in California", Interim Revision 2007, California Department of Conservation.

California Building Standards Commission, 2019 California Building Code, Volumes 1 and 2, Sacramento, California.

California Department of Transportation, 2017, Highway Design Manual.

California Geologic Survey, 2008, Special Publication 117A, Guidelines for Evaluating and Mitigating Seismic Hazards in California.

Churchill, R. (2008) Radon Potential in San Luis Obispo County, California Geological Survey, Department of Conservation, Special Report 208.

Churchill, R. (1997) Radon Mapping Santa Barbara and Ventura Counties, Department of Conservation, Division of Mines and Geology.

County of Los Angeles (2017), Department of Public Works, Geotechnical and Materials Engineering Division, Guidelines for Geotechnical Investigation and Reporting Low Impact Development Stormwater Infiltration. GS200.2. June 30.

Field, E.H., 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, and Zeng, Y., 2013, Uniform California earthquake rupture forecast, version 3 (UCERF3)—The time-independent model: U.S. Geological Survey Open-File Report 2013–1165, 97 p., California Geological Survey Special Report 228, and Southern California Earthquake Center Publication 1792, http://pubs.usgs.gov/of/2013/1165/

Idriss, I. M., Boulanger, R. W., (2008) Soil Liquefaction During Earthquakes, Earthquake Engineering Research Institute.

Ishihara, K., 1985, Stability of Natural Deposits During Earthquakes, Proc 11th International Conference on Soil Mechanics and Foundation Engineering, Vol 1, A. A. Balkema, Rotterdam, The Netherlands, 321-376.

SELECTED REFERENCES (Continued)

Ishihara, K. and Yoshimine, M. (1992). "Evaluation of Settlements in Sand Deposits Following Liquefaction during Earthquakes." Soils and Foundations, Vol. 32, No. 1, March pp. 173-188.

Minor, Scott A., Kellogg, Karl S., Stanley, Richard G., and Brandt, Theodore R., (2007) Geologic Map of the Goleta 7.5' Quadrangle, Santa Barbara County, California. Open File Report 2007-1403.

Post-Tensioning Institute, 2012, Standard Requirements for Design and Analysis of Shallow Post-Tensioned Concrete Foundations on Expansive Soils.

Post-Tensioning Institute, 2004, Design of Post-Tensioned Slabs-on-Ground, Third Edition.

Seed, H. B., Woodward, R. J., Jr and Lundgren, R., 1962, Prediction of swelling potential for compacted clays: J. ASCE, Soil Mechanics and Foundation Division, Vol. 88, No. SM-3, Part 1, pp 53-87.

Structural Engineers Association of California (SEAOC) (1996). Recommended Lateral Force Requirements and Tentative Commentary.

Southern California Earthquake Center (1999), Recommended Procedures for Implementation of DMG Special Publication 117 Guidelines for Analyzing and Mitigating Liquefaction in California.

Tokimatsu, K. and H. B. Seed, 1987, Evaluation of Settlements in Sands due to Earthquake Shaking, ASCE Journal of Geotechnical Engineering, Volume 113, No. 8.

United States Geologic Survey. 2008. Quaternary Fault and Fold Database of the United States. National Hazard Maps.

Youd, T. L. and C. T. Garris, 1995, Liquefaction induced Ground-Surface Description: Journal of Geotechnical Engineering, Vol. 121, No. 11, pp. 805 – 809.

Youd, T. L. and I. M. Idriss, 2001, Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER/NSF Workshop on Evaluation of Liquefaction Resistance of Soils.

FIGURES

N

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FEET

BASEMAP SOURCE: ESRI MAPPING SERVICE

ENGEO Expect Excellence	VICINITY MAP GOLETA TRAIN STATION GOLETA, CALIFORNIA	PROJECT NO.: 16370.000.000 SCALE: AS SHOWN DRAWN BY: QRL CHECKED BY: RHB	FIGURE NO. 1

PATH: G:\DRAFTING\PROJECTS_16000 TO 17999\16370\16370000000\GEX\GOLETA TRAIN STATION.APRX
USER: QLIANG

ORIGINAL FIGURE PRINTED IN COLOR

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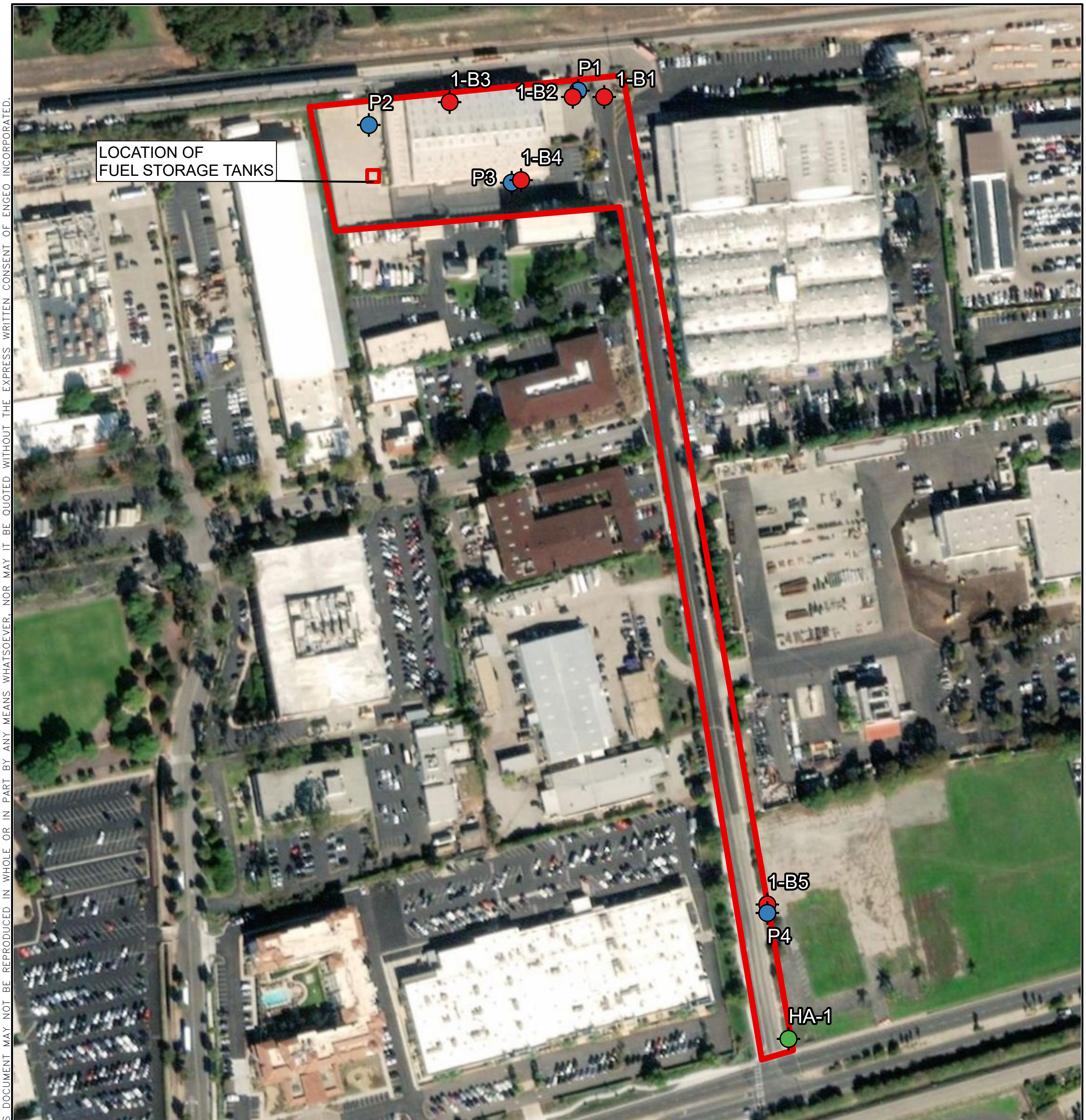

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FEET

N

EXPLANATION

ALL LOCATIONS ARE APPROXIMATE

PROJECT SITE

1-B5 BORING (ENGEO, 2019)

P4 PERCOLATION TEST (ENGEO, 2019)

HA-1 HAND AUGER (ENGEO, 2019)

BASEMAP SOURCE: ESRI MAPPING SERVICE

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SITE PLAN GOLETA TRAIN STATION GOLETA, CALIFORNIA	PROJECT NO. : 16370.000.000		FIGURE NO. 2
	SCALE: AS SHOWN		
	DRAWN BY: QRL	CHECKED BY: RHB	

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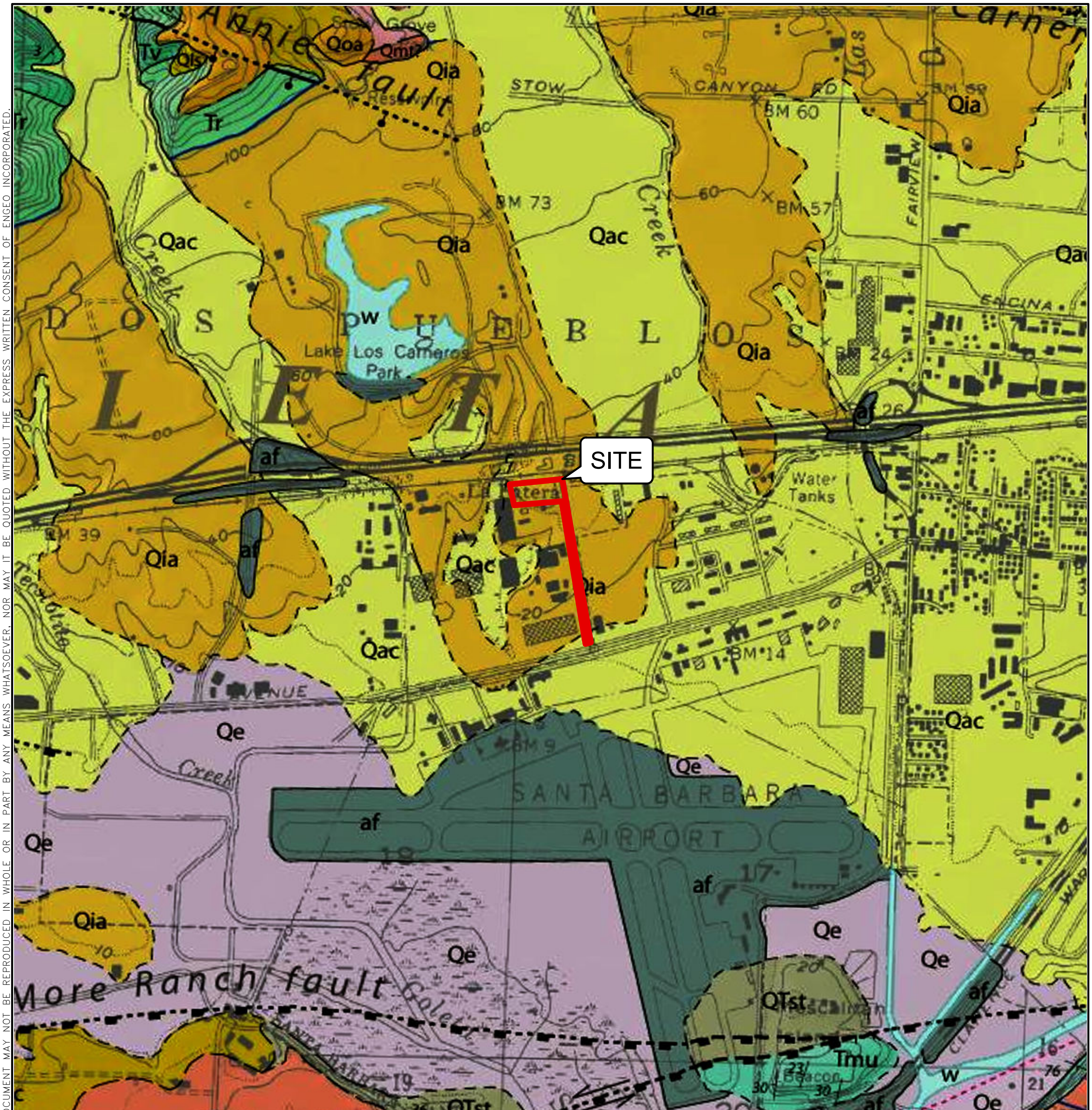

EXPLANATION

ALL LOCATIONS ARE APPROXIMATE

af	ARTIFICIAL FILL
Qe	ESTUARINE DEPOSITS (HOLOCENE)
Qac	ALLUVIUM AND COLLUVIUM (HOLOCENE AND UPPER PLEISTOCENE)
Qls	LANDSLIDE DEPOSITS (HOLOCENE TO MIDDLE PLEISTOCENE)
Qia	INTERMEDIATE ALLUVIAL DEPOSITS (UPPER PLEISTOCENE)
Qmt	MARINE-TERRACE DEPOSITS (UPPER PLEISTOCENE)
Qtst	SILTSTONE UNIT (LOWER PLEISTOCENE AND UPPER PLIOCENE?)
Tmu	UPPER SILICEOUS UNIT (UPPER MIOCENE)
Tr	RINCON SHALE (LOWER MIOCENE)

N

0 750 1,500

FEET

BASEMAP SOURCE: MINOR ET. AL. 2007

ENGEO
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REGIONAL GEOLOGIC MAP

GOLETA TRAIN STATION

GOLETA, CALIFORNIA

PROJECT NO. : 16370.000.000	FIGURE NO. 3
SCALE: AS SHOWN	
DRAWN BY: QRL	CHECKED BY: RHB

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USER: QLIANG

ORIGINAL FIGURE PRINTED IN COLOR

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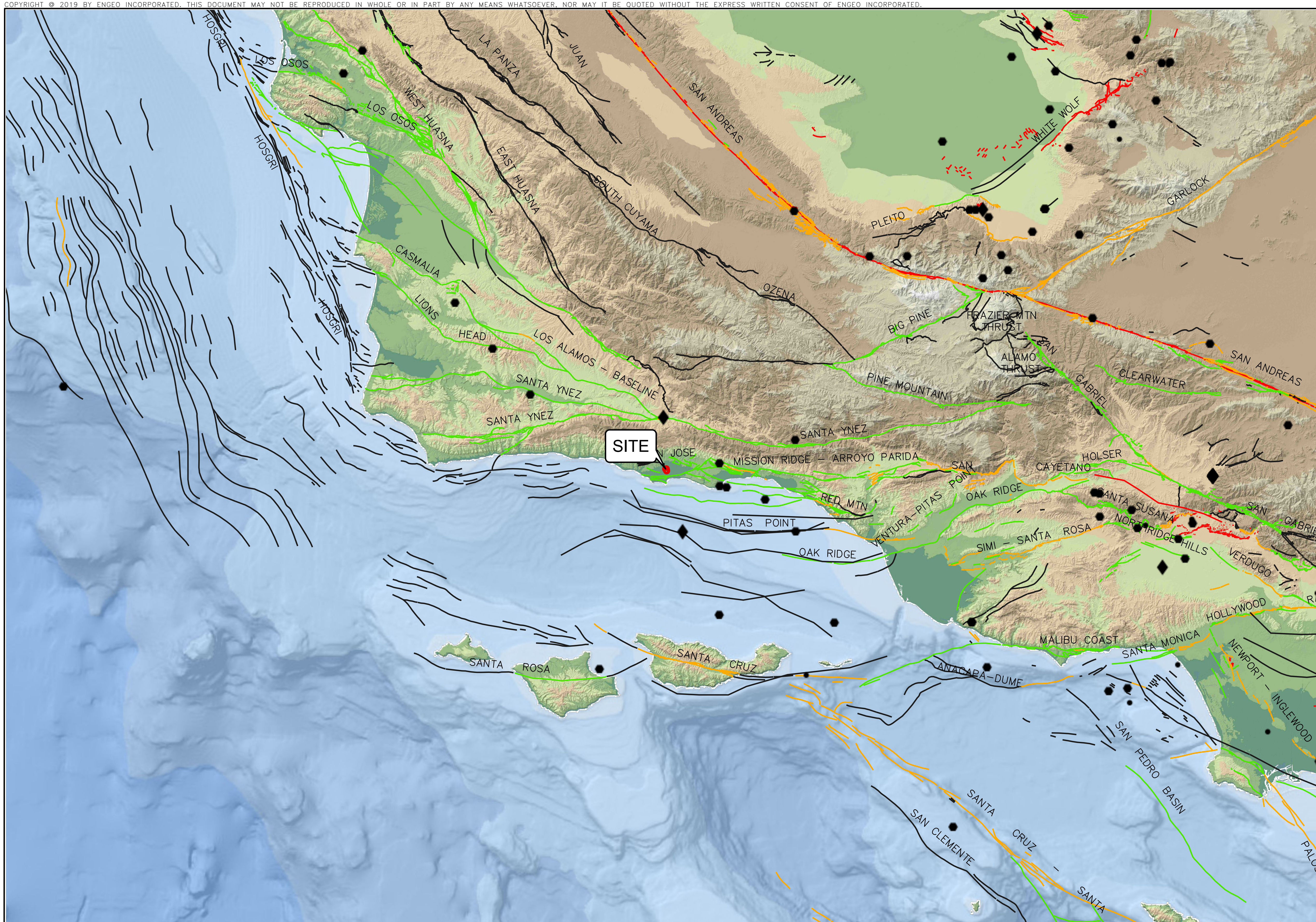

N

0 7.5 15

Miles

EXPLANATION

ALL LOCATIONS ARE APPROXIMATE

EARTHQUAKE

- MAGNITUDE 7+
- MAGNITUDE 6-7
- MAGNITUDE 5-6

USGS QUATERNARY FAULTS

- HISTORICAL
- LATEST QUATERNARY
- LATE QUATERNARY
- UNDIFFERENTIATED QUATERNARY
- HISTORIC BLIND THRUST FAULT ZONE

BASE MAP SOURCE

ESRI, GARMIN, GEBCO, NOAA NGDC, AND OTHER CONTRIBUTORS
COLOR HILLSHADE IMAGE BASED ON THE NATIONAL ELEVATION DATA SET (NED) AT 30 METER RESOLUTION
U.S.G.S. QUATERNARY FAULT DATABASE, 2018
U.S.G.S. HISTORIC EARTHQUAKE DATABASE (1800-PRESENT)

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REGIONAL FAULTING AND SEISMICITY
GOLETA TRAIN STATION
GOLETA, CALIFORNIA

PROJECT NO.: 16370.000.000		FIGURE NO.
SCALE: AS SHOWN		4
DRAWN BY: QRL	CHECKED BY: RHB	

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USER: QLIANG

ORIGINAL FIGURE PRINTED IN COLOR

APPENDIX A

BORING LOG KEY
EXPLORATION LOGS

KEY TO BORING LOGS

MAJOR TYPES			DESCRIPTION
COARSE-GRAINED SOILS MORE THAN HALF OF MAT'L LARGER THAN #200 SIEVE	GRAVELS MORE THAN HALF COARSE FRACTION IS LARGER THAN NO. 4 SIEVE SIZE	CLEAN GRAVELS WITH LESS THAN 5% FINES	GW - Well graded gravels or gravel-sand mixtures GP - Poorly graded gravels or gravel-sand mixtures
		GRAVELS WITH OVER 12 % FINES	GM - Silty gravels, gravel-sand and silt mixtures GC - Clayey gravels, gravel-sand and clay mixtures
	SANDS MORE THAN HALF COARSE FRACTION IS SMALLER THAN NO. 4 SIEVE SIZE	CLEAN SANDS WITH LESS THAN 5% FINES	SW - Well graded sands, or gravelly sand mixtures SP - Poorly graded sands or gravelly sand mixtures
		SANDS WITH OVER 12 % FINES	SM - Silty sand, sand-silt mixtures SC - Clayey sand, sand-clay mixtures
FINE-GRAINED SOILS MORE THAN HALF OF MAT'L SMALLER THAN #200 SIEVE	SILTS AND CLAYS LIQUID LIMIT 50 % OR LESS		ML - Inorganic silt with low to medium plasticity CL - Inorganic clay with low to medium plasticity OL - Low plasticity organic silts and clays
	SILTS AND CLAYS LIQUID LIMIT GREATER THAN 50 %		MH - Elastic silt with high plasticity CH - Fat clay with high plasticity OH - Highly plastic organic silts and clays
	HIGHLY ORGANIC SOILS		PT - Peat and other highly organic soils

For fine-grained soils with 15 to 29% retained on the #200 sieve, the words "with sand" or "with gravel" (whichever is predominant) are added to the group name.

For fine-grained soil with >30% retained on the #200 sieve, the words "sandy" or "gravelly" (whichever is predominant) are added to the group name.

GRAIN SIZES

U.S. STANDARD SERIES SIEVE SIZE: 200, 40, 10, 4
CLEAR SQUARE SIEVE OPENINGS: 3/4 ", 3", 12"

SILTS AND CLAYS	SAND			GRAVEL		COBBLES	BOULDERS
	FINE	MEDIUM	COARSE	FINE	COARSE		

RELATIVE DENSITY

SANDS AND GRAVELS	BLOWS/FOOT (S.P.T.)
VERY LOOSE	0-4
LOOSE	4-10
MEDIUM DENSE	10-30
DENSE	30-50
VERY DENSE	OVER 50

CONSISTENCY

SILTS AND CLAYS	STRENGTH*
VERY SOFT	0-1/4
SOFT	1/4-1/2
MEDIUM STIFF	1/2-1
STIFF	1-2
VERY STIFF	2-4
HARD	OVER 4

MOISTURE CONDITION

DRY	Dusty, dry to touch
MOIST	Damp but no visible water
WET	Visible freewater

SAMPLER SYMBOLS

- Modified California (3" O.D.) sampler
- California (2.5" O.D.) sampler
- S.P.T. - Split spoon sampler
- Shelby Tube
- Dames and Moore Piston
- Continuous Core
- Bag Samples
- Grab Samples
- NR No Recovery

LINE TYPES

- Solid - Layer Break
- Dashed - Gradational or approximate layer break

GROUND-WATER SYMBOLS

- Groundwater level during drilling
- Stabilized groundwater level

(S.P.T.) Number of blows of 140 lb. hammer falling 30" to drive a 2-inch O.D. (1-3/8 inch I.D.) sampler

* Unconfined compressive strength in tons/sq. ft., asterisk on log means determined by pocket penetrometer

ENGEO
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LOG OF BORING 1-B1

ENGEO — Expect Excellence

LATITUDE: 34.437575 LONGITUDE: -119.842536

Geotechnical Exploration
Goleta Train Depot
Goleta, California
16370.000.000

DATE DRILLED: 8/12/2019
HOLE DEPTH: 11.5 ft.
HOLE DIAMETER: 8.0 in.
SURF ELEV (WGS 84): 31 ft.

LOGGED / REVIEWED BY: R. Hildebrant / RHB
DRILLING CONTRACTOR: 2R Drilling
DRILLING METHOD: Hollow Stem Auger
HAMMER TYPE: 140 lb. Auto Trip

Depth in Feet	Elevation in Feet	Sample Type	DESCRIPTION	Log Symbol	Water Level	Blow Count/Foot	Atterberg Limits: Liquid Limit	Atterberg Limits: Plastic Limit	Atterberg Limits: Plasticity Index	Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Shear Strength (psf) *field approximation	Unconfined Strength (tsf) *field approximation	Strength Test Type
			4" ASPHALT PAVEMENT												
			6" AGGREGATE BASE												
	30		SANDY LEAN CLAY TO CLAYEY SAND (CL-SC), dark reddish brown, stiff, moist, fine- to medium-grained sand, R-Value = 7				28	14	14	47					
														1.5*	PP
						6									
5	25					6									
			CLAYEY SAND (SC), dark reddish brown, dense, moist, fine- to medium-grained sand, ~20-25% fines												
10	20					34									

Bottom of boring at approximately 11½ feet below the ground surface
No groundwater encountered during drilling

LOG - GEOTECHNICAL_SU+QU W/ ELEV GOLETA TRAIN STATION REV.GPJ ENGEO INC.GDT 9/19/19

LOG OF BORING 1-B2

ENGEO Expect Excellence

LATITUDE: 34.437549 LONGITUDE: -119.84273

Geotechnical Exploration
Goleta Train Depot
Goleta, California
16370.000.000

DATE DRILLED: 8/12/2019
HOLE DEPTH: 51.5 ft.
HOLE DIAMETER: 8.0 in.
SURF ELEV (WGS 84): 30 ft.

LOGGED / REVIEWED BY: R. Hildebrant / RHB
DRILLING CONTRACTOR: 2R Drilling
DRILLING METHOD: Hollow Stem Auger
HAMMER TYPE: 140 lb. Auto Trip

Depth in Feet	Elevation in Feet	DESCRIPTION	Blow Count/Foot	Liquid Limit	Plastic Limit	Plasticity Index	Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Shear Strength (psf) *field approximation	Unconfined Strength (tsf) *field approximation	Strength Test Type
		4" ASPHALT PAVEMENT										
		SANDY LEAN CLAY (CL), dark reddish brown, hard, moist, manganese nodules, fine- to medium-grained sand		37	16	21	62					
			40					13.9	120	5146	5.15	UC
5	25											
			31	22	14	8	63	16.4	115.7	3073	3.07	UC
			33								4.25*	PP
		CLAYEY SAND (SC), dark reddish brown, medium dense, moist										
10	20											
			47				39	15.6	111.1			
15	15											
			40									
		SILTY SAND (SM), brown, medium dense to dense, moist, fine- to medium-grained sand										
			23				14	6.3				
20	10	Yellowish brown										
			28 50/5"				13					
25	5											

LOG - GEOTECHNICAL_SU+QU W/ ELEV GOLETA TRAIN STATION REV.GPJ ENGEO INC.GDT 9/19/19

ENGEO
Expect Excellence

LOG OF BORING 1-B2

LATITUDE: 34.437549 LONGITUDE: -119.84273

Geotechnical Exploration
Goleta Train Depot
Goleta, California
16370.000.000

DATE DRILLED: 8/12/2019
HOLE DEPTH: 51.5 ft.
HOLE DIAMETER: 8.0 in.
SURF ELEV (WGS 84): 30 ft.

LOGGED / REVIEWED BY: R. Hildebrant / RHB
DRILLING CONTRACTOR: 2R Drilling
DRILLING METHOD: Hollow Stem Auger
HAMMER TYPE: 140 lb. Auto Trip

Depth in Feet	Elevation in Feet	Sample Type	DESCRIPTION	Log Symbol	Water Level	Blow Count/Foot	Atterberg Limits: Liquid Limit	Atterberg Limits: Plastic Limit	Atterberg Limits: Plasticity Index	Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Shear Strength (psf) *field approximation	Unconfined Strength (tsf) *field approximation	Strength Test Type
			medium- to coarse-grained sand			72				25	6.3				
30	0		light yellowish brown, fine- to medium-grained sand			50/5"									
35	-5					57				10	8.1				
40	-10		LEAN CLAY (CL), grayish green, hard, moist, marine shells, iron staining			16					25.5			3.25*	PP
45	-15					78								4.5*	PP
50	-20														

ENGEO
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LOG OF BORING 1-B2

LATITUDE: 34.437549 LONGITUDE: -119.84273

Geotechnical Exploration
Goleta Train Depot
Goleta, California
16370.000.000

DATE DRILLED: 8/12/2019
HOLE DEPTH: 51.5 ft.
HOLE DIAMETER: 8.0 in.
SURF ELEV (WGS 84): 30 ft.

LOGGED / REVIEWED BY: R. Hildebrant / RHB
DRILLING CONTRACTOR: 2R Drilling
DRILLING METHOD: Hollow Stem Auger
HAMMER TYPE: 140 lb. Auto Trip

Depth in Feet	Elevation in Feet	Sample Type	DESCRIPTION	Log Symbol	Water Level	Blow Count/Foot	Atterberg Limits: Liquid Limit	Atterberg Limits: Plastic Limit	Atterberg Limits: Plasticity Index	Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Shear Strength (psf) *field approximation	Unconfined Strength (tsf) *field approximation	Strength Test Type
						83								4.5*	PP
			Bottom of boring at approximately at 51½ ft below the ground surface No groundwater encountered during drilling												

LOG - GEOTECHNICAL_SU+QU W/ ELEV GOLETA TRAIN STATION REV.GPJ ENGEO INC.GDT 9/19/19

ENGEO Expect Excellence

LOG OF BORING 1-B3

LATITUDE: 34.437536 LONGITUDE: -119.843453

Geotechnical Exploration
Goleta Train Depot
Goleta, California
16370.000.000

DATE DRILLED: 8/13/2019
HOLE DEPTH: 38.5 ft.
HOLE DIAMETER: 8.0 in.
SURF ELEV (WGS 84): 34 ft.

LOGGED / REVIEWED BY: R. Hildebrant / RHB
DRILLING CONTRACTOR: 2R Drilling
DRILLING METHOD: Hollow Stem Auger
HAMMER TYPE: 140 lb. Auto Trip

Depth in Feet	Elevation in Feet	DESCRIPTION	Blow Count/Foot	Atterberg Limits: Liquid Limit	Plastic Limit	Plasticity Index	Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Shear Strength (psf) *field approximation	Unconfined Strength (tsf) *field approximation	Strength Test Type
		4" CONCRETE										
		LEAN CLAY (CL), dark reddish brown, hard, moist, [FILL]										
				22	14	8	58					
			22	50	19	31	80	19.8	107.8	3838	3.84	UC
	30		18									
5		SANDY LEAN CLAY (CL), dark reddish brown, very stiff to hard, moist, manganese nodules, fine- to medium-grained sand									3.5*	PP
			36	35	16	19	78	16.9	116.4	5069	5.07	UC
	25											
10			44				63				3.5*	PP
		stiff, fine- to coarse-grained sand										
	20											
15		fine-grained sand	13				81	21				
	15											
20			22				68	18.2				
	10											
25												

LOG - GEOTECHNICAL_SU+QU W/ ELEV GOLETA TRAIN STATION REV.GPJ ENGEO INC.GDT 9/19/19

ENGEO
Expect Excellence

LOG OF BORING 1-B3

LATITUDE: 34.437536 LONGITUDE: -119.843453

Geotechnical Exploration
Goleta Train Depot
Goleta, California
16370.000.000

DATE DRILLED: 8/13/2019
HOLE DEPTH: 38.5 ft.
HOLE DIAMETER: 8.0 in.
SURF ELEV (WGS 84): 34 ft.

LOGGED / REVIEWED BY: R. Hildebrant / RHB
DRILLING CONTRACTOR: 2R Drilling
DRILLING METHOD: Hollow Stem Auger
HAMMER TYPE: 140 lb. Auto Trip

Depth in Feet	Elevation in Feet	Sample Type	DESCRIPTION	Log Symbol	Water Level	Blow Count/Foot	Atterberg Limits: Liquid Limit	Atterberg Limits: Plastic Limit	Atterberg Limits: Plasticity Index	Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Shear Strength (psf) *field approximation	Unconfined Strength (tsf) *field approximation	Strength Test Type
						30				28	10				
			SILTY SAND (SM), brown to pale olive, medium dense, moist, fine- to medium-grained sand												
30	5		wet sample			24				26	25				
			SILT (ML), pale olive, very stiff, moist			14	27	NP	NP	74	26.9			2.25*	PP
35	0		LEAN CLAY (CL), grayish green, very stiff to hard, moist, marine shells			12	38	22	16		27			2.5*	PP
						27 50/5"								4.5*	PP

Bottom of boring at approximately 38½ feet below ground surface
Groundwater encountered at 30 feet during drilling, no groundwater measured at the end of drilling

LOG - GEOTECHNICAL_SU+QU W/ ELEV GOLETA TRAIN STATION REV.GPJ ENGEO INC.GDT 9/19/19

ENGEO
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LOG OF BORING 1-B4

LATITUDE: 34.437154 LONGITUDE: -119.843038

Geotechnical Exploration
Goleta Train Depot
Goleta, California
16370.000.000

DATE DRILLED: 8/12/2019
HOLE DEPTH: 31.5 ft.
HOLE DIAMETER: 8.0 in.
SURF ELEV (WGS 84): 32 ft.

LOGGED / REVIEWED BY: R. Hildebrant / RHB
DRILLING CONTRACTOR: 2R Drilling
DRILLING METHOD: Hollow Stem Auger
HAMMER TYPE: 140 lb. Auto Trip

Depth in Feet	Elevation in Feet	DESCRIPTION	Blow Count/Foot	Atterberg Limits: Liquid Limit	Plastic Limit	Plasticity Index	Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Shear Strength (psf) *field approximation	Unconfined Strength (tsf) *field approximation	Strength Test Type
		5" ASPHALT PAVEMENT										
	30	SANDY SILT (ML), dark reddish brown, hard to very stiff, moist, manganese nodules, fine- to coarse-grained sand, R-Value = <5										
		SANDY LEAN CLAY (ML), dark reddish brown, hard to very stiff, moist, manganese nodules, fine- to coarse-grained sand	24	18	16	2	56	14.2	119.6	1215	1.22	UC
5			15								4.5*	PP
	25	fine-to medium-grained sand	35	34	16	18		15.1	116.6	4190	4.19	UC
10		LEAN CLAY (CL), dark reddish brown, very stiff, moist	14								3.25*	PP
	20											
15			19					19.9	109		3.25*	PP
	15	CLAYEY SAND (SC), brown, medium dense, moist, ~30% fines										
20		SILTY SAND (SM), brown, medium dense, moist, fine- to medium-grained sand	17				33	14.8				
	10											
25		POORLY GRADED SAND TO SILTY SAND (SP-SM), brown, dense, moist, fine- to medium-grained sand										

LOG - GEOTECHNICAL_SU+QU W/ ELEV GOLETA TRAIN STATION REV.GPJ ENGEO INC.GDT 9/19/19

LOG OF BORING 1-B4

ENGEO
Expect Excellence

LATITUDE: 34.437154 LONGITUDE: -119.843038

Geotechnical Exploration
Goleta Train Depot
Goleta, California
16370.000.000

DATE DRILLED: 8/12/2019
HOLE DEPTH: 31.5 ft.
HOLE DIAMETER: 8.0 in.
SURF ELEV (WGS 84): 32 ft.

LOGGED / REVIEWED BY: R. Hildebrant / RHB
DRILLING CONTRACTOR: 2R Drilling
DRILLING METHOD: Hollow Stem Auger
HAMMER TYPE: 140 lb. Auto Trip

Depth in Feet	Elevation in Feet	Sample Type	DESCRIPTION	Log Symbol	Water Level	Blow Count/Foot	Atterberg Limits: Liquid Limit	Atterberg Limits: Plastic Limit	Atterberg Limits: Plasticity Index	Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Shear Strength (psf) *field approximation	Unconfined Strength (tsf) *field approximation	Strength Test Type
			POORLY GRADED SAND TO SILTY SAND (SP-SM), brown, dense, moist, fine- to medium-grained sand			34				9					
	5														
30						44				6					
			Bottom of boring at approximately 31½ feet below the ground surface No groundwater encountered during drilling												

ENGEO Expect Excellence

LOG OF BORING 1-B5

LATITUDE: 34.433692 LONGITUDE: -119.84146

Geotechnical Exploration
Goleta Train Depot
Goleta, California
16370.000.000

DATE DRILLED: 8/12/2019
HOLE DEPTH: 21.5 ft.
HOLE DIAMETER: 8.0 in.
SURF ELEV (WGS 84): 11 ft.

LOGGED / REVIEWED BY: R. Hildebrant / RHB
DRILLING CONTRACTOR: 2R Drilling
DRILLING METHOD: Hollow Stem Auger
HAMMER TYPE: 140 lb. Auto Trip

Depth in Feet	Elevation in Feet	Sample Type	DESCRIPTION	Log Symbol	Water Level	Blow Count/Foot	Atterberg Limits: Liquid Limit	Atterberg Limits: Plastic Limit	Atterberg Limits: Plasticity Index	Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Shear Strength (psf) *field approximation	Unconfined Strength (tsf) *field approximation	Strength Test Type
			3" ASPHALT PAVEMENT												
	10		SANDY LEAN CLAY (CL), dark reddish brown, hard, moist, R-Value = 8				30	14	16	49					
						9								>4.5*	PP
5	5		more clayey			26								>4.5*	PP
			more sandy, manganese nodules			13								4.5*	PP
10	0					17								4.5*	PP
			SILTY SAND (SM), pale yellow, dense, moist, fine- to medium-grained sand, ~15% fines												
15	-5					49									
			CLAYEY SAND (SC), brown, medium dense, wet, fine- to coarse-grained sand, ~20-25% fines		▽										
20	-10					36									

Bottom of boring at approximately 21½ feet below the ground surface
Groundwater measured at 20 feet at the end of drilling

LOG - GEOTECHNICAL_SU+QU W/ ELEV GOLETA TRAIN STATION REV.GPJ ENGEO INC.GDT 9/19/19

LOG OF BORING HA-1

ENGEO
Expect Excellence

LATITUDE: 34.433018 LONGITUDE: -119.841326

Geotechnical Exploration
Goleta Train Depot
Goleta, California
16370.000.000

DATE DRILLED: 7/17/2019
HOLE DEPTH: 13.5 ft.
HOLE DIAMETER: 4.0 in.
SURF ELEV (WGS 84): 9 ft.

LOGGED / REVIEWED BY: R. Hildebrant / RHB
DRILLING CONTRACTOR: N/A
DRILLING METHOD: Hand Auger
HAMMER TYPE: N/A

Depth in Feet	Elevation in Feet	Sample Type	DESCRIPTION	Log Symbol	Water Level	Blow Count/Foot	Atterberg Limits: Liquid Limit	Atterberg Limits: Plastic Limit	Atterberg Limits: Plasticity Index	Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Shear Strength (psf) *field approximation	Unconfined Strength (tsf) *field approximation	Strength Test Type
			CLAYEY SAND TO SANDY CLAY (SC-CL), dark brown, fine- to medium-grained sand, fine- to medium gravel, ~40% fines [FILL]												
5	5		LEAN CLAY WITH SAND (CL), reddish brown, moist, medium plasticity, fine- to medium-grained sand												
	0		CLAYEY SAND (SC), dark red, moist, fine- to medium-grained sand, ~20% fines												
10			LEAN CLAY (CL), dark red, moist, medium plasticity												
			CLAYEY SAND (SC), dark red, moist, fine- to medium-grained sand												
			SANDY CLAY (CL), dark red, moist, fine- to medium-grained sand												

Bottom of boring at approximately 13½ feet below the ground surface
No groundwater encountered during drilling

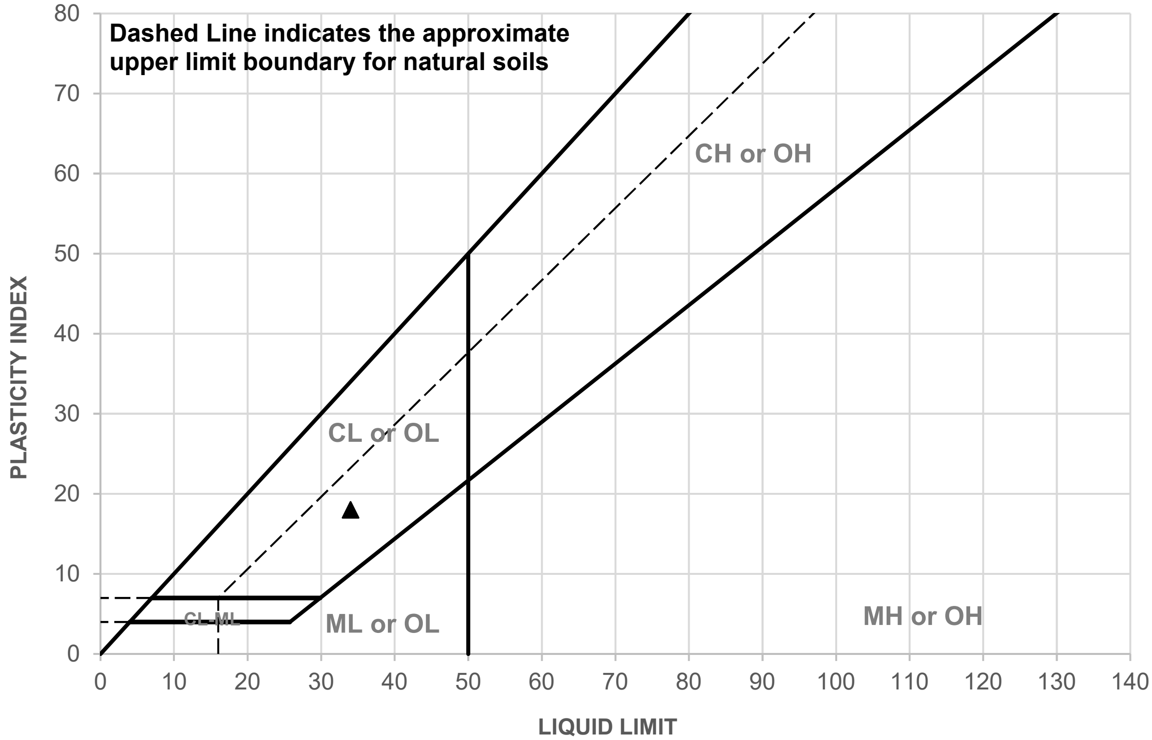
LOG - GEOTECHNICAL_SU+QU W/ ELEV GOLETA TRAIN STATION REV.GPJ ENGEO INC.GDT 9/19/19

APPENDIX B

LABORATORY TEST DATA

Liquid and Plastic Limits Test Report
Unconfined Compression Test
Particle Size Distribution Report
R-Value Test Report
Analytical Results of Soil Corrosion (2 pages)

LIQUID AND PLASTIC LIMITS TEST REPORT
ASTM D4318



	SAMPLE ID	DEPTH	MATERIAL DESCRIPTION	LL	PL	PI
▲	1-B4@8.0 ft	8.0 feet	See exploration logs	34	16	18

	SAMPLE ID	TEST METHOD	REMARKS
▲	1-B4@8.0 ft	PI: ASTM D4318, Wet Method	

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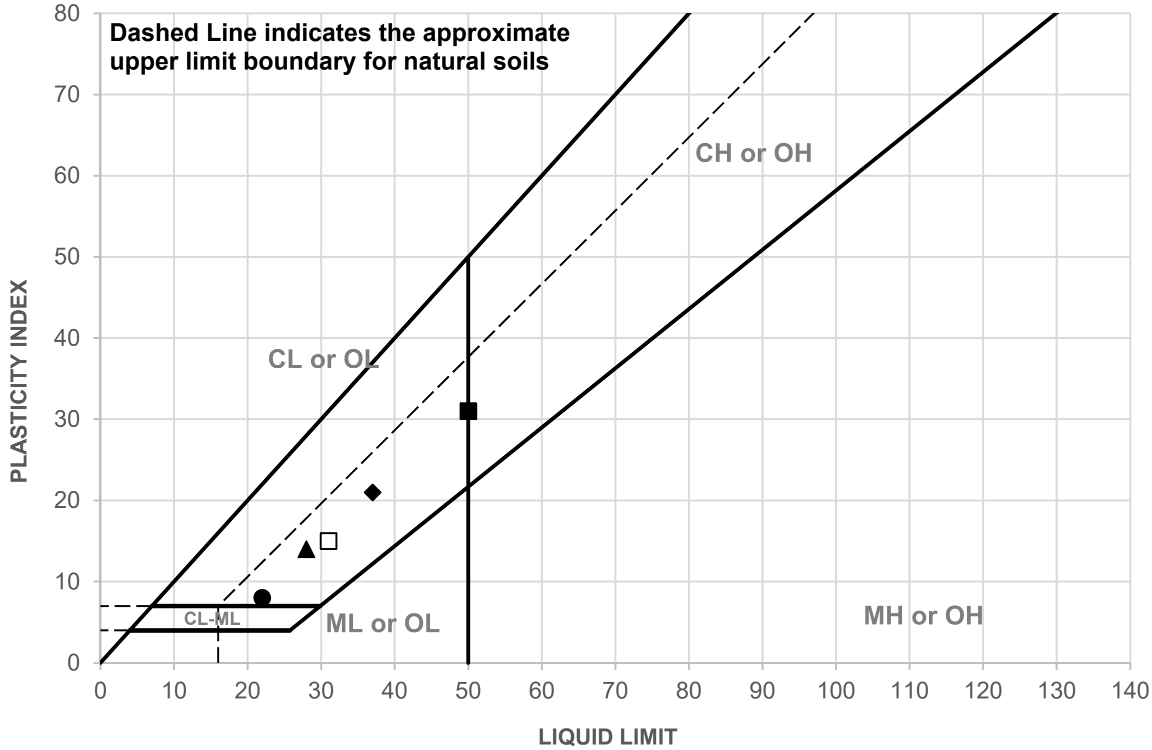
CLIENT: Anil Verma Associates, Inc.
PROJECT NAME: City of Goleta Design for Train Station
PROJECT NO: 16370.000.000
PROJECT LOCATION: Goleta, California
REPORT DATE: 9/19/2019
TESTED BY: G. Criste
REVIEWED BY: D. Seibold

LIQUID AND PLASTIC LIMITS TEST REPORT
ASTM D4318



	SAMPLE ID	DEPTH	MATERIAL DESCRIPTION	LL	PL	PI
▲	1-B1 (Bulk)		See exploration logs	28	14	14
◆	1-B2	1-3 feet	See exploration logs	37	16	21
□	1-B2	6 feet	See exploration logs	31	16	15
●	1-B3	1-3 feet	See exploration logs	22	14	8
■	1-B3	3.5 feet	See exploration logs	50	19	31

	SAMPLE ID	TEST METHOD	REMARKS
▲	1-B1 (Bulk)	PI: ASTM D4318, Wet Method	
◆	1-B2	PI: ASTM D4318, Wet Method	
□	1-B2	PI: ASTM D4318, Wet Method	
●	1-B3	PI: ASTM D4318, Wet Method	
■	1-B3	PI: ASTM D4318, Wet Method	

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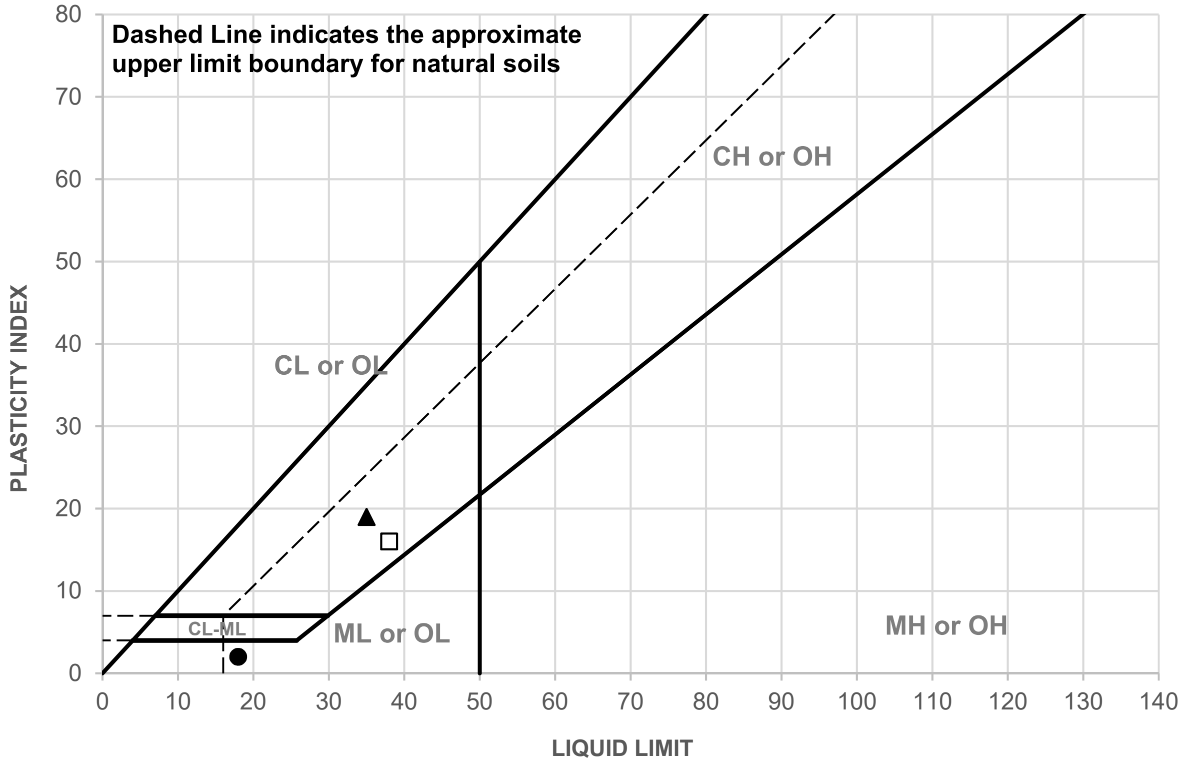
Expect Excellence

CLIENT: Anil Verma Associates, Inc.
PROJECT NAME: City of Goleta Design for Train Station
PROJECT NO: 16370.000.000
PROJECT LOCATION: Goleta, CA
REPORT DATE: 8/29/2019
TESTED BY: L. Santo Domingo
REVIEWED BY: G. Criste

LIQUID AND PLASTIC LIMITS TEST REPORT
ASTM D4318



	SAMPLE ID	DEPTH	MATERIAL DESCRIPTION	LL	PL	PI
▲	1-B3	8.5 feet	See exploration logs	35	16	19
◆	1-B3	32.5 feet	See exploration logs	27	NP	NP
□	1-B3	35 feet	See exploration logs	38	22	16
●	1-B4	3.5 feet	See exploration logs	18	16	2

	SAMPLE ID	TEST METHOD	REMARKS
▲	1-B3	PI: ASTM D4318, Wet Method	
◆	1-B3	PI: ASTM D4318, Wet Method	Could not roll to required 3.2 mm thickness
□	1-B3	PI: ASTM D4318, Wet Method	
●	1-B4	PI: ASTM D4318, Wet Method	

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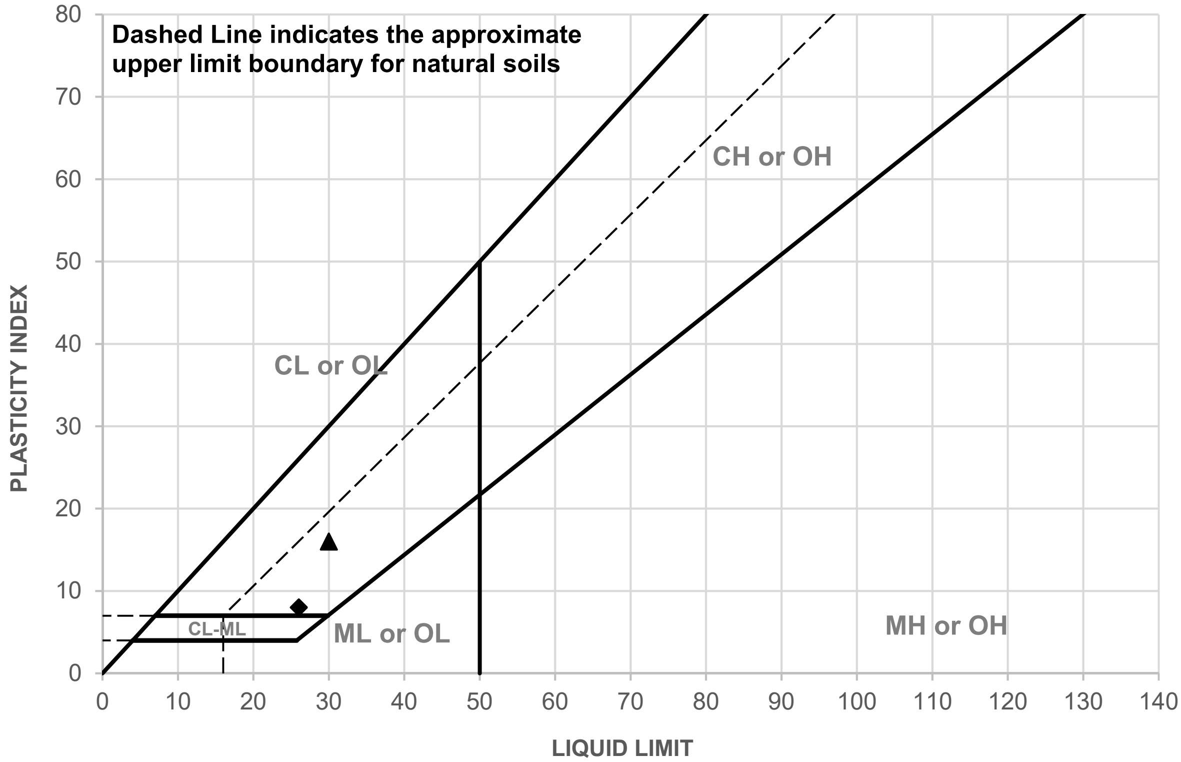
CLIENT: Anil Verma Associates, Inc.
PROJECT NAME: City of Goleta Design for Train Station
PROJECT NO: 16370.000.000
PROJECT LOCATION: Goleta, CA
REPORT DATE: 8/29/2019
TESTED BY: L. Santo Domingo
REVIEWED BY: G. Criste

LIQUID AND PLASTIC LIMITS TEST REPORT
ASTM D4318



	SAMPLE ID	DEPTH	MATERIAL DESCRIPTION	LL	PL	PI
▲	1-B5 (Bulk)		See exploration logs	30	14	16
◆	P2	1-3 feet	See exploration logs	26	18	8

	SAMPLE ID	TEST METHOD	REMARKS
▲	1-B5 (Bulk)	PI: ASTM D4318, Wet Method	
◆	P2	PI: ASTM D4318, Wet Method	

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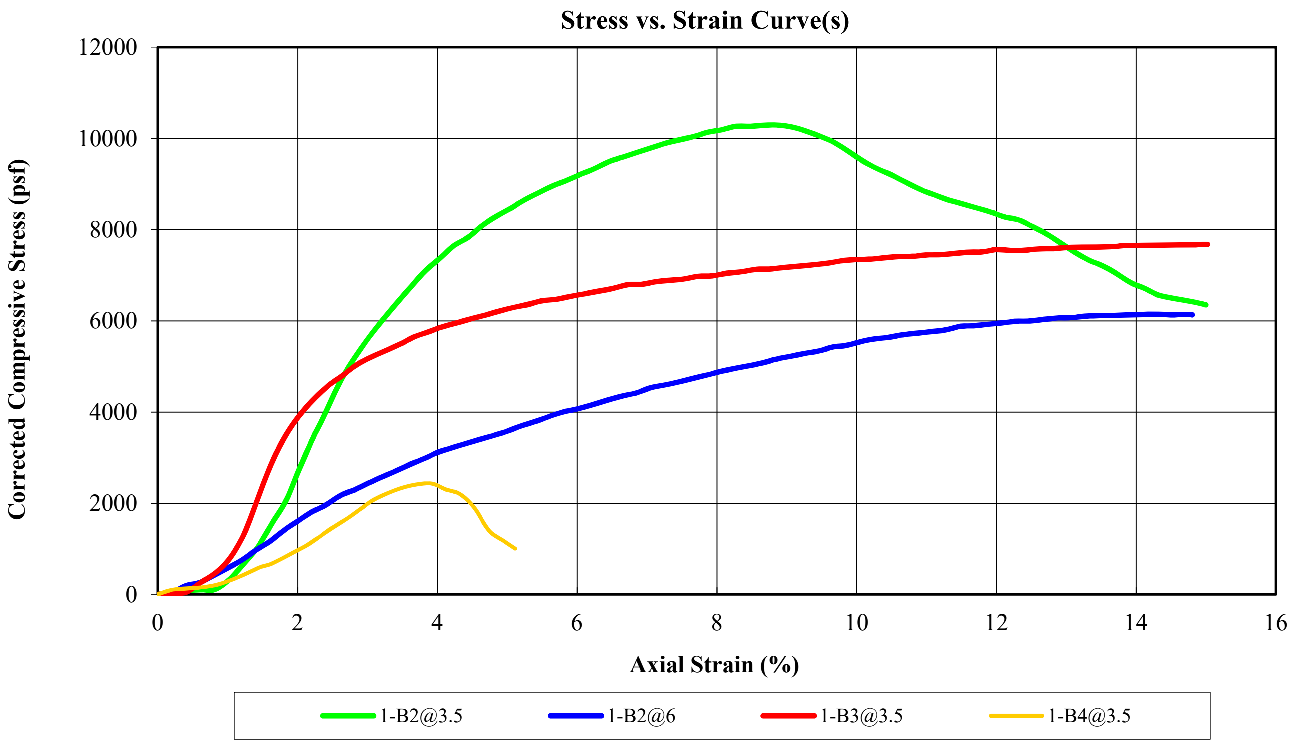
CLIENT: Anil Verma Associates, Inc.
PROJECT NAME: City of Goleta Design for Train Station
PROJECT NO: 16370.000.000
PROJECT LOCATION: Goleta, CA
REPORT DATE: 8/29/2019
TESTED BY: L. Santo Domingo
REVIEWED BY: G. Criste

UNCONFINED COMPRESSION TEST REPORT
(ASTM D2166)



BEFORE TEST	SPECIMEN 1-B2@3.5	1-B2@6	1-B3@3.5	1-B4@3.5
Moisture Content (%)	13.9	16.4	19.8	14.2
Dry Density (pcf)	120.0	115.7	107.8	119.6
Saturation (%)	97.2	100.0	98.3	98.0
Void Ratio	0.38	0.43	0.54	0.38
Diameter (in)	2.410	2.390	2.420	2.420
Height (in)	5.09	5.10	5.05	4.99
Height-To-Diameter Ratio	2.11	2.13	2.09	2.06
TEST DATA				
Unconfined Compressive Strength (psf)	10291	6146	7676	2430
Undrained Shear Strength (psf)	5146	3073	3838	1215
Strain Rate (in./min.)	0.05	0.05	0.05	0.05
Specific Gravity (Assumed)	2.650	2.650	2.650	2.650
Strain at Failure (%)	8.88	14.29	15.02	3.92
Liquid Limit				
Plastic Limit				
Test Remarks				

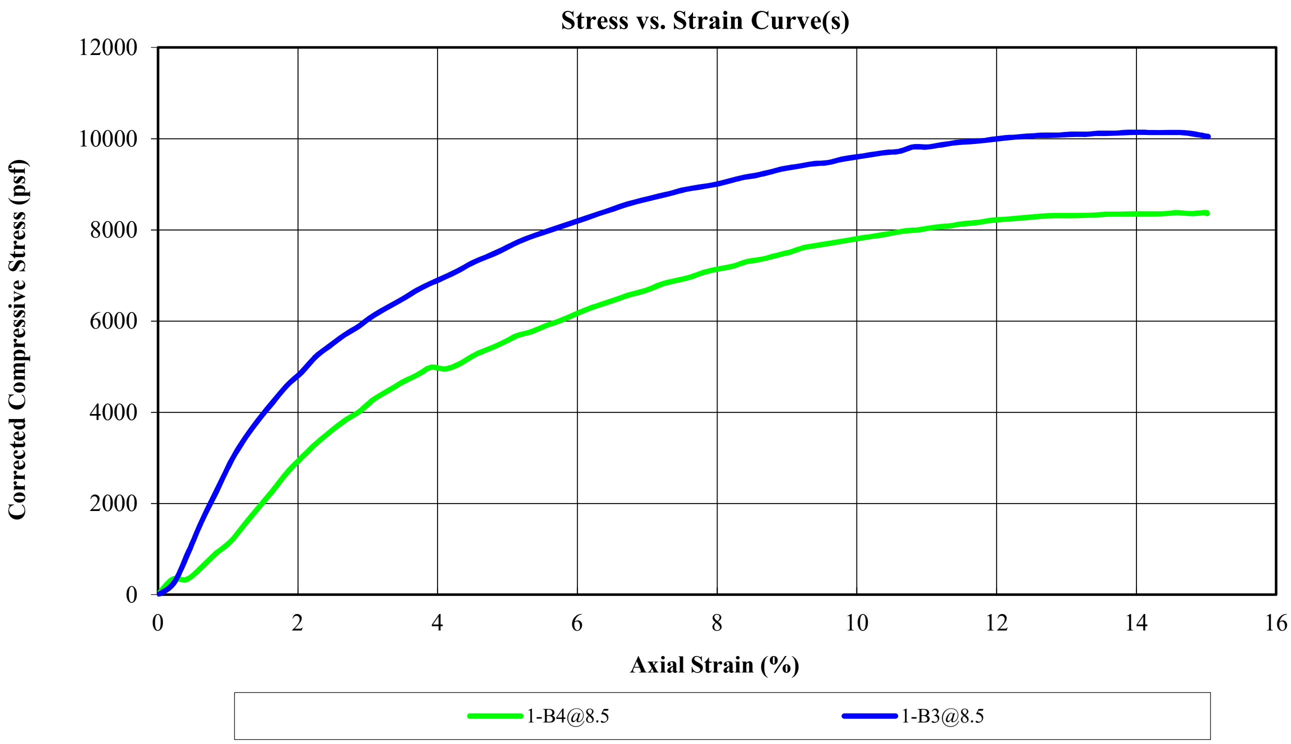
SPECIMEN	DESCRIPTION
1-B2@3.5	See exploration logs
1-B2@6	See exploration logs
1-B3@3.5	See exploration logs
1-B4@3.5	See exploration logs

PROJECT NAME: City of Goleta Design for Train Station
PROJECT NO: 16370.000.000
CLIENT: Anil Verma Associates, Inc.
LOCATION: Goleta, CA
PHASE NO: RIEM

Test Date: 08/28/19
Tested By: M. Quasem
Reviewed By: G. Criste

ENGEO Expect Excellence

UNCONFINED COMPRESSION TEST REPORT
(ASTM D2166)



BEFORE TEST	SPECIMEN 1-B4@8.5	1-B3@8.5
Moisture Content (%)	15.1	16.9
Dry Density (pcf)	116.6	116.4
Saturation (%)	95.6	97.6
Void Ratio	0.42	0.46
Diameter (in)	2.420	2.396
Height (in)	5.00	5.04
Height-To-Diameter Ratio	2.07	2.10
TEST DATA		
Unconfined Compressive Strength (psf)	8380	10138
Undrained Shear Strength (psf)	4190	5069
Strain Rate (in./min.)	0.05	0.05
Specific Gravity (Assumed)	2.650	2.650
Strain at Failure (%)	15.00	13.86
Liquid Limit		
Plastic Limit		
Test Remarks		

SPECIMEN	DESCRIPTION
1-B4@8.5	See exploration logs
1-B3@8.5	See exploration logs

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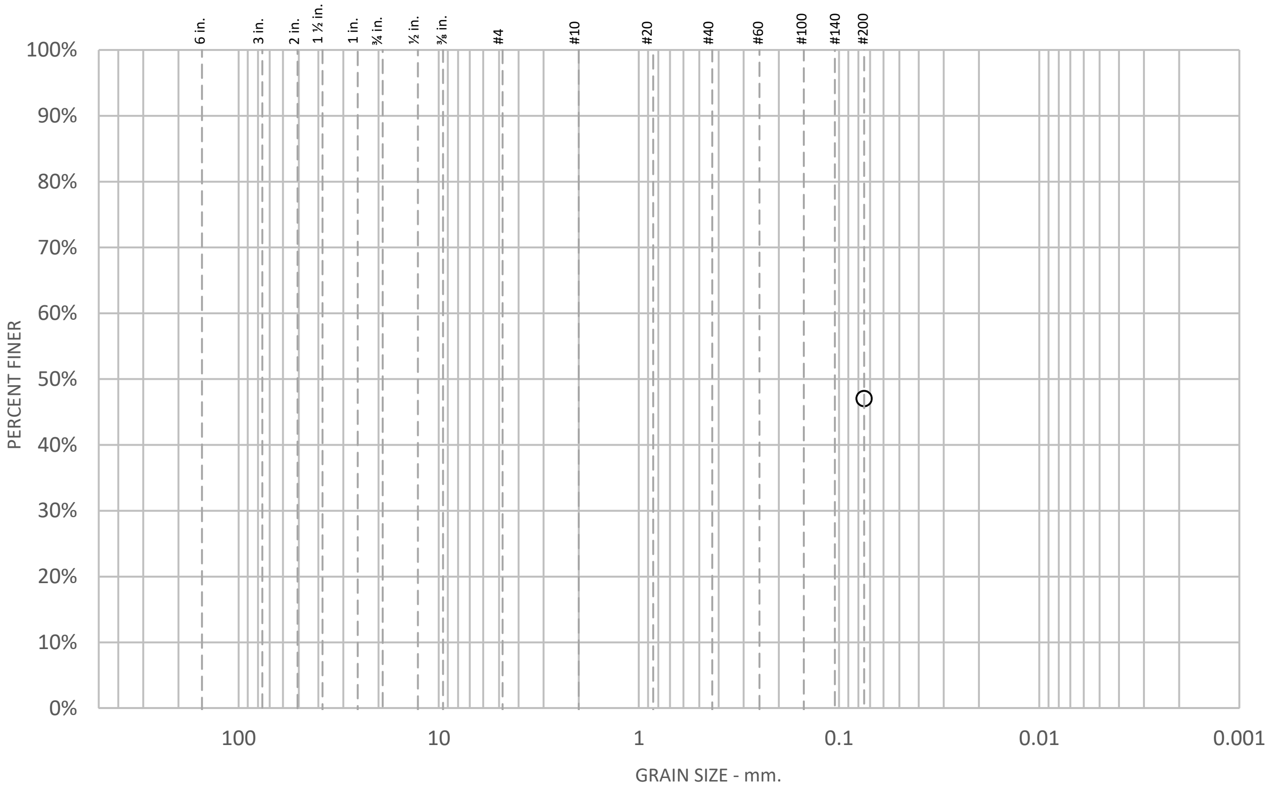
PROJECT NAME: City of Goleta Design for Train Station
PROJECT NO: 16370.000.000
CLIENT: Anil Verma Associates, Inc.
LOCATION: Goleta, CA
PHASE NO: RIEM

Test Date: 08/28/19
Tested By: M. Quasem
Reviewed By: G. Criste

Particle Size Distribution Report



% +75mm	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
						47.0	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	47.0		

* (no specification provided)

Soil Description

See exploration logs

Atterberg Limits

PL = 14 LL = 28 PI = 14

Coefficients

D_{90} = D_{85} = D_{60} =
D_{50} = D_{30} = D_{15} =
D_{10} = C_u = C_c =

Classification

USCS =

Remarks

PI: ASTM D4318, Wet Method
ASTM D1140, Method B
Soak time = 180 min
Dry sample weight = 142.82 g

Sample Number: 1-B1 (Bulk)

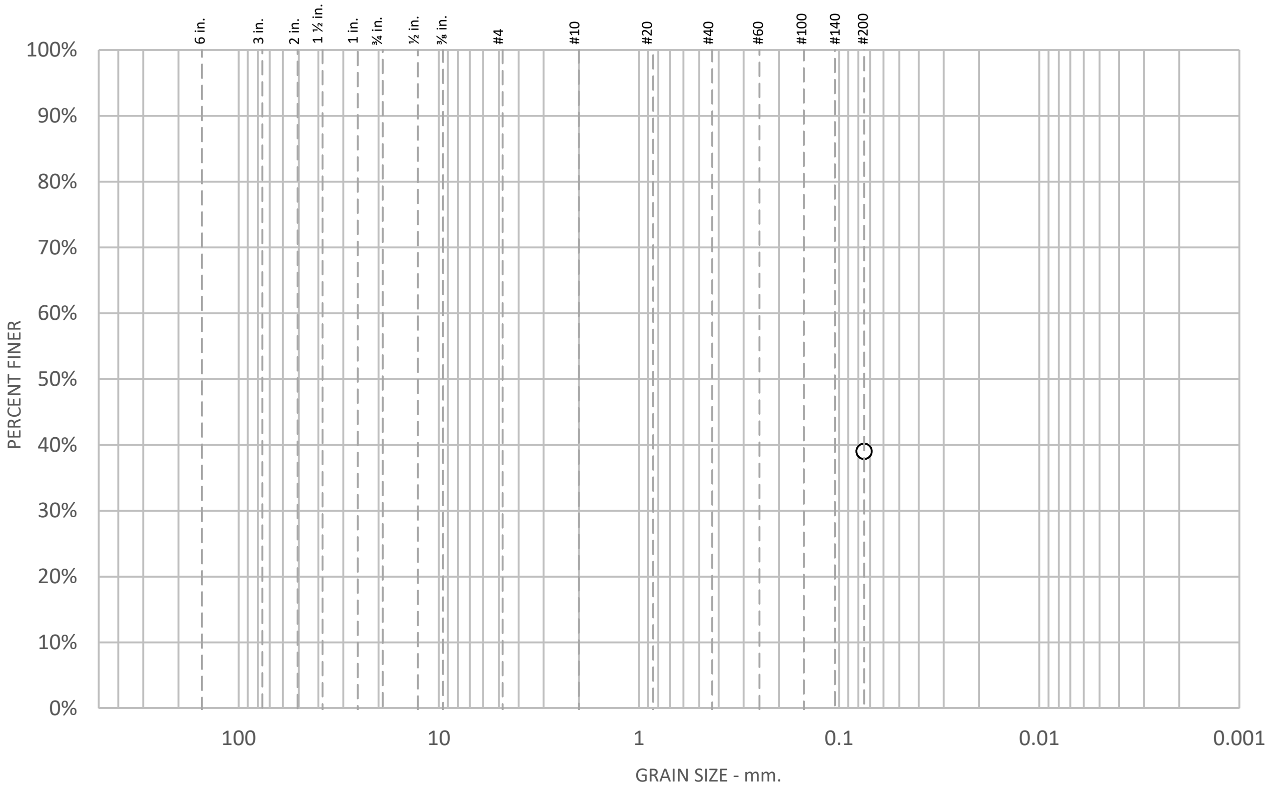
Client: Anil Verma Associates, Inc.
Project Number: 16370.000.000
Project: City of Goleta Design for Trian Station
Date: 8/30/2019
Project location: Goleta, CA

ENGEO Expect Excellence

Tested By: L. Santo Domingo
Checked By: M. Quasem

Test Location: 3420 Fostoria Way, Suite E, Danville, CA 94526

Particle Size Distribution Report



% +75mm	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
						39.0	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	39.0		

* (no specification provided)

Soil Description

See exploration logs

Atterberg Limits

PL = LL = PI =

Coefficients

D_{90} = D_{85} = D_{60} =
D_{50} = D_{30} = D_{15} =
D_{10} = C_u = C_c =

Classification

USCS =

Remarks

ASTM D1140, Method B
Soak time = 180 min
Dry sample weight = 44.85 g

Sample Number: 1-B2 @ 10.5

Client: Anil Verma Associates, Inc. **Project Number:** 16370.000.000
Project: City of Goleta Design for Trian Station **Date:** 8/30/2019
Project location: Goleta, CA

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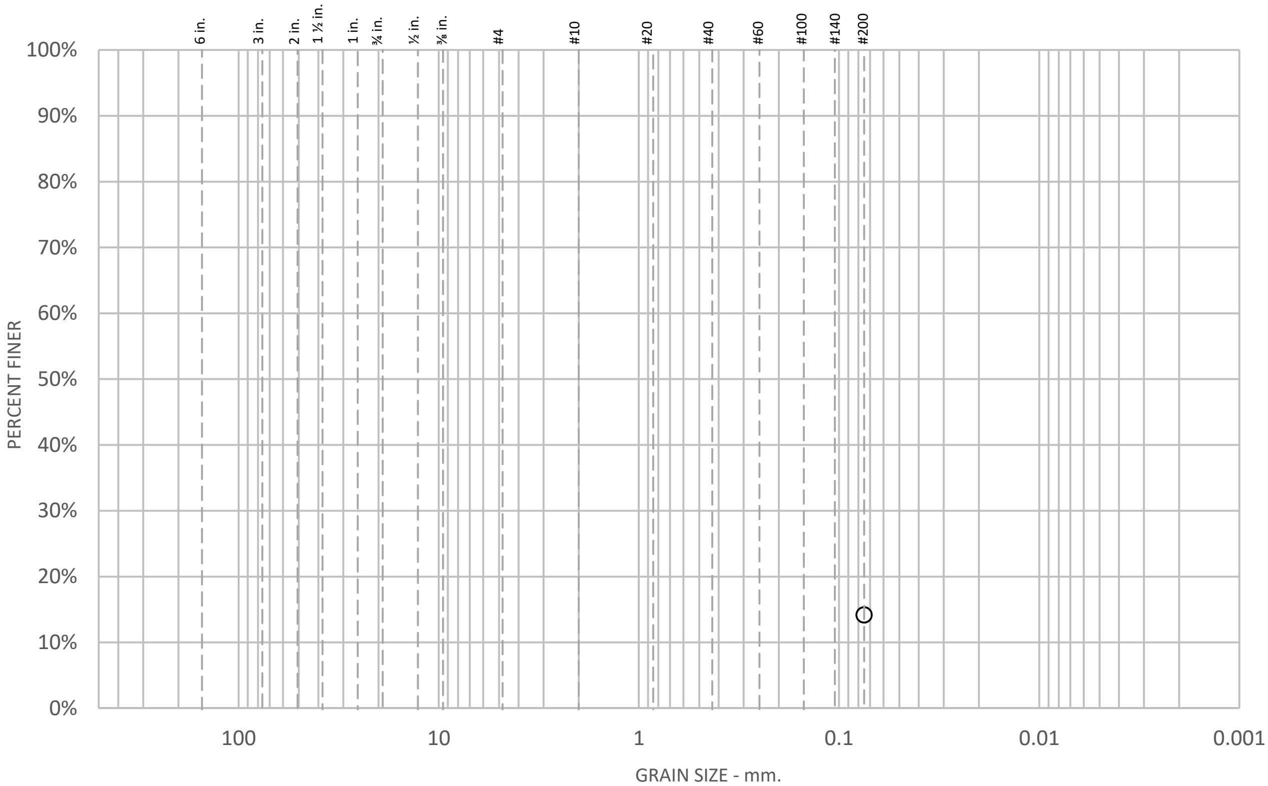
Tested By: L. Santo Domingo **Checked By:** M. Quasem

Test Location: 3420 Fostoria Way, Suite E, Danville, CA 94526

Particle Size Distribution Report



% +75mm	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
						14.2	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	14.2		

* (no specification provided)

Soil Description

See exploration logs

Atterberg Limits

PL = LL = PI =

Coefficients

D_{90} = D_{85} = D_{60} =
D_{50} = D_{30} = D_{15} =
D_{10} = C_u = C_c =

Classification

USCS =

Remarks

ASTM D1140, Method B
Soak time = 180 min
Dry sample weight = 118.52 g

Sample Number: 1-B2 @ 16.5

Client: Anil Verma Associates, Inc. **Project Number:** 16370.000.000
Project: City of Goleta Design for Trian Station **Date:** 8/30/2019
Project location: Goleta, CA

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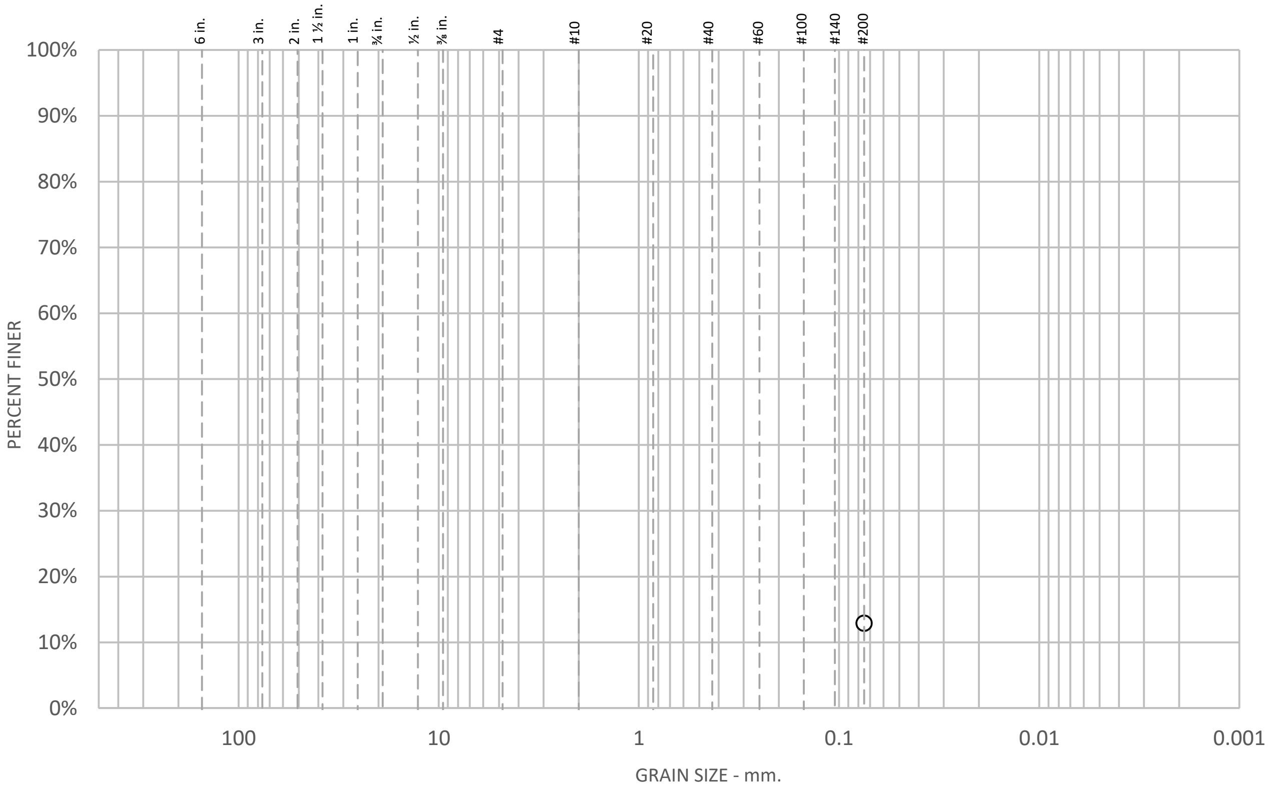
Tested By: L. Santo Domingo **Checked By:** M. Quasem

Test Location: 3420 Fostoria Way, Suite E, Danville, CA 94526

Particle Size Distribution Report



% +75mm	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
						12.9	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	12.9		

* (no specification provided)

Soil Description

See exploration logs

Atterberg Limits

PL = LL = PI =

Coefficients

D_{90} = D_{85} = D_{60} =
D_{50} = D_{30} = D_{15} =
D_{10} = C_u = C_c =

Classification

USCS =

Remarks

ASTM D1140, Method B
Soak time = 180 min
Dry sample weight = 107.75 g

Sample Number: 1-B2 @ 20.5

Client: Anil Verma Associates, Inc.
Project Number: 16370.000.000
Project: City of Goleta Design for Trian Station
Date: 8/30/2019
Project location: Goleta, CA

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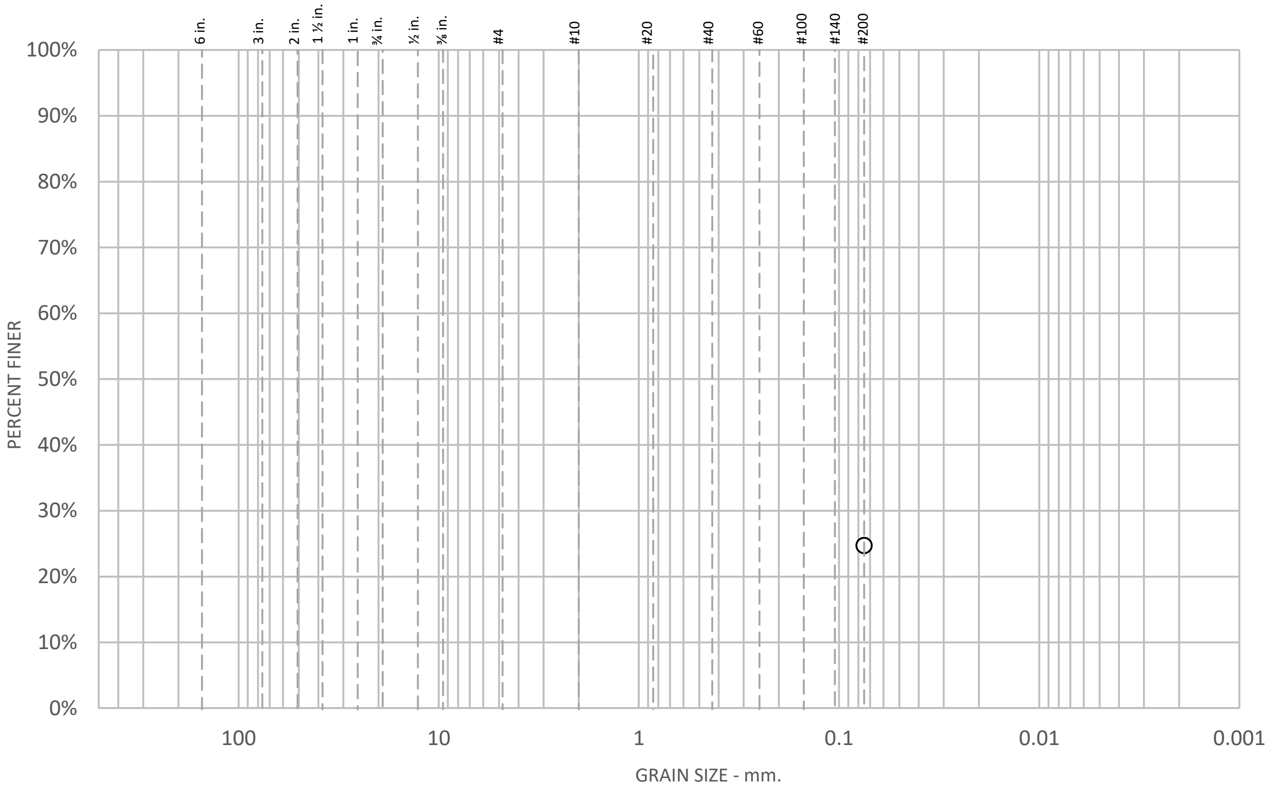
Tested By: L. Santo Domingo
Checked By: M. Quasem

Test Location: 3420 Fostoria Way, Suite E, Danville, CA 94526

Particle Size Distribution Report



% +75mm	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
						24.7	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	24.7		

* (no specification provided)

Soil Description

See exploration logs

Atterberg Limits

PL =　LL =　PI =

Coefficients

D_{90} =　D_{85} =　D_{60} =
D_{50} =　D_{30} =　D_{15} =
D_{10} =　C_u =　C_c =

Classification

USCS =

Remarks

ASTM D1140, Method B
Soak time = 180 min
Dry sample weight = 227.5 g

Sample Number: 1-B2 @ 25.5

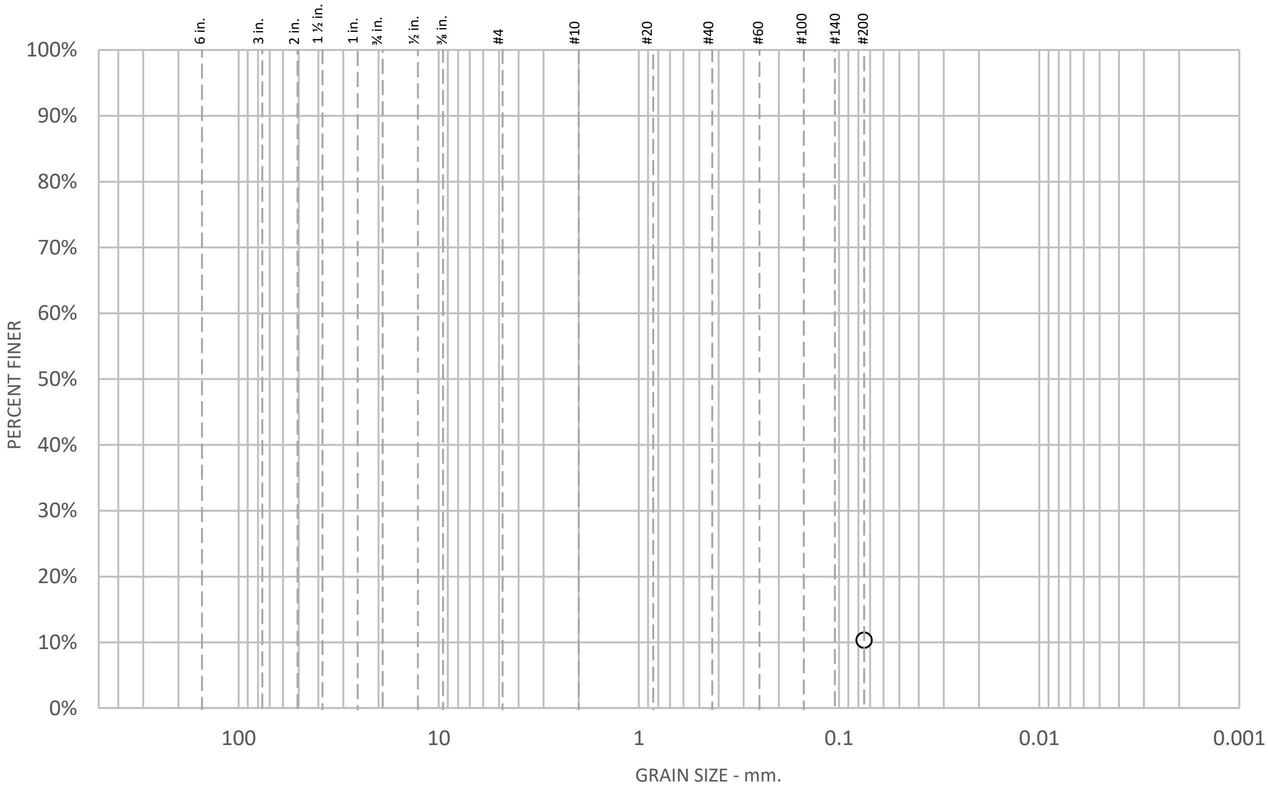
Client: Anil Verma Associates, Inc.
Project Number: 16370.000.000
Project: City of Goleta Design for Trian Station
Date: 8/30/2019
Project location: Goleta, CA

ENGEO Expect Excellence

Tested By: L. Santo Domingo
Checked By: M. Quasem

Test Location: 3420 Fostoria Way, Suite E, Danville, CA 94526

Particle Size Distribution Report



% +75mm	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
						10.3	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	10.3		

* (no specification provided)

Soil Description

See exploration logs

Atterberg Limits

PL = LL = PI =

Coefficients

D_{90} = D_{85} = D_{60} =
D_{50} = D_{30} = D_{15} =
D_{10} = C_u = C_c =

Classification

USCS =

Remarks

ASTM D1140, Method B
Soak time = 180 min
Dry sample weight = 166.76 g

Sample Number: 1-B2 @ 35

Client: Anil Verma Associates, Inc. **Project Number:** 16370.000.000
Project: City of Goleta Design for Trian Station **Date:** 8/30/2019
Project location: Goleta, CA

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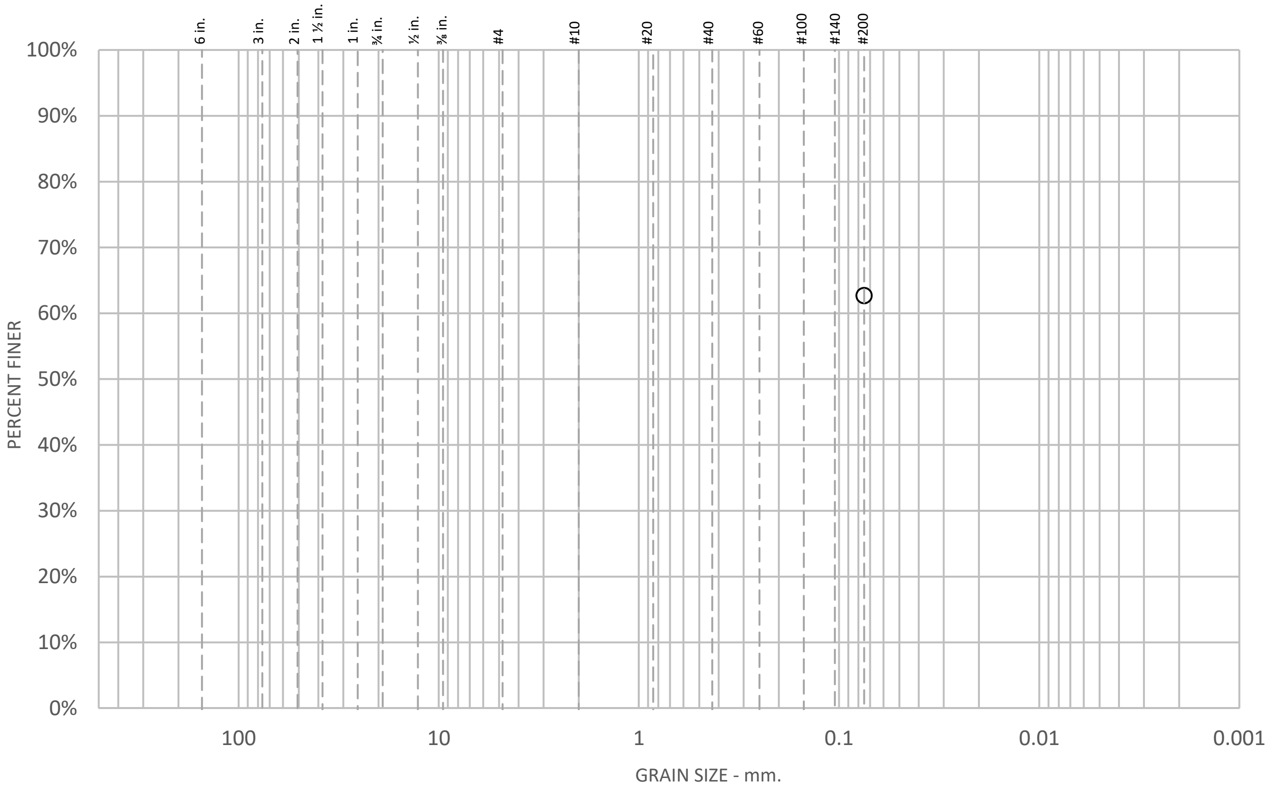
Tested By: L. Santo Domingo **Checked By:** M. Quasem

Test Location: 3420 Fostoria Way, Suite E, Danville, CA 94526

Particle Size Distribution Report



% +75mm	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
						62.7	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	62.7		

* (no specification provided)

Soil Description

See exploration logs

Atterberg Limits

PL = LL = PI =

Coefficients

D_{90} = D_{85} = D_{60} =
D_{50} = D_{30} = D_{15} =
D_{10} = C_u = C_c =

Classification

USCS =

Remarks

ASTM D1140, Method B
Soak time = 180 min
Dry sample weight = 122.7 g

Sample Number: 1-B3 @ 11

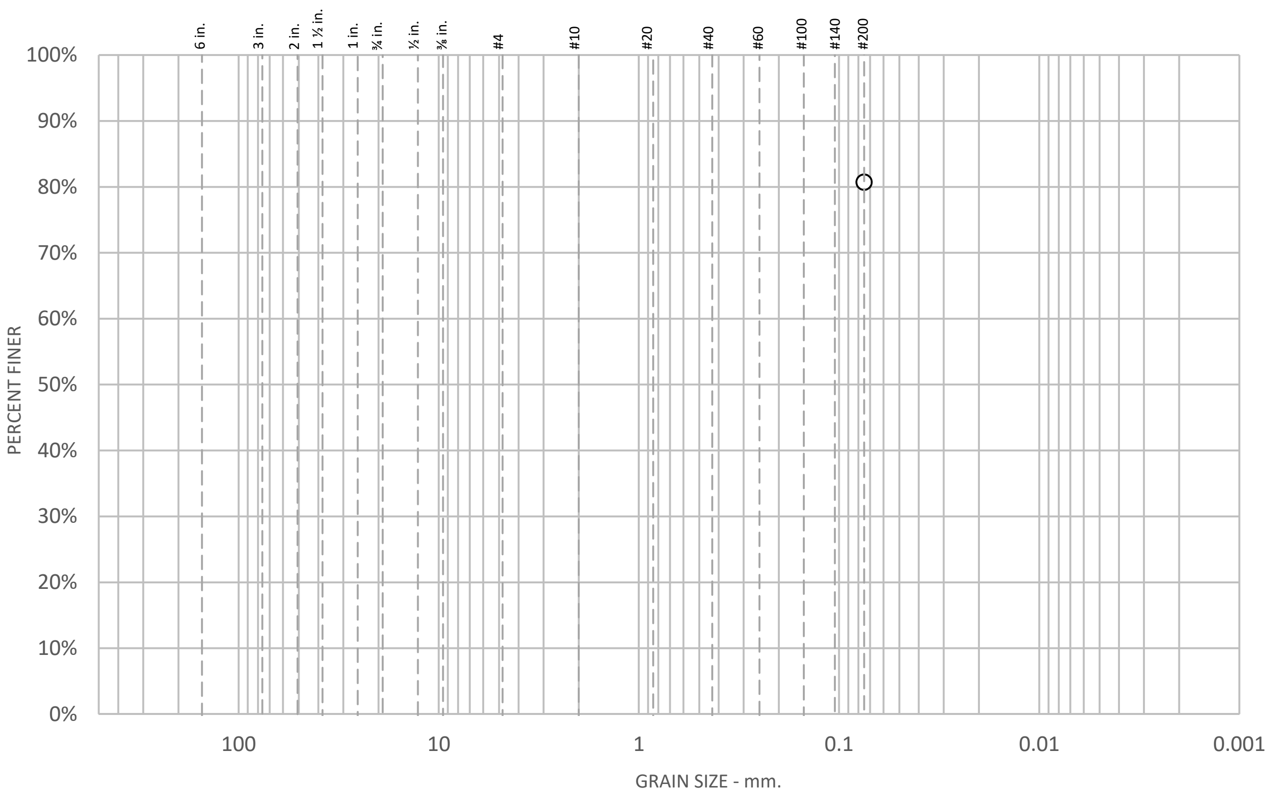
Client: Anil Verma Associates, Inc. **Project Number:** 16370.000.000
Project: City of Goleta Design for Trian Station **Date:** 8/30/2019
Project location: Goleta, CA

ENGEO Expect Excellence

Tested By: L. Santo Domingo **Checked By:** M. Quasem

Test Location: 3420 Fostoria Way, Suite E, Danville, CA 94526

Particle Size Distribution Report



% +75mm	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
						80.7	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	80.7		

* (no specification provided)

Soil Description

See exploration logs

Atterberg Limits

PL = LL = PI =

Coefficients

D_{90} = D_{85} = D_{60} =
D_{50} = D_{30} = D_{15} =
D_{10} = C_u = C_c =

Classification

USCS =

Remarks

ASTM D1140, Method B
Soak time = 180 min
Dry sample weight = 159.2 g

Sample Number: 1-B3 @ 15

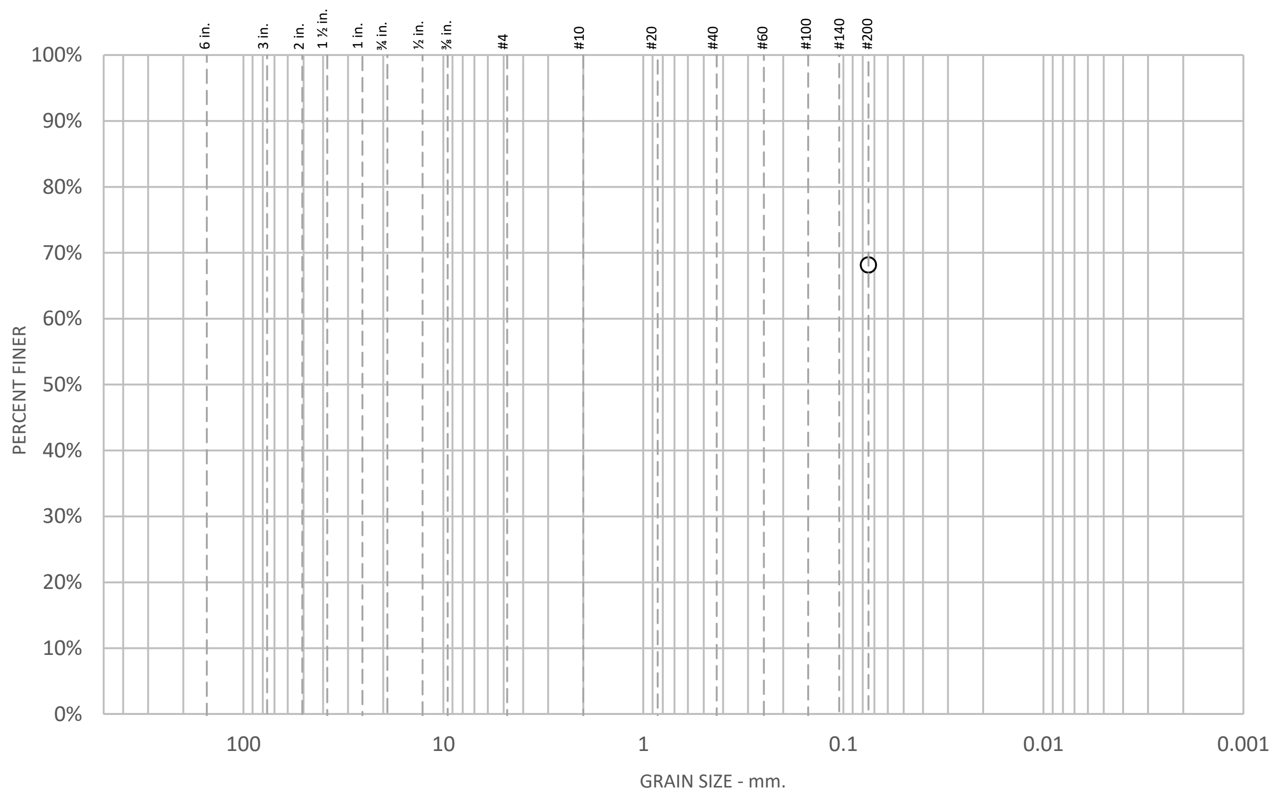
Client: Anil Verma Associates, Inc. **Project Number:** 16370.000.000
Project: City of Goleta Design for Trian Station **Date:** 8/30/2019
Project location: Goleta, CA

ENGEO Expect Excellence

Tested By: L. Santo Domingo **Checked By:** M. Quasem

Test Location: 3420 Fostoria Way, Suite E, Danville, CA 94526

Particle Size Distribution Report



% +75mm	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
						68.1	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	68.1		

* (no specification provided)

Soil Description

See exploration logs

Atterberg Limits

PL = LL = PI =

Coefficients

D_{90} = D_{85} = D_{60} =
D_{50} = D_{30} = D_{15} =
D_{10} = C_u = C_c =

Classification

USCS =

Remarks

ASTM D1140, Method B
Soak time = 180 min
Dry sample weight = 131.42 g

Sample Number: 1-B3 @ 20

Client: Anil Verma Associates, Inc. **Project Number:** 16370.000.000
Project: City of Goleta Design for Trian Station **Date:** 8/30/2019
Project location: Goleta, CA

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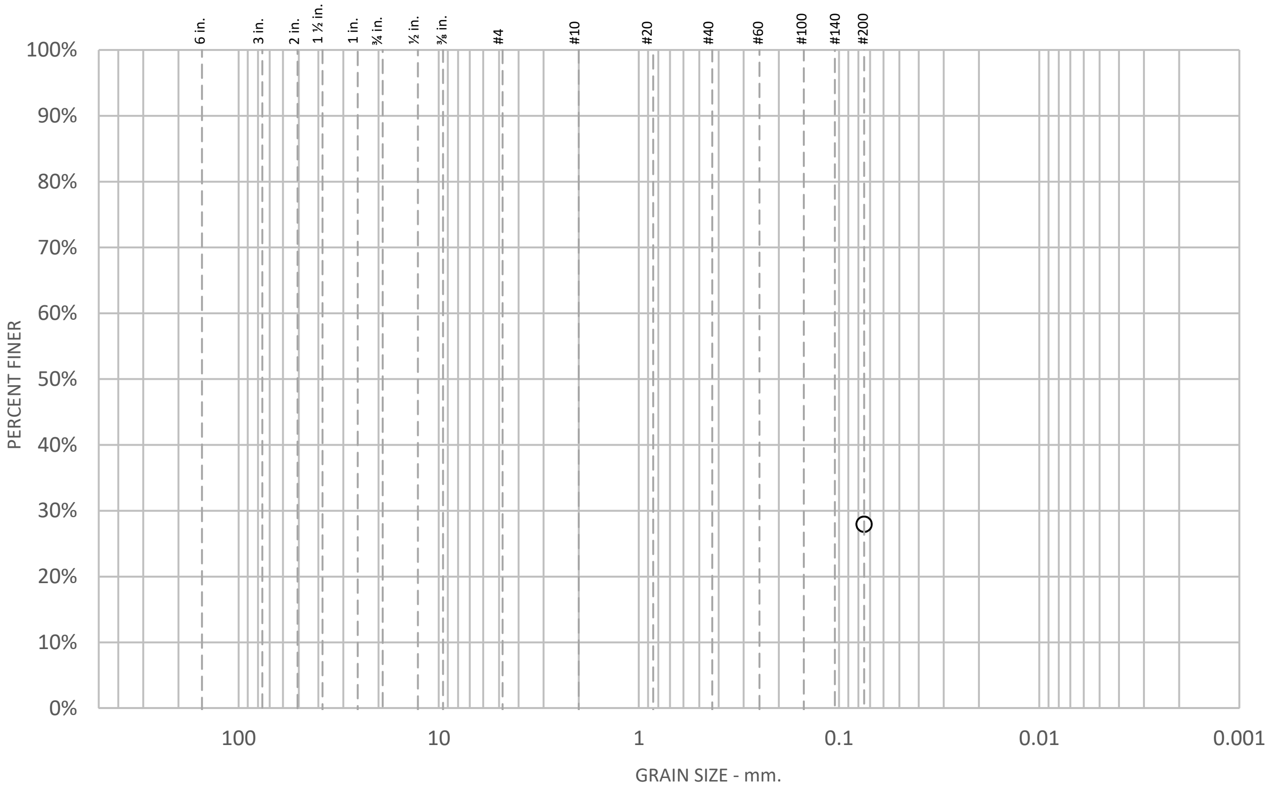
Tested By: L. Santo Domingo **Checked By:** M. Quasem

Test Location: 3420 Fostoria Way, Suite E, Danville, CA 94526

Particle Size Distribution Report



% +75mm	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
						28.0	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	28.0		

* (no specification provided)

Soil Description

See exploration logs

Atterberg Limits

PL = LL = PI =

Coefficients

D_{90} = D_{85} = D_{60} =
D_{50} = D_{30} = D_{15} =
D_{10} = C_u = C_c =

Classification

USCS =

Remarks

ASTM D1140, Method B
Soak time = 180 min
Dry sample weight = 109.97 g

Sample Number: 1-B3 @ 25

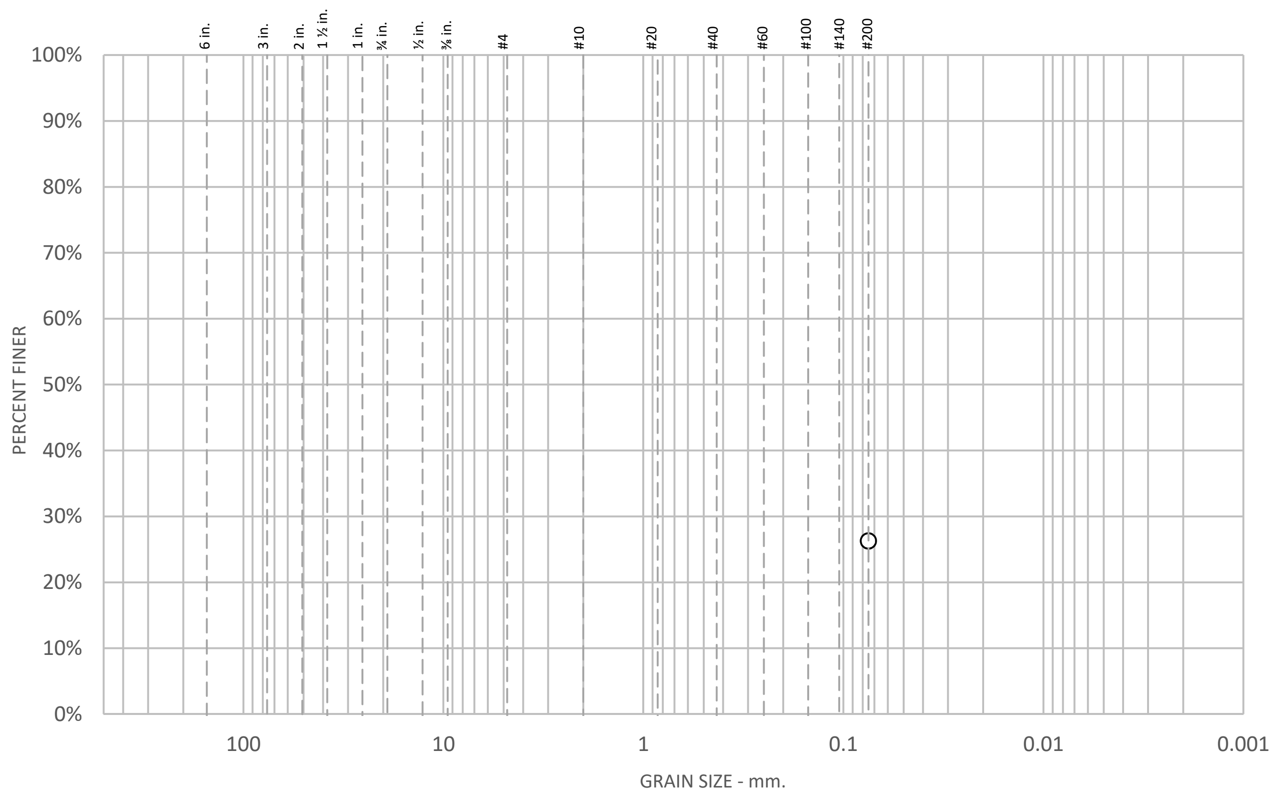
Client: Anil Verma Associates, Inc. **Project Number:** 16370.000.000
Project: City of Goleta Design for Trian Station **Date:** 8/30/2019
Project location: Goleta, CA

ENGEO Expect Excellence

Tested By: L. Santo Domingo **Checked By:** M. Quasem

Test Location: 3420 Fostoria Way, Suite E, Danville, CA 94526

Particle Size Distribution Report



% +75mm	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
						26.3	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	26.3		

* (no specification provided)

Soil Description

See exploration logs

Atterberg Limits

PL = LL = PI =

Coefficients

D_{90} = D_{85} = D_{60} =
D_{50} = D_{30} = D_{15} =
D_{10} = C_u = C_c =

Classification

USCS =

Remarks

ASTM D1140, Method B
Soak time = 180 min
Dry sample weight = 185.99 g

Sample Number: 1-B3 @ 30

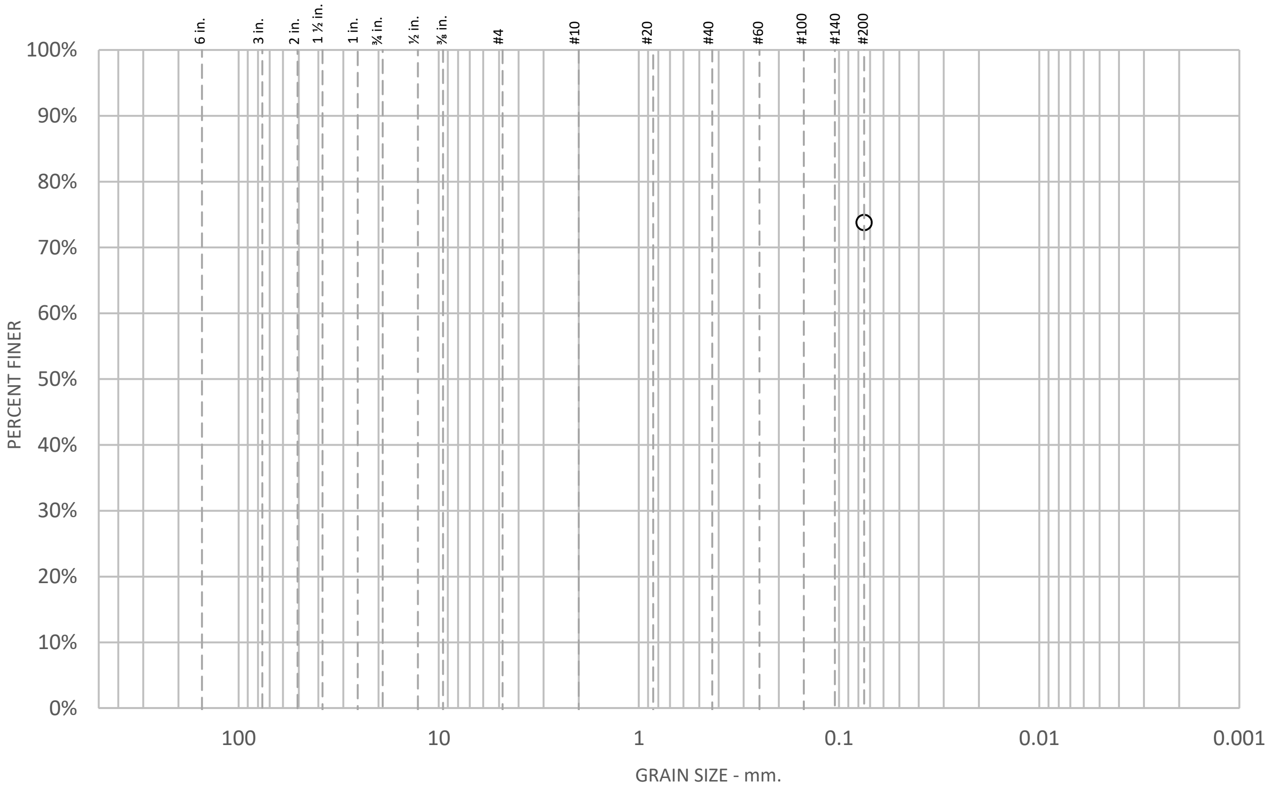
Client: Anil Verma Associates, Inc.
Project Number: 16370.000.000
Project: City of Goleta Design for Trian Station
Date: 8/30/2019
Project location: Goleta, CA

ENGEO Expect Excellence

Tested By: L. Santo Domingo
Checked By: M. Quasem

Test Location: 3420 Fostoria Way, Suite E, Danville, CA 94526

Particle Size Distribution Report



% +75mm	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
						73.8	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	73.8		

* (no specification provided)

Soil Description

See exploration logs

Atterberg Limits

PL = NP LL = NV PI = NP

Coefficients

D_{90} = D_{85} = D_{60} =
D_{50} = D_{30} = D_{15} =
D_{10} = C_u = C_c =

Classification

USCS =

Remarks

PI: ASTM D4318, Wet Method

ASTM D1140, Method B
Soak time = 180 min
Dry sample weight = 109.91 g

Sample Number: 1-B3 @ 32.5

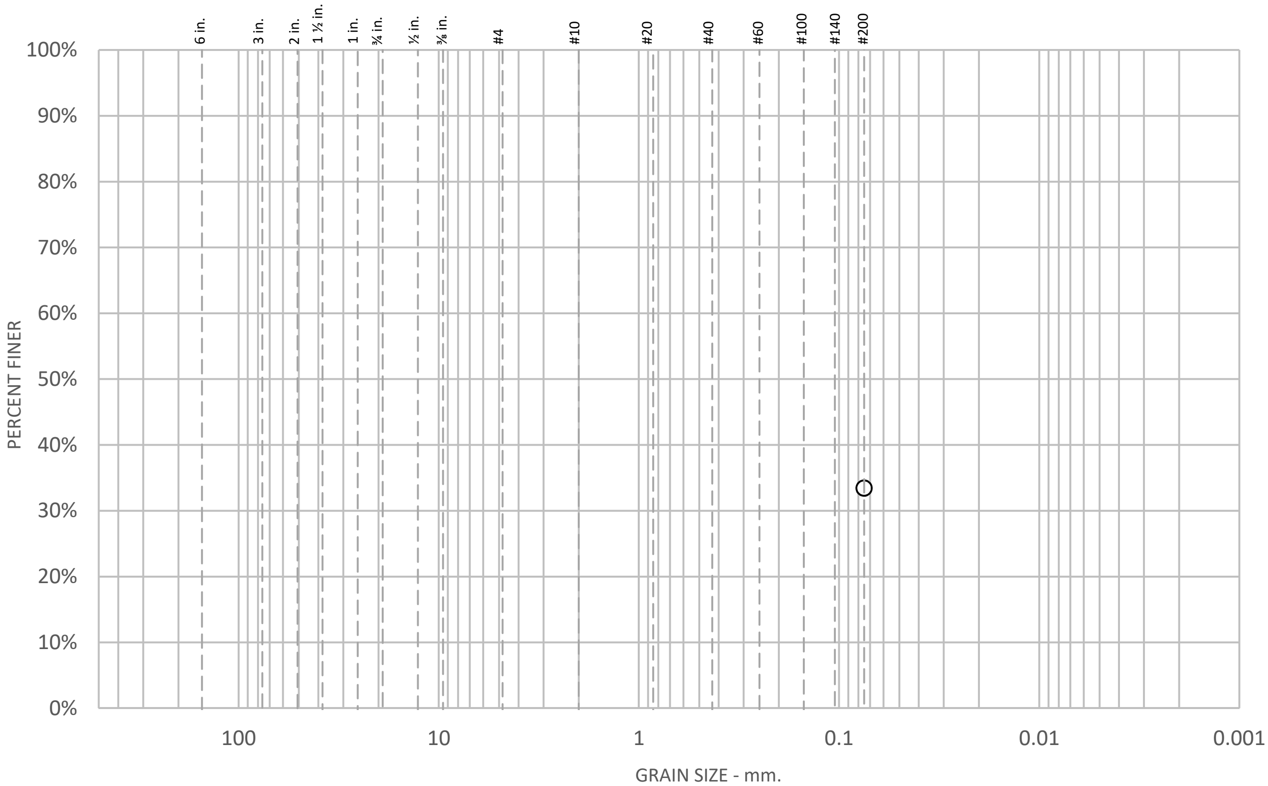
Client: Anil Verma Associates, Inc. **Project Number:** 16370.000.000
Project: City of Goleta Design for Trian Station **Date:** 8/30/2019
Project location: Goleta, CA

ENGEO Expect Excellence

Tested By: L. Santo Domingo **Checked By:** M. Quasem

Test Location: 3420 Fostoria Way, Suite E, Danville, CA 94526

Particle Size Distribution Report



% +75mm	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
						33.4	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	33.4		

* (no specification provided)

Soil Description

See exploration logs

Atterberg Limits

PL = LL = PI =

Coefficients

D_{90} = D_{85} = D_{60} =
D_{50} = D_{30} = D_{15} =
D_{10} = C_u = C_c =

Classification

USCS =

Remarks

ASTM D1140, Method B
Soak time = 180 min
Dry sample weight = 174.65 g

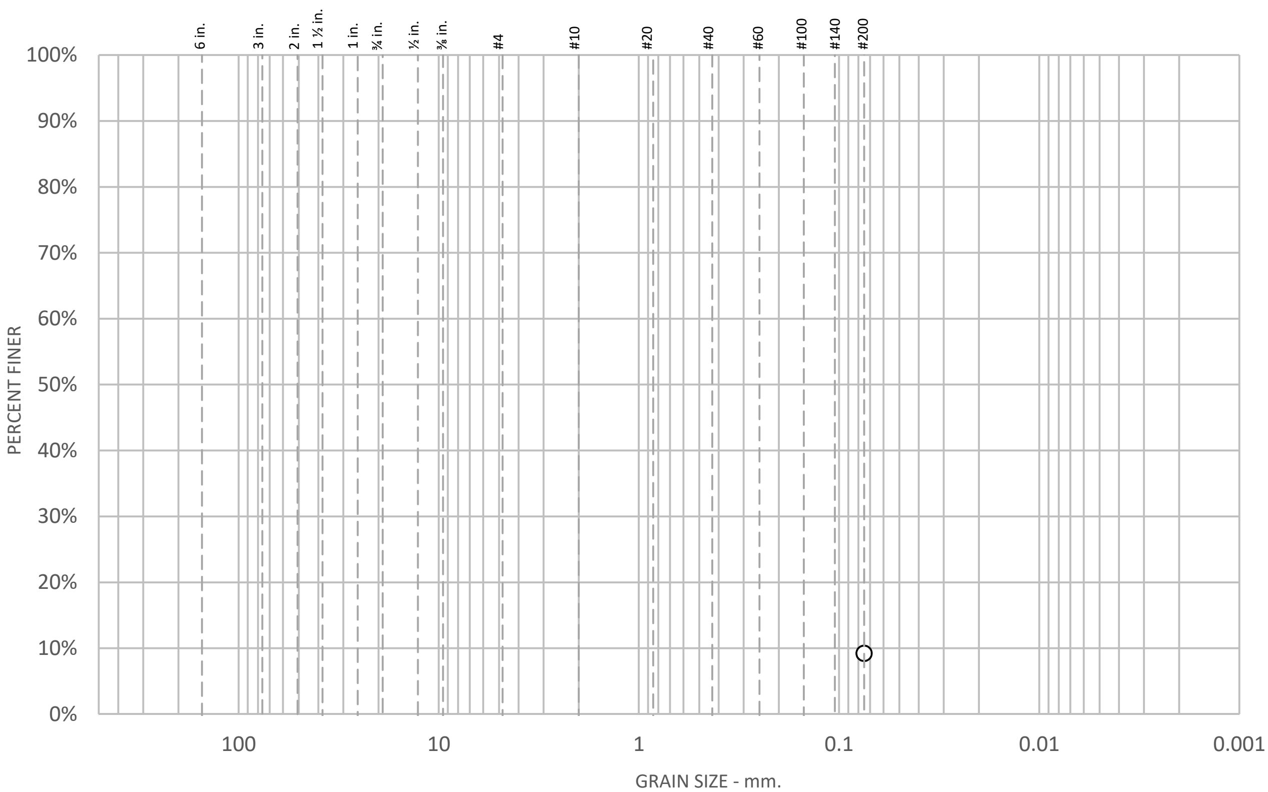
Sample Number: 1-B4 @ 20

Client: Anil Verma Associates, Inc. **Project Number:** 16370.000.000
Project: City of Goleta Design for Trian Station **Date:** 8/30/2019
Project location: Goleta, CA

Tested By: L. Santo Domingo **Checked By:** M. Quasem

Test Location: 3420 Fostoria Way, Suite E, Danville, CA 94526

Particle Size Distribution Report



% +75mm	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
						9.2	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	9.2		

* (no specification provided)

Soil Description

See exploration logs

Atterberg Limits

PL = LL = PI =

Coefficients

D_{90} = D_{85} = D_{60} =
D_{50} = D_{30} = D_{15} =
D_{10} = C_u = C_c =

Classification

USCS =

Remarks

ASTM D1140, Method B
Soak time = 180 min
Dry sample weight = 141.65 g

Sample Number: 1-B4 @ 25

Client: Anil Verma Associates, Inc. **Project Number:** 16370.000.000
Project: City of Goleta Design for Trian Station **Date:** 8/30/2019
Project location: Goleta, CA

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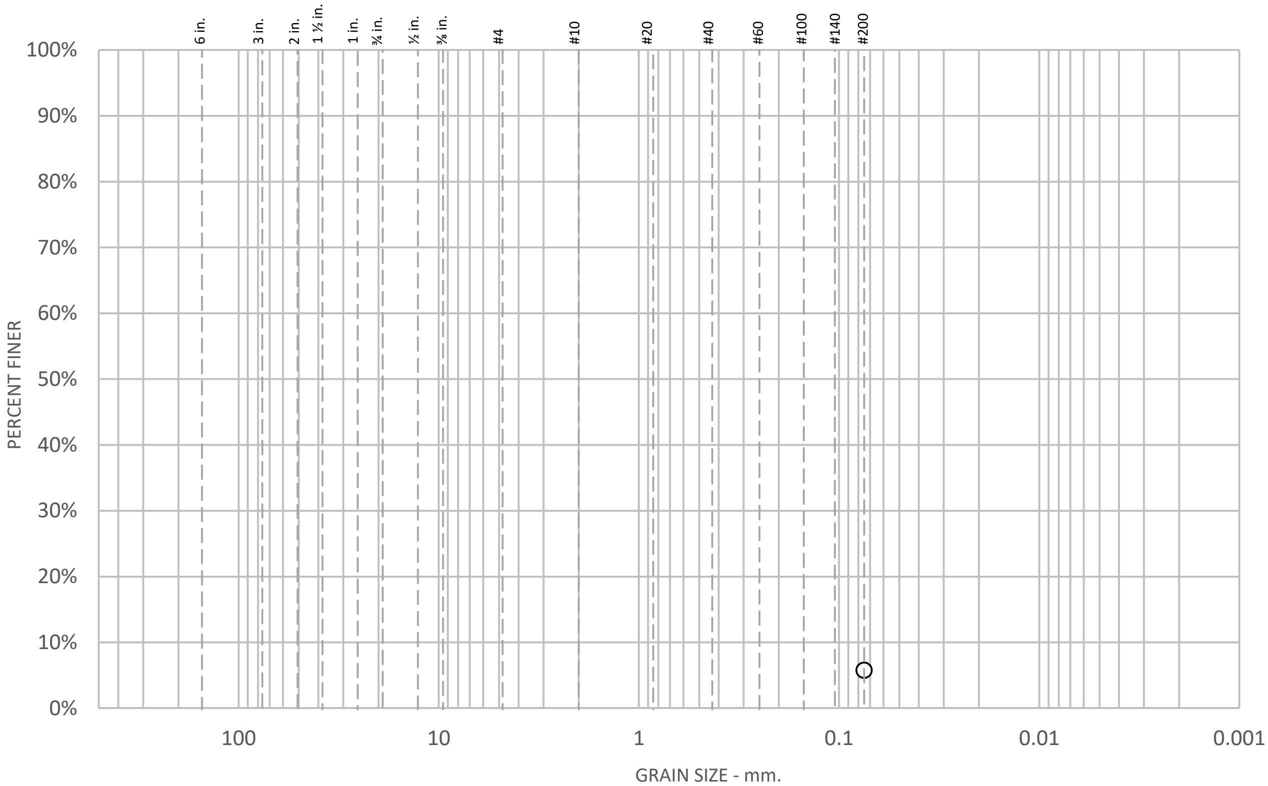
Tested By: L. Santo Domingo **Checked By:** M. Quasem

Test Location: 3420 Fostoria Way, Suite E, Danville, CA 94526

Particle Size Distribution Report



% +75mm	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
						5.8	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	5.8		

* (no specification provided)

Soil Description

See exploration logs

Atterberg Limits

PL = LL = PI =

Coefficients

D_{90} = D_{85} = D_{60} =
D_{50} = D_{30} = D_{15} =
D_{10} = C_u = C_c =

Classification

USCS =

Remarks

ASTM D1140, Method B
Soak time = 180 min
Dry sample weight = 172.09 g

Sample Number: 1-B4 @ 30

Client: Anil Verma Associates, Inc. **Project Number:** 16370.000.000
Project: City of Goleta Design for Trian Station **Date:** 8/30/2019
Project location: Goleta, CA

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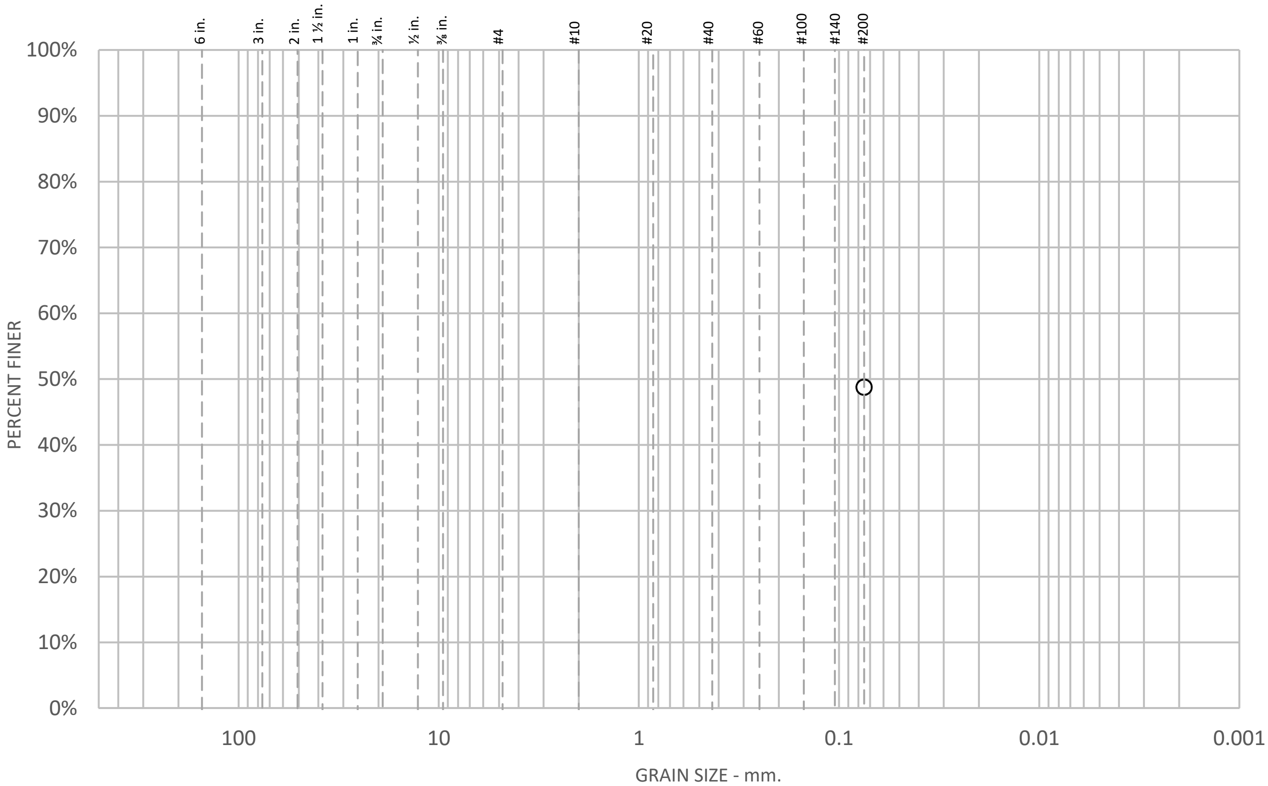
Tested By: L. Santo Domingo **Checked By:** M. Quasem

Test Location: 3420 Fostoria Way, Suite E, Danville, CA 94526

Particle Size Distribution Report



% +75mm	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
						48.8	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	48.8		

* (no specification provided)

Soil Description

See exploration logs

Atterberg Limits

PL = 14 LL = 30 PI = 16

Coefficients

D_{90} = D_{85} = D_{60} =
D_{50} = D_{30} = D_{15} =
D_{10} = C_u = C_c =

Classification

USCS =

Remarks

PI: ASTM D4318, Wet Method

ASTM D1140, Method B
Soak time = 180 min
Dry sample weight = 158.43 g

Sample Number: 1-B5 (Bulk)

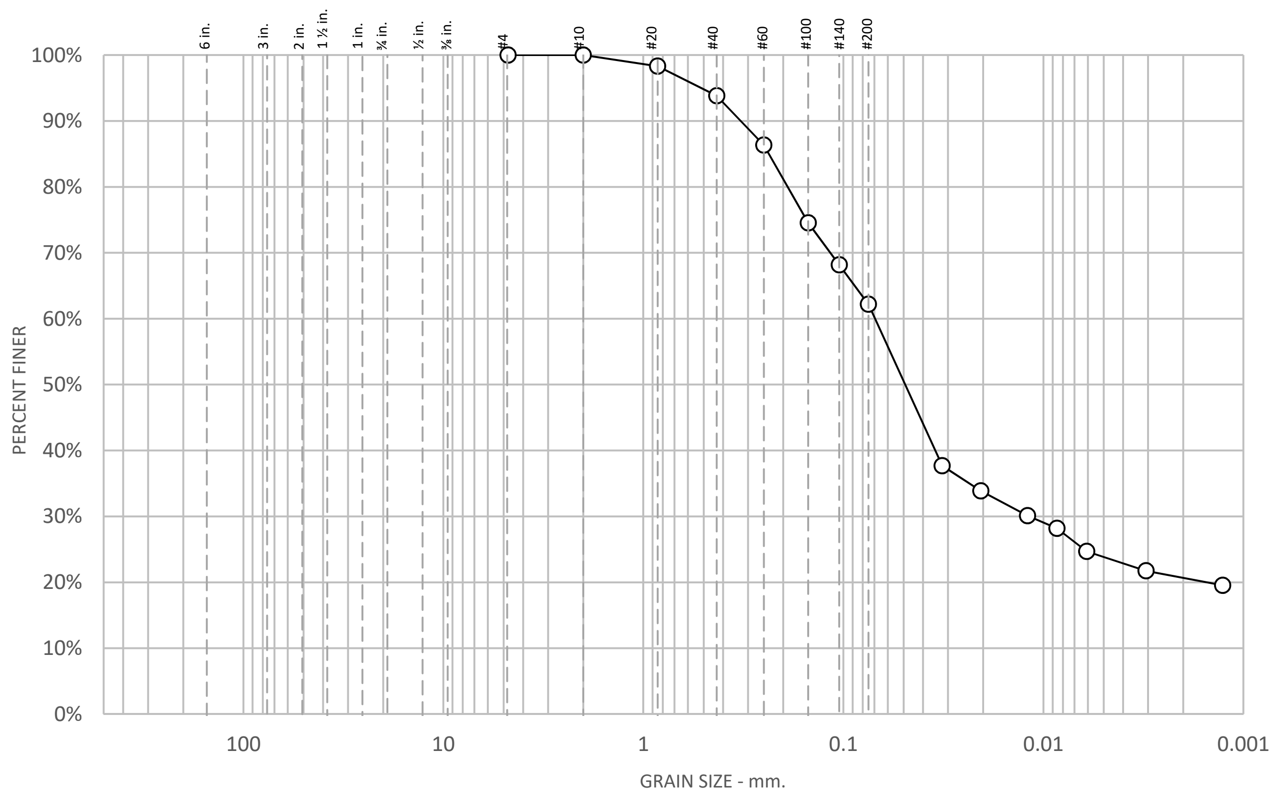
Client: Anil Verma Associates, Inc. **Project Number:** 16370.000.000
Project: City of Goleta Design for Trian Station **Date:** 8/30/2019
Project location: Goleta, CA

ENGEO Expect Excellence

Tested By: L. Santo Domingo **Checked By:** M. Quasem

Test Location: 3420 Fostoria Way, Suite E, Danville, CA 94526

Particle Size Distribution Report



% +75mm	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
				6.2	31.6	41.5	20.7

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#4	100.0		
#10	100.0		
#20	98.3		
#40	93.8		
#60	86.3		
#100	74.5		
#140	68.2		
#200	62.2		
0.0321 mm.	37.7		
0.0206 mm.	33.9		
0.0120 mm.	30.1		
0.0086 mm.	28.2		
0.0061 mm.	24.7		
0.0031 mm.	21.7		
0.0013 mm.	19.5		

* (no specification provided)

Soil Description

See exploration logs

Atterberg Limits

PL = 16 LL = 37 PI = 21

Coefficients

D_{90} = 0.3260 mm D_{85} = 0.2359 mm D_{60} = 0.0695 mm
D_{50} = 0.0491 mm D_{30} = 0.0118 mm D_{15} =
D_{10} = C_u = C_c =

Classification

USCS = CL

Remarks

GS: ASTM D422 ASTM D422
Silt/clay division of 0.002mm used
PI: ASTM D4318, Wet Method
USCS: ASTM D2487

Sample Number: 1-B2 @ 1-3

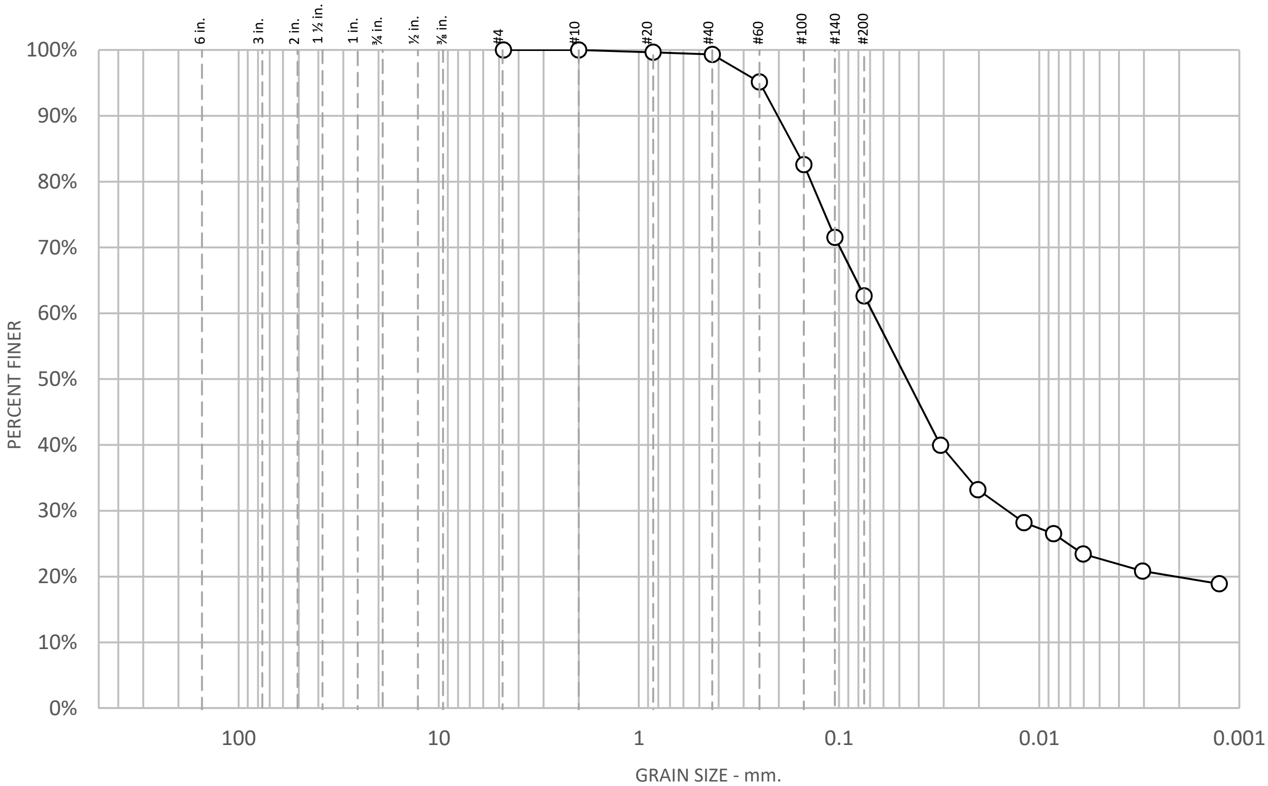
Client: Anil Verma Associates, Inc. **Project Number:** 16370.000.000
Project: City of Goleta Design for Train Station **Date:** 8/30/2019
Project location: Goleta, CA

ENGEO
Expect Excellence

Tested By: L. Santo Domingo **Checked By:** M. Quasem

Test Location: 3420 Fostoria Way, Suite E, Danville, CA 94526

Particle Size Distribution Report



% +75mm	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
				0.7	36.7	42.7	19.9

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#4	100.0		
#10	100.0		
#20	99.7		
#40	99.3		
#60	95.1		
#100	82.6		
#140	71.5		
#200	62.6		
0.0311 mm.	39.9		
0.0202 mm.	33.2		
0.0119 mm.	28.2		
0.0085 mm.	26.5		
0.0060 mm.	23.4		
0.0030 mm.	20.8		
0.0013 mm.	18.9		

* (no specification provided)

Soil Description

See exploration logs

Atterberg Limits

PL = 16 LL = 31 PI = 15

Coefficients

D_{90} = 0.2028 mm D_{85} = 0.1655 mm D_{60} = 0.0677 mm
D_{50} = 0.0460 mm D_{30} = 0.0144 mm D_{15} =
D_{10} = C_u = C_c =

Classification

USCS = CL

Remarks

GS: ASTM D422 ASTM D422
Silt/clay division of 0.002mm used
PI: ASTM D4318, Wet Method
USCS: ASTM D2487

Sample Number: 1-B2 @ 6

Client: Anil Verma Associates, Inc. **Project Number:** 16370.000.000
Project: City of Goleta Design for Train Station **Date:** 8/30/2019
Project location: Goleta, CA

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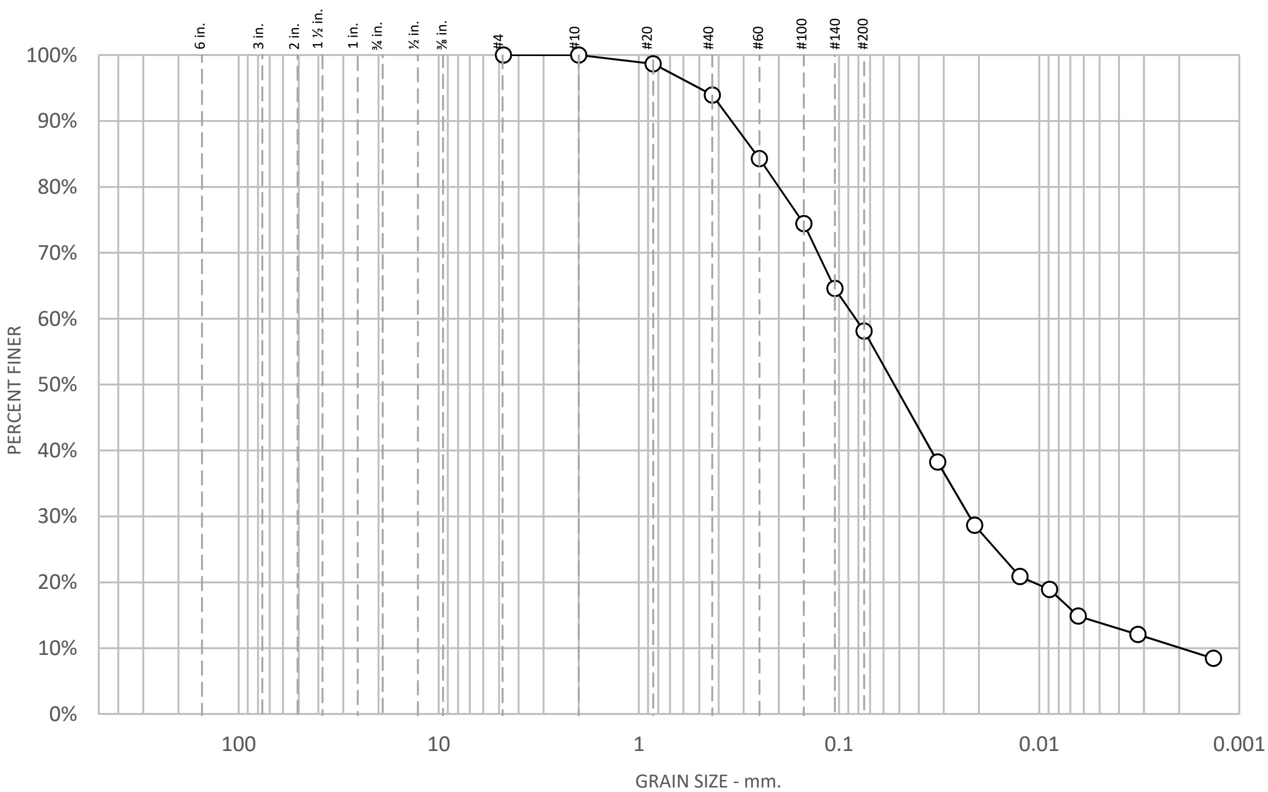
Tested By: L. Santo Domingo **Checked By:** M. Quasem

Test Location: 3420 Fostoria Way, Suite E, Danville, CA 94526

Particle Size Distribution Report



% +75mm	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
				6.1	35.8	48.0	10.1

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#4	100.0		
#10	100.0		
#20	98.7		
#40	93.9		
#60	84.3		
#100	74.4		
#140	64.6		
#200	58.1		
0.0322 mm.	38.2		
0.0210 mm.	28.7		
0.0125 mm.	20.9		
0.0089 mm.	18.9		
0.0064 mm.	14.9		
0.0032 mm.	12.1		
0.0013 mm.	8.5		

* (no specification provided)

Soil Description

See exploration logs

Atterberg Limits

PL = 14 LL = 22 PI = 8

Coefficients

D_{90} = 0.3447 mm D_{85} = 0.2604 mm D_{60} = 0.0827 mm
D_{50} = 0.0531 mm D_{30} = 0.0223 mm D_{15} = 0.0065 mm
D_{10} = 0.0020 mm C_u = 42.38 C_c = 3.09

Classification

USCS = CL

Remarks

GS: ASTM D422 ASTM D422
Silt/clay division of 0.002mm used
PI: ASTM D4318, Wet Method
USCS: ASTM D2487

Sample Number: 1-B3 @ 1-3

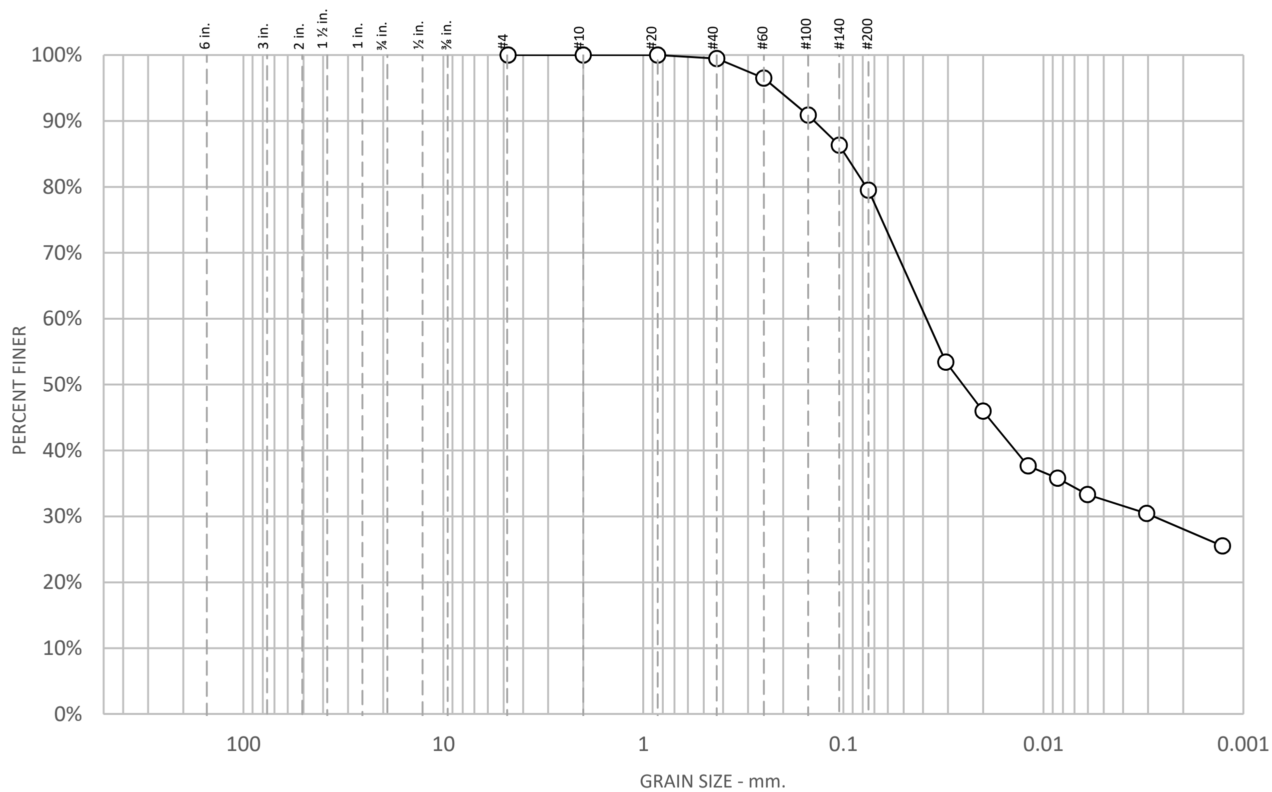
Client: Anil Verma Associates, Inc. **Project Number:** 16370.000.000
Project: City of Goleta Design for Train Station **Date:** 8/30/2019
Project location: Goleta, CA

ENGEO Expect Excellence

Tested By: L. Santo Domingo **Checked By:** M. Quasem

Test Location: 3420 Fostoria Way, Suite E, Danville, CA 94526

Particle Size Distribution Report



% +75mm	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
				0.5	20.0	51.4	28.1

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#4	100.0		
#10	100.0		
#20	100.0		
#40	99.5		
#60	96.5		
#100	90.9		
#140	86.3		
#200	79.5		
0.0307 mm.	53.4		
0.0200 mm.	46.0		
0.0119 mm.	37.7		
0.0085 mm.	35.8		
0.0060 mm.	33.3		
0.0030 mm.	30.5		
0.0013 mm.	25.5		

* (no specification provided)

Soil Description

See exploration logs

Atterberg Limits

PL = 19 LL = 50 PI = 31

Coefficients

D_{90} = 0.1400 mm D_{85} = 0.0984 mm D_{60} = 0.0385 mm
D_{50} = 0.0253 mm D_{30} = 0.0028 mm D_{15} =
D_{10} = C_u = C_c =

Classification

USCS = CL

Remarks

GS: ASTM D422 ASTM D422
Silt/clay division of 0.002mm used
PI: ASTM D4318, Wet Method
USCS: ASTM D2487

Sample Number: 1-B3 @ 3.5

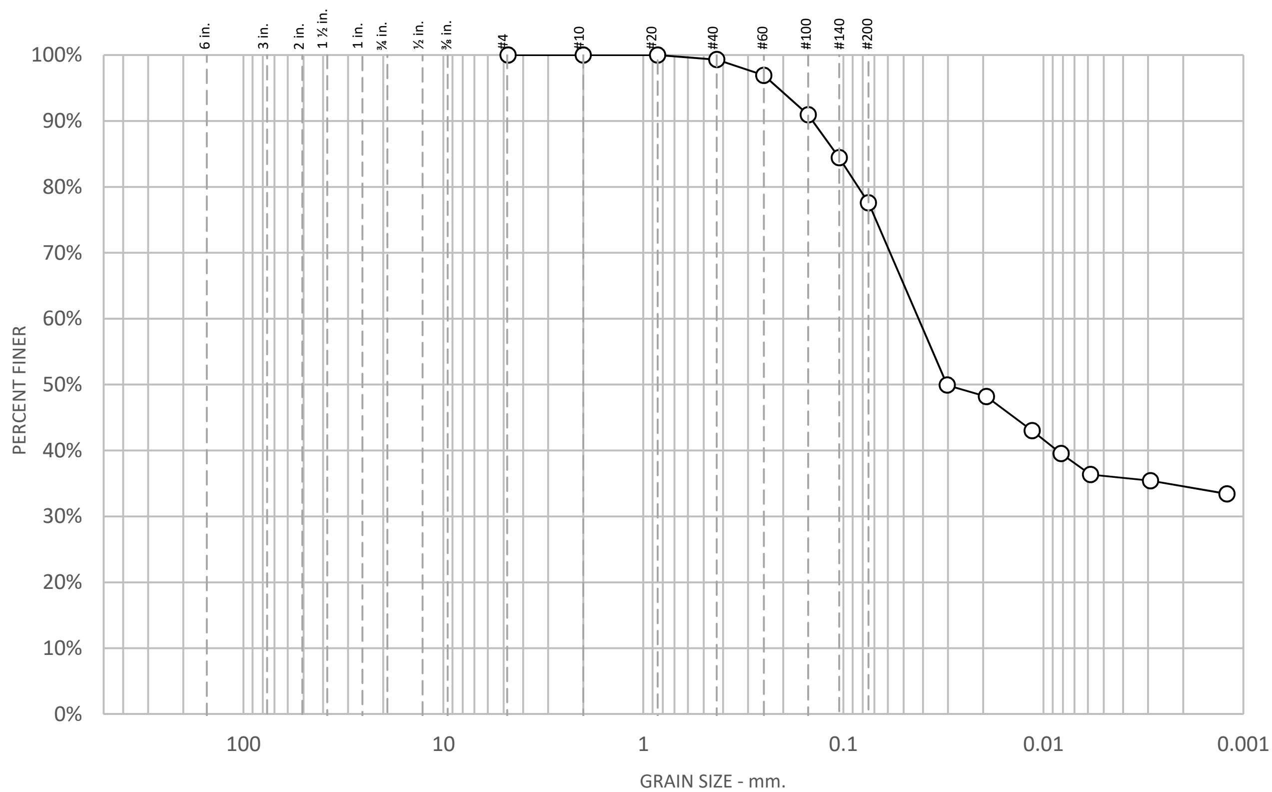
Client: Anil Verma Associates, Inc. **Project Number:** 16370.000.000
Project: City of Goleta Design for Train Station **Date:** 8/30/2019
Project location: Goleta, CA

ENGEO Expect Excellence

Tested By: L. Santo Domingo **Checked By:** M. Quasem

Test Location: 3420 Fostoria Way, Suite E, Danville, CA 94526

Particle Size Distribution Report



% +75mm	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
				0.7	21.7	43.0	34.6

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#4	100.0		
#10	100.0		
#20	100.0		
#40	99.3		
#60	96.9		
#100	90.9		
#140	84.4		
#200	77.6		
0.0303 mm.	49.9		
0.0193 mm.	48.2		
0.0114 mm.	43.0		
0.0082 mm.	39.5		
0.0058 mm.	36.3		
0.0029 mm.	35.4		
0.0012 mm.	33.4		

* (no specification provided)

Soil Description

See exploration logs

Atterberg Limits

PL = 16 LL = 35 PI = 19

Coefficients

D_{90} = 0.1425 mm D_{85} = 0.1084 mm D_{60} = 0.0421 mm
D_{50} = 0.0303 mm D_{30} = D_{15} =
D_{10} = C_u = C_c =

Classification

USCS = CL

Remarks

GS: ASTM D422 ASTM D422
Silt/clay division of 0.002mm used
PI: ASTM D4318, Wet Method
USCS: ASTM D2487

Sample Number: 1-B3 @ 8.5

Client: Anil Verma Associates, Inc. **Project Number:** 16370.000.000
Project: City of Goleta Design for Train Station **Date:** 8/30/2019
Project location: Goleta, CA

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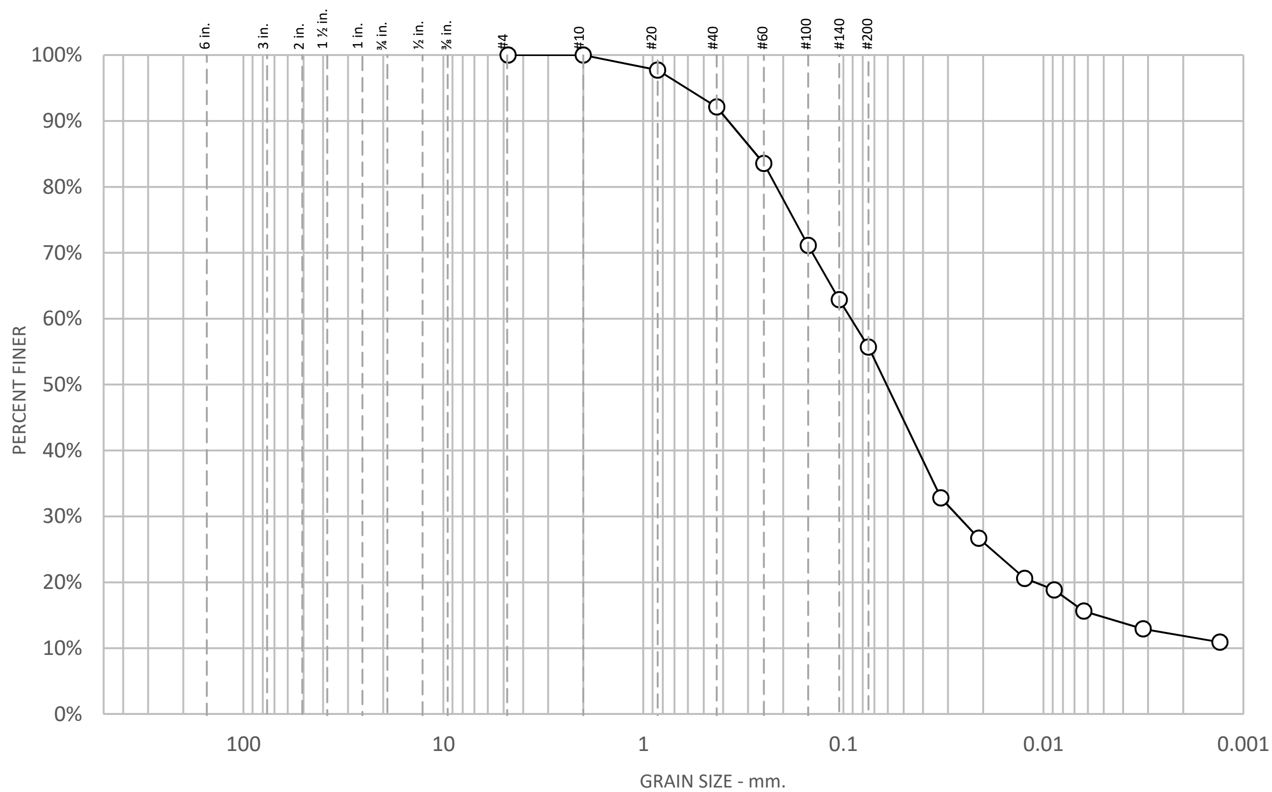
Tested By: L. Santo Domingo **Checked By:** M. Quasem

Test Location: 3420 Fostoria Way, Suite E, Danville, CA 94526

Particle Size Distribution Report



% +75mm	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
				7.9	36.4	43.8	11.9

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#4	100.0		
#10	100.0		
#20	97.7		
#40	92.1		
#60	83.5		
#100	71.1		
#140	62.9		
#200	55.7		
0.0326 mm.	32.8		
0.0211 mm.	26.7		
0.0124 mm.	20.6		
0.0088 mm.	18.8		
0.0063 mm.	15.6		
0.0032 mm.	12.9		
0.0013 mm.	10.9		

* (no specification provided)

Soil Description

See exploration logs

Atterberg Limits

PL = 16 LL = 18 PI = 2

Coefficients

D_{90} = 0.3761 mm D_{85} = 0.2742 mm D_{60} = 0.0917 mm
D_{50} = 0.0609 mm D_{30} = 0.0267 mm D_{15} = 0.0054 mm
D_{10} = C_u = C_c =

Classification

USCS = CL

Remarks

GS: ASTM D422 ASTM D422
Silt/clay division of 0.002mm used
PI: ASTM D4318, Wet Method
USCS: ASTM D2487

Sample Number: 1-B4 @ 3.5

Client: Anil Verma Associates, Inc. **Project Number:** 16370.000.000
Project: City of Goleta Design for Train Station **Date:** 8/30/2019
Project location: Goleta, CA

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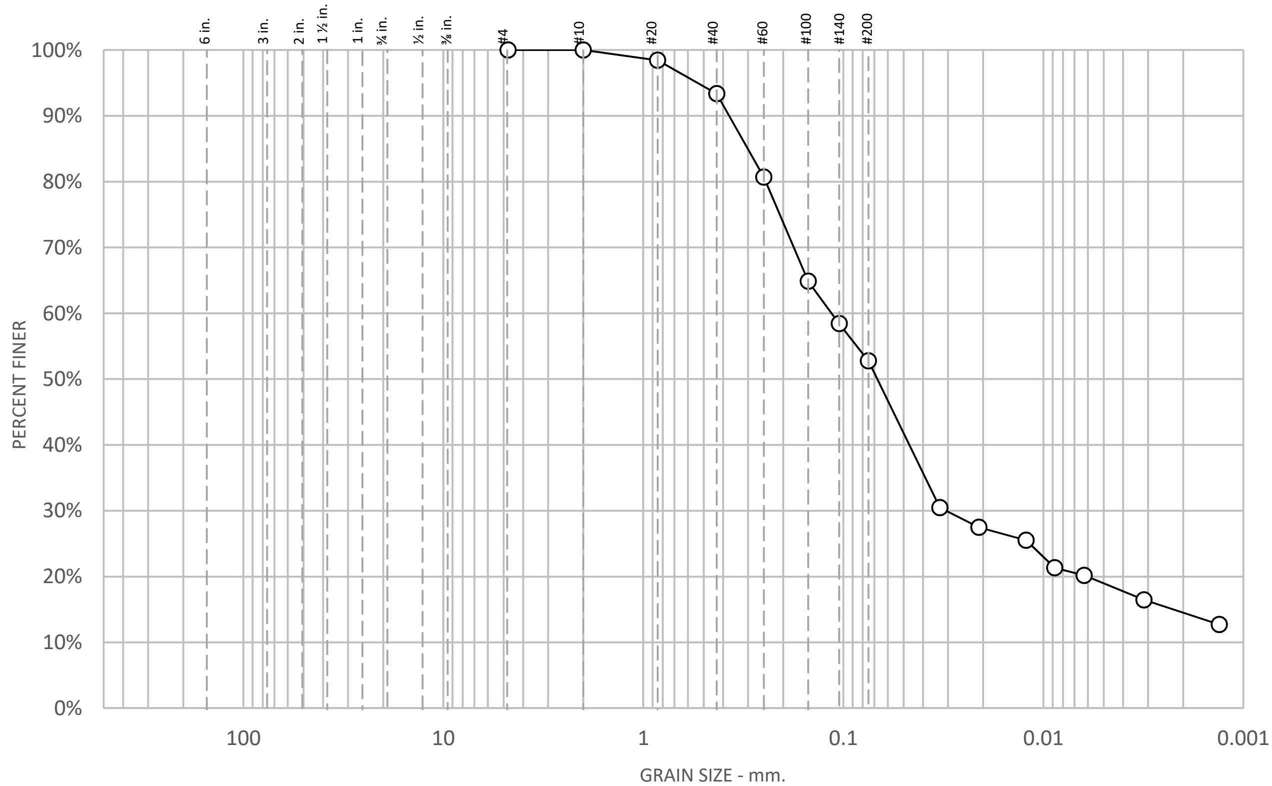
Tested By: L. Santo Domingo **Checked By:** M. Quasem

Test Location: 3420 Fostoria Way, Suite E, Danville, CA 94526

Particle Size Distribution Report



% +75mm	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
				6.6	40.6	38.3	14.5

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#4	100.0		
#10	100.0		
#20	98.4		
#40	93.4		
#60	80.7		
#100	64.9		
#140	58.4		
#200	52.8		
0.0329 mm.	30.5		
0.0210 mm.	27.5		
0.0122 mm.	25.5		
0.0088 mm.	21.3		
0.0063 mm.	20.2		
0.0031 mm.	16.5		
0.0013 mm.	12.7		

* (no specification provided)

Soil Description

See exploration logs

Atterberg Limits

PL = 18 LL = 26 PI = 8

Coefficients

D_{90} = 0.3724 mm D_{85} = 0.3008 mm D_{60} = 0.1146 mm
D_{50} = 0.0677 mm D_{30} = 0.0307 mm D_{15} = 0.0022 mm
D_{10} = C_u = C_c =

Classification

USCS = CL

Remarks

GS: ASTM D422 ASTM D422
Silt/clay division of 0.002mm used
PI: ASTM D4318, Wet Method
USCS: ASTM D2487

Sample Number: P2 @ 1-3

Client: Anil Verma Associates, Inc. **Project Number:** 16370.000.000
Project: City of Goleta Design for Train Station **Date:** 8/30/2019
Project location: Goleta, CA

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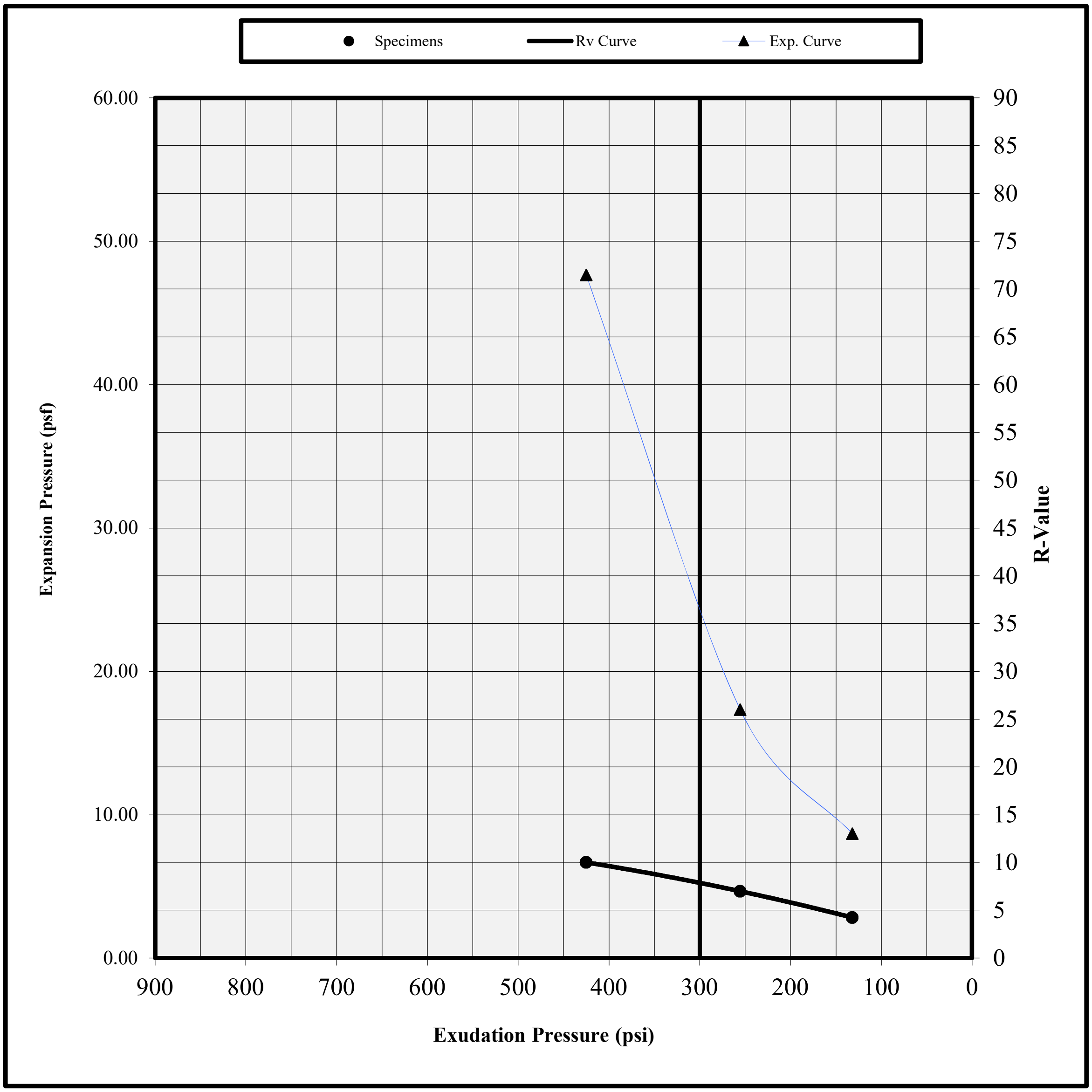
Tested By: L. Santo Domingo **Checked By:** M. Quasem

Test Location: 3420 Fostoria Way, Suite E, Danville, CA 94526

R VALUE TEST REPORT
CTM-301



Sample ID/Location: 1-B1
Description: See Exploration Logs
Test remarks:

Specimen	Specimen 1	Specimen 2	Specimen 3
Exudation Pressure (p.s.i.)	425	255	132
Expansion dial (0.0001")	11	4	2
Expansion Pressure (p.s.f.)	48	17	9
Resistance Value, "R"	10	7	4
% Moisture at Test	13.3	14.7	16.9
Dry Density at Test, p.c.f.	119.3	114.7	110.4
"R" Value at Exudation Pressure of 300 psi.	7		
Expansion Pressure (psf) at Exudation Pressure of 300 psi.	25		

PROJECT NAME: City of Goleta Design for Train Station
DATE: 08/23/19
PROJECT NUMBER: 16370.000.000
CLIENT: Anil Verma Associates, Inc.
PHASE NUMBER: REIM

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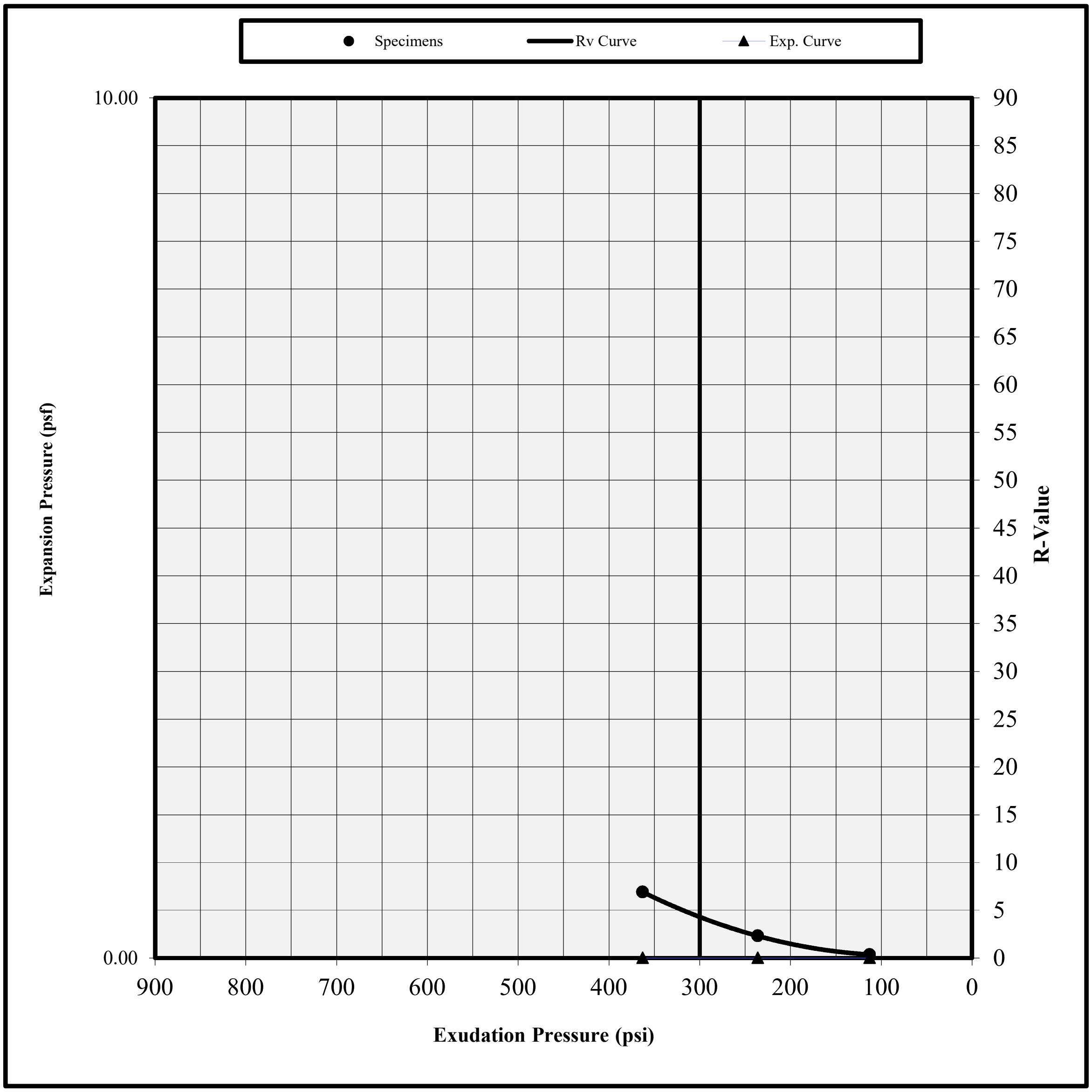
Tested by: W. Miller
Reviewed by: G. Criste

Lab Address : 3420 Fostoria Way, Suite E, Danville, CA 94526

R VALUE TEST REPORT
CTM-301



Sample ID/Location: 1-B4
Description: See Exploration Logs
Test remarks:

Specimen	Specimen 1	Specimen 2	Specimen 3
Exudation Pressure (p.s.i.)	363	236	113
Expansion dial (0.0001")	0	0	0
Expansion Pressure (p.s.f.)	0	0	0
Resistance Value, "R"	7	2	0
% Moisture at Test	10.8	15.0	16.9
Dry Density at Test, p.c.f.	119.9	113.8	112.6
"R" Value at Exudation Pressure of 300 psi.	Less Than 5		
Expansion Pressure (psf) at Exudation Pressure of 300 psi.	0		

PROJECT NAME: City of Goleta Design for Train Station
PROJECT NUMBER: 16370.000.000
CLIENT: Anil Verma Associates, Inc.
PHASE NUMBER: REIM

DATE: 08/24/19

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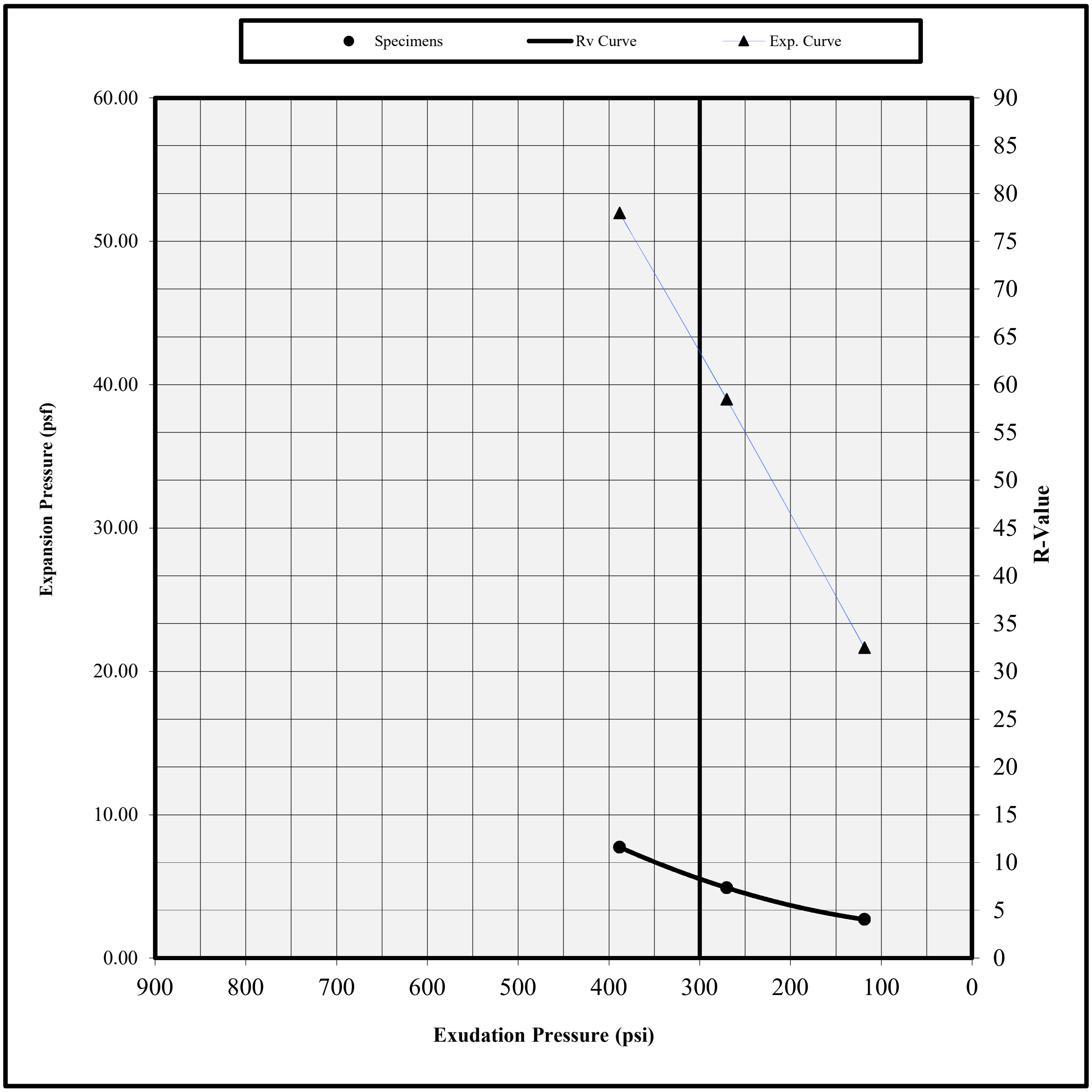
Tested by: W. Miller
Reviewed by: G. Criste

Lab Address : 3420 Fostoria Way, Suite E, Danville, CA 94526

R VALUE TEST REPORT
CTM-301



Sample ID/Location: 1-B5
Description: See Exploration Logs
Test remarks:

Specimen	Specimen 1	Specimen 2	Specimen 3
Exudation Pressure (p.s.i.)	388	270	119
Expansion dial (0.0001")	12	9	5
Expansion Pressure (p.s.f.)	52	39	22
Resistance Value, "R"	12	7	4
% Moisture at Test	15.5	17.4	18.9
Dry Density at Test, p.c.f.	112.6	109.3	106.6
"R" Value at Exudation Pressure of 300 psi.	8		
Expansion Pressure (psf) at Exudation Pressure of 300 psi.	43		

PROJECT NAME: City of Goleta Design for Train Station
PROJECT NUMBER: 16370.000.000
CLIENT: Anil Verma Associates, Inc.
PHASE NUMBER: REIM
DATE: 08/24/19

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Tested by: W. Miller
Reviewed by: G. Criste

Lab Address : 3420 Fostoria Way, Suite E, Danville, CA 94526

California State Certified Laboratory No. 2153

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

4 September, 2019

Job No. 1908138
Cust. No. 13096

Mr. Randy Hildebrant
ENGEO Inc.
2646 Santa Maria Way, Suite 107
Santa Maria, CA 93455

Subject: Project No.: 16370.000.000
Project Name: Goleta, CA
Corrosivity Analysis – ASTM Test Methods

Dear Mr. Hildebrant:

Pursuant to your request, CERCO Analytical has analyzed the soil samples submitted on August 19, 2019. Based on the analytical results, this brief corrosivity evaluation is enclosed for your consideration.

Based upon the resistivity measurements, Samples No.001 & No.002 are classified as "corrosive" and Sample No.003 is classified as "moderately corrosive". All buried iron, steel, cast iron, ductile iron, galvanized steel and dielectric coated steel or iron should be properly protected against corrosion depending upon the critical nature of the structure. All buried metallic pressure piping such as ductile iron firewater pipelines should be protected against corrosion.

The chloride ion concentrations reflect none detected with a reporting limit of 15 mg/kg.

The sulfate ion concentrations ranged from 25 to 43 mg/kg and are determined to be insufficient to damage reinforced concrete structures and cement mortar-coated steel at these locations.

The pH of the soils ranged from 7.71 to 8.85, which does not present corrosion problems for buried iron, steel, mortar-coated steel and reinforced concrete structures.

The redox potentials ranged from 180 to 210-mV. Sample No.001 is indicative of potentially "moderately corrosive" soils and the remaining samples are indicative of potentially "slightly corrosive" soils resulting from anaerobic soil conditions,

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

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

Very truly yours,
CERCO ANALYTICAL, INC.

for

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

JDH/jdl
Enclosure

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

Client:	ENGEO Incorporated
Client's Project No.:	16370.000.000
Client's Project Name:	Goleta, CA
Date Sampled:	13-Aug-19
Date Received:	19-Aug-19
Matrix:	Soil
Authorization:	Signed Chain of Custody

Date of Report: 4-Sep-2019

Job/Sample No.	Sample I.D.	Redox (mV)	pH	Conductivity (umhos/cm)	Resistivity (100% Saturation) (ohms-cm)	Sulfide (mg/kg)*	Chloride (mg/kg)*	Sulfate (mg/kg)*
1908138-001	1-B2 @ 1-3'	180	8.85	-	1,900	-	N.D.	30
1908138-002	1-B3 @ 1-3'	200	7.87	-	1,400	-	N.D.	43
1908138-003	1-B4 @ 1-3'	210	7.71	-	4,400	-	N.D.	25

Method:	ASTM D1498	ASTM D4972	ASTM D1125M	ASTM G57	ASTM D4658M	ASTM D4327	ASTM D4327
Reporting Limit:	-	-	10	-	50	15	15
	29-Aug-2019	29-Aug-2019	-	30-Aug-2019	-	3-Sep-2019	3-Sep-2019

Cheryl McMillen
Laboratory Director

* Results Reported on "As Received" Basis

N.D. - None Detected

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

Chain of Custody

Concord, CA 94520-1006
925 462 2771
Fax: 925 462 2775
CERCO analytical

Job No.	CU#	Client Project I.D.
1908138		16370.000.000

Schedule
Analyte

Date Sampled | Date Due

Full Name
Randy Hildebrant

Phone 805-338-4110 X
Fax
Cell

Company and/or Mailing Address
ENGEO, 2646 Suite 107, Santa Maria, CA 93455 ☒

Sample Source
Goleta, CA

ANALYSIS | ASTM

Lab No.	Sample I.D.	Date	Time	Matrix	Contain.	Size	Preserv.	Qty.	Redox Potential	pH	Sulfate	Chloride	Resistivity-100% Saturated			Brief Evaluation						
1	1-B2 @ 1-3 ft	8/13	11:30am	Soil		1 gal		1	x	x	x	x	x			X						
2	1-B3 @ 1-3 ft	8/13	9:00am	Soil		1 qt		1	X	X	X	X	X			X						
3	1-B4 @ 1-3 ft	8/13	11:00am	Soil		1 gal		1	X	X	X	X	X			X						

MATRIX
DW - Drinking Water
GW - Ground Water
SW - Surface Water
WW - Waste Water
Water
SL - Sludge
S - Soil
Product

ABBREVIATIONS
HB - Hosebib
PV - Petcock Valve
PT - Pressure Tank
PH - Pump House
RR - Restroom
GL - Glass
PL - Plastic
ST - Sterile

SAMPLE RECEIPT
Total No. of Containers
Rec'd Good Cond/Cold
Conforms to Record
Temp. a t Lab -°C
Sampler

		Date	Time
Relinquished By:		12/14/2019	9:00
Received By:		8/19/19	0930
Relinquished By:			
Received By:			
Relinquished By:			
Received By:			

Comments:

THERE IS AN ADDITIONAL CHARGE FOR EXTRUDING SOIL FROM METAL TUBES

Email Address: rhildebrant@engeo.com

APPENDIX C

PERCOLATION TEST DATA

Location 1-P1

Date 8/14/2019
Job # 16370.000.000
Job Name Goleta Train Station

Hole diameter (in) 5
Perf pipe diam (in) 4
Depth of hole (ft) 4.5
Gravel thickness ~2"

Saturated water level Surface
Saturation date & time 8/13/2019 12:00pm

Percolation Test Measurements

	Notes	Time	Depth to Water from Reference Point (ft)	Total Head (in)	Elapsed Time (min.)	Change in Water Level (ft)	Prec. Rate (m.p.i)
1	12" Starting head	8:53 AM	4.16	12			
2		9:42 AM	4.13	12.4	49	-0.03	-
3		10:03 AM	4.13	12.4	21	0	-
4		10:23 AM	4.12	12.5	20	-0.01	-
5		10:43 AM	4.12	12.5	20	0	-
6		11:03 AM	4.11	12.6	20	-0.01	-
7		11:23 AM	4.11	12.6	20	0	-
8		12:23 PM	4.1	12.7	60	-0.01	-
9		12:53 PM	4.08	13.0	30	-0.02	-
10		1:23 PM	4.08	13.0	30	0	-
11		2:00 PM	4.08	13.0	37	0	-
12		2:30 PM	4.07	13.1	30	-0.01	-
13		3:00 PM	4.07	13.1	30	0	-
14		3:30 PM	4.07	13.1	30	0	-
15		4:00 PM	4.07	13.1	30	0	-
16		4:30 PM	4.07	13.1	30	0	-
17		5:00 PM	4.07	13.1	30	0	-
18		5:30 PM	4.07	13.1	30	0	-
19	8/15/2019	8:24 AM	4.12	12.5	894	0.05	1490

Comments:

Water standing in hole, excess water removed to establish 12 inches of water at start of test

Location 1-P2

Date 8/14/2019
Job # 16370.000.000
Job Name Goleta Train Station

Hole diameter (in) 5
Perf pipe diam (in) 4
Depth of hole (ft) 4
Gravel thickness ~2"

Saturated water level Surface
Saturation date & time 8/13/2019 12:20pm

Percolation Test Measurements

	Notes	Time	Depth to Water from Reference Point (ft)	Total Head (in)	Elapsed Time (min.)	Change in Water Level (ft)	Prec. Rate (m.p.i)
1	12" Starting head	9:20 AM	4.01	12			
2		9:47 AM	3.95	12.72	27	-0.06	-
3		10:05 AM	3.95	12.72	18	0	-
4		10:32 AM	3.95	12.72	27	0	-
5		10:52 AM	3.95	12.72	20	0	-
6		11:12 AM	3.96	12.6	20	0.01	-
7		11:32 AM	3.96	12.6	20	0	-
8		12:31 PM	3.94	12.84	59	-0.02	-
9		1:01 PM	3.95	12.72	30	0.01	-
10		1:31 PM	3.94	12.84	30	-0.01	-
11		2:04 PM	3.94	12.84	33	0	-
12		2:34 PM	3.95	12.72	30	0.01	-
13		3:04 PM	3.94	12.84	30	-0.01	-
14		3:34 PM	3.95	12.72	30	0.01	-
15		4:04 PM	3.94	12.84	30	-0.01	-
16		4:34 PM	3.94	12.84	30	0	-
17		5:04 PM	3.94	12.84	30	0	-
18		5:34 PM	3.95	12.72	30	0.01	-
19	8/15/2019	8:41 AM	4.03	11.76	907	0.08	945

Comments:

Water standing in hole, excess water removed to establish 12 inches of water at start of test

ENGEO
Expect Excellence

Location 1-P3

Date 8/14/2019
Job # 16370.000.000
Job Name Goleta Train Station

Hole diameter (in) 5
Perf pipe diam (in) 4
Depth of hole (ft) 4.35
Gravel thickness ~2"

Saturated water level Surface
Saturation date & time 8/13/2019 12:10pm

Percolation Test Measurements

	Notes	Time	Depth to Water from Reference Point (ft)	Total Head (in)	Elapsed Time (min.)	Change in Water Level (ft)	Prec. Rate (m.p.i)
1	12" Starting head	8:23 AM	4.02	12			
2		9:39 AM	4.02	12	76	0	-
3		10:00 AM	4.04	11.76	21	0.02	87
4		10:28 AM	4.06	11.52	28	0.02	117
5		10:48 AM	4.06	11.52	20	0	-
6		11:08 AM	4.07	11.4	20	0.01	167
7		11:28 AM	4.07	11.4	20	0	-
8		12:28 PM	4.09	11.16	60	0.02	250
9		12:58 PM	4.09	11.16	30	0	-
10		1:28 PM	4.09	11.16	30	0	-
11		2:02 PM	4.1	11.04	34	0.01	283
12		2:32 PM	4.11	10.92	30	0.01	250
13		3:02 PM	4.11	10.92	30	0	-
14		3:32 PM	4.12	10.8	30	0.01	250
15		4:02 PM	4.12	10.8	30	0	-
16		4:32 PM	4.13	10.68	30	0.01	250
17		5:02 PM	4.13	10.68	30	0	-
18		5:32 PM	4.14	10.56	30	0.01	250
19	8/15/2019	7:49 AM	4.3	8.64	857	0.16	446

Comments:

Water standing in hole, excess water removed to establish 12 inches of water at start of test

ENGEO
Expect Excellence

Lot # 1-P4

Date 8/14/2019
Job # 16370.000.000
Job Name Goleta Train Station

Hole diameter (in) 7.5
Perf pipe diam (in) 4
Depth of hole (ft) 4.05
Gravel thickness ~2"

Saturated water level Surface
Saturation date & time 8/12/2019 4:00pm

Percolation Test Measurements

	Notes	Time	Depth to Water from Reference Point (ft)	Total Head (in)	Elapsed Time (min.)	Change in Water Level (ft)	Prec. Rate (m.p.i)
1	12" Starting head	9:34 AM	4.15	12			
2		9:54 AM	4.15	12	20	0	-
3		10:14 AM	4.17	11.76	20	0.02	83
4		10:37 AM	4.18	11.64	23	0.01	192
5		10:57 AM	4.19	11.52	20	0.01	167
6		11:17 AM	4.19	11.52	20	0	-
7		11:37 AM	4.2	11.4	20	0.01	167
8		12:34 PM	4.22	11.16	57	0.02	238
9		1:04 PM	4.24	10.92	30	0.02	125
10		1:34 PM	4.25	10.8	30	0.01	250
11		2:08 PM	4.27	10.56	34	0.02	142
12		2:38 PM	4.28	10.44	30	0.01	250
13		3:08 PM	4.28	10.44	30	0	-
14		3:38 PM	4.29	10.32	30	0.01	250
15		4:08 PM	4.3	10.2	30	0.01	250
16		4:38 PM	4.31	10.08	30	0.01	250
17		5:08 PM	4.32	9.96	30	0.01	250
18		5:38 PM	4.33	9.84	30	0.01	250
19	8/15/2019	9:13 AM	4.53	7.44	935	0.2	390

Comments:

ENGEO

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