APPENDIX F

GEOTECHNICAL INVESTIGATION

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MONTE VISTA MEMORIAL GARDENS
3656 LAS COLINAS ROAD
LIVERMORE, CALIFORNIA

GEOTECHNICAL EXPLORATION

SUBMITTED TO
Mike Kliment
Monte Vista Memorial Investment Group, LLC
189 Contractors Avenue
Livermore, CA 94551

PREPARED BY
ENGEO Incorporated

December 21, 2018

PROJECT NO.
15426.000.000

Expect Excellence

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GEOTECHNICAL
ENVIRONMENTAL
WATER RESOURCES
CONSTRUCTION SERVICES

Project No.
15426.000.000

December 21, 2018

Mr. Mike Kliment
Monte Vista Memorial Investment Group, LLC
189 Contractors Avenue
Livermore, CA 94551

Subject: Monte Vista Memorial Gardens
3656 Las Colinas Road
Livermore, California

GEOTECHNICAL EXPLORATION

Dear Mr. Kliment:

ENGEO prepared this geotechnical report for Monte Vista Memorial Investment Group as outlined in our agreement dated September 6, 2018. 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

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1.0 INTRODUCTION

1.1 PURPOSE AND SCOPE

ENGEO prepared this geotechnical report for design of a memorial cemetery in Livermore, California. We prepared this report as outlined in our agreement dated September 9, 2018. Memorial Investment Group authorized ENGEO to perform the following scope of services:

- Subsurface field exploration
- Soil laboratory testing
- Data analysis and conclusions
- Report preparation

For our use, we received a Preliminary Grading Plan prepared by ACS Consultant Engineer, dated May 5, 2018, delivered on September 23, 2018.

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.

1.2 PROJECT LOCATION

The subject site is located in Livermore, California, just north of Interstate 580 as shown on the Vicinity Map (Figure 1). The site is approximately 66 acres and is bisected by Arroyo Las Positas in the southeast, splitting the site into two areas. The site generally consists of a relatively flat lowland valley area in the southeast, with gently sloping hills and valleys to the north and west. The localized ridges and valleys are oriented roughly north-south in the northern portion of the property, and roughly east-west in the western portion of the property, with valleys draining toward Arroyo Las Positas. Site slope gradients range from 2.5:1 to 10:1 (horizontal:vertical) in the surrounding hills (with the steepest slopes in the southwest), and the lowland valley area has a slope gradient shallower than 25:1 (horizontal:vertical). Furthermore, the site is bordered by an existing residence to the east and private undeveloped grazing land to the west and north. Currently, the portion of the site to the east of Arroyo Las Positas contains paved roadways with no further development, and the area on the west side of the arroyo remains undeveloped.

Arroyo Las Positas, an existing creek running northeast-southwest, bisects the property. The creek is roughly 14 to 16 feet deeper than adjacent terrain with bank gradients that are generally 1.5:1 (horizontal:vertical) or flatter. The creek banks are well vegetated with grasses and shrubs. Recent bank erosion as well as some near-vertical creek bank sections were also observed along the creek. Refer to the Site Plan (Figure 2) for approximate locations of near-vertical creek bank sections along the creek and additional information.

The Site Plan (Figure 2) shows the boundaries of the project site and approximate locations of our explorations.

1.3 PROJECT DESCRIPTION

Based on our discussion with Monte Vista Memorial Investment Group, LLC and review of the proposed development plan (Figure 8) and information provided, we understand that the following general site improvements are proposed:

1. Earthwork cuts and fills of up to approximately 30 and 25 feet, respectively.

2. Two manmade lakes extending approximately 8 to 25 feet below planned future grade connected by a manmade creek.

3. Construction of a manmade island within a manmade lake in the southwestern portion of the site.

4. A two-story funeral home with an underground basement extending one level below grade (three stories total), and an adjacent one-story pavilion located on the eastern side of Arroyo Las Positas. For design purposes, we assumed the funeral home basement may extend up to 12 feet below existing grade.

5. Construction of two vehicular bridges crossing Arroyo Las Positas.

6. Construction of a pedestrian walkway connecting the manmade island to the lakeshore.

7. Paved streets, parking, and drive lanes.

8. Pedestrian walkways and boardwalks through a reconstructed wetland and connecting to the manmade island.

9. Solar trellis structures located in the southwestern portion of the site.

10. Utilities and other infrastructure improvements.

11. Retaining walls up to 25 feet in height, associated with a manmade waterfall, bridge abutments and office building basement.

12. Exterior concrete flatwork.

2.0 FINDINGS

2.1 FIELD EXPLORATION

Our field exploration included drilling 15 borings, excavating 8 test pits and advancing 6 Cone Penetration Test (CPT) soundings at various locations on the site. We performed our field exploration between October 5 and October 10, 2018. We also performed geologic field mapping concurrent with field exploration activities.

The location and elevations (NAVD88 Datum) of our explorations are approximate and were estimated by pacing from features shown on the site plan, they should be considered accurate only to the degree implied by the method used.

2.1.1 Borings

We observed drilling of 15 borings at the locations shown on the Site Plan, Figure 2. An ENGEO representative observed the drilling and logged the subsurface conditions at each location. We retained a truck-mounted drill rig and crew to advance 6 of the borings using 6-inch-diameter mud rotary drilling methods and retained a separate truck-mounted drill rig and crew to advance the remaining 9 borings using 6-inch-diameter solid flight auger drilling methods. The borings were advanced to depths ranging from 10 to 50 feet below existing grade. We permitted and backfilled the borings in accordance with the requirements of Zone 7 Water Agency.

We obtained bulk soil samples from drill cuttings and retrieved disturbed samples at various intervals in the borings using standard penetration tests and, 2½-inch O.D. split-spoon sampler.

The standard penetration resistance blow counts were obtained by an automatic trip hammer, which drops a 140-pound hammer through a 30-inch free fall. The samplers were driven 18 inches and the number of blows was 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 report logs in Appendix A. The logs depict subsurface conditions at the exploration locations for the date of exploration; however, subsurface conditions may vary with time.

2.1.2 Test Pits

We observed excavation of 8 test pits at the locations shown on the Site Plan, Figure 2. An ENGEO representative observed the test pit excavation and logged the subsurface conditions at each location. We retained a DS 480 backhoe to excavate the test pits using a 4-foot-wide bucket and logged the type, location, and uniformity of the underlying soil/rock. The maximum depth excavated by the test pits was approximately 15 feet.

We obtained bulk disturbed soil samples from the test pits using hand-sampling techniques. The test pit logs present descriptions and graphically depict the subsurface conditions encountered.

We used the field logs to develop the report logs in Appendix B. The logs depict subsurface conditions at the exploration locations for the date of exploration; however, subsurface conditions may vary with time.

2.1.3 Cone Penetration Tests

We retained a Cone Penetration Test (CPT) track rig to push the cone penetrometer to a maximum depth of about 50 feet. The CPT track rig has a 20-ton compression-type cone with a 15-square-centimeter (cm^2) base area, an apex angle of 60 degrees, and a friction sleeve with a surface area of 225 cm^2. The cone, connected with a series of rods, is pushed into the ground at a constant rate. Cone readings are taken at approximately 5-cm intervals with a penetration rate of 2 cm per second in accordance with ASTM D-5778. Measurements include the tip resistance to penetration of the cone (Qc), the resistance of the surface sleeve (Fs), and pore pressure (U) (Robertson and Campanella, 1988). CPT logs are presented in Appendix C.

The CPT subcontractor conducted pore pressure dissipation testing in 1-CPT3, 1-CPT4 and 1-CPT6. At these locations, the CPT cone was halted at select depths, and the variation of the penetration pore pressure with time was measured until the pore pressure stabilized. The pore water dissipation test was only able to successfully estimate the ground water elevation in 1-CPT4, while the other tests yielded inconclusive results. Results of the pore-pressure dissipation tests are included in Appendix C.

2.1.4 Geologic Field Mapping

During our field explorations, an ENGEO geologist observed the surface conditions and visible geologic features at the site. We mapped the geologic features and summarize our findings on the Site Plan, Figure 2.

2.2 SITE BACKGROUND

The property is currently vacant of any structures besides a small pump house on the east site of Arroyo Las Positas, and a gravel roadway which connects the property southeast of Arroyo Las Positas to Las Colinas Road. The remaining area of the site is covered with seasonal grass.

We reviewed historic aerial photographs using www.historicaerials.com, Google Earth, and aerials available through the University of California, Santa Barbara with various readily available aerial photographs spanning from 1940 to 2017. Based on these aerial photographs, the subject property appears to have been used primarily for agricultural purposes in the 1940s, with a single house located just to the east of the property along with several dirt roads in the surrounding region. Beginning in the 1950s, row crops disappeared from the site and the site was used primarily for grazing land. Construction of Interstate 580, located immediately south of the site, also began in the 1950s. In addition, an embankment was constructed along Las Colinas Road to facilitate construction of the Las Colinas Road overpass. Since construction of these improvements, there has been little change to the site, with the exception of stockpile placement and the installation of a well at the southeastern portion of the site in June 2012. Based on our review of historic aerial photographs covering the site, it appears that aside from construction activities near the site, the site itself has remained relatively unchanged.

Additionally, available historic topographic maps were reviewed. The historic topographic maps support our observations made during the aerial photograph review, illustrating a relatively unchanged surface with little to no change in elevation between 1906 and 2015.

2.3 GEOLOGY AND SEISMICITY

2.3.1 Geology

The site is situated within the Livermore Valley basin, approximately 5 miles west of the Altamont Pass. Based on our geologic mapping and subsurface explorations, as well as review of regional geologic maps, the site is underlain by young colluvial and alluvial deposits, as well as older Livermore Gravel deposits (Figure 3). Faulting in this area is common, and the region around the site has experienced folding and at least two uplift events. Bedding was observed to be shallowly dipping to relatively flat, however Barlock (1988) shows a roughly N70°W trending gently dipping anticline transecting the site in the northwest. This indicates that bedding is generally gently dipping to the southwest in the southwestern portion of the property and to the northeast in the northeastern portion of the property. Landslide mapping, by Nilsen (1975), indicates no landslides within the project boundaries – no hillside landslides were observed or encountered onsite during our geotechnical explorations.

Barlock (1988, 1989) describes the units onsite as Holocene alluvium, and late Miocene to early Pleistocene Livermore Gravels. Furthermore, Dibblee (1980) confirms both the unit geology, and the presence of regional folding onsite. We observed these units onsite, as well as Holocene colluvium and residual soil.

In general, the site is blanketed by roughly 2 to 6 feet of colluvium or residual soil, with the units thickening towards the lowland valley near Arroyo Las Positas. The colluvium was generally observed to be dark gray to dark olive brown silty to sandy fat clay (CH), with a medium to high plasticity. The colluvial deposits were generally observed to be very stiff to hard, and ranged from dry to moist. These deposits also displayed some weak- to well-developed partings and contained various concentrations of carbonate blebs and streamers, with some indications of paleosols occurring at depth.

The geologic unit underlying the surficial soils and alluvium at the site is interpreted as belonging to the Upper Livermore Gravels unit. The Upper Livermore Gravels is described by Barlock (1989) as, “composed predominantly of clasts of Franciscan graywacke, lithic sandstone, metamorphic rock, volcanic rock, and traces of fine-grained vein quartz. Thick, horizontally bedded, clast-supported, well imbricated, gravel beds interlayered with planar cross-bedded and trough cross-bedded sandstone intervals are typical. Indistinctly bedded, matrix-supported, cobble to boulder gravel occurs rarely. The Upper Livermore represents deposition by gravelly braided streams on an alluvial fan.” Based on the site geomorphology, as interpreted in stereo-paired aerial photographs and Google Earth, the flatlands immediately surrounding Arroyo Las Positas indicate the presence of paleo channels, terraces, and flood deposits. This is consistent with the regional geologic mapping which describes the alluvium as “unconsolidated sand and gravel, recent terrace deposits, stream deposits, and cemented fanglomerate” (Barlock, 1988). In addition, we encountered a very fine-grained silty sand overlying the colluvium in the southeastern portion of the property on the north bank of Arroyo Las Positas, which we have interpreted as relatively recent flood deposits.

In general, the observed site geology consisted of light-gray to dark yellowish- to dark reddish-brown massive fine- to coarse-grained sand, silt, and clay units, with plus-or-minus well-rounded gravels of varying size. These sand, silt, and clay units are interlain by clast-supported well-imbricated conglomerate (or gravel) beds. The sand and clay units vary from having a low to high expansion potential, and are generally medium stiff to hard clays, and loose to very dense sands. Moisture-content increases with depth from dry to wet. The provenance of the observed units appears to be Franciscan derived. These observations are consistent with the previously mapped regional geologic units and interpretations of Barlock and Dibblee (1988/1989; 1980).

For simplicity, we have used our aerial photo interpretations and field observations to create a geologic map of the site (Figure 2). However, the Upper Livermore Gravels, which are interpreted as having been deposited in a braided stream environment, are overlain by younger alluvial deposits. This basically means that the older alluvial deposits of the Upper Livermore Gravels blend into the younger alluvial deposits above them. As a result, we have labelled areas as being alluvium or Upper Livermore Gravels on the geologic map. In general, it is believed that the alluvium (Qal) is at least 1 to 2 feet thick where noted on the site geology (Figure 2).

2.3.2 Seismicity

The Livermore area contains numerous active earthquake faults. Nearby active faults include the Greenville, Mount Diablo Thrust, Calaveras, Great Valley, Hayward, and the Green Valley. 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).

Numerous small earthquakes occur every year in the San Francisco Bay Region, and larger earthquakes have been recorded and can be expected to occur in the future. Figure 5 shows the approximate locations of these faults and significant historic earthquakes recorded within the San Francisco Bay Region.

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 the 2008 National Hazards Program the nearest active fault is the Greenville Connected, which is mapped approximately 3.3 miles east of the site. This fault is considered capable of a moment magnitude earthquake of 7.0. Other active faults in the region are summarized in the table below and include the Mount Diablo Thrust fault approximately 4.0 miles away, capable of a moment magnitude of 6.8 and the Calaveras fault approximately 9.0 miles away capable of a moment magnitude of 6.8.

TABLE 2.3.2-1: Active Faults Capable of Producing Significant Ground Shaking at the Site

FAULT NAME	DISTANCE FROM SITE (MILES)	DIRECTION FROM SITE	MAXIMUM MOMENT MAGNITUDE
Greenville Connected	3.3	East	7.0
Mount Diablo Thrust	4.0	Northwest	6.5
Calaveras	9.0	West	7.0
Great Valley	13.0	East	6.9
Hayward	14.9	West	7.3
Green Valley Connected	18.6	Northwest	6.8

The third version of Uniform California Earthquake Forecast (UCERF3) developed by the Working Group on California Earthquake Probabilities (Field et al., 2013) provides updated estimates of the 30-year probability of various magnitudes earthquakes in the San Francisco Bays Area. The results of the study are summarized in the following table:

TABLE 2.3.2-2: 30-Year Probability of Earthquake in the Bay Area

EARTHQUAKE MAGNITUDE	30-YEAR PROBABILITY OF ONE OR MORE EVENTS
5 or Greater	100%
6 or Greater	98%
7 or Greater	51%
8 or Greater	4%

The state of California Seismic Hazard Zones map by California Geologic Survey maps the area around the creek as lying within a potential liquefaction hazard zone (Figure 4). Witter (2006) also maps the alluvial deposits, near Arroyo Las Positas, as having moderate liquefaction susceptibility. The evaluation of liquefaction hazards are provided later in this report. In addition, the state of California Seismic Hazard Zones map by California Geologic Survey maps the hillside areas located at the northern and western portions of the site as earthquake-induced landslide zones (Figure 4). The evaluation of hazards are provided later in this report.

2.4 SURFACE CONDITIONS

As described earlier, the site is bisected by Arroyo Las Positas. The area on the west side of the arroyo, which will comprise the cemetery and a majority of the improvement is undeveloped and is currently used as grazing land. This portion of the site contains areas of dilapidated fences and is bordered by sloping hillsides along the west and north and I-580 to the south. In general, these hillsides are gradually sloping, with slopes ranging from 4:1 to 10:1 (horizontal:vertical), however steeper areas are located in the southwest portion of the site with slopes inclined at a maximum of approximately 2.5:1. The site elevations on the west side of the arroyo range from 662 feet (NAVD88) along the north and west borders, to 491 feet near the arroyo.

On the east side of Arroyo Las Positas, the site is relatively flat with site grades ranging from 490 feet near the arroyo to 493 feet at the eastern border of the property. The entrance to the site lies on a fill slope with an approximate slope gradient of 2.5:1 (horizontal:vertical). Additionally, this side of the property contains minor improvements such as a gravel road, and existing electrical lines and water lines. Furthermore, tilling and disturbed soils were observed at the surface within the project site east of the arroyo.

2.5 SUBSURFACE CONDITIONS

Overall, soils found at the site generally consist of interbedded layers of fine- and coarse-grained material associated with alluvial deposits and the Livermore Gravel Formation. In general, the upper approximately 2 to 10 feet consisted of predominately medium to high plasticity clay with moderate to high expansive potential except at 1-B13, which consisted of very dense sand in the upper 15 feet underlain by stiff lean clay. An approximately 5- to 10-foot-thick layer of generally medium dense to very dense coarse-grained material consisting of clayey sand, clayey gravel, silty sand, sand, and gravel underlies the surficial clay layer. Below this granular layer lies hard lean clay and silty clay with varying amounts of sand and gravel representative of the Livermore Gravel Formation that extended throughout the remainder of the borings.

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 groundwater in several of our subsurface boring explorations. Due to the mud rotary drilling methods, we did not measure groundwater in select boring locations. We summarize our observations in the table below:

TABLE 2.6-1: Groundwater Observations

EXPLORATION LOCATION	APPROX. DEPTH TO GROUNDWATER (FEET)	APPROX. GROUNDWATER ELEVATION (FEET)
1-B2	14	476
1-B3	15	476
1-B6	16	471
1-B14	14	477
1-B8	5	487
1-B9	14	478

We performed pore-pressure-dissipation tests at select CPT locations. We calculated the groundwater elevation at each location based on the pore pressure dissipation test results. The table below provides a summary of the calculated groundwater depth and elevation at the CPT locations.

TABLE 2.6-2: Groundwater Elevation Based on Pore Pressure Dissipation Tests

CPT LOCATION	DEPTH OF CONE (FEET)	MEASURED PORE PRESSURE (PSI)	CALCULATED GROUNDWATER DEPTH (FEET)	CALCULATED GROUNDWATER ELEVATION* (FEET)
1-CPT4	44.4	11.5	34.3	491

**Elevation Datum NAVD88*

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

2.7 LABORATORY TESTING

We performed laboratory tests on selected soil samples to evaluate their engineering properties. For this project, we performed moisture content, dry density, strength testing, plasticity index, hydrometer, resistance value, compaction, permeability sulfate and soil corrosion potential testing. Moisture contents and dry densities are recorded on the boring logs in Appendix A; other laboratory data is included in Appendix B.

2.8 LABORATORY TESTING

As previously discussed, two manmade lakes are planned to be constructed onsite. Through conversations with Monte Vista Memorial Investment Group, we understand the lake liner will be constructed with native clay onsite. We performed hydraulic conductivity testing on representative in-situ liner and remolded bulk soil samples that we collected on the site to evaluate the use of onsite soil for this purpose. The bulk soil sample was prepared to 95 percent relative compaction (ASTM 1557) prior to performing the hydraulic conductivity test. The results of the hydraulic conductivity tests are summarized below.

TABLE 2.8.-1: Laboratory Hydraulic Conductivity Test Results (ASTM D5084, Method C)

FACTOR OF SAFETY				
LOCATION	SOIL CLASSIFICATION	REMOLDED RELATIVE PERCENT COMPACTION	HYDRAULIC CONDUCTIVITY (CM/S) TEST RUN 1	HYDRAULIC CONDUCTIVITY (CM/S) TEST RUN 2
1-B5 @ 38 feet	Lean Clay	---	1.37×10^{-6}	1.17×10^{-6}
TP-4 @ 11-12 feet	Lean Clay	95%	2.09×10^{-5}	2.09×10^{-5}

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 and specifications.

The primary geotechnical concerns that could affect development on the site are presence of expansive soil, seismic ground motions, liquefaction settlement and shallow groundwater. We summarize our conclusions below.

3.1 EXPANSIVE SOIL

We observed potentially expansive clay near the surface of the site at a majority of exploration locations with exception to Boring 1-B13. Our laboratory testing indicates that these soils exhibit moderate 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 and/or (3) using a layer of select fill below building locations.

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. Expansive soil mitigation recommendations are presented in Section 5.2 of this report.

3.2 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, soil liquefaction, lateral spreading, and landslides. 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, tsunamis, ground lurching or seiches is considered low to negligible at the site.

3.2.1 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.

3.2.2 Ground Shaking

An earthquake of moderate to high magnitude generated within the San Francisco Bay 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 latest 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).

3.2.3 2016 California Building Code (CBC) Seismic Design Parameters

The 2016 CBC utilizes design criteria set forth in the 2010 ASCE 7 Standard. Based on the subsurface conditions encountered, we characterized the site as Site Class D in accordance with the 2016 CBC. We provide the 2016 CBC seismic design parameters in Table 3.2.3-1 below, which include design spectral response acceleration parameters based on the mapped Risk-Targeted Maximum Considered Earthquake (MCER) spectral response acceleration parameters.

TABLE 3.2.3-1: 2016 CBC Seismic Design Parameters, Latitude: 37.70394 **Longitude:** -121.75845

PARAMETER	VALUE
Site Class	D
Mapped MCE_R Spectral Response Acceleration at Short Periods, S_S (g)	1.65
Mapped MCE_R Spectral Response Acceleration at 1-second Period, S_1 (g)	0.61
Site Coefficient, F_A	1.00
Site Coefficient, F_V	1.50
MCE_R Spectral Response Acceleration at Short Periods, S_{MS} (g)	1.65
MCE_R Spectral Response Acceleration at 1-second Period, S_{M1} (g)	0.92
Design Spectral Response Acceleration at Short Periods, S_{DS} (g)	1.10
Design Spectral Response Acceleration at 1-second Period, S_{D1} (g)	0.61
Mapped MCE Geometric Mean (MCE_G) Peak Ground Acceleration, PGA (g)	0.63
Site Coefficient, F_{PGA}	1.00
MCE_G Peak Ground Acceleration adjusted for Site Class effects, PGA_M (g)	0.63

3.2.4 Liquefaction

Soil liquefaction results from loss of strength during cyclic loading, such as imposed by earthquakes. Soils most susceptible to liquefaction are clean, loose, saturated, uniformly graded, fine-grained sands below the groundwater table. Empirical evidence indicate that low plasticity silt and clay are also potentially liquefiable, though this phenomenon is commonly referred to as cyclic softening. For the purpose of this report, we will refer to cyclic softening as liquefaction. When seismic ground shaking occurs, the soil is subjected to cyclic shear stresses that can cause excess hydrostatic pressure to develop.

As previously discussed, the subsurface soils consist of mostly clay and silty clay, with interbedded layers of silty sand, sandy silt, and poorly graded sand. We used visual classification, in-situ dilatancy test, and index testing results from the boring soil samples in conjunction with the Bray and Sancio (2006) screening criteria to determine which of the samples of fine-grained soils from the borings may be liquefiable. We also used these data to establish a relationship between soil that is identified as potentially liquefiable in the CPTs by comparing them to nearby borings. To perform this comparison, we compared the calculated Soil Behavior Type Index (I_C) to soil zones that were considered potentially liquefiable in the adjacent borings. This comparison allows us to calibrate the results of CPT testing at this site with soil behavior recovered from our borings. The following nearby borings and CPTs were used to perform these calibrations.

Nearby Boring and CPT Pairs

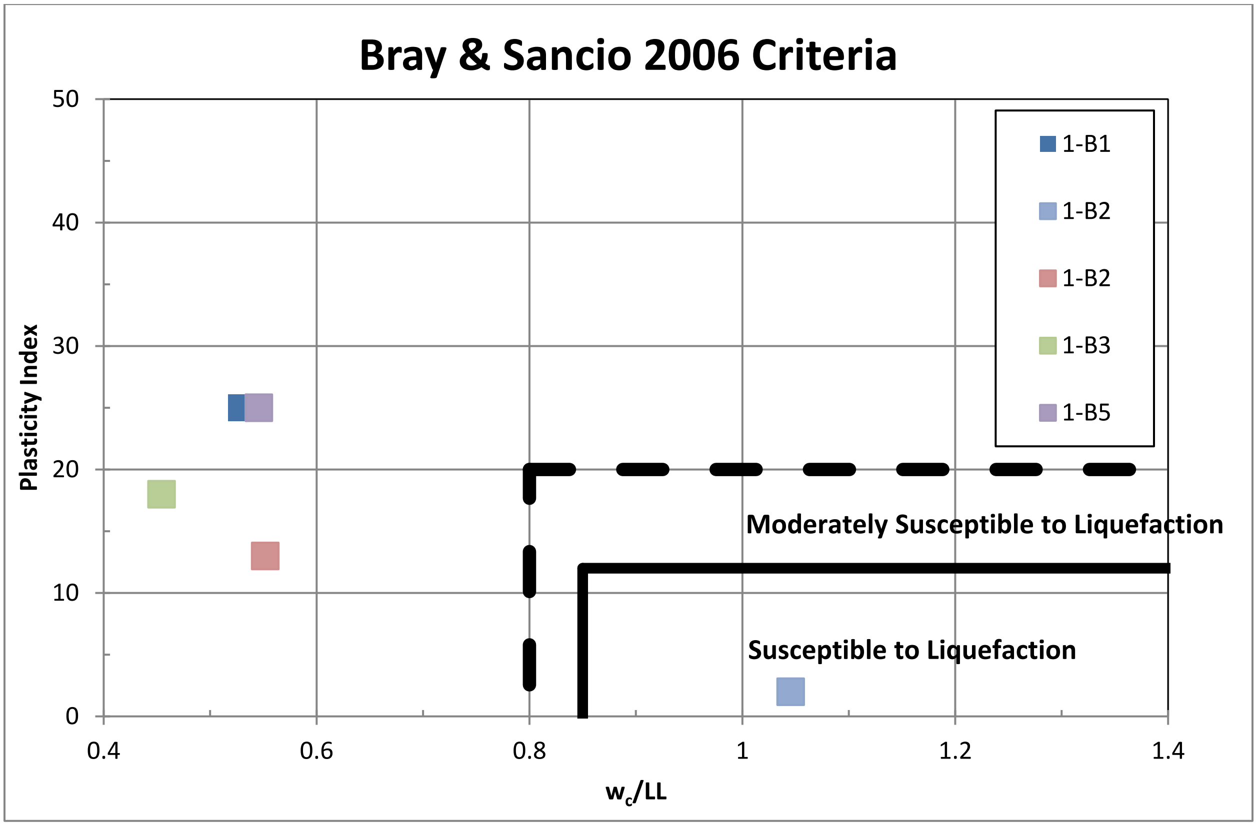
Pair 1: 1-B1 and 1-CPT5
Pair 2: 1-B2 and 1-CPT3
Pair 3: 1-B3 and 1-CPT2
Pair 4: 1-B5 and 1-CPT4

Four soil samples, were plotted well outside the limits of susceptibility to liquefaction according to the Bray and Sancio procedure (Chart 3.2.4-1), and had a soil behavior index (I_c) ranging from of 2.25 to 2.44, as shown in Table 3.2.4-1. Based on this screening (Bray and Sancio, 2006) we established an I_c cutoff value of 2.30 in areas where there is no adjacent boring or lab testing, as the boundary where liquefaction will not occur in clay-like soils at the site. This value represents the I_c value that plasticity index and fines content testing indicate that are clay, which is not susceptible to liquefaction.

TABLE 3.2.4-1: "Clay-like" Soil Samples

BORING	SAMPLE DEPTH (FEET)	I_c
1-B1	20	2.40
1-B2	16.5	2.20
1-B2	28	2.44
1-B3	16	2.29
1-B5	28	2.25

CHART 3.2.4-1: Bray and Sancio (2006) Screening of I_C > 2.2 Soils



We evaluated the data from CPTs for triggering of liquefaction using the calibrated I_C value to represent transitions in soil type and behavior. In performing our analysis, we assumed a design groundwater level of 10 feet below existing grade across the site. Furthermore, we used the mapped maximum considered earthquake (MCE) geometric mean peak ground acceleration (PGA_M) of 0.63g based on the 2016 California Building Code. We assumed a moment magnitude of 7.0 for our analyses to represent the highest level of ground shaking on the controlling faults. As discussed earlier, we also used an I_C cut-off of 2.30, based on our site-specific data, for areas where there was no adjacent soil boring or laboratory tests.

We utilized the software package, CLiq Version 2.2.1.4 by Geologismiki Geotechnical Software, to evaluate liquefaction susceptibility from the CPT data. We performed our analyses using the method outlined by Robertson (2009). Based off the I_c analysis, we omitted non-liquefiable layers from the above Cliq analysis. Additionally, we estimated liquefaction settlement in granular materials at boring locations where there was no accompanying CPT using Standard Penetration (SPT) blow counts, converted Modified California sampler blow counts and the above seismic input parameters as outlined by Idriss and Boulanger (2008). Final estimated liquefaction-induced settlements are summarized below.

TABLE 3.2.4-2: Summary of Liquefaction-Induced Settlement Calculations

EXPLORATION LOCATION	TOTAL SETTLEMENT (INCHES)
1-CPT1	0.1
1-CPT2/1-B3	0.2
1-CPT3/1-B2	1.3
1-CPT4/1-B5	1.0
1-CPT5/1-B1	0.5
1-CPT6	0.6
1-B4	0.0
1-B6	0.6
1-B7	2.5
1-B8	0.2
1-B11	1.0
1-B12	0.5

The estimated liquefaction-induced settlement in the overall site area is up to 2.5 inches; however, this is an isolated area located in the eastern portion of the site, near the entrance. If restricted to areas where vertical structures and bridges are to be constructed, estimated liquefaction-induced settlement is a maximum of 1.3 inches.

3.2.5 Liquefaction-Induced Surface Rupture

In order for liquefaction-induced ground failure to occur, the pore water pressure generated within the liquefied strata must exert a sufficient 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 300 exploratory borings, 15 different earthquakes, and several ranges of recorded peak ground acceleration.

The potentially liquefiable soils at the site are generally thin layers of alluvial soils with a minimum 14-foot cap of non-liquefiable soil. Based on the above studies, the potentially liquefiable soils are capped by a sufficient thickness of non-liquefiable soils to prevent venting and surface rupture or sand boils during a strong seismic event.

3.2.6 Lateral Spreading

Lateral spreading is a failure within weak soils, typically due to liquefaction, which causes a soil mass to move toward a free face, such as an open channel, or down a gentle slope. There are relatively thin layers of potentially susceptible liquefiable layers at the project, however these soils were not encountered above the creek flow line at exploration locations adjacent to Arroyo Las Positas, and generally appear discontinuous across the site and below any free face condition. Therefore, lateral spreading is considered a low risk in our opinion.

Although lateral spreading is considered a low risk, there is potential for creek bank instability considering that portions of Arroyo Las Positas are deeply incised, there is evidence of recent bank erosion as well as some near-vertical creek bank sections. This concern is further analyzed and discussed in a subsequent section.

3.2.7 Earthquake-Induced Landsliding

No indications of previous deep-seated landsliding were observed during the field exploration at the site, and no features indicative of deep-seated slope instability were observed in historical aerial photographs of the site. Therefore, based on our observations in the field and due to the consistency of material encountered during our subsurface exploration, the potential for deep-seated earthquake-induced landsliding is low.

3.3 CREEK BANK STABILITY AND SETBACKS

As discussed above, Arroyo Las Positas, an existing creek running northeast-southwest, bisects the property. The creek is roughly 14 to 16 feet deep from adjacent terrain with in general 1.5:1 (horizontal:vertical) or flatter bank gradients. The creek banks are well vegetated with grasses and shrubs. Recent bank erosion as well as some near-vertical creek bank sections were also observed along the creek. Refer to the Site Plan (Figure 2) for approximate locations of bank failures along the creek and additional information.

Based on analysis, the top of the existing creek bank may experience some movement and displacement as a result of a strong earthquake event and/or continued erosion. Proposed improvements located within 40 feet from the top of the creek bank may experience some movement. We recommend that proposed improvements be set back a distance of at least 40 feet from the top of the existing creek bank (riparian edge), or at least beyond a 3:1 line of projection extending up from the base of the creek bank, whichever is greater. Additional measures could be used to reduce the above recommended distances, such as a geotechnical corrective grading, geogrid reinforcement, ground improvement, buried retaining walls, and/or sheet piling. These options require site-specific analyses and could be assessed once grading plans are further refined.

3.4 SOIL CORROSION POTENTIAL

As part of this study, we obtained two representative soil samples and submitted to a qualified analytical lab, CERCO, for determination of pH, resistivity, sulfate, and chloride. Additionally, we tested five soil samples in our laboratory for sulfate ion concentration determination. The results, which include a brief corrosivity evaluation of the tested soil sample by CERCO, are included in Appendix B and summarized in the table below.

TABLE 3.4-1: Corrosivity Test Results

SAMPLE LOCATION	DEPTH (FT)	REDOX (mV)	PH	RESISTIVITY (OHMS-CM)	CHLORIDE (MG/KG)	SULFATE (MG/KG)
1-B2	11.5	230	7.38	1,700	62	120
1-B3	32.0	330	8.67	1,200	61	40
1-B1*	5.5	---	---	---	---	N.D.**
1-B2*	25.0	---	---	---	---	N.D.**
1-B3*	11.5	---	---	---	---	N.D.**
1-B4*	21.0	---	---	---	---	N.D.**
1-B6*	5.5	---	---	---	---	1000

* ASTM D4327

** None Detected

The 2016 CBC references the 2014 American Concrete Institute Manual, ACI 318-14, Section 19.3.1 for concrete durability requirements. ACI Table 19.3.1.1 provides the following exposure categories and classes, and Table 19.3.2.1 provides requirements for concrete in contact with soil based upon the exposure class.

TABLE 3.4-2: ACI Table 19.3.1.1: Exposure Categories and Classes

CATEGORY	SEVERITY	CLASS	CONDITION	
F Freezing and thawing	Not Applicable	F0	Concrete not exposed to freezing-and-thawing cycles	
	Moderate	F1	Concrete exposed to freezing-and-thawing cycles and occasional exposure to moisture	
	Severe	F2	Concrete exposed to freezing-and-thawing cycles and in continuous contact with moisture	
	Very Severe	F3	Concrete exposed to freezing-and-thawing cycles and in continuous contact with moisture and exposed to deicing chemicals	
			WATER- SOLUBLE SULFATE IN SOIL % BY WEIGHT*	**DISSOLVED SULFATE IN WATER MG/KG (PPM)****
S Sulfate	Not applicable	S0	$SO_4 < 0.10$	$SO_4 < 150$
	Moderate	S1	$0.10 \le SO_4 < 0.20$	$150 \le SO_4 \le 1,500$ seawater
	Severe	S2	$0.20 \le SO_4 \le 2.00$	$1,500 \le SO_4 \le 10,000$
	Very severe	S3	$SO_4 > 2.00$	$SO_4 > 10,000$
			CONDITION	
P Requiring low permeability	Not applicable	P0	In contact with water where low permeability is not required.	
	Required	P1	In contact with water where low permeability is required.	
C Corrosion protection of reinforcement	Not applicable	C0	Concrete dry or protected from moisture	
	Moderate	C1	Concrete exposed to moisture but not to external sources of chlorides	
	Severe	C2	Concrete exposed to moisture and an external source of chlorides from deicing chemicals, salt, brackish water, seawater, or spray from these sources	

* Percent sulfate by mass in soil determined by ASTM C1580
** Concentration of dissolved sulfates in water in ppm determined by ASTM D516 or ASTM D4130

In accordance with the criteria presented in the above table, these soils are categorized as Not Applicable for all classes except , and are within the F0 freeze-thaw class, S0 sulfate exposure class, P0 exposure class The presence of groundwater indicates the soils should be classified as C1 corrosion class. Cement type, water-cement ratio, and concrete strength, are not specified for these ranges.

Considering a 'Not Applicable' sulfate exposure, there is no requirement for cement type or water-cement ratio, however, a minimum concrete compressive strength of 2,500 psi is specified by the building code. For this sulfate range, the structural designer may consider Type II cement and a concrete mix design for foundations and building slabs-on-grade that incorporates a maximum water-cement ratio of 0.50. It should be noted, however, that the structural engineering design requirements for concrete may result in more stringent concrete specifications.

A brief corrosivity evaluation of the site soils is provided by CERCO in Appendix B. If desired to investigate this further, we recommend a corrosion consultant be retained to evaluate if specific corrosion recommendations are advised for the project. Note that ASTM Test Method D4327 was used in lieu of the ACI-designated sulfate test methods as it provides better test results.

3.5 EXCAVATABILITY

We used a CAT 313L Excavator during our exploratory test pit work. Based upon our observation and experience, we provide the following conclusions regarding excavation resistance at the site:

1. Conventional grading and backhoe equipment will likely be able to excavate the soil deposits.

2. We observed the upper 15 feet of the site soils to be moderately cemented at depth. Conventional grading and backhoe equipment will likely be able to excavate the site soils using light to moderate effort. Deeper grading excavations may encounter lenses of gravel that may require moderate effort with a CAT D8 or larger bulldozer, equipped with single or multi-shank rippers.

We provide the above excavatability information for general planning purposes only. This information is not intended for bidding purposes.

3.6 SHALLOW GROUNDWATER

Groundwater was encountered at depths of 5 to 16 feet below the existing ground surface during field exploration activities at select exploration locations. Refer to the table in the previous section.

Based on the above, we recommend considering a design high groundwater depth of 5 feet below existing grade for project design such as planned roadway improvements on the eastern portion of the site in the vicinity of 1-B7 through 1-B9 and 1-B15. We recommend considering a design groundwater depth of 10 feet below existing grade for project design such as the funeral home building, bridge improvements, and cemetery improvements on the remaining portions of the site.

Fluctuations in groundwater levels should be expected during seasonal changes or over a period of years because of precipitation changes, perched zones, changes in drainage patterns, and irrigation.

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 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

The relative compaction and optimum moisture content of soil and aggregate base referred to in this report are based on the most recent ASTM D1557 test method. Compacted soil is not acceptable if it is unstable. It should exhibit only minimal flexing or pumping, as observed by an ENGEO representative.

As used in this report, 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" in this report 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 DISTURBED NEAR-SURFACE SOIL

As described previously, we anticipate the presence of disturbed near-surface soil throughout the site. Such soil can undergo excessive settlement, especially under new fill or building loads. The proposed building foundation excavation will remove a portion of the disturbed soil. However, in areas outside of the proposed excavation for the basement level of the building, we recommend removal of the material down to undisturbed soil as determined by an ENGEO representative, which we anticipate to be approximately 12 to 24 inches below ground surface. The bottom of the removed area should then be scarified and moisture conditioned before placing new engineered fill. Fill placement specifications may be found in the Fill Compaction Section.

5.2 GENERAL SITE CLEARING

Areas to be developed should be cleared of surface and subsurface deleterious materials, including existing building foundations, slabs, buried 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 Fill Compaction Section. ENGEO should be retained to observe and test backfilling.

Following clearing, strip the site to remove surface organic materials. Strip organics from the ground surface to a depth of at least 2 to 3 inches below the surface. Remove strippings from the site or, if considered suitable by the landscape architect and owner, use them in landscape fill.

It may also be feasible to mulch organics in place, depending on the amount and type of vegetation present at the time of grading as well as the proposed mulching method. If desired, ENGEO can evaluate site vegetation at the time of grading to assess the feasibility of mulching organics in place.

5.3 OVER-OPTIMUM SOIL MOISTURE CONDITIONS

The contractor should anticipate encountering excessively over-optimum (wet) soil moisture conditions during winter or spring grading, during or following periods of rain or high flow in Arroyo Las Positas, and within 5 feet of the groundwater level. In addition, wet soil conditions may be found during excavation for the basement level of the funeral home as well as along the entrance roadway in the eastern portion of the site. 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-flyash, 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 8 inches in maximum dimension. ENGEO should be made aware if any import material is to be used and allowed to sample and test proposed imported fill materials at least 5 days prior to delivery to the site.

5.5 FILL COMPACTION

5.5.1 Grading in Structural Areas

Perform subgrade compaction prior to fill placement, following cutting operations, and in areas left at grade as follows.

1. Scarify to a depth of at least 12 inches.

2. Moisture condition soil to at least 4 percentage points over the optimum moisture content; **and**

3. Compact the soil to between 87 and 92 percent relative compaction. Compact the upper 6 inches of finish pavement subgrade to at least 90 percent relative compaction prior to aggregate base placement.

After the subgrade has been compacted, place and compact acceptable fill as follows:

1. Spread fill in loose lifts that do not exceed 12 inches.

2. Moisture condition lifts to at least 4 percentage points over the optimum moisture content; **and**

3. Compact fill to between 87 and 92 percent relative compaction; compact the upper 6 inches of fill in pavement areas to at least 90 percent relative compaction prior to aggregate base placement.

Compact the pavement Caltrans Class 2 Aggregate Base section to at least 95 percent relative compaction (ASTM D1557). Moisture condition aggregate base to or slightly above the optimum moisture content prior to compaction.

5.5.2 Underground Utility Backfill

The contractor is responsible for conducting trenching and shoring in accordance with CALOSHA requirements. Project consultants involved in utility design should specify pipe-bedding materials.

Place and compact granular trench backfill as follows:

1. Trench backfill should have a maximum particle size of 6 inches.

2. Moisture condition trench backfill to a minimum moisture content of optimum. Moisture condition backfill outside the trench.

3. Place fill in loose lifts not exceeding 12 inches.
   and

4. Compact fill to a minimum of 90 percent relative compaction (ASTM D1557).

Where utility trenches cross perimeter building foundations, backfill with native clay soil for pipe bedding and backfill for a distance of 2 feet on the exterior side of the foundation. This will help prevent the normally granular bedding materials from acting as a conduit for water to enter beneath the building. As an alternative, a sand cement slurry (minimum 28-day compressive strength of 500 psi) may be used in place of native clay soil in both sides of the foundation.

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.

5.5.3 Landscape Fill

Process, place and compact fill in accordance with the Fill Compaction Section except compact to at least 85 percent relative compaction (ASTM D1557).

5.6 SLOPES

5.6.1 Gradients

We recommend the following slope gradient guidelines for cut and fill slopes:

TABLE 5.6.1-1: Slope Gradient Guidelines

SLOPE GRADIENT (HORIZONTAL:VERTICAL)	CUT SLOPE HEIGHT (FEET)	FILL SLOPE HEIGHT (FEET)
2:1	10 or less	10 or less
3:1	Up to 50	Up to 50

Where slopes higher or steeper than those recommended above are desired, or based upon final grading plan slope stability analysis, supplemental slope stabilization techniques such as slope rebuilding, use of select fill materials, or incorporation of geogrid-reinforcing materials may be required.

To improve performance of slopes against erosion, in addition to typical erosion control protection such as hydroseeding or other techniques, we recommend that all finished slopes (cut and fill) receive roughly a 6-inch-thick layer of track-walked moistened strippings placed on a roughened, moistened slope. This will promote quick revegetation of slopes that will help hinder slope erosion.

The contractor is responsible to construct temporary construction slopes in accordance with CALOSHA requirements.

5.6.2 Fill Placed on Existing Slopes

We recommend keying and benching where fills are placed on original grade with a gradient of 8:1 or steeper.

Construct a minimum 18-foot-wide key inward from the toe of the new fill slope. Extend the key at least 2 feet below original grade into firm competent soil/rock, as evaluated by ENGEO. Slope the key bottom at least 2 percent downward toward the heel of the key. Deeper keys may be recommended by ENGEO based on actual soil/rock conditions observed during construction.

Cut benches into original grade after the key has been nearly filled and compacted in accordance with Fill Compaction Section. Construct benches into original slope grade as filling proceeds every 2 feet vertically, to remove loose soil/rock. Deeper bench depths may be recommended by ENGEO depending on actual conditions observed during construction. Bench widths may vary depending on the original slope grade and actual bench depth. Keyway and bench subdrain alternatives are presented on Figure 6.

Planned slopes will be reviewed and analyzed with respect to slope stability as part of the 40-scale grading plan review, at which time applicable remedial grading plans showing locations of keyways, select fill, and subdrains will be prepared. Supplemental stability analysis will also be performed as part of this review process to confirm minimum Factors of Safety will be achieved.

5.6.3 Subsurface Drainage

Subsurface drainage systems should be installed in keyways and swales or natural drainage areas. Typical keyway subdrains are shown on Figure 7.

We recommend that ENGEO be retained to review the final grading plans and show the approximate locations of recommended subdrains on a remedial grading plan. Depending on the actual conditions encountered during grading, similar subsurface drainage facilities may be recommended within low-lying areas. Subdrains should also be added where wet conditions are encountered during grading.

5.7 MANMADE LAKES

Based on preliminary plans and discussions with Monte Vista Memorial Investment Group, we understand two manmade lakes are to be constructed as a part of the overall site development. These lakes and associated stream connecting the two lakes will serve to catch a portion of the stormwater runoff from the site and will be continually circulated throughout both lakes.

One lake is to be located in the northern portion of the site, and will have a maximum depth elevation of 508 feet (approximately 8 feet below final grade). Additionally, this lake will be bisected by a concrete waterfall with the upslope side of the lake being at most 3 feet deep adjacent to the back of the wall and up 15 feet deep on the downslope side. The other lake will be located in the southwestern portion of the site and will have a maximum depth elevation of 489 feet (approximately 25 feet below final grade). The material excavated from these two areas will be used in the construction of the internal roadway throughout the site. Excavation of the manmade lakes and associated stream connecting these two features should comply with the recommendations provided in the Slope Gradient section.

5.7.1 Southern Lake Island

The construction of the southern manmade lake also includes the creation of an island near the northeast edge of the manmade lake. This island will house a pagoda chapel structure and pedestrian walkway connecting the island to the lakeshore. We understand that the island is to be constructed of engineered fill after the main excavation of the lake is completed. Based on preliminary grading plans, the finished grade of this island is set at 513 feet. Furthermore, the planned bottom elevation of the lake, at its deepest point, is approximately 489 feet. We recommend that fill slopes adhere to the recommendations provided in the Slope Gradient Section. All engineered fill should be placed in general conformance with the recommendations provided in the Fill Compaction section.

We recommend that ENGEO be retained to review the final grading plans and lake designs to confirm they are designed in general accordance with our recommendations, and provide supplemental recommendations as needed.

5.7.2 Lake Design Considerations

As described in Section 2.8, we performed hydraulic conductivity tests on in-situ and remolded samples of the native clay to evaluate the reuse as a clay liner. Based on our lab testing and review of existing data, it is our opinion that the onsite near surface clay soils may be suitable to use as material for the manmade lake liners; however, the lake designer should review the laboratory testing and confirm if the resulting hydraulic conductivity parameters are acceptable.

Once final grading plans of lake have been finalized, and in coordination with the lake designer, we can perform infiltration testing or seepage analysis to supplement the above recommendations and determine if an additive to reduce the hydraulic conductivity, such as bentonite, will be necessary.

5.8 SITE DRAINAGE

5.8.1 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. The latest California Building Code Section 1804.4 specifies minimum slopes of 5 percent away from foundations. Where development conditions 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.

5.9 STORMWATER INFILTRATION AND SELECT PROJECT RISK LEVEL FACTORS

Due to the density of the site soils and fines content (percentage passing the No. 200 sieve) generally exceeding 30 percent, the near-surface site soils are expected to have a low permeability value for stormwater infiltration in grassy swales or permeable pavers, unless subdrains are installed. Therefore, Best Management Practices should assume that limited stormwater infiltration will occur at the site.

5.10 STORMWATER BIORETENTION AREAS

We understand bioretention areas are planned as part of the overall site development; therefore, we recommend that, when practical, they be planned a minimum of 5 feet away from structural site improvements, such as buildings, streets, retaining walls, and sidewalks/driveways. When this is not practical, bioretention areas located within 5 feet of structural site improvements can either:

1. Be constructed with structural side walls capable of withstanding the loads from the adjacent improvements, or

2. Incorporate filter material compacted to between 85 and 90 percent relative compaction (ASTM D1557, latest edition) and a waterproofing system designed to reduce the potential for moisture transmission into the subgrade soil beneath the adjacent improvement.

In addition, one of the following options should be followed.

1. We recommend that bioretention design incorporate a waterproofing system lining the bioswale excavation and a subdrain, or other storm drain system, to collect and convey water to an approved outlet. The waterproofing system should cover the bioretention area excavation in such a manner as to reduce the potential for moisture transmission beneath the adjacent improvements.

2. Alternatively, and with some risk of movement of adjacent improvements, if infiltration is desired, we recommend the perimeter of the bioretention areas be lined with an HDPE tree root barrier that extends at least 1 foot below the bottom of the bioretention areas/infiltration trenches.

Site improvements located adjacent to bioretention areas that are underlain by base rock, sand, or other imported granular materials, should be designed with a deepened edge that extends to the bottom of the imported material underlying the improvement.

Where adjacent site improvements include buildings greater than three stories, streets steeper than 3 percent, or design elements subject to lateral loads (such as from impact or traffic patterns), additional design considerations may be recommended. If the surface of the bioretention area is depressed, the slope gradient should follow the slope guidelines described in earlier section(s) of this document. In addition, although not recommended, if trees are to be planted within bioretention areas, HDPE Tree Boxes that extend below the bottom of the bioretention system should be installed to reduce potential impact to subdrain systems that may be part of the bioretention area design. For this condition, the waterproofing system should be connected to the HPDE Tree Box with a waterproof seal.

Given the nature of bioretention systems and possible proximity to improvements, we recommend ENGEO be retained to review design plans and provide testing and observation services during the installation of linings, compaction of the filter material, and connection of designed drains.

It should be noted that the contractor is responsible for conducting all excavation and shoring in a manner that does not cause damage to adjacent improvements during construction and future maintenance of the bioretention areas. As with any excavation adjacent to improvements, the contractor should reduce the exposure time such that the improvements are not detrimentally impacted.

5.11 LANDSCAPING CONSIDERATION

As the near-surface soils are moderately to highly expansive, we recommend greatly restricting the amount of surface water infiltration near structures, pavements, flatwork, and slabs-on-grade. This may be accomplished by:

- Selecting landscaping that requires little or no watering, especially within 3 feet of structures, slabs-on-grade, or pavements.
- Using low precipitation sprinkler heads.
- Regulating the amount of water distributed to lawn or planter areas by installing timers on the sprinkler system.
- Providing surface grades to drain rainfall or landscape watering to appropriate collection systems and away from structures, slabs-on-grade, or pavements.
- Preventing water from draining toward or ponding near building foundations, slabs-on-grade, or pavements.
- Avoiding open planting areas within 3 feet of the building perimeter.

We recommend that these items be incorporated into the landscaping plans.

5.12 REMEDIAL GRADING PLANS

Due to the complex geology, and hillside topography, we recommend that ENGEO be retained to prepare remedial grading plans, for this project once final grading plans have been completed. This is important to clarify our geotechnical recommendations related to keyways, benches and subdrains. In preparing these plans, we intend to overlay the grading plans with graphic representations of our grading and subsurface drainage recommendations presented in this report. This allows the unique hillside geotechnical recommendations to be clearly displayed on the grading plans. This can assist in obtaining more accurate earthwork bids as well as clarifying the geotechnical recommendations as they apply to the final grading plan.

6.0 FOUNDATION RECOMMENDATIONS

We developed foundation recommendations using data obtained from our field exploration, laboratory test results, and engineering analysis. We recommend the funeral home and pavilion building foundations consist of a conventional structural mat foundation. Furthermore, we developed deep foundation recommendations for the two single span bridges and auxiliary solar panel trellis and pedestrian boardwalk through the wetland area.

6.1 CONVENTIONAL MAT FOUNDATION

The proposed funeral home and pavilion building can be supported on a structural mat foundation. The mat foundations should be designed to impose a maximum allowable uniform bearing pressure of 1,500 pounds per square foot (psf) for dead plus long-term live loads. The allowable bearing capacity may be increased to 2,000 psf in areas of loading concentration. These values may be increased by one-third when considering transient loads, such as wind or seismic. We recommend that structural mat foundations be designed for an edge-cantilever distance of 5 feet, and unsupported interior free span of 10 feet. A modulus of subgrade reaction of 75 kips per cubic foot can be used for engineered fill or native soil.

The design should incorporate 1½-inch total and ¾-inch differential settlement due to liquefaction settlement. The differential settlement may be assumed to occur over a horizontal distance of 30 feet or between adjacent column supports, whichever is closer.

Differential settlement between the proposed at-grade and below grade portions of the funeral home structure is also a geotechnical concern considering the structure will have a multi-level building pad. Assuming the at-grade and below-grade portions of the building are structurally connected, we recommend that foundation and structural design incorporate an additional ½ inch of differential settlement between the at-grade and below grade portions of the structure.

Resistance to lateral loads may be provided by frictional resistance between the foundation concrete and the subgrade soils and by passive earth pressure acting against the side of the foundation. A coefficient of friction of 0.30 can be used between concrete and the subgrade. If a waterproofing membrane is placed below the mat, a coefficient of friction of 0.20 should be used. Passive pressures can be taken as equivalent to the pressure developed by a fluid having a weight of 250 pounds per cubic foot (pcf).

6.1.1 Below-Grade Building Pad Subgrade Preparation

Based on our field exploration and laboratory testing, we anticipate low to moderately expansive soils will be encountered at the below-grade levels of the structure. Depending on the depth of the basement excavation and time of construction, the subgrade soils may be weak and/or near saturation. We recommend assuming the upper 18 inches of subgrade soils will require stabilization prior to improvements construction. This may be accomplished by overexcavation and one or more of the following options:

- Construction of a working pad of 18 inches of clean crushed rock and incorporating a geotextile stabilization fabric if needed.
- Construction of a lean concrete rat slab.
- Chemical treatment of the subgrade soils.

Even after stabilization, the building pad will be susceptible to disturbance under construction equipment loads. The contractor should limit the use of heavy and/or rubber-tired equipment on the subgrade to reduce potential for creation of unstable areas. Where the subgrade is disturbed during construction, the disturbed material should be removed and replaced with crushed rock or lean concrete.

6.1.2 Buoyancy Impacts

The below-grade basement may be founded below the 10-foot design groundwater level and may be subject to buoyancy impacts. The foundation should be designed to resist hydrostatic uplift pressures due to the design groundwater level of 10 feet below existing grade. Uplift resistance can be provided by the weight of the foundation elements and the dead loads of the building. The Structural Engineer should evaluate the buoyancy uplift on the structure and determine if additional resistance is necessary. Viable alternatives for added uplift resistance include hold-down piers or anchors. These can be designed as active or passive systems for which ENGEO can provide more details as necessary.

6.1.3 Waterproofing Considerations

The mat foundation and basement walls should be waterproofed and designed to resist hydrostatic and/or uplift pressures. The waterproofing should be designed by a specialty consultant that specializes in permanent waterproofing construction. Waterstops should be placed at all construction joints.

6.2 BUILDING RETAINING WALLS

It is anticipated the funeral home building will include below-grade retaining walls up to 12 feet in height. The building retaining walls should be designed to resist lateral earth pressures from natural materials and/or backfill and from any surcharge loads. Provided that adequate drainage is included as recommended below, the restrained walls may be designed using an at-rest equivalent fluid pressure of 80 pcf. The design should account for one-half of any vertical surcharge loads applied as a uniform lateral load to the top 10 feet of the wall.

If the structure is designed to resist hydrostatic pressures because of limited below-grade drainage, then the building walls should have drainage facilities above the design groundwater depth of 10 feet below existing grade to reduce the potential for build-up of hydrostatic pressures. The wall design should include an additional 40 pcf hydrostatic pressure for depths greater than the design depth to groundwater of 10 feet below ground surface.

We recommend the seismic performance of the basement retaining walls be evaluated using an active equivalent fluid weight of 50 pcf for drained conditions and an active equivalent fluid weight of 90 pcf for undrained conditions, and a seismic increment of 25 pcf, in accordance with Lew, et al. (2010). This evaluation should be separate from the static design using at-rest earth pressures. Passive pressures acting on foundations should be designed in accordance with the recommendations in Section 6.1 above.

6.2.1 Wall Drainage

In general, all walls retaining more than 2 feet of soil that are not designed to resist hydrostatic pressures should be provided with drainage facilities to prevent the build-up of hydrostatic pressures behind the walls. Wall drainage may be provided using a 4-inch-diameter perforated pipe embedded in either free-draining gravel surrounded by synthetic filter fabric (minimum 6-ounce) or Class 2 permeable material (Part 2 of Supplemental Recommendations, Section 2.05B). The width of the drain blanket should be at least 12 inches, and the drain blanket should extend to about 1 foot below the finished grades. The upper 1 foot of wall backfill should consist of compacted site soils. Drainage should be collected into solid pipes and directed to an outlet approved by the Civil Engineer. Synthetic filter fabric should meet the minimum requirement listed in the Supplemental Recommendations and be preapproved by the Geotechnical Engineer prior to delivery.

Design details for draining the below grade retaining walls above the groundwater level should be determined during the design process. A sump system may be needed for drainage unless the storm drain system will allow for gravity connection and outfall. Construct either graded rock drains or geosynthetic drainage composites behind the retaining walls to reduce hydrostatic lateral forces. For rock drain construction, we recommend two types of rock drain alternatives:

1. A minimum 12-inch-thick layer of Class 2 Permeable Filter Material (Caltrans Specification 68-1.025) placed directly behind the wall, or

2. A minimum 12-inch-thick layer of washed, crushed rock. Envelop rock in a minimum 6-ounce, nonwoven geotextile filter fabric.

For both types of rock drains:

- Place the rock drain directly behind the walls of the structure.

- Extend rock drains from a depth of 10 feet below the existing ground surface to within 12 inches of the top of the wall.

- Place a minimum of 4-inch-diameter perforated pipe at the base of the drain material, inside the rock drain and fabric, with perforations placed down.

- Place pipe at a gradient of at least 1 percent to direct water away from the wall by gravity to a sump or drainage facility.

ENGEO should review and approve geosynthetic composite drainage systems prior to use.

6.2.2 Wall Backfill

Backfill behind retaining walls should be placed and compacted in accordance with fill placement recommendations. Use light compaction equipment within 5 feet of the wall face. If moderate to heavy compaction equipment is used, the walls should be temporarily braced to avoid excessive wall movement. Alternatively, the wall design can incorporate additional surcharge loading to allow moderate to heavy equipment.

6.3 TEMPORARY EXCAVATIONS

The Contractor should be familiar with applicable local, state, and federal regulations, including the current Occupational Safety and Health Administration (OSHA) Excavation and Trench Safety Standards. It is the responsibility of the Contractor to provide stable, safe trench and construction slope conditions and to follow OSHA safety requirements. Since excavation procedures may be dangerous, it is also the responsibility of the Contractor to provide a trained “competent person” as defined by OSHA to supervise all excavation operations, ensure that all personnel are working in safe conditions and have thorough knowledge of OSHA excavation safety requirements.

6.4 TEMPORARY DEWATERING

Temporary dewatering during construction may be necessary to keep the excavation and working areas reasonably dry. Dewatering should be performed in a manner such that water levels are maintained not less than 2 feet below the bottom of excavation prior to and continuously during shoring installation. As the excavations progress, it may be necessary to dewater the soils ahead of the excavation, such as by continuous pumping from sumps, to control the tendency for the bottom of the excavation to heave under hydrostatic pressures and to reduce inflow of water or soil from beneath temporary shoring, should shoring be utilized. An active dewatering system such as dewatering wells should be considered but may be inefficient and cost-prohibitive considering the clayey site soils. Selection of temporary dewatering method(s) should be coordinated with selection of temporary shoring systems.

Ultimately, the selection of equipment and method should be determined by the contractor/designer. The dewatering system implemented should be selected to have minimal impact on the groundwater level surrounding the proposed excavations. The dewatering system should be designed to prevent pumping soil fines with the discharge water. Uncontrolled dewatering could cause settlement of the general area and affect existing improvements in the vicinity of the site. Therefore, adjacent improvements should be monitored for vertical movement during construction.

Groundwater management including temporary storage in Baker tanks (or similar) and testing should be considered prior to discharge of generated water. Requirements of potential receiving facilities should be determined in advance of construction. Impacted groundwater may require discharge to a specialty facility.

7.0 BRIDGE FOUNDATION RECOMMENDATIONS

We understand that two vehicular bridges are proposed to span across Arroyo Las Positas. Based on preliminary plans, the bridges will be single-span of approximately 80 to 100 feet, and we estimate each bridge will be approximately 30 feet wide. We recommend the bridge abutments be supported on cast-in-drilled-hole (CIDH) pier foundations.

As discussed, we also provide preliminary design recommendations to support the bridge abutments on helical pile foundations.

The recommendations below should be considered preliminary in nature. Once additional information, including foundation type(s) and loading are developed, we should revisit and update our recommendations.

7.1 DRILLED PIERS

Based on the soil conditions encountered at the site, we recommend that the vehicular bridges be supported on cast-in-drilled-hole (CIDH) straight-shaft friction piers. The soil cirteria for the drilled piers are listed below. The proposed minimum pier depths for each support is based on estimated loads from previous single span bridge projects.

- Minimum diameter: 2 feet.
- Minimum pier depth: Southern Bridge: 40 feet at both abutments
  Northern Bridge: 40 feet at both abutments

TABLE 7.1-1: Ultimate Skin Friction Values for Southern Bridge

LOCATION	ELEVATION (FEET)	ULTIMATE SKIN FRICTION-STATIC (PSF)	ULTIMATE SKIN FRICTION-SEISMIC (PSF)
West and East Abutments Approximate Surface Elevation = 487 feet	482 – 487	0	400*
	477 – 482	500	400*
	472 – 477	700	600*
	462 – 472	700	200*
	Below 462	2,500	2,500

**Apply as a downdrag load, factor of safety should not be applied.*

TABLE 7.1-2: Ultimate Skin Friction Values for Northern Bridge

LOCATION	ELEVATION (FEET)	ULTIMATE SKIN FRICTION-STATIC (PSF)	ULTIMATE SKIN FRICTION-SEISMIC (PSF)
North Abutment Approximate Surface Elevation = 487 feet	482 – 487	0	0
	472 – 482	500	400*
	467 – 472	700	200*
	Below 467	2,500	2,500
South Abutment Approximate Surface Elevation = 487 feet	482 – 487	0	0
	472 – 482	500	400*
	462 – 472	700	200*
	Below 462	2,500	2,500

** Apply as a downdrag load, factor of safety should not be applied.*

The Structural Engineer should design the foundation elements for the actual loading requirements, including the steel reinforcement.

Research has shown that the lateral capacity of a group of piles is generally less than that of a single pile for pile spacings less than 6 to 8 pile diameters. For pile groups with a minimum spacing of 3 pile diameters, we recommend reducing the single pile allowable lateral capacities by the following percentages in Table 7.1-3.

TABLE 7.1-3: Group Reduction Percentages

NO. OF PILES IN GROUP	PERCENTAGE TO REDUCE SINGLE PILE CAPACITY BY
2	25
4	30
9	43
16	48
25	54

Please contact us if group reduction percentages are needed for additional pile group configurations.

Based on the shallow groundwater and the granular layers found in the exploratory points, dewatering and casing of the proposed piers may be necessary.

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.

Due to the potential for caving, 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.

7.1.1 Lateral Pile Capacities

We anticipate the computer program, L-Pile, will be used in pile lateral loading computations. Based on our field data, the soil parameters for the computer input were developed. The following tables list the input criteria for the computer software.

TABLE 7.1.1-1: Southern Bridge - East and West Abutment

ELEVATION (FEET)	GENERALIZED SOIL PROFILE	L-PILE SOIL TYPE	SOIL STRENGTH (KSF) OR FRICTION ANGLE (DEGREES)	k (PCI)	E_{50}	EFFECTIVE UNIT WEIGHT (PCF)
Above 477	Sandy Clay	Stiff Clay	1.0	500	0.007	120
Between 477 and 462*	Clayey Sand	Sand	30°	60	---	60
Below 462	Clay	Very Stiff Clay	4.0	1,000	0.005	60

*Use liquefaction P-Multiplier of 0.12 for this layer.

TABLE 7.1.1-2: Northern Bridge – North Abutment

ELEVATION (FEET)	GENERALIZED SOIL PROFILE	L-PILE SOIL TYPE	SOIL STRENGTH (KSF) OR FRICTION ANGLE (DEGREES	k (PCI)	E_{50}	EFFECTIVE UNIT WEIGHT (PCF)
Above 477	Sandy Clay	Stiff Clay	1.0	500	0.007	120
Between 472 and 477	Sandy Clay	Stiff Clay	1.0	500	0.007	60
Between 467 and 472*	Clayey Sand	Sand	30°	60	---	60
Below 467	Clay	Very Stiff Clay	4.0	1,000	0.005	60

*Use liquefaction P-Multiplier of 0.12 for this layer.

TABLE 7.1.1-3: Northern Bridge – South Abutment

ELEVATION (FEET)	GENERALIZED SOIL PROFILE	L-PILE SOIL TYPE	SOIL STRENGTH (KSF) OR FRICTION ANGLE (DEGREES	k (PCI)	E_{50}	EFFECTIVE UNIT WEIGHT (PCF)
Above 477	Sandy Clay	Stiff Clay	1.0	500	0.007	120
Between 472 and 477	Sandy Clay	Stiff Clay	1.0	500	0.007	60
Between 462 and 472*	Clayey Sand	Sand	30°	60	---	60
Below 462	Clay	Very Stiff Clay	4.0	1,000	0.005	60

*Use liquefaction P-Multiplier of 0.12 for this layer.

7.2 HELICAL PILES

We provide the following preliminary design recommendations for use in the design of the helical anchor foundations for the bridges:

TABLE 7.2-1: Preliminary Design Recommendations

LOCATION	EMBEDMENT ELEVATION (FEET)	ALLOWABLE END BEARING CAPACITY (PSF)
Southern Bridge	461	11,000 x Area*
Northern Bridge	460	11,000 x Area*

*Area = Area of the circular plate

The above design capacities are based off a 50-foot embedment depth of the helical pile and one helical plate at the end of the pile. If multiple plates are to be used and the spacing of each helix is greater than 3B (where B is the diameter of the helical plate) from one another, the above capacities can be multiplied by the number of helical plates along the pile. Additionally, no more than five helical plates may be used along the pile.

Uplift capacities can utilize the end bearing capacities and design methodology presented above, however a reduction factor of 0.8 should be applied to the end bearing capacity values to account for soil disturbance above the helical plate as a result of installation.

Finally, due to the limited capabilities of vertical helical piles to resist lateral loads, we recommend a number of helical piles be battered to resist lateral loads. Lateral load analysis may utilize the input criteria presented in the previous Lateral Pile Capacity Section.

7.2.1 Wing Walls and Abutment Walls

If backfilled with onsite soils, wing and abutment walls should be designed for lateral fluid pressure as provided in the following Lateral Soil Pressure Section. Additionally, we recommend the retaining walls include dynamic seismic earth pressures. If Caltrans structural back fill material is used behind wing and abutment walls, the associated Caltrans loading criteria should be assumed.

Applicable loading, including surcharges due to traffic, buildings, stockpiles, construction equipment, etc. should be incorporated into shoring design when the surcharge loading is situated above a 1:1 line of projection extending up the bottom of wall. Appropriate safety factors against overturning and sliding should be incorporated into the design calculations.

Foundation plans should be submitted to the Geotechnical Engineer for review when they are available.

8.0 SITE RETAINING WALLS

8.1 LATERAL SOIL PRESSURES

Unrestrained drained retaining walls constructed on level ground may be designed using the following active equivalent fluid weights in pounds per cubic foot (pcf).

TABLE 8.1-1: Active Equivalent Fluid Pressures

BACKFILL SLOPE CONDITION (HORIZONTAL:VERTICAL)	ACTIVE PRESSURE (POUNDS PER CUBIC FOOT)
Level	50
3:1	60
2:1	70

Appropriate surcharge loads from vehicles, sidewalk/hardscape, buildings, and other potential surcharge loadings, as applicable, should be incorporated when the surcharge loading is situated above a 1:1 (horizontal:vertical) line of projection extending up from the bottom of the wall. If needed, vertical surcharge loads may be applied as uniform, horizontal surcharge loading equal to 50 percent of the vertical surcharge load. Unless appropriate surcharge loading for construction equipment is incorporated in the wall designs, light hand-operated equipment should be used during backfill compaction of engineered fill and improvement construction behind the walls, to reduce potential for possible overstressing of the walls

8.1.1 Wall Seismic Design

Seismic conditions should be considered in the design of the perimeter retaining walls. Under seismic conditions, the active incremental seismic force along the face of a retaining wall should be added to the static active pressures, and can be calculated as follows:

$$\Delta P = 14 \times H^2$$

H is the design height of the wall (in feet) and ΔP is the active incremental seismic force in pounds per foot of wall. This force has a horizontal direction and should be applied at $^1/_3$ x H from the base of the wall. This force should be combined with the appropriate active equivalent pressure.

8.2 RETAINING WALL DRAINAGE

Drainage facilities should be installed behind retaining walls to prevent the build-up of hydrostatic pressures on the walls. Wall drainage may be provided using 4-inch-diameter perforated (SDR 35 or approved equivalent) pipe encapsulated in either Class 2 permeable material, or free-draining gravel surrounded by synthetic filter fabric.

The width of the gravel-type drain blanket should be at least 12 inches. The drain blanket should extend from the base of the wall to about 1 foot below the finished grade at top of wall. The upper 1 foot of wall backfill should consist of clayey soil or other approved, relatively impervious material.

If preapproved by the Geotechnical Engineer, prefabricated wall drain panels could be considered in lieu of the granular drain blanket above the pipe system. Drainage should be collected by solid pipes and directed to an outlet approved by the Civil Engineer.

8.3 FOUNDATIONS

8.3.1 Shallow Continuous Footings

We recommend that retaining wall footings be designed using an allowable bearing pressure of 2,500 pounds per square foot (psf) for dead-plus-live-loading conditions. This value may be increased by one-third when evaluating the short-term effects of wind or seismic loading.

For a level foreground condition, the footing should be embedded at least 24 inches below lowest adjacent grade. We recommend a minimum footing thickness of 12 inches. Actual footing design (sizing, reinforcement, etc.) should be determined by the structural engineer based on structural design considerations. Footings located adjacent to utility trenches should have their bearing surfaces below an imaginary 1:1 plane projected upward from the bottom edge of the trench to the footing.

Passive pressures acting on footing foundations may be assumed as 250 pcf. Unless the surface directly in front of the wall is confined by a slab or pavement, we recommend starting passive pressure resistance at a depth of 1 foot below lowest adjacent grade, or that depth necessary to achieve a horizontal distance of 10 feet between the outer base edge of the footing and nearest free face, whichever is shallower. The friction factor for sliding resistance may be assumed as 0.30. Appropriate safety factors against overturning and sliding should be incorporated into the design calculations.

8.3.2 Drilled Pier Foundations

We recommend concrete waterfall retaining wall at the upper lake be supported on CIDH piers because of the sloping foreground below the wall. Additionally, we understand that a number of auxiliary structures may be constructed using drilled piers including: solar trellises, pedestrian boardwalks, and a pagoda chapel located on the island within the southern lake. The following recommendations should be used for design of these structures:

TABLE 8.3.2-1: Design Parameters for Drilled Piers

PIER DESIGN ELEMENT	AUXILIARY STRUCTURE DESIGN PARAMETERS	CONCRETE WATERFALL DESIGN PARAMETER
Minimum pier diameter:	12 inches.	12 inches.
Minimum pier depth:	8 feet	10 feet
Downward load capacity (allowable skin friction):	350 psf. This value may be increased by one-third when considering seismic or wind loads. Exclude the upper 2 feet of the pier shaft from pier load capacity computations	500 psf. This value may be increased by one-third when considering seismic or wind loads. Exclude the upper 2 feet of the pier shaft from pier load capacity computations

PIER DESIGN ELEMENT	AUXILIARY STRUCTURE DESIGN PARAMETERS	CONCRETE WATERFALL DESIGN PARAMETER
Minimum pier spacing:	3 pier diameters, center-to-center	3 pier diameters, center-to-center
Passive Resistance Pressure:	250 pcf acting on 2 times the pier diameter. This value may be increased by one-third when considering seismic or wind loads. Passive resistance may start at the depth required to provide 10 feet of lateral confinement in front of the drilled piers. The passive resistance may be applied over two pier diameters	300 pcf acting on 2 times the pier diameter. This value may be increased by one-third when considering seismic or wind loads. Passive resistance may start at the depth required to provide 10 feet of lateral confinement in front of the drilled piers. The passive resistance may be applied over two pier diameters.

Appropriate safety factors against bending of wall elements and pier embedment should be incorporated into the design calculations. Actual pier depths and spacing should be determined by the structural engineer based on structural design considerations.

9.0 PAVEMENT DESIGN

9.1 FLEXIBLE PAVEMENTS

We obtained a representative bulk sample of the surface soil from the site area and performed R-value tests to provide data for pavement design. The results of the test are included in Appendix B and indicate an R-value of 11 and 15. Because surface soils vary across the site, it is our opinion that an R-value of 10 is applicable for design. Using estimated traffic indices for various pavement loading requirements, we developed the following recommended pavement sections using Topic 633 of the Caltrans Highway Design Manual (including the asphalt factor of safety), presented in the table below.

TABLE 9.1-1: Recommended Asphalt Concrete Pavement Sections

TRAFFIC INDEX	SECTION	
	ASPHALT CONCRETE (INCHES)	CLASS 2 AGGREGATE BASE (INCHES)
5	3	11
6	3.5	14
7	4	17

The civil engineer should determine the appropriate traffic indices based on the estimated traffic loads and frequencies.

9.2 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:

- Use a minimum section of 6 inches of Portland Cement concrete over 12 inches of Caltrans Class 2 Aggregate Base.

- Concrete pavement should have a minimum 28-day compressive strength of 3,500 psi.
- Provide minimum control joint spacing in accordance with Portland Cement Association guidelines.

9.3 SUBGRADE AND AGGREGATE BASE COMPACTION

Compact finish subgrade and aggregate base in accordance with Fill Compaction Section. Aggregate Base should meet the requirements for ¾-inch maximum Class 2 AB in accordance with Section 26-1.02B of the latest Caltrans Standard Specifications.

9.4 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 base rock 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.

10.0 SLABS-ON-GRADE

10.1 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. Compact the aggregate base to at least 90 percent relative compaction (ASTM D1557). Consideration should be given to thicken flatwork edges to at least 10 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. Construct control and construction joints in accordance with current Portland Cement Association Guidelines.

11.0 GROUND HEAT EXCHANGE

Based on our findings and review of the proposed development, we consider the site to be *highly* suitable for using a Ground Heat-Exchange (GHX) system to achieve energy savings and to potentially eliminate the need for outdoor air conditioner units, if desired.

For the thermal properties of the soil and groundwater conditions at the site, an open-loop GHX system would likely be well suited and could be implemented on select buildings, or integrated into a project-wide system.

As project planning progresses into architectural design, we can meet with you, your architect, and your MEP designer to further assess and develop GHX energy saving opportunities and efficiencies.

12.0 LIMITATIONS AND UNIFORMITY OF CONDITIONS

This report presents geotechnical recommendations for design of the improvements discussed in Section 1.3 for the Monte Vista Memorial Investment Group 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 owner 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 developers, owners, buyers, 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 or provide insurance; 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 is representative of the actual subsurface conditions across the site. Considering possible underground variability of soil, rock, stockpiled material, 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, notify ENGEO 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, notify the proper regulatory officials 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 on-site 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.

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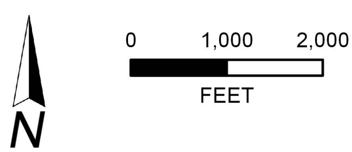
FIGURES



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VICINITY MAP
3656 LAS COLINAS ROAD
LIVERMORE, CALIFORNIA

PROJECT NO. : 15426.000.000	FIGURE NO. 1
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FEET

EXPLANATION
ALL LOCATIONS ARE APPROXIMATE

— — —	GEOLOGIC CONTACT	**Qaf**	ARTIFICIAL FILL
1-CPT5	CONE PENETRATION TEST (ENGEO, 2018)	**Qc**	COLLUVIUM
1-TP8	TEST PIT (ENGEO, 2018)	**Qhal**	HOLOCENE ALLUVIUM
1-B15	SOLID FLIGHT AUGER BORING (ENGEO, 2018)	**Qal**	OLDER ALLUVIUM AND ALLUVIAL TERRACE DEPOSITS
1-B6	MUD ROTARY BORING (ENGEO, 2018)	**QTlu**	LIVERMORE GRAVELS

BASE MAP SOURCE: GOOGLE EARTH MAPPING SERVICE

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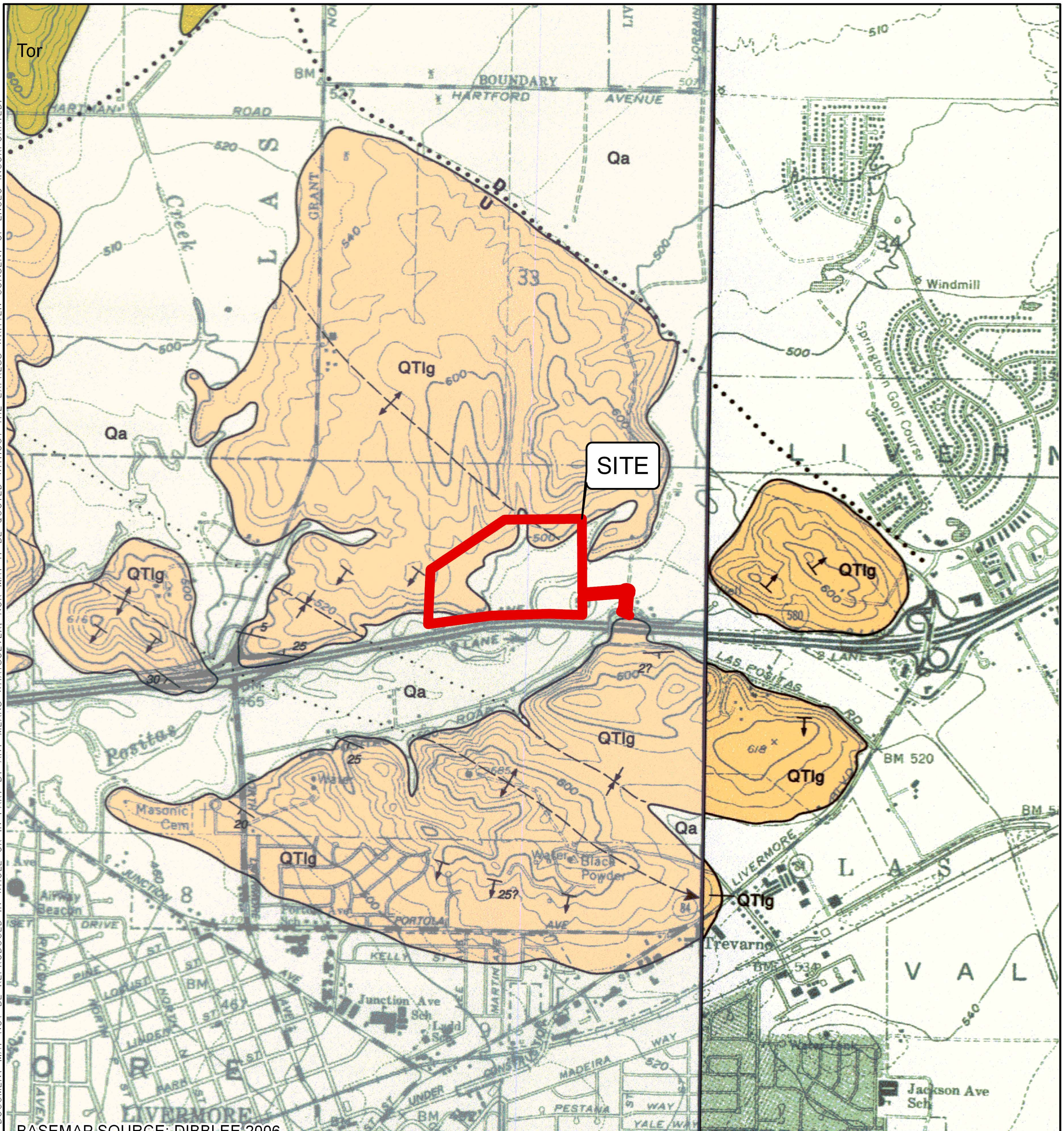
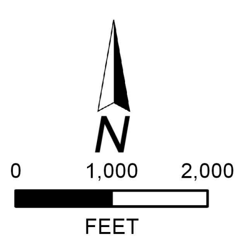
SITE PLAN
3656 LAS COLINAS ROAD
LIVERMORE, CALIFORNIA

PROJECT NO.: 15426.000.000		FIGURE NO. 2
SCALE: AS SHOWN		
DRAWN BY: LL	CHECKED BY: AHF	

ORIGINAL FIGURE PRINTED IN COLOR





EXPLANATION

ALL LOCATIONS ARE APPROXIMATE

Qa	ALLUVIUM GRAVEL
QTlg	LIVERMORE GRAVEL
Tor	ORINDA FORMATION

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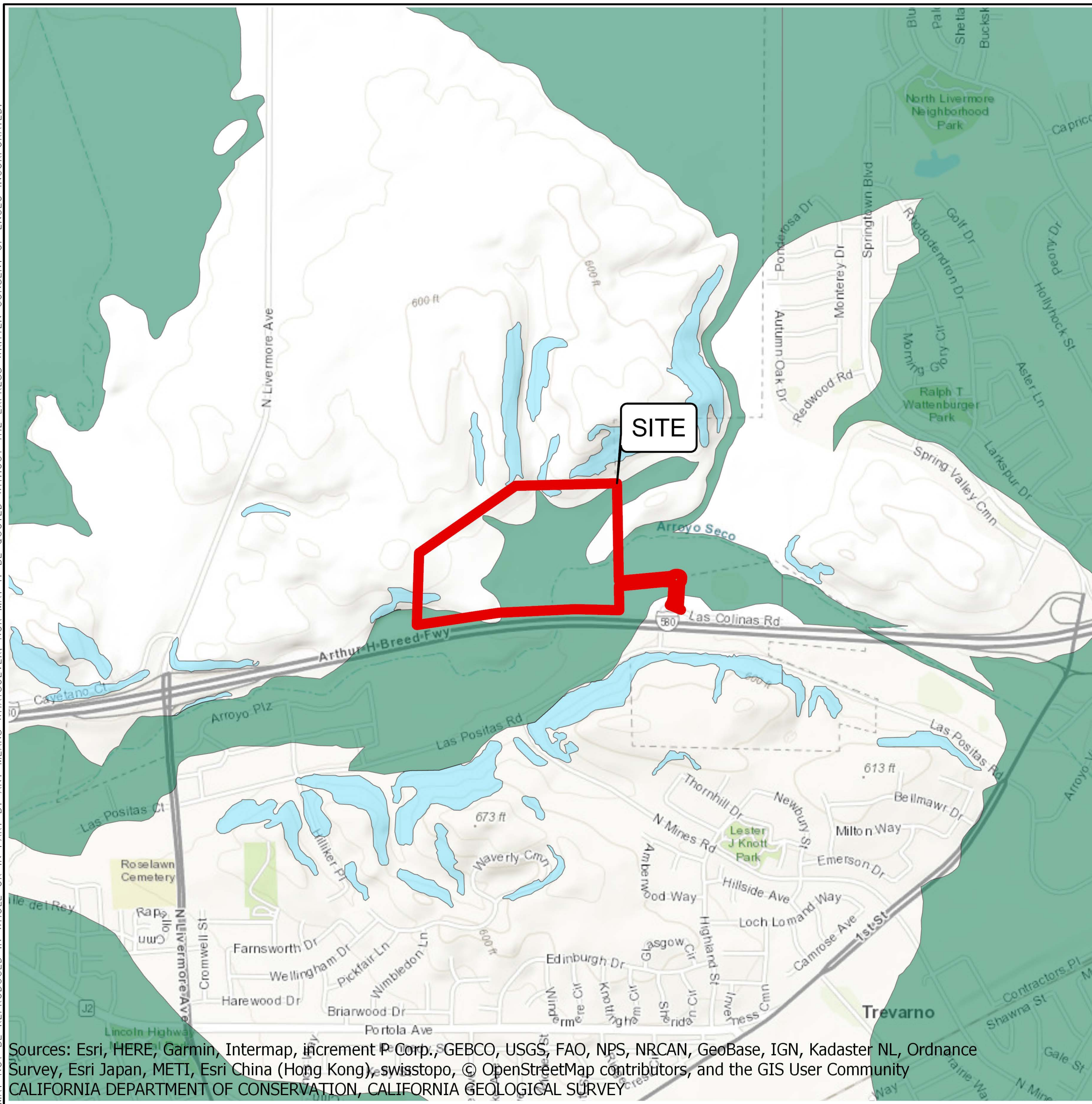
REGIONAL GEOLOGIC MAP
3656 LAS COLINAS ROAD
LIVERMORE, CALIFORNIA

PROJECT NO. : 15426.000.000		FIGURE NO. 3
SCALE: AS SHOWN		
DRAWN BY: QRL	CHECKED BY: AHF	

PATH: G:\DRAFTING\DRAFTING2_DWG_13000 PLUS\15426\000\112018-GEX\GIS\15426.APRX
USER: QLIANG

ORIGINAL FIGURE PRINTED IN COLOR



Sources: Esri, HERE, Garmin, Intermap, increment P Corp., GEBCO, USGS, FAO, NPS, NRCAN, GeoBase, IGN, Kadaster NL, Ordnance Survey, Esri Japan, METI, Esri China (Hong Kong), swisstopo, © OpenStreetMap contributors, and the GIS User Community
CALIFORNIA DEPARTMENT OF CONSERVATION, CALIFORNIA GEOLOGICAL SURVEY

EXPLANATION

ALL LOCATIONS ARE APPROXIMATE

N

0 1,000 2,000 FEET

Liquefaction Zone
Areas where historical occurrence of liquefaction, or local geological,geotechnical and ground water conditions indicate a potential for permanent ground displacements such that mitigation as defined in Public Resources Code Section 2693(c) would be required

Earthquake-Induced Landslide Zones
Areas where previous occurrence of landslide movement, or local topographic, geological, geotechnical and subsurface water conditions indicate a potential for permanent ground displacements such that mitigation as defined in Public Resources Code Section 2693(c) would be required.

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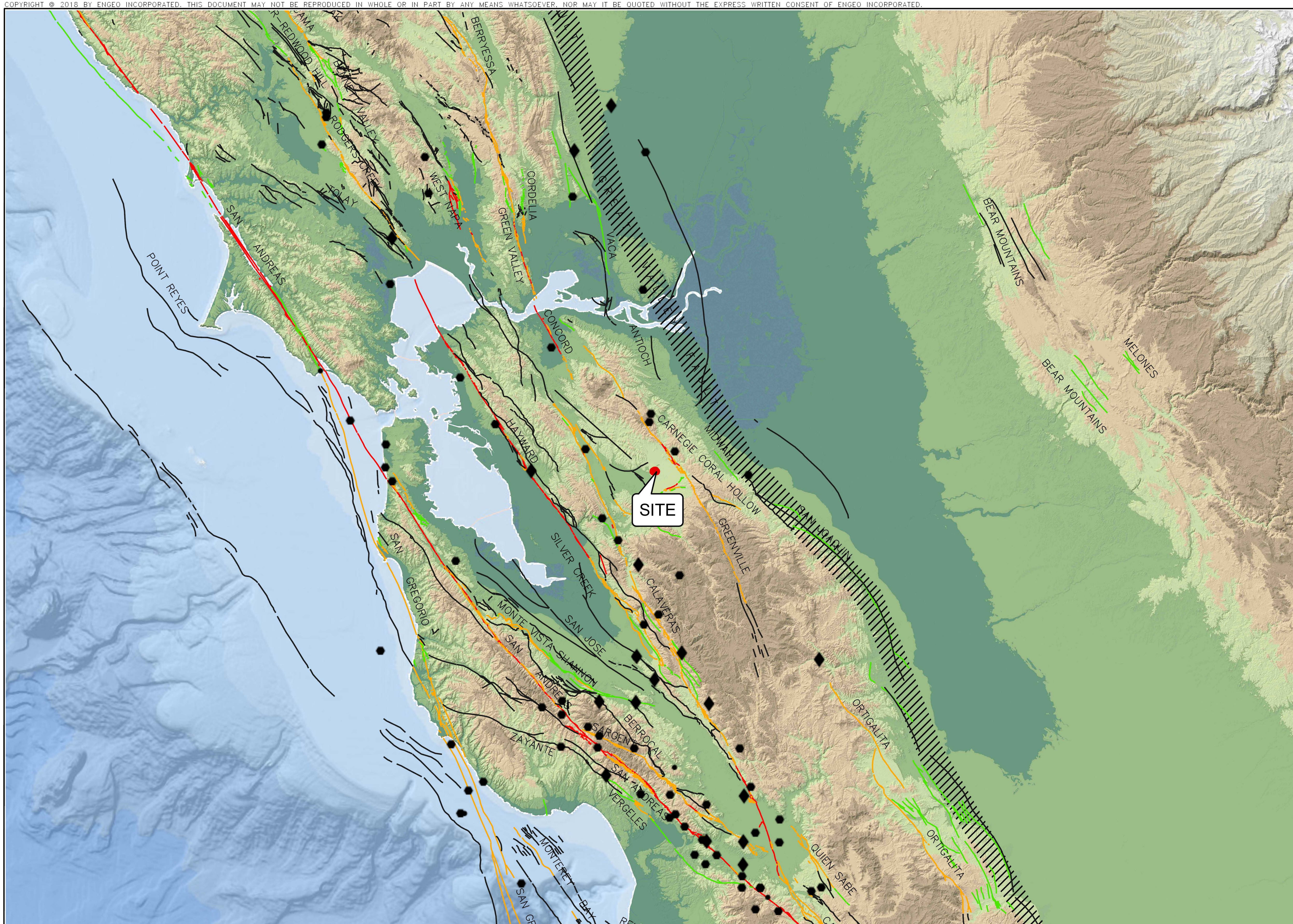
SEISMIC HAZARDS ZONE MAP

3656 LAS COLINAS ROAD
LIVERMORE, CALIFORNIA

PROJECT NO. : 15426.000.000	FIGURE NO. 4
SCALE: AS SHOWN	
DRAWN BY:QRL	CHECKED BY:AHF

PATH: G:\DRAFTING\DRAFTING2_DWG_13000 PLUS\15426\000\112018-GEX\GIS\15426.APRX
USER: QLIANG

ORIGINAL FIGURE PRINTED IN COLOR



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

3656 LAS COLINAS ROAD

LIVERMORE, CALIFORNIA

PROJECT NO.: 15426.000.000	FIGURE NO. 5
SCALE: AS SHOWN	
DRAWN BY: QRL	CHECKED BY: AHF

PATH: G:\DRAFTING\DRAFTING2_DWG_13000 PLUS\15426\000\GIS\15426.APRX

ORIGINAL FIGURE PRINTED IN COLOR

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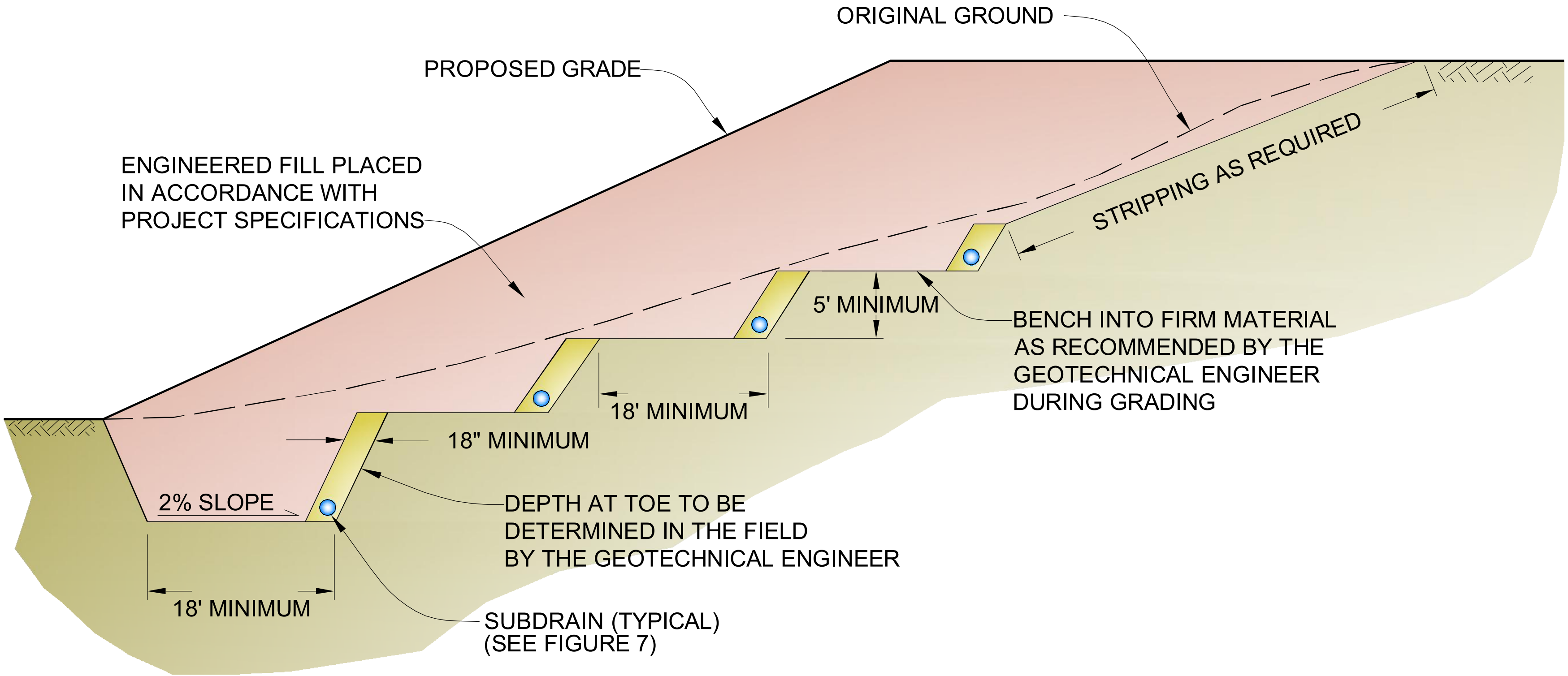
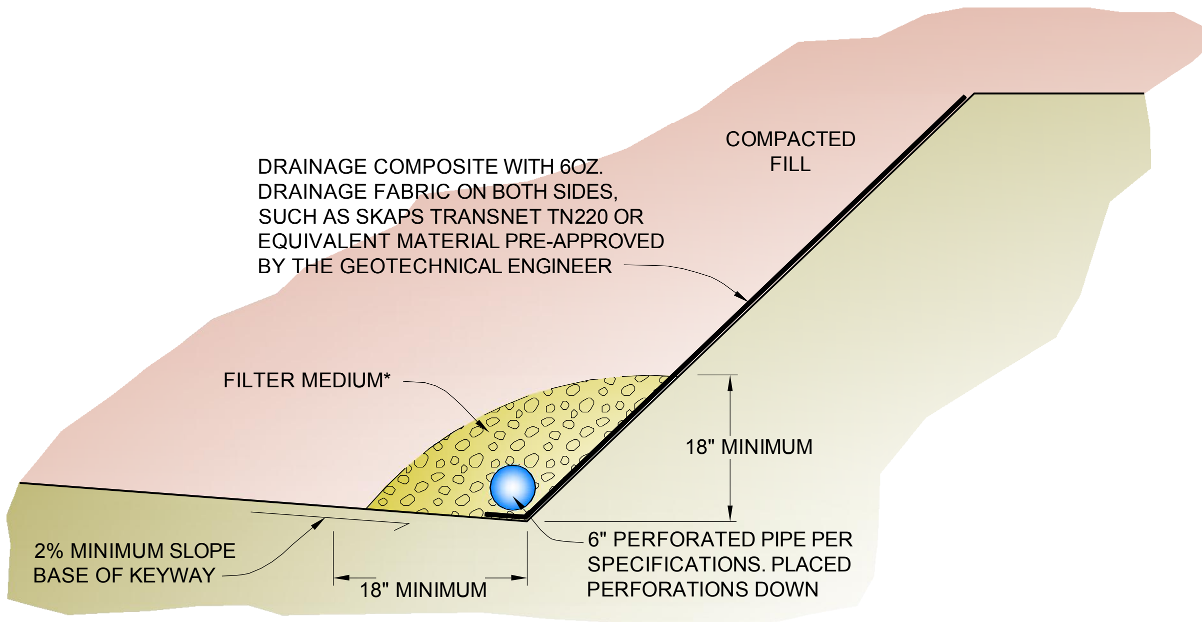
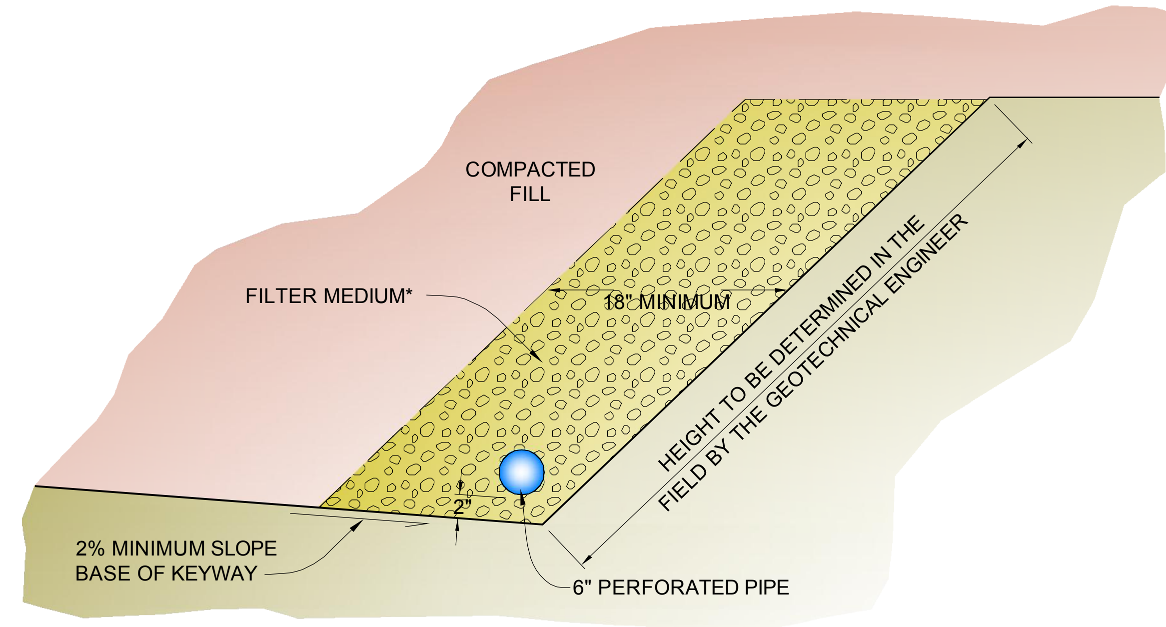


ENGEO Expect Excellence	TYPICAL KEYWAY DETAIL 3656 LAS COLINAS ROAD LIVERMORE, CALIFORNIA	PROJECT NO.: 15426.000.000 SCALE: NO SCALE DRAWN BY: LL / CHECKED BY: AHF	FIGURE NO. 6

ORIGINAL FIGURE PRINTED IN COLOR



KEYWAY SUBDRAIN - OPTION 1



KEYWAY SUBDRAIN - OPTION 2

*FILTER MEDIUM

ALTERNATIVE A

CLASS 2 PERMEABLE MATERIAL

MATERIAL SHALL CONSIST OF CLEAN, COARSE SAND AND GRAVEL OR CRUSHED STONE, CONFORMING TO THE FOLLOWING GRADING REQUIREMENTS:

SIEVE SIZE	% PASSING SIEVE
1"	100
3/4"	90-100
3/8"	40-100
#4	25-40
#8	18-33
#30	5-15
#50	0-7
#200	0-3

ALTERNATIVE B

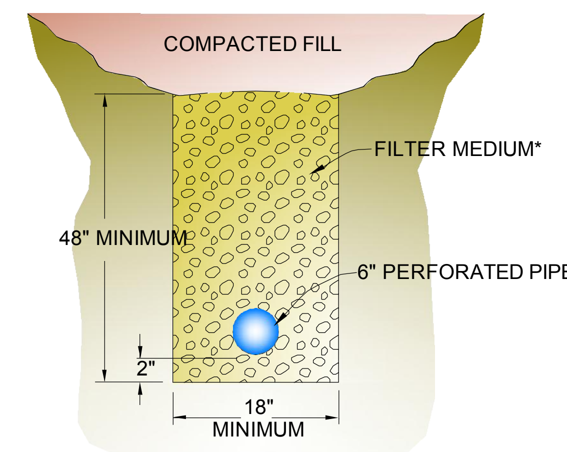
CLEAN CRUSHED ROCK OR GRAVEL WRAPPED IN FILTER FABRIC

ALL FILTER FABRIC SHALL MEET THE FOLLOWING MINIMUM AVERAGE ROLL VALUES UNLESS OTHERWISE SPECIFIED BY ENGEO:

GRAB STRENGTH (ASTM D-4632)	180 lbs
MASS PER UNIT AREA (ASTM D-4751)	6 oz/yd ^{2}
APPARENT OPENING SIZE (ASTM D-4751)	70-100 U.S. STD. SIEVE
FLOW RATE (ASTM D-4491)	80 gal/min/ft
PUNCTURE STRENGTH (ASTM D-4833)	80 lbs

NOTES:

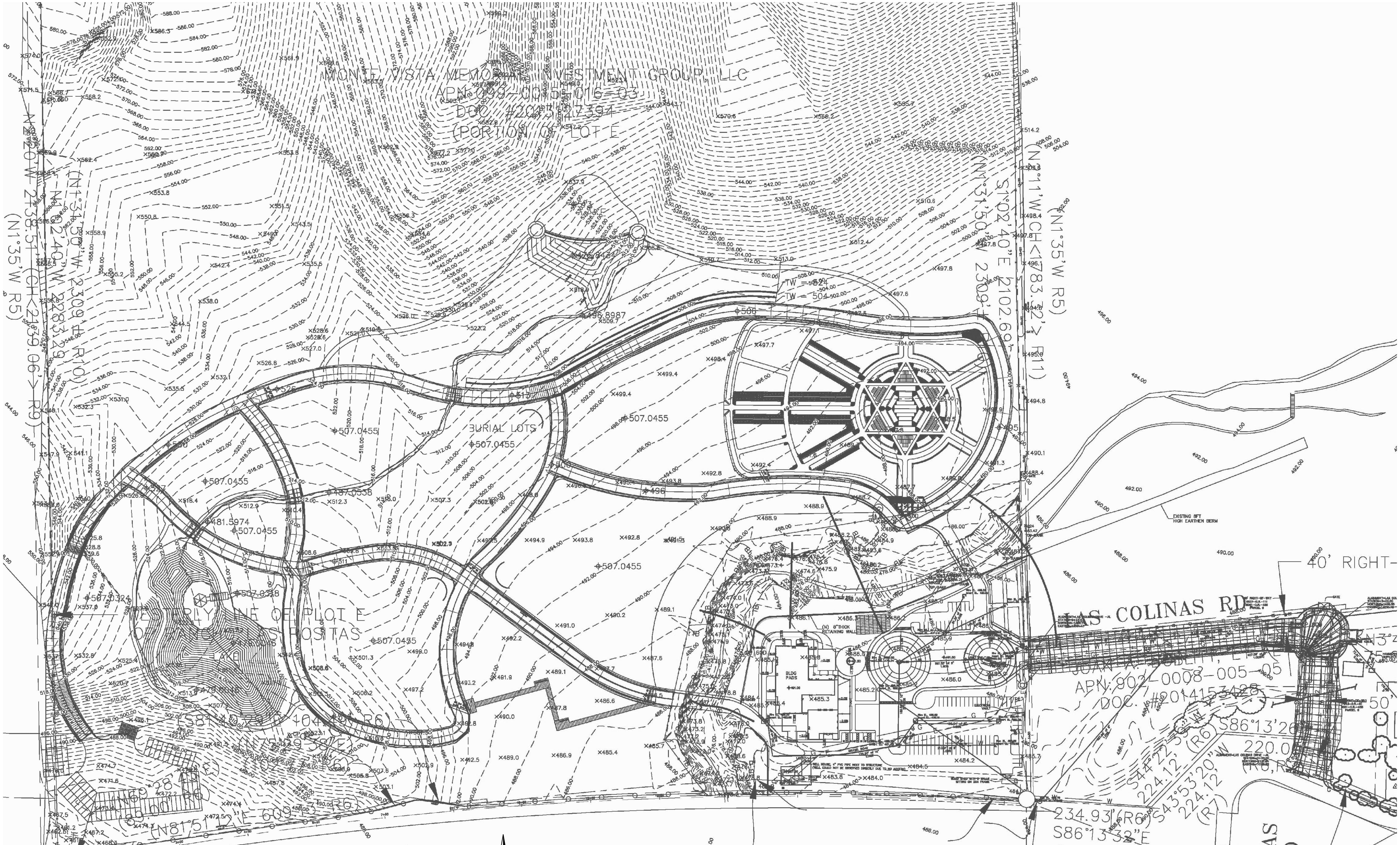
1. ALL PIPE JOINTS SHALL BE GLUED
2. ALL PERFORATED PIPE PLACED PERFORATIONS DOWN
3. 1% FALL (MINIMUM) ON ALL TRENCHES AND DRAIN LINES



SWALE SUBDRAIN

ENGEO
Expect Excellence

SUBDRAIN AND SWALE DETAILS	PROJECT NO.: 15426.000.000	FIGURE NO.
3656 LAS COLINAS ROAD	SCALE: NO SCALE	7
LIVERMORE, CALIFORNIA	DRAWN BY: LL / CHECKED BY: AHF	



BASE MAP SOURCE: ACS CONSULTING ENGINEERS, 2018

ENGEO Expect Excellence	PROPOSED DEVELOPMENT 3656 LAS COLINAS ROAD LIVERMORE, CALIFORNIA	PROJECT NO.: 15426.000.000 SCALE: AS SHOWN DRAWN BY: LL CHECKED BY: AHF FIGURE NO. 8

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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
- 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 INCORPORATED

ENGEO
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LOG OF BORING 1-B1

LATITUDE: -121.758879 LONGITUDE: 37.704003

Geotechnical Exploration
3656 Las Colinas Road
Livermore, CA
15426.000.000

DATE DRILLED: 10/8/2018
HOLE DEPTH: Approx. 51½ ft.
HOLE DIAMETER: 4.0 in.
SURF ELEV (NAVD88): Approx. 491 ft.

LOGGED / REVIEWED BY: S. Waganaar /
DRILLING CONTRACTOR: H1 Drilling
DRILLING METHOD: SFA, Switch to Mud
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
	490		FAT CLAY (CH), dark gray, hard, slightly moist, high plasticity, trace organics and fine sand.												
5	485		SANDY LEAN CLAY (CL), pale olive, hard, moist, medium plasticity, some fine to coarse sand, trace fine gravel			33	43	13	30		16	114	4417		UC
10	480		POORLY GRADED SAND WITH CLAY (SP), yellowish brown, medium dense to dense, slightly moist, sand is fine to coarse subangular to subrounded, trace fines and gravel.			30				10					
15	475		LEAN CLAY WITH GRAVEL AND SAND (CL), pale olive mottled with grayish green, very stiff, slightly moist, medium plasticity, some fine to coarse gravel, trace sand and organics.			44					17	107	2152		UC
20			switched to mud rotary												

LOG OF BORING 1-B1

ENGEO
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LATITUDE: -121.758879 LONGITUDE: 37.704003

Geotechnical Exploration
3656 Las Colinas Road
Livermore, CA
15426.000.000

DATE DRILLED: 10/8/2018
HOLE DEPTH: Approx. 51½ ft.
HOLE DIAMETER: 4.0 in.
SURF ELEV (NAVD88): Approx. 491 ft.

LOGGED / REVIEWED BY: S. Waganaar /
DRILLING CONTRACTOR: H1 Drilling
DRILLING METHOD: SFA, Switch to Mud
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
	470		SANDY LEAN CLAY (CL), pale olive, hard, moist, low to medium plasticity, trace fine sand. [LIVERMORE GRAVELS]			35	44	19	25	70	23	101	5421		UU
25			Trace organics and seams of fine sand												
	465					52								4.5*	PP
						57				87	21	107			
30	460														
			Increasing sand and gravel content			50/4"									
			POORLY GRADED GRAVEL (GP), yellowish brown, very dense, wet, fine to coarse gravel. [LIVERMORE GRAVELS]												
35	455		CLAYEY SAND (SC), dark yellowish brown to olive brown, very dense, saturated, sand is fine to coarse, subangular to subrounded, some fines content. [LIVERMORE GRAVELS]												
						50/2"				14	20				
40															

LOG OF BORING 1-B1

ENGEO — Expect Excellence

LATITUDE: -121.758879 LONGITUDE: 37.704003

Geotechnical Exploration
3656 Las Colinas Road
Livermore, CA
15426.000.000

DATE DRILLED: 10/8/2018
HOLE DEPTH: Approx. 51½ ft.
HOLE DIAMETER: 4.0 in.
SURF ELEV (NAVD88): Approx. 491 ft.

LOGGED / REVIEWED BY: S. Waganaar /
DRILLING CONTRACTOR: H1 Drilling
DRILLING METHOD: SFA, Switch to Mud
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
	450		CLAYEY SAND (SC), dark yellowish brown to olive brown, very dense, saturated, sand is fine to coarse, subangular to subrounded, some fines content. [LIVERMORE GRAVELS]												
			LEAN CLAY (CL), olive, moist, medium plasticity, medium toughness, trace fine sand and gravel, active weathering. [LIVERMORE GRAVELS]												
						56	43	20	23	90	23	104		4.5*	PP
45	445														
50			Calcium carbonate veins												
	440					40									
			End boring at 51.5 feet below ground surface. Groundwater not encountered during drilling.												

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LOG OF BORING 1-B2

LATITUDE: -121.759259 LONGITUDE: 37.703817

Geotechnical Exploration
3656 Las Colinas Road
Livermore, CA
15426.000.000

DATE DRILLED: 10/8/2018
HOLE DEPTH: Approx. 51½ ft.
HOLE DIAMETER: 4.0 in.
SURF ELEV (NAVD88): Approx. 490 ft.

LOGGED / REVIEWED BY: S. Waganaar /
DRILLING CONTRACTOR: H1 Drilling
DRILLING METHOD: SFA, Switch to Mud
HAMMER TYPE: 140 lb. Auto Trip

Depth in Feet	Elevation in Feet	DESCRIPTION	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
		FAT CLAY (CH), dark gray, hard, slightly moist, high plasticity, trace organics and fine sand.										
		SANDY LEAN CLAY (CL), pale olive to olive, stiff to very stiff, moist, medium plasticity, some fine to medium sand.										
5	485											
			14	41	16	25		18	104	1396		UC
		POORLY GRADED GRAVELLY SAND (SP), yellowish brown, medium dense, moist, gravel is fine to medium, some fine to coase subangular sand, trace fines.										
10	480											
			56									
		grades to more gravelly and becomes wet	28				8					
		switched to mud rotary										
15	475	CLAYEY SAND (SC), olive brown to dark yellowish brown, very dense, low plasticity, sand is fine to medium, some fines.										
			50/2"									
			54	22	20	2	23	20	107			
20	470											

LOG OF BORING 1-B2

LATITUDE: -121.759259 LONGITUDE: 37.703817

Expect Excellence

Geotechnical Exploration
3656 Las Colinas Road
Livermore, CA
15426.000.000

DATE DRILLED:	10/8/2018
HOLE DEPTH:	Approx. 51½ ft.
HOLE DIAMETER:	4.0 in.
SURF ELEV (NAVD88):	Approx. 490 ft.
LOGGED / REVIEWED BY:	S. Waganaar /
DRILLING CONTRACTOR:	H1 Drilling
DRILLING METHOD:	SFA, Switch to Mud
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
			CLAYEY SAND (SC), olive brown to dark yellowish brown, very dense, low plasticity, sand is fine to medium, some fines. increasing fines conent			34				30	17				
			switched to mud rotatary												
25	465		LEAN CLAY (CL), pale olive, very stiff to hard, moist, low to medium plasticity, trace fines. [LIVERMORE GRAVELS]			36								4.0*	PP
			increasing sand content			68	29	16	13		16	96	5965		UC
30	460														
			decreasing sand content			50					26	100	6284		UU
35	455														
			interbedded fine sand seams			62									
40	450														

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LOG OF BORING 1-B2

LATITUDE: -121.759259 LONGITUDE: 37.703817

Geotechnical Exploration
3656 Las Colinas Road
Livermore, CA
15426.000.000

DATE DRILLED: 10/8/2018
HOLE DEPTH: Approx. 51½ ft.
HOLE DIAMETER: 4.0 in.
SURF ELEV (NAVD88): Approx. 490 ft.

LOGGED / REVIEWED BY: S. Waganaar /
DRILLING CONTRACTOR: H1 Drilling
DRILLING METHOD: SFA, Switch to Mud
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
			LEAN CLAY (CL), pale olive, very stiff to hard, moist, low to medium plasticity, trace fines. [LIVERMORE GRAVELS]												
			POORLY GRADED SAND (SP), yellowish brown, medium dense, saturated, sand is fine to medium grained. [LIVERMORE GRAVELS]			55					20	107		4.0*	PP
45	445		SANDY LEAN CLAY (CL), pale olive to olive, very stiff, moist, low to medium plasticity, some fine sand, calcium carbonate veins. [LIVERMORE GRAVELS]												
50	440		color changes to olive and becomes hard, decreasing sand content			48								4.5*	PP
			End boring at 51.5 feet below ground surface. Groundwater encountered at approximately 14 feet below ground surface.												

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LOG OF BORING 1-B3

LATITUDE: -121.757617　　LONGITUDE: 37.704775

Geotechnical Exploration
3656 Las Colinas Road
Livermore, CA
15426.000.000

DATE DRILLED: 10/9/2018
HOLE DEPTH: Approx. 50 ft.
HOLE DIAMETER: 4.0 in.
SURF ELEV (NAVD88): Approx. 491 ft.

LOGGED / REVIEWED BY: S. Waganaar /
DRILLING CONTRACTOR: H1 Drilling
DRILLING METHOD: SFA, Switch to Mud
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
			FAT CLAY (CH), dark gray, hard, slightly moist, high plasticity, trace organics and fine sand.												
	490														
5															
	485		increasing sand content, trace weathered gravels			47	56	17	39		18	93	7558		UC
			CLAYEY SAND (SC), olive to grayish green, loose, moist, sand is fine to medium, subrounded to subangular, some fines.												
10															
	480					10									
			LEAN CLAY (CL), olive to olive brown, stiff, moist, medium plasticity, trace fine sand.												
						15									
			increasing sand content		▽										
15															
	475		CLAYEY SAND (SC), olive to olive brown, medium dense, moist, sand is fine to coarse, trace fines.			58	35	17	18	19	16	118			
			switched to mud rotary												
						20									
20															

LOG OF BORING 1-B3

ENGEO
Expect Excellence

LATITUDE: -121.757617 LONGITUDE: 37.704775

Geotechnical Exploration
3656 Las Colinas Road
Livermore, CA
15426.000.000

DATE DRILLED: 10/9/2018
HOLE DEPTH: Approx. 50 ft.
HOLE DIAMETER: 4.0 in.
SURF ELEV (NAVD88): Approx. 491 ft.

LOGGED / REVIEWED BY: S. Waganaar /
DRILLING CONTRACTOR: H1 Drilling
DRILLING METHOD: SFA, Switch to Mud
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
	470		LEAN CLAY (CL), olive to yellowish brown, hard, moist, medium plasticity, trace fine sand and gravel. [LIVERMORE GRAVELS]			59					24	102	6196		UU
25	465		POORLY GRADED GRAVEL (SP), olive brown, very dense, saturated, sand is fine to coarse, trace fines and gravel. [LIVERMORE GRAVELS]			50/3"									
			CLAYEY SAND (SC), olive brown, dense, saturated, sand is fine to coarse, subrounded, some fines and trace gravel.[LIVERMORE GRAVELS]			40					13				
30	460		LEAN CLAY (CL), light grayish green mottled with reddish yellow, hard, moist, low to medium plasticity, trace sand and gravel. [LIVERMORE GRAVELS]			29									
35	455		increasing plasticity, slow dilation			61					25	101	7025		UU
40															

ENGEO
Expect Excellence

LOG OF BORING 1-B3

LATITUDE: -121.757617 LONGITUDE: 37.704775

Geotechnical Exploration
3656 Las Colinas Road
Livermore, CA
15426.000.000

DATE DRILLED: 10/9/2018
HOLE DEPTH: Approx. 50 ft.
HOLE DIAMETER: 4.0 in.
SURF ELEV (NAVD88): Approx. 491 ft.

LOGGED / REVIEWED BY: S. Waganaar /
DRILLING CONTRACTOR: H1 Drilling
DRILLING METHOD: SFA, Switch to Mud
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
	450		LEAN CLAY (CL), light grayish green mottled with reddish yellow, hard, moist, low to medium plasticity, trace sand and gravel. [LIVERMORE GRAVELS]												
			color change to olive mottled with light grayish green, trace weathered gravels and calcium carbonate veins			50/5"									
45	445														
			color change to olive brown			72					25	101			
50			End boring at 50.0 feet below ground surface. Groundwater encountered at approximately 15 feet below ground surface.												

LOG - GEOTECHNICAL_SU+QU W/ ELEV 10042018 GEX BX.GPJ ENGEO INC.GDT 12/13/18

ENGEO
Expect Excellence

LOG OF BORING 1-B4

LATITUDE: -121.760394 LONGITUDE: 37.706317

Geotechnical Exploration
3656 Las Colinas Road
Livermore, CA
15426.000.000

DATE DRILLED: 10/9/2018
HOLE DEPTH: Approx. 49½ ft.
HOLE DIAMETER: 4.0 in.
SURF ELEV (NAVD88): Approx. 525 ft.

LOGGED / REVIEWED BY: S. Waganaar /
DRILLING CONTRACTOR: H1 Drilling
DRILLING METHOD: SFA, Switch to Mud
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
			SANDY FAT CLAY (CH), dark gray, hard, slightly moist, high plasticity												
			SILTY SAND (SM), light yellowish brown, dense, slightly moist, fine to coarse sand, some fines content, trace fine gravel.												
5	520														
						46				50	15	103			
10	515														
			color change to yellowish brown, decreasing gravel content												
						55									
			FAT CLAY (CH), yellowish brown, hard, moist, high plasticity, trace fine sand.											>4.5*	PP
15	510														
						36	52	20	32		28	91.3	1484		UC
			LEAN CLAY (CL), yellowish brown mottled with olive, hard, moist, trace fine sand and calcium carbonate. [LIVERMORE GRAVELS]												
20	505														

ENGEO
Expect Excellence

LOG OF BORING 1-B4

LATITUDE: -121.760394 LONGITUDE: 37.706317

Geotechnical Exploration
3656 Las Colinas Road
Livermore, CA
15426.000.000

DATE DRILLED: 10/9/2018
HOLE DEPTH: Approx. 49½ ft.
HOLE DIAMETER: 4.0 in.
SURF ELEV (NAVD88): Approx. 525 ft.

LOGGED / REVIEWED BY: S. Waganaar /
DRILLING CONTRACTOR: H1 Drilling
DRILLING METHOD: SFA, Switch to Mud
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
			LEAN CLAY (CL), yellowish brown mottled with olive, hard, moist, trace fine sand and calcium carbonate. [LIVERMORE GRAVELS]			43					28	96.3	3650		UU
			switched to mud rotary												
25	500														
			decreasing sand content			60									
30	495		LEAN CLAY (CL), olive mottled with yellowish olive, hard, moist, low to medium plasticity, some fine sand, trace carbonates. [LIVERMORE GRAVELS]												
						31									
						24				88	27	96.1			
35	490														
			decreasing sand content			82									
40	485														

ENGEO
Expect Excellence

LOG OF BORING 1-B4

LATITUDE: -121.760394 LONGITUDE: 37.706317

Geotechnical Exploration
3656 Las Colinas Road
Livermore, CA
15426.000.000

DATE DRILLED: 10/9/2018
HOLE DEPTH: Approx. 49½ ft.
HOLE DIAMETER: 4.0 in.
SURF ELEV (NAVD88): Approx. 525 ft.

LOGGED / REVIEWED BY: S. Waganaar /
DRILLING CONTRACTOR: H1 Drilling
DRILLING METHOD: SFA, Switch to Mud
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
			SILTY SAND (SM), olive, very dense, moist, sand is fine to coarse, trace fine gravel. [LIVERMORE GRAVELS]												
						77				46	20	108			
45	480														
			increasing gravel content			50/5"									
			End boring at 49.5 feet below ground surface. Groundwater not encountered during drilling.												

LOG OF BORING 1-B5

LATITUDE: -121.76312 LONGITUDE: 37.703837

Geotechnical Exploration
3656 Las Colinas Road
Livermore, CA
15426.000.000

DATE DRILLED: 10/10/2018
HOLE DEPTH: Approx. 50 ft.
HOLE DIAMETER: 4.0 in.
SURF ELEV (NAVD88): Approx. 519 ft.

LOGGED / REVIEWED BY: A. Light /
DRILLING CONTRACTOR: H1 Drilling
DRILLING METHOD: SFA, Switch to Mud
HAMMER TYPE: 140 lb. Auto Trip

Depth in Feet	Elevation in Feet	DESCRIPTION	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
		FAT CLAY (CH), dark gray, medium stiff to stiff, slightly moist, high plasticity, some fine gravel.										
	515											
5		SILTY SAND (SM), gray with light yellowish brown, medium dense, slightly moist, sand is fine grained, some fines.	32				46					
	510											
10		color changes to olive, increasing fines content	35									
		LEAN CLAY (CL), olive, hard, moist, low to medium plasticity, trace fine sand. [LIVERMORE GRAVELS]									>4.5*	PP
			26	42	20	22		23				
	505	SANDY LEAN CLAY (CL), olive mottled with yellowish brown, hard, moist, low to medium plasticity, trace fine to coarse sand. [LIVERMORE GRAVELS]										
15			47				72	22	117	5047		UC
		switched to mud rotary										
	500											
20												

LOG OF BORING 1-B5

ENGEO
Expect Excellence

LATITUDE: -121.76312 LONGITUDE: 37.703837

Geotechnical Exploration
3656 Las Colinas Road
Livermore, CA
15426.000.000

DATE DRILLED: 10/10/2018
HOLE DEPTH: Approx. 50 ft.
HOLE DIAMETER: 4.0 in.
SURF ELEV (NAVD88): Approx. 519 ft.

LOGGED / REVIEWED BY: A. Light /
DRILLING CONTRACTOR: H1 Drilling
DRILLING METHOD: SFA, Switch to Mud
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
			LEAN CLAY (CL), olive, hard, moist, low to medium plasticity, trace sand and carbonates. [LIVERMORE GRAVELS]			52					32	106		>4.5*	PP
25	495														
						42	46	21	25	83	25	100			
30	490		SANDY SILT (ML), olive, hard, moist, trace fine sand. [LIVERMORE GRAVELS]												
						50				57	18	111			
35	485		LEAN CLAY (CL), olive, hard, moist, low to medium plasticity, trace fine to coarse sand. [LIVERMORE GRAVELS]												
						45	40	20	20	93					
40	480														

LOG OF BORING 1-B5

ENGEO Expect Excellence

LATITUDE: -121.76312 LONGITUDE: 37.703837

Geotechnical Exploration
3656 Las Colinas Road
Livermore, CA
15426.000.000

DATE DRILLED: 10/10/2018
HOLE DEPTH: Approx. 50 ft.
HOLE DIAMETER: 4.0 in.
SURF ELEV (NAVD88): Approx. 519 ft.

LOGGED / REVIEWED BY: A. Light /
DRILLING CONTRACTOR: H1 Drilling
DRILLING METHOD: SFA, Switch to Mud
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
			LEAN CLAY (CL), olive mottled with gray, hard, moist, low to medium plasticity, trace fine to coarse sand and carbonates. [LIVERMORE GRAVELS]												
						45					26	97			
	475														
45															
	470		increasing fine sand content			46									
50			End boring at 50.0 feet below ground surface. Groundwater not encountered during drilling.												

ENGEO
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LOG OF BORING 1-B6

LATITUDE: -121.758387 LONGITUDE: 37.705501

Geotechnical Exploration
3656 Las Colinas Road
Livermore, CA
15426.000.000

DATE DRILLED: 10/10/2018
HOLE DEPTH: Approx. 51½ ft.
HOLE DIAMETER: 4.0 in.
SURF ELEV (NAVD88): Approx. 495 ft.

LOGGED / REVIEWED BY: A. Light /
DRILLING CONTRACTOR: H1 Drilling
DRILLING METHOD: SFA, Switch to Mud
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
			FAT CLAY (CH), dark gray to grayish brown, stiff, slightly moist, high plasticity, trace organics and fine sand.												
5	490		color changes pale olive to olive.			27	67	19	48	48	23	102		1.5*	PP
10	485		LEAN CLAY WITH GRAVEL (CL), olive to light grayish green, medium stiff, moist, medium plasticity, trace weathered gravel.			18					26	103	987		UC
15	480		CLAYEY SAND (SC), olive to light grayish green, medium dense, saturated, sand is fine to coarse, trace fines and weathered gravels. [LIVERMORE GRAVELS]		▽	32									
						24				13	19				
			switched to mud rotary												
			color changes to olive brown												
20	475														

LOG - GEOTECHNICAL_SU+QU W/ ELEV 10042018 GEX BX.GPJ ENGEO INC.GDT 12/13/18

LOG OF BORING 1-B6

LATITUDE: -121.758387 LONGITUDE: 37.705501

Geotechnical Exploration
3656 Las Colinas Road
Livermore, CA
15426.000.000

DATE DRILLED: 10/10/2018
HOLE DEPTH: Approx. 51½ ft.
HOLE DIAMETER: 4.0 in.
SURF ELEV (NAVD88): Approx. 495 ft.

LOGGED / REVIEWED BY: A. Light /
DRILLING CONTRACTOR: H1 Drilling
DRILLING METHOD: SFA, Switch to Mud
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
			LEAN CLAY WITH SAND (CL), olive mottled with yellowish brown, hard, moist, medium plasticity, trace fine to medium sand, calcium carbonate veins. [LIVERMORE GRAVELS]			36								4.5*	PP
25	470		GRAVELLY LEAN CLAY WITH SAND (CL), olive mottled with yellowish brown, hard, moist, medium plasticity, trace fine to medium sand, calcium carbonate veins. [LIVERMORE GRAVELS]			30				53					
30	465		increasing sand content			47					21	109		3.75*	PP
35	460		trace gravel			50/5"									
40	455														

ENGEO
Expect Excellence

LOG OF BORING 1-B6

LATITUDE: -121.758387 LONGITUDE: 37.705501

Geotechnical Exploration
3656 Las Colinas Road
Livermore, CA
15426.000.000

DATE DRILLED: 10/10/2018
HOLE DEPTH: Approx. 51½ ft.
HOLE DIAMETER: 4.0 in.
SURF ELEV (NAVD88): Approx. 495 ft.

LOGGED / REVIEWED BY: A. Light /
DRILLING CONTRACTOR: H1 Drilling
DRILLING METHOD: SFA, Switch to Mud
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
			CLAYEY GRAVEL (GP), olive brown mottled with white, dense, moist, gravels are weathered and friable, some fines and trace fine to coarse sand. [LIVERMORE GRAVELS]			76									
			SANDY CLAY (CL), olive brown, dense, moist, low plasticity, some fine sand. [LIVERMORE GRAVELS]												
45	450		LEAN CLAY WITH SAND (CL), olive mottled with light grayish green, hard, moist, medium plasticity, friable, trace fine to coarse sand and calcium carbonate veins. [LIVERMORE GRAVELS]			74									
50	445					29									
			End boring at 51.5 feet below ground surface. Groundwater encountered at approximately 16 feet below ground surface.												

ENGEO
Expect Excellence

LOG OF BORING 1-B7

LATITUDE: -121.754982 LONGITUDE: 37.703983

Geotechnical Exploration
3656 Las Colinas Road
Livermore, CA
15426.000.000

DATE DRILLED: 10/4/2018
HOLE DEPTH: Approx. 16½ ft.
HOLE DIAMETER: 4.0 in.
SURF ELEV (NAVD88): Approx. 492 ft.

LOGGED / REVIEWED BY: B. Xu /
DRILLING CONTRACTOR: West Coast Exploration
DRILLING METHOD: Solid Flight Auger
HAMMER TYPE: 140 lb. Rope and Cathead

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
			LEAN CLAY (CL), dark gray to brownish gray, very stiff, moist												
	490														
			With coarse gravels			24					22				
5					▽										
						10								2.75*	PP
			Increasing fine to coarse grained sand												
			Becomes soft increasing sand content												
	485														
			SILTY SAND WITH GRAVEL (SM), gray, loose, wet												
10															
						7				27	20				
	480														
15															
			color changes to light yellowish brown and becomes dense			43									
			End boring at approximately 16.5 feet below ground surface. Groundwater was encountered at approximately 10 feet below ground surface.												

LOG OF BORING 1-B8

ENGEO
Expect Excellence

LATITUDE: -121.756418 LONGITUDE: 37.704111

Geotechnical Exploration
3656 Las Colinas Road
Livermore, CA
15426.000.000

DATE DRILLED: 10/4/2018
HOLE DEPTH: Approx. 16½ ft.
HOLE DIAMETER: 4.0 in.
SURF ELEV (NAVD88): Approx. 492 ft.

LOGGED / REVIEWED BY: B. Xu /
DRILLING CONTRACTOR: West Coast Exploration
DRILLING METHOD: Solid Flight Auger
HAMMER TYPE: 140 lb. Rope and Cathead

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
			LEAN CLAY (CL), gray to brownish gray, very stiff, moist												
	490														
5						27					34	87		1.75*	PP
	485														
10			color changes to brownish gray mottled with greenish yellow			27								2.5*	PP
	480														
			SILTY SAND (SM), gray, medium dense, very moist, some fines.		∇										
15						24									
			End boring at approximately 16.5 feet below ground surface. Groundwater was encountered at approximately 14.5 ft below ground surface.												

ENGEO
Expect Excellence

LOG OF BORING 1-B9

LATITUDE: -121.757068 LONGITUDE: 37.704082

Geotechnical Exploration
3656 Las Colinas Road
Livermore, CA
15426.000.000

DATE DRILLED: 10/4/2018
HOLE DEPTH: Approx. 11½ ft.
HOLE DIAMETER: 4.0 in.
SURF ELEV (NAVD88): Approx. 491 ft.

LOGGED / REVIEWED BY: B. Xu /
DRILLING CONTRACTOR: West Coast Exploration
DRILLING METHOD: Solid Flight Auger
HAMMER TYPE: 140 lb. Rope and Cathead

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
	490		FAT CLAY (CH), gray to brownish gray, very stiff, moist, high plasticity												
5						27								2.75*	PP
			POORLY GRADED SAND (SP), yellowish red mottled with very dark gray, moist, trace fines.												
	485		SANDY CLAY (CL), pale olive mottled with very dark gray, very stiff, moist, medium plasticity, trace fine sand.												
						22					24	98		2.75*	PP
10			color changes to dark grayish brown, becomes medium stiff to stiff												
	480					30								0.75*	PP
			color changes to pale olive mottled with dark gray											1.75*	PP
			End boring at approximately 11.5 feet below ground surface. Groundwater was not encountered during drilling.												

LOG OF BORING 1-B10

ENGEO Expect Excellence

LATITUDE: -121.75818 LONGITUDE: 37.703375

Geotechnical Exploration
3656 Las Colinas Road
Livermore, CA
15426.000.000

DATE DRILLED: 10/4/2018
HOLE DEPTH: Approx. 11½ ft.
HOLE DIAMETER: 4.0 in.
SURF ELEV (NAVD88): Approx. 489 ft.

LOGGED / REVIEWED BY: B. Xu /
DRILLING CONTRACTOR: West Coast Exploration
DRILLING METHOD: Solid Flight Auger
HAMMER TYPE: 140 lb. Rope and Cathead

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
			LEAN SILTY CLAY (CL), gray to brownish gray, very stiff, moist to slightly moist, medium plasticity, some fine sand, trace of organics												
	485					22					25	83			
5			No organics, becomes hard												
						22								4.5*	PP
	480														
10			color changes to olive mottled with gray												
						27								2.5*	PP
			End boring at approximately 11.5 feet below ground surface. Groundwater was not encountered during drilling.												

LOG - GEOTECHNICAL_SU+QU W/ ELEV 10042018 GEX BX.GPJ ENGEO INC.GDT 12/13/18

ENGEO
Expect Excellence

LOG OF BORING 1-B11

LATITUDE: -121.758798 LONGITUDE: 37.704886

Geotechnical Exploration
3656 Las Colinas Road
Livermore, CA
15426.000.000

DATE DRILLED:	10/4/2018	LOGGED / REVIEWED BY:	B. Xu /
HOLE DEPTH:	Approx. 16½ ft.	DRILLING CONTRACTOR:	West Coast Exploration
HOLE DIAMETER:	4.0 in.	DRILLING METHOD:	Solid Flight Auger
SURF ELEV (NAVD88):	Approx. 494 ft.	HAMMER TYPE:	140 lb. Rope and Cathead

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
			SANDY LEAN CLAY (CL), dark yellowish brown, hard, moist, some fine-to-coarse sand, with fine gravel.												
	490					27	46	13	33		18	81		>4.5*	PP
5			Increasing sand content.			40								>4.5*	PP
	485														
10			POORLY GRADED GRAVEL (GP), pale yellow to pale olive, medium dense, moist			35									
	480		SANDY SILT (ML), pale yellow mottled with yellowish red, medium dense, moist												
15						20				77					
			End boring at approximately 16.5 feet below ground surface. Groundwater was not encountered during drilling.												

LOG OF BORING 1-B12

ENGEO
Expect Excellence

LATITUDE: -121.757627 LONGITUDE: 37.704127

Geotechnical Exploration
3656 Las Colinas Road
Livermore, CA
15426.000.000

DATE DRILLED: 10/4/2018
HOLE DEPTH: Approx. 21½ ft.
HOLE DIAMETER: 4.0 in.
SURF ELEV (NAVD88): Approx. 491 ft.

LOGGED / REVIEWED BY: B. Xu /
DRILLING CONTRACTOR: West Coast Exploration
DRILLING METHOD: Solid Flight Auger
HAMMER TYPE: 140 lb. Rope and Cathead

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
			FAT CLAY (CH), dark gray, hard, moist, trace fine sand.												
	490														
5	485					42					25	92	5494		UC
10	480		color changes to light gray to pale olive, increasing sand content, becomes very stiff			26					19	109		3.25*	PP
			SANDY SILT (ML), dark gray, trace fines content and fine gravel.												
15	475					25				76					
						27									
20															

LOG OF BORING 1-B12

ENGEO Expect Excellence

LATITUDE: -121.757627 LONGITUDE: 37.704127

Geotechnical Exploration
3656 Las Colinas Road
Livermore, CA
15426.000.000

DATE DRILLED: 10/4/2018
HOLE DEPTH: Approx. 21½ ft.
HOLE DIAMETER: 4.0 in.
SURF ELEV (NAVD88): Approx. 491 ft.

LOGGED / REVIEWED BY: B. Xu /
DRILLING CONTRACTOR: West Coast Exploration
DRILLING METHOD: Solid Flight Auger
HAMMER TYPE: 140 lb. Rope and Cathead

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
	470		SANDY SILT (ML), dark gray, trace fines content and fine gravel. color changes to reddish brown with olive			21								2.5*	PP
			End boring at approximately 21.5 feet below ground surface. Groundwater was encountered at approximately 14 feet below ground surface.												

LOG - GEOTECHNICAL_SU+QU W/ ELEV 10042018 GEX BX.GPJ ENGEO INC.GDT 12/13/18

LOG OF BORING 1-B13

ENGEO — Expect Excellence

LATITUDE: -121.760909 LONGITUDE: 37.704085

Geotechnical Exploration
3656 Las Colinas Road
Livermore, CA
15426.000.000

DATE DRILLED: 10/4/2018
HOLE DEPTH: Approx. 21½ ft.
HOLE DIAMETER: 4.0 in.
SURF ELEV (NAVD88): Approx. 496 ft.

LOGGED / REVIEWED BY: B. Xu /
DRILLING CONTRACTOR: West Coast Exploration
DRILLING METHOD: Solid Flight Auger
HAMMER TYPE: 140 lb. Rope and Cathead

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
	495		SILTY SAND WITH GRAVEL (SM), yellowish brown, very dense, moist, some fine gravel and trace fines.												
5	490					50/6"				15					
			Trace coarse gravel.												
10	485		color changes to reddish yellow			50/6"					19	106			
15	480		LEAN CLAY (CL), yellowish brown, hard, moist, with trace of fine sands. [LIVERMORE GRAVELS]			38								>4.5*	PP
20															

LOG OF BORING 1-B13

ENGEO
Expect Excellence

LATITUDE: -121.760909 LONGITUDE: 37.704085

Geotechnical Exploration
3656 Las Colinas Road
Livermore, CA
15426.000.000

DATE DRILLED: 10/4/2018
HOLE DEPTH: Approx. 21½ ft.
HOLE DIAMETER: 4.0 in.
SURF ELEV (NAVD88): Approx. 496 ft.

LOGGED / REVIEWED BY: B. Xu /
DRILLING CONTRACTOR: West Coast Exploration
DRILLING METHOD: Solid Flight Auger
HAMMER TYPE: 140 lb. Rope and Cathead

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
	475		LEAN CLAY (CL), yellowish brown, hard, moist, with trace of fine sands. [LIVERMORE GRAVELS]			50/6"					25	100		>4.5*	PP
			End boring at approximately 21.5 feet below ground surface. Groundwater was not encountered during drilling.												

ENGEO
Expect Excellence

LOG OF BORING 1-B14

LATITUDE: -121.760515 LONGITUDE: 37.705616

Geotechnical Exploration
3656 Las Colinas Road
Livermore, CA
15426.000.000

DATE DRILLED: 10/4/2018
HOLE DEPTH: Approx. 11½ ft.
HOLE DIAMETER: 4.0 in.
SURF ELEV (NAVD88): Approx. 508 ft.

LOGGED / REVIEWED BY: B. Xu /
DRILLING CONTRACTOR: West Coast Exploration
DRILLING METHOD: Solid Flight Auger
HAMMER TYPE: 140 lb. Rope and Cathead

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
			SANDY CLAY (CL), dark yellowish brown, hard, moist, some medium to coarse sand, with trace gravels.												
	505					50				70	45	74		>4.5*	PP
5			Some fine to coarse sand and fine gravel			65								>4.5*	PP
	500														
10			Trace fine-to-medium-grained sands			50/6"								>4.5*	PP
			End boring at approximately 11.5 feet below ground surface. Groundwater was not encountered during drilling.												

LOG OF BORING 1-B15

ENGEO
Expect Excellence

LATITUDE: -121.755233 LONGITUDE: 37.704226

Geotechnical Exploration
3656 Las Colinas Road
Livermore, CA
15426.000.000

DATE DRILLED: 10/4/2018
HOLE DEPTH: Approx. 11½ ft.
HOLE DIAMETER: 4.0 in.
SURF ELEV (NAVD88): Approx. 492 ft.

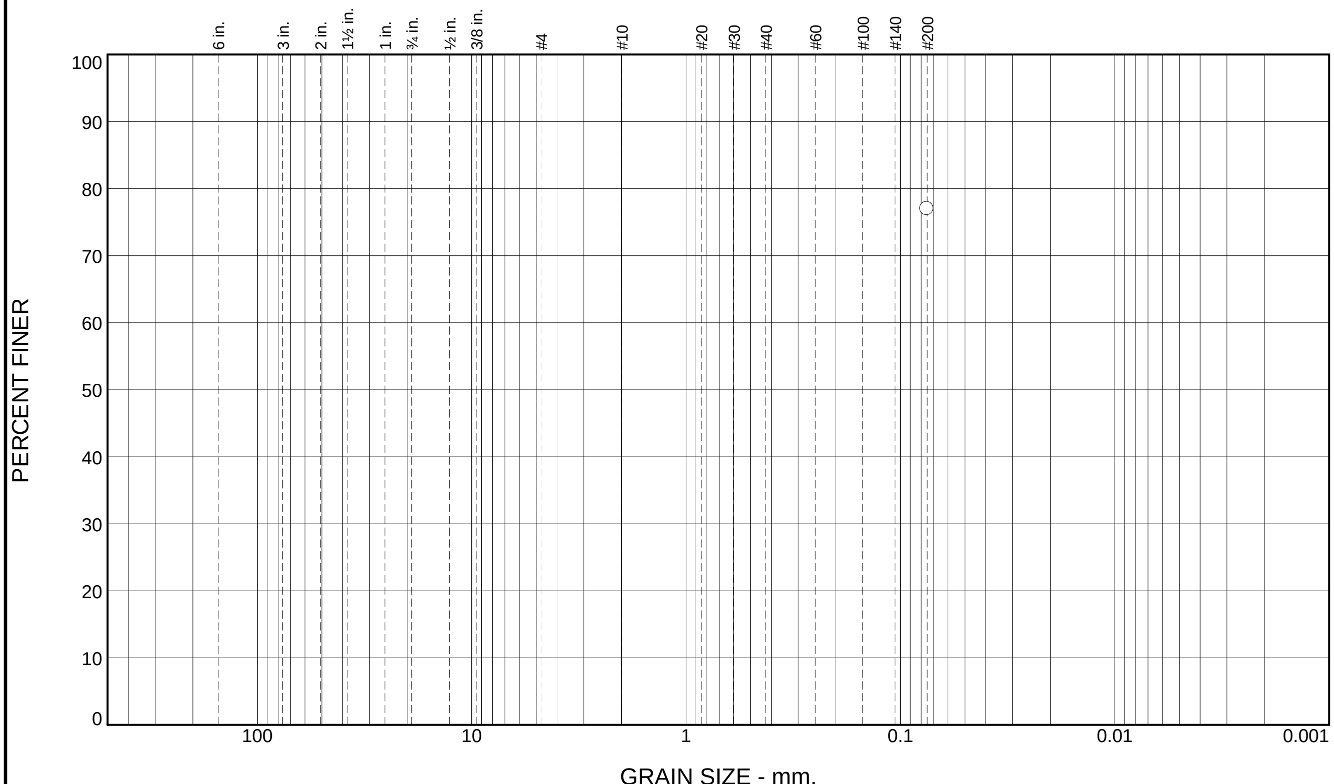
LOGGED / REVIEWED BY: B. Xu /
DRILLING CONTRACTOR: West Coast Exploration
DRILLING METHOD: Solid Flight Auger
HAMMER TYPE: 140 lb. Rope and Cathead

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
			LEAN CLAY (CL), brownish gray with pale olive												
	490														
			FAT CLAY (CH), very dark gray, medium stiff, moist, high plasticity			16								1.25*	PP
5															
						20	63	20	43		31				
	485														
10			color changes to dark gray to brownish gray												
						24								1*	PP
			End boring at approximately 11.5 feet below ground surface. Groundwater was not encountered during drilling.												

APPENDIX B

LABORATORY TEST DATA

Particle Size Distribution Report



% +75mm	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
						77.0	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	77.0		

* (no specification provided)

Material 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= AASHTO=

Remarks

ASTM D1140, Method B
Dry Sample Weight = 138.07; Soak Time = 31.81

Sample Number: 1-B11 @ 15

Date: 11/5/2018

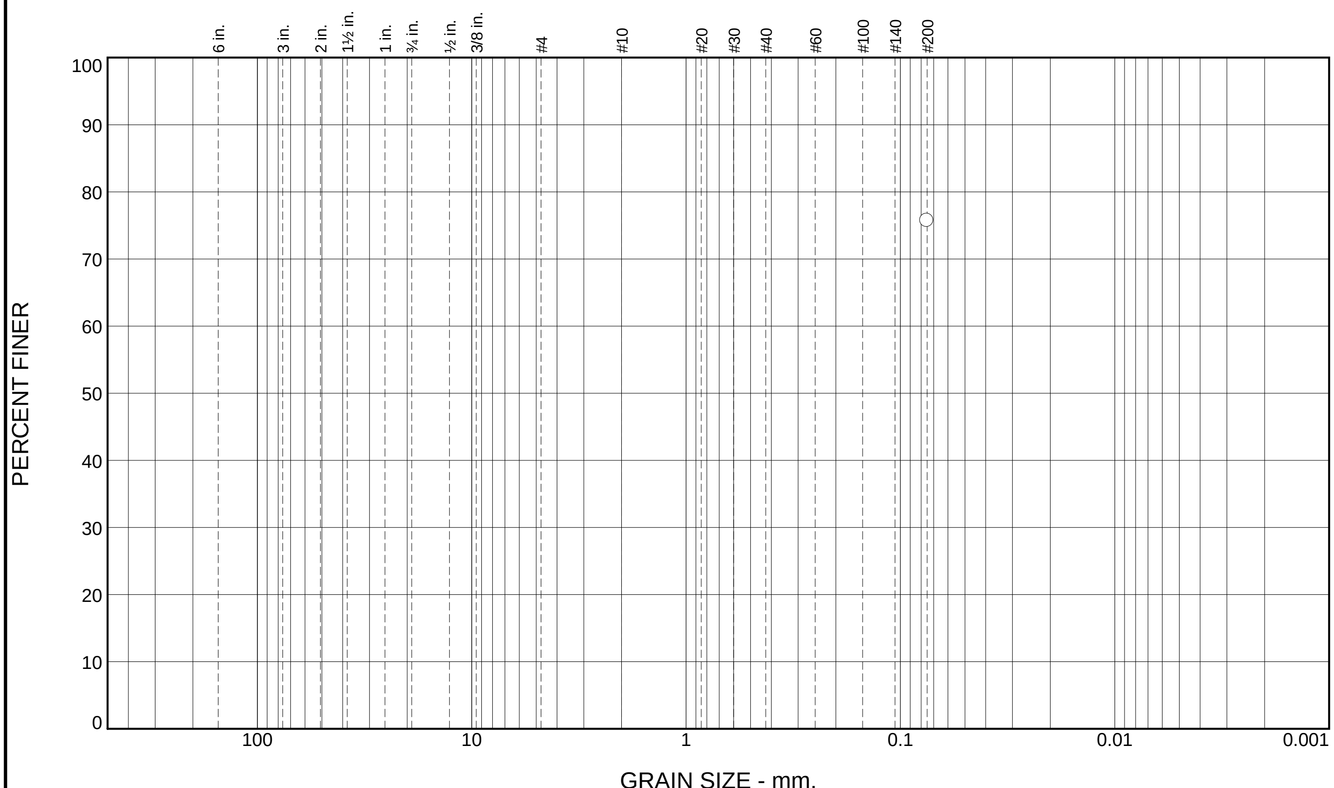
ENGEO INCORPORATED

Client: Monte Vista Memorial Investment Group, LLC
Project: 3656 Las Colinas Road

Project No: 15426.000.000

Tested By: M. Bromfield **Checked By:** M. Quasem

Particle Size Distribution Report



% +75mm	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
						75.7	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	75.7		

* (no specification provided)

Material 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= AASHTO=

Remarks

ASTM D1140, Method B
Dry Sample Weight = 163.83; Soak Time = 4 hrs

Sample Number: 1-B12 @ 15.5

Date: 11/5/2018

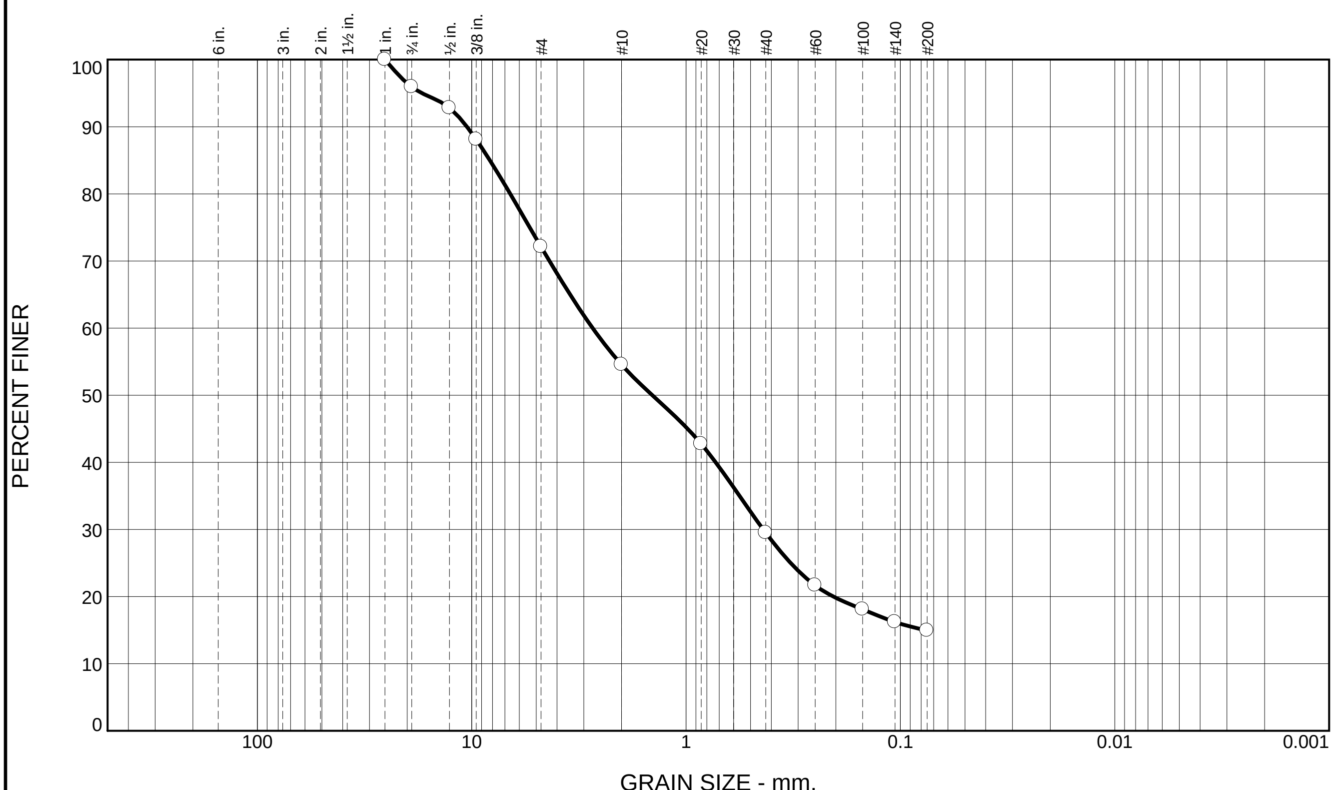
ENGEO INCORPORATED

Client: Monte Vista Memorial Investment Group, LLC
Project: 3656 Las Colinas Road

Project No: 15426.000.000

Tested By: M. Bromfield **Checked By:** M. Quasem

Particle Size Distribution Report



% +75mm	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	4.1	23.8	17.5	25.1	14.6	14.9	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1	100.0		
3/4	95.9		
1/2	92.8		
3/8	88.1		
#4	72.1		
#10	54.6		
#20	42.7		
#40	29.5		
#60	21.7		
#100	18.1		
#140	16.2		
#200	14.9		

* (no specification provided)

Material Description

See exploration logs

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 10.5265 D_{85}= 8.2257 D_{60}= 2.7271
D_{50}= 1.4374 D_{30}= 0.4366 D_{15}= 0.0767
D_{10}= C_u= C_c=

Classification

USCS= AASHTO=

Remarks

ASTM D6913

Sample Number: 1-B13 @ 5

Date: 11/6/18

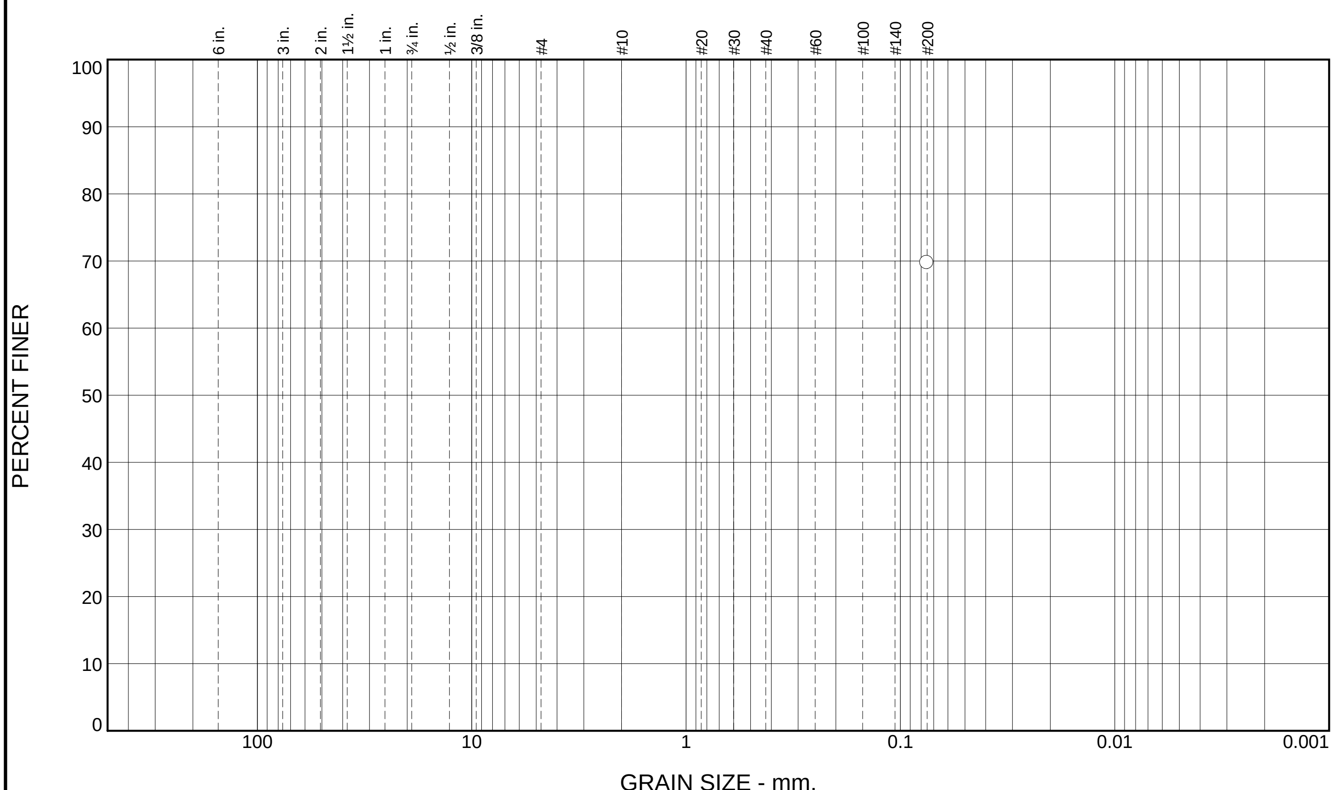
ENGEO INCORPORATED

Client: Monte Vista Memorial Investment Group, LLC
Project: 3656 Las Colinas Road

Project No: 15426.000.000

Tested By: M. Quasem **Checked By:** M. Bromfield

Particle Size Distribution Report



% +75mm	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
						69.7	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	69.7		

* (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= AASHTO=

Remarks

ASTM D1140, Method B
Dry Sample Weight = 65.34; Soak Time = 4 hrs

Sample Number: 1-B14 @ 4

Date: 11/6/18

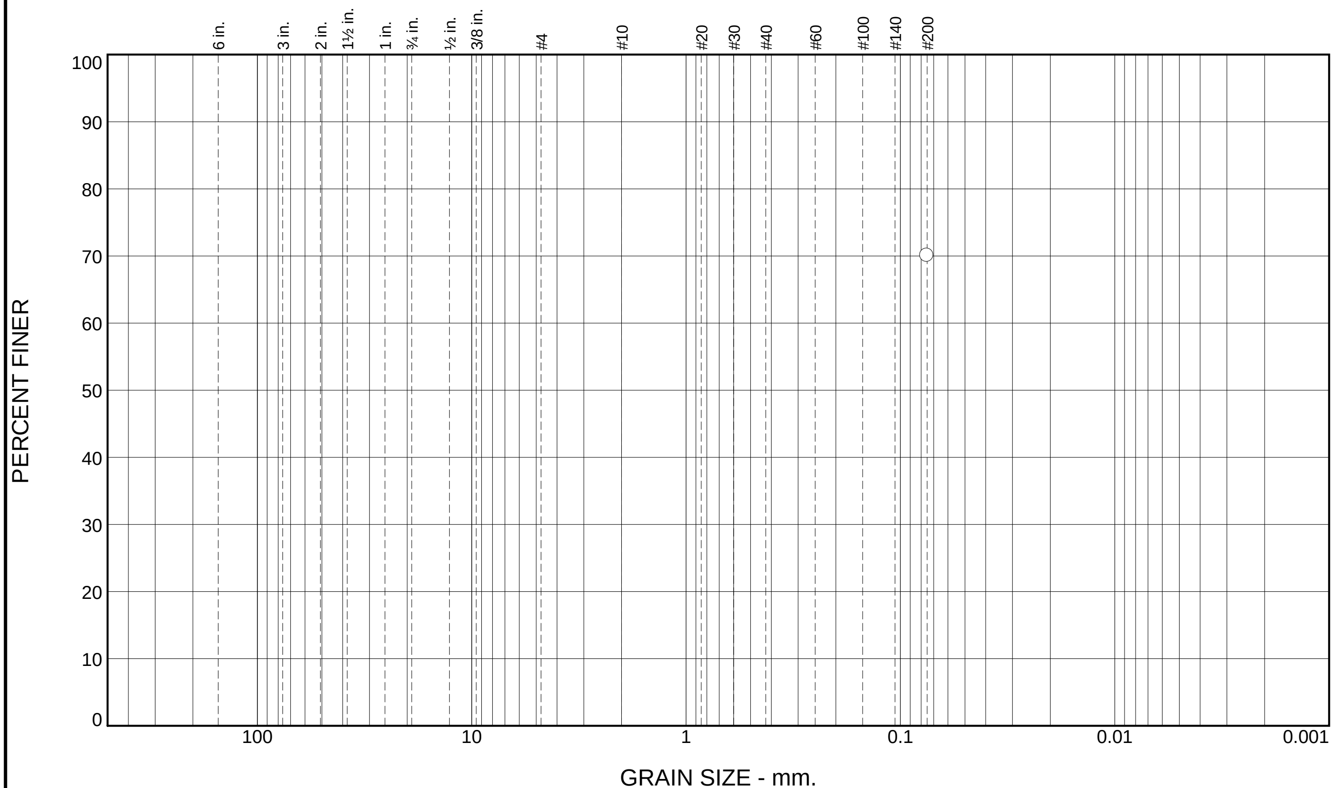
ENGEO INCORPORATED

Client: Monte Vista Memorial Investment Group, LLC
Project: 3656 Las Colinas Road

Project No: 15426.000.000

Tested By: M. Quasem **Checked By:** M. Bromfield

Particle Size Distribution Report



% +75mm	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
						70.1	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	70.1		

* (no specification provided)

Material Description

See exploration logs

Atterberg Limits

PL= 19 LL= 44 PI= 25

Coefficients

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

Classification

USCS= AASHTO=

Remarks

GS: ASTM D1140
PI: ASTM D4318, Wet method

Sample Number: 1-B1 @ 21

Date: 11/6/18

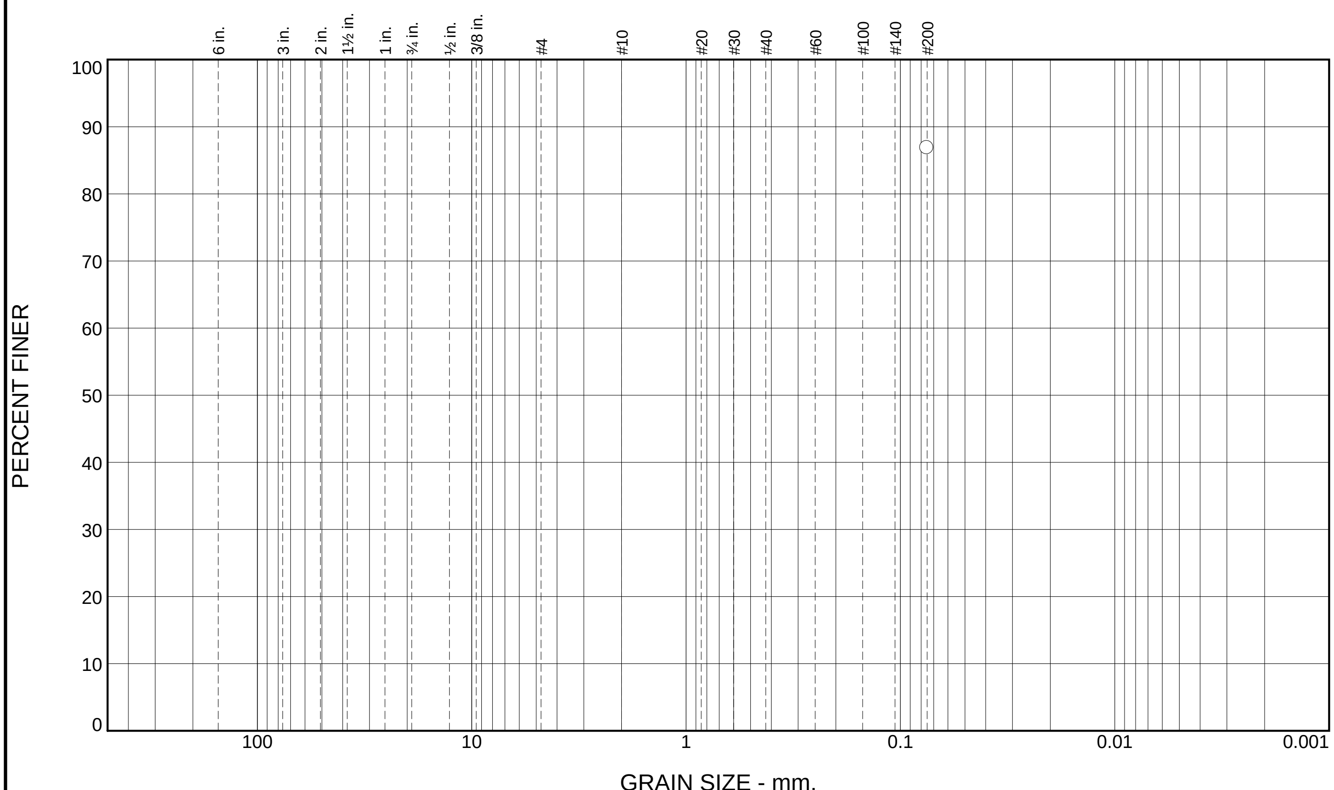
ENGEO INCORPORATED

Client: Monte Vista Memorial Investment Group, LLC
Project: 3656 Las Colinas Road

Project No: 15426.000.000

Tested By: M. Quasem **Checked By:** M. Bromfield

Particle Size Distribution Report



% +75mm	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
						86.8	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	86.8		

* (no specification provided)

Material Description

See exploration logs

Atterberg Limits

PL= 19 LL= 41 PI= 22

Coefficients

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

Classification

USCS= AASHTO=

Remarks

GS: ASTM D1140, Method B
Dry Sample Weight = 42.55; Soak Time = 4 hrs
PI: ASTM D4318, Wet method

Sample Number: 1-B1 @ 28

Date: 11/5/2018

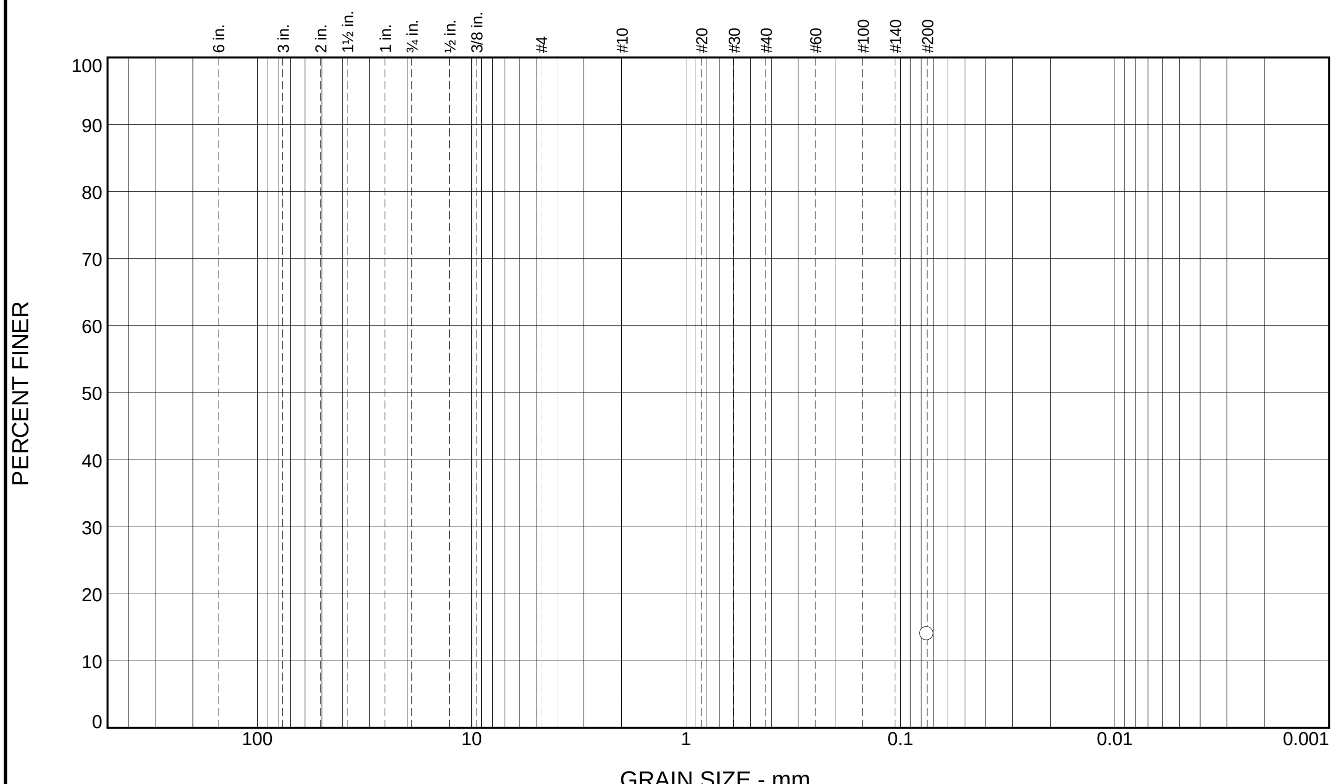
ENGEO INCORPORATED

Client: Monte Vista Memorial Investment Group, LLC
Project: 3656 Las Colinas Road

Project No: 15426.000.000

Tested By: M. Bromfield **Checked By:** M. Quasem

Particle Size Distribution Report



% +75mm	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
						14.0	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	14.0		

* (no specification provided)

Material 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= AASHTO=

Remarks

ASTM D1140, Method B
Dry Sample Weight = 182.95; Soak Time = 4 hrs

Sample Number: 1-B1 @ 38

Date: 11/5/2018

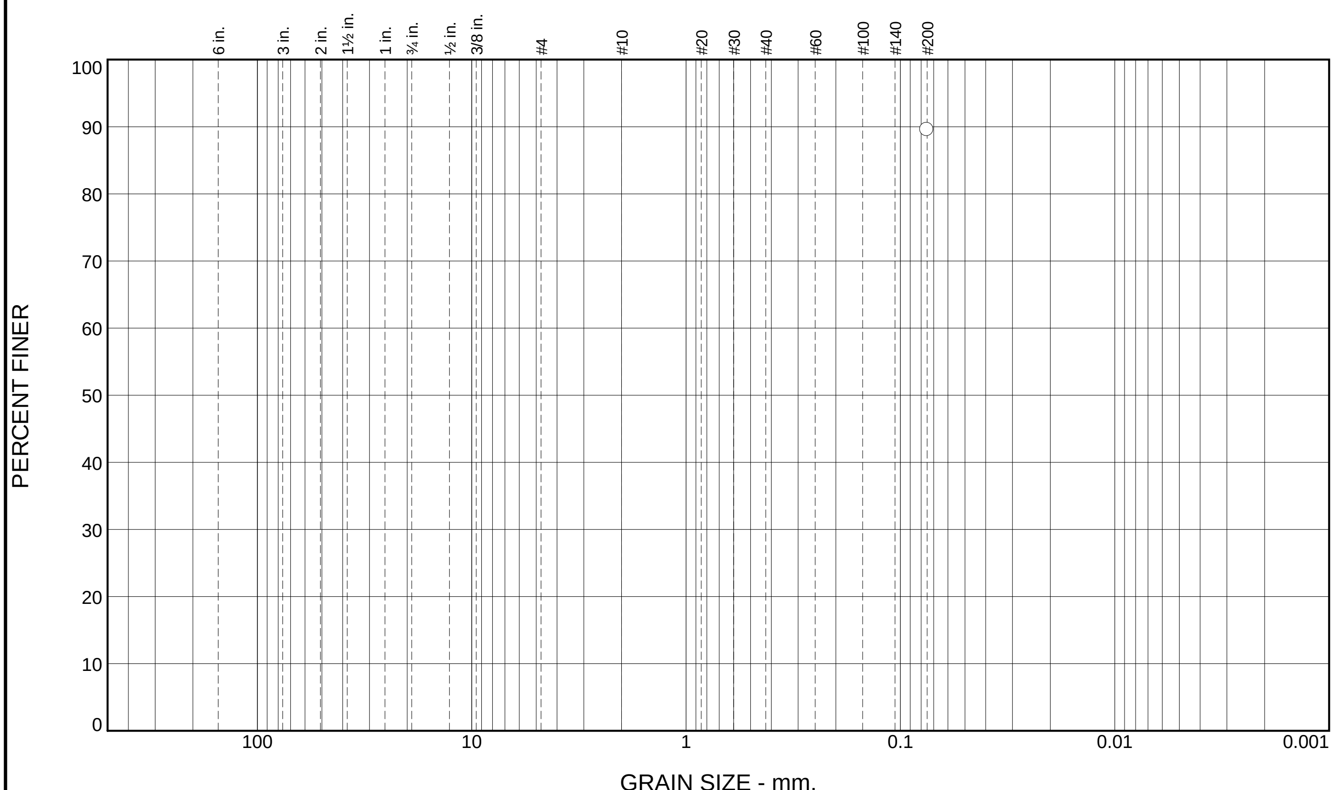
ENGEO INCORPORATED

Client: Monte Vista Memorial Investment Group, LLC
Project: 3656 Las Colinas Road

Project No: 15426.000.000

Tested By: M. Bromfield **Checked By:** M. Quasem

Particle Size Distribution Report



% +75mm	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
						89.5	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	89.5		

* (no specification provided)

Material 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= AASHTO=

Remarks

GS: ASTM D1140, Method B
Dry Sample Weight = 110.02; Soak Time = 4 hrs
PI: ASTM D4318, Wet method

Sample Number: 1-B1 @ 43

Date: 11/5/2018

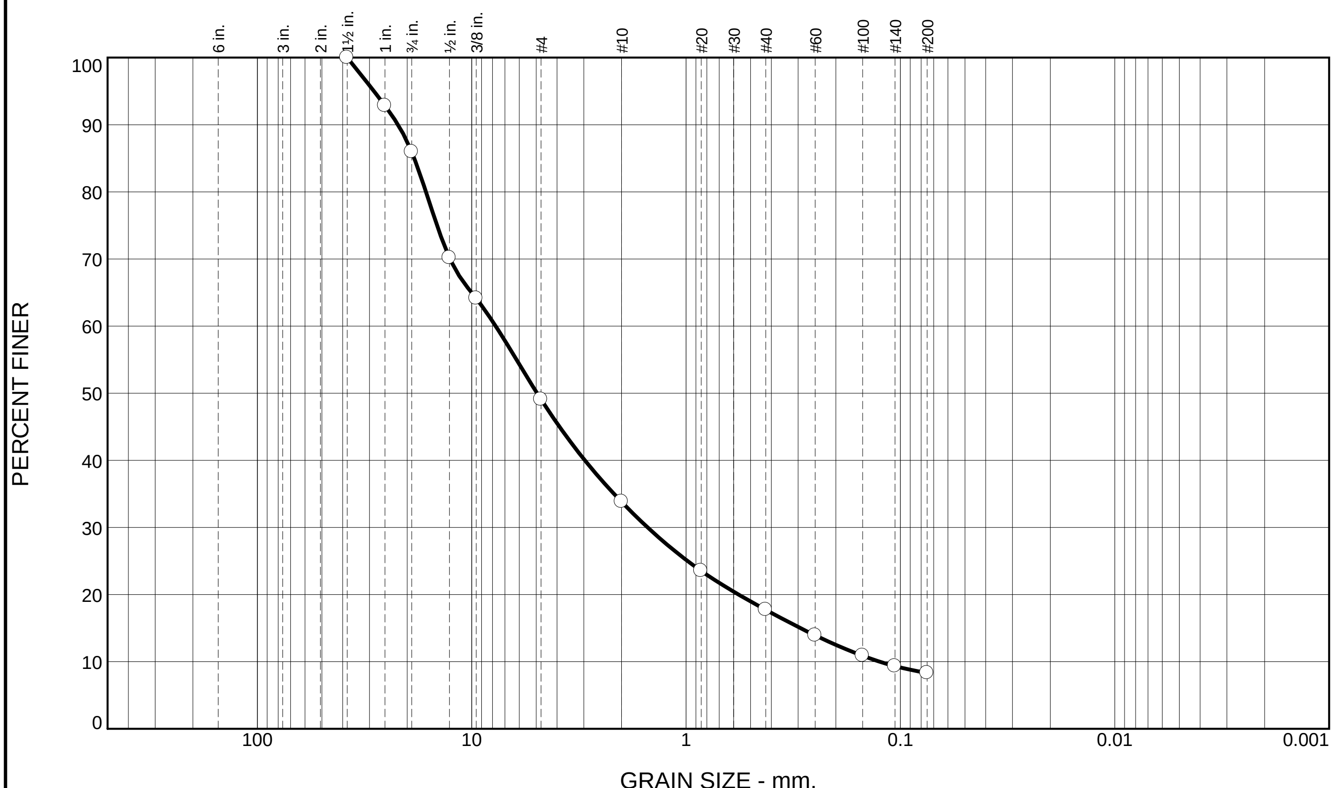
ENGEO INCORPORATED

Client: Monte Vista Memorial Investment Group, LLC
Project: 3656 Las Colinas Road

Project No: 15426.000.000

Tested By: M. Bromfield **Checked By:** M. Quasem

Particle Size Distribution Report



% +75mm	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	14.1	36.8	15.3	16.1	9.4	8.3	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1-1/2	100.0		
1	92.8		
3/4	85.9		
1/2	70.2		
3/8	64.1		
#4	49.1		
#10	33.8		
#20	23.5		
#40	17.7		
#60	13.9		
#100	10.9		
#140	9.3		
#200	8.3		

* (no specification provided)

Material Description

See exploration logs

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 22.0699 D_{85}= 18.5381 D_{60}= 7.7306
D_{50}= 4.9557 D_{30}= 1.5106 D_{15}= 0.2928
D_{10}= 0.1251 C_u= 61.79 C_c= 2.36

Classification

USCS= AASHTO=

Remarks

ASTM D6913

Sample Number: 1-B2 @ 11.5

Date: 11/6/18

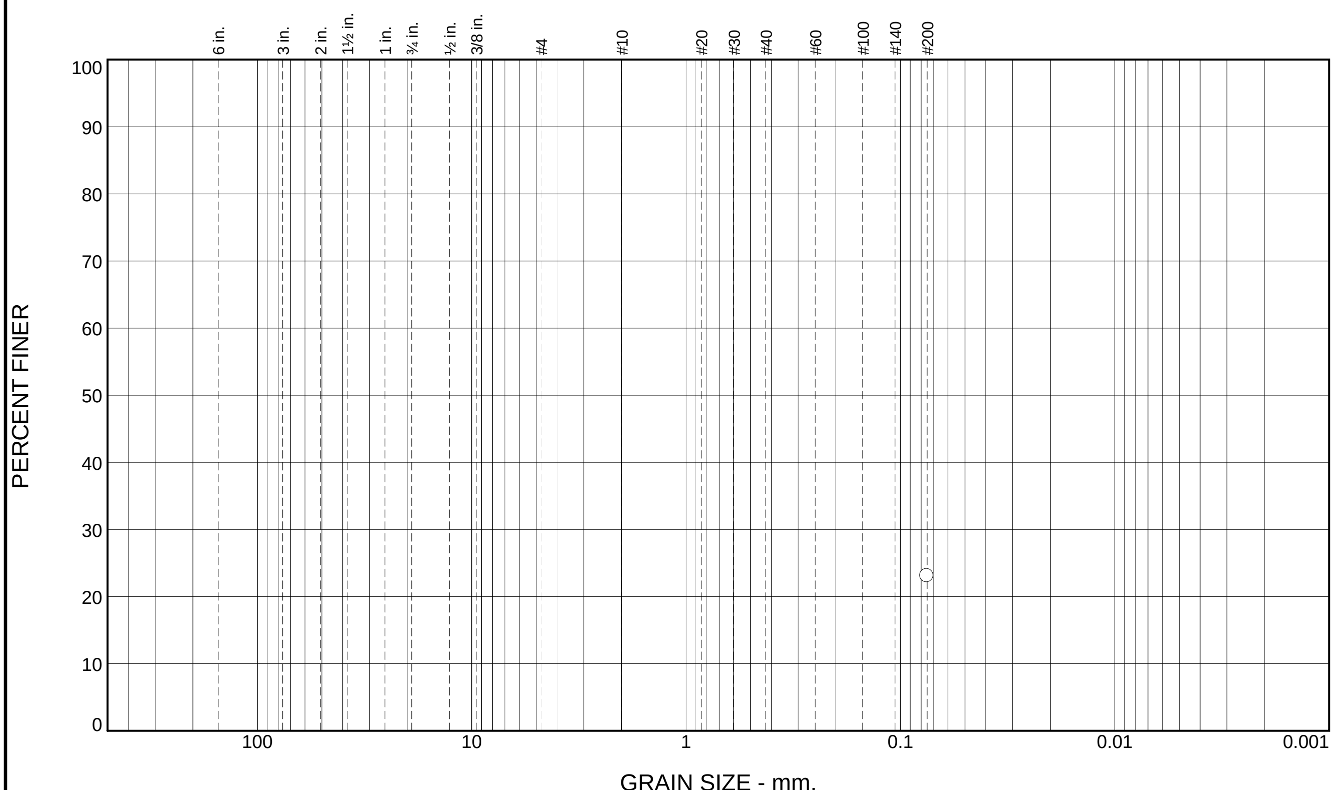
ENGEO INCORPORATED

Client: Monte Vista Memorial Investment Group, LLC
Project: 3656 Las Colinas Road

Project No: 15426.000.000

Tested By: M. Quasem **Checked By:** M. Bromfield

Particle Size Distribution Report



% +75mm	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
						23.0	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	23.0		

* (no specification provided)

Material Description

See exploration logs

Atterberg Limits

PL= 20 LL= 22 PI= 2

Coefficients

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

Classification

USCS= AASHTO=

Remarks

GS: ASTM D1140
PI: ASTM D4318, Wet method

Sample Number: 1-B2 @ 16.5

Date: 11/6/18

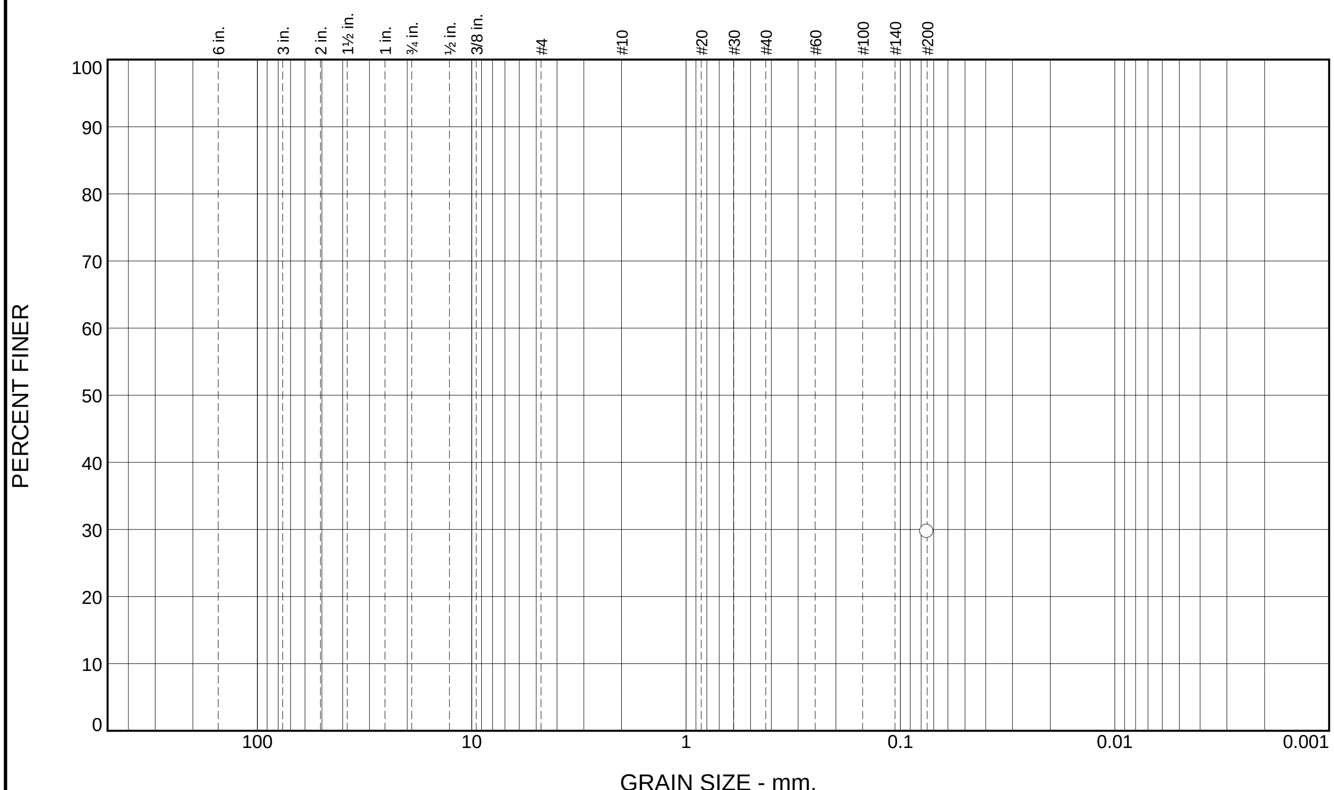
ENGEO INCORPORATED

Client: Monte Vista Memorial Investment Group, LLC
Project: 3656 Las Colinas Road

Project No: 15426.000.000

Tested By: M. Quasem **Checked By:** M. Bromfield

Particle Size Distribution Report



% +75mm	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
						29.6	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	29.6		

* (no specification provided)

Material 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= AASHTO=

Remarks

ASTM D1140, Method B
Dry Sample Weight = 164.64; Soak Time = 4 hrs

Sample Number: 1-B2 @ 20

Date: 11/5/2018

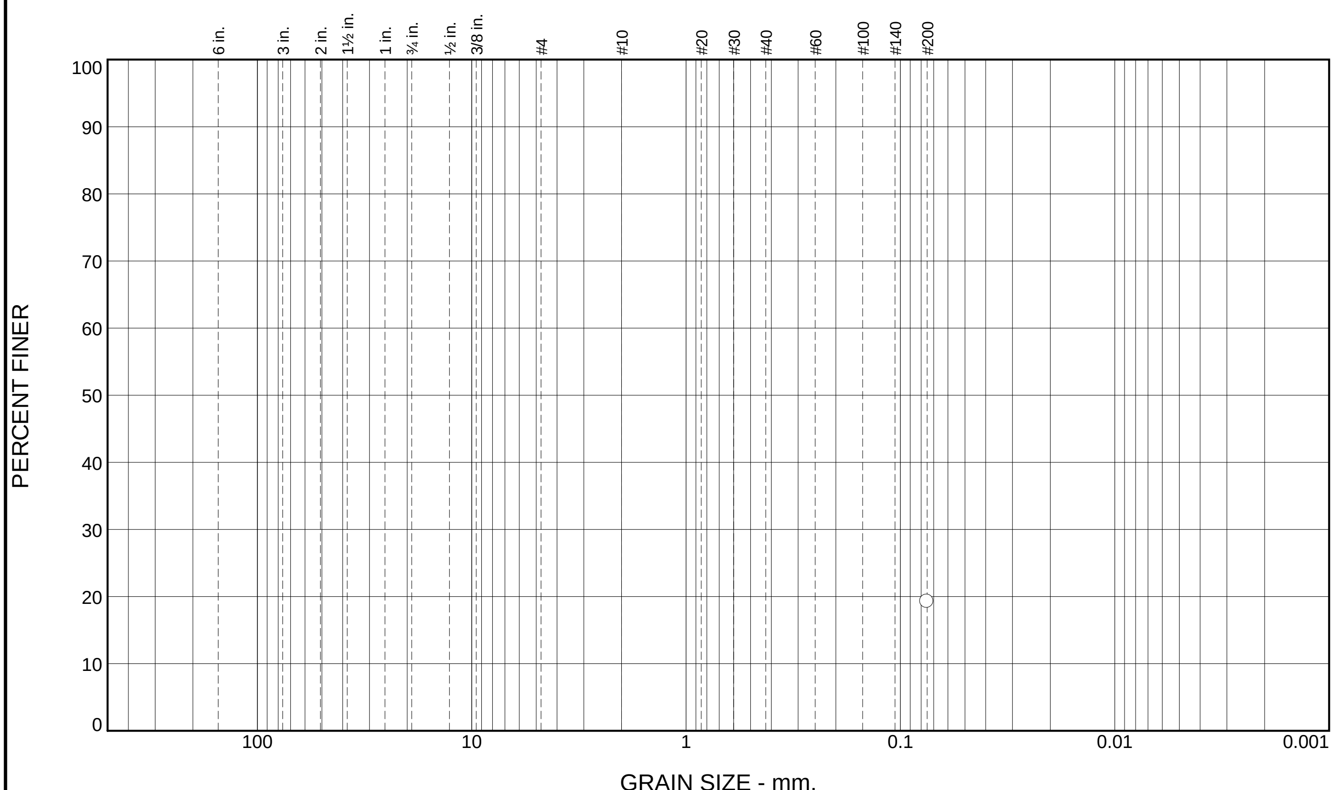
ENGEO INCORPORATED

Client: Monte Vista Memorial Investment Group, LLC
Project: 3656 Las Colinas Road

Project No: 15426.000.000

Tested By: M. Bromfield **Checked By:** M. Quasem

Particle Size Distribution Report



% +75mm	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
						19.2	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	19.2		

* (no specification provided)

Material Description

See exploration logs

Atterberg Limits

PL= 17 LL= 35 PI= 18

Coefficients

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

Classification

USCS= AASHTO=

Remarks

GS: ASTM D1140, Method B
Dry Sample Weight = 283.86; Soak Time = 4 hrs
PI: ASTM D4318, Wet method

Sample Number: 1-B3 @ 16

Date: 11/5/2018

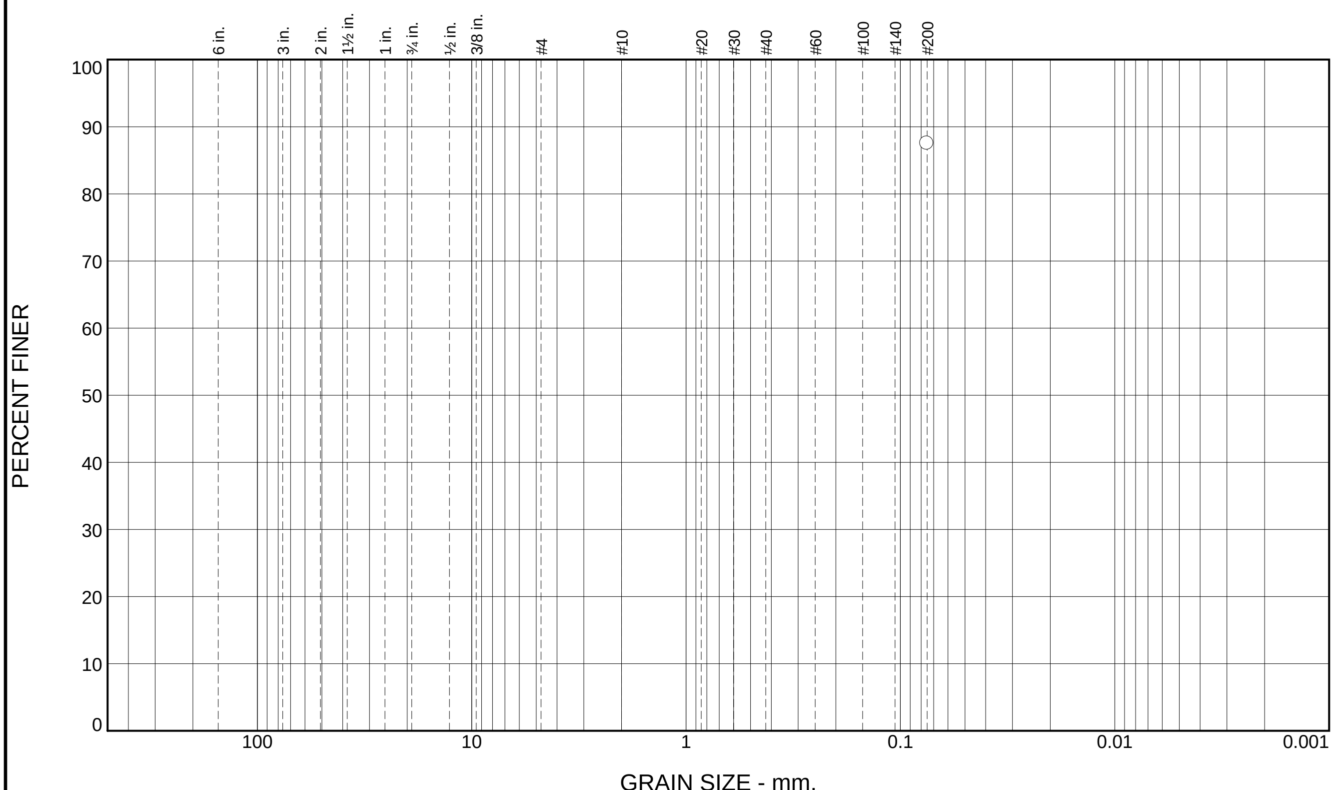
ENGEO INCORPORATED

Client: Monte Vista Memorial Investment Group, LLC
Project: 3656 Las Colinas Road

Project No: 15426.000.000

Tested By: M. Bromfield **Checked By:** M. Quasem

Particle Size Distribution Report



% +75mm	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
						87.5	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	87.5		

* (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= AASHTO=

Remarks

ASTM D1140, Method B
Dry Sample Weight = 118.31; Soak Time = 4 hrs

Sample Number: 1-B4 @ 33.5

Date: 11/26/2018

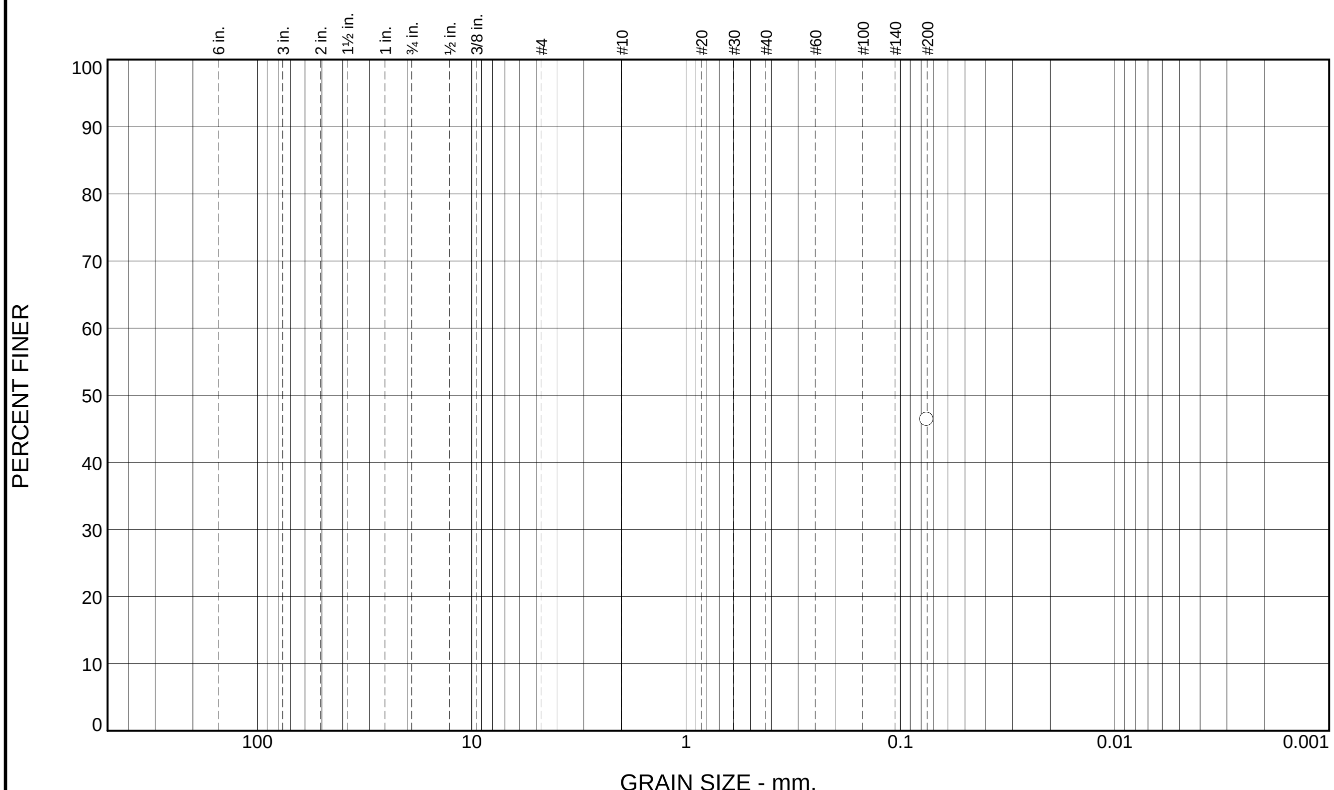
ENGEO INCORPORATED

Client: Monte Vista Memorial Investment Group, LLC
Project: 3656 Las Colinas Road

Project No: 15426.000.000

Tested By: M. Bromfield **Checked By:** M. Quasem

Particle Size Distribution Report



% +75mm	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
						46.3	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	46.3		

* (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= AASHTO=

Remarks

ASTM D1140, Method B
Dry Sample Weight = 145.69; Soak Time = 4 hrs

Sample Number: 1-B4 @ 43

Date: 11/26/2018

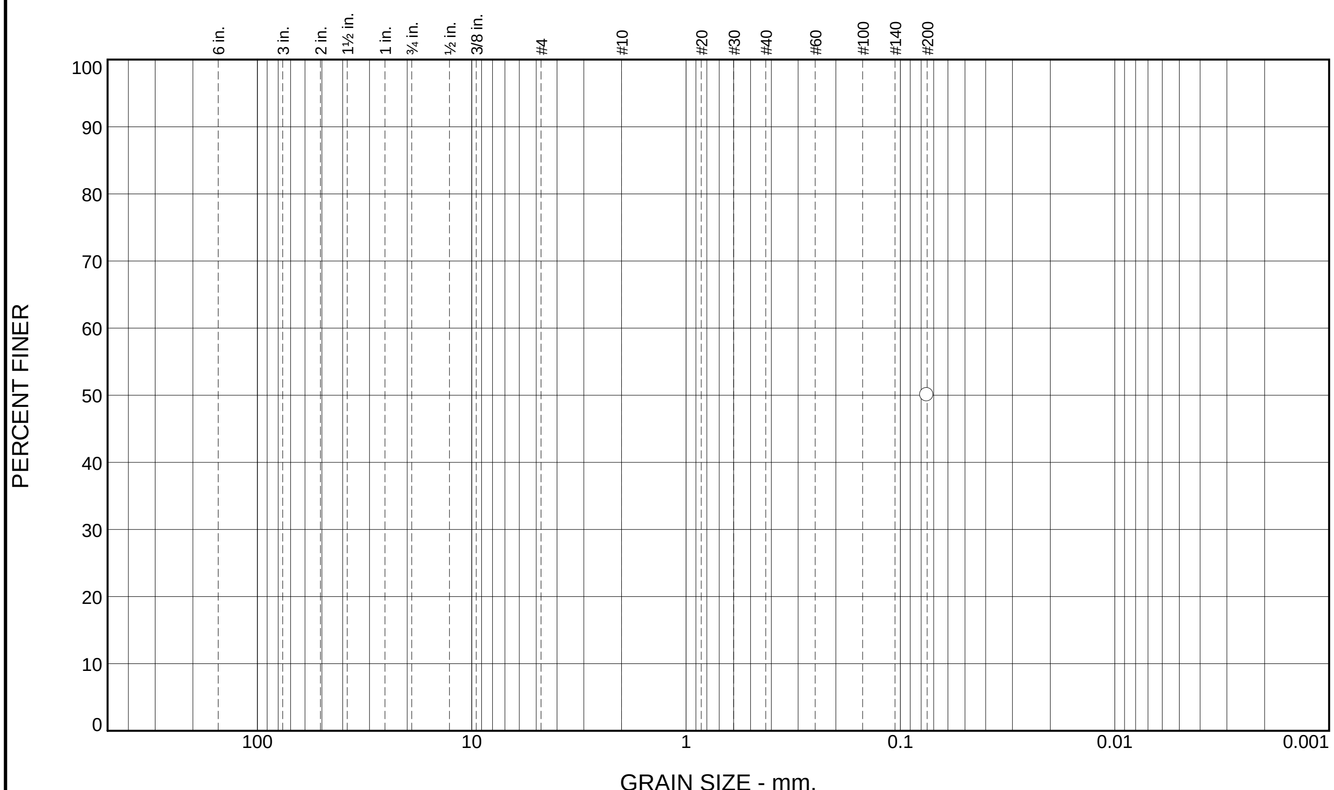
ENGEO INCORPORATED

Client: Monte Vista Memorial Investment Group, LLC
Project: 3656 Las Colinas Road

Project No: 15426.000.000

Tested By: M. Bromfield **Checked By:** M. Quasem

Particle Size Distribution Report



% +75mm	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
						50.0	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	50.0		

* (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= AASHTO=

Remarks

ASTM D1140, Method B
Dry Sample Weight = 207.55; Soak Time = 4 hrs

Sample Number: 1-B4 @ 6

Date: 11/26/2018

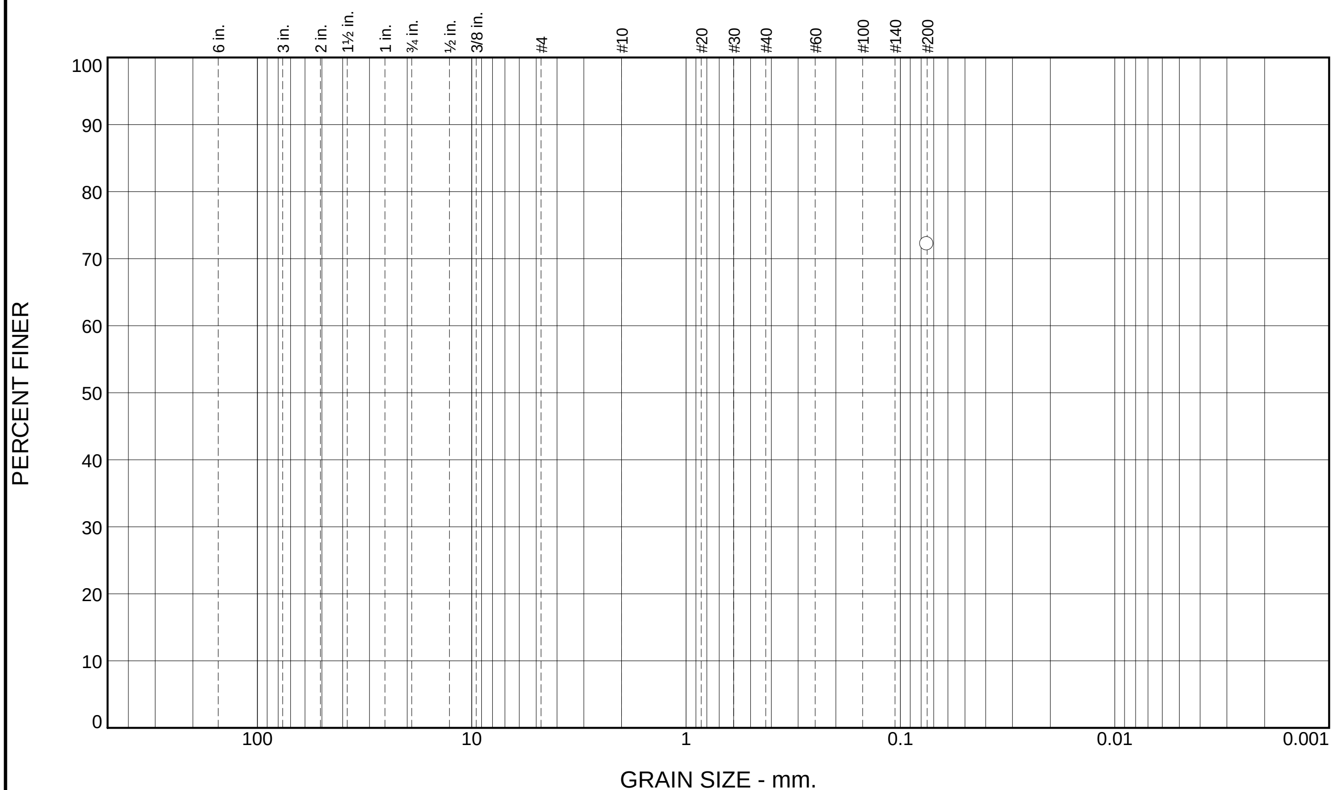
ENGEO INCORPORATED

Client: Monte Vista Memorial Investment Group, LLC
Project: 3656 Las Colinas Road

Project No: 15426.000.000

Tested By: M. Bromfield **Checked By:** M. Quasem

Particle Size Distribution Report



% +75mm	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
						72.2	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	72.2		

* (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= AASHTO=

Remarks

ASTM D1140, Method B
Dry Sample Weight = 109.94; Soak Time = 4 hrs

Sample Number: 1-B5 @ 16

Date: 11/26/2018

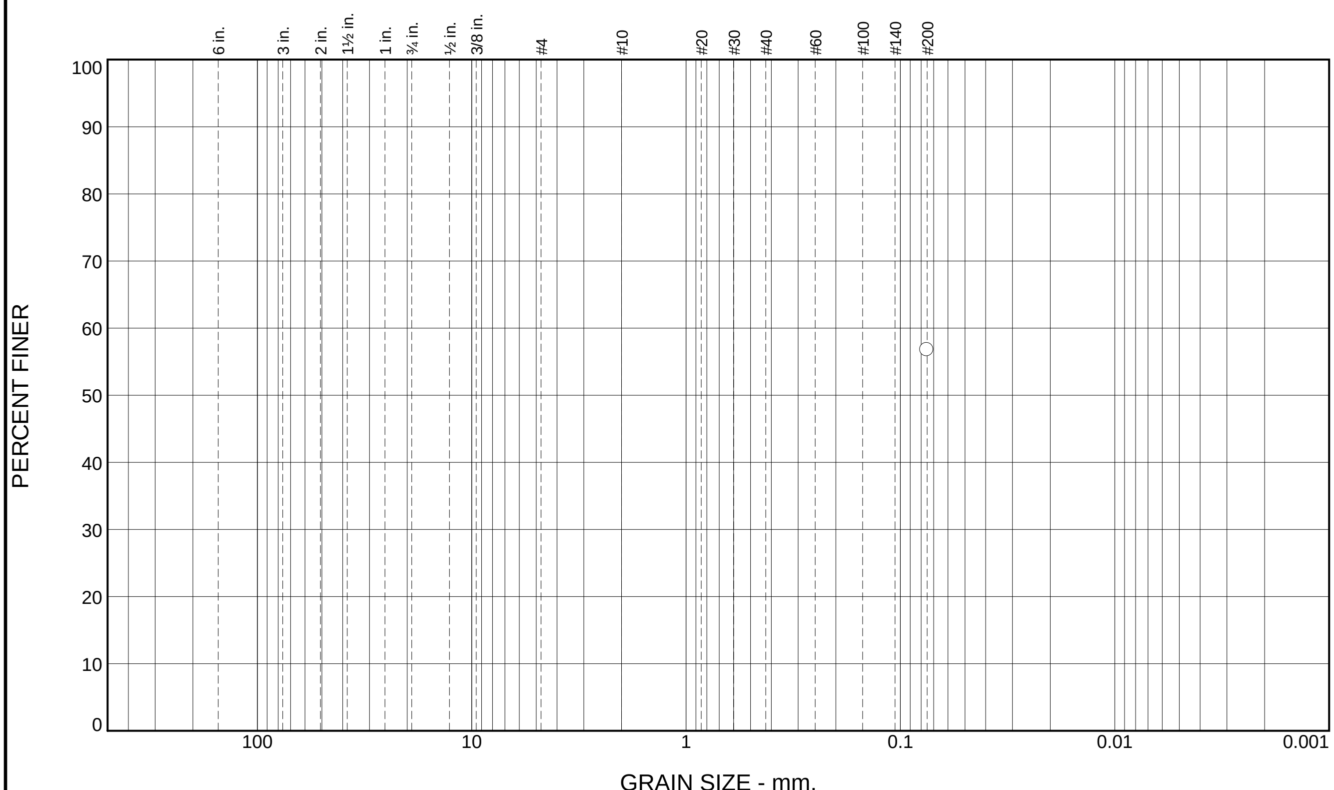
ENGEO INCORPORATED

Client: Monte Vista Memorial Investment Group, LLC
Project: 3656 Las Colinas Road

Project No: 15426.000.000

Tested By: M. Bromfield **Checked By:** M. Quasem

Particle Size Distribution Report



% +75mm	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
						56.7	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	56.7		

* (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= AASHTO=

Remarks

ASTM D1140, Method B
Dry Sample Weight = 238.5; Soak Time = 4 hrs

Sample Number: 1-B5 @ 33

Date: 11/28/2018

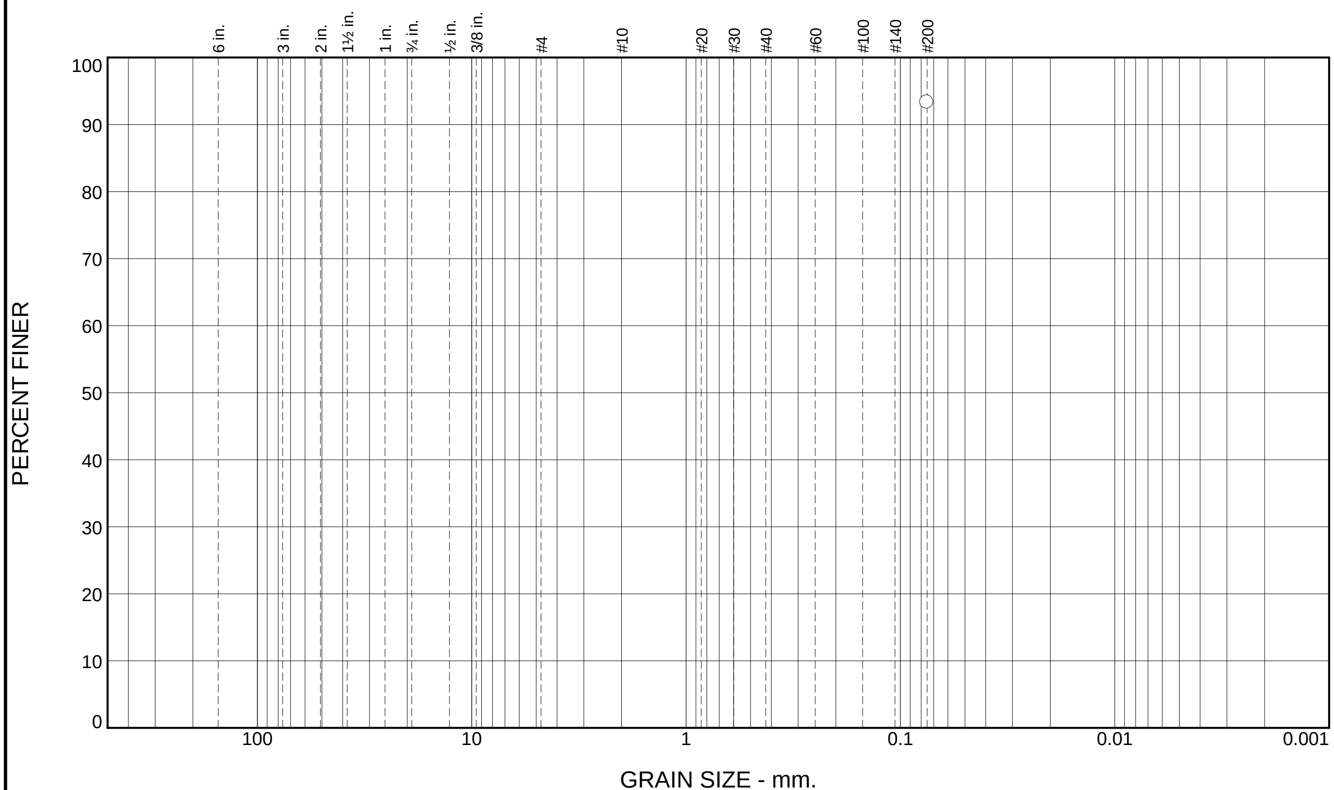
ENGEO INCORPORATED

Client: Monte Vista Memorial Investment Group, LLC
Project: 3656 Las Colinas Road

Project No: 15426.000.000

Tested By: M. Bromfield **Checked By:** M. Quasem

Particle Size Distribution Report



% +75mm	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
						93.3	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	93.3		

* (no specification provided)

Soil Description

See exploration logs

Atterberg Limits

PL= 20 LL= 40 PI= 20

Coefficients

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

Classification

USCS= AASHTO=

Remarks

GS: ASTM D1140, Method B
Dry Sample Weight = 55.33, Soak Time = 4 hrs
PI: ASTM D4318, Wet method

Sample Number: 1-B5 @ 38

Date: 11/28/2018

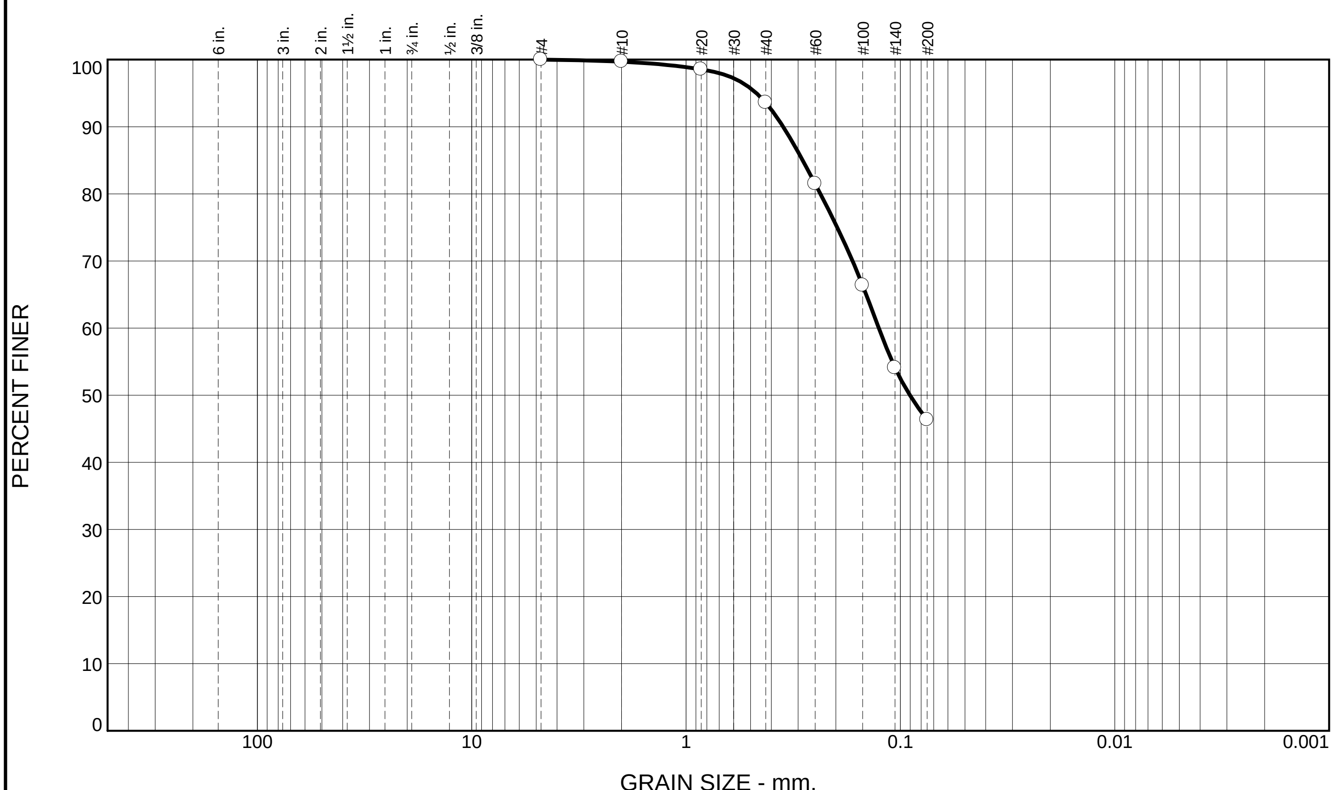
ENGEO INCORPORATED

Client: Monte Vista Memorial Investment Group, LLC
Project: 3656 Las Colinas Road

Project No: 15426.000.000

Tested By: M. Bromfield **Checked By:** M. Quasem

Particle Size Distribution Report



% +75mm	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	0.0	0.3	6.1	47.3	46.3	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#4	100.0		
#10	99.7		
#20	98.5		
#40	93.6		
#60	81.5		
#100	66.4		
#140	54.1		
#200	46.3		

* (no specification provided)

Soil Description

See exploration logs

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 0.3525 D_{85}= 0.2860 D_{60}= 0.1261
D_{50}= 0.0903 D_{30}= D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= AASHTO=

Remarks

ASTM D6913, Method A

Sample Number: 1-B5 @ 5.5

Date: 11/26/2018

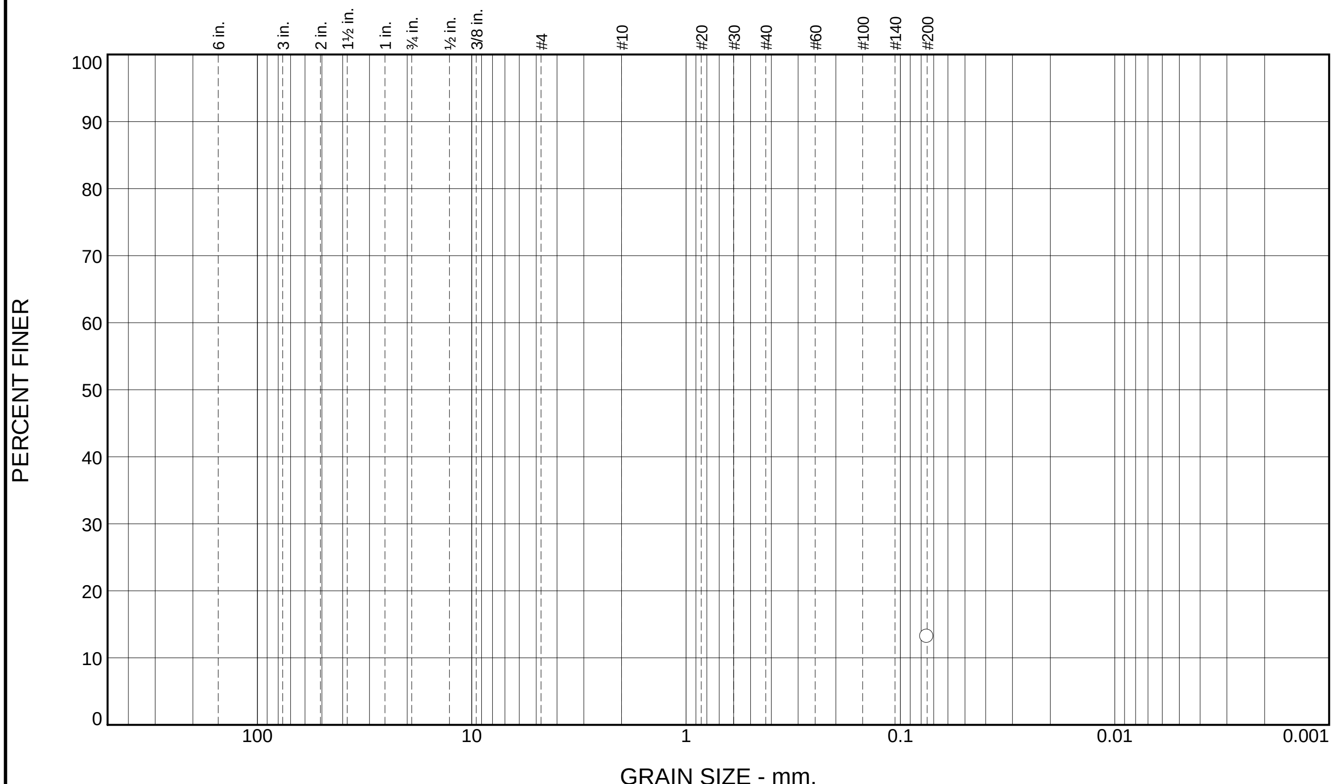
ENGEO INCORPORATED

Client: Monte Vista Memorial Investment Group, LLC
Project: 3656 Las Colinas Road

Project No: 15426.000.000

Tested By: M. Bromfield **Checked By:** M. Quasem

Particle Size Distribution Report



% +75mm	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
						13.2	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	13.2		

* (no specification provided)

Material Description

See exploration logs

Atterberg Limits

PL= LL= PI=

Coefficients

D90= D85= D60=
D50= D30= D15=
D10= Cu= Cc=

Classification

USCS= AASHTO=

Remarks

ASTM D1140, Method B
Dry Sample Weight = 334.5; Soak Time = 4 hrs

Sample Number: 1-B6 @ 17

Date: 11/5/2018

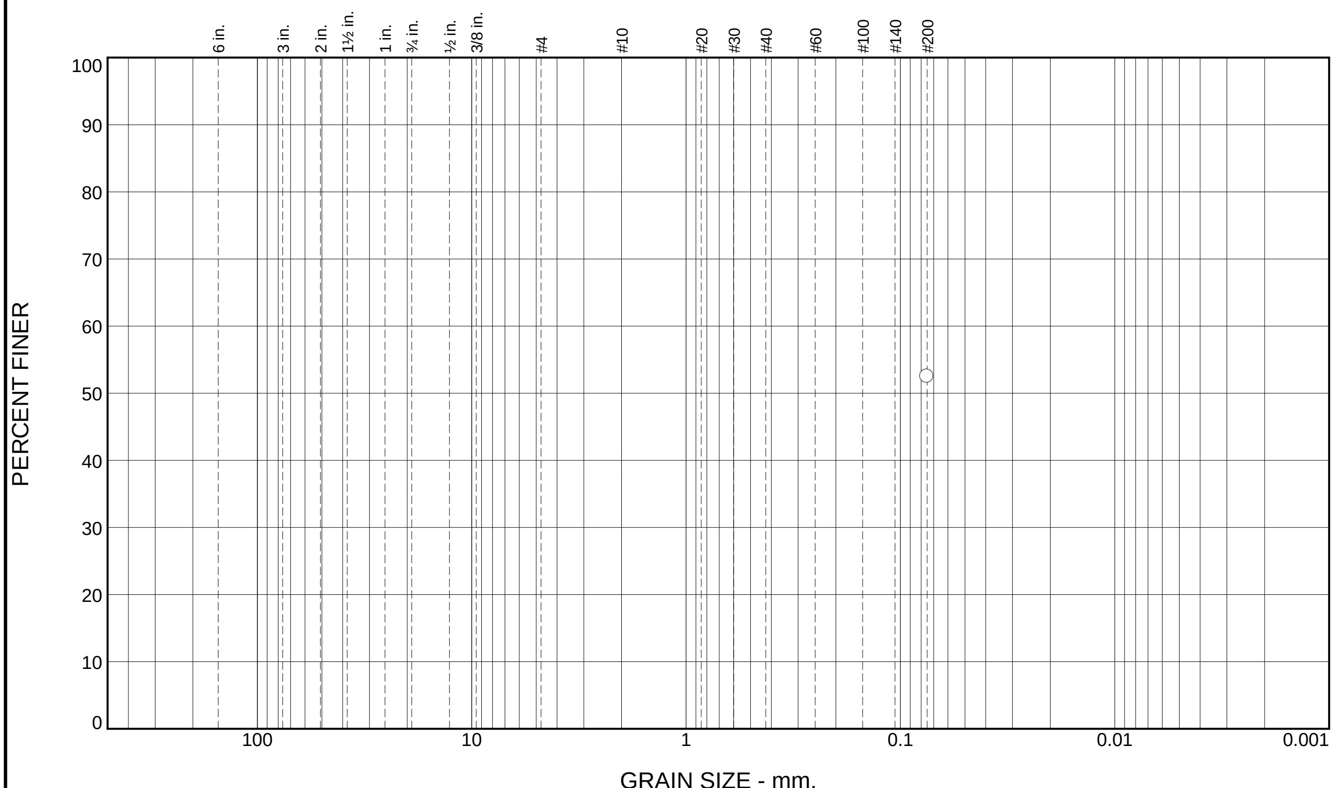
ENGEO INCORPORATED

Client: Monte Vista Memorial Investment Group, LLC
Project: 3656 Las Colinas Road

Project No: 15426.000.000

Tested By: M. Bromfield **Checked By:** M. Quasem

Particle Size Distribution Report



% +75mm	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
						52.5	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	52.5		

* (no specification provided)

Material 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= AASHTO=

Remarks

ASTM D1140, Method B
Dry Sample Weight = 143.8; Soak Time = 68.33

Sample Number: 1-B6 @ 25

Date: 11/5/2018

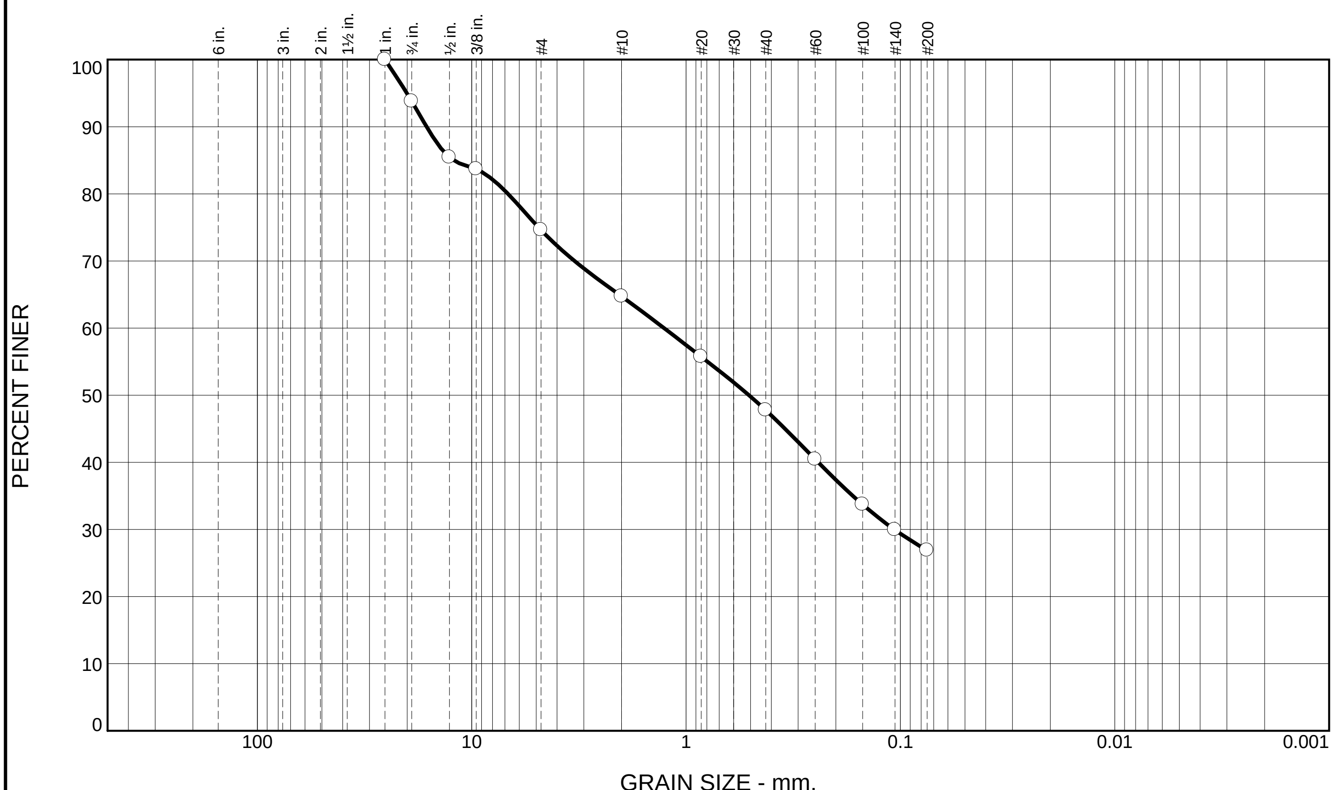
ENGEO INCORPORATED

Client: Monte Vista Memorial Investment Group, LLC
Project: 3656 Las Colinas Road

Project No: 15426.000.000

Tested By: M. Bromfield **Checked By:** M. Quasem

Particle Size Distribution Report



% +75mm	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	6.2	19.2	9.9	16.9	20.9	26.9	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1	100.0		
3/4	93.8		
1/2	85.4		
3/8	83.7		
#4	74.6		
#10	64.7		
#20	55.7		
#40	47.8		
#60	40.4		
#100	33.7		
#140	29.9		
#200	26.9		

* (no specification provided)

Material Description

See exploration logs

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 16.2311 D_{85}= 12.1555 D_{60}= 1.2673
D_{50}= 0.5088 D_{30}= 0.1069 D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= AASHTO=

Remarks

ASTM D6913

Sample Number: 1-B7 @ 10

Date: 11/6/18

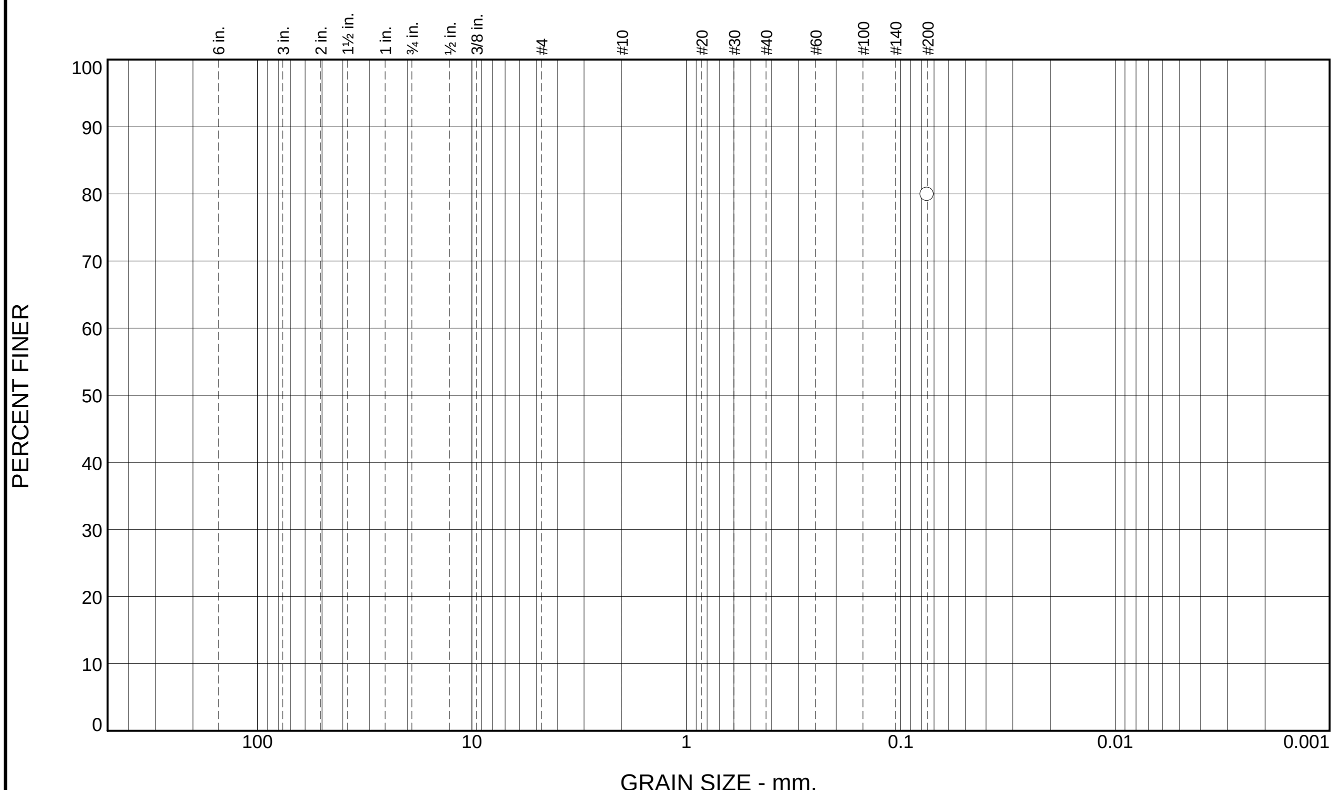
ENGEO INCORPORATED

Client: Monte Vista Memorial Investment Group, LLC
Project: 3656 Las Colinas Road

Project No: 15426.000.000

Tested By: M. Quasem **Checked By:** M. Bromfiled

Particle Size Distribution Report



% +75mm	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
						79.9	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	79.9		

* (no specification provided)

Soil Description

See exploration logs

Atterberg Limits

PL= 21 LL= 41 PI= 20

Coefficients

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

Classification

USCS= AASHTO=

Remarks

GS: ASTM D1140, Method B
Dry Sample Weight = 176.83; Soak Time = 4 hrs
PI: ASTM D4318, Wet method

Sample Number: 1-TP4 @ 11-12

Date: 11/28/2018

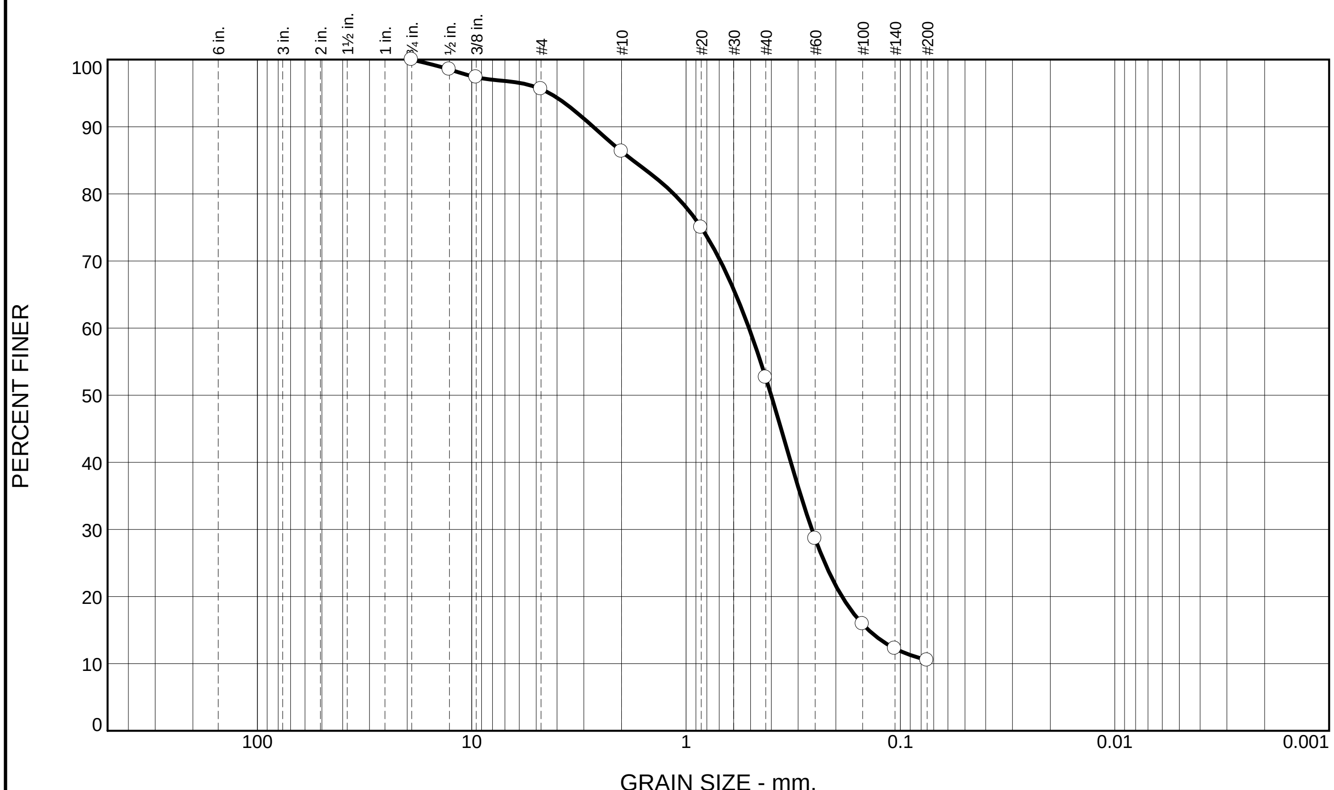
ENGEO INCORPORATED

Client: Monte Vista Memorial Investment Group, LLC
Project: 3656 Las Colinas Road

Project No: 15426.000.000

Tested By: M. Bromfield **Checked By:** M. Quasem

Particle Size Distribution Report



% +75mm	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	4.4	9.3	33.7	42.1	10.5	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3/4	100.0		
1/2	98.5		
3/8	97.4		
#4	95.6		
#10	86.3		
#20	75.0		
#40	52.6		
#60	28.6		
#100	15.9		
#140	12.2		
#200	10.5		

* (no specification provided)

Material Description

See exploration logs

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 2.7117 D_{85}= 1.7699 D_{60}= 0.5087
D_{50}= 0.4010 D_{30}= 0.2591 D_{15}= 0.1406
D_{10}= C_u= C_c=

Classification

USCS= AASHTO=

Remarks

GS: ASTM D422; Specific gravity = 2.706, ASTM D854
Silt/clay division of 0.002 was used

Sample Number: 1-B1 @ 10

Date: 11/6/18

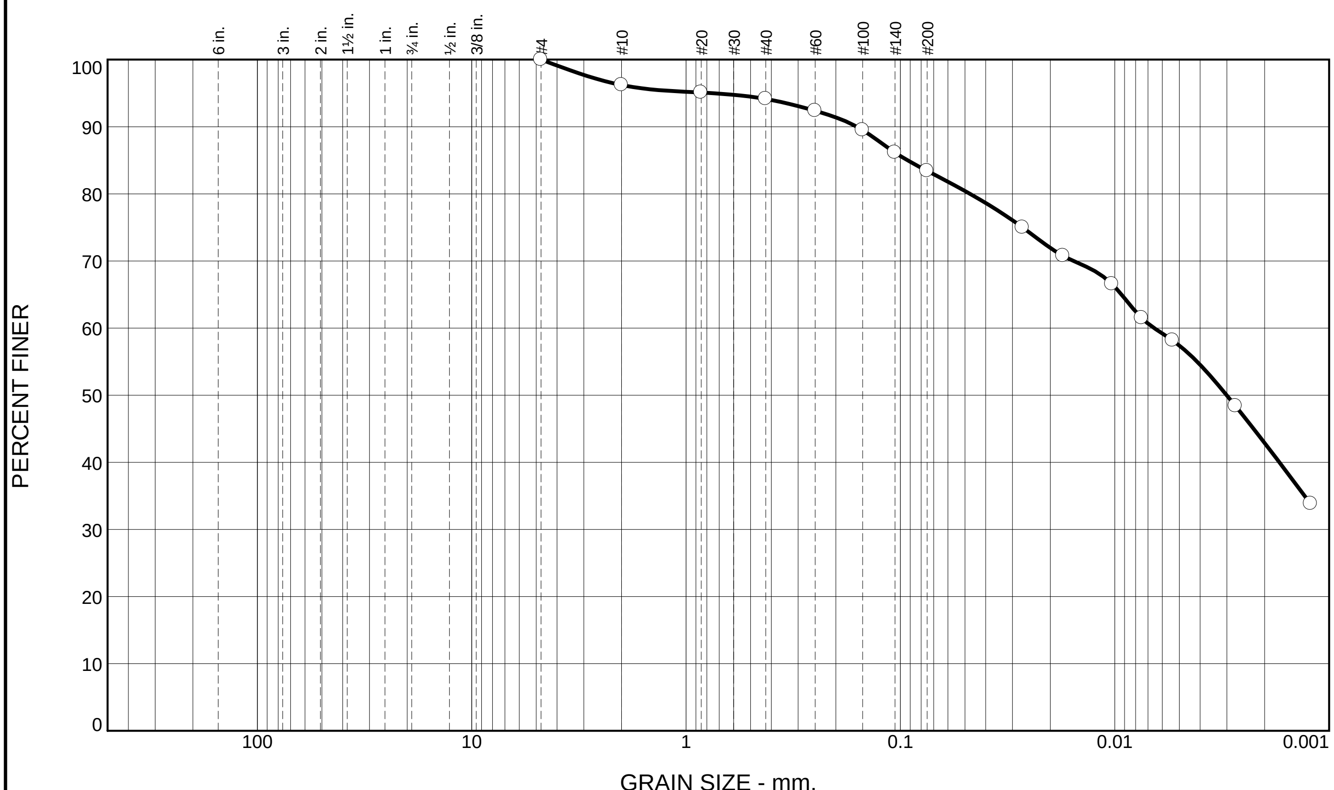
ENGEO INCORPORATED

Client: Monte Vista Memorial Investment Group, LLC
Project: 3656 Las Colinas Road

Project No: 15426.000.000

Tested By: M. Quasem **Checked By:** M. Bromfield

Particle Size Distribution Report



% +75mm	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	0.0	3.8	2.1	10.7	40.5	42.9

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#4	100.0		
#10	96.2		
#20	95.1		
#40	94.1		
#60	92.4		
#100	89.5		
#140	86.1		
#200	83.4		
0.0269 mm.	75.0		
0.0174 mm.	70.8		
0.0103 mm.	66.6		
0.0075 mm.	61.5		
0.0054 mm.	58.2		
0.0027 mm.	48.4		
0.0012 mm.	33.8		

* (no specification provided)

Soil Description

See exploration logs

Atterberg Limits

PL= 21 LL= 46 PI= 25

Coefficients

D_{90}= 0.1597 D_{85}= 0.0927 D_{60}= 0.0066
D_{50}= 0.0030 D_{30}= D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= CL AASHTO= A-7-6(21)

Remarks

GS: ASTM D422, Specific Gravity = 2.715, ASTM D854
PI: ASTM D4318, Wet method; USCS: ASTM D2487
Silt/clay division of .002 mm was used.

Sample Number: 1-B5 @ 28

Date: 11/28/2018

ENGEO INCORPORATED

Client: Monte Vista Memorial Investment Group, LLC
Project: 3656 Las Colinas Road

Project No: 15426.000.000

Tested By: M. Bromfield **Checked By:** M. Quasem

MOISTURE CONTENT DETERMINATION

ASTM D2216

BORING/SAMPLE ID	1-B3@11.5							
DEPTH (ft)	11.5							
Method A or B	B							
%MOISTURE	27.6							

BORING/SAMPLE ID								
DEPTH (ft)								
Method A or B								
%MOISTURE								

BORING/SAMPLE ID								
DEPTH (ft)								
Method A or B								
%MOISTURE								

BORING/SAMPLE ID								
DEPTH (ft)								
Method A or B								
%MOISTURE								

BORING/SAMPLE ID								
DEPTH (ft)								
Method A or B								
%MOISTURE								

PROJECT NAME: 3656 Las Colinas Road
DATE: 11/15/18
PROJECT NUMBER: 15426.000.000
CLIENT: Monte Vista Memorial investment Group, LLC
PHASE NUMBER: 001

ENGEO INCORPORATED

Tested by: R. Montalvo   Reviewed by: M. Gilbert

MOISTURE-DENSITY DETERMINATION
ASTM D7263

BORING ID:	1-B1	1-B1	1-B1	1-B2	1-B2	1-B2	1-B3	1-B3
DEPTH (ft.):	28.0	38.0	43.0	16.5	20.0	44.0	16.0	26.5
MOISTURE CONTENT (%):	21.2	19.7	23.3	20.1	16.9	20.1	15.9	13.2
DRY DENSITY (lbs/ft^3):	107.1		103.5	106.8		106.5	118.0	

BORING ID:	1-B3	1-B6	1-B6	1-B6	1-B7	1-B7	1-B8	1-B9
DEPTH (ft.):	50.0	6.0	17.0	31.0	4.0	10.0	5.5	8.0
MOISTURE CONTENT (%):	25.0	23.3	18.0	21.2	22.2	20.1	33.9	24.1
DRY DENSITY (lbs/ft^3):	101.3	101.9		108.8			86.6	97.8

BORING ID:	1-B10	1-B11	1-B12	1-B13	1-B13	1-B14	1-B15	
DEPTH (ft.):	4.0	4.0	10.0	10.0	20.0	4.0	5.5	
MOISTURE CONTENT (%):	25.3	17.8	18.5	19.1	25.3	45.1	30.6	
DRY DENSITY (lbs/ft^3):	83.3	80.8	109.3	106.1	99.9	74.0		

Testing remarks: For moisture content only, ASTM D2216

PROJECT NAME: 3656 Las Colinas Road
PROJECT NUMBER: 15426.000.000
CLIENT: Monte Vista Memorial Investment Group, LLC
PHASE NUMBER: 001

DATE: 11/05/18

ENGEO
Expect Excellence

Tested by: M. Quasem   Reviewed by: W. Miller

MOISTURE-DENSITY DETERMINATION
ASTM D7263

BORING ID:	1-B4	1-B4	1-B4	1-B5	1-B5	1-B5	1-B5	1-B5
DEPTH (ft.):	6.0	33.5	43.0	11.5	21.0	28.0	33.0	43.0
MOISTURE CONTENT (%):	14.8	26.7	20.1	23.1	21.3	25.1	18.3	26.0
DRY DENSITY (lbs/ft^3):	102.6	96.1	107.9		106.0	99.5	111.1	97.4

Testing remarks: For moisture content only, ASTM D2216

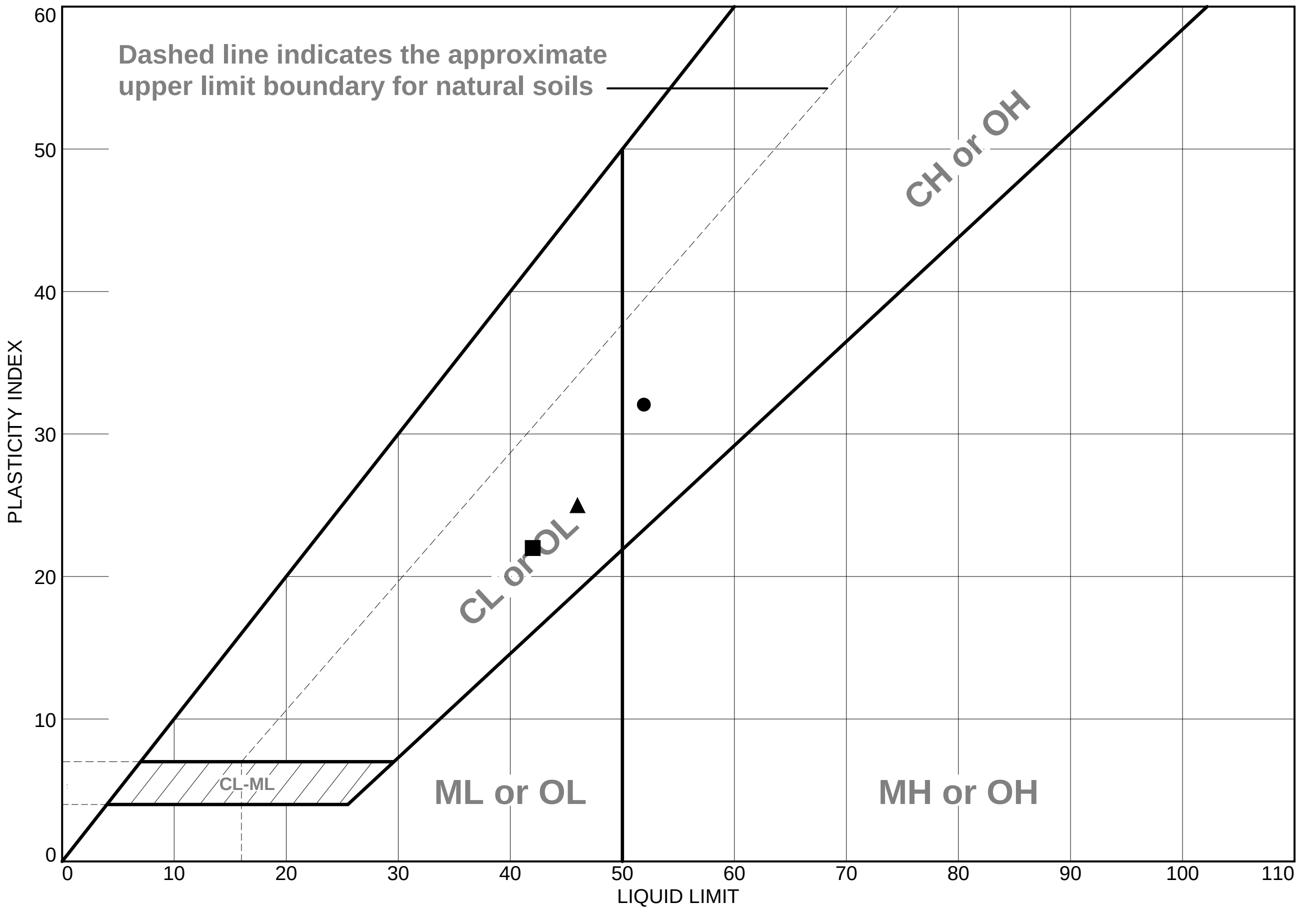
PROJECT NAME: 3656 Las Colinas Road
PROJECT NUMBER: 15426.000.000
CLIENT: Monte Vista Memorial Investment Group, LLC
PHASE NUMBER: 001

DATE: 11/22/18

ENGEO
Expect Excellence

Tested by: M. Bromfeild   Reviewed by: M. Quasem

LIQUID AND PLASTIC LIMITS TEST REPORT



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	See exploration logs	52	20	32			
■	See exploration logs	42	20	22			
▲	See exploration logs	46	21	25			

Project No. 15426.000.000 **Client:** Monte Vista Memorial Investment Group, LLC

Project: 3656 Las Colinas Road

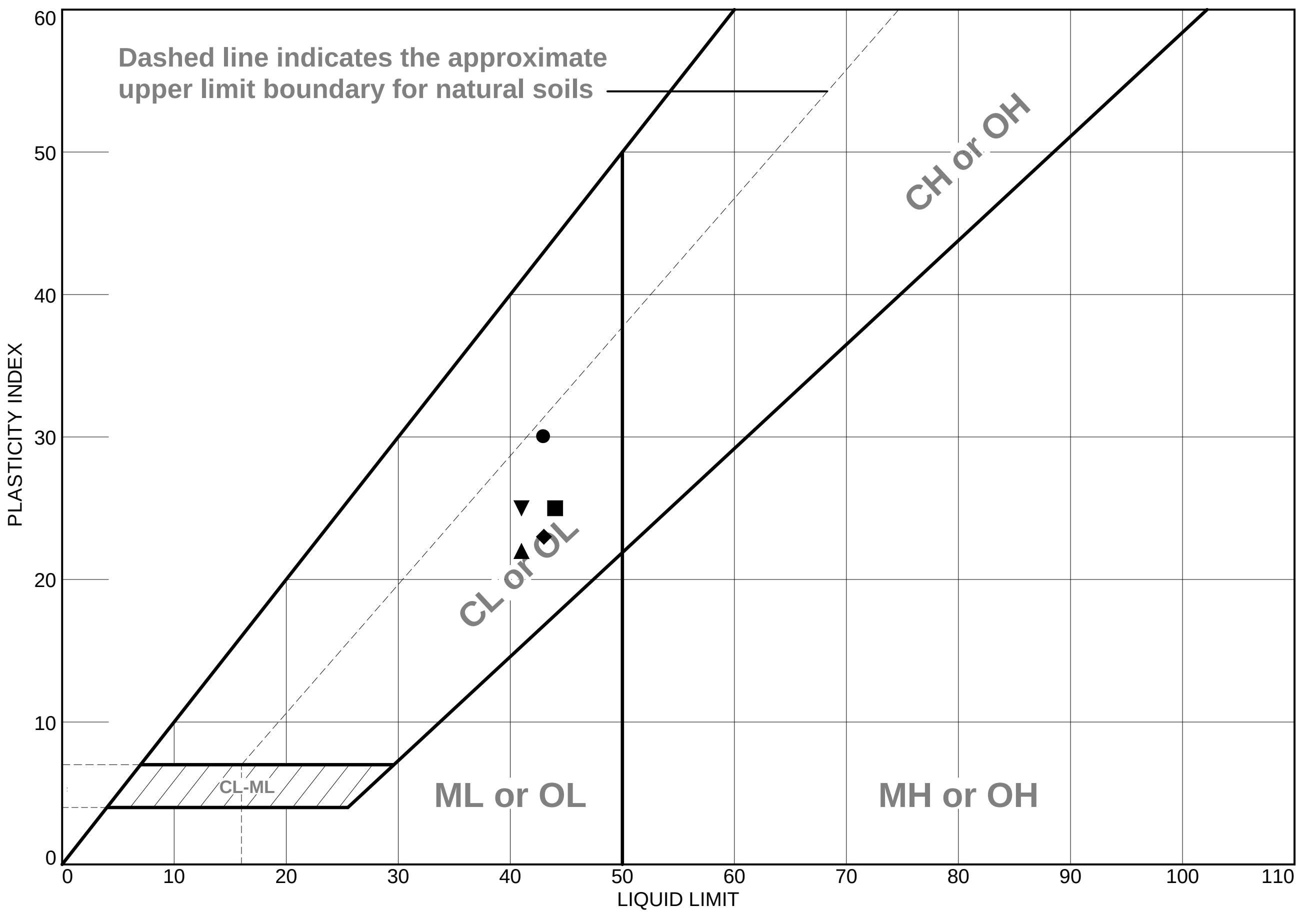
●**Sample Number:** 1-B4 @ 16
■**Sample Number:** 1-B5 @ 11.5
▲**Sample Number:** 1-B5 @ 28

ENGEO INCORPORATED

Remarks:
●ASTM D4318, Wet method
■ASTM D4318, Wet method
▲PI: ASTM D4318, Wet method
GS: ASTM D422

Tested By: M. Bromfield **Checked By:** M. Quasem

LIQUID AND PLASTIC LIMITS TEST REPORT



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	See exploration logs	43	13	30			
■	See exploration logs	44	19	25		70.1	
▲	See exploration logs	41	19	22		86.8	
◆	See exploration logs	43	20	23		89.5	
▼	See exploration logs	41	16	25			

Project No. 15426.000.000 **Client:** Monte Vista Memorial Investment Group, LLC

Project: 3656 Las Colinas Road

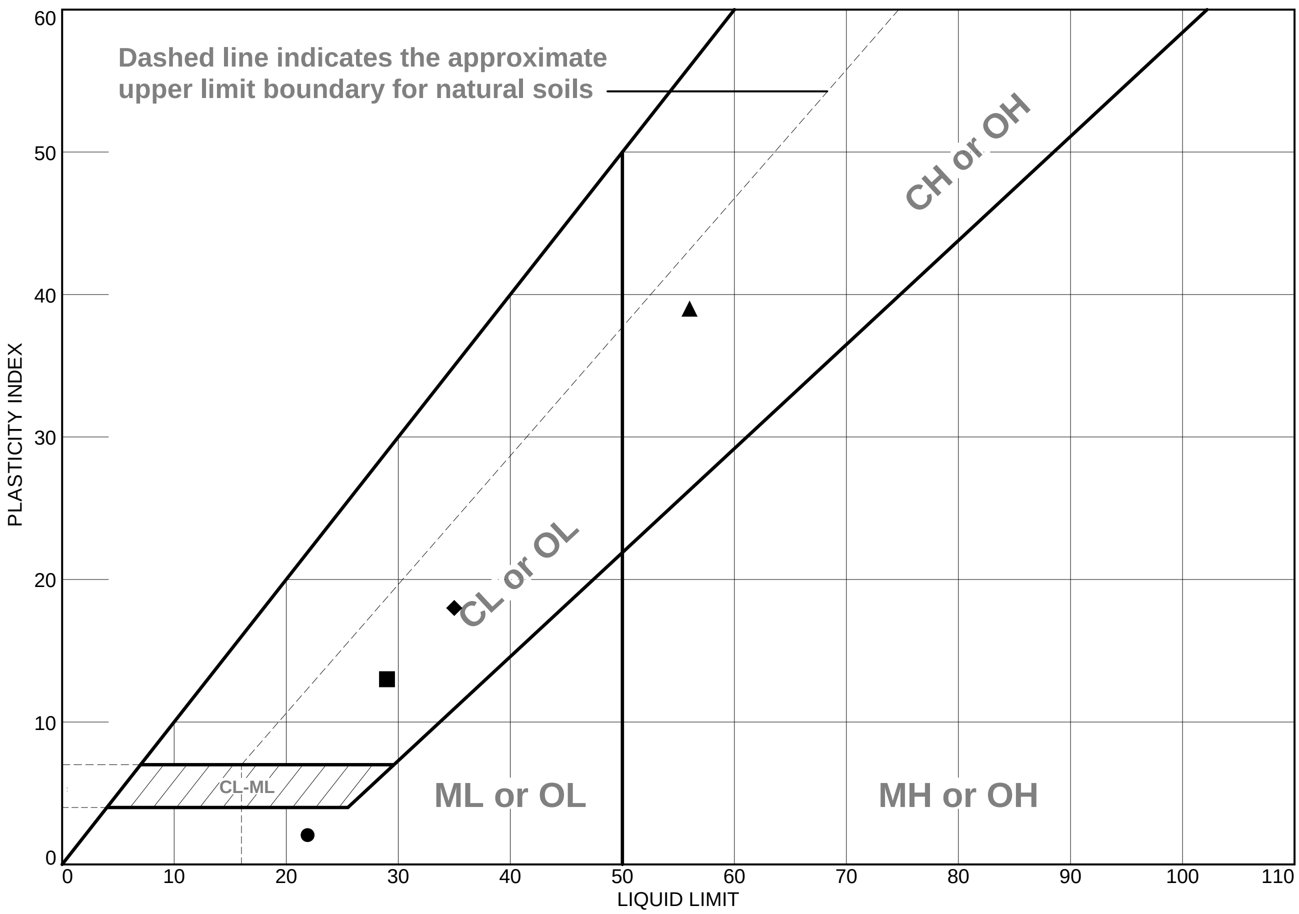
●**Sample Number:** 1-B1 @ 6
■**Sample Number:** 1-B1 @ 21
▲**Sample Number:** 1-B1 @ 28
◆**Sample Number:** 1-B1 @ 43
▼**Sample Number:** 1-B2 @ 6

ENGEO INCORPORATED

Remarks:
●ASTM D4318, Wet method
■PI: ASTM D4318, Wet method
GS: ASTM D1140
▲PI: ASTM D4318, Wet method
GS: ASTM D1140, Method B
◆PI: ASTM D4318, Wet method
GS: ASTM D1140
▼ASTM D4318, Wet method

Tested By: ○ M. Quasem □ M. Quasem △ M. Bromfield ◇ M. Quasem ▽ M. Quasem **Checked By:** M. Bromfield

LIQUID AND PLASTIC LIMITS TEST REPORT



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	See exploration logs	22	20	2		23.0	
■	See exploration logs	29	16	13			
▲	See exploration logs	56	17	39			
◆	See exploration logs	35	17	18		19.2	

Project No. 15426.000.000 **Client:** Monte Vista Memorial Investment Group, LLC

Project: 3656 Las Colinas Road

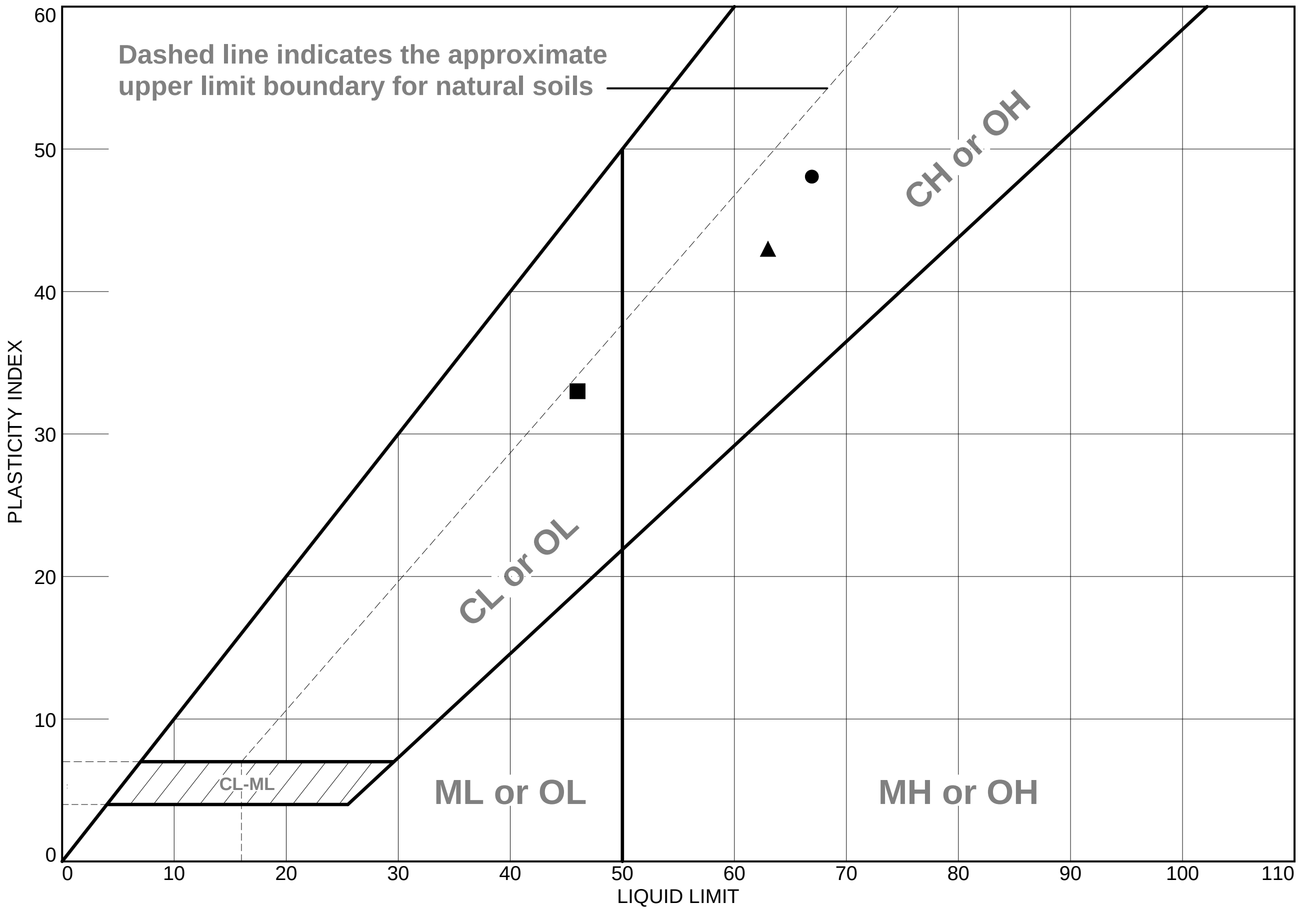
●**Sample Number:** 1-B2 @ 16.5
■**Sample Number:** 1-B2 @ 28
▲**Sample Number:** 1-B3 @ 6
◆**Sample Number:** 1-B3 @ 16

ENGEO INCORPORATED

Remarks:
●PI: ASTM D4318, Wet method
GS: ASTM D1140, Method B
■ASTM D4318, Wet method
▲ASTM D4318, Wet method
◆PI: ASTM D4318, Wet method
GS: ASTM D1140, Method B

Tested By: ○ M. Bromfield □ M. Quasem △ M. Quasem ◇ M. Bromfield **Checked By:** M. Quasem

LIQUID AND PLASTIC LIMITS TEST REPORT



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	See exploration logs	67	19	48			
■	See exploration logs	46	13	33			
▲	See exploration logs	63	20	43			

Project No. 15426.000.000 **Client:** Monte Vista Memorial Investment Group, LLC

Project: 3656 Las Colinas Road

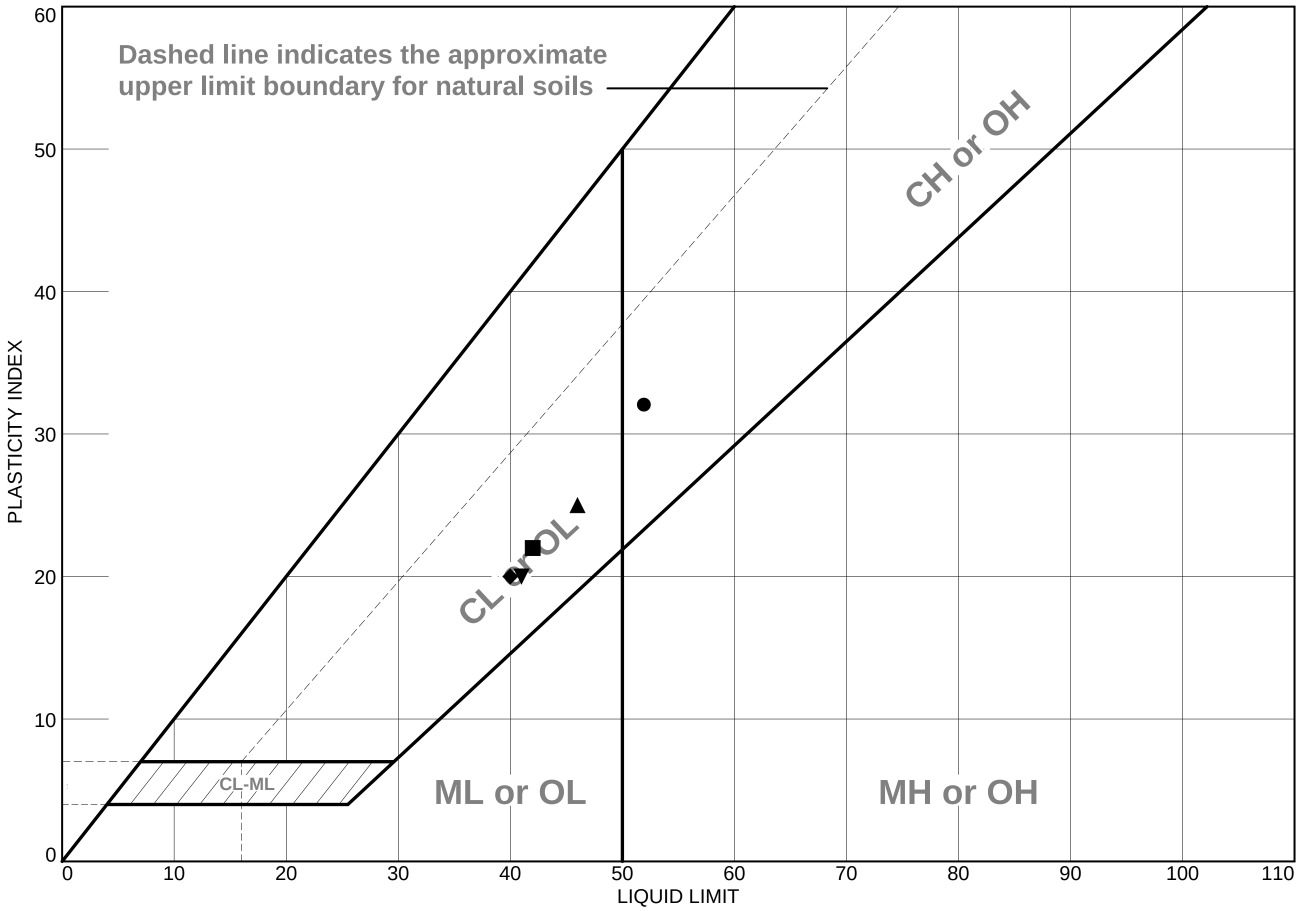
●**Sample Number:** 1-B6 @ 6
■**Sample Number:** 1-B11 @ 4
▲**Sample Number:** 1-B15 @ 5.5

ENGEO INCORPORATED

Remarks:
●ASTM D4318, Wet method
■ASTM D4318, Wet method
▲ASTM D4318, Wet method

Tested By: ○ M. Bromfield □ M. Quasem △ M. Quasem **Checked By:** M. Quasem

LIQUID AND PLASTIC LIMITS TEST REPORT



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	See exploration logs	52	20	32			
■	See exploration logs	42	20	22			
▲	See exploration logs	46	21	25			
◆	See exploration logs	40	20	20		93.3	
▼	See exploration logs	41	21	20		79.9	

Project No. 15426.000.000 **Client:** Monte Vista Memorial Investment Group, LLC

Project: 3656 Las Colinas Road

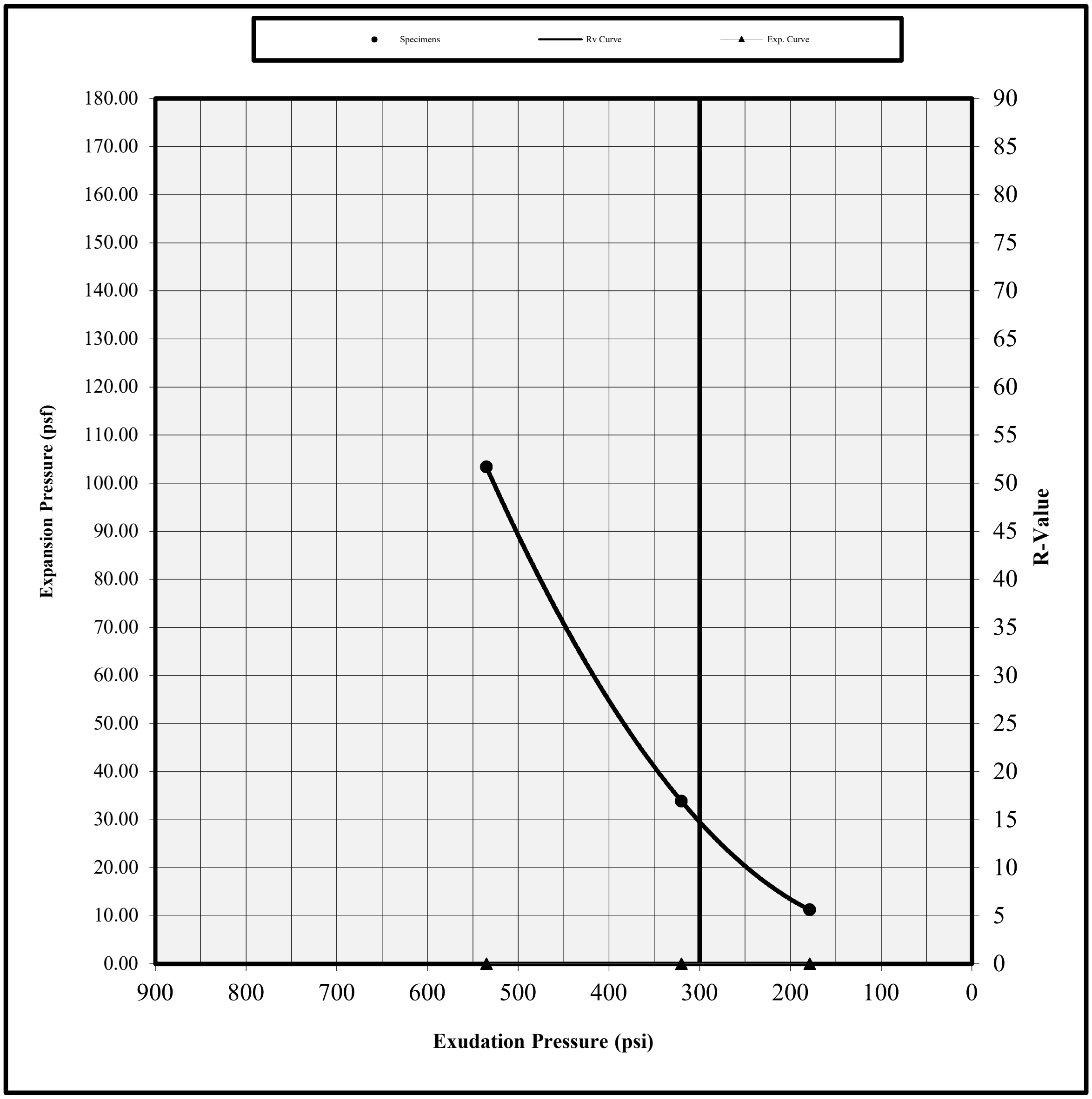
●**Sample Number:** 1-B4 @ 16
■**Sample Number:** 1-B5 @ 11.5
▲**Sample Number:** 1-B5 @ 28
◆**Sample Number:** 1-B5 @ 38
▼**Sample Number:** 1-TP4 @ 11-12

ENGEO INCORPORATED

Remarks:
●ASTM D4318, Wet method
■ASTM D4318, Wet method
▲PI: ASTM D4318, Wet method
GS: ASTM D422
◆PI: ASTM D4318, Wet method
GS: ASTM D1140, Method B
▼PI: ASTM D4318, Wet method
GS: ASTM D1140, Method B

Tested By: M. Bromfield **Checked By:** M. Quasem

R VALUE TEST REPORT
ASTM D2844



Sample ID/Location: 1-TP2@4.5
Description: See exploration logs

Specimen	Specimen 1	Specimen 2	Specimen 3
Exudation Pressure (p.s.i.)	535	320	179
Expansion dial (0.0001")	0	0	0
Expansion Pressure (p.s.f.)	0	0	0
Resistance Value, "R"	52	17	6
% Moisture at Test	11.8	14.4	16.1
Dry Density at Test, p.c.f.	117.5	117.1	113.4
Minimum Design R-Value	NA		
"R" Value at Exudation Pressure of 300 psi.	15		
Expansion Pressure (psf) at Exudation Pressure of 300 psi.	0		

PROJECT NAME: 3656 Las Colina Road
DATE: 11/19/18
PROJECT NUMBER: 15426.000.000
CLIENT: Monte Vista Memorial Investment Group, LLC
PHASE NUMBER: 1

ENGEO
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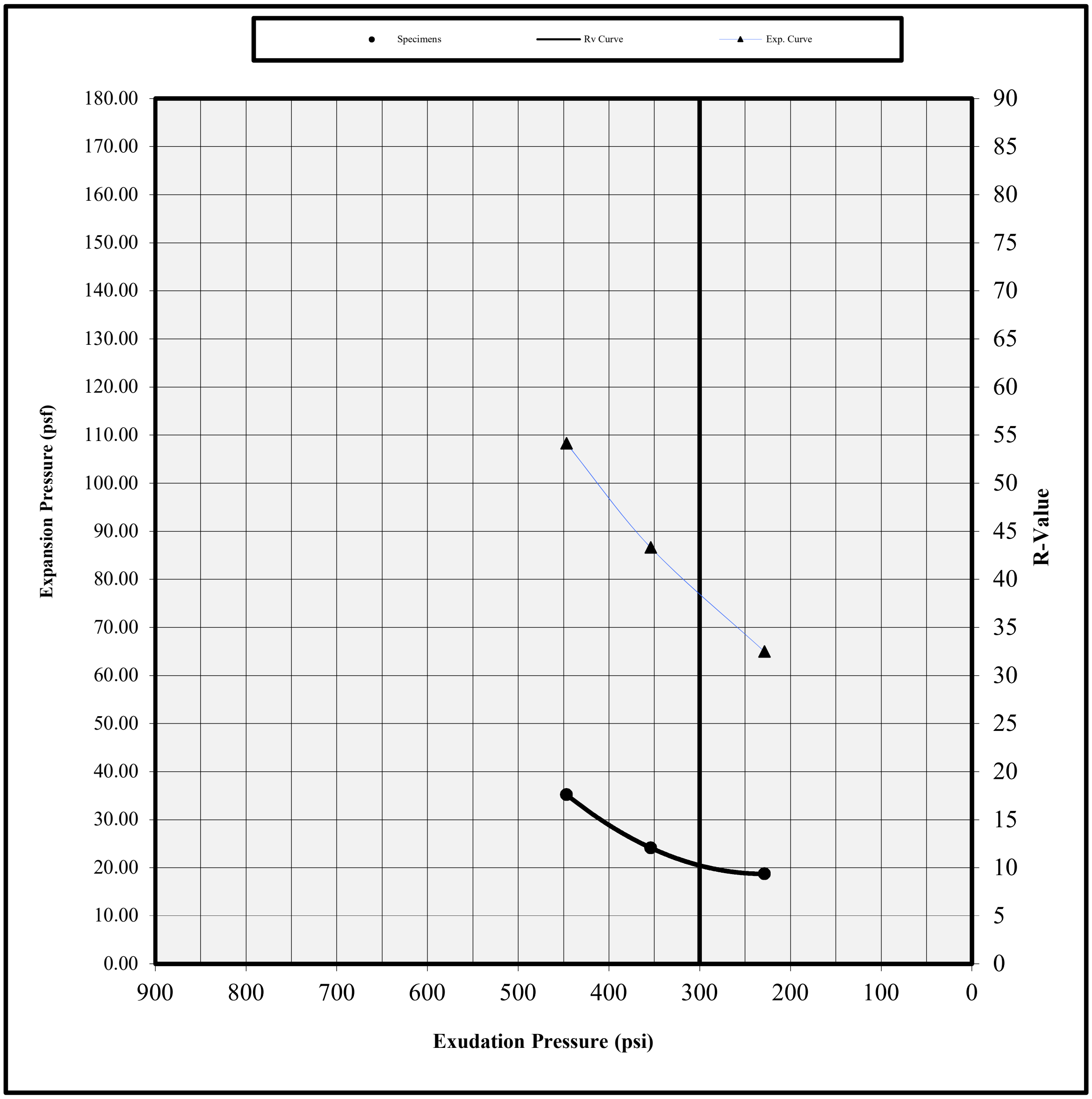
Tested by: R. Montalvo
Reviewed by: M. Gilbert

Lab Address: 2213 Plaza Drive, Rocklin, CA 95765

R VALUE TEST REPORT
ASTM D2844



Sample ID/Location: Bulk 1 ft. to 1.5 ft.
Description: See exploration logs

Specimen	Specimen 1	Specimen 2	Specimen 3
Exudation Pressure (p.s.i.)	447	354	229
Expansion dial (0.0001")	25	20	15
Expansion Pressure (p.s.f.)	108	87	65
Resistance Value, "R"	18	12	9
% Moisture at Test	23.8	25.2	27.0
Dry Density at Test, p.c.f.	98.4	96.2	93.0
Minimum Design R-Value	NA		
"R" Value at Exudation Pressure of 300 psi.	11		
Expansion Pressure (psf) at Exudation Pressure of 300 psi.	78		

PROJECT NAME: 3656 Las Colina Road
PROJECT NUMBER: 15426.000.000
CLIENT: Monte Vista Memorial Investment Group, LLC
PHASE NUMBER: 1

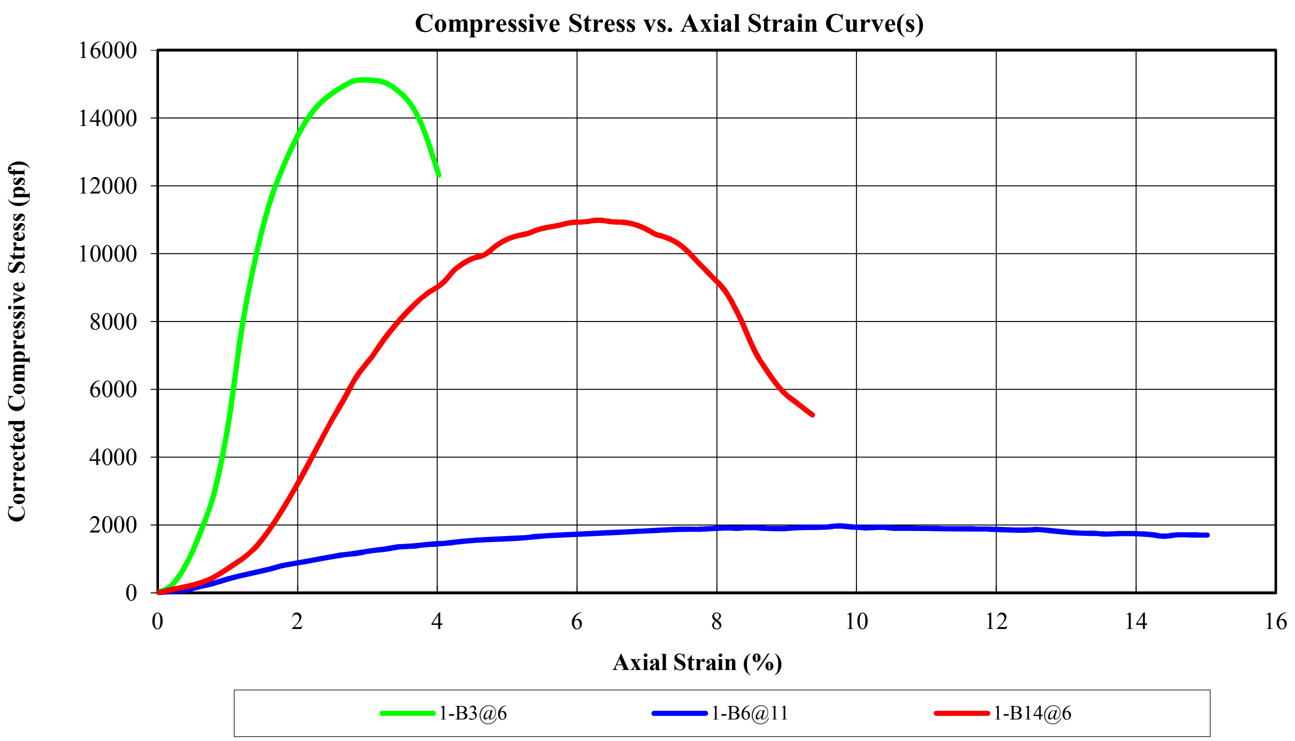
DATE: 11/19/18

ENGEO Expect Excellence

Tested by: R. Montalvo   Reviewed by: M. Gilbert

Lab Address: 2213 Plaza Drive, Rocklin, CA 95765

UNCONFINED COMPRESSION TEST REPORT
(ASTM D2166)



BEFORE TEST	SPECIMEN 1-B3@6	1-B6@11	1-B14@6
Moisture Content (%)	17.6	26.0	25.0
Dry Density (pcf)	110.2	103.2	96.2
Saturation (%)	93.2	100.0	92.0
Void Ratio	0.50	0.60	0.72
Diameter (in)	2.412	2.410	2.419
Height (in)	5.11	5.06	5.05
Height-To-Diameter Ratio	2.12	2.10	2.09
TEST DATA			
Unconfined Compressive Strength (psf)	15117	1973	10988
Undrained Shear Strength (psf)	7558	987	5494
Strain Rate (in./min.)	0.05	0.05	0.05
Specific Gravity (Assumed)	2.650	2.650	2.650
Strain at Failure (%)	3.03	9.73	6.30
Liquid Limit	-	-	-
Plastic Limit	-	-	-
Test Remarks			

SPECIMEN	DESCRIPTION
1-B3@6	See exploration logs
1-B6@11	See exploration logs
1-B14@6	See exploration logs

PROJECT NAME: 3656 Las Colinas Road
PROJECT NO: 15426.000.000
CLIENT: Monte Vista Memorial Investment Group, LLC
LOCATION: Livermore, CA
PHASE NO: 001

Test Date: 11/05/18
Tested By: M. Quasem
Reviewed By: M. Bromfield

ENGEO
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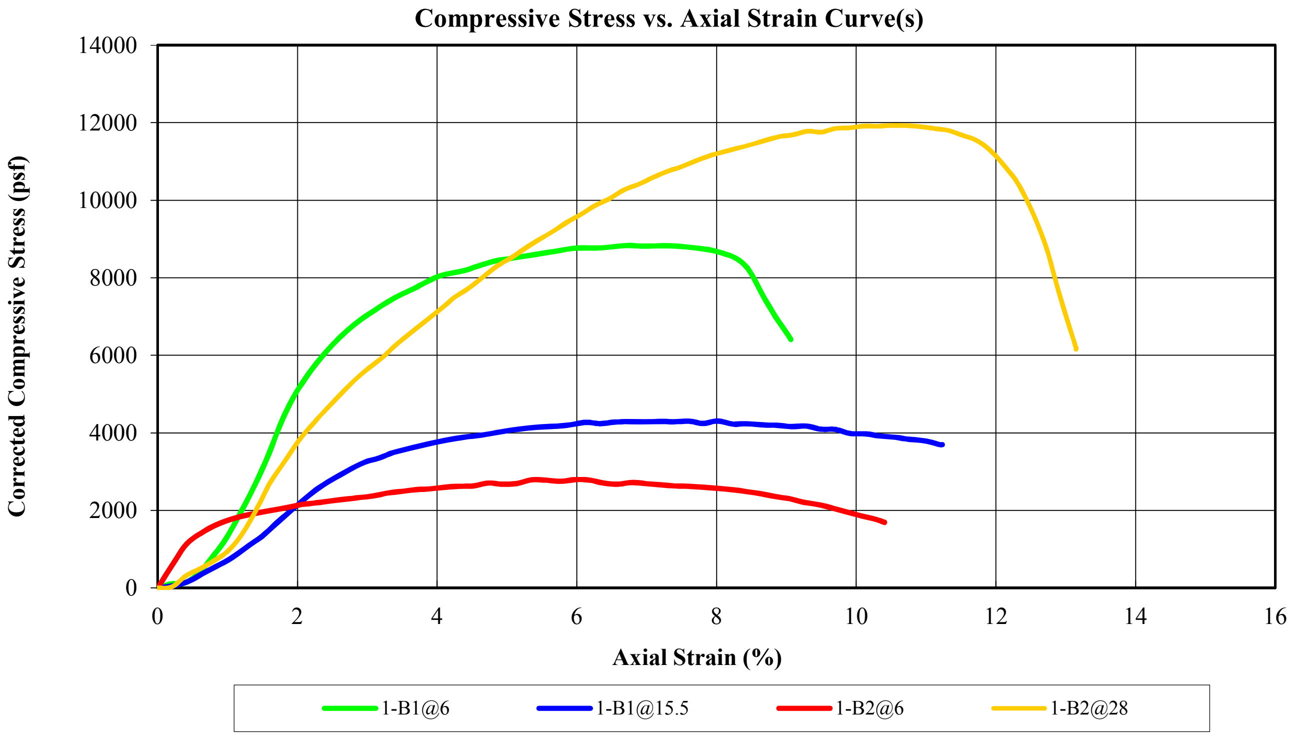
UNCONFINED COMPRESSION TEST REPORT
(ASTM D2166)



	SPECIMEN			
BEFORE TEST	**1-B1@6**	**1-B1@15.5**	**1-B2@6**	**1-B2@28**
Moisture Content (%)	15.7	16.6	17.7	15.8
Dry Density (pcf)	114.3	107.4	103.5	115.2
Saturation (%)	93.0	81.4	78.3	96.2
Void Ratio	0.45	0.54	0.60	0.44
Diameter (in)	2.420	2.418	2.420	2.400
Height (in)	5.34	4.87	4.99	5.09
Height-To-Diameter Ratio	2.21	2.01	2.06	2.12
TEST DATA				
Unconfined Compressive Strength (psf)	8834	4304	2791	11930
Undrained Shear Strength (psf)	4417	2152	1396	5965
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 (%)	6.73	8.02	5.97	10.51
Liquid Limit	-	-	-	-
Plastic Limit	-	-	-	-
Test Remarks				

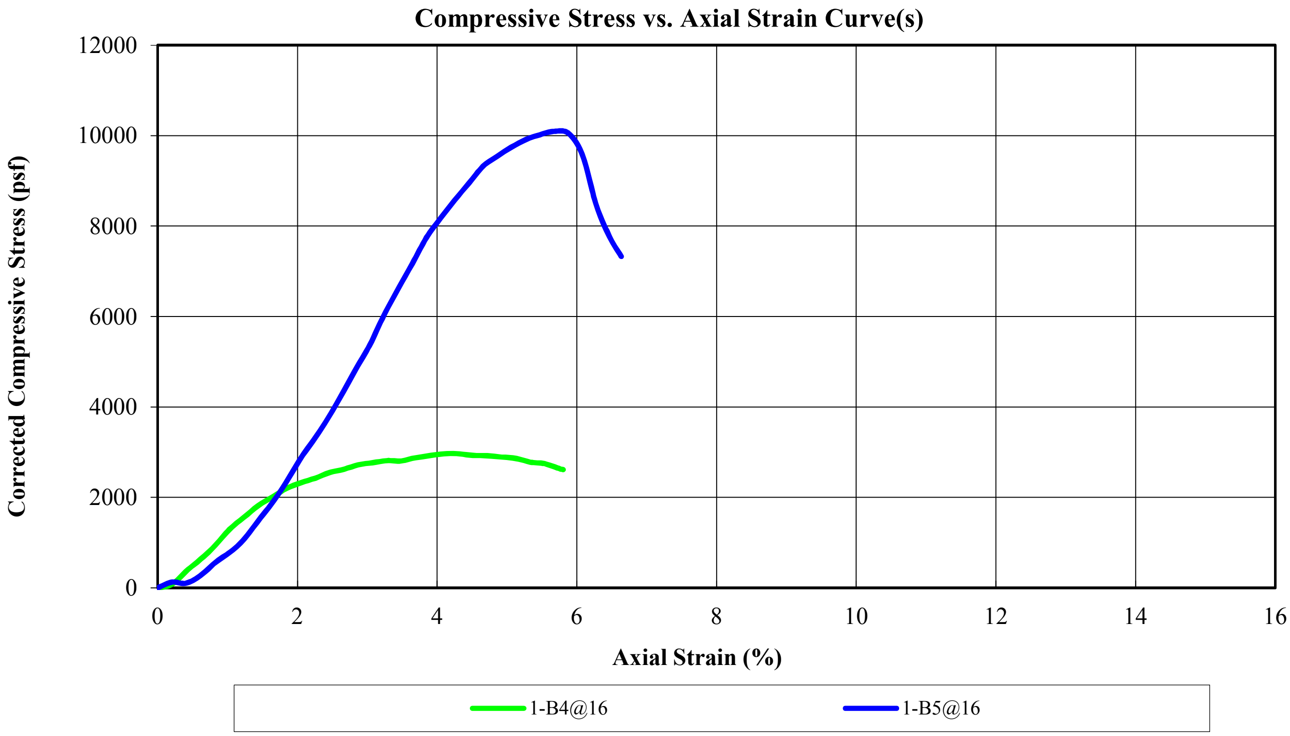
SPECIMEN	DESCRIPTION
1-B1@6	See exploration logs
1-B1@15.5	See exploration logs
1-B2@6	See exploration logs
1-B2@28	See exploration logs

ENGEO
Expect Excellence

PROJECT NAME: 3656 Las Colinas Road
PROJECT NO: 15426.000.00
CLIENT: Monte Vista Memorial Investment Group, LLC
LOCATION: Livermore, CA
PHASE NO: 001

Test Date: 11/05/18
Tested By: M. Quasem
Reviewed By: M. Bromfield

UNCONFINED COMPRESSION TEST REPORT
(ASTM D2166)



BEFORE TEST	SPECIMEN 1-B4@16	SPECIMEN 1-B5@16
Moisture Content (%)	27.8	21.5
Dry Density (pcf)	91.3	117.0
Saturation (%)	90.7	100.0
Void Ratio	0.81	0.41
Diameter (in)	2.380	2.427
Height (in)	5.01	5.07
Height-To-Diameter Ratio	2.11	2.09
TEST DATA		
Unconfined Compressive Strength (psf)	2968	10094
Undrained Shear Strength (psf)	1484	5047
Strain Rate (in./min.)	0.05	0.05
Specific Gravity (Assumed)	2.650	2.650
Strain at Failure (%)	4.11	5.67
Liquid Limit	-	-
Plastic Limit	-	-
Test Remarks		

SPECIMEN	DESCRIPTION
1-B4@16	See exploration logs
1-B5@16	See exploration logs

PROJECT NAME: 3656 Las Colinas Road
PROJECT NO: 15426.000.000
CLIENT: Monte Vista Memorial Investment Group, LLC
LOCATION: Livermore, CA
PHASE NO: 001

Test Date: 11/21/18
Tested By: M. Quasem
Reviewed By: M. Bromfield

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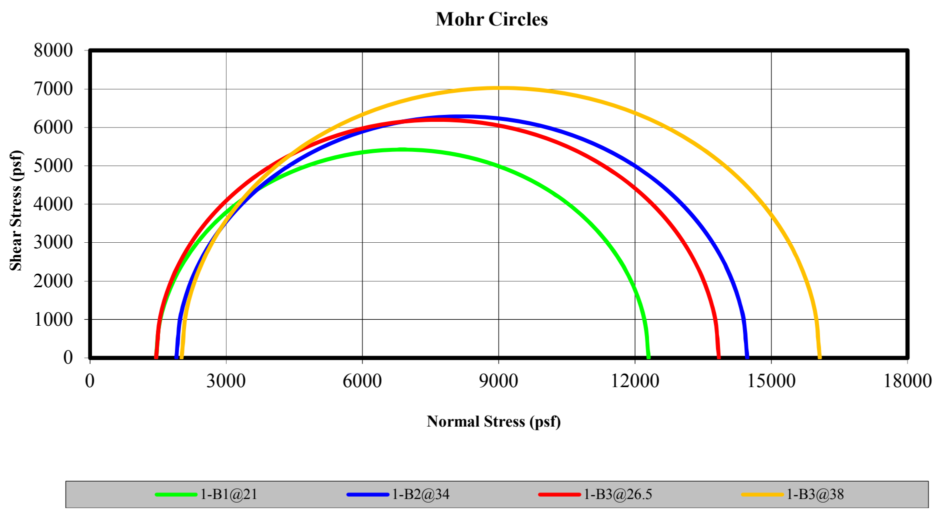
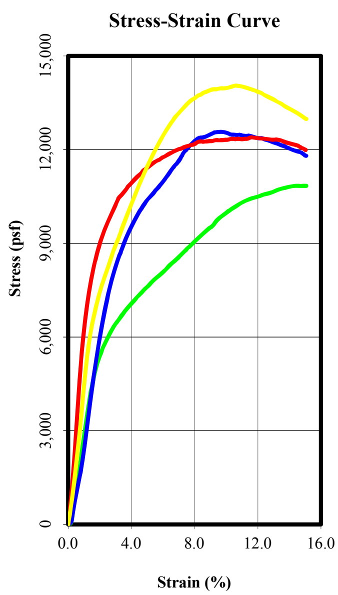
Isotropic Unconsolidated Undrained Triaxial Test

(ASTM D2850)

Tested By: M. Quasem   Date: 11/5/2018
Checked By: M. Bromfield   Date: 11/05/18





	Specimen			
Before Test	**1-B1@21**	**1-B2@34**	**1-B3@26.5**	**1-B3@38**
Water Content (%)	23.27	26.39	24.29	25.31
Dry Density (pcf)	100.80	99.33	101.53	100.52
Saturation (%)	96.16	100.00	99.34	100.00
Void Ratio	0.64	0.72	0.66	0.68
Diameter (in)	2.430	2.399	2.418	2.420
Height (in)	5.060	5.100	5.051	5.030
Height-to-Diameter Ratio	2.082	2.126	2.089	2.079
ASTM D4318 - Wet Method				
Liquid Limit	-	-	-	-
Plastic Limit	-	-	-	-
ASTM D854 - Measured				
Specific Gravity (Assumed)	2.650	2.740	2.700	2.700
After Test	**1-B1@21**	**1-B2@34**	**1-B3@26.5**	**1-B3@38**
Water Content (%)	23.27	26.39	24.29	25.31
Saturation (%)	96.16	100.00	99.34	100.00
Strain Rate (in/min)	0.05	0.05	0.05	0.05
Peak Deviator Stress (psf)	10842.6	12568.7	12391.9	14050.1
Axial Strain @ Failure (%)	14.524	9.672	11.560	10.607
	Cell Pressure			
Cell (psf)	1454.4	1900.8	1454.4	2016.0
Back (psf)	n/a	n/a	n/a	n/a
	Principle Stresses at Failure			
σ1 (psf)	12297.0	14469.5	13846.3	16066.1
σ3 (psf)	1454.4	1900.8	1454.4	2016.0
Corrected Peak Deviator Stress				

Mohr-Coulomb Parameters with a Non-zero Friction Angle (Ø≠0)			**Cohesion at Failure with a Zero Friction Angle (Ø=0)**			
Cohesion, c (psf)	0.0		5421.3	6284.3	6195.9	7025.1
Friction Angle Ø	0.00		n/a	n/a	n/a	n/a

Project Information	
Project Name:	3656 Las Colinas Road
Project Number:	15426.000.000
Project Location:	Livermore, CA
Client:	Monte Vista Memorial Investment Group, LLC
Description:	See exploration logs
Test Remarks:	

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PERMEABILITY TEST REPORT

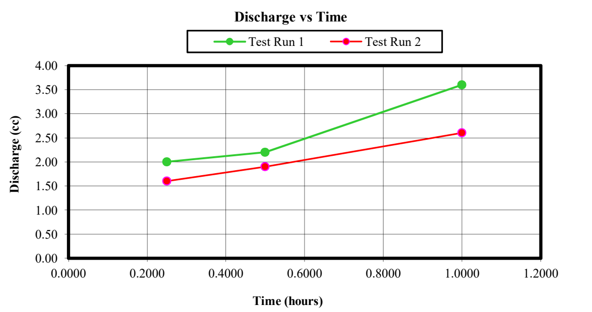
ASTM D5084, Method C

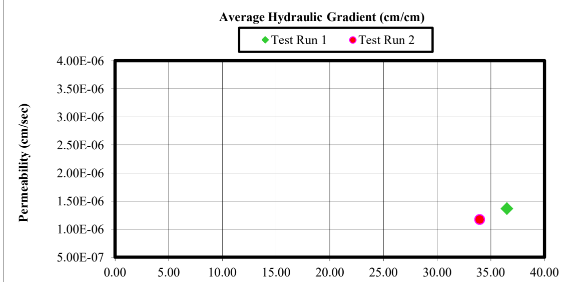
TEST DATA

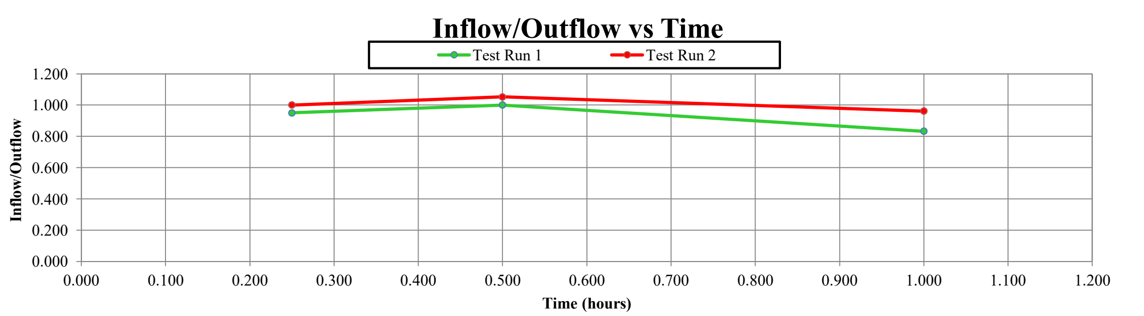
	Run 1	Run 2
Specimen Diameter (in.):	2.400	2.400
Specimen Height (in.):	2.055	2.055
Dry Density Before Test (pcf):	98.6	98.6
Moisture Content Before Test (%):	26.0%	26.0%
Moisture Content After Test (%):	24.4%	24.4%
Cell Pressure During Test (psi):	36.1	36.1
Back Pressure During Test (psi):	25.0	25.0
Effective Stress (σ_3) (psf):	1598.4	1598.4
Flow Rate (cc/sec):	1.48E-03	1.19E-03
Permeability (cm/sec):	1.37E-06	1.17E-06
B-Value:	0.99	0.99
Permeant Fluid:	Tap Water	

SPECIMEN DATA

Sample Identification:	1-B5@38			
Sample Type:	undisturbed			
%Gravel:	0.0	%Sand:	%Silt/Clay:	
Atterberg Limits:	LL: 40	PL: 20	PI: 20	USCS:
Post Test Length Change(in.):	0.0181			
Maximum Dry Density (pcf):	n/a			
Optimum Moisture Content (%):	n/a			
Compaction (%):	n/a			
Compaction Method:	Tamping a known mass to a known volume			
Permeameter Type:	**Flexible Wall**			
Type of Test:	**Falling Head--Rising Tailwater Pressure**			
Remarks:	Test results have been corrected to reflect a temperature of 20° Celsius.			







	Before Test	After test
Specimen Data	Initial	Final
Diameter, in	2.422	2.400
Height, in	2.073	2.055
Area, in^2	4.606	4.524
Volume, ft^3	0.0055	0.0054
Specific Gravity of Sample (measured)	2.689	2.689
Weight of Sample, g	311.1	307.1
Total Volume (cc)	156.46	152.33
Vol. of Solids (cc)	93.02	90.56
Vol. of Voids (cc)	63.45	61.77
Porosity	41.3306	39.7381
Void Ratio, e	0.7017	0.6568
Saturation %	99.75%	100.00%
Tare Weight, g	158.9	0.0
Gross Wet Soil Weight, g	191.33	313.82
Gross Dry Soil Weight, g	184.64	252.18
Water Content, %	26.0%	24.4%
Wet Unit Weight, pcf	124.3	126.0
Dry Unit Weight, pcf	98.6	101.3
Hydraulic Gradient, Test Run 1		36.48
Hydraulic Gradient, Test Run 2		33.95
Δ[Hydraulic Gradient] (Run 1 - Run 2)		2.53

Project Name: 3656 Las Colinas Road
Project Location: Livermore, California
Project Number: 15426.000.000
Phase Number: 001
Tested By: G. Criste **Reviewed by:** D. Seibold
Date: 12/05/18

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Laboratory Address: 3420 Fostoria Way, Suite E, Danville, CA 94526. Phone No. (925)355-9047.

Remolding Specifications for Permeability

PROJECT NAME: **3656 Las Colinas Road**
PROJECT NUMBER: **15426.000.000**
CLIENT: **Monte Vista Memorial Investment Group, LLC**
PHASE NUMBER: **001** **DATE:** **11/28/18**

SAMPLE ID: **1-TP4@11-12**

Curve Maximum Density:	107.8	pcf	**Height (in) :**	**2.000**
Curve Optimum Moisture:	15.8	%	**Diam (in) :**	**2.421**

Moisture Content of sample to be remolded:

Tare name:	
Wet soil + tare:	442.10
Dry soil + tare:	418.49
Tare weight:	256.21
	14.55 %

Vol (ft^3) : 0.0053

Compact To: **95** % at **2** % over optimum moisture

Desired moisture content at remolding/compaction:

15.8	+	2.0	=	17.8 %
17.8	-	14.55	=	3.2511 g

*Water to be added (g): **3.25** to **114.55** g soil

Weight of remolding material (g): **300.00**
Water to be added to remolding material (g): **8.51**

WET DENSITY CALCULATION:

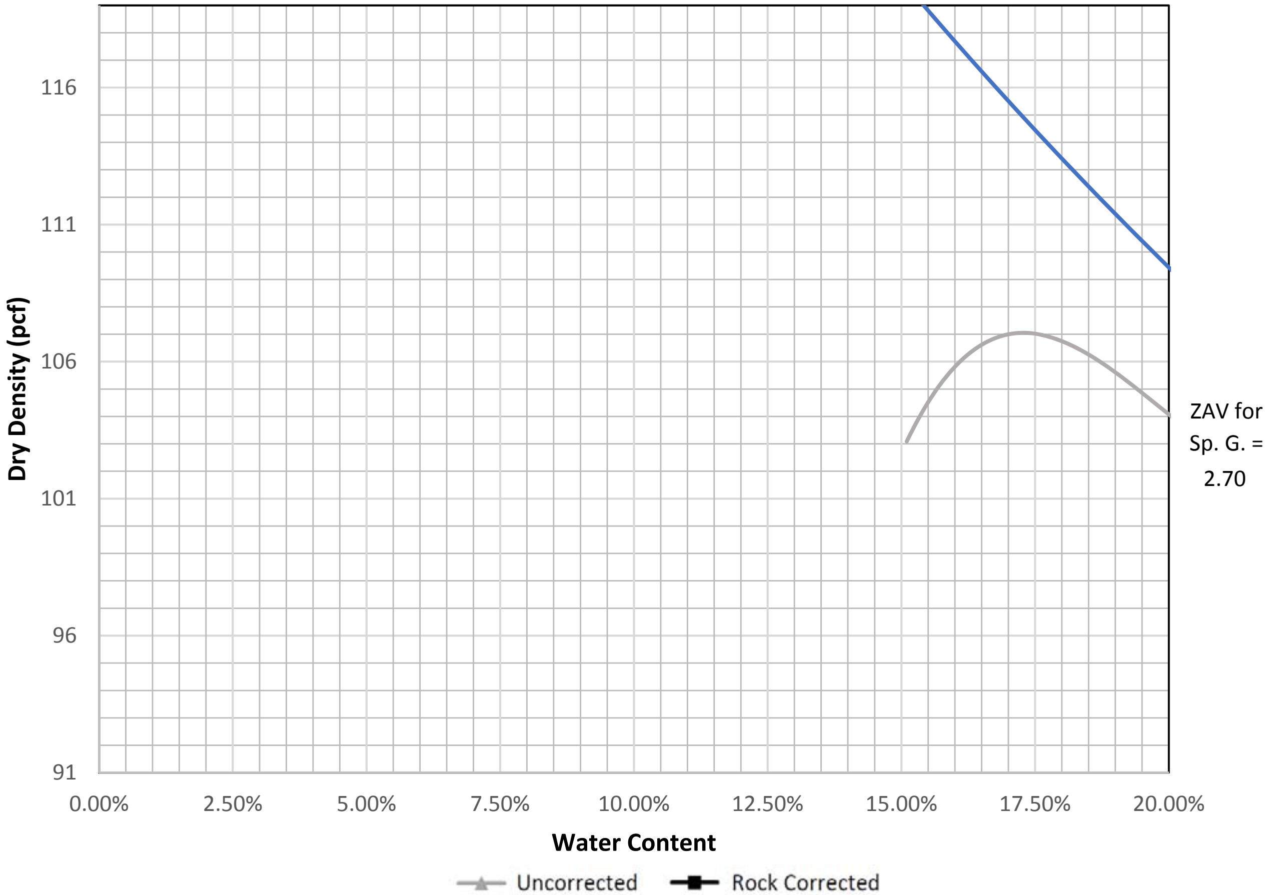
Dry Density (remold): 0.95 * 107.8 = 102.4 lb/ft^3
Wet density (lb/ft^3): 120.64 [dry density * (1+(MC/100))]

Multiply wet density (pcf) w/ volume (cf) and by 453.6 grams per lb. to get sample size for remolding:

120.6 lb/ft^3 * 0.0053 ft^3 * 453.6 = 291.6 g

291.56 grams of soil needed for remolding
145.78 g per lift (2 lifts)

COMPACTION TEST RESULT for Curve No. N/A



Test specification: Method A

Elev/ Depth	Classification		Nat. Moist.	Sp.G.	LL	PI	% > 3/8 in.	% > No.200
	USCS	AASHTO						
				2.7				

ROCK CORRECTED RESULTS	UNCORRECTED	MATERIAL DESCRIPTION
Maximum dry density = 0 pcf	107.1 pcf	Yellowish brown sandy CLAY
Optimum moisture = 0%	17.3%	**ASTM METHODS USED:**

Client: Monte Vista Memorial Investment Group, LLC
Project: 3656 Las Colinas Road
Project No. 15426.000.000 **Date:**
Location: 1-TP4 @ 11-12 **Curve No.** N/A

Remarks:

ASTM C127
ASTM D4318
ASTM D6913

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Tested By: G. Criste **Checked By:** M. Bromfield

PERMEABILITY TEST REPORT

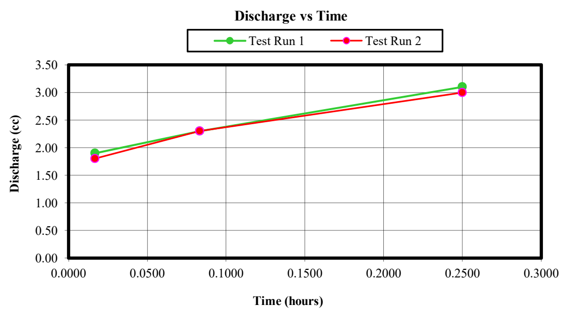
ASTM D5084, Method C

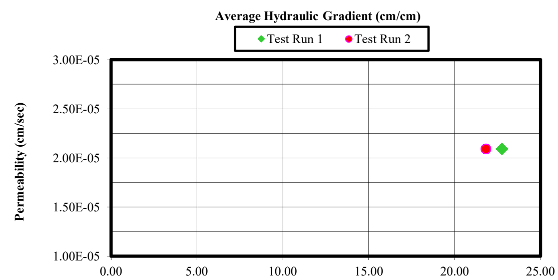
TEST DATA

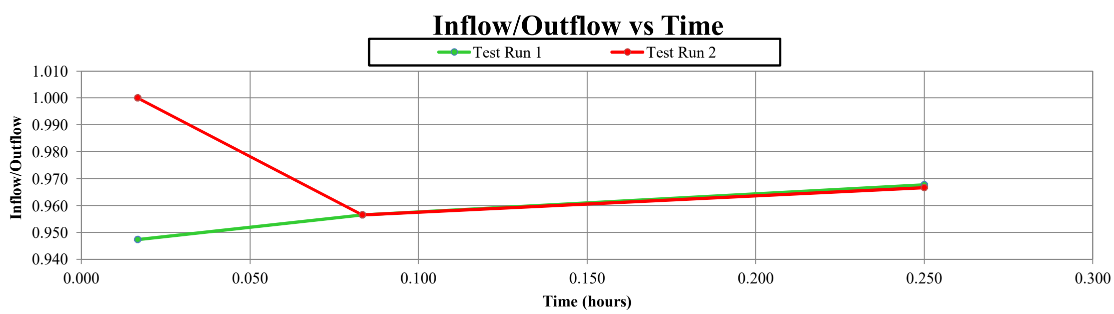
	Run 1	Run 2
Specimen Diameter (in.):	2.424	2.424
Specimen Height (in.):	1.998	1.998
Dry Density Before Test (pcf):	101.7	101.7
Moisture Content Before Test (%):	17.8%	17.8%
Moisture Content After Test (%):	24.5%	24.5%
Cell Pressure During Test (psi):	24.7	24.7
Back Pressure During Test (psi):	15.0	15.0
Effective Stress (σ_3) (psf):	1396.8	1396.8
Flow Rate (cc/sec):	1.43E-02	1.37E-02
Permeability (cm/sec):	2.09E-05	2.09E-05
B-Value:	0.99	0.99
Permeant Fluid:	Tap Water	

SPECIMEN DATA

Sample Identification:	1-TP4@11-12			
Sample Type:	Remolded			
%Gravel:	0.0	%Sand: 20.1	%Silt/Clay: 79.9	
Atterberg Limits:	LL: 41	PL: 21	PI: 20	USCS: CL
Post Test Length Change(in.):	0.0048			
Maximum Dry Density (pcf):	107.8			
Optimum Moisture Content (%):	15.8			
Compaction (%):	95%			
Compaction Method:	Tamping a known mass to a known volume			
Permeameter Type:	**Flexible Wall**			
Type of Test:	**Falling Head--Rising Tailwater Pressure**			
Remarks:	Test results have been corrected to reflect a temperature of 20° Celsius. Specimen remolded to 95%RC of 107.8pcf at 17.8%MC (+2% over opt)			







	Before Test	After test
Specimen Data	Initial	Final
Diameter, in	2.431	2.424
Height, in	2.003	1.998
Area, in^2	4.641	4.616
Volume, ft^3	0.0054	0.0053
Specific Gravity of Sample (measured)	2.743	2.743
Weight of Sample, g	291.8	308.4
Total Volume (cc)	152.32	151.16
Vol. of Solids (cc)	90.63	89.94
Vol. of Voids (cc)	61.69	61.22
Porosity	40.7024	40.2488
Void Ratio, e	0.6837	0.6709
Saturation %	71.35%	100.00%
Tare Weight, g	0.0	85.5
Gross Wet Soil Weight, g	100.00	396.22
Gross Dry Soil Weight, g	84.90	335.12
Water Content, %	17.8%	24.5%
Wet Unit Weight, pcf	119.7	127.5
Dry Unit Weight, pcf	101.7	102.4
Hydraulic Gradient, Test Run 1		22.75
Hydraulic Gradient, Test Run 2		21.83
Δ[Hydraulic Gradient] (Run 1 - Run 2)		0.92

Project Name: 3656 Las Colinas Road
Project Location: Livermore, California
Project Number: 15426.000.000
Phase Number: 001
Tested By: G. Criste **Reviewed by:** D. Seibold
Date: 12/12/18

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WATER SOLUBLE SULFATES IN SOILS

ASTM C1580

Sample number	Sample Location / ID	Matrix	Water Soluble Sulfate % by mass
1	1-B1@5.5	soil	ND
2	1-B2@25	soil	ND
3	1-B3@11.5	soil	ND

Remarks: Results are reported to the nearest 100mg/kg. Anything less than 50mg/kg will be reported as 'ND' for Not-Detectable.

PROJECT NAME: 3656 Las Colinas Road
PROJECT NUMBER: 15426.000.000
CLIENT: Monte Vista Memorial Investment Group, LLC
PHASE NUMBER: 001

DATE: 11/15/18

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Lab Address: 2213 Plaza Drive, Rocklin, CA 95765. Phone No. (916) 279-2698

WATER SOLUBLE SULFATES IN SOILS

ASTM C1580

Sample number	Sample Location / ID	Matrix	Water Soluble Sulfate % by mass
1	1-B4 @ 21	soil	ND

Remarks: Results are reported to the nearest 100mg/kg. Anything less than 50mg/kg will be reported as 'ND' for Not-Detectable.

PROJECT NAME: 3656 Las Colinas Road
PROJECT NUMBER: 15426.000.000
CLIENT: Monte Vista Memorial Investment Group, LLC
PHASE NUMBER: 001

DATE: 11/26/18

ENGEO
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Tested by: M. Quasem   Reviewed by: M. Bromfield

Lab Address: 3420 Fostoria Way Suite E, Danville, CA 94526. Phone No. (925) 355-9047

WATER SOLUBLE SULFATES IN SOILS

ASTM C1580

Sample number	Sample Location / ID	Matrix	Water Soluble Sulfate % by mass
1	1-B6 @ 5.5	soil	0.01

Remarks: Results are reported to the nearest 100mg/kg. Anything less than 50mg/kg will be reported as 'ND' for Not-Detectable.

PROJECT NAME: 3656 Las Colinas Road
PROJECT NUMBER: 15426.000.000
CLIENT: Monte Vista Memorial Investment Group, LLC
PHASE NUMBER: 001

DATE: 11/02/18

ENGEO
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Tested by: M. Quasem

Reviewed by: M. Bromfield

Lab Address: 3420 Fostoria Way Suite E, Danville, CA 94526. Phone No. (925) 355-9047

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

15 November, 2018

Job No. 1811013
Cust. No. 10169

Mr. Eric Keifer
ENGEO Inc.
2010 Crow Canyon Place, Suite 250
San Ramon, CA 94583

Subject: Project No.: 15426.000.000
Project Name: 3656 Las Colinas Road
Corrosivity Analysis – ASTM Test Methods

Dear Mr. Keifer:

Pursuant to your request, CERCO Analytical has analyzed the soil samples submitted on November 05, 2018. Based on the analytical results, this brief corrosivity evaluation is enclosed for your consideration.

Based upon the resistivity measurements, both sample is classified as "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 were 61 mg/kg & 62 mg/kg and are determined to be insufficient to attack steel embedded in a concrete mortar coating.

The sulfate ion concentration were 40 mg/kg & 120 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 were 7.38 & 8.67, which does not present corrosion problems for buried iron, steel, mortar-coated steel and reinforced concrete structures.

The redox potentials were 230-mV & 330-mV and 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.

Cheryl McMillen for

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

JDH/jdl
Enclosure

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

Client:	ENGEO Incorporated
Client's Project No.:	15426.000.000
Client's Project Name:	3656 Las Colinas Road
Date Sampled:	8-Oct-18
Date Received:	5-Nov-18
Matrix:	Soil
Authorization:	Signed Chain of Custody

Date of Report: 15-Nov-2018

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)*
1811013-001	1-B2 @ 11.5'	230	7.38	-	1,700	-	62	120
1811013-002	1-B3 @ 32'	330	8.67	-	1,200	-	61	40

Method:	ASTM D1498	ASTM D4972	ASTM D1125M	ASTM G57	ASTM D4658M	ASTM D4327	ASTM D4327
Reporting Limit:	-	-	10	-	50	15	15
	14-Nov-2018	13-Nov-2018	-	15-Nov-2018	-	13-Nov-2018	13-Nov-2018

* Results Reported on "As Received" Basis

Cheryl McMillen
Laboratory Director

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

18110/13

CU#10869

CHAIN OF CUSTODY RECORD

PROJECT NUMBER: 15426.000.000	PROJECT NAME: 3656 Las Colinas Road
SAMPLED BY: (SIGNATURE/PRINT) Spencer Wagnaar	
PROJECT MANAGER: Eric Keifer	
ROUTING: E-MAIL	swaganaar@engeo.com; EKiefer@engeo.com

	SAMPLE NUMBER	DATE	TIME	MATRIX	NUMBER OF CONTAINERS	CONTAINER SIZE	PRESERVATIVE	Redox, pH, sulfate, resistivity (100% sat.) chloride	REMARKS REQUIRED DETECTION LIMITS
001	1-B2 @ 11.5 ft.	10/08/18	9:00 AM	soil	1	bag	none	X	Sample with brief evaluation
002	1-B3 @ 32 ft.	10/08/18	11:00 AM	soil	1	bag	none	X	Sample with brief evaluation

RELINQUISHED BY: (SIGNATURE)	DATE/TIME	RECEIVED BY: (SIGNATURE)	RELINQUISHED BY: (SIGNATURE)	DATE/TIME	RECEIVED BY: (SIGNATURE)
[signature]	11/5/18 1:17pm	[signature]	[signature]	11/5/18 1350	[signature]
RELINQUISHED BY: (SIGNATURE)	DATE/TIME	RECEIVED BY: (SIGNATURE)	RELINQUISHED BY: (SIGNATURE)	DATE/TIME	RECEIVED BY: (SIGNATURE)
RELINQUISHED BY: (SIGNATURE)	DATE/TIME	RECEIVED FOR LABORATORY BY: (SIGNATURE)	DATE/TIME		

Standard 10- day turnaround

ENGEO INCORPORATED
2010 CROW CANYON PLACE, SUITE 250
SAN RAMON, CALIFORNIA 94583
(925) 837-2973 FAX (888) 279-2698
WWW.ENGEO.COM

DISTRIBUTION: ORIGINAL ACCOMPANIES SHIPMENT; COPY TO PROJECT FIELD FILES

APPENDIX C

TEST PIT LOGS

ENGEO Expect Excellence	# Test Pit Log	Test Pit Number **1-TP1**
Project Name: **Las Colinas**		Lat.: **37.70641**
Project Location: **Livermore, California**		Long.: **-121.76015**
Project No.: **15426.000.000**	Logged By: **Eric M Kiefer** / Contractor: **Shryock Grading**	Equipment: **CAT 313L**
Date Started: **10/3/18**	Date Completed: **10/3/18** / Total Depth: **11 ft**	Groundwater: **N/A**

Depth (ft)	Soil/Rock Descriptions
0 – 3.5	**Sandy fat CLAY (CH)** – Dark gray, very stiff to hard (PP = 3.5 to >4.5 tsf), dry to slightly moist (well drained), some partings developed, carbonate streamers very common. [COLLUVIUM/TOPSOIL]
3.5 – 8	**Gravelly lean CLAY (CL)** – Yellowish brown to dark yellowish brown, hard (PP = >4.5 tsf), partings well developed, well rounded gravels up to approximately 1.5 inches, carbonate streamers and blebs, some gravels may be completely replaced. [LIVERMORE GRAVELS]
8 – 11	**Gravelly lean CLAY (CL)** – Yellowish brown, hard (operator commented "got much harder" around 8 feet), well rounded gravels up to approximately 1 inch, gravels appear more common, gravels appear to be highly or completely weathered. [LIVERMORE GRAVELS]

*Bottom of test pit at 11 feet. No groundwater encountered. No Caving. Excavator started having trouble excavating at approximately 11 feet.

Test Pit Log

Test Pit Number
1-TP2

Project Name: **Las Colinas**			Lat.: **37.70635**
Project Location: **Livermore, California**			Long.: **-121.76067**
Project No.: **15426.000.000**	Logged By: **Eric M Kiefer**	Contractor: **Shryock Grading**	Equipment: **CAT 313L**
Date Started: **10/3/18**	Date Completed: **10/3/18**	Total Depth: **Approx. 13.5 ft**	Groundwater: **N/A**

Depth (ft)	Soil/Rock Descriptions
0 – 3.5	**Sandy fat CLAY (CH)** – Dark gray, hard (PP = >4.5 tsf), dry to slightly moist (well drained), some partings, some carbonate streamers. [COLLUVIUM/TOPSOIL]
3.5 – 5	**Sandy lean CLAY/Clayey SAND (CL/SC)** – Yellowish brown, hard (PP = >4.5 tsf), slightly moist to dry, some partings, massive. [LIVERMORE GRAVELS]
5 – 7.5	**Silty SAND (SM)** – Light yellowish brown, very dense (PP = >4.5 tsf), dry to slightly moist, medium to fine grained sand, poorly graded (well sorted), very faint evidence of rock structure, massive. [LIVERMORE GRAVELS]
7.5 – ~12	**Gravelly SAND (SP)** – Light yellowish brown to pale olive, very dense (PP = >4.5 tsf), slightly moist, well rounded gravels up to approximately 1 inch, thinly bedded (gravels showing some imbrication), strike and dip 265°, 31° (?) bedding. Gravel size increased to up to 3 inches at approximately 11 feet. [LIVERMORE GRAVELS]
~12 – ~13.5	**Silty clayey SAND (SM/SC)** – Light yellowish brown to pale olive, very dense, slightly moist, clay cemented, some trace rock structure visible, massive. [LIVERMORE GRAVELS]

*Bottom of test pit at approximately 13.5 feet. No groundwater encountered. No Caving.

Test Pit Log

Test Pit Number
1-TP3

Project Name: **Las Colinas**			Lat.: **37.70569**
Project Location: **Livermore, California**			Long.: **-121.76163**
Project No.: **15426.000.000**	Logged By: **Eric M Kiefer**	Contractor: **Shryock Grading**	Equipment: **CAT 313L**
Date Started: **10/3/18**	Date Completed: **10/3/18**	Total Depth: **11.5 feet**	Groundwater: **N/A**

Depth (ft)	Soil/Rock Descriptions
0 – 6	**Sandy fat CLAY (CH)** – Dark gray, very stiff to hard (PP = 3.5 to >4.5 tsf), dry to slightly moist, some partings, some carbonate streamers and blebs. [COLLUVIUM/TOPSOIL]
6 – 8.5	**Sandy CLAY with gravels (CL)** – Yellowish brown, hard (PP = >4.5 tsf), dry to slightly moist, medium to fine grained sand, well rounded to rounded gravels up to approximately 3 inches, gravels highly to completely weathered. [LIVERMORE GRAVELS]
8.5 – 11.5	**Silty CLAY with gravels (CL)** – Yellowish brown to pale olive or olive brown, hard (PP= >4.5 tsf), rounded to well rounded gravels up to approximately 1 inch, gravels highly to completely weathered (some gravels weathered to white clay), carbonate blebs and streamers common, partings well developed, some rock structure. [LIVERMORE GRAVELS]

*Bottom of test pit at 11.5 feet. No groundwater encountered. No Caving.

Test Pit Log

Test Pit Number

1-TP4

Project Name: **Las Colinas**			Lat.: **37.70643**
Project Location: **Livermore, California**			Long.: **-121.75849**
Project No.: **15426.000.000**	Logged By: **Eric M Kiefer**	Contractor: **Shryock Grading**	Equipment: **CAT 313L**
Date Started: **10/3/18**	Date Completed: **10/3/18**	Total Depth: **Approx. 13 ft**	Groundwater: **N/A**

Depth (ft)	Soil/Rock Descriptions
0 – 3	**Sandy fat CLAY (CH)** – Dark gray, Hard (PP = >4.5 tsf), dry to slightly moist, some partings developed, no visible carbonate streamers. [COLLUVIUM/TOPSOIL]
3 – 8	**Gravelly SAND with clay (SP)** – Yellowish red to dark yellowish brown, dense to very dense (PP = 2.5 to >4.5 tsf), moist, sand medium to coarse grained, rounded to well rounded gravels up to approximately 4 inches, well developed imbrication in gravels. [LIVERMORE GRAVELS]
8 – ~13	**Sandy lean CLAY (CL)** – Olive brown to yellowish brown, hard (PP = >4.5 tsf), slightly moist to moist, partings well developed, some rock structure, carbonate streamers and blebs common. [LIVERMORE GRAVELS] **Lab Results** (Sampled at approximately 11-12 feet) • Particle Size Distribution – Passing #200 Sieve = 79.9% • Atterberg Limits – PL = 21; LL = 41; PI = 20

*Bottom of test pit at approximately 13 feet. No groundwater encountered. No Caving.

ENGEO
Expect Excellence

Test Pit Log

Test Pit Number
1-TP5

Project Name: **Las Colinas**			Lat.: **37.70539**
Project Location: **Livermore, California**			Long.: **-121.75784**
Project No.: **15426.000.000**	Logged By: **Eric M Kiefer**	Contractor: **Shryock Grading**	Equipment: **CAT 313L**
Date Started: **10/3/18**	Date Completed: **10/3/18**	Total Depth: **Approx. 14 ft**	Groundwater: **N/A**

Depth (ft)	**Soil/Rock Descriptions**
0 – 2	**Silty SAND (SM)** – Light gray, very dense (PP = >4.5 tsf), dry (well drained), fine to very fine grained sand, some carbonate streamers and partings. [ALLUVIUM/OVERBANK DEPOSIT]
2 – 3	**Sandy fat CLAY (CH)** – Olive brown to dark yellowish brown, hard (PP = >4.5 tsf), moist, some partings. [COLLUVIUM/TOPSOIL]
3 – 8	**Sandy lean CLAY with some gravels (CL)** –Dark yellowish brown to yellowish brown, dense to very dense (PP = 4.0 to >4.5 tsf), moist, well graded, some well rounded gravels up to ½ inch, sand fine to coarse grained. [LIVERMORE GRAVELS]
8 – ~13	**Clayey SAND with gravels (SC)** – Yellowish red to dark yellowish brown, dense to very dense (PP = 4.0 to >4.5 tsf), moist, well graded, some well rounded gravels up to ½ inch, sand fine to coarse grained. [LIVERMORE GRAVELS]
~13 – ~14	**Gravelly SAND (SP)** – Dark reddish brown to yellowish brown, dense to very dense, moist, well rounded gravels up to 6+ inches. [LIVERMORE GRAVELS]

*Bottom of test pit at approximately 14 feet. No groundwater encountered. No Caving.

ENGEO Expect Excellence	Test Pit Log		Test Pit Number 1-TP6
Project Name: **Las Colinas**			Lat.: **37.70450**
Project Location: **Livermore, California**			Long.: **-121.76003**
Project No.: **15426.000.000**	Logged By: **Eric M Kiefer**	Contractor: **Shryock Grading**	Equipment: **CAT 313L**
Date Started: **10/3/18**	Date Completed: **10/3/18**	Total Depth: **Approx. 12.5 ft**	Groundwater: **N/A**

Depth (ft)	Soil/Rock Descriptions
0 – 4	**Sandy fat CLAY (CH)** – Dark gray, hard (PP = >4.5 tsf), dry to slightly moist, some partings, some carbonate streamers. [COLLUVIUM/TOPSOIL]
4 – ~12.5	**Sandy CLAY with some gravels (CL)** – Dark yellowish brown to dark brown, hard (PP = >4.5 tsf), moist, some partings, some carbonate streamers, some rock structure. Sandy gravel lens at approximately 5.5 feet, rounded to well rounded gravels up to 1.5 inches. Carbonate streamers and blebs more common starting at approximately 6 feet. Moisture increasing at approximately 12.5 feet. [LIVERMORE GRAVELS]
*Bottom of test pit at approximately 12.5 feet. No groundwater encountered. No Caving.	

ENGEO Expect Excellence	Test Pit Log		Test Pit Number 1-TP7
Project Name: **Las Colinas**			Lat.: **37.70547**
Project Location: **Livermore, California**			Long.: **-121.76330**
Project No.: **15426.000.000**	Logged By: **Eric M Kiefer**	Contractor: **Shryock Grading**	Equipment: **CAT 313L**
Date Started: **10/3/18**	Date Completed: **10/3/18**	Total Depth: **Approx. 14 ft**	Groundwater: **N/A**

Depth (ft)	Soil/Rock Descriptions
0 – 4	**Sandy fat CLAY (CH)** – Dark gray, hard (PP = >4.5 tsf), dry to slightly moist, some partings. [COLLUVIUM/TOPSOIL]
4 – ~14	**Clayey SAND/Sandy lean CLAY (SC/CL)** – Light yellowish brown to pale olive, hard (PP = >4.5 tsf), slightly moist to moist, partings well developed, some rock structure, massive, carbonate blebs and streamers common (less common below approximately 5.5 feet), trace gravels up to approximately 0.5 inches. Got harder around 12 feet – continued to get harder with depth. [LIVERMORE GRAVELS]
*Bottom of test pit at approximately 14 feet. No groundwater encountered. No Caving.	

ENGEO Expect Excellence	# Test Pit Log	Test Pit Number **1-TP8**	
Project Name: **Las Colinas**			Lat.: **37.70410**
Project Location: **Livermore, California**			Long.: **-121.76206**
Project No.: **15426.000.000**	Logged By: **Eric M Kiefer**	Contractor: **Shryock Grading**	Equipment: **CAT 313L**
Date Started: **10/3/18**	Date Completed: **10/3/18**	Total Depth: **Approx. 14 ft**	Groundwater: **N/A**

Depth (ft)	Soil/Rock Descriptions
0 – 3	**Sandy fat CLAY (CH)** – Dark gray, hard (PP = >4.5 tsf), dry to slightly moist, some partings, trace carbonate blebs and streamers. [COLLUVIUM/TOPSOIL]
3 – 6	**Sandy lean CLAY (CL)** – Pale olive to dark yellowish brown, hard (PP = >4.5 tsf), slightly moist to moist, fine grained sand, carbonate blebs and streamers common from approximately 3-4 feet. [LIVERMORE GRAVELS]
6 – 6.25	**Sandy GRAVEL (GP)** – Dark gray mottled with dark yellowish brown, very dense (PP = >4.5 tsf), moist, well rounded gravels up to 0.5 inches, gravels imbricated. [LIVERMORE GRAVELS]
6.25 – ~14	**Sandy lean CLAY (CL)** – Dark yellowish brown to olive brown, hard (PP = >4.5 tsf), moist, trace gravels – well rounded, carbonate streamers and blebs, well developed partings and some rock structure – becomes more well developed below approximately 9 feet. [LIVERMORE GRAVELS]

*Bottom of test pit at approximately 14 feet. No groundwater encountered. No Caving.

APPENDIX D

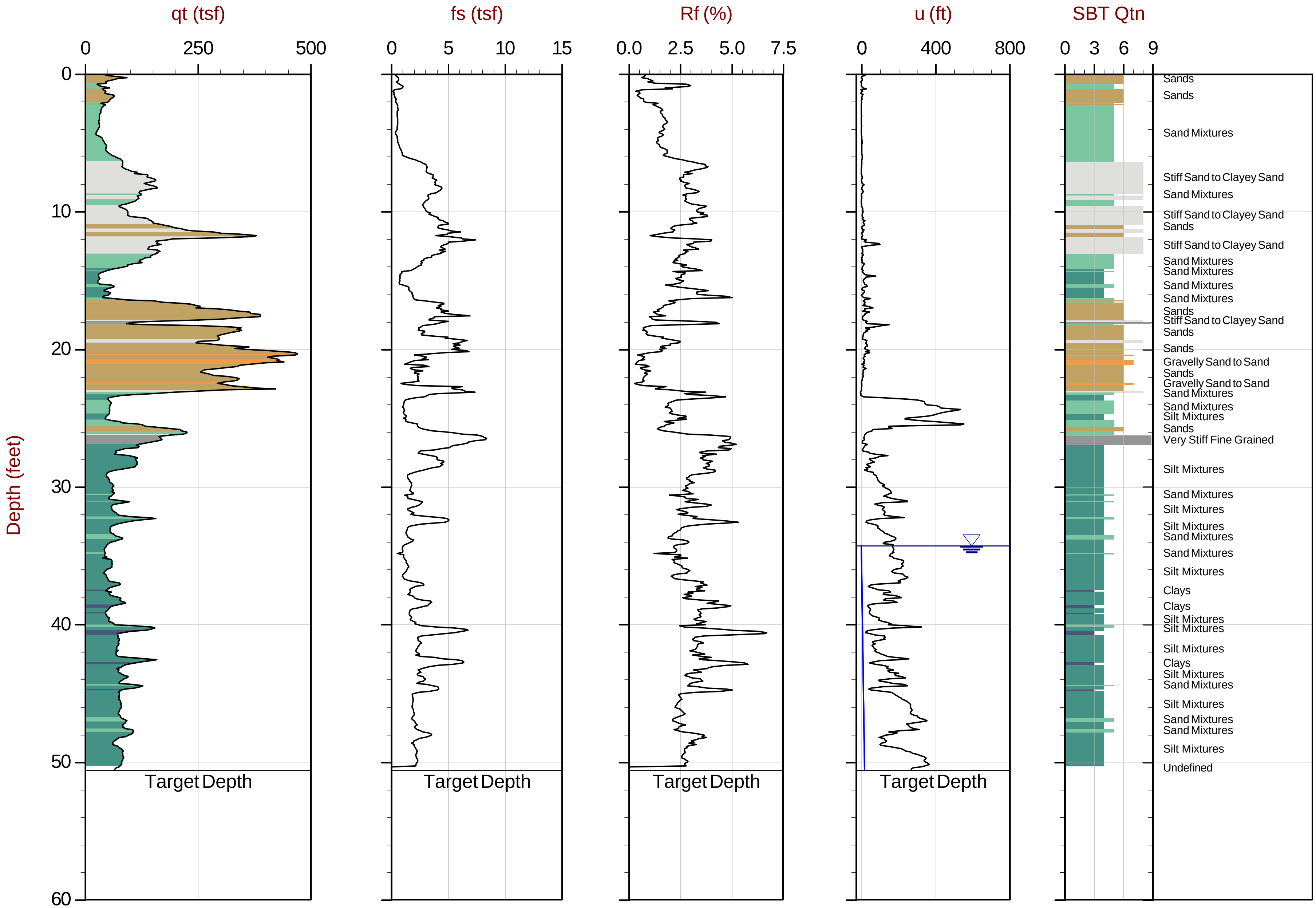
CONE PENETRATION TEST LOGS

CONETEC

ENGEO

Job No: 18-56165
Date: 2018-10-03 09:39
Site: Las Colinas

Sounding: 1-CPT1
Cone: 383:T1500F15U500



Max Depth: 15.425 m / 50.61 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 18-56165_1CP1.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 10N N: 4173832m E: 609296m
Sheet No: 1 of 1

Equilibrium Pore Pressure (Ueq) Assumed Ueq Dissipation, Ueq achieved Dissipation, Ueq not achieved Hydrostatic Line

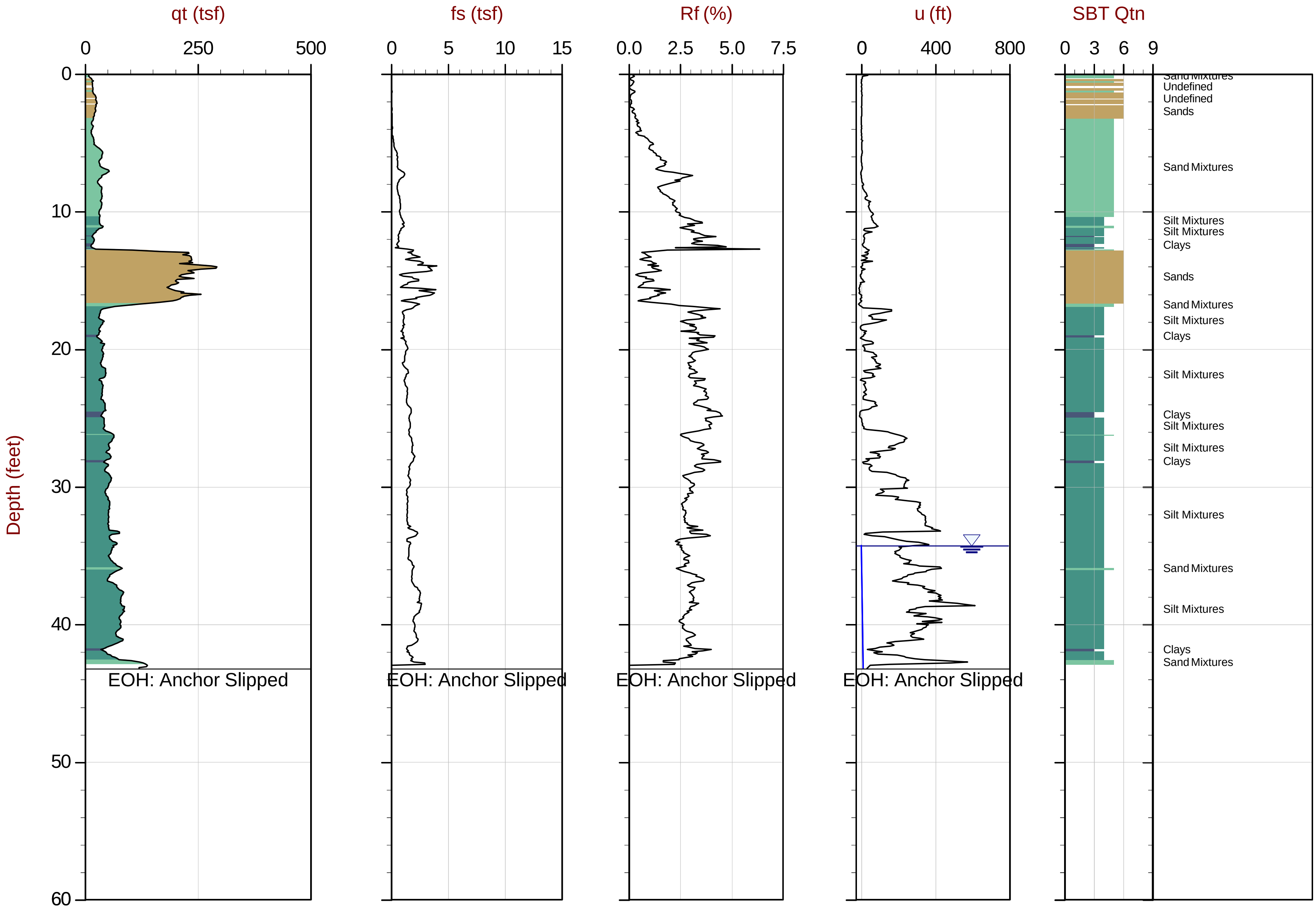
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

ENGEO

Job No: 18-56165
Date: 2018-10-03 14:27
Site: Las Colinas

Sounding: 1-CPT2
Cone: 383:T1500F15U500



Max Depth: 13.175 m / 43.22 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 18-56165_1CP2.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 10N N: 4173772m E: 609526m
Sheet No: 1 of 1

Equilibrium Pore Pressure (Ueq) | Assumed Ueq | Dissipation, Ueq achieved | Dissipation, Ueq not achieved | Hydrostatic Line

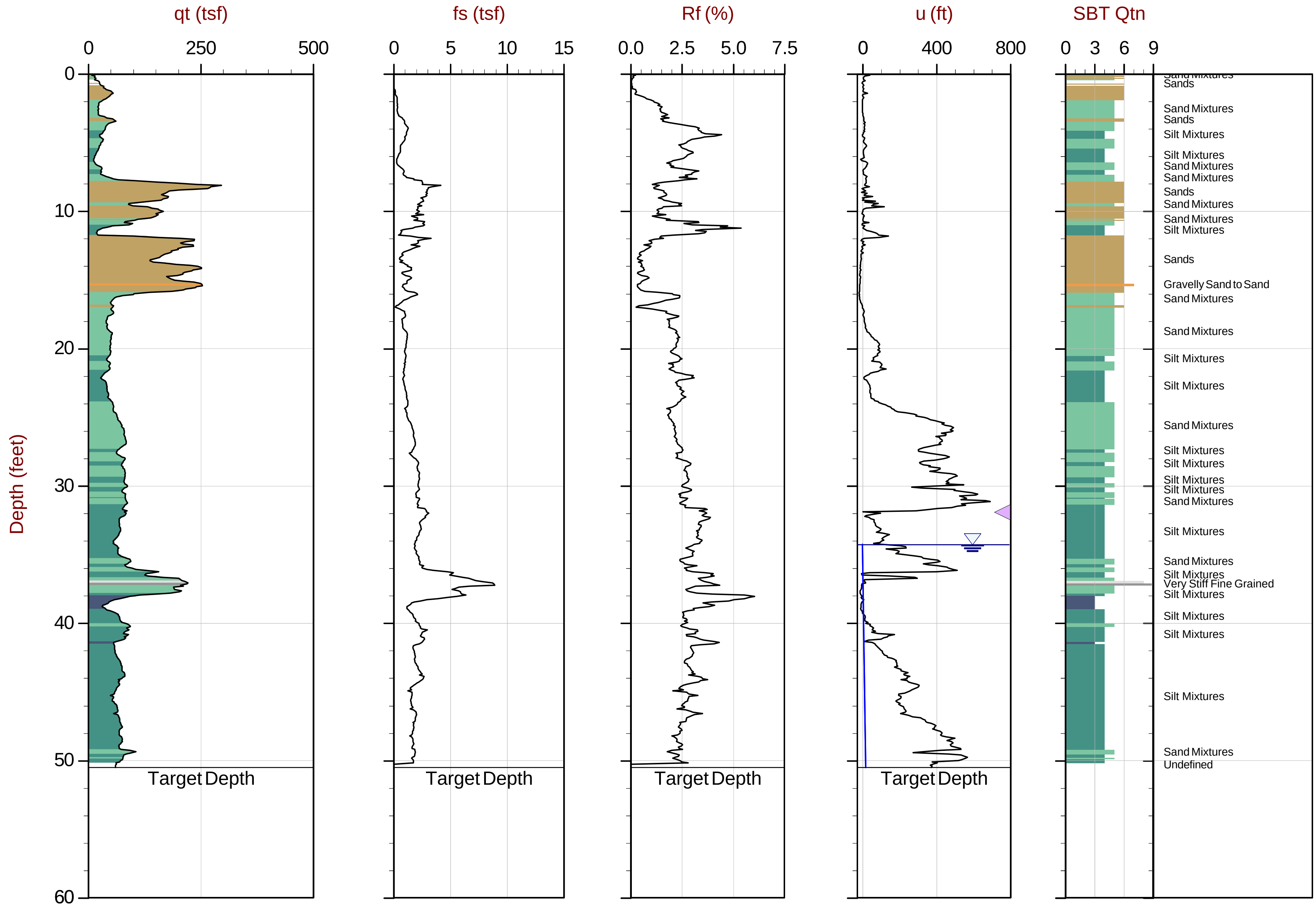
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

ENGEO

Job No: 18-56165
Date: 2018-10-03 10:42
Site: Las Colinas

Sounding: 1-CPT3
Cone: 383:T1500F15U500



Max Depth: 15.400 m / 50.52 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 18-56165_1CP3.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 10N N: 4173665m E: 609327m
Sheet No: 1 of 1

Equilibrium Pore Pressure (Ueq) Assumed Ueq Dissipation, Ueq achieved Dissipation, Ueq not achieved Hydrostatic Line

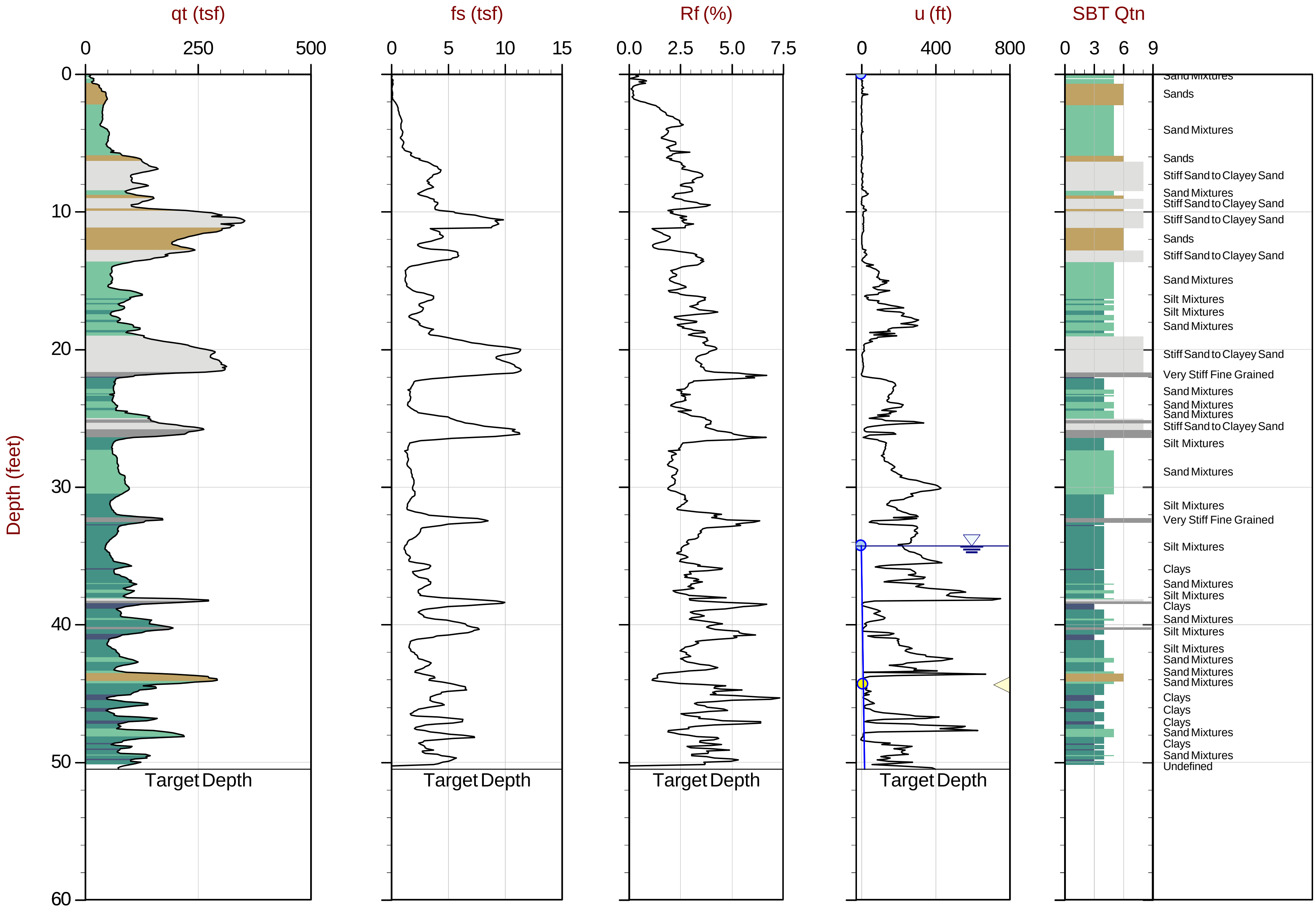
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

ENGEO

Job No: 18-56165
Date: 2018-10-03 11:42
Site: Las Colinas

Sounding: 1-CPT4
Cone: 383:T1500F15U500



Max Depth: 15.400 m / 50.52 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 18-56165_1CP4.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 10N N: 4173726m E: 609001m
Sheet No: 1 of 1

Equilibrium Pore Pressure (Ueq) | Assumed Ueq | Dissipation, Ueq achieved | Dissipation, Ueq not achieved | Hydrostatic Line

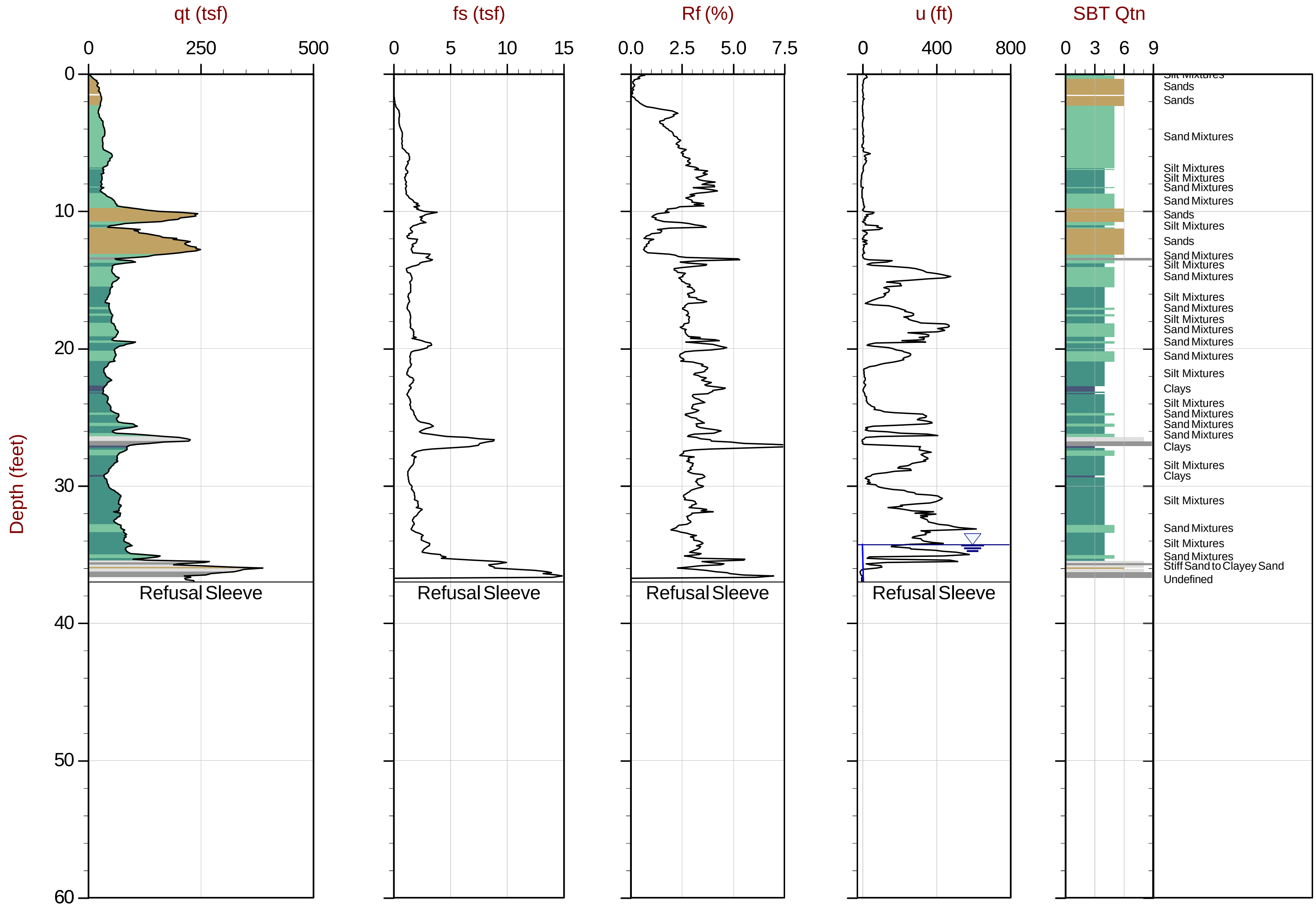
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

ENGEO

Job No: 18-56165
Date: 2018-10-03 15:31
Site: Las Colinas

Sounding: 1-SCPT5
Cone: 383:T1500F15U500



Max Depth: 11.275 m / 36.99 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 18-56165_1SCP5.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 10N N: 4173695m E: 609406m
Sheet No: 1 of 1

Equilibrium Pore Pressure (Ueq) Assumed Ueq Dissipation, Ueq achieved Dissipation, Ueq not achieved Hydrostatic Line

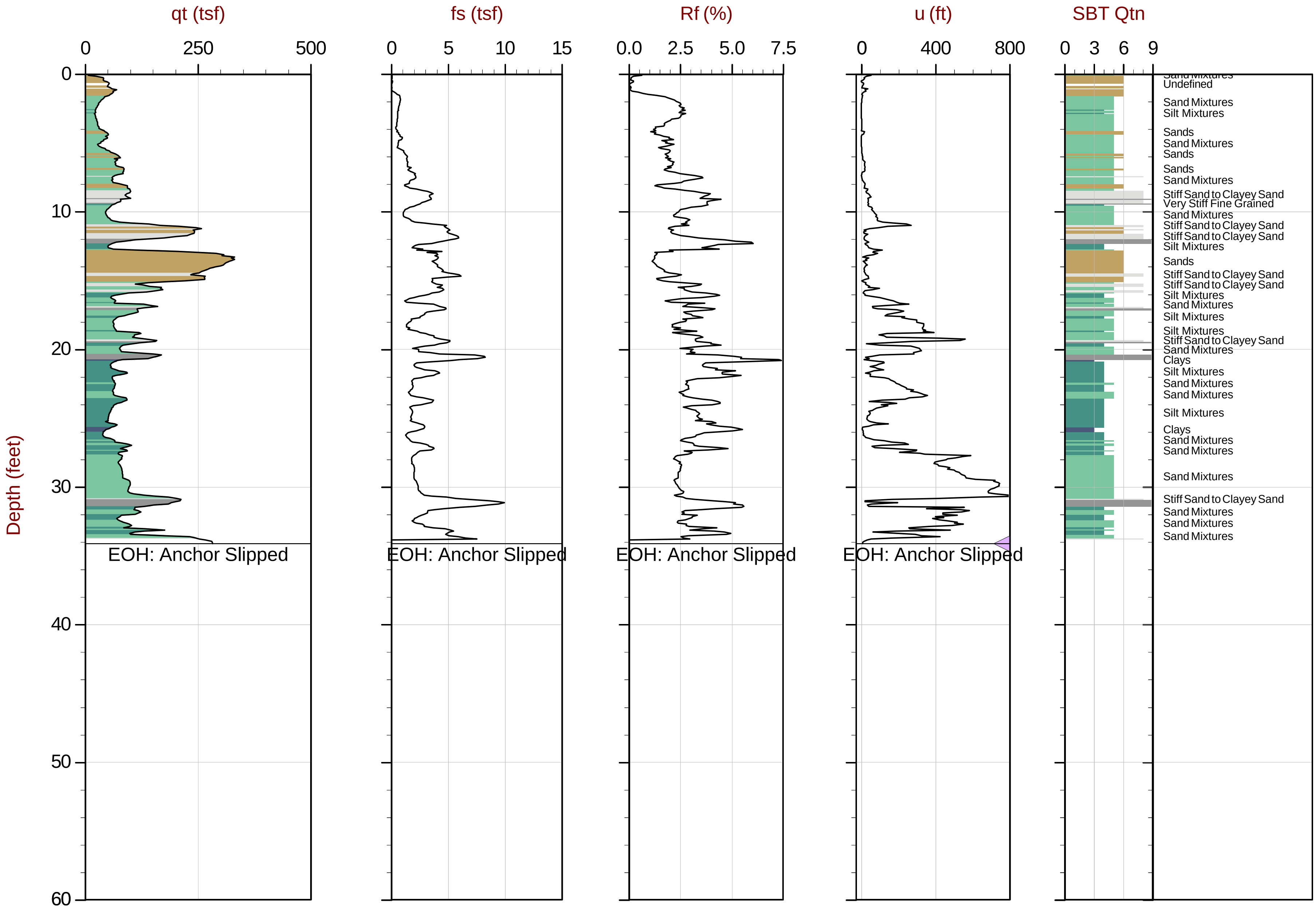
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

ENGEO

Job No: 18-56165
Date: 2018-10-03 12:58
Site: Las Colinas

Sounding: 1-CPT6
Cone: 383:T1500F15U500



Max Depth: 10.400 m / 34.12 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 18-56165_1CP6.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 10N N: 4173838m E: 609072m
Sheet No: 1 of 1

Equilibrium Pore Pressure (Ueq) Assumed Ueq Dissipation, Ueq achieved Dissipation, Ueq not achieved Hydrostatic Line

The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

ENGEO
Expect Excellence