Appendix E:
Geology and Soils Supporting Information

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E.1 - Existing Site Conditions

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Project No.
9181.100.000

September 2, 2011

Mr. David R. Baker
O'Brien Land Company, LLC
3031 Stanford Ranch Road, Suite 2-310
Rocklin, CA 95765

Subject: AMD Trust Site
Deer Hill Road
Lafayette, California

EXISTING SITE CONDITIONS

Dear Mr. Baker

We have prepared this letter to comment on the existing soil and topographic conditions at the AMD trust site. The proposed development will include some grading and re-configuration of site topography but will to a large extent utilize existing artificially-created landforms that were created by past site uses as described below.

SITE USE HISTORY

Review of aerial photographs from 1928 to 2005 shows that the site was undeveloped until the existing residence was constructed in 1941. The garage and one of the two small offices were constructed sometime between 1946 and 1958. The other small office appears to have been constructed sometime between 1965 and 1974. Contra Costa County documents indicate that Independent Construction Company was issued a quarry permit for the site, which was active from 1967 to 1970. This quarry use pre-dates the Surface Mine Reclamation Act (SMARA), which would have required reclamation and stabilization of quarry slopes and re-vegetation of the site. Aerial photographs from1968 and 1969 show that grading for the construction of Deer Hill Road and Highway 24 was in progress at that time, and that excavations were in progress across most of the AMD Trust property. A comparison of USGS topography to existing topography shows that cuts of as much as 60 to 80 feet were made on the site as part of quarry operations. We understand that the excavated material was used as fill in the adjacent road and highway construction. Based on review of aerial photos, some form of quarry operation or minor grading activity occurred at the site through the early 1990s. The site was used as a container storage site from the late 1990s almost to the present time. Figure 1 depicts areas of past disturbance at the site related to both quarry activity, road construction and other site uses.

EXISTING CONDITIONS

Approximately 85 percent of the area of the AMD Trust property has been disturbed by past site use, as depicted on Figure 1. A comparison of USGS topography to existing topography shows that cuts of as much as 60 to 80 feet were made on the site as part of quarry operations. Areas adjacent to Highway 24 and Deer hill road were filled to create road embankments. The current topography is a series of artificial terraces and graded slopes upon which natural soils and native vegetation are absent. The exposed soils in graded areas consist of nutrient-poor bedrock or rock-derived gravelly soil. Many existing slopes are eroding and locally unstable.

PROPOSED PROJECT

The proposed project will largely occupy the existing artificially-created site landforms but will include geotechnical measures to stabilize slopes and reduce erosion. Over-steepened cut slopes will be graded to flatter inclinations, loose, eroding and unstable soils will be removed and replaced with stabilized engineered fills, and surface drainage will be improved and controlled by the storm drain collection system. In addition, the project will include water quality treatment facilities that will reduce sediment discharge from the site. Currently exposed bare soil areas on slopes and existing terraces will be vegetated by proposed landscaping. In general, the proposed development will improve stability, reduce erosion and improve the quality of existing runoff water.

CONCLUSIONS

Approximately 85 percent of the area of AMD Trust property is currently in a disturbed and non-native condition due to past site use as a pre-SMARA quarry which was never reclaimed in accordance with more recent State requirements. The proposed development will improve slope stability, reduce erosion control site runoff and improve the water quality of site runoff.

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

Sincerely,

ENGEO Incorporated

PROFESSIONAL GEOLOGIST
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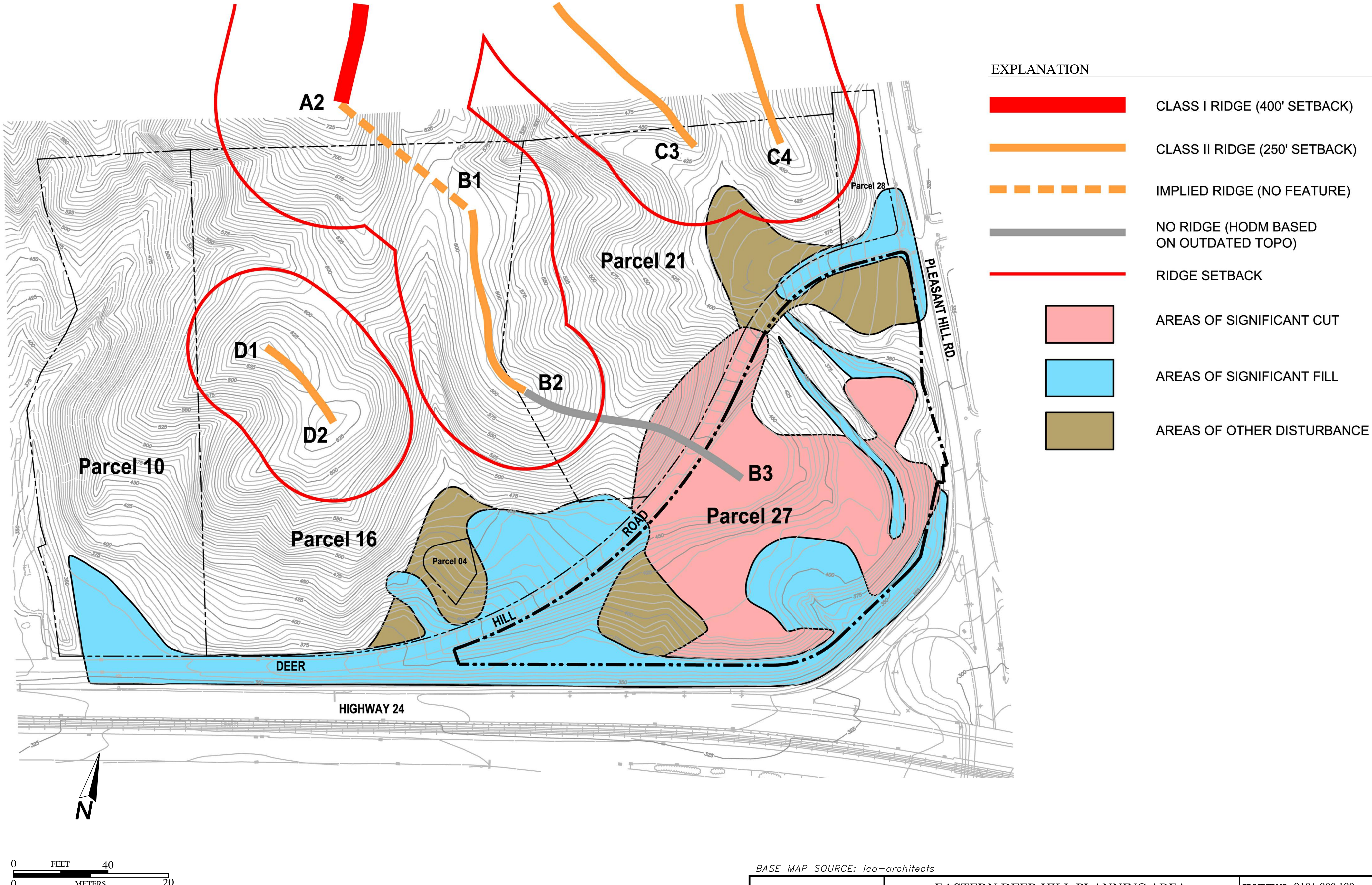
Principal

Attachment: Figure 1

Cc: Norm Dyer, LCA



BASE MAP SOURCE: lca-architects

ENGEO
Expect Excellence

EASTERN DEER HILL PLANNING AREA
AREAS OF GRADING DISTURBANCE
AMD TRUST PROPERTY
LAFAYETTE, CALIFORNIA

PROJECT NO.: 9181.000.100		FIGURE NO.
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ORIGINAL FIGURE PRINTED IN COLOR

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E.2 - Updated Geotechnical Report

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UPDATED GEOTECHNICAL REPORT

THE HOMES AT DEER HILL
LAFAYETTE, CALIFORNIA

Submitted to:
Mr. David R. Baker
O'Brien Land Company, LLC
3031 Stanford Ranch Road, Suite 2-310
Rocklin, California

Prepared by:
ENGEO Incorporated

April 3, 2014

Project No.
9181.200.000

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Project No.
9181.200.000

April 3, 2014

Mr. David R. Baker
O'Brien Land Company, LLC
3031 Stanford Ranch Road, Suite 2-310
Rocklin, CA 95765

Subject: The Homes at Deer Hill (Tract 9369)
Deer Hill Road
Lafayette, California

UPDATED GEOTECHNICAL REPORT

Dear Mr. Baker:

As requested, we completed this updated geotechnical report for the proposed Homes at Deer Hill project (formerly the Terraces of Lafayette) in Lafayette, California. The accompanying report presents our field exploration and laboratory testing with our conclusions and recommendations regarding residential development at the site.

Our findings indicate that the study area is suitable for the proposed development provided the recommendations and guidelines provided in this report are implemented during project planning and construction. We are pleased to have been of service to you on this project and are prepared to consult further with you and your design team as the project progresses.

Sincerely,

ENGEO Incorporated

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1.0 INTRODUCTION

1.1 PURPOSE AND SCOPE

The purpose of this geotechnical report is to provide updated conclusions and recommendations based on a reevaluation of the geotechnical considerations due to changes to the project plans. ENGEO prepared a previous geotechnical report in March 2011, which provided recommendations for a proposed multi-family residential development at the site. Since the time the previous report was prepared, the plans have changed to a single-family residential development. This report also considers updates to seismic design criteria included in the 2013 California Building Code. As part of our scope, we performed the following services.

- Review of available literature, previous reports, and geologic maps for the study area.
- Subsurface exploration consisting of three additional test pits.
- Laboratory testing of materials sampled during the field exploration.
- Engineering analyses.
- Report preparation summarizing our conclusions and recommendations for the proposed development.

We prepared this report exclusively for O'Brien Land Company, LLC and their design team consultants. ENGEO should review any changes made in the character, design or layout of the development to modify the conclusions and recommendations contained in this report, as necessary. This document may not be reproduced in whole or in part by any means whatsoever, nor may it be quoted or excerpted without the express written consent of ENGEO.

1.2 SITE LOCATION AND DESCRIPTION

The project site is located southeast of Deer Hill Road and northwest of the intersection of Pleasant Hill Road and Highway 24 in Lafayette, California (Figure 1). According to the Tract 9369 Vesting Tentative Map prepared by BKF (March 6, 2014), the project site encompasses roughly 22 acres. Cuts and fills related to grading for Deer Hill Road, Highway 24 and a quarry operation have altered the original topography of the site. Several existing structures, including a residence and maintenance buildings, are present in the eastern portion of the site. An existing paved driveway off Deer Hill Road provides access to the residence and existing buildings, and an unimproved dirt road provides access to the portions of the site that were quarried in the past.

The current topography of the project site can generally be characterized as four relatively flat-lying areas (terraces) separated by slopes that vary from inclinations of 1.5:1 to 4:1 (horizontal:vertical). The majority of the site is grass covered with trees flanking the paved driveway, existing residence and drainage at the eastern portion of the site. Current elevations range from a high of about 463 feet above mean sea level (msl) on the northernmost terrace adjacent to Deer Hill Road to a low of about 330 feet above msl at the drainage near Pleasant Hill Road at the eastern edge of the site. The Mokelumne aqueduct parallels the southeastern and southern project site boundary.

1.3 PROPOSED DEVELOPMENT

The Vesting Tentative Map prepared by BKF, dated March 6, 2014, shows the development of 44 single-family residences, a soccer field, appurtenant streets, utilities, parking and common areas. We understand that the existing residence and maintenance buildings will be demolished as part of the development. Based on the grading plan, grading will consist of cuts up to approximately 40 feet deep and fills up to approximately 40 feet thick, with graded slopes up to 60 feet high at inclinations of approximately 2:1 (horizontal:vertical) or flatter. Current plans also indicate proposed terraced retaining walls along the 2:1 slope at the southwestern corner of the project. We anticipate one- to two-story, above-grade structures of wood-frame construction for the residential buildings. Therefore, the building loads are expected to be relatively light.

1.4 HISTORY OF SITE

We reviewed stereo-paired aerial photographs of the site from various years between 1928 and 2005. Review of the photos indicate the site was relatively undeveloped until sometime between 1954 and 1957 when a residence and several small structures were constructed in the northeastern portion of the site. Historic documents indicate that Contra Costa County issued a quarry permit for the site to Independent Construction Company around 1967; this was around the same time as the grading for Deer Hill Road and Highway 24, which is evident in both 1968 and 1969 aerial photos of the site. Based on review of aerial photos, some form of quarry operation or minor grading activity occurred at the site through the early 1990s. The site was used as a container storage site from the late 1990s almost to the present time.

1.5 PREVIOUS GEOTECHNICAL AND GEOLOGICAL STUDY

1.5.1 Preliminary Geotechnical Feasibility Report, ENGEO, March 2011

In March 2011, ENGEO performed a preliminary geotechnical feasibility investigation for a proposed multi-family residential development at the site. This previous study included a review of geologic literature and maps, a geologic reconnaissance of the site, examination of aerial photographs, collection of four surface samples for evaluation of index soil properties, and preparation of a report. No subsurface exploration was undertaken for the preparation of the preliminary report. The laboratory analyses from this study are presented in Appendix B. The study concluded that proposed residential development of the property was feasible provided the project was appropriately designed to address the geologic and geotechnical hazards identified in the report.

1.5.2 Geotechnical Exploration, ENGEO, Revised September 2, 2011 (August 18, 2011)

In the summer of 2011, ENGEO performed a geotechnical exploration at the site. At the time of this exploration, the proposed project consisted of a multi-family residential development. Our previous exploration included excavation and logging of 30 test pits and drilling and logging of 6 exploratory borings to a maximum depth of approximately 51½ feet below existing grade. A description of the subsurface conditions encountered during this previous exploration is included in Section 2 of this report and the report logs are included in Appendix A. The approximate locations of the previous borings and test pits are included on Figure 3 of this report. Select samples collected during this previous exploration were tested in our laboratory for various soil characteristics. The laboratory results are included in Appendix B of this report. The previous exploration concluded that the study area appears to be suitable for residential development provided that the project is appropriately designed for the geologic and geotechnical hazards identified in the report.

2.0 GEOLOGY AND SEISMICITY

2.1 GEOLOGIC SETTING

The site is located within the Coast Ranges physiographic province of California. The Coast Ranges physiographic province is typified by a system of northwest-trending, fault-bounded mountain ranges and intervening alluviated valleys. Reliez Valley is located east of the site. The valley floor is covered with alluvium derived largely from the surrounding hills, including those onsite.

Bedrock in the Coast Ranges consists of igneous, metamorphic and sedimentary rocks that range in age from Jurassic to Pleistocene. The present physiography and geology of the Coast Ranges are the result of deformation and deposition along the tectonic boundary between the North American plate and the Pacific plate. Plate boundary fault movements are largely concentrated along the well-known fault zones, which in the area include the San Andreas, Hayward, and Calaveras faults, as well as other lesser-order faults.

2.1.1 Site Geology

According to published maps covering the site by Dibblee (2005) and Graymer (1994), the project site is underlain by late to middle Miocene marine sedimentary rock primarily consisting of sandstone (Figure 2). Based on mapping by Dibblee, the site is underlain by marine sandstone and clay shale/siltstone of the Monterey Formation. According to Graymer, bedrock underlying the majority of the site comprises the Briones Formation (Tbr – Miocene) with Neroly Formation (Tn) underlying the westernmost corner of the project site. At the property, the bedding within the bedrock units generally strikes northwest–southeast and dips moderately towards the southwest. Exposures of this bedrock unit were generally observed to be weak to moderately strong, closely fractured and moderately weathered.

2.1.2 Geologic Mapping

During our exploration, an ENGEO geologist performed geologic mapping at the site. Figure 3 shows the areal extent of the geologic units mapped. We provide a description of the subsurface conditions encountered during our exploration within these geologic units in Section 3 of this report.

2.2 FAULTING AND SEISMICITY

Because of the presence of nearby active faults[1], the Bay Area Region is considered seismically active. Numerous small earthquakes occur every year in the region, and large (>M7) earthquakes have been recorded and can be expected to occur in the future. The site is not located within a State of California Earthquake Fault Zone. Figure 4 shows the approximate location of active and potentially active faults and significant historic earthquakes mapped within the San Francisco Bay Region. Based on the USGS Quaternary Fault and Fold Database (QFFD), the nearest active fault is the Northern Calaveras fault located approximately 4.5 miles south of the site. Other active faults located near the site include the Concord-Green Valley fault, located approximately 5 miles to the east of the site, and the Hayward fault, located approximately 8 miles to the west.

Based on an evaluation of the termination of the northern Calaveras fault by Unruh and Kelson (2002), the Lafayette fault, which is located approximately 200 feet west of the project site, is considered to be a potentially active right-lateral strike-slip fault that is interpreted as one of a series of structures that may accommodate slip on the northern Calaveras fault. According to the State of California, a fault is considered to be "active" if it has had identifiable movement within the last 11,000 years; the time period for a "potentially active fault" is 2 million years.

The Uniform California Earthquake Rupture Forecast (UCERF, 2008) evaluated the 30-year probability of a M6.7 or greater earthquake occurring on the known active fault systems in the Bay Area, including the Calaveras fault. The UCERF generated an overall probability of 63 percent for the Bay Area as whole, and a probability of 7 percent for the Calaveras fault, 3 percent for the Concord-Green Valley fault, and 31 percent for the Hayward fault.

3.0 FIELD EXPLORATION

The sections below summarize both our recent and previous (2011) field exploration activities and laboratory testing; as well as ground surface, subsurface, and groundwater conditions.

3.1 FIELD LOGGING

The field exploration for this study was conducted on March 4, 2014, and consisted of excavating 3 additional test pits to a maximum depth of 26 feet below existing grade. Previous exploration of the site was conducted on June 1 and 2, and June 14 and 15, 2011, and consisted of excavating 30 test pits to a maximum depth of 19 feet below existing grade and drilling 6 exploratory borings to a maximum depth of approximately 51½ feet below existing grade. The approximate locations of test pits and borings are shown on Figure 3. The test pits were performed using a track-mounted excavator and the borings were performed using a truck-mounted B-58 drill rig equipped with 4-inch-diameter solid flight augers. Exploration locations were established by handheld GPS and visual sighting from existing features and should be considered accurately located only to the degree implied by the methods used.

[1] An active fault is defined by the California Geological Survey as one that has had surface displacement within Holocene time (about the last 11,000 years) (Hart, 1997).

The test pits and borings were logged in the field by an ENGEO geologist. Bulk soil samples were collected from the test pits for laboratory testing. In addition, 2½-inch-diameter stainless steel liners were used to collect soil samples within Test Pit 2TP-1 for laboratory testing. Soil samples were collected from the borings using either a 2½-inch inside diameter (I.D.) California-type split-spoon sampler fitted with 6-inch-long stainless steel and brass liners or a 2-inch outside diameter (O.D.) Standard Penetration Test split-spoon sampler. The penetration of the samplers into the native materials was recorded as the number of blows needed to drive the sampler 18 inches in 6-inch increments. The boring logs record blow count results as the actual number of blows required for the last 1 foot of penetration; no conversion factors have been applied. The samplers were driven with a 140-pound hammer falling a distance of 30 inches employing an automatic trip system. The field logs were then used to develop the report boring logs, which are presented in Appendix A.

The boring and test pit logs depict subsurface conditions at the time the exploration was conducted. Subsurface conditions at other locations may differ from conditions occurring at these locations, and the passage of time may result in altered subsurface conditions. In addition, stratification lines represent the approximate boundaries between soil types, and the transitions may be gradual.

3.2 SUBSURFACE CONDITIONS

As discussed in Section 2 of this report, we performed mapping of the geologic units at the site, which are shown on Figure 3. We provide a description of the subsurface conditions encountered during our exploration within these geologic units below. The boring and test pit logs included in Appendix A can be referenced for more specific subsurface conditions encountered during our exploration.

3.2.1 Existing Fill (Qaf)

Existing undocumented fill (Qaf) is present in the two former swales at the southern portion of the site (Figure 3). The fill in southernmost portions of the two swales appears to have been placed during grading for Highway 24 in the late 1960s. In general, the existing fill consisted of moist, very stiff to hard, silty clay and sandy clay with angular gravel-sized sandstone fragments, and few cobble-sized sandstone fragments. Fill in these areas displayed horizontal layering indicative of fill placement in lifts. Fill thickness in the swales is approximately15 feet.

Undocumented fill is also present in the southwestern portion of the site in an existing 2:1 fill slope associated with the grading for Deer Hill Road in the late 1960s (Figure 3). In general, the fill is bedrock derived and consists of dense, silty gravel and sandy gravel. Fill in this area also displayed horizontal layering indicative of fill placement in lifts.

In the northeastern portion of the site, minor amounts of fill associated with the access roads to the existing residence and the mid-level terrace are present. This fill generally comprises 3 to 5 feet of very stiff, moist silty clay with gravel-sized sandstone fragments.

In addition to the existing fills described above, we observed that the mid-slope, level terrace is blanketed by a 6- to 12-inch layer of road grindings. These were likely placed at some point following the quarry operation at the site.

3.2.2 Landslide Debris (Qls)

Previous landslide mapping by Nilsen (1975) and Haydon (1996) shows roughly four landslides at the site. Based on our subsurface exploration and detailed field mapping, we identified one possible earthflow in the northeastern portion of the site (Figure 3). Previous grading and quarrying operations at the site have removed most of the landslides identified on the referenced geologic maps and upon exploration were determined to be deposits of colluvium (described below). The earthflow is approximately 15 feet in depth and comprises silty clay. The earthflow exhibited no signs of recent activity through cracking or displacement near the head scarp or additional sloughing of surficial soils.

3.2.3 Colluvium (Qc)

Where not stripped away by previous grading and quarrying activities, colluvial deposits are present below fills placed in the two swales located in the southern portion of the site (Figure 3). We have also mapped colluvium in two smaller swales located in the northeastern portion of the site (Figure 3). In general, the colluvium consists of moist, stiff to very stiff, lean clay with moderate compressibility and dense clayey sand. Two Plasticity Index (PI) tests were performed on this unit that resulted in a PI range of 19 to 23.

3.2.4 Pleistocene-age Alluvial Deposits (Qal)

Pleistocene-age alluvial deposits (Qal) are present in the relatively flat lying northeastern area of the site near the intersection of Deer Hill Road and Pleasant Hill Road (Figure 3). In general, the alluvium is fine-grained consisting of stiff to very stiff lean clay and sandy clay with moderate compressibility. Two PI tests were performed on this unit that resulted in a PI range of 30 to 41.

3.2.5 Miocene Briones Formation (Tbr)

According to published maps covering the site by Dibblee (2005) and Graymer (1994), the project site is underlain by late to middle Miocene marine sedimentary rock primarily consisting of sandstone. Based on mapping by Dibblee, the site is underlain by marine sandstone, clay shale/siltstone of the Monterey Formation. According to Graymer, bedrock underlying the majority of the site comprises the Briones Formation (Tbr – Miocene) with Neroly Formation (Tn) underlying the westernmost corner of the project site.

Based on our mapping, bedrock at the site consists primarily of Miocene Briones Formation sandstone with some siltstone interbeds. Bedding within the bedrock units generally strikes west–northwest to east-northeast and dips 30 to 60 degrees towards the south. A solid-flight auger boring (B-3) was advanced to near refusal at a depth of 20.5 feet within the sandstone unit in an area of previous and proposed cut on the uppermost terrace adjacent to Deer Hill Road.

This sandstone can be described as weak to medium strong, closely fractured, and moderately weathered.

3.3 LABORATORY TESTING

Select samples recovered during our subsurface exploration were tested to determine various soil characteristics as presented on the following table.

TABLE 3.3-1
Laboratory Testing

Soil Characteristic	Testing Method	Location of Results
Unconsolidated Undrained Triaxial Compression	ASTM D-3080	Appendix B
Natural Unit Weight and Moisture Content	ASTM D-2216	Appendix A
Plasticity Index	ASTM D-4318	Appendix B
Grain Size Distribution	ASTM D-422	Appendix B
Compaction Curve	ASTM D-1557	Appendix B
Sulfate Testing in Soils	Cal Trans 417	Appendix B
Direct Shear	ASTM D-3080	Appendix B

The laboratory test results are shown on the borelogs (Appendix A), with individual test results presented in Appendix B.

3.4 GROUNDWATER

Groundwater was encountered in the two northernmost borings (B-1 and B-2) at a depth of approximately 13 to 14 feet below existing grades. Perched groundwater was also encountered at depths of 4 and 9 feet in Test Pits TP-8 and 2TP-3, respectively. Fluctuations in groundwater levels occur seasonally and over a period of years because of variations in precipitation, temperature, irrigation, and other factors.

4.0 DISCUSSION AND CONCLUSIONS

Based on our findings and results of engineering analyses, it is our opinion that the site is feasible for construction of the proposed residential development from a geotechnical standpoint. We evaluated the site with respect to known geologic and other hazards common to the greater San Francisco Bay Region. The primary hazards and the risks associated with these hazards with respect to the planned development are discussed in the following sections of this report.

4.1 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, ground lurching, soil liquefaction, lateral spreading, and densification. Based on topographic and lithologic data, risk from earthquake-induced regional subsidence/uplift is considered negligible at the site. The following sections present a discussion of these hazards as they apply to the site.

4.1.1 Ground Rupture

As previously discussed, the site is not located within a State of California Earthquake Fault Zone. Based on our field mapping, review of aerial photographs and the results of our field exploration, it is our opinion that fault-related ground rupture is unlikely at the subject property.

4.1.2 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, all structures should be designed using sound engineering judgment and the 2013 California Building Code (CBC) requirements, as a minimum.

4.1.3 Ground Lurching

Ground lurching is a result of the rolling motion imparted to the ground surface during energy released by an earthquake. Such rolling motion can cause ground cracks to form in weaker soils. The potential for the formation of these cracks is considered greater at contacts between deep alluvium and bedrock. Such an occurrence is possible at the site as in other locations in the Bay Area, but based on the site location, it is our opinion that the offset is expected to be minor.

4.1.4 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. Empirical evidence indicates that loose to medium dense gravels, silty sands, low-plasticity silts, and some low-plasticity clays are also potentially liquefiable.

According to the USGS Liquefaction Susceptibility map for the central San Francisco Bay Region (2006), the northeastern portion of the site, just southwest of the intersection of Pleasant Hill Road and Deer Hill Road, is mapped as an area potentially susceptible to liquefaction. We evaluated the liquefaction potential of the subsurface soil by drilling two test borings (B-1 and B-2) in this area and collecting soil samples. Borings B-1 and B-2 encountered stiff to very stiff clay to the depth explored. The results of our laboratory testing on samples collected from our test borings indicate the clay has PIs ranging from 30 to 41. Based on our analysis, the potential for liquefaction at the site is low.

4.1.5 Lateral Spreading

Lateral spreading involves lateral ground movements caused by seismic shaking. These lateral ground movements are often associated with a weakening or failure of an embankment or soil mass overlying a layer of liquefied or weak soils. Due to the low potential for liquefaction at the site, the potential for lateral spreading at the site is considered low.

4.1.6 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 considered low.

As discussed previously in our report, we did identify one possible relatively shallow earthflow in the northeastern portion of the site at the approximate location shown on Figure 3. We summarize our evaluation of the potential for earthquake-induced movement of this landslide below.

4.2 SLOPE STABILITY

4.2.1 Methods of Analysis

We performed two-dimensional limit-equilibrium slope stability analyses of critical slopes with the computer slope stability software Slide Version 6.0 using Spencer's method (Spencer, 1967). We selected critical slopes for slope stability analyses (Cross Sections 1-1', 2-2', and 3-3'). Cross Section 1-1 is at the location of a proposed 2:1 (horizontal to vertical) slope with terraced retaining walls at the southwestern portion of the site. Figure 3 shows the locations of Cross Sections 1-1, 2-2, and 3-3 and the profiles of the Cross Sections are included on Figure 5. A conservative groundwater table was assumed at roughly 5 to 20 feet below existing grade depending on location. For pseudostatic stability analyses, we used ground motions corresponding to a seismic event with a probability of exceedance of 10 percent in 50 years based on the United States Geological Survey 2008 Seismic Hazard Map.

4.2.2 Estimation of Shear Strength

We performed a direct shear test on a remolded sample of bedrock from Test Pit TP-2 to estimate drained strength parameters for engineered fill. The sample was compacted to 90 percent relative compaction at 2 percentage points above optimum moisture content. To estimate undrained strength parameters for engineered fill, we performed an unconsolidated undrained triaxial compression test on remolded samples of bedrock from Test Pit 2TP-2. The samples were remolded to 92 percent relative compaction at 2 percentage points above optimum moisture content. We estimated shear strength parameters for the existing fill placed as part of the Highway 24 and Deer Hill Road improvements from SPT blow counts obtained from our test borings drilled as part of this study. To estimate undrained shear strengths of the colluvium, we performed an unconsolidated undrained triaxial compression test on a sample of the colluvium collected from Test Pit 2TP-1. The results of field strength tests were used to estimate undrained shear strengths for the alluvium. Drained shear strength parameters for the colluvium and alluvium were estimated from data published by Stark and Eid (1997) using index properties. We also estimated the strengths of the landslide debris using index properties. The sandstone bedrock material was modeled using equivalent Mohr-Columb strength parameters derived from the Generalized Hoek-Brown strength function.

TABLE 4.2.2-1
Summary of Shear Strength Parameters

Material	Drained Strength Parameters		Undrained Strength Parameters	
	Cohesion (psf)	Friction Angle (deg)	Cohesion (psf)	Friction Angle (deg)
Existing Fill (Qaf)	0	30	0	30
Engineered Fill (Qf) – proposed	0	33	500	23
Colluvium (Qc)	0	30	1,000	0
Alluvium (Qal)	0	30	1,500	0
Landslide	0	24	0	24
Bedrock (Tbr)	1,000	40	1,000	40

4.2.3 Results of Static Slope Stability Analyses

Appendix C shows the results of our static stability analyses for proposed slopes shown on Cross Sections 1-1', 2-2', and 3-3' with consideration to long-term conditions. The results are summarized in Table 4.2.3-1. The results for Cross Sections 2-2' indicate a factor of safety above commonly accepted criteria. However, the results for Cross Sections 1-1' and 3-3' indicate mitigation will be required to reduce the risk of static (long-term) slope stability affecting proposed improvements. We provide recommendations for mitigation of potential long-term slope instability in Section 5 of this report.

TABLE 4.2.3-1
Static Slope Stability

Section	Minimum Static Fs
1-1'	1.1
2-2'	1.7
3-3'	1.3

4.2.4 Results of Seismic Slope Stability Analyses

We used the Anderson (2008) simplified Newmark analysis method to estimate seismically induced deformation for the slopes shown on Cross Sections 1-1', 2-2', and 3-3'based on the seismic yield coefficient obtained from pseudo-static analyses summarized in the table below. As discussed above, the yield coefficient was then used in combination with expected site ground motions corresponding to a seismic event with a probability of exceedance of 10 percent in 50 years based on the United States Geological Survey 2008 Seismic Hazard Map in our analyses. We include a summary of calculated seismic slope deformation for the cross sections analyzed in the table below.

TABLE 4.2.4-1
Pseudo-Static Slope Stability

Cross Section	Seismic Yield Coefficient (g)	Seismic Slope Deformation (inches)
1-1'	0.07	22
2-2'	0.20	1
3-3'	0.15	4

These estimated deformations correspond to the mean value. It is important to note that developers of this approach (as well as developers of similar approaches) consider the results of these analyses to be indices of expected seismic performance and not predictions of actual slope displacements. Based on guidance in California Geological Survey Special Publication 117A, the slope deformation estimated for Cross Sections 2-2' and 3-3' is unlikely to correspond to significant ground deformation. However, the estimated slope deformation for Cross Section 1-1' is likely to correspond to significant ground deformation. Accordingly, mitigation will be required for the proposed fill slope shown on Cross Section 1-1' to reduce the risk of seismically-induced slope deformation affecting proposed improvements. We provide recommendations for mitigation of potential seismic slope instability in Sections 5 of this report.

4.3 EXPANSIVE SOIL

Our laboratory testing indicates that the soils and bedrock at the site generally exhibit low to moderate shrink/swell potential with variations in moisture content. Laboratory testing on a near-surface soil sample collected from Boring B-1 indicates the soil in the northern portion of the site, in the area of the proposed parking lot, has a high expansion potential. 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. Expansive soil mitigation recommendations are presented in Sections 6 and 11 of this report.

4.4 EXISTING FILLS AND COLLUVIUM

In general, existing fills are present along the Caltrans right-of-way in the southern portion of the site and south of Deer Hill Road in the southwestern portion of the site. These fills were placed during previous grading for Highway 24 and Deer Hill Road. At some locations, the existing fills were placed directly on top of native colluvium. Existing fills and colluvium could undergo vertical movement that is not easily characterized and could ultimately be inadequate to effectively support the proposed engineered fill and building loads. Based on the proposed development plan, proposed fills, fill slopes, and some building pads will be situated in areas where existing fills and colluvium were encountered. Recommendations for addressing existing fills and colluvium are presented in Section 5 of this report.

4.5 COMPRESSIBLE SOIL

Fill up to approximately 40 feet thick is planned at the site, with the majority of the fill to be placed over bedrock. Approximately 10 feet of fill is planned at the northern portion of the site and will be placed over alluvium. Based on our subsurface exploration, laboratory test results, and the proposed grading and development layout described in Section 1.3, it is our opinion that the majority of any settlement from consolidation of the overconsolidated alluvial soil will occur during fill placement and will not significantly affect the proposed development. In order to confirm our opinion, ENGEO should be retained to review final grading and site improvement plans and observe and test earthwork construction at the site.

4.6 SHALLOW GROUNDWATER AND DEWATERING

Perched groundwater was encountered as shallow as 4 feet below existing grade at the time of our exploration. As a result, relatively shallow groundwater is present at the site at times during the year. While we do not anticipate below-grade levels for any of the structures, excavations to mitigate potential hazards or for planned cuts or utilities may encounter groundwater depending upon the time of year of construction. The need for temporary dewatering should be considered.

4.7 2013 CALIFORNIA BUILDING CODE SEISMIC PARAMETERS

The 2013 California Building Code (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 2013 CBC. We provide the 2013 CBC seismic design parameters in Table 4.7-1 below, which include design spectral response acceleration parameters based on the mapped Risk-Targeted Maximum Considered Earthquake (MCE_R) spectral response acceleration parameters.

TABLE 4.7-1
2013 CBC Seismic Design Parameters

Parameter	Design Value
Site Class	D
Mapped MCE_R Spectral Response Acceleration at Short Periods, S_S (g)	1.62
Mapped MCE_R Spectral Response Acceleration at 1-second Period, S_1 (g)	0.60
Site Coefficient, F_A	1.0
Site Coefficient, F_V	1.5
MCE_R Spectral Response Acceleration at Short Periods, S_{MS} (g)	1.62
MCE_R Spectral Response Acceleration at 1-second Period, S_{M1} (g)	0.90
Design Spectral Response Acceleration at Short Period, S_{DS} (g)	1.08
Design Spectral Response Acceleration at 1-second Period, S_{D1} (g)	0.6

4.8 CORROSIVITY CONSIDERATIONS

Two selected soil samples were collected for soluble sulfate concentration testing. These tests provide an indication of the corrosion potential of the soil environment on buried concrete structures. According to the sulfate test results, the sulfate ion concentration ranges from 5 to 3882 mg/kg of water-soluble sulfate (SO_4) concentration levels. The CBC references the 2008 American Concrete Institute Manual, ACI 318 (Chapter 4) for concrete requirements. ACI provides the following sulfate exposure categories, classes and concrete requirements in contact with soil based upon the exposure risk.

TABLE 4.8-1
Sulfate Exposure Categories and Classes

Sulfate Exposure Category S	Exposure Class	Water- Soluble Sulfate in Soil % by Weight
Not Applicable	S0	$SO_4 < 0.10$
Moderate	S1	$0.10 \leq SO_4 < 0.20$
Severe	S2	$0.20 \leq SO_4 \leq 2.00$
Very Severe	S3	$SO_4 > 2.00$

TABLE 4.8-2
Requirements for Concrete by Exposure Class

Exposure Class	Max w/cm	Min f'c (psi)	Cement Type ASTM C150	Cement Type ASTM C595	Cement Type ASTM C1157	Calcium Chloride Admixture
S0	N/A	2500	No Type restriction	No Type restriction	No Type restriction	No restriction
S1	0.5	4000	II†‡	IP(MS), IS(<70), (MS)	MS	No restriction
S2	0.45	4500	V‡	IP(HS), IS(<70), (HS)	HS	Not permitted
S3	0.45	4500	V + pozzolan or slag§	IP(HS) + pozzolan or slag or IS(<70) (HS) + pozzolan or slag§	HS + pozzolan or slag§	Not permitted

Notes:
† For seawater exposure, other types of portland cements with tricalcium aluminate (C_3A) contents up to 10 percent are permitted if the w/cm does not exceed 0.40.
‡ Other available types of cement such as Type III or Type I are permitted in Exposure Classes S1 or S2 if the C_3A contents are less than 8 or 5 percent, respectively.
§ The amount of the specific source of the pozzolan or slag to be used shall not be less than the amount that has been determined by service record to improve sulfate resistance when used in concrete containing Type V cement. Alternatively, the amount of the specific source of the pozzolan or slag to be used shall not be less than the amount tested in accordance with ASTM C1012 and meeting the criteria in ACI 4.5.1.

In accordance with the criteria presented above, the highest test result is classified in the "Severe" sulfate exposure class. Cement type, maximum water-cement ratio, and minimum concrete strength for this exposure class are specified in the table above.

Testing was not completed for all depths of potential embedment. Once more specifics of the proposed improvements are known, we can provide additional testing and/or guidance regarding the exposure risk for sulfates.

4.9 EXCAVATABILITY

Based on our field exploration, it is our opinion that the site soils and bedrock should be rippable with conventional heavy construction equipment, such as a Caterpillar D-9 or larger. Localized cemented lenses or beds may be encountered, which will likely require considerable ripping effort and generate oversized material (greater than 6 inches in diameter). Backhoes may experience difficulty excavating in some of the less weathered bedrock. We anticipate that heavy-duty excavators with rock buckets should be capable of trenching the materials; however, in some instances significant difficulty may be encountered and should be anticipated.

We provide this information for general planning purposes only. This information is not intended for bidding purposes.

4.10 CONCLUSIONS

From a geologic and geotechnical standpoint, the study area appears to be suitable for residential development provided the recommendations provided in this report and other sound engineering practices are incorporated in the design and construction of the project. As discussed above and based on this geotechnical exploration and review of previous studies, the main geologic/geotechnical considerations to be addressed at the site are summarized below. The recommendations in subsequent sections of this report address these considerations.

- Slope stability
- Existing fill
- Expansive soils

5.0 EARTHWORK RECOMMENDATIONS

5.1 GRADING

The following grading recommendations are provided for the project based upon the current plan prepared by prepared by BKF (date March 6, 2014). The grading recommendations provided in this report are appropriate for planning purposes for the entire site. Development of the final grading plans should be coordinated with the Geotechnical Engineer and Engineering Geologist in order to tailor the plans to accommodate known soil and geologic hazards and to improve the overall stability of the site. The final 40-scale grading plans for the project should be reviewed by the Geotechnical Engineer. Detailed locations of keyways, subdrains and subexcavation areas will be outlined on these plans during our review, as applicable.

The Geotechnical Engineer should be notified at least 3 days prior to grading in order to coordinate its schedule with the grading contractor. Grading operations should be observed and tested by the Geotechnical Engineer.

5.2 SELECTION OF MATERIALS

With the exception of some construction debris (wood, brick, metal, etc.), trees, organically contaminated materials (soil which contains more than 3 percent organic content by weight), and environmentally impacted soils, we anticipate the site soils and bedrock derived materials are suitable for use as general fill. Other materials and debris, including trees with their root balls, should be removed from the project site. We recommend that fill material derived from low-plasticity bedrock or low-plasticity granular soil be used for the construction of fill slopes with inclinations steeper than 3:1 (horizontal to vertical) and heights over 10 feet. Low-plasticity bedrock and soil is defined here as material with a Plasticity Index less than 12.

Oversized soil or rock materials (those exceeding two-thirds of the lift thickness or 6 inches in dimension, whichever is less) should be removed from the fill and broken down to meet this requirement or otherwise off-hauled.

The Geotechnical Engineer should be informed when import materials are planned for the site. Import materials should be submitted to, and approved by, the Geotechnical Engineer prior to delivery at the site.

5.3 DEMOLITION AND STRIPPING

Site preparation should commence with removal of site vegetation, structures, and surface and subsurface improvements. Following the demolition of existing improvements, site development should include removal of debris, loose soil, and soft compressible materials in any location to be graded. Any soft compressible soils should be removed from areas to receive fill or structures, or those areas to serve as borrow. Vegetation and debris should be separately stockpiled from soft compressible material and existing soil fill.

If desired, reuse of the existing asphalt concrete grindings within future paved areas could be considered from a geotechnical standpoint. The material should be broken down, but not pulverized, to meet a 6-inch or less particle size and placed in a separate stockpile outside the limits of grading until used within street areas below subgrade. The asphaltic concrete grindings should be thoroughly mixed with soil and placed as engineered fill below street or parking lot subgrade elevations. Reuse of existing paving materials as engineered fill within future streets could add a "green" recycling component to the project and also save costs to export and depose these materials. Reuse of this material as part of the planned pavement section or placement of this material within future building pads is not recommended.

No loose or uncontrolled backfilling of depressions resulting from demolition and stripping or other soil removal should be permitted.

5.4 EXISTING FILLS, COLLUVIUM, AND LANDSLIDE DEBRIS

Based on our field exploration, existing undocumented fill is present along the Caltrans right-of-way in the southern portion of the site and south of Deer Hill Road in the southwestern portion of the site. These fills were placed during previous grading for Highway 24 and Deer Hill Road.

Existing fills and compressible soils are unsuitable to remain below proposed structures and should be subexcavated to expose underlying competent native soils that are approved by the Geotechnical Engineer. The base of the excavations should be processed, moisture conditioned, as needed, and compacted in accordance with the subsequent recommendations for engineered fill.

Based on our field exploration, colluvial soils and landslide debris are present underlying the existing fills and within swales at portions of the site as shown on Figure 3. Colluvium, compressible soils, and landslide debris are unsuitable to remain below proposed structures and should be subexcavated to expose underlying competent native soils that are observed by the Geotechnical Engineer. The base of the excavations should be processed, moisture conditioned, as needed, and compacted in accordance with the subsequent recommendations for engineered fill.

5.5 TOE KEYWAYS

Construction of subsurface drainage within keyways at the toes of proposed fill slopes will be required to mitigate potential slope stability hazards. We anticipate that typical keyway designs will consist of 24 to 30-foot-wide keyways constructed to a minimum depth of 5 to 30 feet, or extending below existing fills and colluvium, and at least 3 feet into competent native materials, whichever is deeper. Subsurface drainage systems should be installed within the keyways as recommended in a subsequent section. A typical keyway detail is presented on Figure 6. At some locations, keyway drainage is not possible due to unavailable subdrain outfall elevations. In these cases, keyways should be designed for undrained conditions. Keyways should be backfilled with material derived from low-plasticity bedrock or low-plasticity granular soil (material with a Plasticity Index less than 12) compacted to at least 95 percent relative compaction at 2 percent above optimum moisture content. Geogrid is recommended within keyways at some locations as discussed in Section 5.6 below.

Actual subsurface mitigation configurations (including size and depths of keyways) will be shown on the final 40-scale remedial grading plans and after additional detailed slope stability analyses have been performed where necessary. Fills should be adequately keyed and benched into competent material or bedrock materials as evaluated by the Geotechnical Engineer during fill slope construction. Observation and evaluation of exposed conditions by the Geotechnical Engineer in the field will allow for modifications to the actual depth and location of the keyways, subexcavated benches, and locations of subdrains on actual field conditions and geometry exposed during grading. Figure 5 includes conceptual remedial grading measures for Cross Sections 1-1', 2-2', and 3-3'.

5.6 GEOGRID REINFORCEMENT

As discussed above, results of slope stability analyses of proposed fill slopes shown on Cross Sections 1-1' and 3-3' indicate mitigation is required. We recommend that geogrid reinforcement be placed in engineered fills for toe keyways and slopes to reduce the potential static and seismic slope instability at these locations. The geogrid-reinforced fill material should be derived from low-plasticity bedrock or low-plasticity granular soil (material with a Plasticity Index less than 12). We performed slope stability analyses to evaluate conceptual mitigation using geogrid-reinforced engineered fill, the results of which are included in Appendix C.

The results of static stability analysis for the proposed geogrid-reinforced engineered fill slopes shown on Cross Sections 1-1' and 3-3' indicate factors of safety in conformance with commonly accepted criteria. For Cross Section 1-1', we used the Anderson (2008) simplified Newmark analysis method to estimate seismically induced deformation based on the seismic yield coefficient obtained from pseudo-static analyses included in Appendix C. We estimate a seismic slope deformation of approximately 4 inches for Cross Section 1-1'. These estimated deformations correspond to the mean value. It is important to note that developers of this approach (as well as developers of similar approaches) consider the results of these analyses to be indices of expected seismic performance and not predictions of actual slope displacements. Based on guidance in California Geological Survey Special Publication 117A, the slope deformation estimated for Cross Section 1-1' with geogrid-reinforced engineered fill is unlikely to correspond to significant ground deformation.

In addition to mitigation of potential static and seismic slope instability, we recommend the use of biaxial geogrid within the outer portion of slopes that do not conform to the slope gradient guidelines provided below to reduce the risk of local surficial failures. A detailed design of the proposed geogrid-reinforced fill at the locations referenced above should be performed as part of corrective grading plan development once final 40-scale grading plans are available for the project.

5.7 SUBSURFACE DRAINAGE FACILITIES

Subsurface drainage systems are planned for keyways, and at the base of removal areas, as a minimum. Secondary bench subdrains may also be required, depending upon the height of the fill slope and the slope of the underlying native terrain. In addition, observed seepage areas or suspected spring areas should be controlled in development areas through the use of subdrains. Positive fall of at least ½ (selectively) to 1 percent towards an approved outlet should also be provided for all subdrains. As noted above, some keyways will be designed for saturated conditions due to the lack of suitable subdrain outfall locations.

The recommended locations of the subdrains will be approximately located on the corrective grading plans used during site grading. We provide general details for these on Figure 7. As shown on Figure 7, subdrain systems should consist of a minimum 6-inch-diameter perforated pipe encased in Caltrans Class 2 permeable material or crushed rock wrapped in filter fabric.

Subdrain pipe should conform to the specifications below unless otherwise recommended by ENGEO in the field.

- For design depths less than 30 feet, the following pipe types are appropriate:
  - Perforated ABS Solid Wall SDR 35 (ASTM D-2751)
  - Perforated PVC Solid Wall SDR 35 (ASTM D-3034)
  - Perforated PVC A-2000 (ASTM F949)
  - Perforated Corrugated HDPE double-wall (AASHTO M-252 or M-294, Caltrans Type S, 50 psi minimum stiffness)
  - Double-Drained High Flow Profile Polypropylene Composite (ASTM D-1621)

- For design depths less than 50 feet, the following pipe types are appropriate:
  - Perforated PVC SDR 23.5 Solid Wall (ASTM D-3034)
  - Perforated Schedule 40 PVC Solid Wall (ASTM-1785)
  - Perforated ABS SDR 23.5 Solid Wall (ASTM D-2751)
  - Perforated ABS DWV/Sch. 40 (ASTM D-2661 and D-1527)
  - Perforated Corrugated HDPE double-wall (AASHTO M-252 or M-294, Caltrans Type S, 70 psi minimum stiffness)
  - Double-Drained High Flow Profile HDPE Composite (ASTM D-3350)

Discharge from the subdrains will generally be low, but in some instances may be continuous. Subdrains should outlet into the storm drain system or other approved outlets and their locations should be surveyed and documented by the project Civil Engineer for future maintenance.

Not all sources of seepage are evident during the time of field work because of the intermittent nature of some of these conditions and their dependence on long-term climatic conditions. Furthermore, new sources of seepage may be created by a combination of changed topography, manmade irrigation patterns, and potential utility leakage. Since uncontrolled water movements are one of the major causes of detrimental soil movements, it is of utmost importance that a Geotechnical Engineer be advised of any seepage conditions so that remedial action may be initiated, if necessary

5.8 GRADED SLOPES

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

TABLE 5.8-1
Slope Gradient Guidelines

Slope Gradient (horizontal:vertical)	Cut Slope Height (feet)	Fill Slope Height (feet)
2:1	50 or less	50 or less
3:1	Greater than 50	Greater than 50

Based on the grading plan prepared by BKF, dated March 21, 2011, and the subsurface conditions, we anticipate that the majority of material generated by cuts will be derived from

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low-plasticity bedrock. The fill slope criteria provided for 2:1 (horizontal to vertical) slopes in Table 5.8-1 are based on the assumption that the fill material used in the zone extending a distance of at least 1½ times the height of the slope laterally from the slope face will be derived from low-plasticity bedrock or low-plasticity granular soil. Low-plasticity bedrock and soil is defined here as material with a Plasticity Index less than 12. If other material is used for fill slope construction, we recommend a maximum fill slope height of 10 feet for 2:1 slopes. In accordance with the 2013 CBC requirements, we recommend that slopes with inclinations steeper than 3:1 be graded with terraces at least 6 feet in width at not more than 30-foot vertical intervals.

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 or incorporation of geogrid-reinforcing materials may be required. For example, the proposed fill slope shown on Cross Section 2-2' of Figure 5, which is situated below Lots 35 and 36, is shown on the grading plan at an inclination of 2:1 and a height greater than 50 feet. Therefore, we recommend the use of biaxial geogrid within the outer portion of the slope at this location to reduce the risk of surficial failures. Additionally, cut-fill transition slopes should be overexcavated and reconstructed as engineered fill slopes.

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 analyses will also be performed as part of this review process to confirm minimum factors of safety will be achieved.

During grading, cut slopes should be observed and mapped by an engineering geologist. If adverse conditions are observed in the field during grading, it may be necessary to reconstruct the slopes as engineered fill slopes.

5.8.1 Erosion Control

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. Additionally, 2:1 slopes should be provided with erosion control protection such as Rhino Snot Soil Stabilizer or other equivalent soil stabilization product.

5.9 SLOPE SETBACKS

The recommended slope setbacks for habitable structures are variable depending on slope height and soil conditions. Slope setbacks are intended to reduce the potential effects of long-term slope creep and possible earthquake-induced slope displacements on structures. For structures adjacent to fill slopes, we recommend a minimum setback of at least 15 feet or one-third of the slope height, whichever is greater, from the tops of slopes. For higher slopes, the minimum setback can be reduced to as little as 15 feet if the slope is provided with geogrid reinforcement designed for the specific slope condition. For structures adjacent to cut slopes in bedrock, we recommend a minimum setback of 15 feet from the top of slope.

We recommend a minimum setback of 15 feet from the toe of slopes for habitable structures to reduce the risk of adverse impacts from potential slope movement under static or seismic loading conditions.

5.10 CUT AND CUT-FILL TRANSITION LOTS

We recommend that the upper 2 feet of subgrade soils in areas of cut and cut-fill transitions be made uniform by subexcavating the soil and replacing it as engineered fill. This condition will be achieved as a result of remedial grading operations. This requirement will provide a relatively uniform, moisture conditioned state for the foundation subgrade soils. We provide recommendations for fill placement in a subsequent section of this report.

5.11 DIFFERENTIAL FILL THICKNESS

For subexcavation activities that create a differential fill thickness across individual building pads, mitigation to achieve a similar fill thickness across the pad is beneficial for the performance of a shallow foundation system. We recommend a maximum differential fill thickness of 10 feet across individual building pads to reduce the risk of differential settlement. For a differential fill thickness exceeding 10 feet across an individual pad, we recommend performing subexcavation activities to bring this vertical distance to within the 10-foot tolerance and replacement of this material as engineered fill. As a minimum, the subexcavation area should include the entire structure footprint plus 5 feet beyond the edges of the building footprint.

5.12 FILL PLACEMENT

Once a suitable firm base is achieved for general fill areas, the exposed non-yielding surface should be scarified to a depth of 12 inches, moisture conditioned, and recompacted to provide adequate bonding with the initial lift of fill. All fills should be placed in thin lifts, with the lift thickness not to exceed 8 inches or the depth of penetration of the compaction equipment used, whichever is less.

The following compaction control requirements should be applied to keyway backfill:

Test Procedures:	ASTM D-1557.
Required Moisture Content:	Not less than 2 percentage points above optimum moisture content.
Minimum Relative Compaction:	Not less than 95 percent.

The following compaction control requirements should be applied to general fill areas:

Test Procedures:	ASTM D-1557.
Required Moisture Content:	Not less than 3 percentage points above optimum moisture content.
Minimum Relative Compaction:	Not less than 90 percent.

5.13 MONITORING AND TESTING

It is important that all site preparations for site grading be performed under the observation of the Geotechnical Engineer's field representative. The Geotechnical Engineer's field representative should observe all graded area preparation, including demolition and stripping. The final grading plans should be submitted to the Geotechnical Engineer for review.

6.0 FOUNDATION RECOMMENDATIONS

The primary consideration for foundation design at the site is expansive soil. Alternatives for addressing the effects of the expansive soil on building foundations include post-tensioned mat foundations or grading building pads with non-expansive select fill. We anticipate that a post-tensioned mat foundation bearing on compacted fill would be preferred for support of the proposed residential structures. 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 is extremely difficult to remoisturize clayey soils without excavation, moisture conditioning, and recompaction.

6.1 POST-TENSIONED MAT FOUNDATIONS

Post-tensioned (PT) mat foundations should be designed using the criteria presented in Table 6.1-1 below. These mats should have a minimum thickness of 10 inches and be thickened to at least 12 inches at the perimeter. PT mats should be designed for an average allowable bearing pressure of 1,000 pounds per square foot (psf) for dead plus live loads, with maximum localized bearing pressures of 1,500 psf for column or wall loads. Allowable bearing pressures can be increased by ⅓ for wind or seismic loads.

TABLE 6.1-1
Post-Tension Design Criteria

Condition	Center Lift	Edge Lift
Edge Moisture Variation Distance, e_m (feet)	9.0	4.6
Differential Soil Movement, y_m (inches)	0.3	0.7

7.0 INTERIOR SLABS-ON-GRADE

7.1 SLAB MOISTURE VAPOR REDUCTION

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

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

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

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

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

8.0 EXTERIOR SLABS-ON-GRADE

Exterior flatwork includes items such as concrete sidewalks, steps, and outdoor courtyards exposed to foot traffic only. Concrete flatwork should have a minimum thickness of 4 inches. Control and construction joints should be constructed in accordance with current Portland Cement Association Guidelines.

Exterior slabs-on-grade should be designed specifically for their intended use and loading requirements. Cracking of conventional slabs should be expected due to concrete shrinkage. Slabs-on-grade should be reinforced for control of cracking, and frequent control joints should be provided to control the cracking. Reinforcement should be designed by the Structural Engineer. In our experience, welded wire mesh may not be sufficient to control slab cracking. As a minimum, exterior slabs-on-grade should be reinforced with No. 3 bars spaced 18 inches on center each way.

A 4-inch-thick layer of clean crushed rock or gravel should be placed under slabs. Exterior slabs should be constructed with thickened edges extending at least beneath the granular material into compacted soil to reduce water infiltration. Slabs should slope away from the buildings at a slope of at least 2 percent to prevent water from flowing toward the building.

9.0 RETAINING WALLS

9.1 CANTILEVER RETAINING WALLS

Unrestrained drained retaining walls constructed on level ground and up to 10 feet in height may be designed using active equivalent fluid pressures as follows.

TABLE 9.1-1
Active Equivalent Fluid Pressures

Backfill Slope Condition (horizontal:vertical)	Active Pressure (pounds per cubic foot)
Level	45
3:1	60
2:1	70

Restrained walls should be designed as drained retaining walls using an at-rest fluid pressure of 70 pcf for level backfill conditions. Restrained walls should be designed to resist an additional uniform pressure equivalent to 35 percent of any surcharge loads and restrained walls should be designed to resist an additional uniform pressure equivalent to 50 percent of any surcharge loads applied at the surface.

Seismic loading for walls with retained heights greater than 6 feet should be considered in accordance with ASCE 7-10. We recommend a dynamic seismic lateral earth pressure corresponding to 15H, where H is the height of the retaining wall and the seismic earth pressure (in psf) has a uniform distribution. When considering seismic earth pressures for unrestrained and restrained retaining walls, the recommended seismic earth pressure increment should be added to the active earth pressures provided above.

Passive pressures acting on foundations may be assumed as 300 pounds per cubic foot (pcf) provided that the area in front of the retaining wall is level for a distance of at least 10 feet or three times the depth of foundation and keyway, whichever is greater. The upper 1 foot of soil should be excluded from passive pressure computations. The friction factor for sliding resistance may be assumed as 0.35. It is recommended that retaining wall footings be designed using an allowable bearing pressure of 2,500 pounds per square foot (psf). Appropriate safety factors against overturning and sliding should be incorporated into the design calculations.

All retaining walls 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. 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 be preapproved by the Geotechnical Engineer prior to delivery.

All backfill should be placed in accordance with the recommendations provided above for engineered fill. Light equipment should be used during backfill compaction to reduce possible overstressing of the walls. The foundation details and structural calculations for retaining walls should be submitted for review.

9.2 MECHANICALLY STABILIZED EARTH WALLS

As an alternative to cantilever retaining walls, we are also providing mechanically stabilized earth (MSE) wall recommendations and design criteria. Based on the proposed site retaining wall layout, segmental blocks with fiber glass pin connections for geogrid (e.g. Keystone Standard 21½-inch locks or equivalent) may be used. Seismic loading for walls with retained heights greater than 6 feet should be considered using design earthquake ground motions as discussed in ASCE 7-10. The walls should also consider any surcharge loads applied at the surface such as those imposed by traffic or adjacent structures.

We have assumed that the proposed wall will be founded on prepared subgrade in conformance with recommendations for fill placement provided in Section 5 of this report. In addition, we have assumed that material derived from low-plasticity bedrock or low-plasticity granular soil (material with a Plasticity Index less than 12) will be used as the foundation fill, retained soil, and reinforced fill soil for the MSE walls. Accordingly, the following soil material parameters should be incorporated in the MSE wall design.

TABLE 9.2-1
Soil Material Parameters

Condition	Cohesion (c') (pcf)	Friction Angle (∅') (degrees)	Unit Weight (γ) (pcf)
Reinforced Fill	0	33	120
Retained Soil	0	33	120
Foundation Fill	0	33	120

We recommend that the following minimum factors of safety be incorporated in the MSE wall design.

TABLE 9.2-2
External Stability

Condition	Safety Factor (Static/Seismic)
Sliding	1.5 / 1.1
Bearing Capacity	2.0 / 1.5
Overturning	2.0 / 1.5

TABLE 9.2-3
Internal Stability

Condition	Safety Factor (Static/Seismic)
Pull-out Resistance	1.5 / 1.1

10.0 EXCAVATIONS AND TEMPORARY SHORING SYSTEMS

Excavations, including utility trenches, should be properly excavated and shored, as applicable, to create a stable and safe condition. It is the responsibility of the Contractor to provide such stable, safe trench and construction slope conditions and to follow OSHA safety requirements. Since excavation procedures may be very 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.

While not anticipated at this time, recommendations for shoring design can be provided upon request. The contractor should be responsible for the design and construction of all shoring and underpinning systems and the safety of all workers within excavations.

11.0 PAVEMENT DESIGN

The following pavement sections have been determined based on an estimated R-value of 5, for a Traffic Index of 5 and 6, and according to the method contained in Topic 608 of Highway Design Manual by Caltrans. As discussed above, laboratory test results on soil samples collected in the proposed parking lot area in northern portion of the site indicate the soils have a high potential for shrink and swell resulting from moisture variation. Settlement and heave from shrink and swell could adversely impact pavements in this area. In order to reduce this risk, we recommend the use of non-expansive fill within the upper 12 inches of pavement subgrade, which could include non-expansive fill (PI less than 12) or lime treatment of expansive subgrade soil. ENGEO should be consulted to provide supplemental recommendations if lime treatment of the parking lot subgrade soil is planned.

TABLE 11.0-1
Pavement Sections

Traffic Index	HMA (inches)	Class 2 AB (inches)
5.0	3.0	10
6.0	3.5	13

AB –Caltrans Class 2 aggregate base (R-value of 78)

Pavement construction and all materials (hot mix asphalt and aggregate base) should comply with the requirements of the Standard Specifications of the State of California Division of Highways, City of Lafayette requirements and the following minimum requirements.

- All pavement subgrades should be scarified to a depth of 10 to 12 inches below finished subgrade elevation, moisture conditioned to 2 percentage points above optimum moisture content, and compacted to at least 95 percent relative compaction.

- Subgrade soils should be in a stable, non-pumping condition at the time aggregate baserock materials are placed and compacted. Proof-rolling with a heavy wheel-loaded piece of construction equipment should be implemented. Yielding materials should be appropriately mitigated, with suitable mitigation measures developed in coordination with the client, contractor and Geotechnical Engineer.

- Adequate provisions must be made such that the subgrade soils and aggregate baserock materials are not allowed to become saturated.

- Aggregate baserock materials should meet current Caltrans specifications for Class 2 aggregate baserock and should be compacted to at least 95 percent of maximum dry density at a moisture content of at least optimum. Proof-rolling with a heavy wheel-loaded piece of construction equipment should be implemented after placement and compaction of the aggregate base. Yielding materials should be appropriately mitigated, with suitable mitigation measures developed in coordination with the client, contractor and Geotechnical Engineer.

- Hot mix asphalt paving materials should meet current Caltrans specifications.

- All concrete curbs separating pavement and irrigated landscaped areas should extend into the subgrade and below the bottom of adjacent aggregate baserock materials. An undercurb drain could also be considered to help collect and transport subsurface seepage.

12.0 DRAINAGE

The building pads must be positively graded at all times to provide for rapid removal of surface water runoff away from the foundation systems, and to prevent ponding of water under foundations or seepage toward the foundation systems at any time during or after construction. Ponded water will cause undesirable soil swell and loss of strength. As a minimum requirement, finished grades should have slopes of at least 5 percent within 10 feet, as applicable, from the exterior walls and at right angles to allow surface water to drain positively away from the structures. For paved areas, the slope gradient can be reduced to 2 percent.

All surface water should be collected and discharged into outlets approved by the Civil Engineer. Landscape mounds must not interfere with this requirement. In addition, each lot should drain individually by providing positive drainage or sufficient area drains around the building to remove excessive surface water.

All roof stormwater should be collected and directed to downspouts. Stormwater from roof downspouts should not be allowed to discharge directly onto the ground surface. We recommend downspouts discharge at least 5 feet away from foundations and the minimum gradient within

5 feet from the foundation should be increased from 3 to 5 percent. Alternatively, engineered stormwater systems can be developed under the guidance of ENGEO.

The occurrence of surface water infiltrating, ponding, and saturating the foundation soils can cause loss of soil strength and undesirable shrinking/swelling of the foundation soils. For structural mat foundation systems, if at any time adequate drainage away from the foundation cannot be achieved, then additional measures to hinder saturation of foundation soils must be provided. This may be accomplished by installing a perimeter subdrain system. Under no circumstance should the subdrain facilities be connected to the surface water collection system.

13.0 REQUIREMENTS FOR LANDSCAPING IRRIGATION

The geotechnical foundation design parameters contained in this report have considered the swelling potential of some of the site soils; however, it is important to recognize that swell in excess of that anticipated is possible under adverse drainage or irrigation conditions. Therefore, planted areas should be avoided immediately adjacent to the buildings. If planting adjacent to a structure is desired, the use of watertight planter boxes with controlled discharge or the use of plants that require very little moisture is recommended.

Sprinkler systems should not be installed where they may cause ponding or saturation of foundation soils within 3 feet from walls. Such ponding or saturation could result in undesirable soil swell, loss of compaction and consequent foundation and slab movements. Irrigation of landscaped areas should be strictly limited to that necessary to sustain vegetation. The Landscape Architect and prospective owners should be informed of the surface drainage and irrigation requirements included in this report.

14.0 UTILITIES

It is recommended that utility trench backfilling be done under the observation of a Geotechnical Engineer. Ideally, pipe zone backfill (i.e., material beneath and immediately surrounding the pipe) should consist of native material less than ¾ inch in maximum dimension compacted in accordance with recommendations provided above for engineered fill. Trench zone backfill (i.e. material placed between the pipe zone backfill and the ground surface) should also consist of native soil compacted in accordance with recommendations for engineered fill. Controlled density fill is also suitable for pipe zone and trench zone backfill.

If required by local agencies, where import material is used for pipe zone backfill, we recommend it consist of quarry fines, fine- to medium-grained sand, or a well-graded mixture of sand and gravel and that this material not be used within 2 feet of finish subgrades. This material should be compacted to at least 90 percent relative compaction at a moisture content of not less than optimum.

In general, uniformly graded gravel should not be used for pipe or trench zone backfill due to the potential for migration of soil into the relatively large void spaces present in this type of material and for movement of water along trenches backfilled with this type of material. If uniformly graded gravel is used, we recommend that it be encapsulated in 6-ounce filter fabric. Providing outlet locations into manholes or catch basins for water collected in granular trench backfill should also be considered.

All utility trenches entering building or paved areas should be provided with a soil plug (seal) where the trenches pass under or through the building perimeter or curb lines. The soil plug should extend at least 3 feet to both sides of the crossing and should be placed below, around, and above the utility pipe such that it is entirely in contact with the trench walls and pipe. This is to prevent surface water percolation into the import sand or gravel pipe zone backfill under foundations and pavements where such water would remain trapped in a perched condition.

Care should be exercised where utility trenches are located beside foundation areas. Utility trenches constructed parallel to foundations should be located entirely above a plane extending down from the lower edge of the footing at an angle of 45 degrees. Utility companies and Landscape Architects should be made aware of this information.

Compaction of backfill by jetting should not be allowed at this site. If there appears to be a conflict between the City or other Agency requirements and the recommendations contained in this report, this should be brought to the Owner's attention for resolution prior to submitting bids.

15.0 LIMITATIONS AND UNIFORMITY OF CONDITIONS

This report is issued with the understanding that it is the responsibility of the owner to transmit the information and recommendations of this report to developers, owners, buyers, architects, engineers, and designers for the project so that the necessary steps can be taken by the contractors and subcontractors to carry out such recommendations in the field. The conclusions and recommendations contained in this report are solely professional opinions.

The professional staff of ENGEO Incorporated strives to perform its services in a proper and professional manner with reasonable care and competence but is not infallible. There are risks of earth movement and property damages inherent in land development. 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 preparation of ENGEO's report. 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-study area 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.

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LIST OF FIGURES

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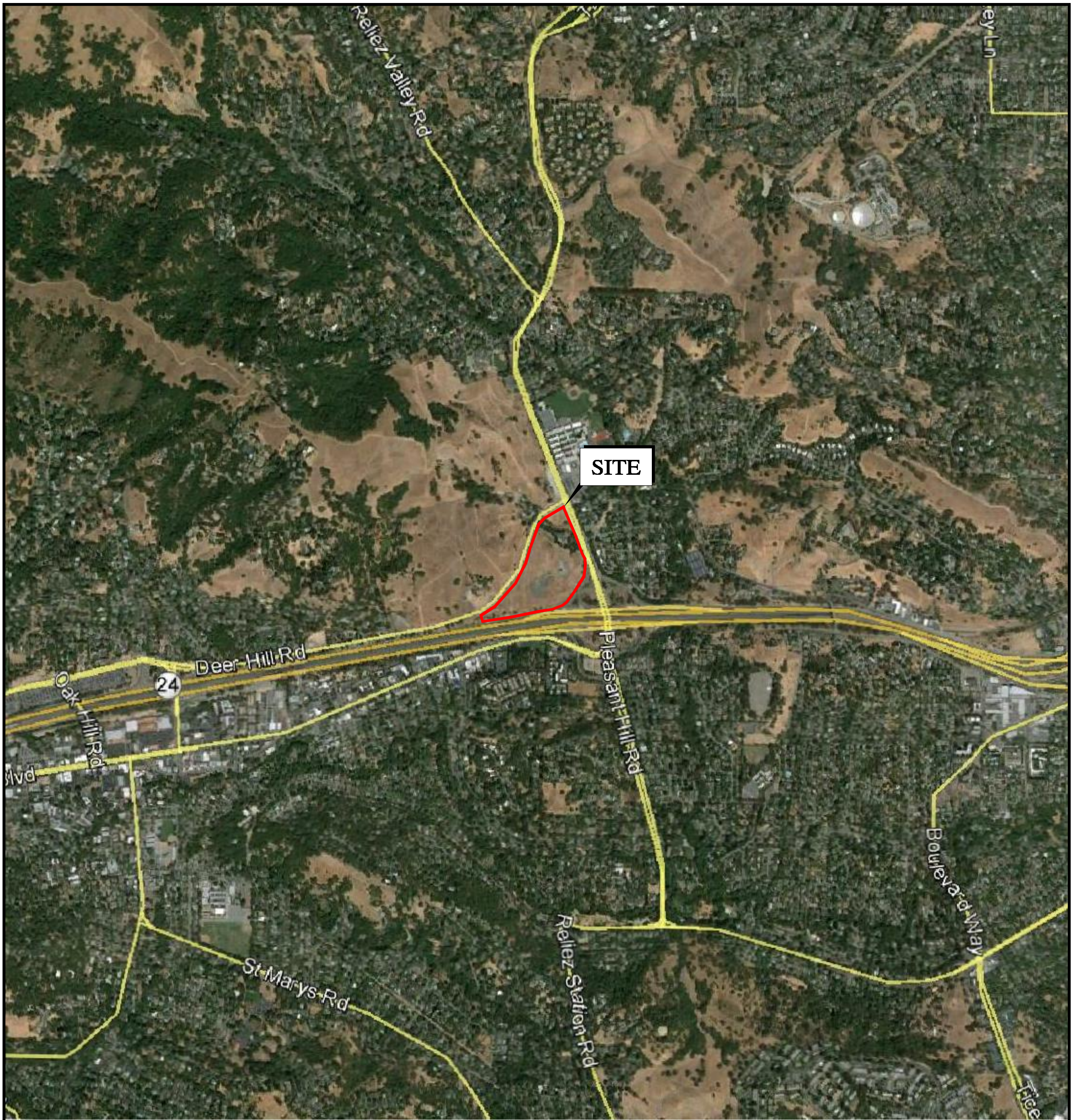

N

0 FEET 2000
0 METERS 1000

BASE MAP SOURCE: GOOGLE EARTH PRO

ENGEO
Expect Excellence

VICINITY MAP
THE TERRACES OF LAFAYETTE
LAFAYETTE, CALIFORNIA

PROJECT NO.: 9181.200.000	
SCALE: AS SHOWN	
DRAWN BY: SRP	CHECKED BY: PCG

FIGURE NO. 1

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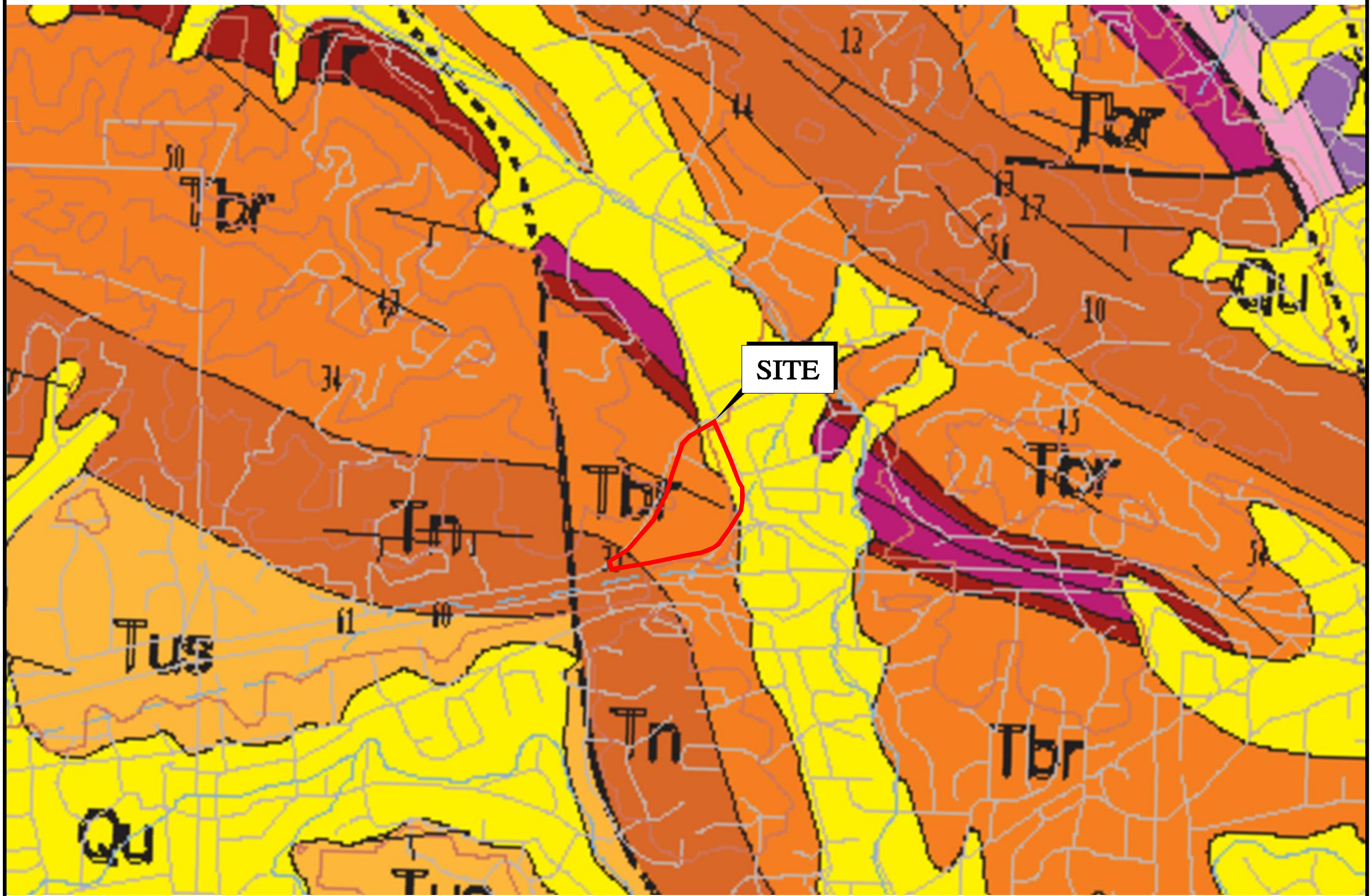

EXPLANATION

BEDROCK CONTACT-DASHED WHERE GRADATIONAL OR APPROXIMATELY LOCATED

FAULT-DASHED WHERE INFERRED, DOTTED WHERE CONCEALED, QUERIED WHERE EXISTENCE IS DOUBTFUL; DOUBLE ARROWS INDICATE STRIKE-SLIP MOVEMENT

STRIKE AND DIP OF STRATA

INCLINED

Qu	UNDIVIDED QUATERNARY DEPOSITS
Tor	ORINDA FORMATION
Tus	UNAMED SEDIMENTARY AND VOLCANIC ROCKS
Tn	NEROLY FORMATION
Tbr	BRIONES FORMATION

N

0 FEET 2000
0 METERS 1000

BASE MAP SOURCE: GRAYMER, 1994

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REGIONAL GEOLOGIC MAP
THE TERRACES OF LAFAYETTE
LAFAYETTE, CALIFORNIA

PROJECT NO.: 9181.200.000		FIGURE NO
SCALE: AS SHOWN		2
DRAWN BY: SRP	CHECKED BY: PCG	

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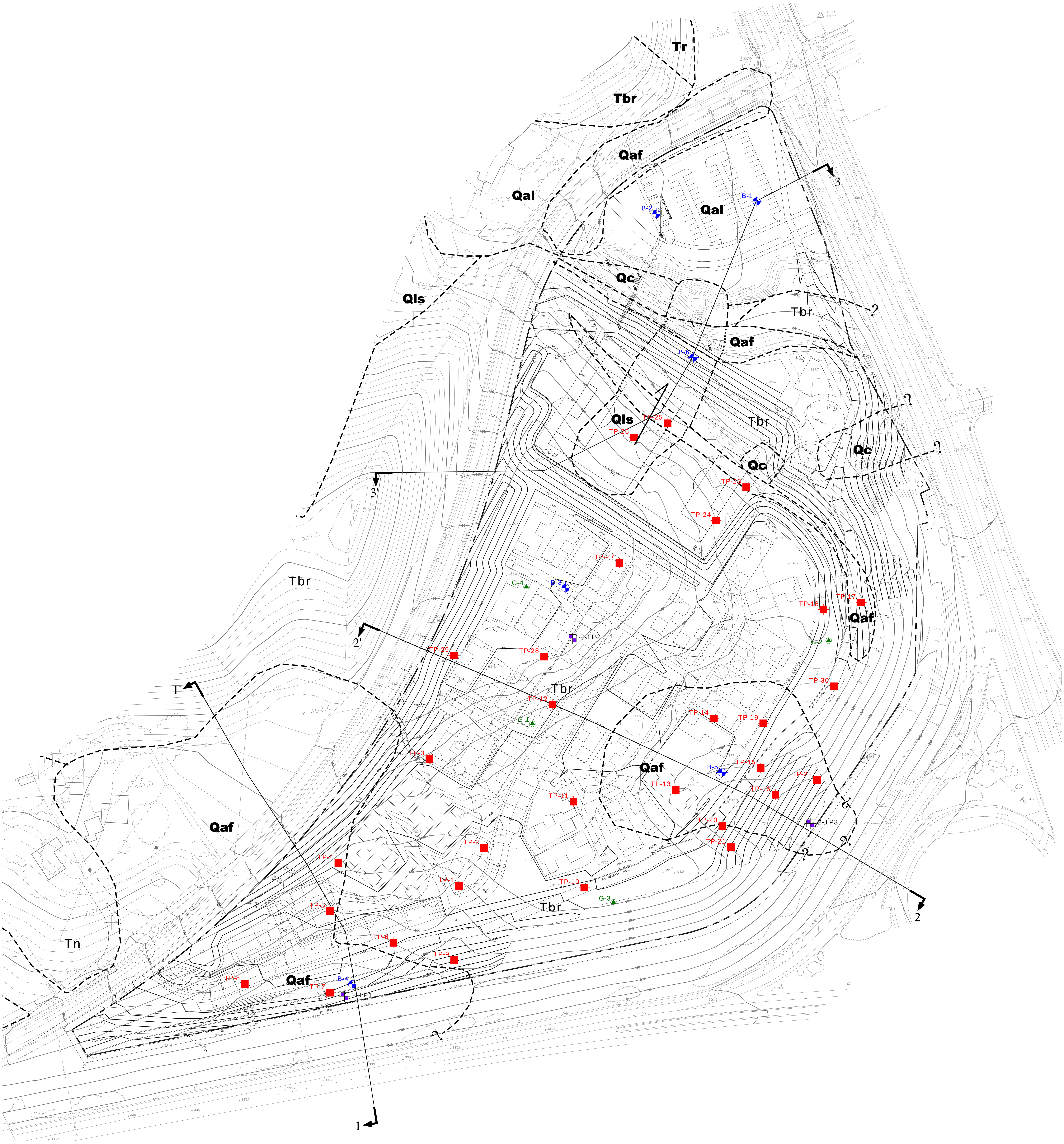

EXPLANATION

ALL LOCATIONS ARE APPROXIMATE

Qaf EXISTING FILL

Qls LANDSLIDE

Qal ALLUVIUM

Qc COLLUVIUM

EARTHFLOW

DEEP-SEATED

Tr RODEO SHALE

Tbr BRIONES SANDSTONE

Tn NEROLY FORMATION

G-4 GRAB SAMPLE LOCATION (ENGEO, 2011)

TP-29 TEST PIT LOCATION (ENGEO, 2011)

2-TP3 TEST PIT LOCATION (ENGEO, 2014)

B-6 BORING LOCATION (ENGEO, 2011)

BASE MAP SOURCE: UNKNOWN

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SITE PLAN AND GEOLOGIC MAP
THE TERRACES OF LAFAYETTE
LAFAYETTE, CALIFORNIA

PROJECT NO.: 9181.200.000
SCALE: AS SHOWN
DRAWN BY: SRP
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FIGURE NO. 3

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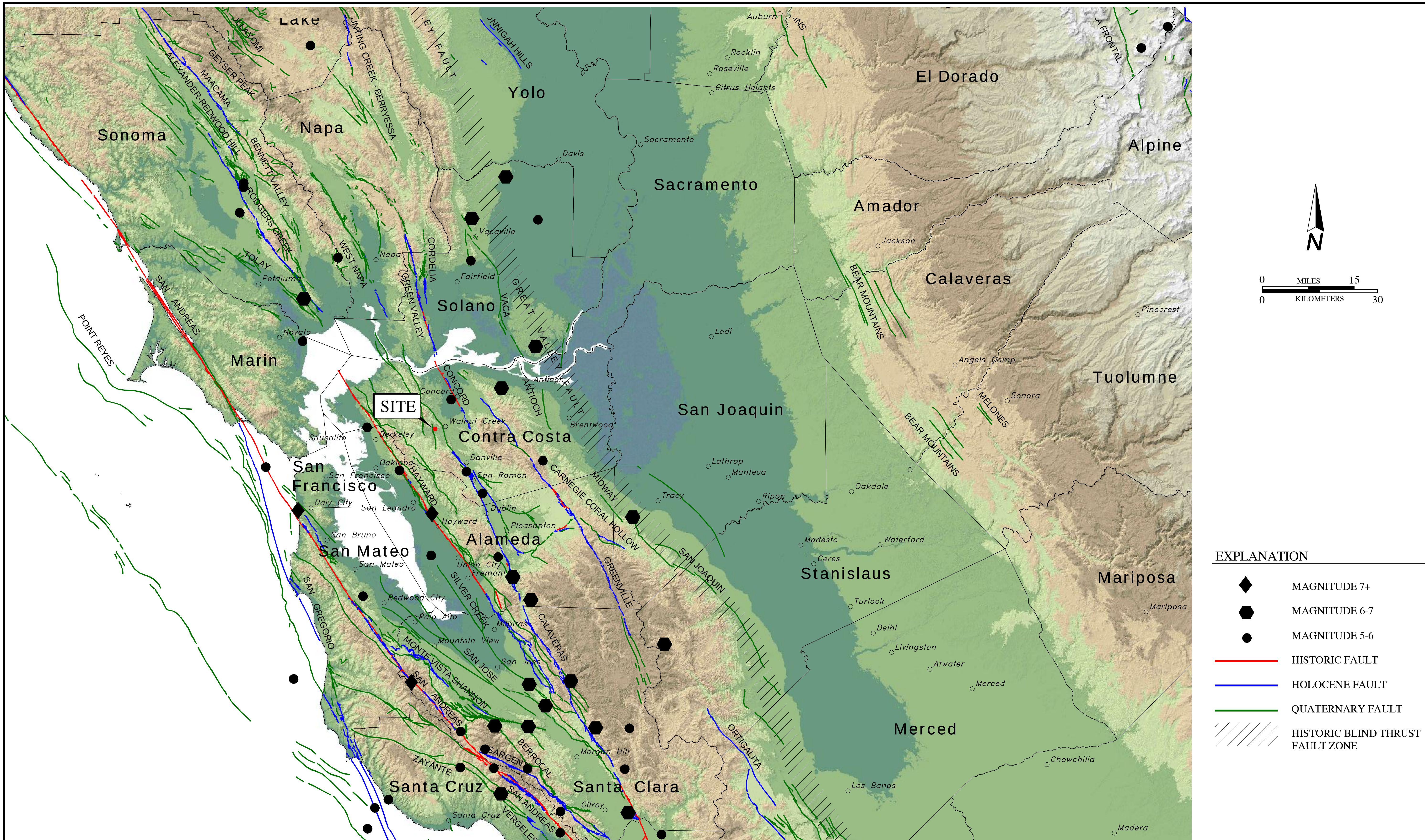

BASE MAP SOURCE:
COLOR HILLSHADE IMAGE BASED ON THE NATIONAL ELEVATION DATASET (NED) AT 30 METER RESOLUTION
U.S.G.S. QUATERNARY FAULT DATABASE, NOVEMBER, 2010
U.S.G.S. HISTORIC EARTHQUAKE DATABASE (1800–2000)

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REGIONAL FAULTING AND SEISMICITY
THE TERRACES OF LAFAYETTE
LAFAYETTE, CALIFORNIA

PROJECT NO.: 9181.200.000	FIGURE NO
SCALE: AS SHOWN	4
DRAWN BY: SRP	CHECKED BY: PCG

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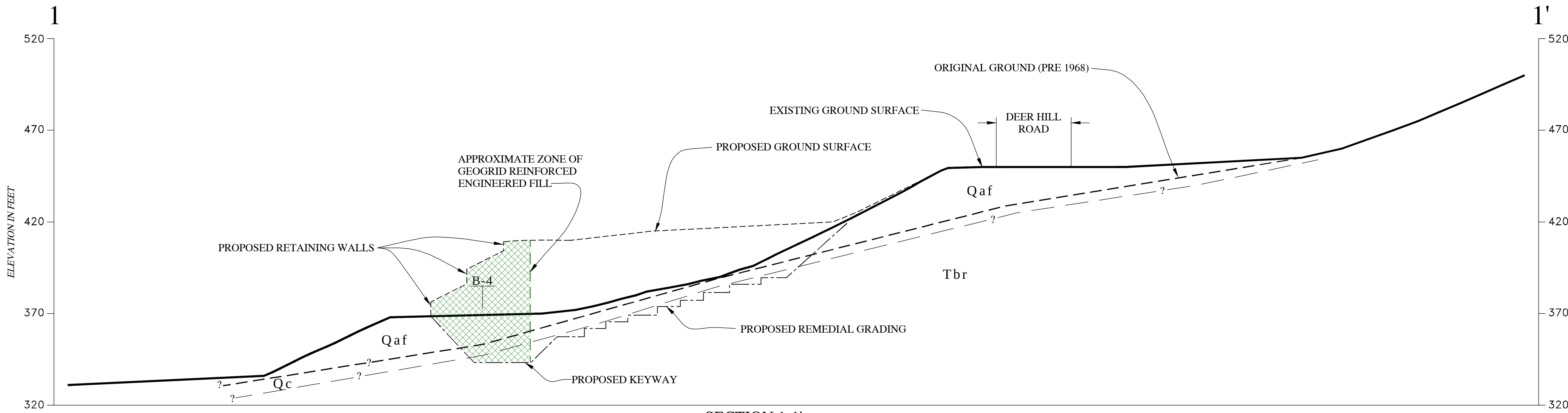
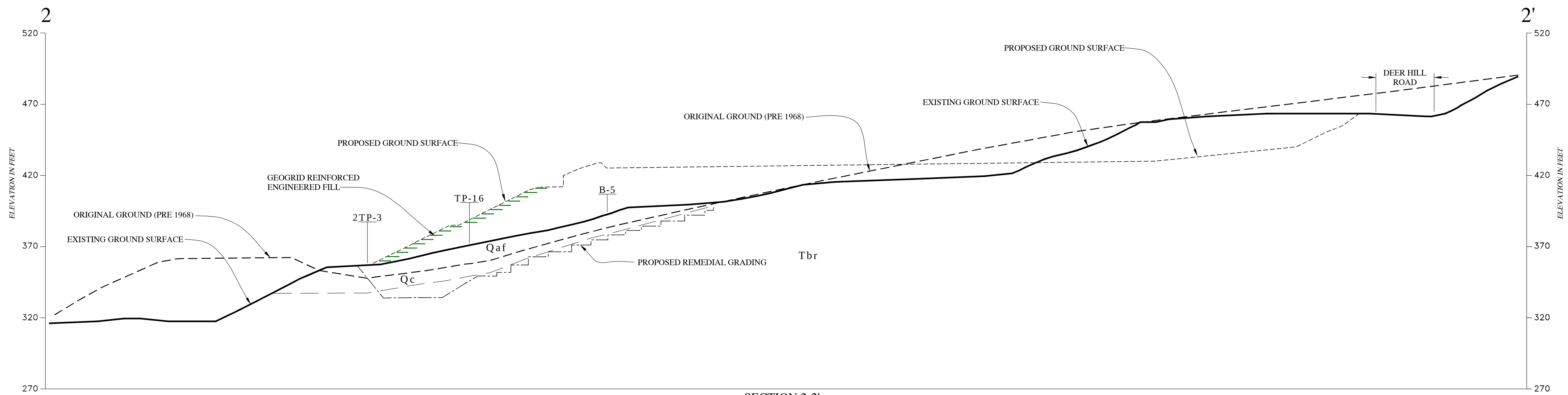


SECTION 1-1'
1"=50'



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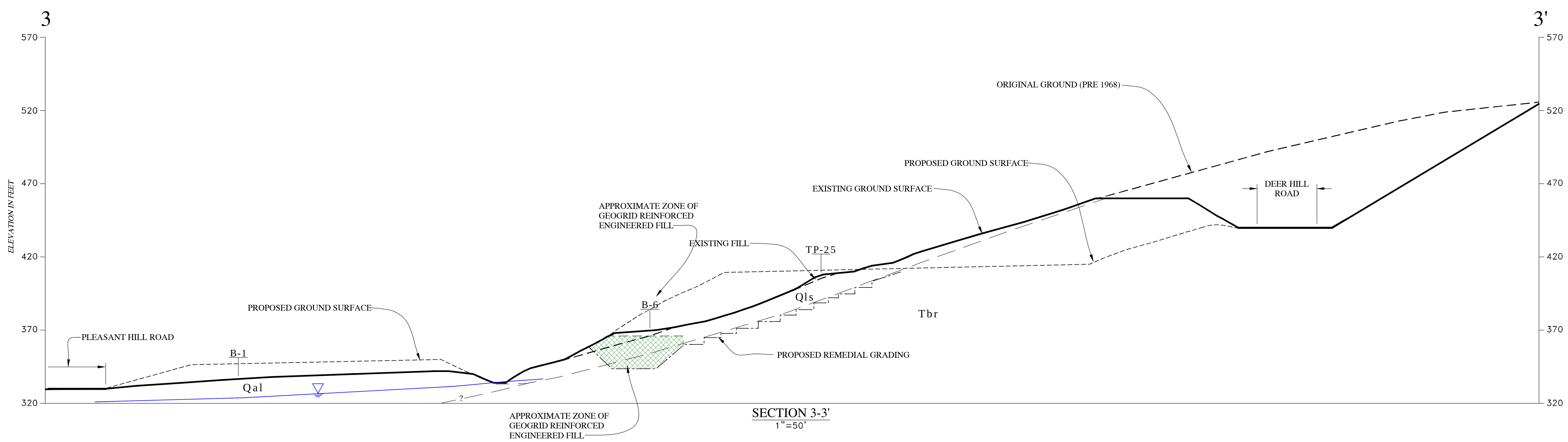

SECTION 3-3'
1"=50'

EXPLANATION

Qaf EXISTING FILL
Qc COLLUVIUM
Qls LANDSLIDE
Tbr BRIONES FORMATION

ENGEO Expect Excellence	CROSS SECTIONS THE TERRACES OF LAFAYETTE LAFAYETTE, CALIFORNIA	PROJECT NO.: 9181.200.000 SCALE: AS SHOWN DRAWN BY: SRP CHECKED BY: PCG	FIGURE NO. 5

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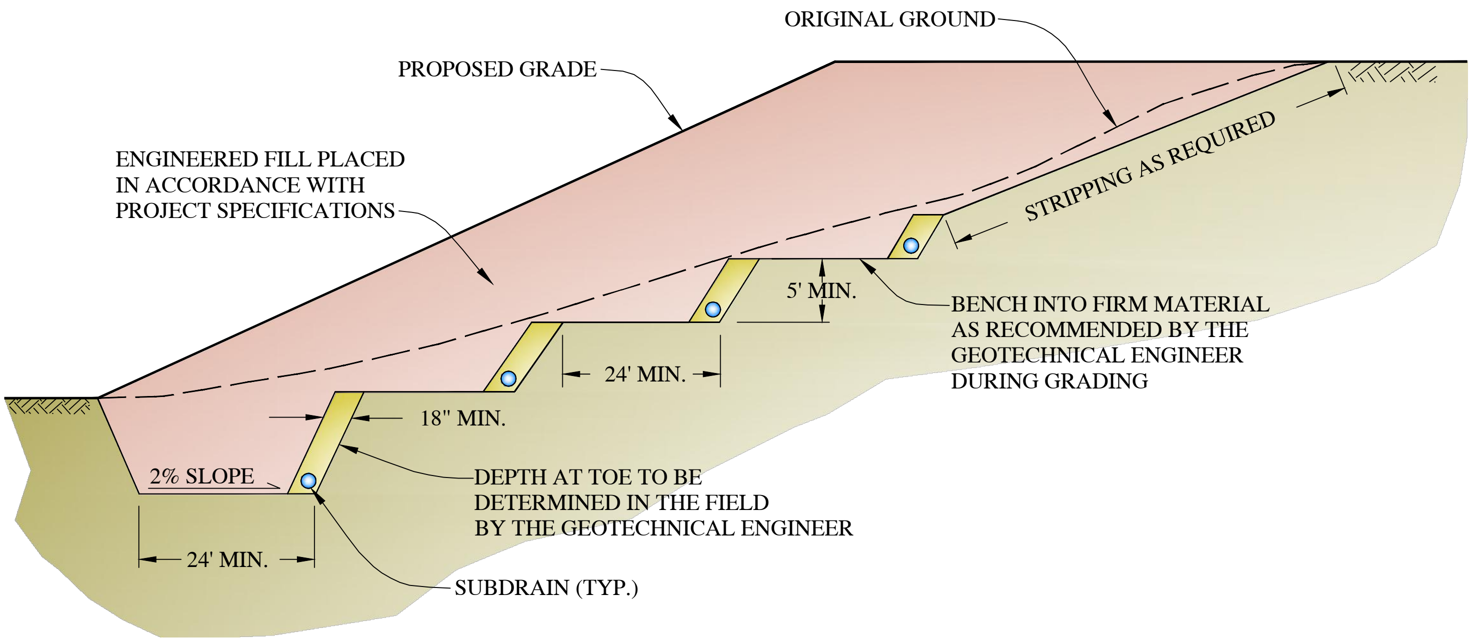

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TYPICAL KEYWAY DETAIL

THE TERRACES OF LAFAYETTE

LAFAYETTE, CALIFORNIA

PROJECT NO.: 9181.200.000	FIGURE NO. 6
SCALE: NO SCALE	
DRAWN BY: SRP	CHECKED BY: PCG

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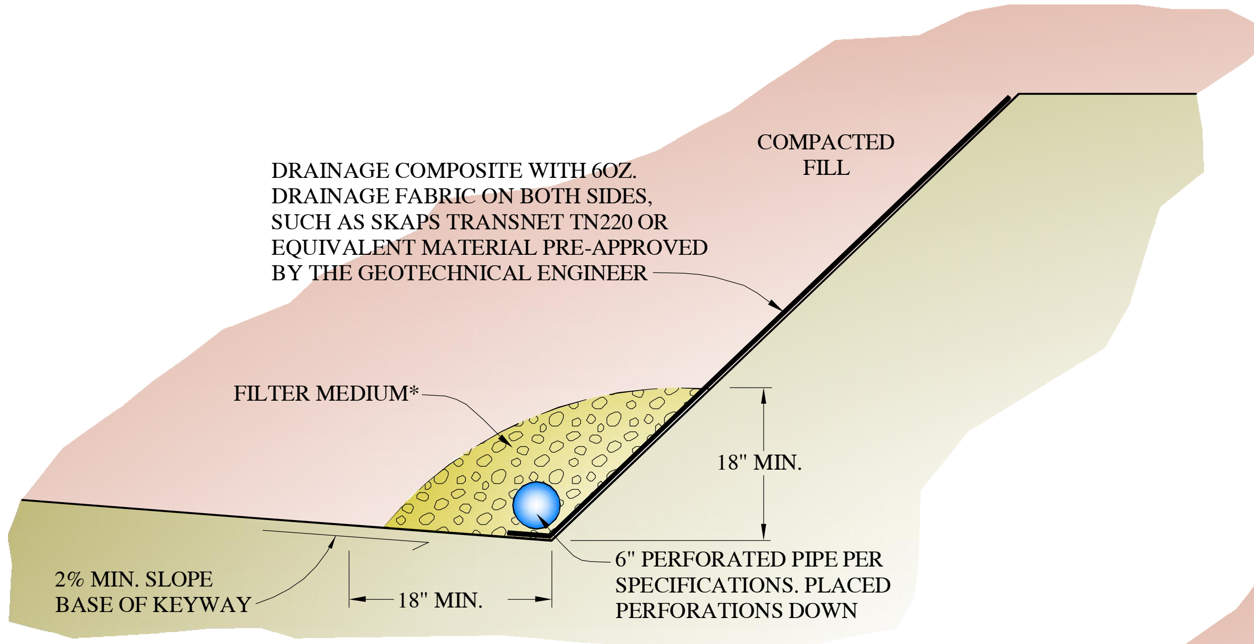
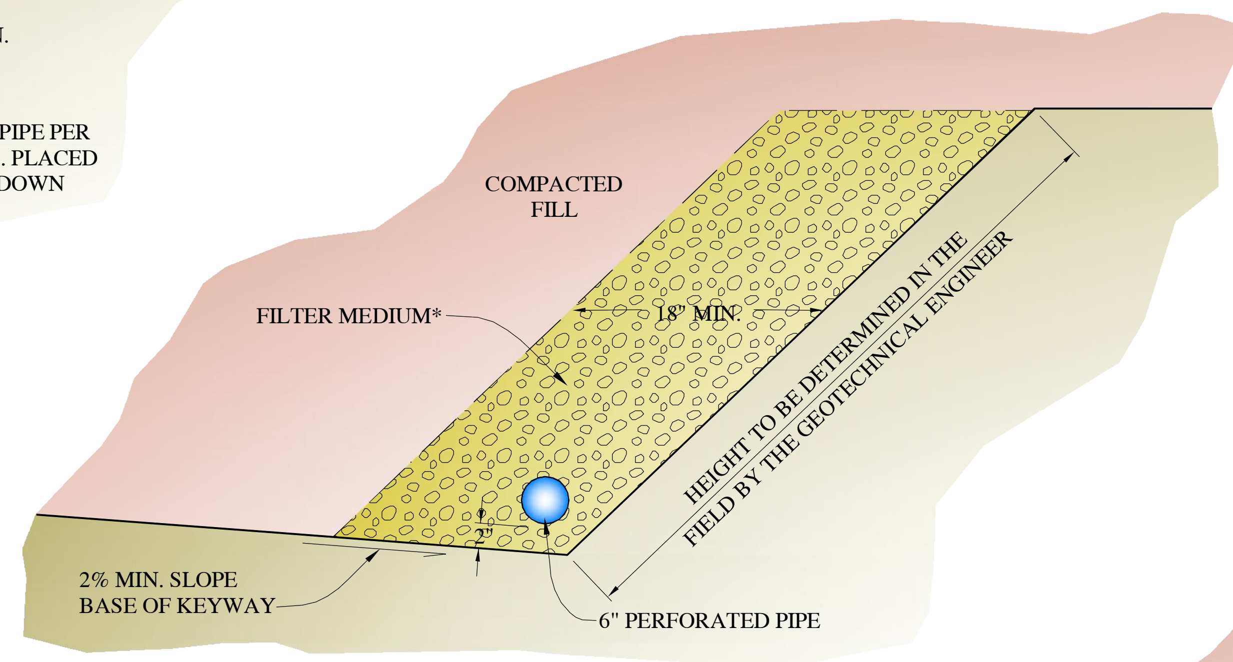
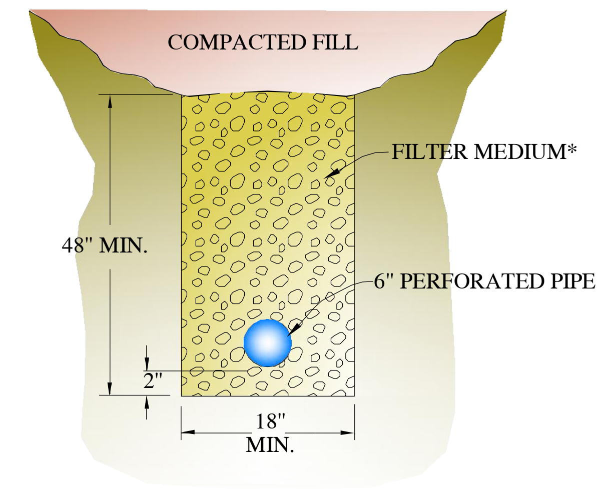


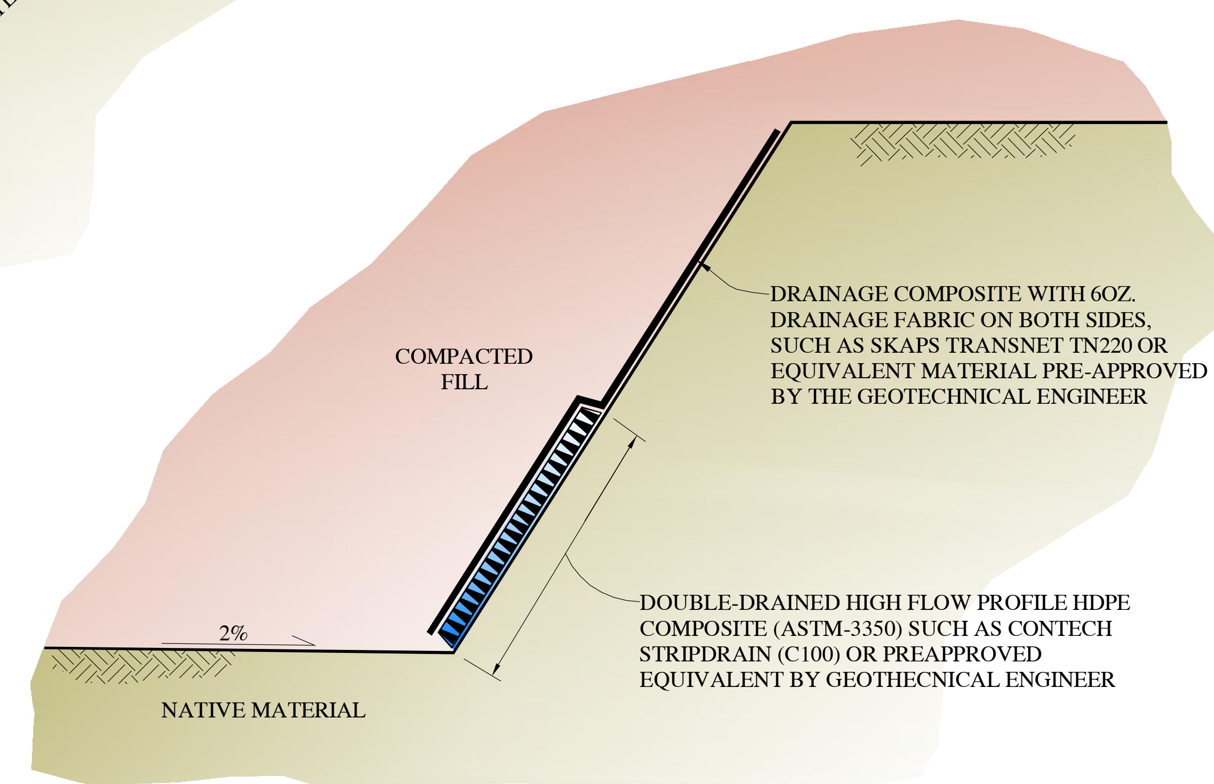
KEYWAY SUBDRAIN - OPTION 1



KEYWAY SUBDRAIN - OPTION 2



SWALE SUBDRAIN



ALTERNATE KEYWAY SUBDRAIN - OPTION 3

(FOR DEPTHS LESS THAN 30 FEET)

*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*

ENGEO Expect Excellence

TYPICAL SUBDRAIN DETAILS

THE TERRACES OF LAFAYETTE

LAFAYETTE, CALIFORNIA

PROJECT NO.: 9181.200.000

SCALE: NO SCALE

DRAWN BY: SRP | CHECKED BY: PCG

FIGURE NO. 7

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APPENDIX A

Boring Logs
Test Pit Logs
(ENGEO 2014 and 2011)

APPENDIX A

ENGEO Expect Excellence		TEST PIT LOG
Terraces at Lafayette Lafayette, California 9181.200.000		Logged By: J. White Logged Date: 3/7/2014 and 3/10/2014
Test Pit Number	Depth (Feet)	Description
2-TP1	0 – 5	SANDY CLAY (CL), brown, stiff, moist to wet at fence line, with fine to coarse gravel and rock fragments up to 6 inches, fine to coarse grained sand. (Fill)
	5 – 6	FAT CLAY (CH), dark brown, very stiff, moist, with gravel, roots at 5 feet, PP=3.0. (Fill)
	6 – 9.5	CLAYEY SAND (SC), brown, dense, moist, fine to coarse sand, with fine to coarse gravel and rock fragments, sandstone cobbles and few boulders up to 2 feet across. (Fill)
	9.5 – 10	SANDY CLAY (CL), brown to very dark brown, stiff, moist, with fine gravel to cobbles/ rock fragments. (Fill)
	10 – 15	CLAYEY SAND (SC), dark brown to brownish gray, dense, moist, minor seepage at 10 feet, fine to coarse grained sand, gravel to boulders up to 2 feet across, layering indicative of fill. (Fill)
	15 – 24	LEAN CLAY with sand (CL), black, stiff to very stiff, moist, few coarse gravels and rock fragments, organic odor, few rootlets, minor grass-line observed at 15 feet (contact); PP=2.5 at 16 feet, 3.0 at 17 feet, 3.5 at 18 feet, 3.0 at 19 feet, 3.0 at 21 feet, 3.5 at 23 feet, 3.5 at 24 feet. At 21 feet, becomes very dark brown, very stiff, some minor pedogenic development, few clay filled tubular pores, few fine gravels. (Qc)
	24 – 25	CLAYEY SAND (SC), brown, very dense, moist, siltstone rock fragments. (Residual Soil)
	25 – 26	SILTSTONE, brown to gray, weak, closely fractured, moderately weathered, iron staining along fracture surfaces. (Bedrock) Bottom at 26 feet
2-TP2	0 – 3	SANDSTONE, brown, medium strong to strong, closely fractured, moderately weathered, iron staining along fractures. (Bedrock) Bottom at 3 feet

TEST PIT LOG

Terraces at Lafayette Lafayette, California 9181.200.000	Logged By: J. White Logged Date: 3/7/2014 and 3/10/2014

Test Pit Number	Depth (Feet)	Description
2-TP3	0 – 9	SANDY CLAY (CL), very dark brown mixed with dark brown, very stiff, moist, with fine to coarse gravel and rock fragments, few cobbles up to 10 inches. At 3 feet, becomes brown, with layers of clayey sand, layering indicative of fill. At 5 feet, dark brown. At 9 feet, wet, medium stiff, seepage from sidewalls. (Fill)
	9 – 19	SANDY CLAY (CL), black, stiff, moist, few sandstone fragments, minor organics. At 13 feet, very stiff. At 15 feet, becomes dark brown, very stiff, moist, some blocky pedogenic structure, minor clay films on gravels. (Qc)
	19 – 20	CLAYEY SAND (SC), brown to olive brown, dense, moist, siltstone rock fragments. (Residual Soil)
	20 – 22	Interbedded SILTSTONE and SANDSTONE, brown to olive brown, extremely weak, very closely fractured, moderately weathered. (Bedrock) Logged from surface after 6 feet due to caving in. Bottom at 22 feet.

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
Expect Excellence

ENGEO INCORPORATED

LOG OF BORING B-1

Geotechnical Exploration
The Terraces of Lafayette
Lafayette, California
9181.100.000

DATE DRILLED: 6/14/2011
HOLE DEPTH: Approx. 51½ ft.
HOLE DIAMETER: 4.0 in.
SURF ELEV (msl): Approx. 337 ft.

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

Depth in Feet	Depth in Meters	DESCRIPTION	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)	Unconfined Strength (tsf) *field approx
		SILTY SAND (SM) mixed with mulch. (fill)									
	1	SILTY CLAY (CL), very dark brown, stiff, moist, few fine gravels.		19	59	18	41				1.5*
	2	SANDY CLAY (CL), very dark grayish brown, very stiff, moist, with fine gravel ad few sandstone fragments.		53					23.4	101.2	3.5*
		Brown mottled with gray, with fine gravel and 1/8 to 1/4 inch sandstone fragments, few manganese nodules.									
10	3			46					23.9	98.9	3.5*
	4		▽								
	5	Becomes dark yellowish brown, stiff, wet.		32					26.7	97	2*
20	6	Harder drilling. Gray and yellowish brown, medium stiff, moist, with subrounded 1/4 to 1 inch sandstone fragments.		39							
	7										
	8			16							
		Becomes dark yellowish brown, very moist, fine to medium grained sand. Becomes loose.		9	46	16	30	65	30.5		
30	9	Becomes medium dense, 1/4-inch sandstone fragments		14					29.8		
	10										
	11	Same as above.		11					29.2		
40	12	Brown mottled with dark gray, very stiff, with fine gravel, some manganese.		16					27.7		2.5*
	13										
	14	Becomes stiff, few 1/16 to 1/8 inch sandstone fragments.		32							1.5*
50	15	Becomes very stiff, with subrounded to rounded sandstone fragments.		34							3.5*
		Bottom of boring at 51.5 feet, groundwater at 13 feet.									

LOG - GEOTECHNICAL 9181100000 GINT LOGS.GPJ ENGEO INC.GDT 8/18/11

ENGEO INCORPORATED

LOG OF BORING B-2

Geotechnical Exploration
The Terraces of Lafayette
Lafayette, California
9181.100.000

DATE DRILLED: 6/14/2011
HOLE DEPTH: Approx. 31½ ft.
HOLE DIAMETER: 4.0 in.
SURF ELEV (msl): Approx. 351 ft.

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

Depth in Feet	Depth in Meters	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)	Unconfined Strength (tsf) *field approx
	1		SILTY CLAY (CL), very dark grayish brown, very stiff, moist, with fine gravel.			22					26.7	96	3*
	2		Mottled with brown, with sand.			38							3*
10	3		Dark brown mottled with olive gray, few 1/8-inch sandstone fragments, few manganese nodules.			42					24.7	100	2.75*
	4 5		Yellowish brown mottled with gray, with 1/8 to 1/4 inch sandstone fragments.		∇	34					26.5	100.4	3*
20	6 7		Same as above.			18					28.2	95.9	2.5*
	8		Increasing sand content.			26					26.8	97.6	2.5*
30	9		Same as above.			29					21.5	100.9	2.5*
			Bottom of boring at 31.5 feet, groundwater encountered at 14 feet.										

ENGEO INCORPORATED

LOG OF BORING B-3

Geotechnical Exploration
The Terraces of Lafayette
Lafayette, California
9181.100.000

DATE DRILLED: 6/15/2011
HOLE DEPTH: Approx. 20½ ft.
HOLE DIAMETER: 4.0 in.
SURF ELEV (msl): Approx. 462 ft.

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

Depth in Feet	Depth in Meters	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)	Unconfined Strength (tsf) *field approx
			SANDSTONE, bluish gray with brown, weak, closely fractured, moderately weathered, fine grained, some iron staining.										
	1												
	2					50/6"							
10	3		Becomes dark bluish gray.			60/3"							
	4												
	5		Same as above.			84/6"							
20	6		Same as above.			68/6"							
			Bottom of boring at 20.5 feet, no groundwater encountered.										

LOG OF BORING B-4

ENGEO INCORPORATED

Geotechnical Exploration
The Terraces of Lafayette
Lafayette, California
9181.100.000

DATE DRILLED: 6/15/2011
HOLE DEPTH: Approx. 28½ ft.
HOLE DIAMETER: 4.0 in.
SURF ELEV (msl): Approx. 366 ft.

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

Depth in Feet	Depth in Meters	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)	Unconfined Strength (tsf) *field approx
			SILTY CLAY (CL), very dark brown with dark yellowish brown, hard, moist, with 1/4 to 2 inch sandstone fragments. (fill)			50/6"							
	1										21.1	104.1	
	2												
			Increasing sand content, few bluish gray sandstone fragments.			50/4"							
10	3												
	4		Same as above.			50/6"					21.1	101.5	
	5		SILTY CLAY (CL), very dark brown, very stiff, moist, with fine gravel and sandstone fragments.										
20	6					28					25.5	98.2	3*
			Harder drilling.										
	7		SANDSTONE, bluish gray with brown, weak, closely fractured, moderately weathered, some iron staining, fine grained.			50/6"							
	8												
			Same as above.			50/6"							
			Bottom of boring at 28.5 feet, no groundwater encountered.										

LOG - GEOTECHNICAL 9181100000 GINT LOGS.GPJ ENGEO INC.GDT 8/18/11

ENGEO INCORPORATED

LOG OF BORING B-5

Geotechnical Exploration
The Terraces of Lafayette
Lafayette, California
9181.100.000

DATE DRILLED: 6/15/2011
HOLE DEPTH: Approx. 19½ ft.
HOLE DIAMETER: 4.0 in.
SURF ELEV (msl): Approx. 397 ft.

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

Depth in Feet	Depth in Meters	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)	Unconfined Strength (tsf) *field approx
			SILTY SAND (SM), dark yellowish brown to yellowish brown, dense, moist, with 1/4 to 2 inch sandstone fragments.										
	1												
						34					24.2	97.3	
	2												
10	3		SILTY CLAY (CL), very dark gray, stiff, moist, with fine gravel, few rootlets.			24					29.1	92.8	2*
	4												
			SANDSTONE, dark bluish gray, weak, closely fractured, moderately weathered, medium grained.			50/6"					23.8	100.3	
	5												
			Becomes very dark brown, medium strong.			50/6"							
			Bottom of boring at 19.5 feet, no groundwater encountered.										

LOG - GEOTECHNICAL 9181100000 GINT LOGS.GPJ ENGEO INC.GDT 8/18/11

LOG OF BORING B-6

ENGEO INCORPORATED

Geotechnical Exploration
The Terraces of Lafayette
Lafayette, California
9181.100.000

DATE DRILLED: 6/15/2011
HOLE DEPTH: Approx. 25½ ft.
HOLE DIAMETER: 4.0 in.
SURF ELEV (msl): Approx. 370 ft.

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

Depth in Feet	Depth in Meters	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)	Unconfined Strength (tsf) *field approx
			Asphalt.										
			Aggregate base.										
	1		SILTY CLAY (CL), very dark brown mixed with yellowish brown, very stiff, moist. (fill)										
	2		SILTY CLAY (CL), very dark brown with reddish brown mottles, very stiff, moist, with fine gravel, few rootlets.			39					25.3	95.6	3.5*
10	3		Brown, hard, moist, with fine sand, few roots.			62							4.5*
	4												
	5		SANDSTONE, olive brown with yellowish brown, extremely weak, closely fractured, iron staining, fine to medium grained.			66							
20	6		Interbedded SILTSTONE and SANDSTONE, olive brown, extremely weak, closely fractured, highly weathered, iron staining.			40							
	7		SANDSTONE, light gray, weak, closely fractured, fine to medium grained.			50/6"							
			Bottom of boring at 25.5 feet, no groundwater encountered.										

ENGEO INCORPORATED	TEST PIT LOG	
Terraces at Lafayette Lafayette, California 9181.100.000	Logged By: J. White Logged Date: 6/1/11 to 6/2/11	
Test Pit Number	Depth (Feet)	Description
TP-1	0 – 2	SILTY CLAY (CL), very dark gray, very stiff, with sand and fine gravel, rootlets in upper 6 inches.
	2 – 4 ½	SANDSTONE, yellowish brown and gray, very weak, closely fractured, thinly bedded, iron staining along fracture surfaces, few siltstone interbeds towards the east end of trench. Bedding from west to east - N81E/40S, N70E/34S, N80W/30S.
TP-2	0 – 2	SILTY CLAY (CL), very dark gray, very stiff, with sand and fine gravel, few sandstone fragments.
	2 – 3	SANDY CLAY (CL), very dark brown, very stiff, with fine gravel and carbonate nodules.
	3 – 5 ½	SANDSTONE, olive brown and brown, weak, closely fractured, thinly bedded, highly weathered, some iron staining.
TP-3	0 – 4	SANDSTONE, yellowish brown and gray, weak to medium strong, closely fractured, moderately weathered, iron staining along fracture surfaces. Bedding N60W/50S.
TP-4	0 – 3	SANDY GRAVEL (GM), dense, dry, rootlets, few silty clay blocks, bedrock derived fill. (fill).
	3 – 4	SILTY CLAY (CL), very dark gray, very stiff, moist, with fine gravel and sandstone fragments, few rootlets.
	4 – 6 ½	SANDSTONE, dark yellowish brown, weathers to dark reddish brown, very weak, closely fractured, thinly bedded, iron staining along fracture surfaces.

TEST PIT LOG

ENGEO INCORPORATED	TEST PIT LOG
Terraces at Lafayette Lafayette, California 9181.100.000	Logged By: J. White Logged Date: 6/1/11 to 6/2/11

Test Pit Number	Depth (Feet)	Description
TP-5	0 – 4	SILTY GRAVEL (GM), dark yellowish brown, very dense, moist, with sand and sandstone fragments, bedrock derived fill, layering indicative of fill. (fill).
	4 – 5	SANDY CLAY (CL), very dark brown, very stiff, moist, with sandstone fragments.
	5 – 7	SANDSTONE, light gray and yellowish brown, weak, closely fractured, thickly bedded, highly weathered, some iron staining.
TP-6	0 – 1 ½	SANDY CLAY (CL), very dark brown, stiff, moist, with sandstone fragments.
	1 ½ - 5	SANDSTONE, gray and reddish brown, weak, closely fractured, thinly bedded, highly weathered, abundant iron staining. Bedding N62E/49S.
TP-7	0 – 9	SILTY GRAVEL (GM), yellowish brown, dense, moist, bedrock derived fill, few sandstone blocks over 6-inches, horizontal layering indicative of fill. (fill).
	9 – 12	SANDY CLAY (CL), bluish gray mixed with brown, very stiff, moist, with sandstone fragments. (fill).
TP-8	0 – 6	CLAYEY GRAVEL (GC), dark yellowish brown, dense, wet, sandstone blocks and fragments, water began seeping in at 4-feet and filled bottom of pit.

TEST PIT LOG

Terraces at Lafayette Lafayette, California 9181.100.000		Logged By: J. White Logged Date: 6/1/11 to 6/2/11
Test Pit Number	**Depth (Feet)**	**Description**
TP-9	0 – 2	SILTY CLAY (CL), very dark gray, very stiff, moist, with fine gravel and sandstone fragments, few rootlets.
	2 – 4	SANDSTONE, dark yellowish brown and reddish brown, weak, closely fractured, thinly bedded, highly weathered, abundant iron staining.
TP-10	0 – 3	SANDSTONE, brown, medium strong, closely fractured, thinly bedded, iron staining, highly weathered, coarse grained. Bedding N59W/39S
TP-11	0 – 1	Loose mixture of asphalt and aggregate base. (fill).
	1 – 3	SANDSTONE, brown and dark yellowish brown, medium strong, closely fractured, thickly bedded, highly weathered.
TP-12	0 – 3	SILTSTONE, brown, very weak, very closely fractured, very thinly bedded, highly weathered.
	3 – 6	SILTSTONE, bluish gray, medium strong, closely fractured, thickly bedded, moderately weathered.
TP-13	0 – 5	SILTY CLAY and SANDSTONE mixture, dark brown and yellowish brown, dense, moist, layering indicative of fill. (fill).
	5 – 8	SILTY CLAY (CL), very dark brown, very stiff, moist, with fine gravel and sandstone fragments.
	8 – 11	SILTSTONE, dark olive brown, very weak, very closely fractured, thinly bedded, some iron staining.

TEST PIT LOG

ENGEO INCORPORATED	TEST PIT LOG	
Terraces at Lafayette Lafayette, California 9181.100.000		Logged By: J. White Logged Date: 6/1/11 to 6/2/11
Test Pit Number	**Depth (Feet)**	**Description**
TP-14	0 – 1	Loose mixture of asphalt and aggregate base. (fill).
	1 – 4	SANDY CLAY and SANDSTONE mixture, dense, moist, horizontal layering indicative of fill.
	4 – 6	Interbedded SANDSTONE and SILTSTONE and shale, very weak, very closely fractured, very thinly bedded to laminated, highly weathered, abundant iron staining. Bedding N60E/34S
TP-15	0 – 15	SANDY CLAY and SANDSTONE mixture, dense, moist, horizontal layering indicative of fill. (fill)
	15 – 17	SILTY CLAY (CL), very dark brown, very stiff, moist, with fine gravel and siltstone fragments.
	17 – 19	SILTSTONE, olive brown, very weak, closely fractured, thinly bedded, highly weathered, iron staining.
TP-16	0 – 13	SILTY CLAY and SANDSTONE mixture, dense, moist, horizontal layering indicative of fill. (fill)
	13 – 17	SANDY CLAY (CL), very dark brown, very stiff, moist, with fine gravel and siltstone fragments.
	17 – 20 (maximum depth)	SANDY CLAY (CL), dark olive brown and brown, very stiff, very moist, few dark brown mottles, with sandstone fragments.

ENGEO INCORPORATED	# TEST PIT LOG
Terraces at Lafayette Lafayette, California 9181.100.000	Logged By: J. White Logged Date: 6/1/11 to 6/2/11

Test Pit Number	Depth (Feet)	Description
TP-17	0 – 4	SILTY CLAY and SANDSTONE mixture, dense, moist, horizontal layering indicative of fill. (fill)
	4 – 7	Interbedded SANDSTONE and SILTSTONE, brown with olive brown, very weak, very closely fractured, thinly bedded, highly weathered. Bedding N30E/ 59S
TP-18	0 – 3	Interbedded SANDSTONE and SILTSTONE, brown with olive brown, very weak, very closely fractured, thinly bedded, highly weathered.
TP-19	0 – 5	SILTY CLAY and SANDSTONE mixture, dense, moist, horizontal layering indicative of fill. (fill)
	5 – 7	Interbedded SANDSTONE and SILTSTONE, brown with olive brown, very weak, very closely fractured, thinly bedded, highly weathered.
TP-20	0 – 2	SILTY CLAY (CL), very dark gray, very stiff, moist, with sandstone fragments.
	2 – 7	SILTSTONE, olive brown, extremely weak, upper 2 feet crushed, very closely fractured, thinly bedded, highly weathered.
TP-21	0 – 6	Interbedded SILSTONE and SANDSTONE, brown, weak, very closely fractured, thinly bedded, highly weathered, iron staining.

TEST PIT LOG

Terraces at Lafayette Lafayette, California 9181.100.000		Logged By: J. White Logged Date: 6/1/11 to 6/2/11
Test Pit Number	**Depth (Feet)**	**Description**
TP-22	0 – 2	SANDY CLAY and SANDSTONE mixture, dense, moist, horizontal layering indicative of fill, few blocks over 6 inches. (fill)
	2 – 6	SILTY CLAY (CL), very dark brown, becomes dark brown at 4 feet, very stiff, moist, with fine gravel and siltstone fragments.
	6 – 9	SILTSTONE, olive brown, very weak, closely fractured, thinly bedded, highly weathered, iron staining along fracture surfaces.
TP-23	0 – 2	SANDY CLAY and SILTSTONE/ SANDSTONE mixture, dense, moist, horizontal layering indicative of fill. (fill)
	2 – 3	SILTY CLAY (CL), very dark brown, very stiff, moist, with sandstone fragments.
	3 – 5	SANDSTONE, yellowish brown, weak, closely fractured, thickly bedded, iron staining along fracture surfaces, difficult to excavate.
TP-24	0 – 3	Interbedded SILTSTONE and SANDSTONE, weak, closely fractured, upper 1 foot is crushed, very thinly bedded, highly weathered, iron staining. Bedding N62W/ 55S.
TP-25	0 – 2	SANDY CLAY and SANDSTONE mixture, dense, moist, horizontal layering indicative of fill. (fill)
	2 – 13	SILTY CLAY (CL), very dark gray, becomes dark olive gray at 8 feet, very stiff, moist, well developed ped surfaces.
TP-26	0 – 3	SILTY CLAY (CL), very dark brown, very stiff, moist, with fine gravel.
	3 – 6	SILTY CLAY (CL), dark olive brown, very stiff, very moist, few dark brown mottles, few sandstone fragments.

ENGEO INCORPORATED	TEST PIT LOG	
Terraces at Lafayette Lafayette, California 9181.100.000	Logged By: J. White Logged Date: 6/1/11 to 6/2/11	

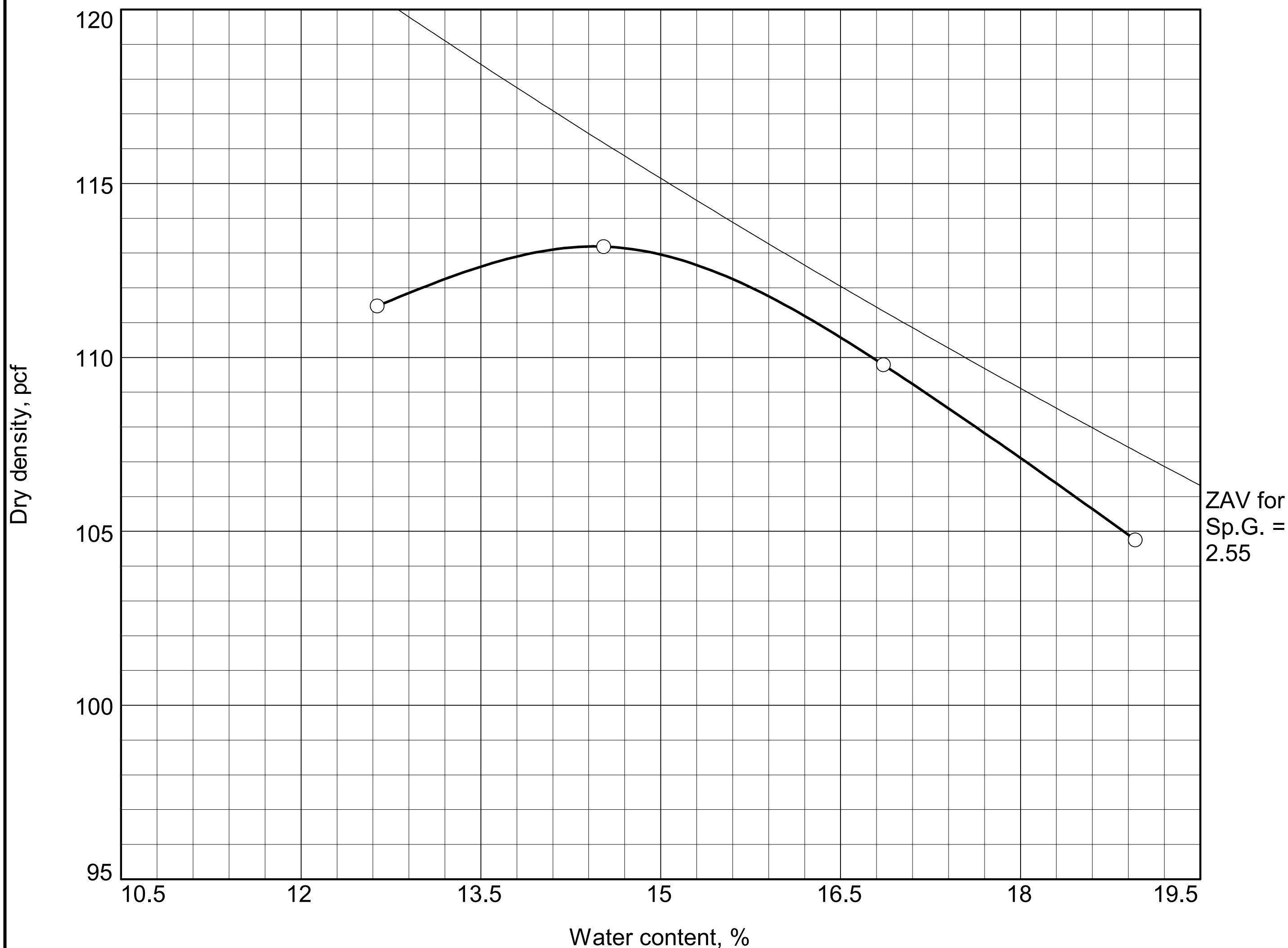
Test Pit Number	Depth (Feet)	Description
	6 – 9	SANDSTONE, dark yellowish brown, weak closely fractured, thinly bedded, highly weathered, coarse grained. Bedding N71W/64S.
TP-27	0 – 3	Interbedded SANDSTONE and SILTSTONE, reddish brown and olive brown, weak, closely fractured, thinly bedded, highly weathered, iron staining.
TP-28	0 – 1	SANDSTONE, Brown, very closely fractured, highly weathered, roots.
	1 – 4	Interbedded SILTSTONE and SANDSTONE, brown, weak, closely fractured, thickly bedded, highly weathered.
TP-29	0 – 2	SANDSTONE, brown to bluish gray at 2 feet, medium strong, closely fractured, thickly bedded, highly weathered to freshly weathered at bottom, difficult to excavate.
TP-30	0 – 2 ½	SANDSTONE, brown and gray, medium strong, closely fractured, thickly bedded, moderately weathered, iron staining.

APPENDIX B

Laboratory Analysis
(ENGEO 2014 and 2011)

APPENDIX B

COMPACTION TEST REPORT For Curve No. 2-TP2



Test specification: ASTM D 1557-07 Method A Modified

Elev/ Depth	Classification		Nat. Moist.	Sp.G.	LL	PI	% > #4	% < No.200
	USCS	AASHTO						

TEST RESULTS	MATERIAL DESCRIPTION
Maximum dry density = 113.2 pcf Optimum moisture = 14.5 %	See exploration logs

Project No. 9181.200.000 **Client:** O'Brien Land Company, LLC

Project: Terraces of Lafayette Eng Consult

Date: 3.13.14

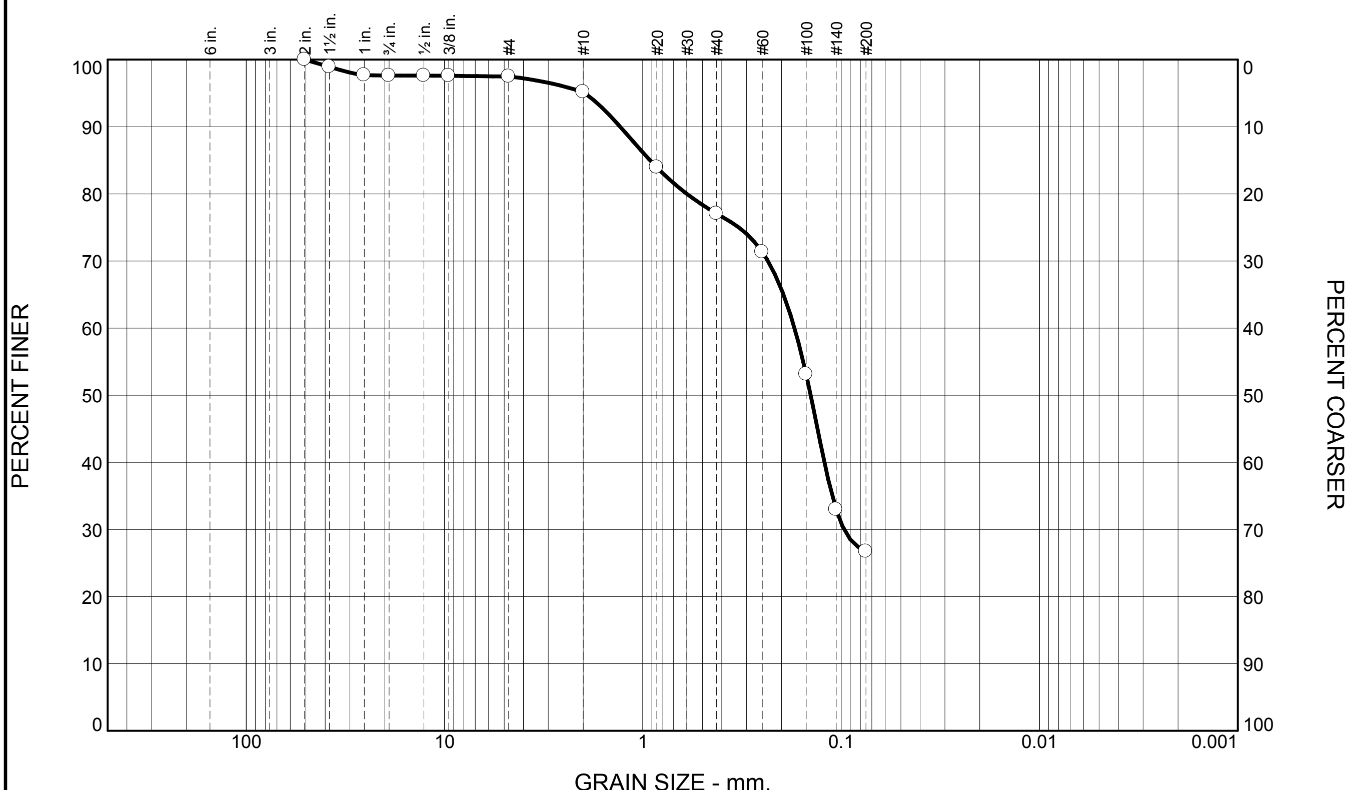
○ **Location:** TP2 **Sample Number:** 2-TP2

Remarks:

ENGEO INCORPORATED

Tested By: JL/RB **Checked By:** DS

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	2.4	0.1	2.3	18.1	50.4	26.7	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
2	100.0		
1 1/2	98.9		
1	97.7		
3/4	97.6		
1/2	97.6		
3/8	97.6		
#4	97.5		
#10	95.2		
#20	83.9		
#40	77.1		
#60	71.3		
#100	53.1		
#140	33.0		
#200	26.7		

* (no specification provided)

Soil Description

See exploration logs

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 1.3010 D_{85}= 0.9193 D_{60}= 0.1718
D_{50}= 0.1424 D_{30}= 0.0968 D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= AASHTO=

Remarks

Sample Number: 2-TP2@ 3 **Depth:** 3

Date: 03/17/14

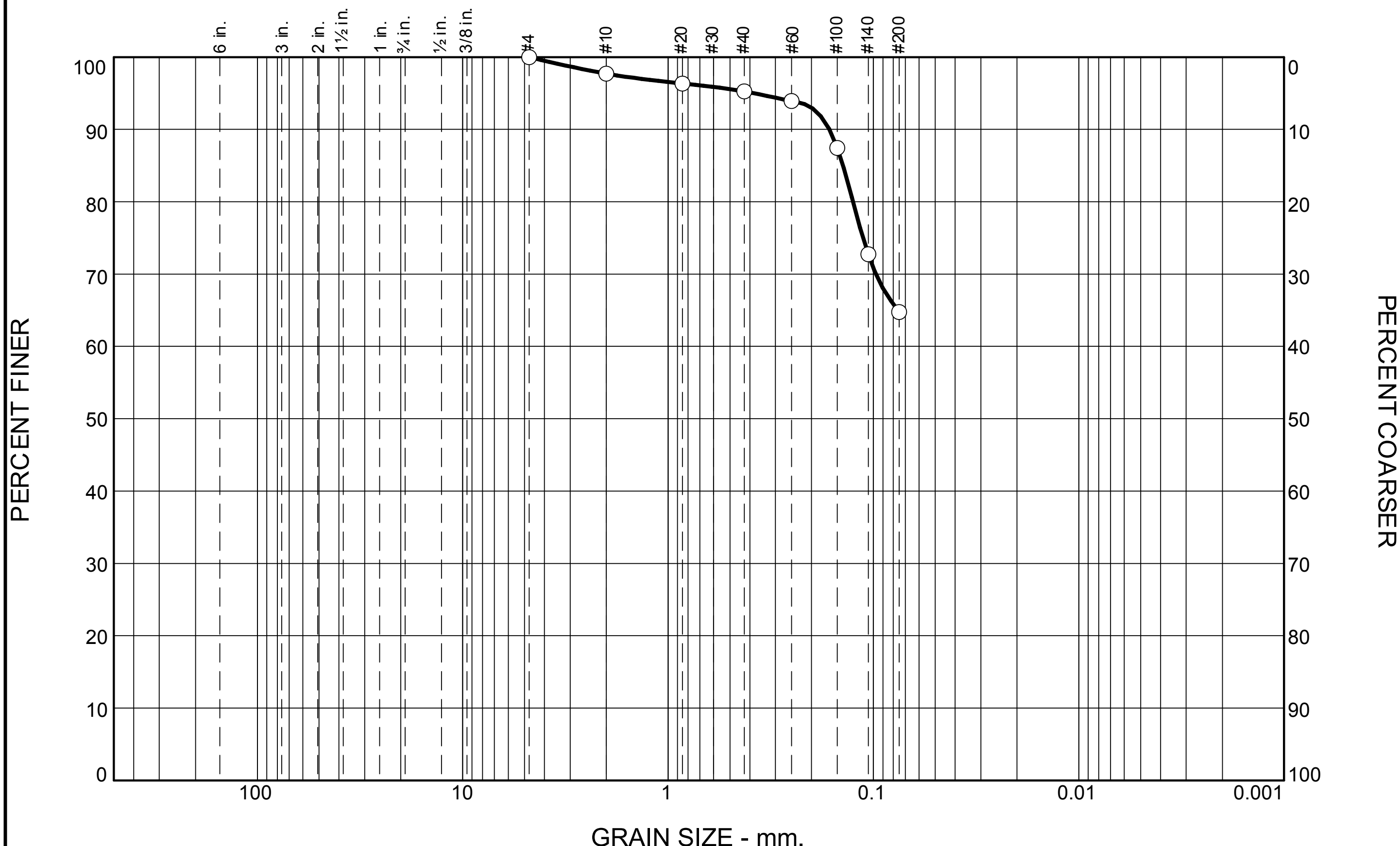
ENGEO INCORPORATED

Client: O'Brien Land Company, LLC
Project: Terraces of Lafayette Eng Consult

Project No: 9181.200.000

Tested By: JAL **Checked By:** GC

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	0.0	2.3	2.5	30.4	64.8	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#4	100.0		
#10	97.7		
#20	96.3		
#40	95.2		
#60	93.9		
#100	87.4		
#140	72.7		
#200	64.8		

* (no specification provided)

Soil Description

See exploration logs

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 0.1639 D_{85}= 0.1407 D_{60}=
D_{50}= D_{30}= D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= AASHTO=

Remarks

GS: ASTM D422

Sample Number: 2-TP3 @ 5 **Depth:** 5.0 feet

Date: 03/21/14

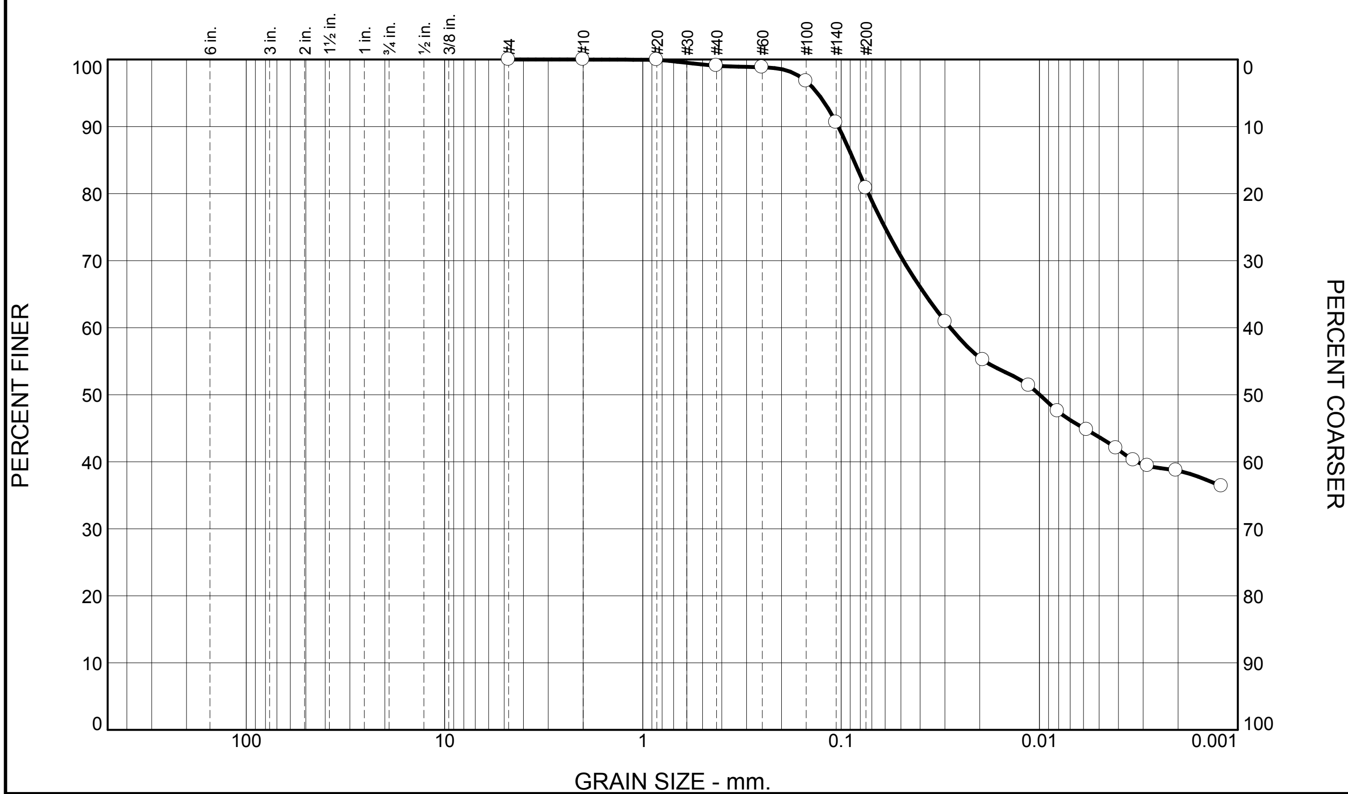
ENGEO INCORPORATED

Client: O'Brien Land Company, LLC
Project: Terraces of Lafayette Eng Consult

Project No: 9181.200.000

Tested By: JAL **Checked By:** DS

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	0.0	0.0	0.9	18.3	37.1	43.7

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#4	100.0		
#10	100.0		
#20	100.0		
#40	99.1		
#60	98.8		
#100	96.8		
#140	90.6		
#200	80.8		
0.0298 mm.	60.9		
0.0193 mm.	55.2		
0.0113 mm.	51.4		
0.0081 mm.	47.6		
0.0058 mm.	44.8		
0.0041 mm.	42.0		
0.0034 mm.	40.2		
0.0029 mm.	39.4		
0.0020 mm.	38.7		
0.0012 mm.	36.4		

* (no specification provided)

Soil Description

See exploration logs

Atterberg Limits

PL= 16 LL= 35 PI= 19

Coefficients

D_{90}= 0.1035 D_{85}= 0.0865 D_{60}= 0.0282
D_{50}= 0.0100 D_{30}= D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= CL AASHTO= A-6(14)

Remarks

GS: ASTM D422; PI: ASTM D4318; USCS: ASTM D2487

Sample Number: 2-TP1 @ 17 **Depth:** 17.0 feet

Date: 03/20/14

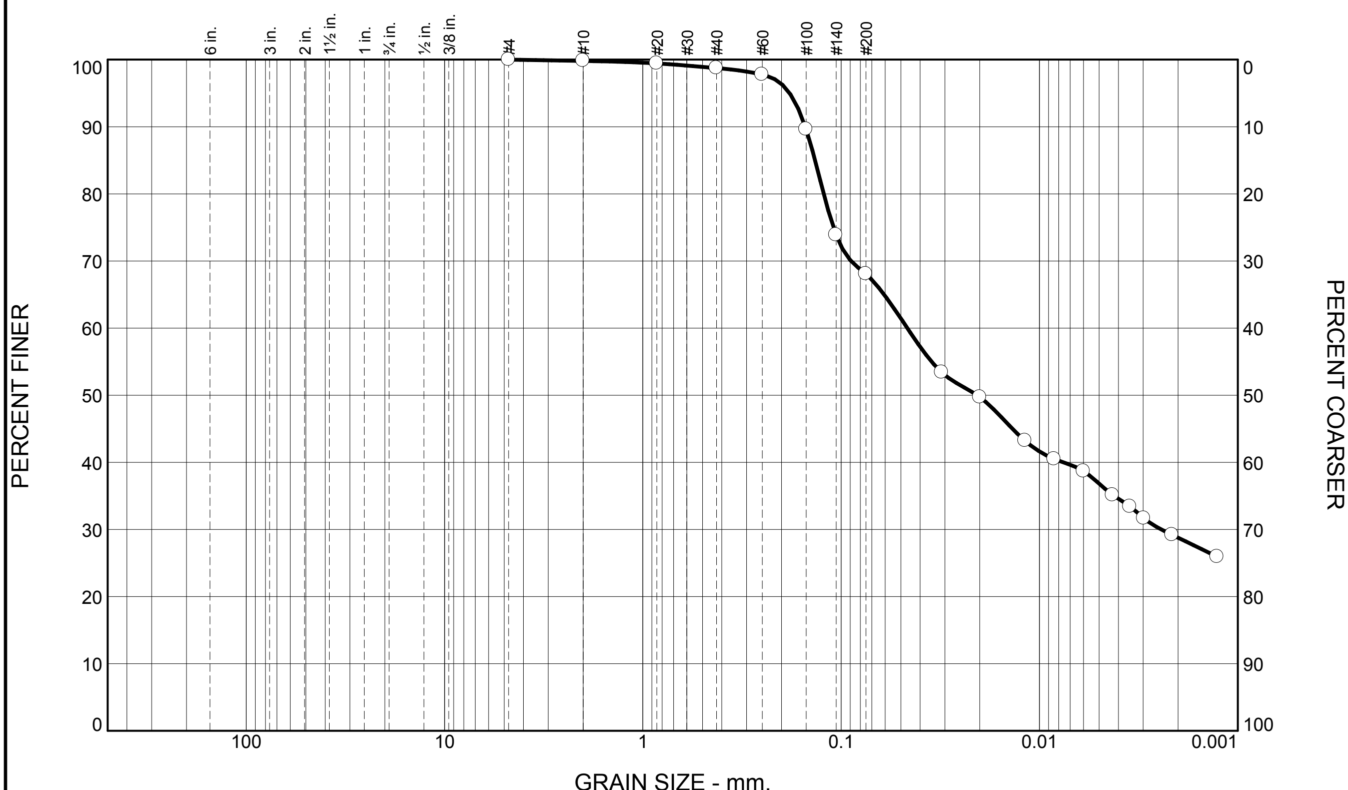
ENGEO INCORPORATED

Client: O'Brien Land Company, LLC
Project: Terraces of Lafayette Eng Consult

Project No: 9181.200.000

Tested By: JAL **Checked By:** GC

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	0.0	0.2	1.1	30.6	31.3	36.8

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#4	100.0		
#10	99.8		
#20	99.4		
#40	98.7		
#60	97.8		
#100	89.6		
#140	73.9		
#200	68.1		
0.0311 mm.	53.4		
0.0199 mm.	49.7		
0.0118 mm.	43.2		
0.0084 mm.	40.5		
0.0060 mm.	38.7		
0.0043 mm.	35.1		
0.0035 mm.	33.4		
0.0030 mm.	31.6		
0.0021 mm.	29.2		
0.0013 mm.	25.9		

* (no specification provided)

Soil Description

See exploration logs

Atterberg Limits

PL= 20 LL= 43 PI= 23

Coefficients

D_{90}= 0.1515 D_{85}= 0.1354 D_{60}= 0.0464
D_{50}= 0.0207 D_{30}= 0.0024 D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= CL AASHTO= A-7-6(14)

Remarks

GS: ASTM D422; PI: ASTM D4318; USCS: ASTM D2487

Sample Number: 2-TP3 @ 9 **Depth:** 9.0 feet

Date: 03/20/14

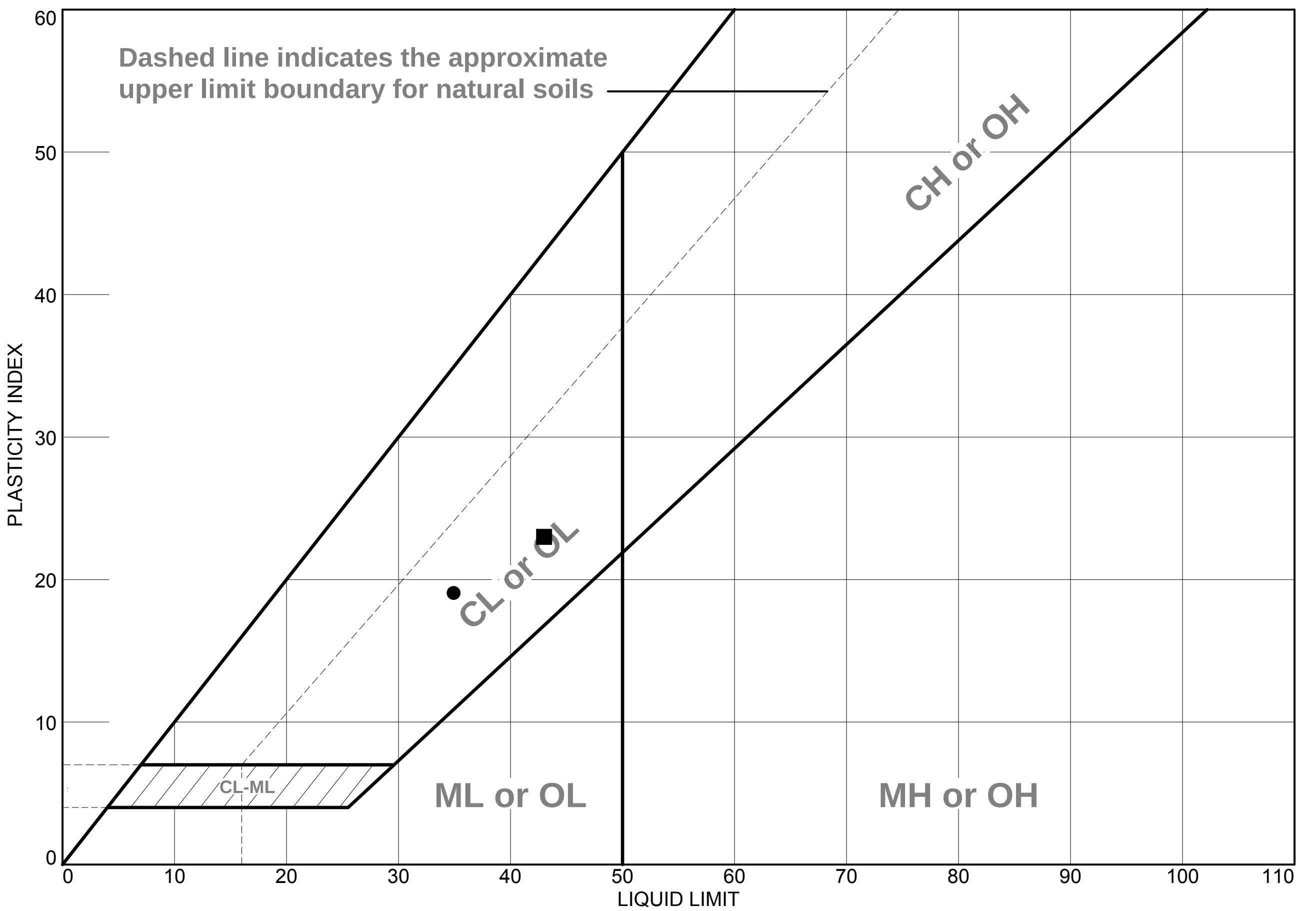
ENGEO INCORPORATED

Client: O'Brien Land Company, LLC
Project: Terraces of Lafayette Eng Consult

Project No: 9181.200.000

Tested By: JAL

LIQUID AND PLASTIC LIMITS TEST REPORT



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	See exploration logs	35	16	19	99.1	80.8	CL
■	See exploration logs	43	20	23	98.7	68.1	CL

Project No. 9181.200.000 **Client:** O'Brien Land Company, LLC

Project: Terraces of Lafayette Eng Consult

●**Depth:** 17.0 feet **Sample Number:** 2-TP1 @ 17

■**Depth:** 9.0 feet **Sample Number:** 2-TP3 @ 9

Remarks:

●PI: ASTM D4318; GS: ASTM D422; USCS: ASTM D2487

■PI: ASTM D4318; GS: ASTM D422; USCS: ASTM D2487

ENGEO INCORPORATED

Tested By: RB **Checked By:** GC

ENGEO

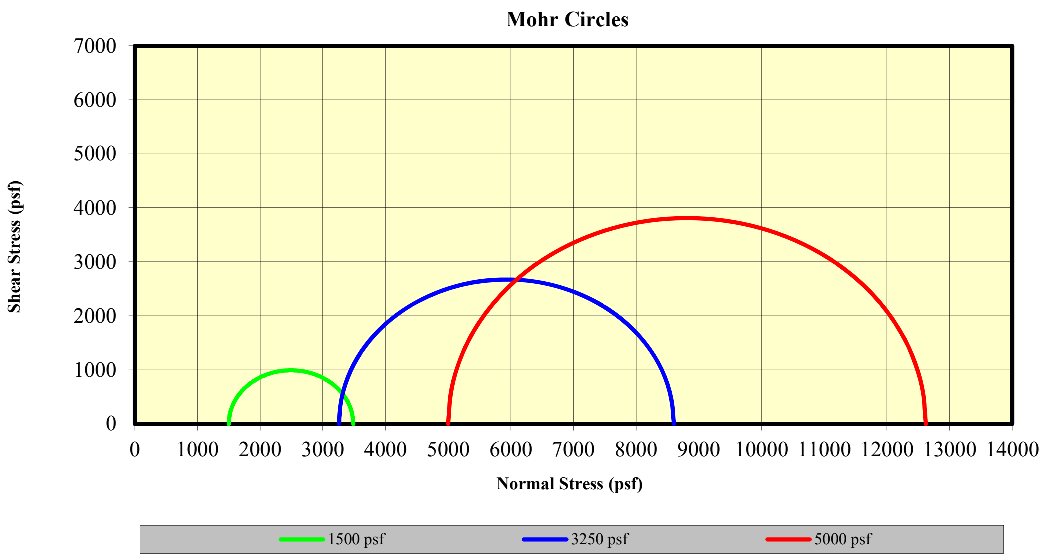
Unconsolidated Undrained Triaxial Test (ASTM D2850)

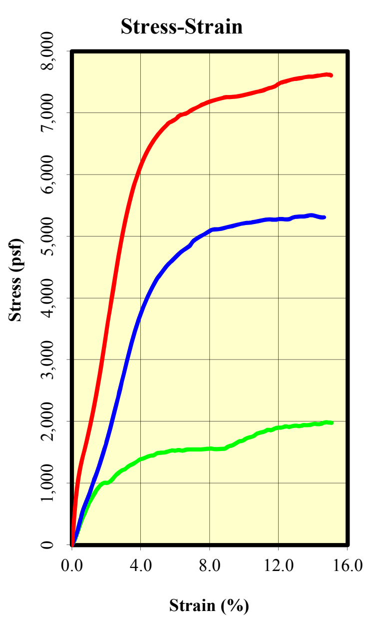
Date: 3.26.14

Checked By: D. Seibold

Date: 3.25.14

Tested By: G. Criste



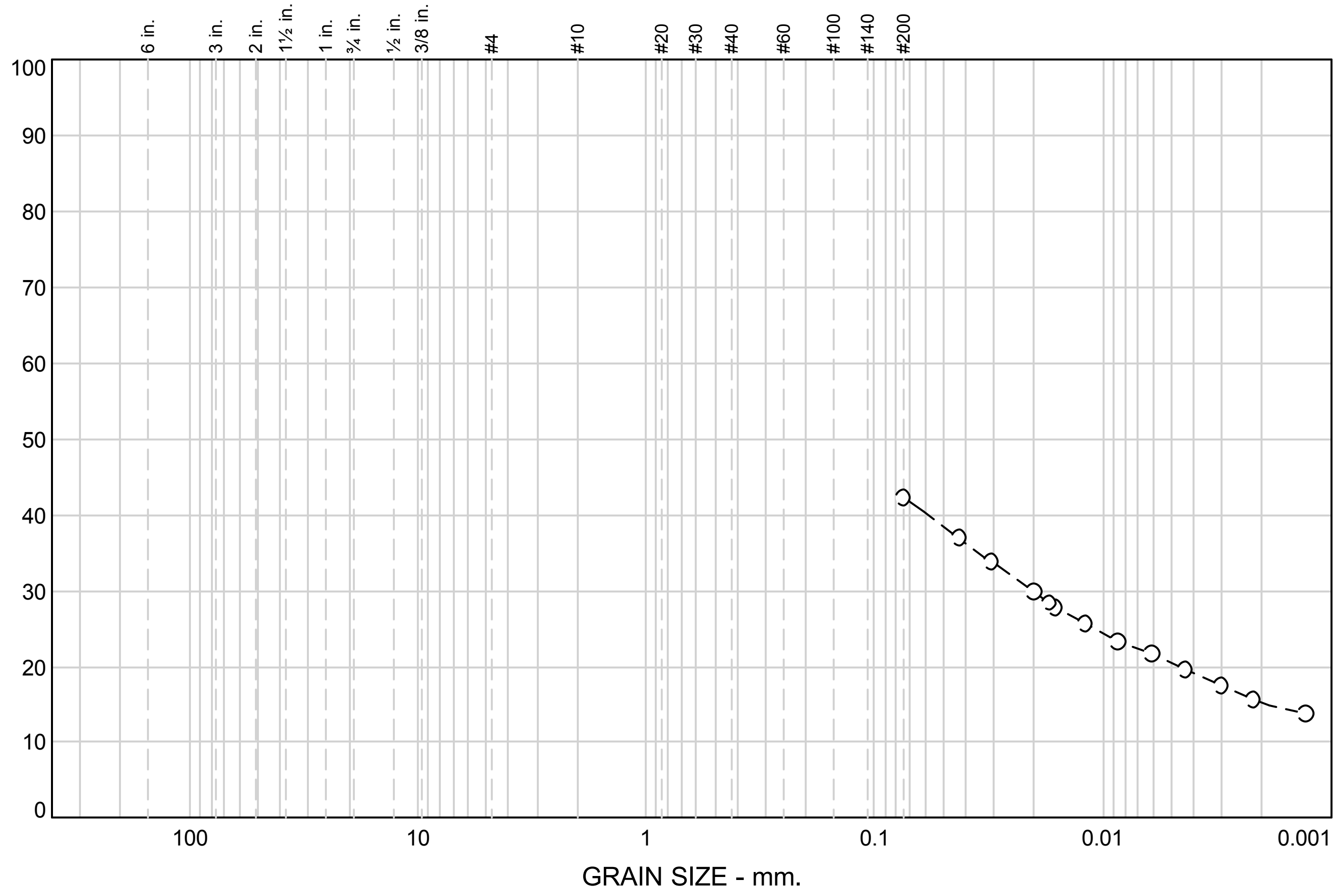


Before Test	Specimen 1500 psf	3250 psf	5000 psf	
Water Content (%)	16.50	16.50	16.50	
Dry Density (pcf)	102.64	102.67	103.53	
Saturation (%)	68.20	68.26	69.71	
Void Ratio	0.66	0.66	0.65	
Diameter (in)	2.430	2.436	2.435	
Height (in)	5.060	5.044	5.018	
Liquid Limit	-	-	-	
Plastic Limit	-	-	-	
Specific Gravity	2.730	2.730	2.730	
Height-to-Diam. Ratio	2.082	2.071	2.061	
After Test	**1500 psf**	**3250 psf**	**5000 psf**	
Water Content (%)	25.57	24.79	24.63	
Saturation (%)	100.00	100.00	100.00	
Strain Rate (in/min)	0.05	0.05	0.05	
Peak Deviator Stress (psf)	1986.0	5345.0	7622.7	
Axial Strain @ Failure (%)	14.880	13.891	14.796	
	Cell Pressure			
Cell (psf)	1497.6	3254.4	4996.8	
Back (psf)	n/a	n/a	n/a	
	Principle Stresses at Failure			
σ1 (psf)	3483.6	8599.4	12619.5	
σ3 (psf)	1497.6	3254.4	4996.8	

Mohr-Coulomb Parameters with a Non-zero Friction Angle (Ø≠0)			Cohesion at Failure with a Zero Friction Angle (Ø=0)			
Cohesion, c (psf)	500		993.0	2672.5	3811.3	
Friction Angle Ø	23		n/a	n/a	n/a	

Project Information			
Project Name:	Terraces of Lafayette Eng Consult		
Project Number:	9181.200.000	Job Number:	9181.200.000
Location:	Lafayette, California	Boring Number:	2-TP2
Client:	O'Brien Land Company, LLC	Sample Number:	2-TP2
Description:	See exploration logs		

Particle Size Distribution Report



% Cobbles	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
						27.0	15.3

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	42.3		
0.0429 mm.	37.0		
0.0310 mm.	33.9		
0.0202 mm.	29.8		
0.0172 mm.	28.4		
0.0162 mm.	27.8		
0.0120 mm.	25.7		
0.0086 mm.	23.3		
0.0061 mm.	21.8		
0.0044 mm.	19.7		
0.0030 mm.	17.6		
0.0022 mm.	15.7		
0.0013 mm.	13.8		

* (no specification provided)

Soil Description

Light yellowish brown silty SANDSTONE.

Atterberg Limits

PL= 28 LL= 34 PI= 6

Coefficients

D_{85}= D_{60}= D_{50}=
D_{30}= 0.0207 D_{15}= 0.0019 D_{10}=
C_u= C_c=

Classification

USCS= SM AASHTO= A-4(0)

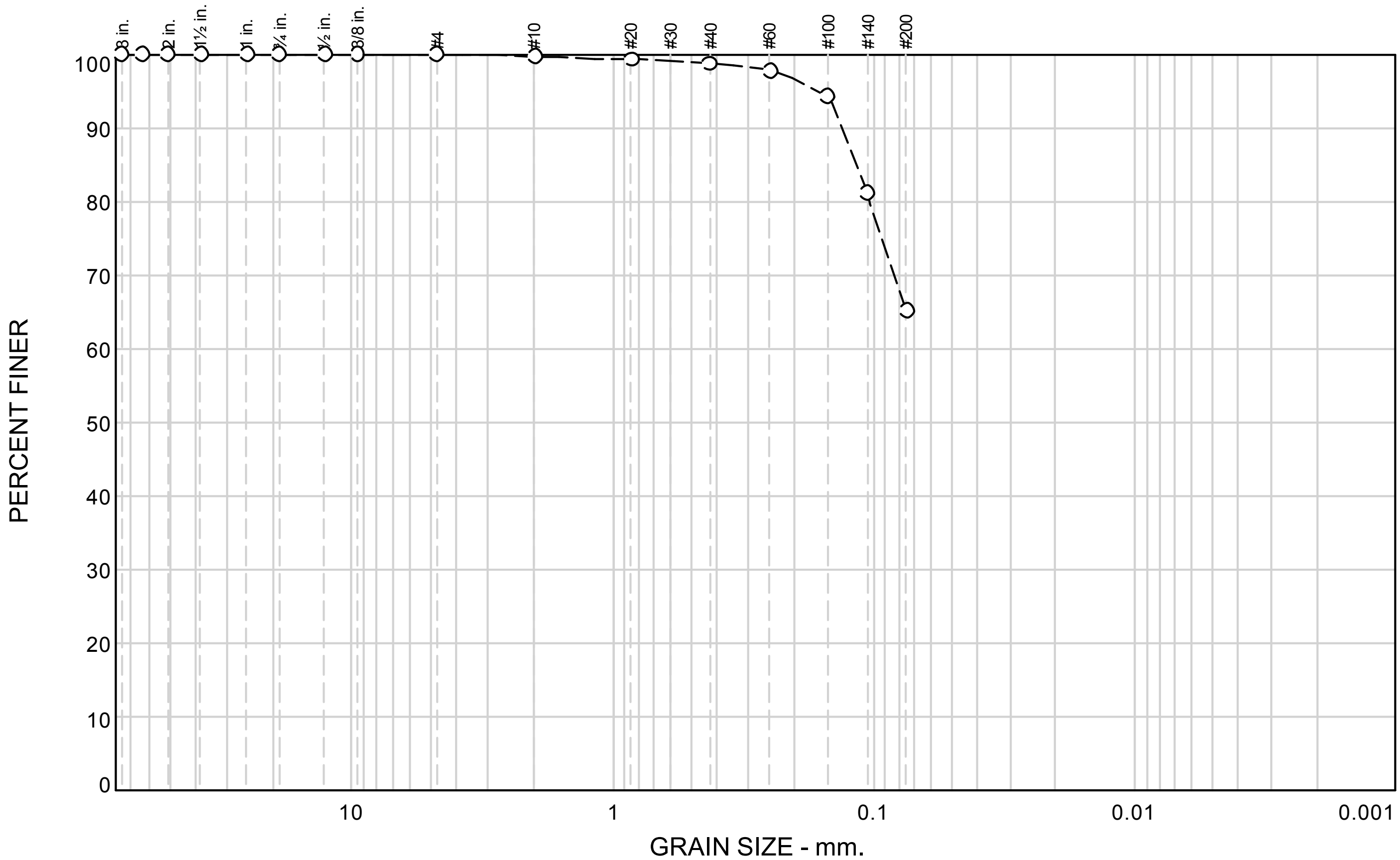
Remarks

Sample No.: #1 @ 6' **Source of Sample:** **Date:** 2/16/11
Location: **Elev./Depth:**

ENGEO INCORPORATED GEOTECHNICAL AND ENVIRONMENTAL CONSULTANTS MATERIALS TESTING

Client:
Project: Terraces of Lafayette
Project No: 9181.000.000

Particle Size Distribution Report



% Cobbles	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	0.0	0.3	1.0	33.4	65.3	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3	100.0		
2-1/2	100.0		
2	100.0		
1-1/2	100.0		
1	100.0		
3/4	100.0		
1/2	100.0		
3/8	100.0		
#4	100.0		
#10	99.7		
#20	99.3		
#40	98.7		
#60	97.8		
#100	94.2		
#140	81.1		
#200	65.3		

* (no specification provided)

Soil Description

See exploration logs

Atterberg Limits

PL= 16 LL= 46 PI= 30

Coefficients

D_{85}= 0.1158 D_{60}= D_{50}=
D_{30}= D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= CL AASHTO= A-7-6(17)

Remarks

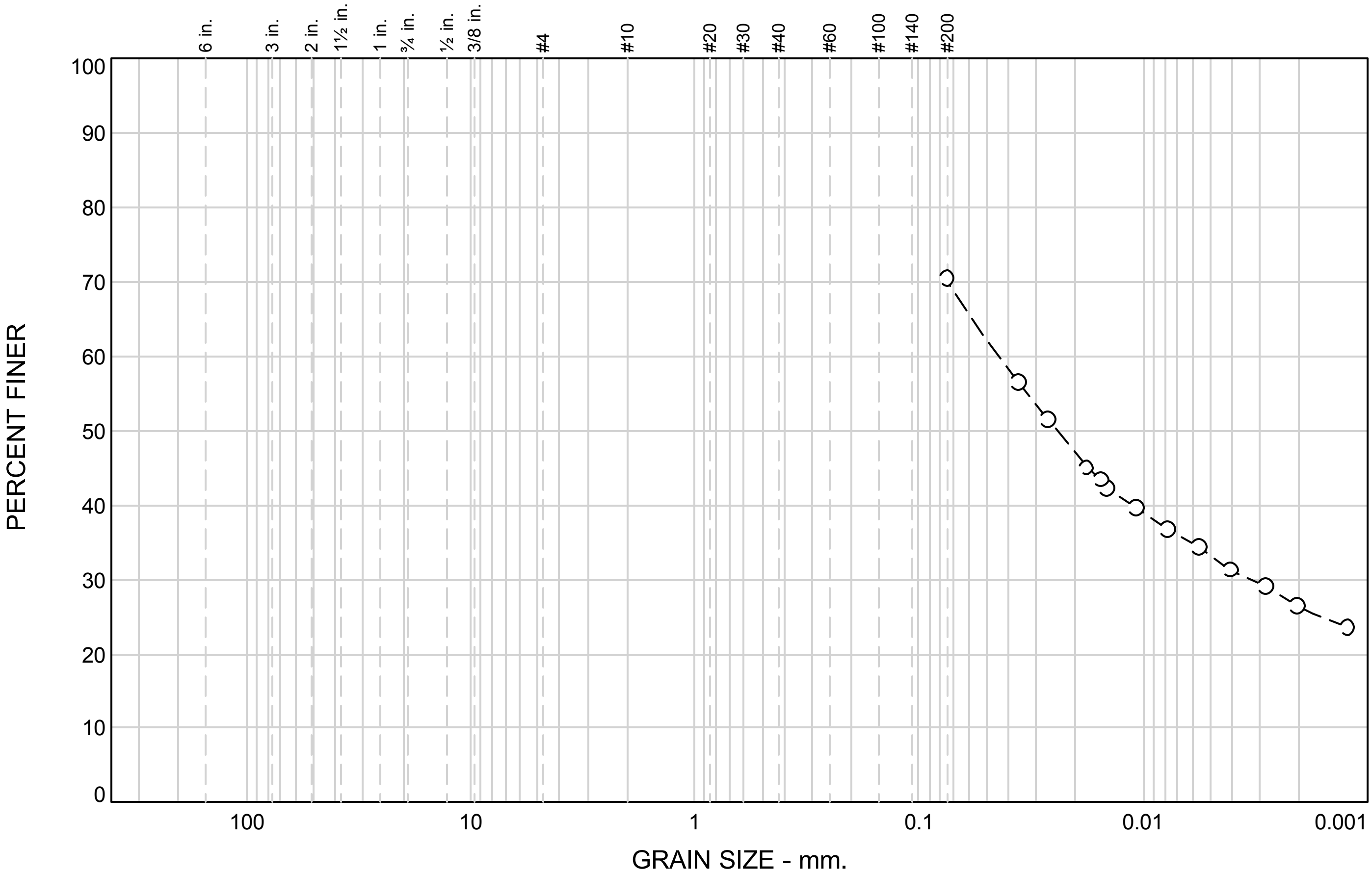
Sample No.: B-1 @ 27.5 **Source of Sample:** **Date:** 06/30/11
Location: B-1 **Elev./Depth:** 27.5 feet

ENGEO INCORPORATED
GEOTECHNICAL AND ENVIRONMENTAL CONSULTANTS
MATERIALS TESTING

Client: O'Brien Land Company, LLC
Project: The Terraces of Lafayette
Project No: 9181.100.000 **Plate**

Tested By: TB **Checked By:** GC

Particle Size Distribution Report



% Cobbles	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
						44.3	26.1

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	70.4		
0.0359 mm.	56.4		
0.0267 mm.	51.6		
0.0180 mm.	45.1		
0.0154 mm.	43.4		
0.0145 mm.	42.4		
0.0108 mm.	39.7		
0.0078 mm.	36.7		
0.0056 mm.	34.4		
0.0041 mm.	31.3		
0.0028 mm.	29.1		
0.0021 mm.	26.4		
0.0012 mm.	23.6		

* (no specification provided)

Soil Description

Brown sandy CLAY.

Atterberg Limits

PL= 20 LL= 43 PI= 23

Coefficients

D_{85}= D_{60}= 0.0440 D_{50}= 0.0245
D_{30}= 0.0033 D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= CL AASHTO= A-7-6(15)

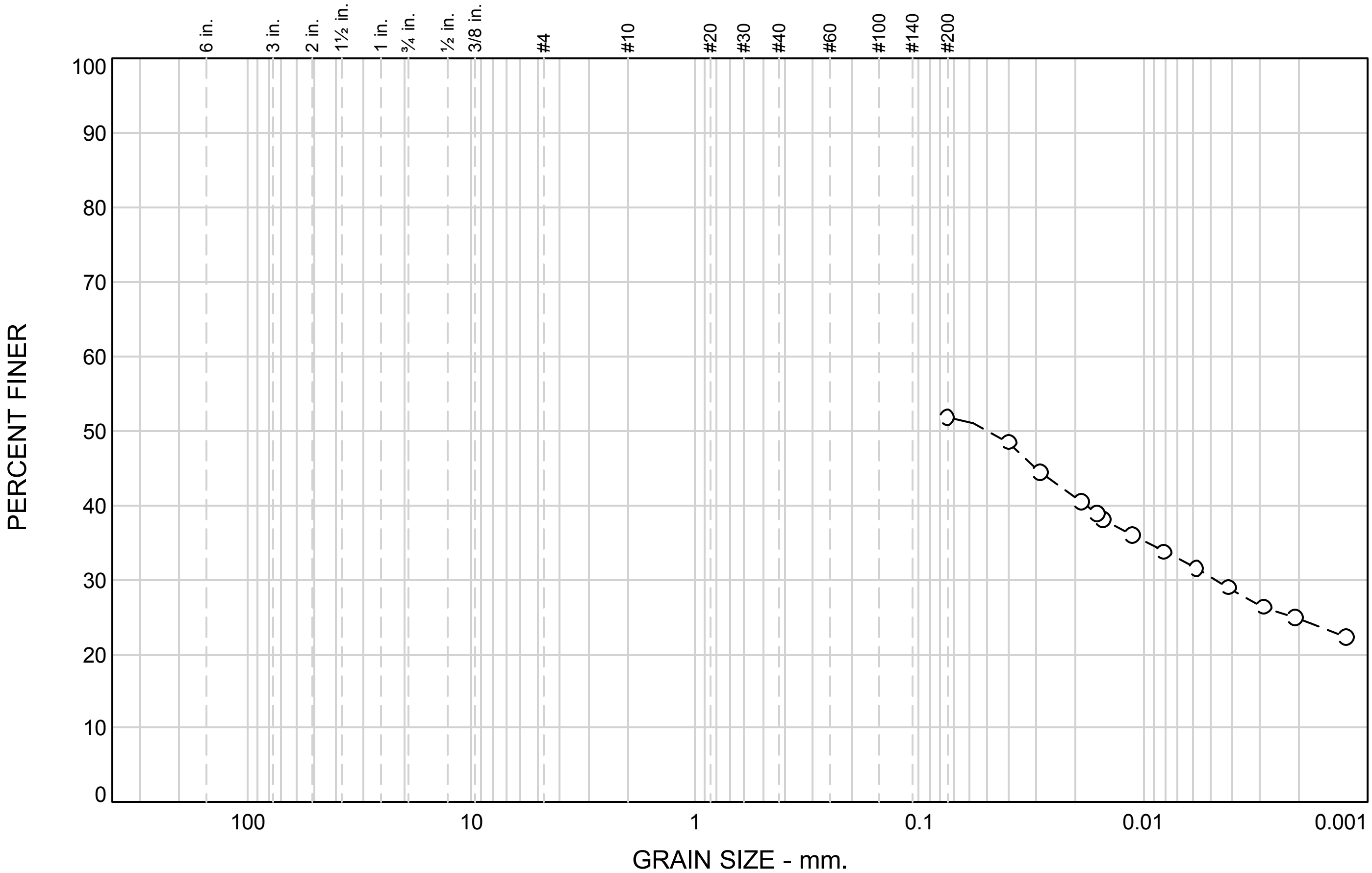
Remarks

Sample No.: #2 @ 6' **Source of Sample:** **Date:** 02/16/11
Location: **Elev./Depth:**

ENGEO INCORPORATED GEOTECHNICAL AND ENVIRONMENTAL CONSULTANTS MATERIALS TESTING

Client:
Project: Terraces of Lafayette
Project No: 9181.000.000

Particle Size Distribution Report



% Cobbles	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
						27.0	24.7

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	51.7		
0.0397 mm.	48.4		
0.0290 mm.	44.5		
0.0189 mm.	40.5		
0.0162 mm.	38.8		
0.0152 mm.	38.1		
0.0113 mm.	36.0		
0.0081 mm.	33.7		
0.0058 mm.	31.5		
0.0042 mm.	28.9		
0.0029 mm.	26.3		
0.0021 mm.	24.9		
0.0012 mm.	22.3		

* (no specification provided)

Soil Description

Brown sandy CLAY with sandstone fragments.

Atterberg Limits

PL= 19 LL= 41 PI= 22

Coefficients

D_{85}= D_{60}= D_{50}= 0.0473
D_{30}= 0.0048 D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= CL AASHTO= A-7-6(8)

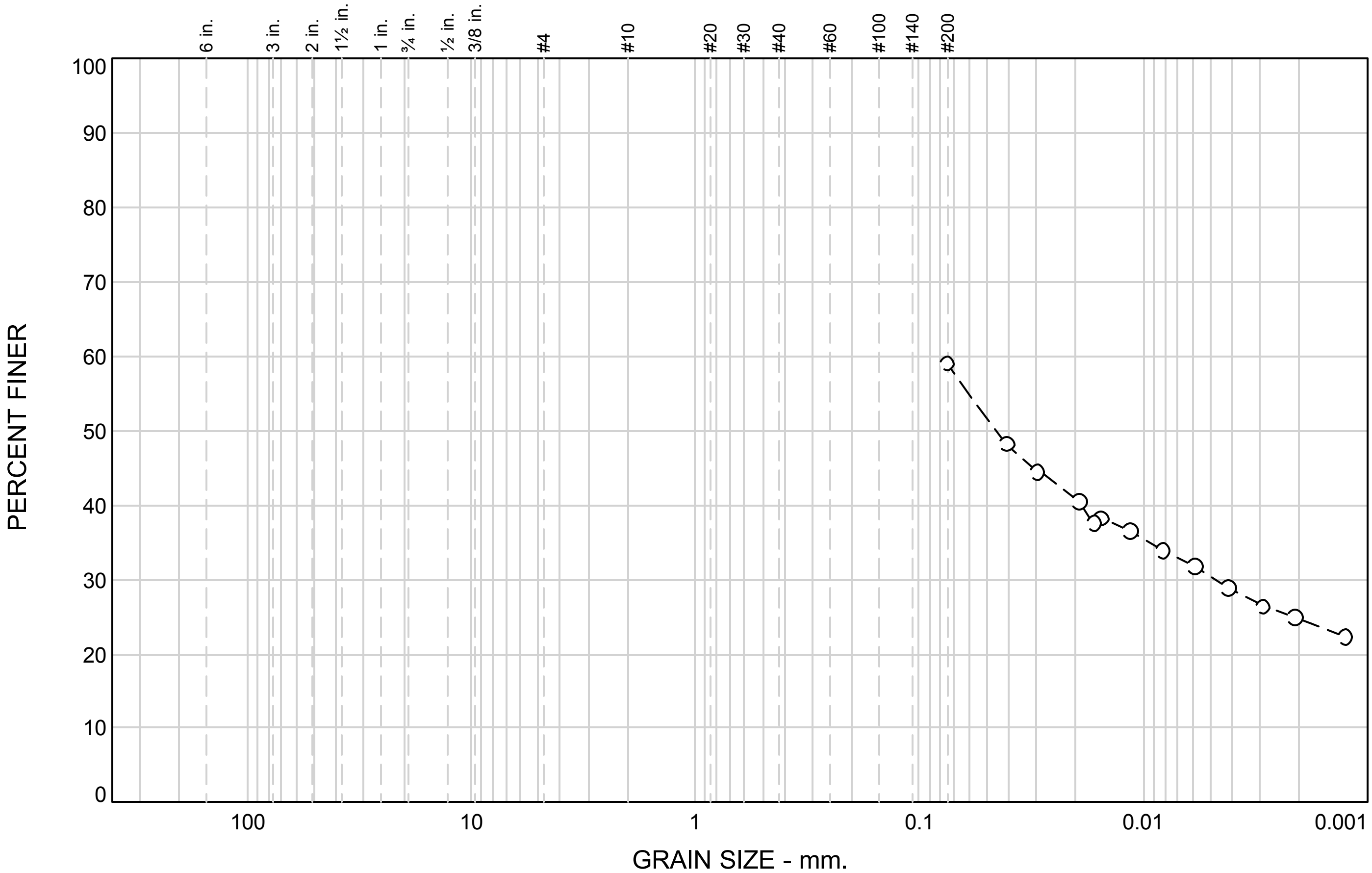
Remarks

Sample No.: #3 @ 0.5' **Source of Sample:** **Date:** 02/16/11
Location: **Elev./Depth:**

ENGEO INCORPORATED GEOTECHNICAL AND ENVIRONMENTAL CONSULTANTS MATERIALS TESTING

Client:
Project: Terraces of Lafayette
Project No: 9181.000.000

Particle Size Distribution Report



% Cobbles	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
						34.4	24.6

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	59.0		
0.0409 mm.	48.2		
0.0297 mm.	44.4		
0.0193 mm.	40.4		
0.0166 mm.	37.6		
0.0155 mm.	38.2		
0.0114 mm.	36.5		
0.0082 mm.	33.9		
0.0059 mm.	31.7		
0.0042 mm.	28.8		
0.0029 mm.	26.4		
0.0021 mm.	24.9		
0.0013 mm.	22.2		

* (no specification provided)

Soil Description

Dark brown sandy CLAY.

Atterberg Limits

PL= 19 LL= 40 PI= 21

Coefficients

D_{85}= D_{60}= D_{50}= 0.0458
D_{30}= 0.0048 D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= CL AASHTO= A-6(10)

Remarks

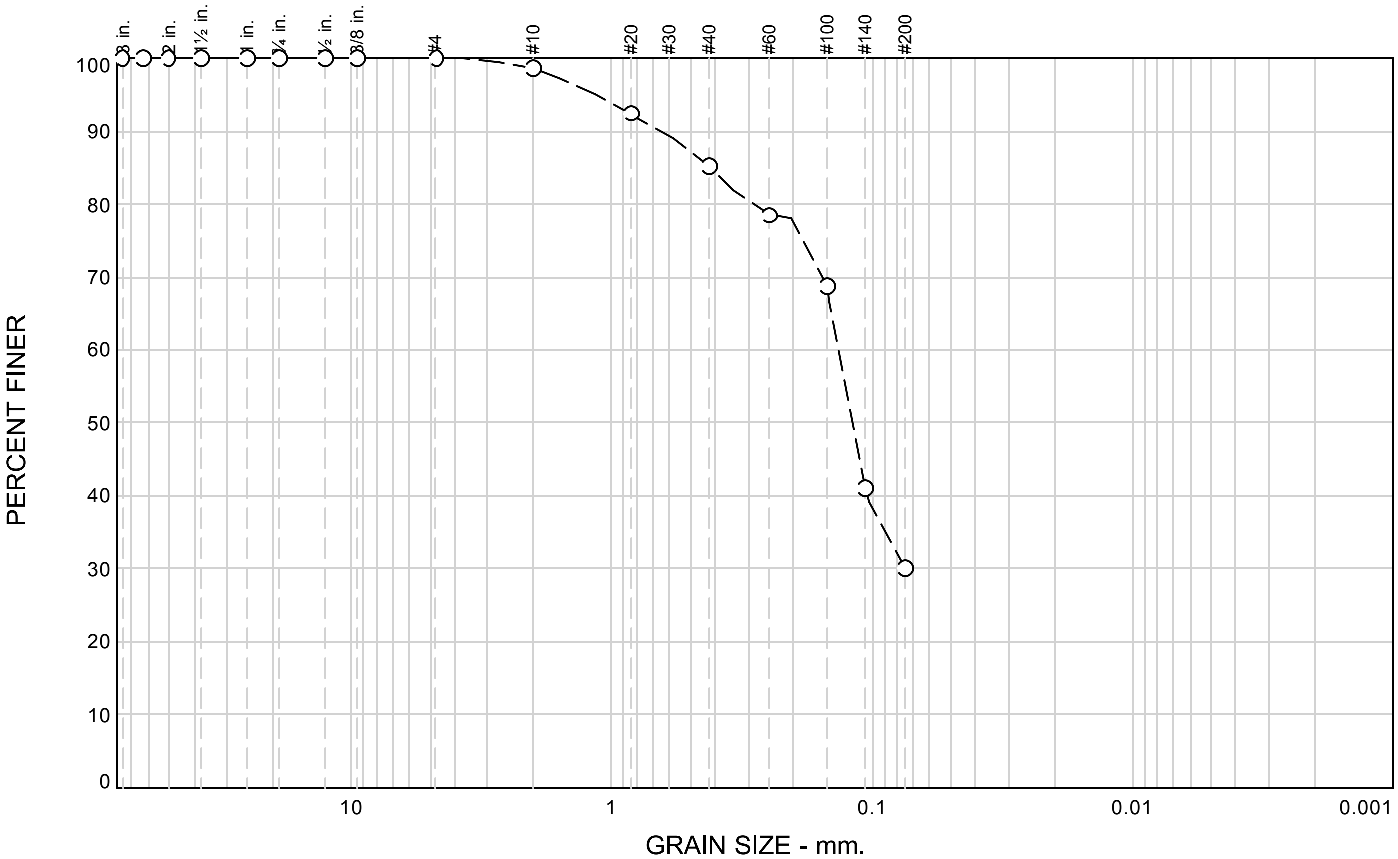
Sample No.: #4 @ 0.5' **Source of Sample:** **Date:** 02/16/11
Location: **Elev./Depth:**

ENGEO INCORPORATED GEOTECHNICAL AND ENVIRONMENTAL CONSULTANTS MATERIALS TESTING

Client:
Project: Terraces of Lafayette

Project No: 9181.000.000

Particle Size Distribution Report



% Cobbles	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	0.0	1.3	13.5	55.1	30.1	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3	100.0		
2.5	100.0		
2	100.0		
1.5	100.0		
1	100.0		
3/4	100.0		
1/2	100.0		
3/8	100.0		
#4	100.0		
#10	98.7		
#20	92.5		
#40	85.2		
#60	78.5		
#100	68.9		
#140	41.0		
#200	30.1		

* (no specification provided)

Soil Description

See exploration logs

Atterberg Limits

PL= 30 LL= 34 PI= 4

Coefficients

D_{85}= 0.4182 D_{60}= 0.1335 D_{50}= 0.1191
D_{30}= D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= SM AASHTO= A-2-4(0)

Remarks

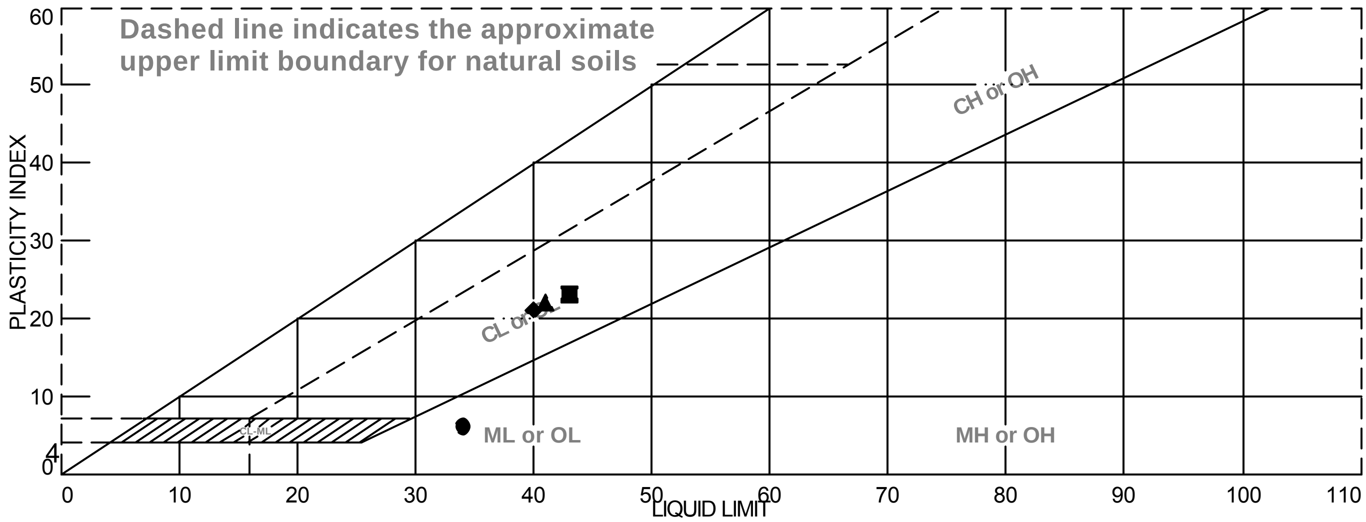
Sample No.: TP-2 **Source of Sample:** **Date:** 06/30/11
Location: TP-2 **Elev./Depth:**

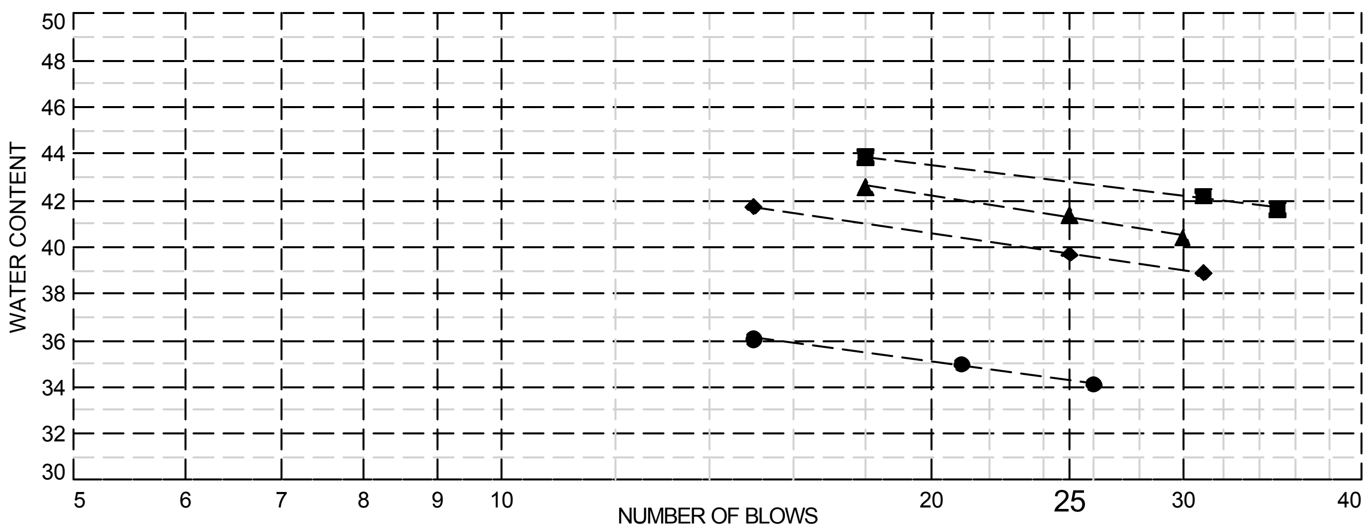
ENGEO INCORPORATED GEOTECHNICAL AND ENVIRONMENTAL CONSULTANTS MATERIALS TESTING

Client: O'Brien Land Company, LLC
Project: The Terraces of Lafayette
Project No: 9181.100.000

Tested By: DS **Checked By:** GC

LIQUID AND PLASTIC LIMITS TEST REPORT





	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	Light yellowish brown silty SANDSTONE.	34	28	6		42.3	SM
■	Brown sandy CLAY.	43	20	23		70.4	CL
▲	Brown sandy CLAY with sandstone fragments.	41	19	22		51.7	CL
◆	Dark brown sandy CLAY.	40	19	21		59.0	CL

Project No. 9181.000.000 **Client:**

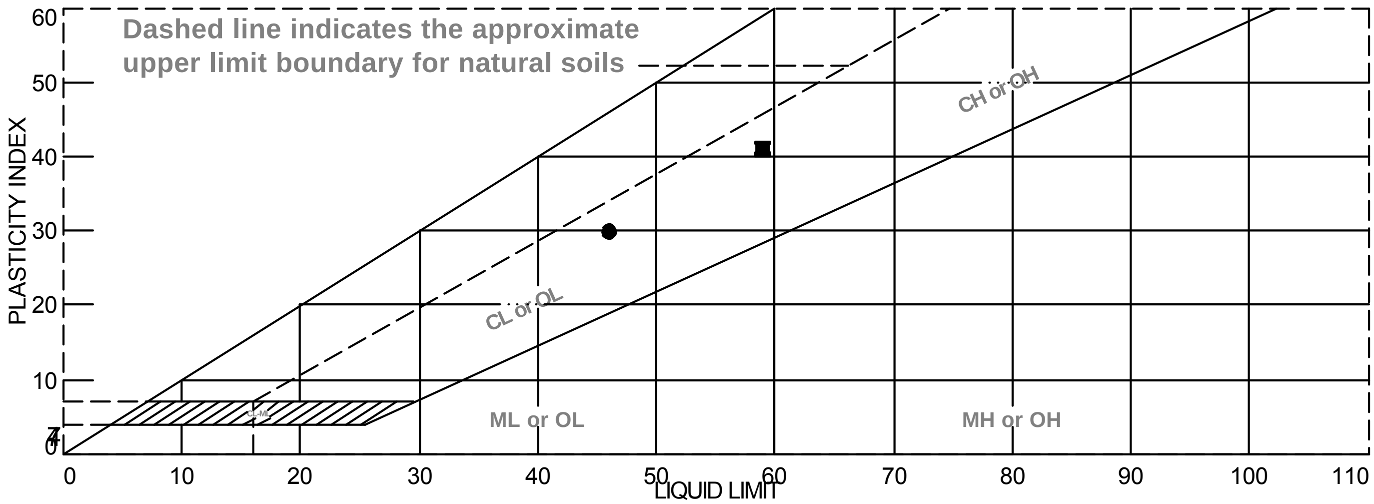
Project: Terraces of Lafayette

●**Sample Number:** #1 @ 6'
■**Sample Number:** #2 @ 6'
▲**Sample Number:** #3 @ 0.5'
◆**Sample Number:** #4 @ 0.5'

ENGEO INCORPORATED GEOTECHNICAL AND ENVIRONMENTAL CONSULTANTS MATERIALS TESTING

Remarks:

LIQUID AND PLASTIC LIMITS TEST REPORT



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	See exploration logs	46	16	30	98.7	65.3	CL
■	See exploration logs	59	18	41			CH

Project No. 9181.100.000 **Client:** O'Brien Land Company, LLC

Project: The Terraces of Lafayette

● **Location:** B-1 **Depth:** 27.5 feet **Sample Number:** B-1 @ 27.5

■ **Location:** B-1 **Depth:** 3 feet **Sample Number:** B-1 @ 3

Remarks:

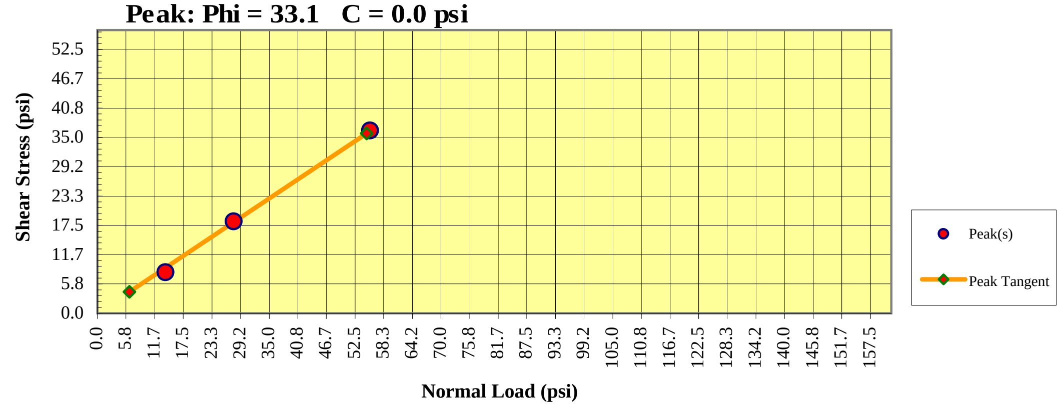
ENGEO INCORPORATED GEOTECHNICAL AND ENVIRONMENTAL CONSULTANTS MATERIALS TESTING

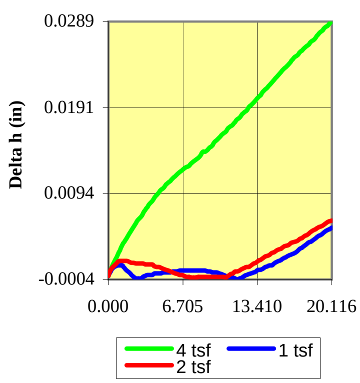
Plate

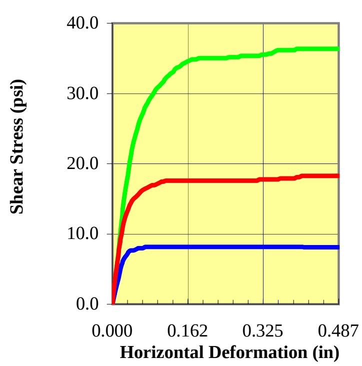
Tested By: GC **Checked By:** DS

Direct Shear Test (ASTM D3080)

Peak: Phi = 33.1 C = 0.0 psi







Initial	Specimen 4 tsf	1 tsf	2 tsf	
Moisture (%)	16.96	16.96	16.96	
Density (pcf)	100.34	100.36	100.46	
Void Ratio	0.736	0.736	0.734	
Saturation (%)	64.30	64.34	64.49	
Diameter (in)	2.420	2.420	2.420	
Height (in)	1.200	1.200	1.200	

Final	4 tsf	1 tsf	2 tsf	
Moisture (%)	25.76	25.72	26.49	
Density (pcf)	97.27	98.15	97.18	
Void Ratio	0.791	0.775	0.792	
Saturation (%)	91.21	92.79	93.50	
Diameter (in)	2.420	2.420	2.420	
Height (in)	1.198	1.199	1.199	
Normal Stress (psi)	55.6	13.9	27.8	
Peak Stress (psi)	36.4	8.1	18.3	
Residual Stress (psi)				
Strain (%)	20.116	20.116	20.116	
Rate (in/min)	0.0003	0.0003	0.0003	

Project Date	
Date	6.27.11

Project:	The Terraces at Lafayette GEX
Location:	Lafayette, California
Project Number:	9181.100.000
Boring Number	TP2
Sample Number:	TP2@4
Depth:	4.0 ft.
Sample Type:	Remolded
Description:	Silty SAND (SM).
Test Type:	Direct Shear, ASTM D3080.
Remarks:	Specimens were remolded to 90% compaction at 2% over optimum moisture content. Max density; 110.7 pcf. Optimum moisture content; 14.8%.

N/A	N/A	N/A	N/A
Failure Photographs			

Date 7.6.11

J. Fippin

Checked By

Date 7.6.11

Date 7.6.11

D. Seibold

Tested By

COMPACTION TEST REPORT

Curve No.: TP-2@4

Project No.: 9181.100.000 **Date:** 06/23/11

Project: The Terraces of Lafayette

Location: TP-2

Elev./Depth: **Sample No.** TP-2

Remarks:

MATERIAL DESCRIPTION

Description: See exploration logs

Classifications - **USCS:** SM **AASHTO:** A-2-4(0)

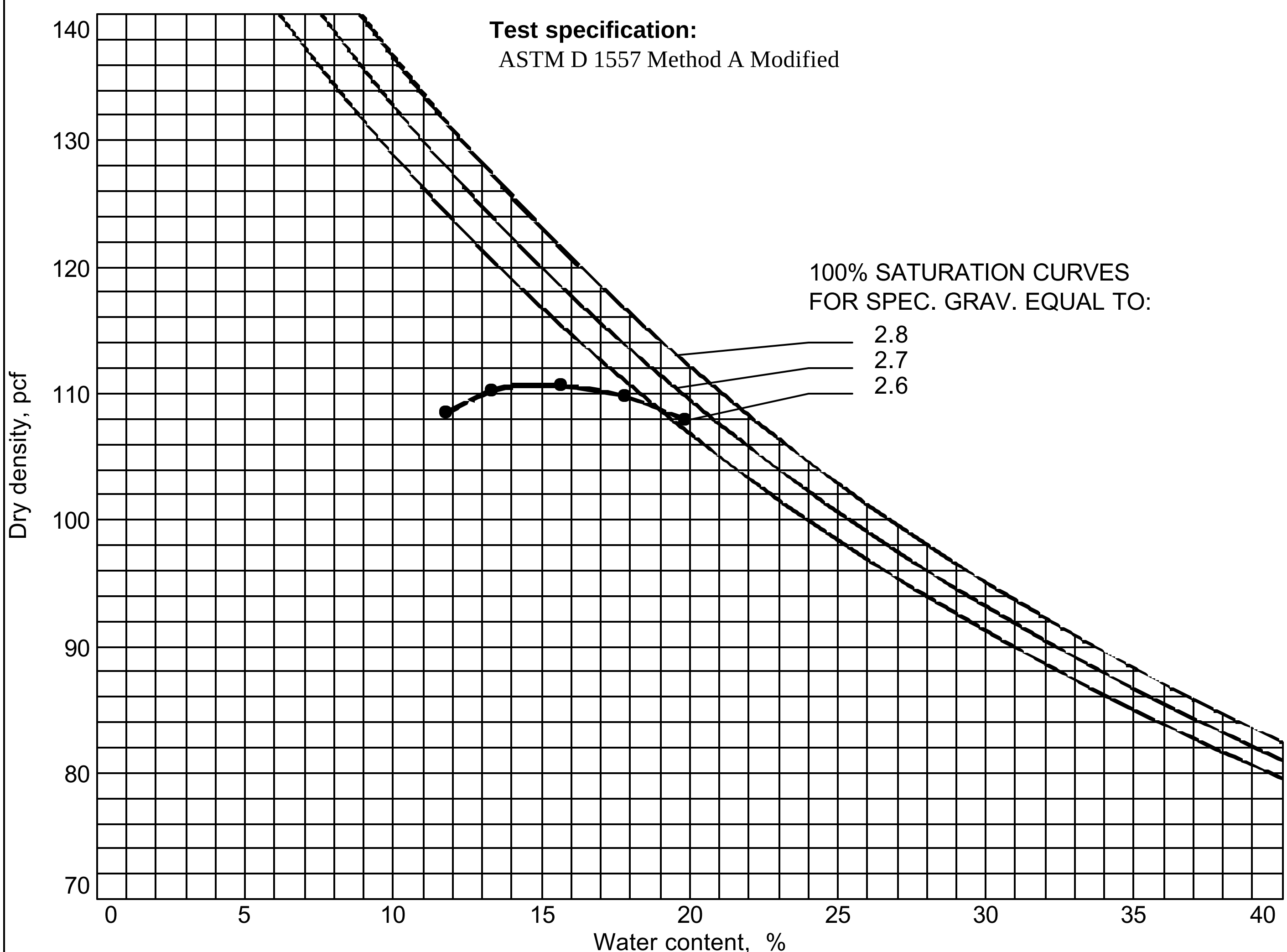
Nat. Moist. = **Sp.G. =**

Liquid Limit = 34 **Plasticity Index =** 4

% > No.4 = 0.0 % **% < No.200 =** 30.1 %

TEST RESULTS
Maximum dry density = 110.7 pcf
Optimum moisture = 14.6 %

Test specification:
ASTM D 1557 Method A Modified



EN GEO Incorporated

SULFATE TEST RESULTS

CALTRANS Test Method 417

Project Name: The Terraces of Lafayette   Project Number: 9181.100.000

Tested By: JG   Date: June 28, 2011

Sample Number	Sample Location	Matrix	Water Soluble Sulfate (SO_4) in Soil	
			mg/kg	% by Weight
1	B-2@1.5'	soil	5	0.000
2	B-3@5'	soil	3882	0.388

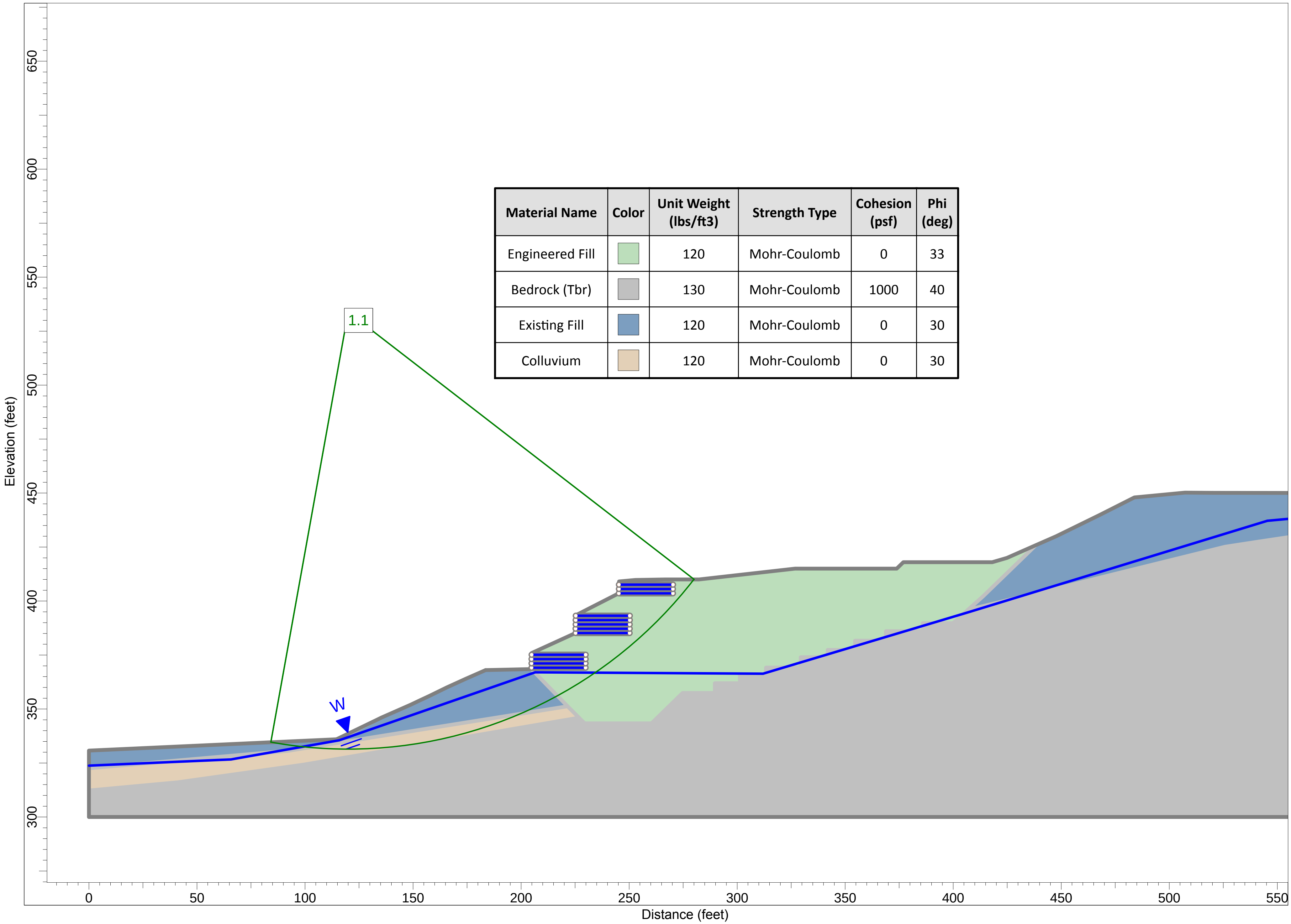
Office: 2010 Crow Canyon Place, Suite 250, San Ramon, CA 94583
Laboratory: 2057 San Ramon Valley Boulevard, San Ramon, CA 94583

APPENDIX C

Slope Stability Analyses Results

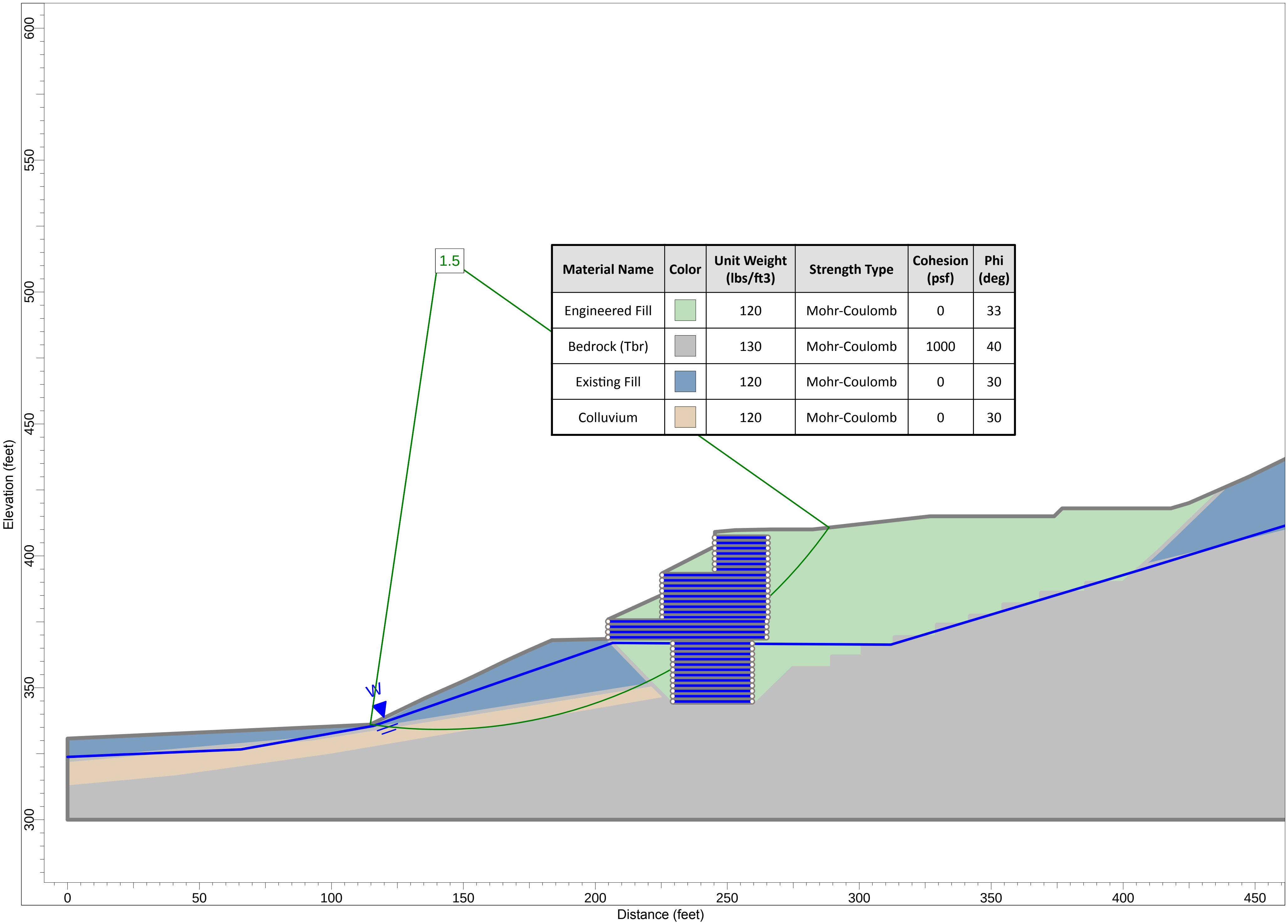
APPENDIX C

Section 1-1' - Static Slope Stability



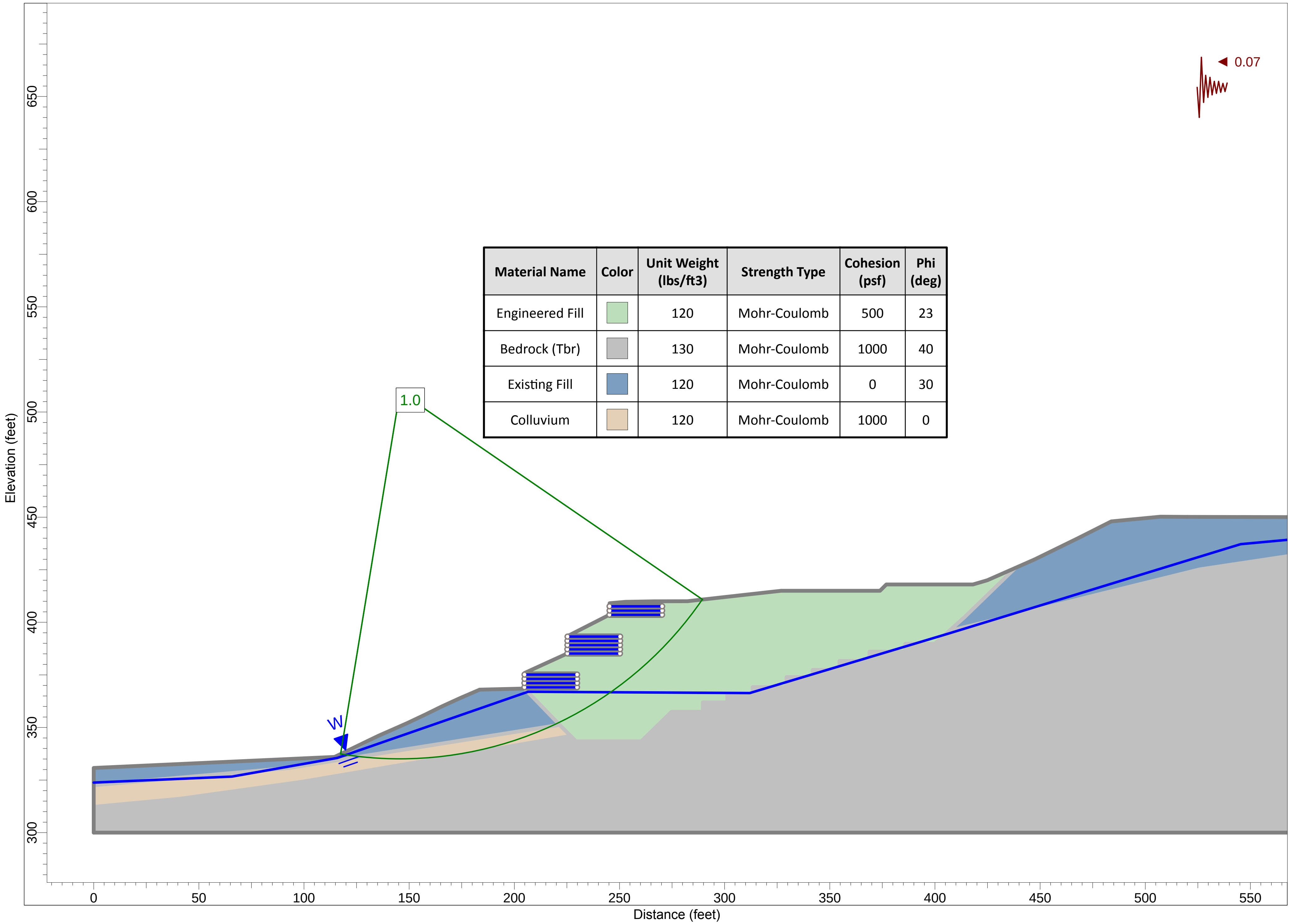
Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Engineered Fill		120	Mohr-Coulomb	0	33
Bedrock (Tbr)		130	Mohr-Coulomb	1000	40
Existing Fill		120	Mohr-Coulomb	0	30
Colluvium		120	Mohr-Coulomb	0	30

Section 1-1' - Static Slope Stability Mitigation



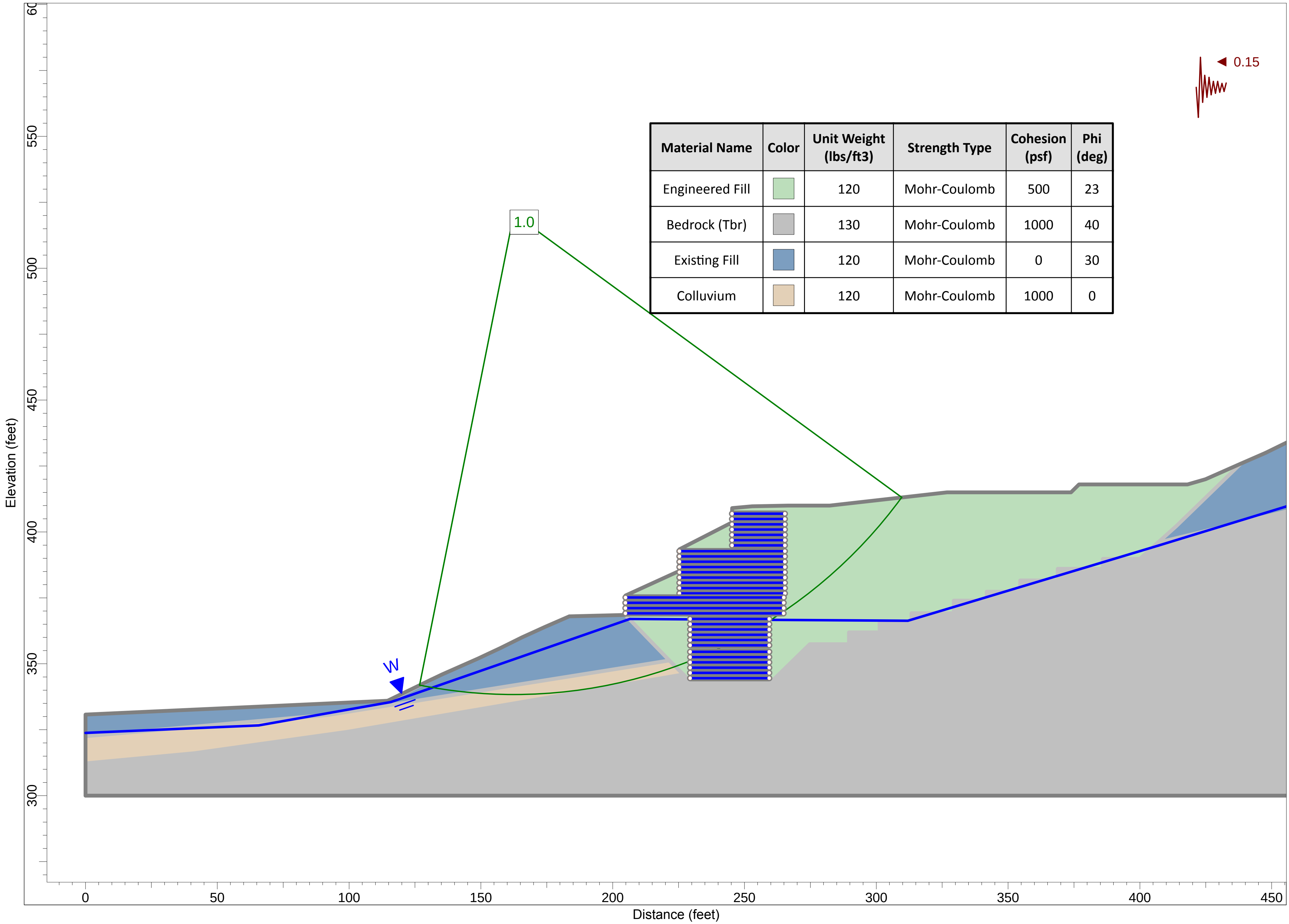
Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Engineered Fill		120	Mohr-Coulomb	0	33
Bedrock (Tbr)		130	Mohr-Coulomb	1000	40
Existing Fill		120	Mohr-Coulomb	0	30
Colluvium		120	Mohr-Coulomb	0	30

Section 1-1' - Seismic Slope Stability



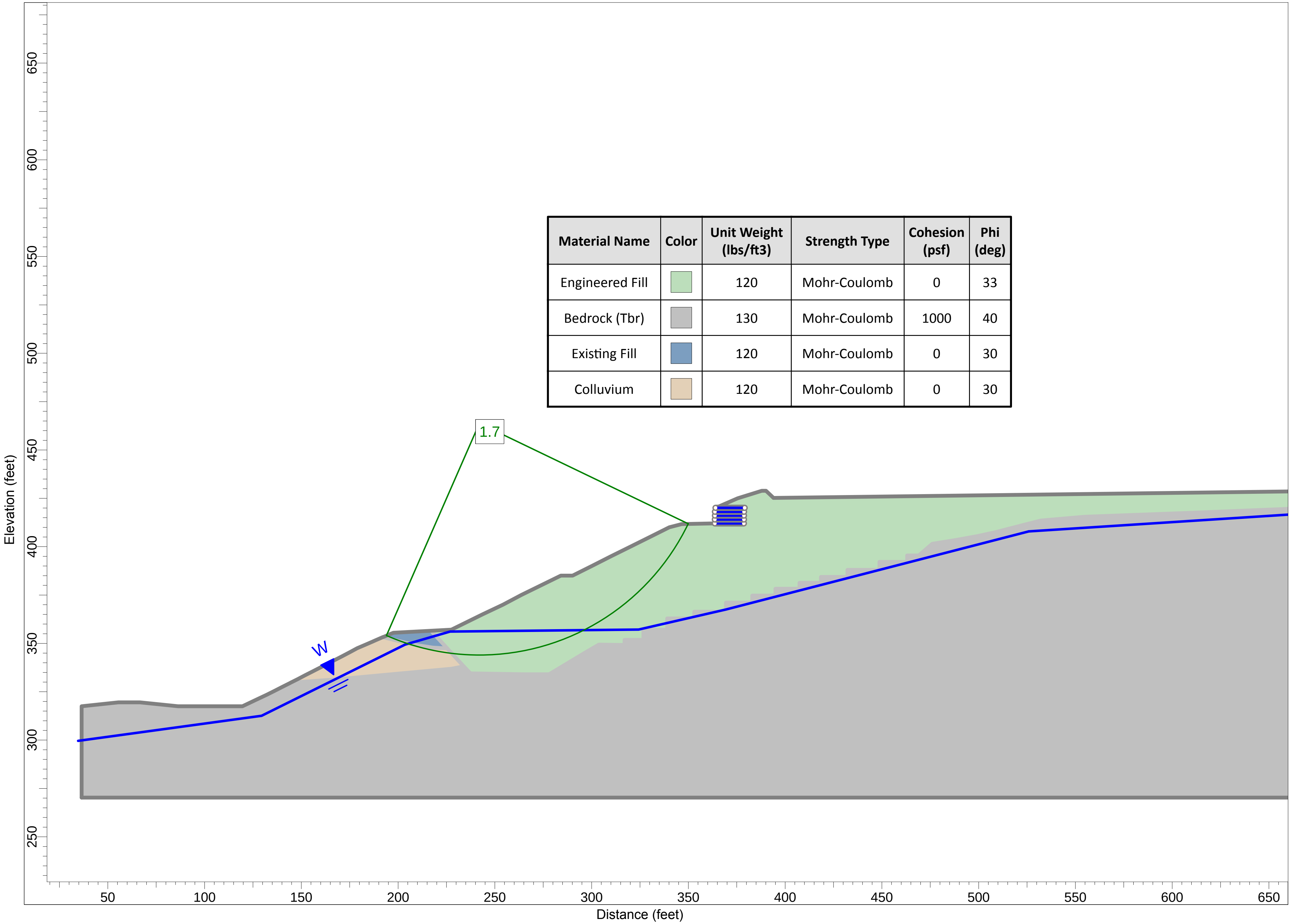
Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Engineered Fill		120	Mohr-Coulomb	500	23
Bedrock (Tbr)		130	Mohr-Coulomb	1000	40
Existing Fill		120	Mohr-Coulomb	0	30
Colluvium		120	Mohr-Coulomb	1000	0

Section 1-1' - Seismic Slope Stability Mitigation



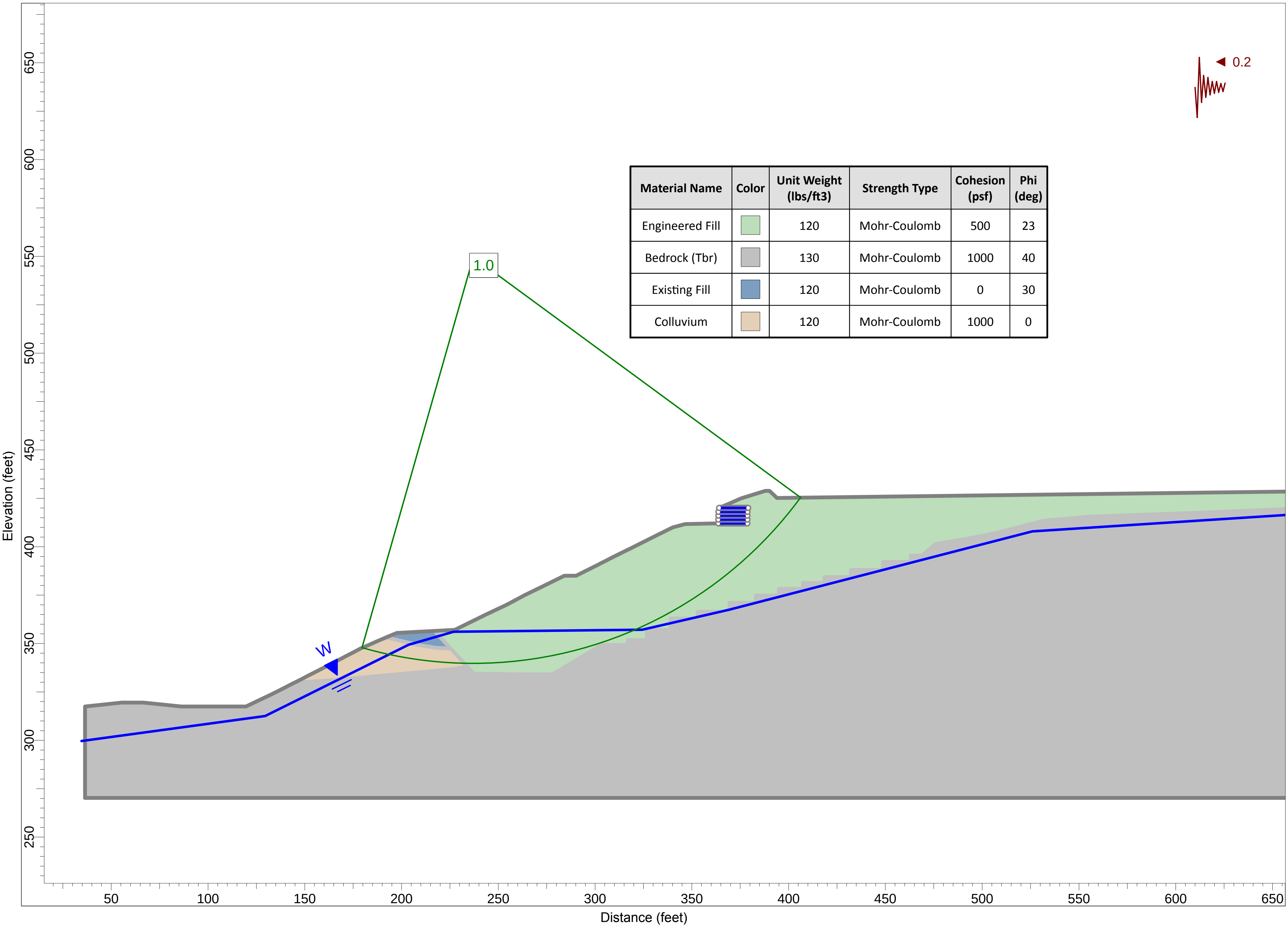
Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Engineered Fill		120	Mohr-Coulomb	500	23
Bedrock (Tbr)		130	Mohr-Coulomb	1000	40
Existing Fill		120	Mohr-Coulomb	0	30
Colluvium		120	Mohr-Coulomb	1000	0

Section 2-2' - Static Slope Stability



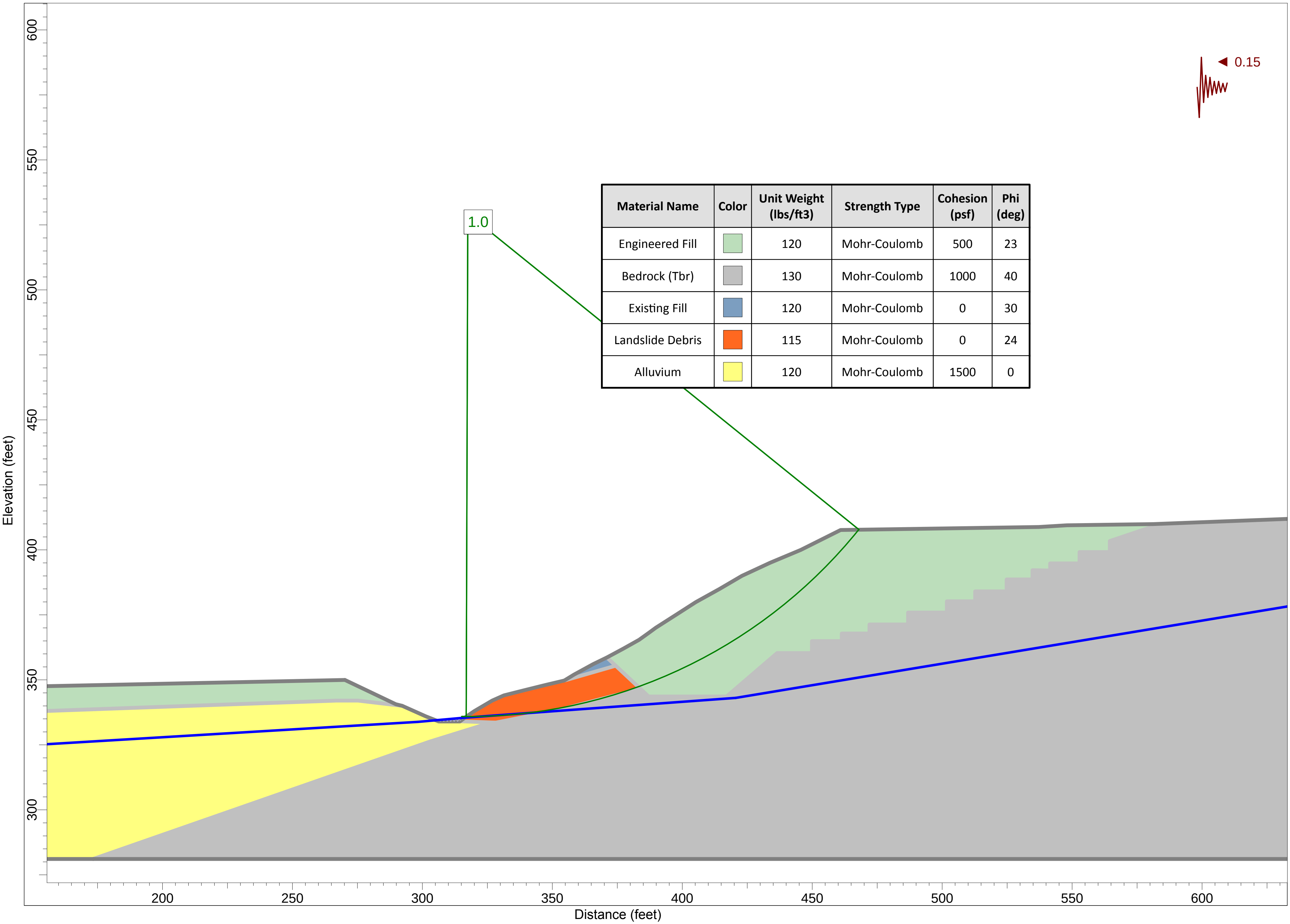
Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Engineered Fill		120	Mohr-Coulomb	0	33
Bedrock (Tbr)		130	Mohr-Coulomb	1000	40
Existing Fill		120	Mohr-Coulomb	0	30
Colluvium		120	Mohr-Coulomb	0	30

Section 2-2' - Seismic Slope Stability



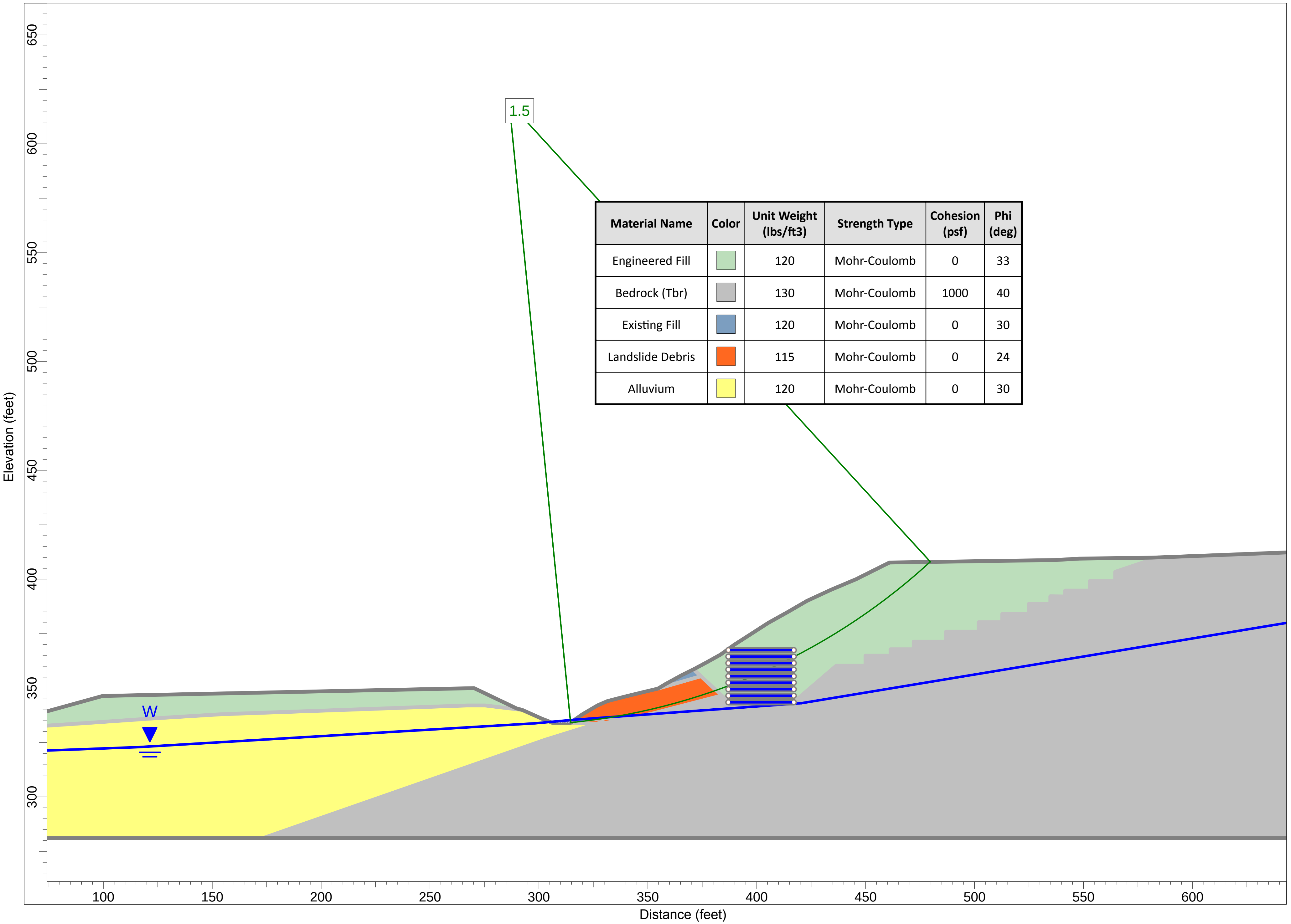
Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Engineered Fill		120	Mohr-Coulomb	500	23
Bedrock (Tbr)		130	Mohr-Coulomb	1000	40
Existing Fill		120	Mohr-Coulomb	0	30
Colluvium		120	Mohr-Coulomb	1000	0

Section 3-3' - Seismic Slope Stability



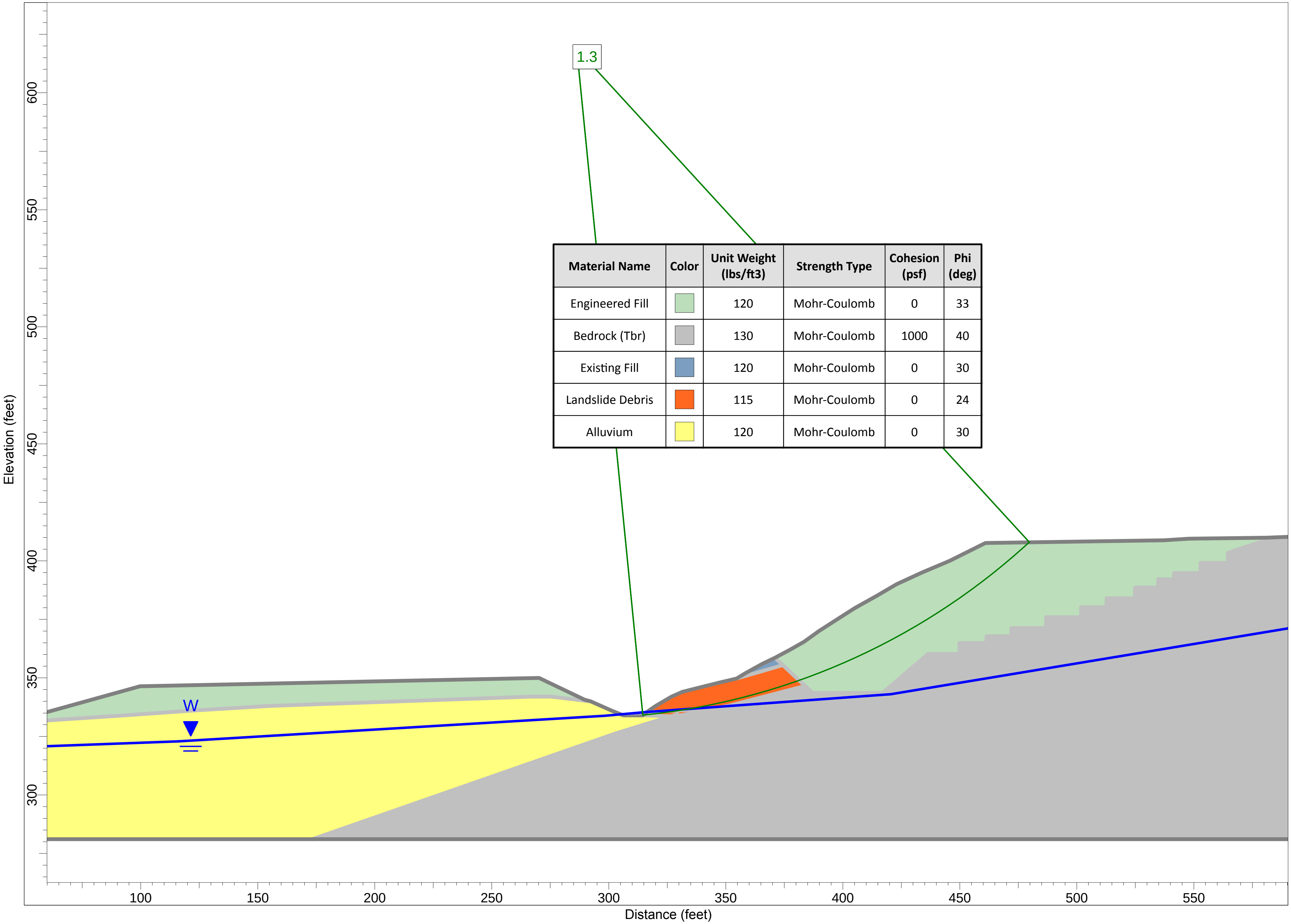
Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Engineered Fill		120	Mohr-Coulomb	500	23
Bedrock (Tbr)		130	Mohr-Coulomb	1000	40
Existing Fill		120	Mohr-Coulomb	0	30
Landslide Debris		115	Mohr-Coulomb	0	24
Alluvium		120	Mohr-Coulomb	1500	0

Section 3-3' - Static Slope Stability Mitigation



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Engineered Fill		120	Mohr-Coulomb	0	33
Bedrock (Tbr)		130	Mohr-Coulomb	1000	40
Existing Fill		120	Mohr-Coulomb	0	30
Landslide Debris		115	Mohr-Coulomb	0	24
Alluvium		120	Mohr-Coulomb	0	30

Section 3-3' - Static Slope Stability



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Engineered Fill		120	Mohr-Coulomb	0	33
Bedrock (Tbr)		130	Mohr-Coulomb	1000	40
Existing Fill		120	Mohr-Coulomb	0	30
Landslide Debris		115	Mohr-Coulomb	0	24
Alluvium		120	Mohr-Coulomb	0	30

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