APPENDIX M

GEOLOGY AND SOILS DATA

GEOTECHNICAL EXPLORATION

THE TERRACES OF LAFAYETTE
LAFAYETTE, CALIFORNIA

Submitted to:
Mr. David R. Baker
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3031 Stanford Ranch Road, Suite 2-310
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Prepared by:
ENGEO Incorporated

August 18, 2011
Revised September 2, 2011

Project No.
9181.100.000

- Expect Excellence -

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9181.100.000

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

Subject: The Terraces of Lafayette
Deer Hill Road
Lafayette, California

GEOTECHNICAL EXPLORATION

Dear Mr. Baker:

As requested, we completed this geotechnical exploration for the proposed Terraces of Lafayette project in Lafayette, California. The accompanying report presents our field exploration and laboratory testing with our conclusions and recommendations regarding multi-family 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 conclusions and recommendations for the proposed Terraces of Lafayette multi-family residential development in Lafayette, California. We performed the following services.

- Review of available literature, previous reports, and geologic maps for the study area.
- Subsurface exploration, consisting of six soil borings and thirty test pits.
- Laboratory testing of materials sampled during the field exploration.
- Geotechnical data 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 Grading and Drainage Plan prepared by BKF (March 21, 2011), 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 grading plan prepared by BKF, dated March 21, 2011, (Project No. 20115003) shows the development of 14 multi-unit apartment buildings, club house, swimming pool and appurtenant street access, 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 50 feet deep and fills up to approximately 40 feet thick, with graded slopes up to 50 feet high at inclinations of approximately 2:1 (horizontal:vertical) or flatter. We anticipate two- to three-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 ranging from 1928 to 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. Our previous study included a review of geologic literature and maps, 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 investigation was undertaken for the preparation of the preliminary report. The laboratory analyses from this study are presented in Appendix B. The previous study concluded that the proposed residential development of the property is feasible 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. Below are descriptions of the geologic units encountered during our exploration of the site (Figure 3).

2.1.2.1 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.

2.1.2.2 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 others 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.

2.1.2.3 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, very stiff, silty clay.

2.1.2.4 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 silty clay and sandy clay. Two PI tests were performed on this unit that resulted in a PI range of 30 to 41.

2.1.2.5 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.

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 2010 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 our 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 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 six exploratory borings to a maximum depth of approximately 51½ feet below existing grade at the approximate locations 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 method 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. 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 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 one 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 LABORATORY TESTING

Select samples recovered during drilling activities were tested to determine various soil characteristics as presented on the following table.

TABLE 3.2-1
Laboratory Testing

Soil Characteristic	Testing Method	Location Of Results
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.

3.3 SUBSURFACE CONDITIONS

Borings B-1 and B-2 were drilled in the area mapped as alluvium in the northeastern portion of the site. A thin 1-foot-thick layer of fill consisting of silty clay and wood chips was encountered at the ground surface in B-1. The alluvium encountered in B-1 and B-2 comprised stiff to very stiff silty clay within the upper 5 to 8 feet overlying stiff to very stiff sandy clay. Two samples from the alluvium were tested for PI and yielded PIs of 30 and 41.

Boring B-3 was drilled in an area of previous cut on the upper terrace adjacent to Deer Hill Road. Sandstone was encountered from the ground surface to the bottom of the boring at a depth of 20.5 feet below existing grade. Very slow drilling conditions (near refusal) were encountered near the bottom of the boring.

Borings B-4 and B-5 and several of the test pits were drilled and excavated in the two southern swales adjacent to the Caltrans right-of-way. Rocky fill consisting of sandstone fragments within a silty clay and silty sand matrix was encountered to a depth of approximately 10 to 16 feet below existing grade in these areas. In general, the fill was layered, moist, dense to very dense or very stiff to hard. Roughly five to eight feet of colluvium was encountered below the fill. The colluvium was generally moist, stiff to very stiff silty clay overlying sandstone bedrock.

Boring B-6 and Test Pits TP-25 and TP-26 were advanced and excavated in an area of a mapped landslide in the northern portion of the site (Figure 3). Landslide debris, generally comprising moist, very stiff to hard silty clay, was encountered to a depth of approximately 15 feet in this area. Bedrock comprising interbedded siltstone and sandstone was encountered below the landslide debris.

Consult Figure 3 and the boring and test pit logs for specific subsurface conditions at each location. We include our exploration logs in Appendix A. The logs describe the soil type, color, consistency, and visual classification in general accordance with the Unified Soil Classification System (USCS). The logs graphically depict the subsurface conditions encountered at the time of the exploration.

3.4 GROUNDWATER

Groundwater was encountered in the two northeasternmost borings (B-1 and B-2) at a depth of approximately 13 to 14 feet below existing grades. Groundwater was also encountered at a depth of 4 feet in Test Pit TP-8. 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 the exploration and laboratory test results, it is our opinion, from a geotechnical standpoint, that the site is feasible for construction of the proposed multi-family residential development. The site was evaluated 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 and tsunamis and seiches 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 2007 California Building Code (CBC) requirements, as a minimum. Seismic design provisions of current building codes generally prescribe minimum lateral forces, applied statically to the structure, combined with the gravity forces of dead-and-live loads. The code-prescribed lateral forces are generally considered to be substantially smaller than the comparable forces that would be associated with a major earthquake. Therefore, structures should be able to: (1) resist minor earthquakes without damage, (2) resist moderate earthquakes without structural damage but with some nonstructural damage, and (3) resist major earthquakes without collapse but with some structural as well as nonstructural damage. Conformance to the current building code recommendations does not constitute any kind of guarantee that significant structural damage would not occur in the event of a maximum magnitude earthquake; however, it is reasonable to expect that a well-designed and well-constructed structure will not collapse or cause loss of life in a major earthquake (SEAOC, 1996).

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 in this area and collecting soil samples. As described in Section 3.3 above, 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.

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 two critical slopes for slope stability analyses Cross Sections 1-1 and 2-2)', Cross Section 1-1 is at the location of the highest and steepest proposed fill slope. Figure 3 shows the location of Cross Section 1-1' and the profile of Cross Section 1-1' is included on Figure 5. A conservative groundwater table was assumed at roughly 5 to 20 feet below existing grade depending on location. For pseudo-static stability analyses we used a seismic coefficient of 0.2g which is one-half of the design peak ground acceleration (PGA) determined in accordance with the 2010 CBC.

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 strength parameters for engineered fill. The sample was compacted to 90 percent relative compaction at 2 percentage points above optimum moisture content. Shear strength parameters for the existing fill placed as part of the Highway 24 and Deer Hill Road improvements were estimated from SPT blow counts obtained from our test borings drilled as part of this study. Shear strength parameters for the colluvium were estimated from data published by Stark and Eid (1997) using Liquid Limit and clay contents The sandstone bedrock material was modeled using the Generalized Hoek-Brown shear-normal function.

TABLE 4.2.2-1
Summary ShearStrength Parameters

Material	Effective-Stress Strength Parameters		Pseudo-Static Strength Parameters	
	Friction Angle (deg)	Cohesion (psf)		
Existing Fill (Qaf)	30	0	30	0
Engineered Fill (Qf, proposed)	33	0	33	0
Colluvium (Qc)	30	0	0	1,500
Bedrock (Tbr)	0	3,000	0	3,000

4.2.3 Results of Slope Stability Analyses

Appendix C shows the results of our static stability for sections Cross Sections 1-1' and 2-2'. The results are summarized in Table 4.2.3-1. The analyses indicate factors of safety above commonly accepted criteria.

TABLE 4.2.3-1
Static Stability

Section	Min Static FS	Min Psuedo-Static FS	Seismic Yield Coefficient (g)
1-1	1.6	1.1	0.22
2-2	2.6	1.6	0.52

4.2.4 Seismic Slope Deformation Analyses

We used the Bray and Travasarou (2007) simplified Newmark analysis method to estimate seismically induced deformation for the slope shown on Cross Section 1-1' based on the seismic yield coefficient obtained from pseudo-static analyses. We used the 2010 CBC Design Spectral Acceleration Values and a Moment Magnitude of 7.3 for the Hayward-Rogers Creek fault in our estimates.

Based on our deformation analysis, the calculated seismic deformation for the top of Cross Section 1-1 is approximately 9 inches. This estimated deformation is the median 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 our experience, this range of movement could potentially result in moderate settlements or ground cracking. The potential for ground deformation can be reduced by mitigation measures such as placement of geogrid layers in fills slopes. Seismic slope displacements will likely diminish with distance from the tops of planned fill slopes. Therefore, we recommend that a minimum setback of 15 feet, or one third the total height of the slope, whichever is greater, from the top of planned fill slopes be established for habitable structures. The total slope height includes the additive height of existing plus proposed final graded slopes. Setbacks on fill slopes could be reduced to a minimum of 15 feet if slopes are constructed with geogrid reinforcement designed for the specific slope condition. In addition, we recommend that structures be supported on relatively rigid foundations in order to reduce the potential for damage to structures in these areas.

4.3 EXPANSIVE SOIL

We observed potentially expansive fat clay near the surface of the site in Boring B-1, and from a surface sample collected as part of our preliminary geotechnical feasibility report. Our laboratory testing indicates that the soils and bedrock at the site exhibit low to high shrink/swell potential with variations in moisture content.

Expansive soils change in volume with changes in moisture. They can shrink or swell and cause heaving and cracking of slabs-on-grade, pavements, and structures founded on shallow foundations. Building damage due to volume changes associated with expansive soils can be reduced by: (1) using a rigid mat foundation that is designed to resist the settlement and heave of expansive soil, (2) deepening the foundations to below the zone of moisture fluctuation, i.e., by using deep footings or drilled piers, and/or (3) using mat or footings at normal shallow depths but bottomed on a layer of select fill having a low expansion potential. Expansive soil mitigation recommendations are presented in Sections 5.11 and 6.0 of this report.

Post-tensioned mat foundations are the preferred foundation system for the residential structures. Design criteria for this foundation type are presented in the following sections.

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.

4.4 EXISTING FILLS AND COMPRESSIBLE SOIL

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.

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, we should be retained to review final grading and site improvement plans, and to observe and test all earthwork construction at the site.

Existing fills could undergo vertical movement that is not easily characterized and could ultimately be inadequate to effectively support the proposed building loads. Based on the proposed development plan, some building pads will be situated in areas where existing fills were observed. In general, existing fills should be overexcavated and replaced as engineered fill. Recommendations for engineered fill placement are presented in a subsequent section.

4.5 SHALLOW GROUNDWATER

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. Temporary dewatering should be considered.

4.6 2010 CALIFORNIA BUILDING CODE SEISMIC PARAMETERS

The site has varying soil and bedrock conditions, which can be generally classified as Site Class D in accordance with the 2010 CBC. The following 2010 California Building Code (CBC) seismic design parameters should be used for design.

TABLE 4.6-1
2010 CBC Seismic Information

Parameter	Design Value
Site Class	D
0.2 second Spectral Response Acceleration, S_S	1.5
1.0 second Spectral Response Acceleration, S_1	0.6
Site Coefficient, F_A	1.0
Site Coefficient, F_V	1.5
Maximum considered earthquake spectral response accelerations for short periods, S_{MS}	1.0
Maximum considered earthquake spectral response accelerations for 1-second periods, S_{M1}	0.9
Design spectral response acceleration at short periods, S_{DS}	1.0
Design spectral response acceleration at 1-second periods, S_{D1}	0.6

4.7 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. The results of the tests are summarized below.

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, Sections 4.2 and 4.3) for concrete requirements. ACI Tables 4.2.1 and 4.3.1 provide the following sulfate exposure categories and classes and concrete requirements in contact with soil based upon the exposure risk.

TABLE 4.7-1
Sulfate Exposure Categories and Classes

Sulfate Exposure Category S	Exposure Class	Water- Soluble Sulfate in Soil	Dissolved Sulfate in Water
		% by Weight	mg/kg (ppm)
Not Applicable	S0	$SO_4 < 0.10$	$SO_4 < 150$
Moderate	S1	$0.10 \leq SO_4 < 0.20$	$150 \leq SO_4 \leq 1{,}500$, seawater
Severe	S2	$0.20 \leq SO_4 \leq 2.00$	$1{,}500 \leq SO_4 \leq 10{,}000$
Very Severe	S3	$SO_4 > 2.00$	$SO_4 > 10{,}000$

TABLE 4.7-2
Requirements for Concrete by Exposure Class

Exposure Class	Max w/cm	Min f'c (psi)	Cement Type			Calcium Chloride Admixture
			ASTM C150	ASTM C595	ASTM C1157	
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 by the CBC 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.

4.8 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 that may require considerable ripping effort and generate oversized material (greater than six 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.

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

4.9 CONCLUSIONS

From a geologic and geotechnical standpoint, the study area appears to be suitable for multi-family residential development. As discussed above and based on this geotechnical exploration and review of previous studies, the main geologic/geotechnical issues to be addressed at the site include the following. The recommendations in subsequent sections consider the hazards and concerns listed below.

- Slope stability
- Existing fill
- Expansive soils
- Shallow groundwater

5.0 EARTHWORK RECOMMENDATIONS

The recommendations included in this report, along with other sound engineering practices, should be incorporated in the design and construction of the project.

5.1 GRADING

The following grading recommendations are provided for the project based upon the current plan prepared by prepared by BKF and LCA Architects (March 21, 2011). 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 meet the requirements of the Guide Contract Specifications included in Appendix D and 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 engineered fill. Other materials and debris, including trees with their root balls, should be removed from the project site.

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 and should conform to the requirements provided in the Guide Contract Specifications (Appendix D).

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

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.

5.5 TOE KEYWAYS

Construction of subdrained toe keyways will be required at the toes of proposed fill slopes to mitigate potential slope stability hazards. We anticipate that typical keyway designs will consist of 18- to 24-foot-wide keyways constructed to a minimum depth of 5 to 25 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.

Actual subsurface mitigation configurations (size and depths) will be shown on the final 40-scale remedial grading plans and after additional detailed slope stability analyses have been performed, as applicable. Fills should be adequately keyed/benched into competent material or bedrock materials, as determined by the Geotechnical Engineer during fill slope construction. The actual depth and location of the keyways, subexcavated benches, and locations of subdrainage may then be slightly modified in the field by the Geotechnical Engineer, based on the 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 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.

The recommended locations of the subdrains will be approximately located on the remedial grading plans used during site grading; however, general details are presented 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. The subdrain pipe and drainage blanket should meet the requirements contained in Section 2.05, Part I of the Guide Contract Specifications. As an alternative, prefabricated geocomposite drainage material (such as SKAPS TNS 220-6) could be considered in lieu of the granular medium above the subdrain zone.

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

5.7 GRADED SLOPES

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

TABLE 5.7-1
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 low-plasticity bedrock. The fill slope criteria provided in Table 5.7-1 are based on the assumption that the fill material will be derived from low-plasticity bedrock or low-plasticity granular soil. Low-plasticity bedrock and soil is defined here as material with a PI 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. 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. Additionally, cut-fill transition slopes should be overexcavated and reconstructed as engineered fill slopes.

In accordance with the 2010 CBC requirements, we recommend that slopes with inclinations greater than 3:1 be graded with terraces at least 6 feet in width at not more than 30-foot vertical intervals.

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

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.

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.

5.8 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 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 if the slope is provided with geogrid reinforcement designed for the specific slope condition. For structures adjacent to cut slopes, we recommend a minimum setback of 15 feet from the top of slope.

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

5.9 CUT, FILL, AND CUT-FILL TRANSITION LOTS

We recommend that the upper 2 feet of subgrade soils be made uniform by subexcavating and replacing 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. Moisture and compaction recommendations are provided in a subsequent section of this report.

5.10 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 that a differential fill thickness of up to 10 feet is acceptable across individual building pads. 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 that the material is replaced 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.

5.11 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 and subexcavated backfill areas:

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.

5.12 MONITORING AND TESTING

It is important that all site preparations for site grading be done 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, following the recommendations contained in the Guide Contract Specifications in Appendix D. The final grading plans should be submitted to the Geotechnical Engineer for review.

6.0 FOUNDATION RECOMMENDATIONS

The primary considerations for foundation design at the site will be the potential for differential settlement of fill and settlement from earthquake-induced ground displacements of fill slopes. The effect of differential settlement can be reduced by establishing slope setbacks as described in Section 5.8, remedial grading, proper fill placement, and by the choice of a proper foundation system. In order to reduce the effects of differential settlement, the foundation can be designed to be sufficiently stiff to move as a rigid unit. For level building pads with structures set back at least 15 feet or ⅓ of the slope height, whichever is greater, from the tops of fill slopes, a heavily reinforced conventional structural mat or post-tensioned mat foundation bearing on prepared native soil/bedrock or compacted fill would be suitable for support of the proposed multi-family residential structures. Alternatively, it would be suitable to support the proposed structures adjacent to fill slopes on heavily reinforced continuous or interconnected footings founded in native soil/bedrock or compacted fill. For level building pads with structures set back at least 15 feet from the tops of cut slopes, a conventional footing with slab-on-grade foundation bearing on prepared native bedrock or compacted fill would be suitable for support of the proposed multi-family residential structures.

The foundation design should consider a 2-inch total settlement. A differential value of 1 inch may be considered for design and should be assumed to act between adjacent column supports or over a 30-foot distance.

6.1 CONVENTIONALLY REINFORCED STRUCTURAL MAT FOUNDATIONS

Conventionally reinforced structural mat foundations for buildings or portions of buildings situated at least 40 feet from the top of an engineered fill slope should be designed for a 3-foot edge-cantilever distance and a 10-foot unsupported interior-span distance provided the soil within 5 feet of finished pad grade, whether fill or native, is non-expansive, defined here as a PI of 12 or less; these structural mats should have a minimum thickness of 10 inches and be thickened to at least 12 inches at the perimeter. Conventionally reinforced structural mats for buildings or portions of buildings within 40 feet of the top of an engineered fill slope should be designed for a 10-foot edge-cantilever distance and a 25-foot unsupported interior-span distance. These structural mats should have a minimum thickness of 12 inches and be thickened to at least 14 inches at the perimeter.

Conventionally reinforced structural mats should be reinforced with top and bottom steel as determined by the structural engineer to provide structural continuity and permit spanning of local irregularities. Mat foundations should be designed to accommodate differential movement without experiencing structural distress to the slabs or excessive deflections in the framing and wall finishes. Mat foundations should be designed for an allowable uniform soil pressure of 1,000 pounds per square foot (psf), 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. Provided the site earthwork is conducted in accordance with the recommendations of this report, a subgrade modulus of 150 psi/in can be used for structural slab design. The thickened edge of the mat should have a minimum width of 12 inches. The minimum backfill height of soil against the mat at the perimeter should be 6 inches. The resistance to lateral loads should be computed using a base friction factor of 0.35 acting between the bottom of the mat and subgrade.

The recommendations for slab moisture vapor reduction in Section 7.0 should be used when water vapor migrating through the slab would be undesirable. The recommendations will reduce, but not stop, upward water vapor transmission through the mat foundations.

6.2 POST-TENSIONED MAT FOUNDATIONS

Post-tensioned (PT) mat foundations for buildings not adjacent to planned fill slopes should be designed using the criteria presented in Table 6.2-1 below provided the soil within 5 feet of finished pad grade, whether fill or native, is non-expansive, defined here as a PI of 12 or less. These structural 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 6.2-1
Post-Tension Design Criteria

Condition	Center Lift	Edge Lift
Edge Moisture Variation Distance, e_m (feet)	9.6	4.6
Differential Soil Movement, y_m (inches)	0.3	0.7

PT mats for buildings located within 40 feet of the top of an engineered fill slopes should be designed using the criteria provided in Table 10 and additionally be able to cantilever for a distance of 10 feet at the perimeter and span 25 feet within interior areas. These PT mats should have a minimum thickness of 12 inches and be thickened to at least 14 inches at the perimeter and should be designed to accommodate differential movement without experiencing structural distress to the slabs or excessive deflections in the framing and wall finishes.

6.3 CONVENTIONAL FOOTINGS WITH SLAB-ON-GRADE

The proposed structures situated at least 40 feet from the top of an engineered fill slope or at least 15 feet from the top of a cut slope can be supported on shallow continuous footings founded in prepared native soil or compacted fill. However, for this foundation type, soil materials, whether fill or native, within 5 feet of finished pad grade should be non-expansive, defined here as a PI of 12 or less.

6.3.1 Footing Dimensions and Allowable Bearing Capacity

We provide the minimum footing dimensions as follows in the Table 6.3.1-1 below. The footings should be interconnected.

TABLE 6.3.1-1
Minimum Footing Dimensions

Footing Type	*Minimum Depth (inches)	Minimum Width (inches)
Continuous	24	18
Isolated	24	18

*below lowest adjacent pad grade

Minimum footing depths shown above are taken from lowest adjacent pad grade.

Foundations should be designed for a maximum allowable bearing pressure of 2,500 pounds per square foot (psf) for dead plus live loads. This bearing capacity can be increased by ⅓ for the short-term effects of wind or seismic loading.

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

6.3.2 Waterstop

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

6.3.3 Reinforcement

The structural engineer should design footing reinforcement to support the intended structural loads without excessive settlement. All footings should be interconnected and should be reinforced with top and bottom steel to provide structural continuity and to permit spanning of local irregularities.

6.3.4 Foundation Lateral Resistance

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

- Passive Lateral Pressure: 300 pcf
- Coefficient of Friction: 0.35

7.0 INTERIOR SLABS-ON-GRADE

7.1 SLAB MOISTURE VAPOR REDUCTION

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

1. Construct a moisture retarder system directly beneath the slab on-grade that consists of the following:

    a. Vapor retarder membrane sealed at all seams and pipe penetrations and connected to all footings. Vapor retarders shall conform to Class A vapor retarder per ASTM E 1745-97 "Standard Specification for Plastic Water Vapor Retarders used in Contact with Soil or Granular Fill under Concrete Slabs". The vapor retarder should be underlain by

    b. Four inches of clean crushed rock (except for post-tensioned mats). Crushed rock should have 100 percent passing the ¾-inch sieve and less than 5 percent passing the No. 4 Sieve.

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

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

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

8.0 EXTERIOR SLABS-ON-GRADE

Exterior flatwork includes items such as concrete sidewalks, steps, and outdoor courtyards exposed to foot traffic only; provide a minimum concrete flatwork 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 (Section 2.04, Part I of Guide Contract Specifications) 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

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 9.0-1
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.

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.

Whenever possible, walls should be located at the toe of slopes, rather than at the top of slopes to create level terrain in front of the walls and terraced retaining walls are not recommended. The Geotechnical Engineer should be consulted on design values where surcharge loads, such as from streets or buildings, are expected, where a downhill slope exists below a proposed wall, or if terraced walls are planned.

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 (Part I of Guide Contract Specifications, Section 2.05B). The width of the drain blanket should be at least 12 inches, and the drain blanket should extend to about 1 foot below the finished grades. The upper 1 foot of wall backfill should consist of compacted site soils. Drainage should be collected into solid pipes and directed to an outlet approved by the Civil Engineer. Synthetic filter fabric should meet the minimum requirement listed in the Guide Contract Specifications (Appendix D) and 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.

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.

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 3 percent within 5 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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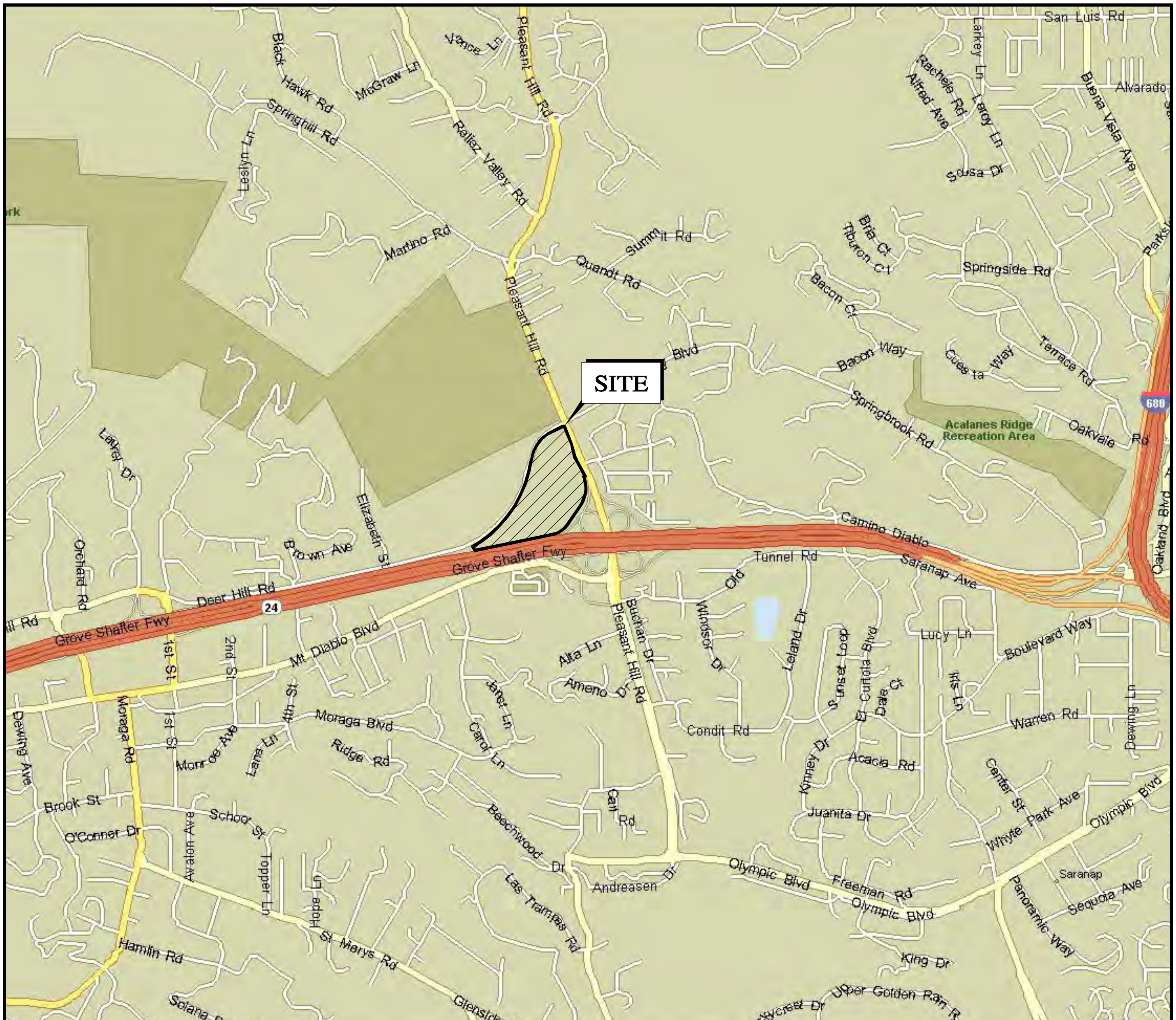
LIST OF FIGURES

FIGURES



0 FEET 2000

0 METERS 1000

BASE MAP SOURCE: MS STREETS AND TRIPS

ENGEO
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VICINITY MAP THE TERRACES OF LAFAYETTE LAFAYETTE, CALIFORNIA	PROJECT NO.: 9181.100.000		FIGURE NO.
	SCALE: AS SHOWN		1
	DRAWN BY: PC	CHECKED BY: PG	

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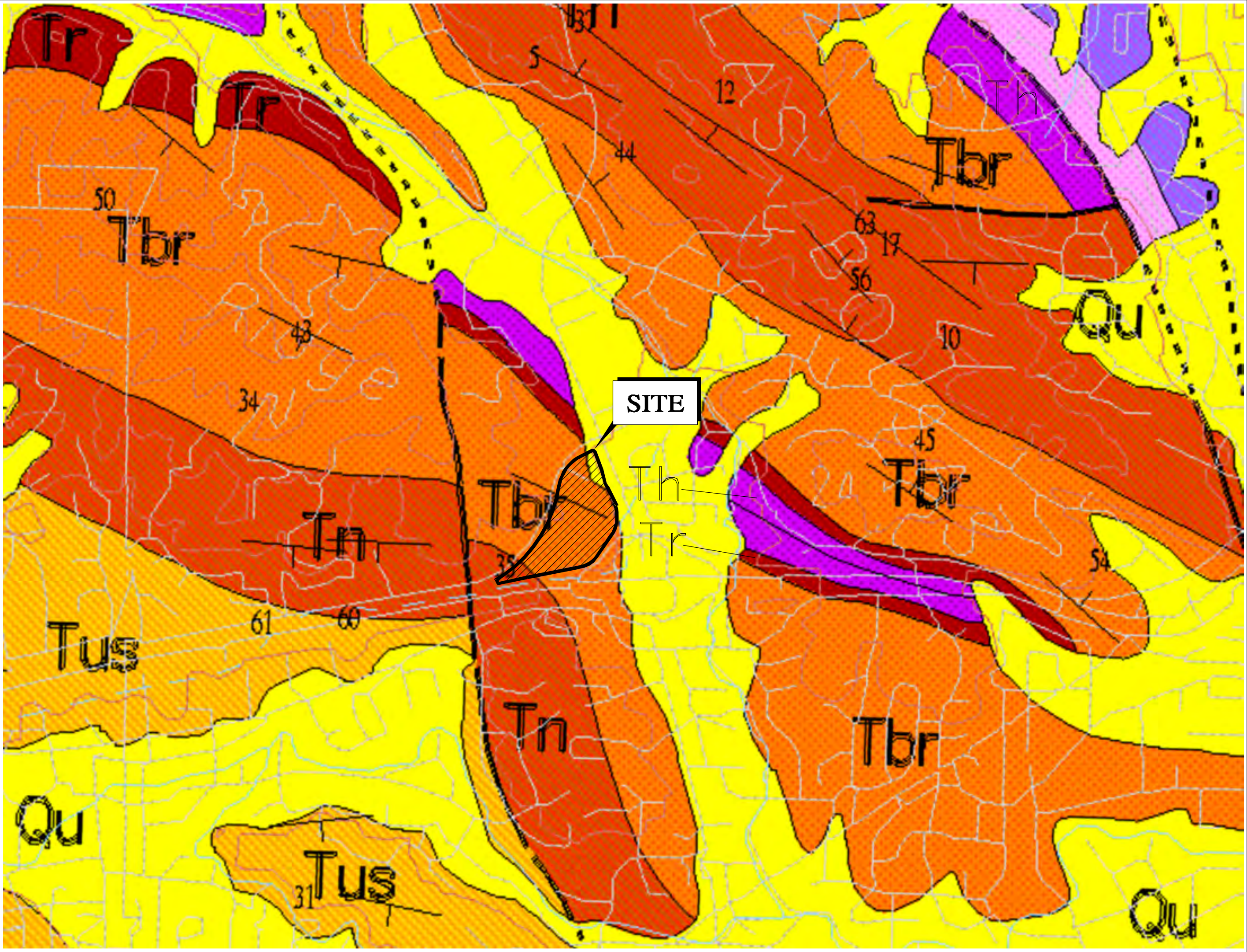

EXPLANATION

BEDROCK CONTACT-DASHED WHERE GRADATIONAL OR APPROXIMATELY LOCATED

FAULT-DASHED WHERE INFERRED, DOTTED WHERE CONCEALED, QUERIED WHERE EXISTENCE IS DOUBTFUL

STRIKE AND DIP OF STRATA

INCLINED

Qu	UNDIVIDED QUATERNARY DEPOSITS
Tus	UNNAMED SEDIMENTARY AND VOLCANIC ROCKS
Tn	NEROLY FORMATION
Tbr	BRIONES FORMATION
Tlj	LAS JUNTAS SHALE
Tr	RODEO SHALE
Th	HAMBRE SANDSTONE
Tme	MEGANOS 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.: 9181.100.000	
SCALE: AS SHOWN	
DRAWN BY: PC	CHECKED BY: PG

FIGURE NO. 2

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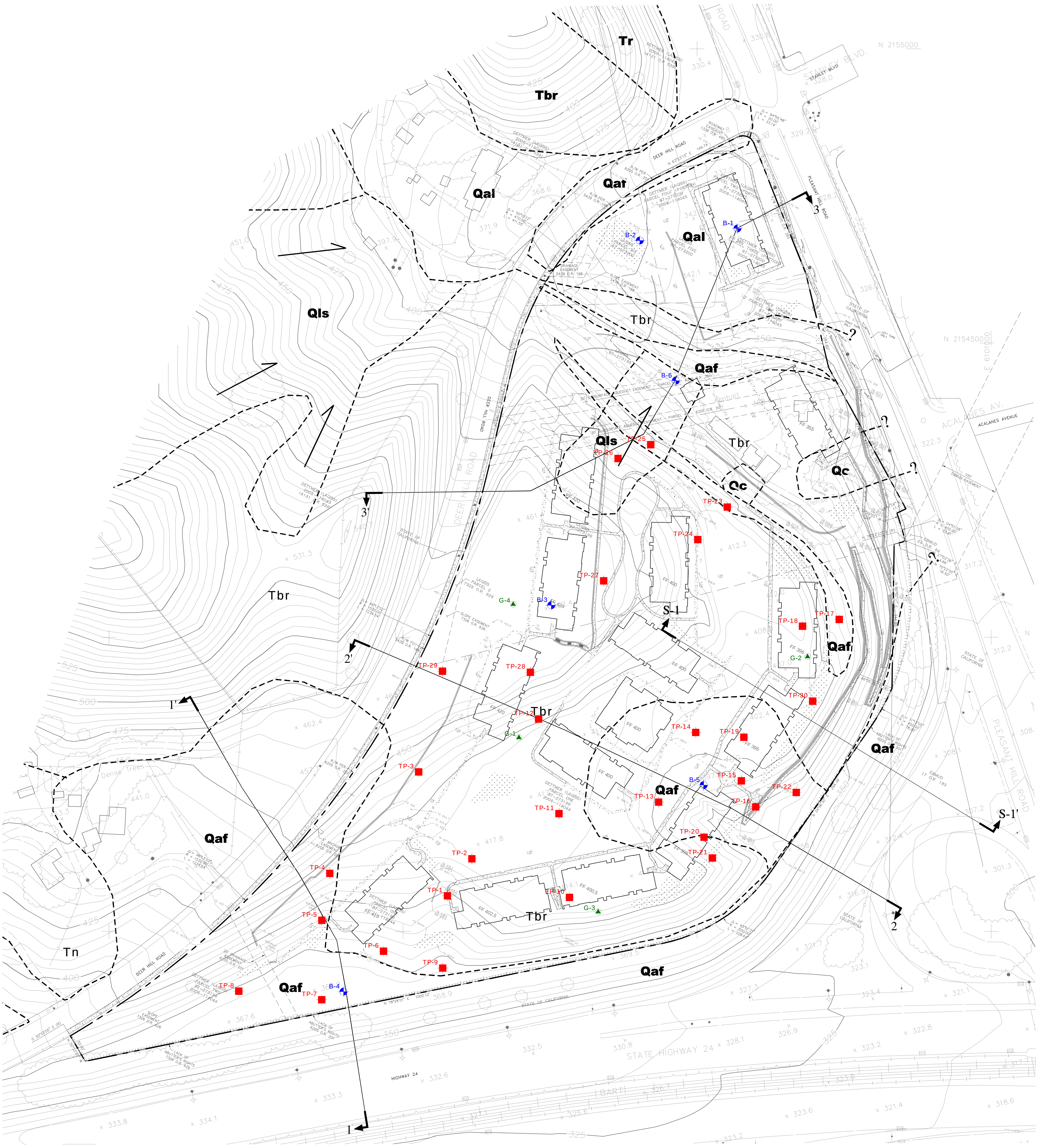


EXPLANATION

Qaf FILL

Qls LANDSLIDE

EARTHFLOW

DEEP-SEATED

Af ARTIFICIAL FILL

Qc COLLUVIUM

Tr RODEO SHALE

Tbr BRIONES SANDSTONE

Tn NEROLY FORMATION

G-4 APPROXIMATE LOCATION OF GRAB SAMPLE

TP-29 APPROXIMATE LOCATION OF TEST PIT

B-6 APPROXIMATE LOCATION OF BORING

N

0 FEET 80

0 METERS 40

BASE MAP SOURCE:

ENGEO Expect Excellence

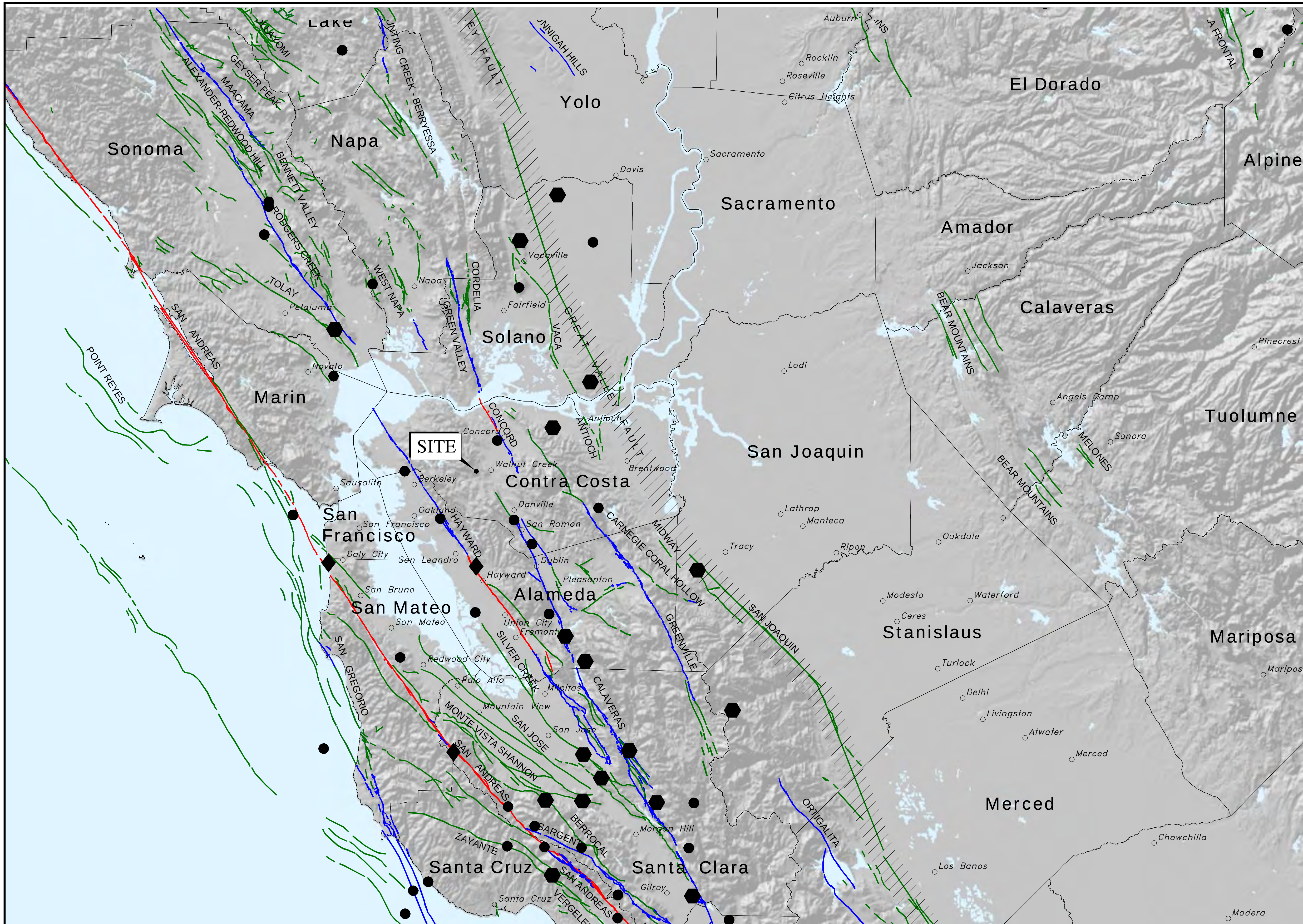
SITE PLAN AND GEOLOGIC MAP

THE TERRACES OF LAFAYETTE

LAFAYETTE, CALIFORNIA

PROJECT NO.: 9181.100.000	FIGURE NO. 3
SCALE: AS SHOWN	
DRAWN BY: PC / CHECKED BY: PG	

ORIGINAL FIGURE PRINTED IN COLOR



N

0 MILES 15

0 KILOMETERS 30

EXPLANATION

- MAGNITUDE 7+
- MAGNITUDE 6-7
- MAGNITUDE 5-6
- HISTORIC FAULT
- HOLOCENE FAULT
- QUATERNARY FAULT
- HISTORIC BLIND THRUST FAULT ZONE

BASE MAP SOURCE:
U.S.G.S. 1-ARC SECOND S.R.T.M. DATABASE
U.S.G.S. QUATERNARY FAULT DATABASE, MARCH, 2006
U.S.G.S. HISTORIC EARTHQUAKE DATABASE (1800-2000)

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REGIONAL FAULTING AND SEISMICITY
THE TERRACES OF LAFAYETTE
LAYAYETTE, CALIFORNIA

PROJECT NO.: 9181.100.000	FIGURE NO
SCALE: AS SHOWN	4
DRAWN BY: PC	CHECKED BY: PG

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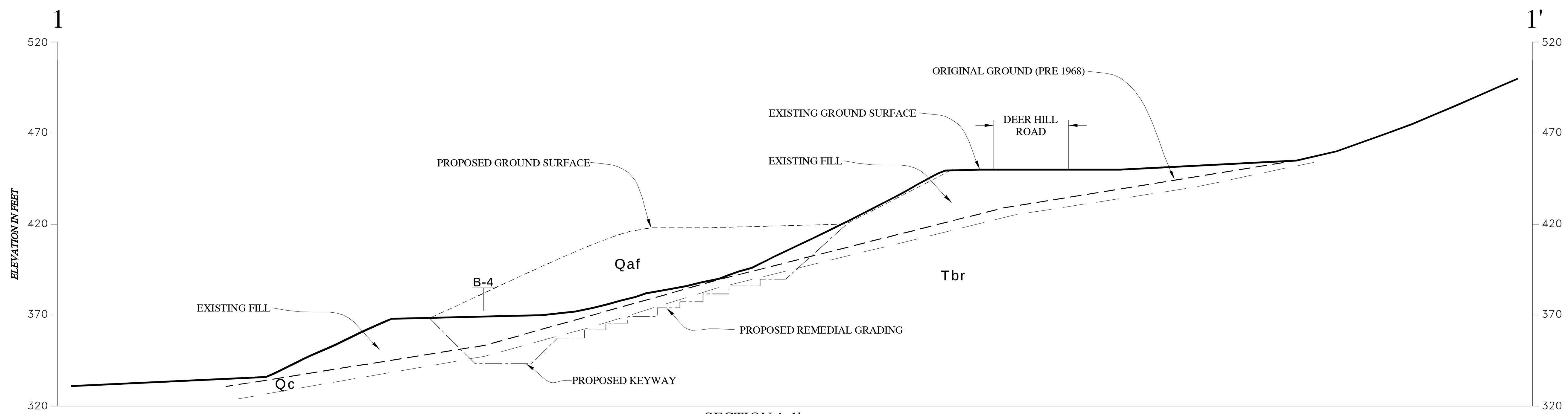
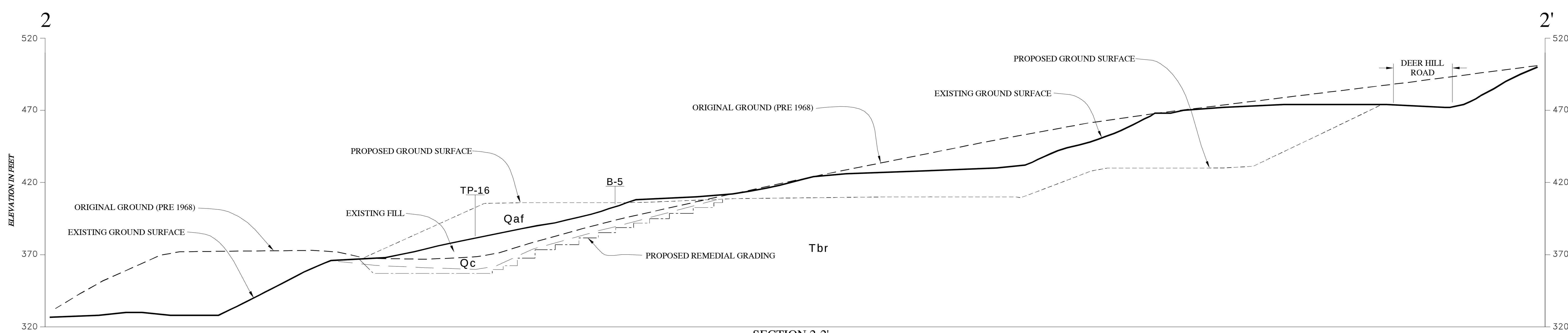
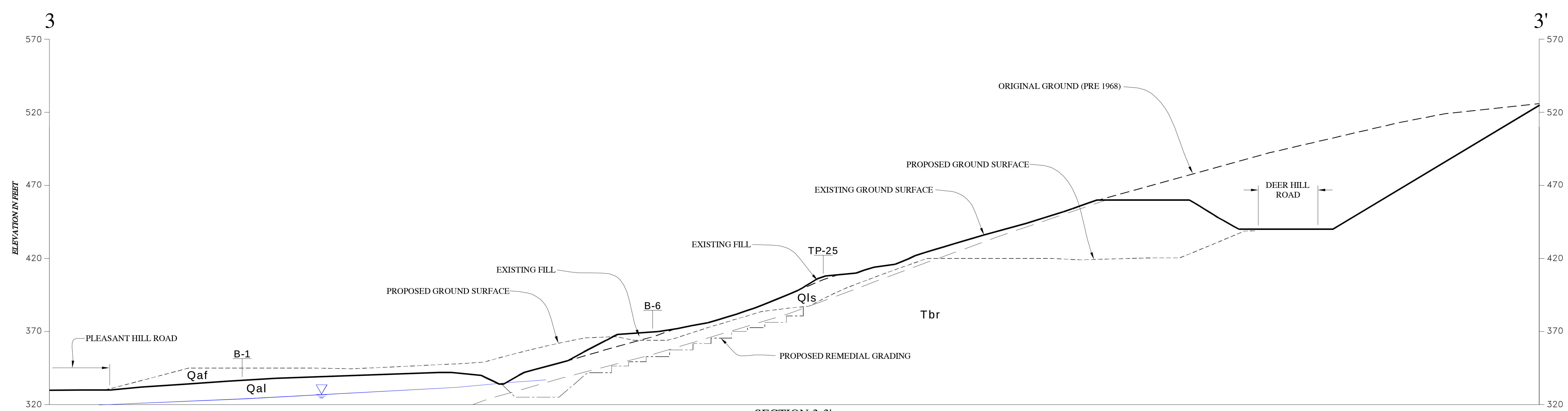


SECTION 1-1'
1"=50'



SECTION 2-2'
1"=50'



SECTION 3-3'
1"=50'

EXPLANATION

Qaf FILL
Qc COLLUVIUM
Qls LANDSLIDE
Tbr BRIONES FORMATION

ENGEO Expect Excellence

CROSS SECTIONS
THE TERRACES OF LAFAYETTE
LAFAYETTE, CALIFORNIA

PROJECT NO.: 9181.100.000
SCALE: AS SHOWN
DRAWN BY: PC
CHECKED BY: PG
FIGURE NO 5

ORIGINAL FIGURE PRINTED IN COLOR

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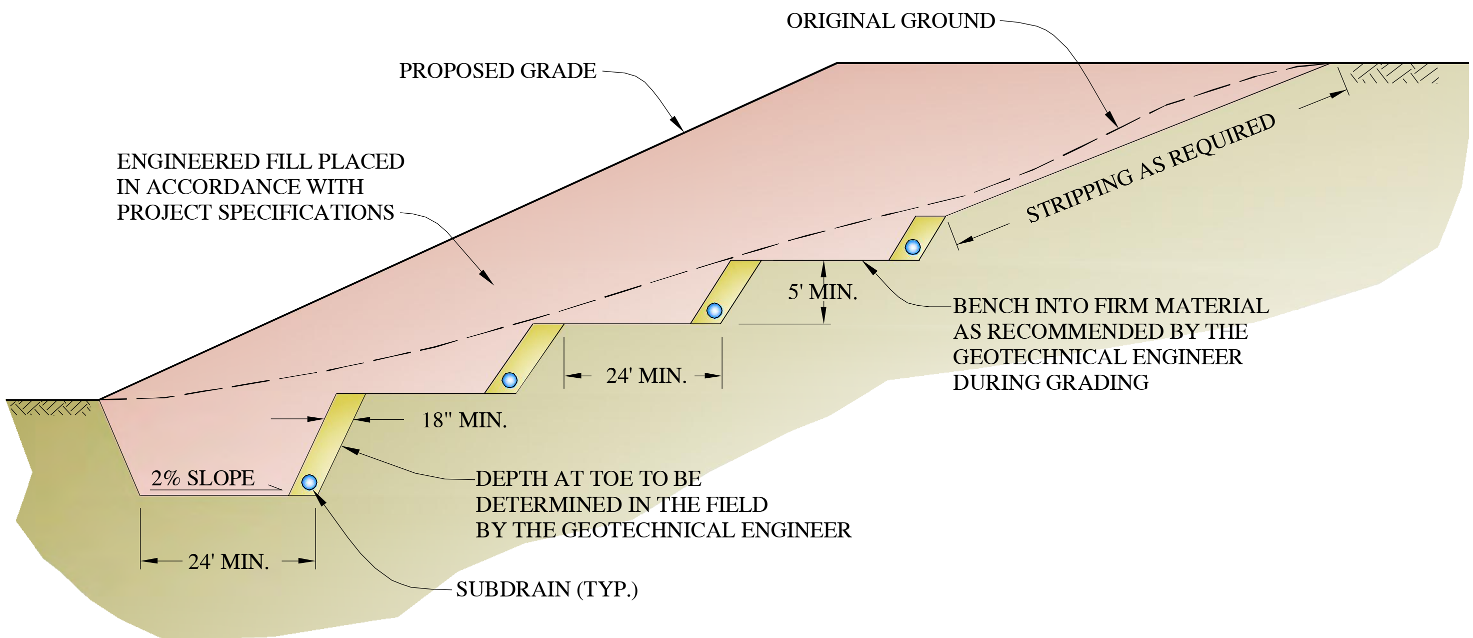

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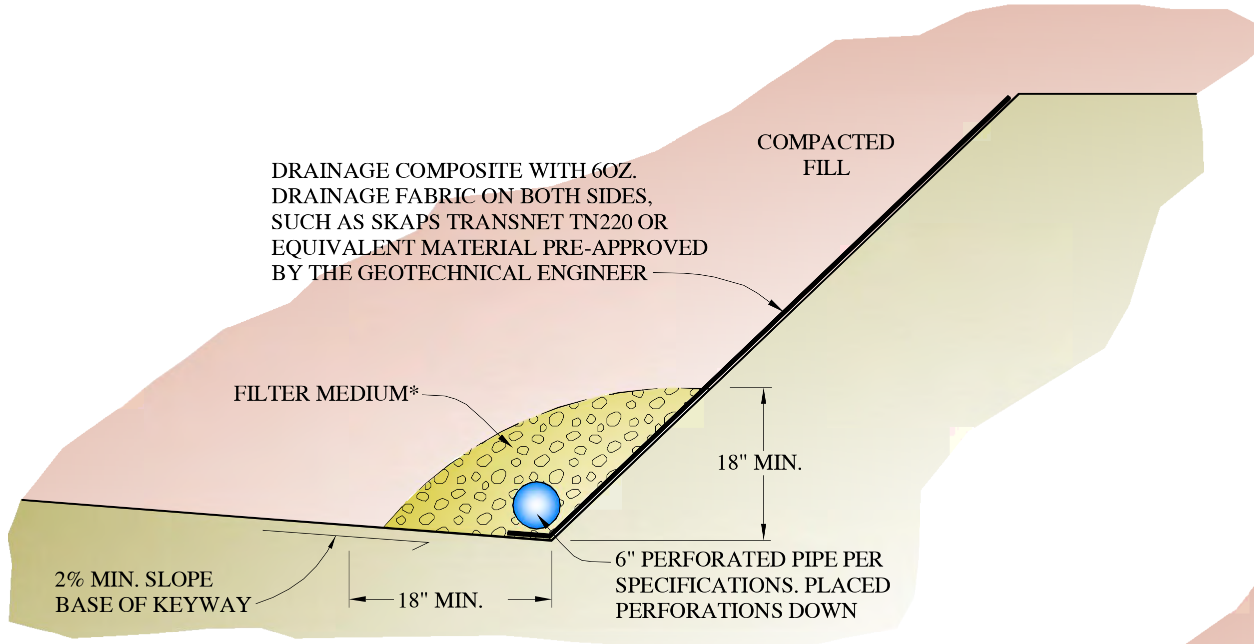
TYPICAL KEYWAY DETAIL

THE TERRACES OF LAFAYETTE

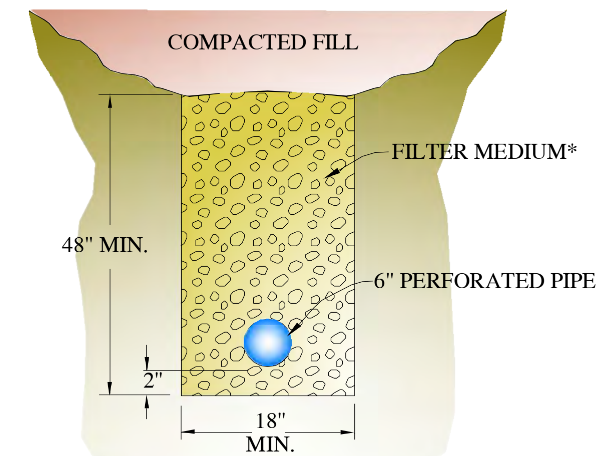
LAFAYETTE, CALIFORNIA

PROJECT NO.: 9181.100.000	FIGURE NO. 6
SCALE: NO SCALE	
DRAWN BY: PC	CHECKED BY: PG

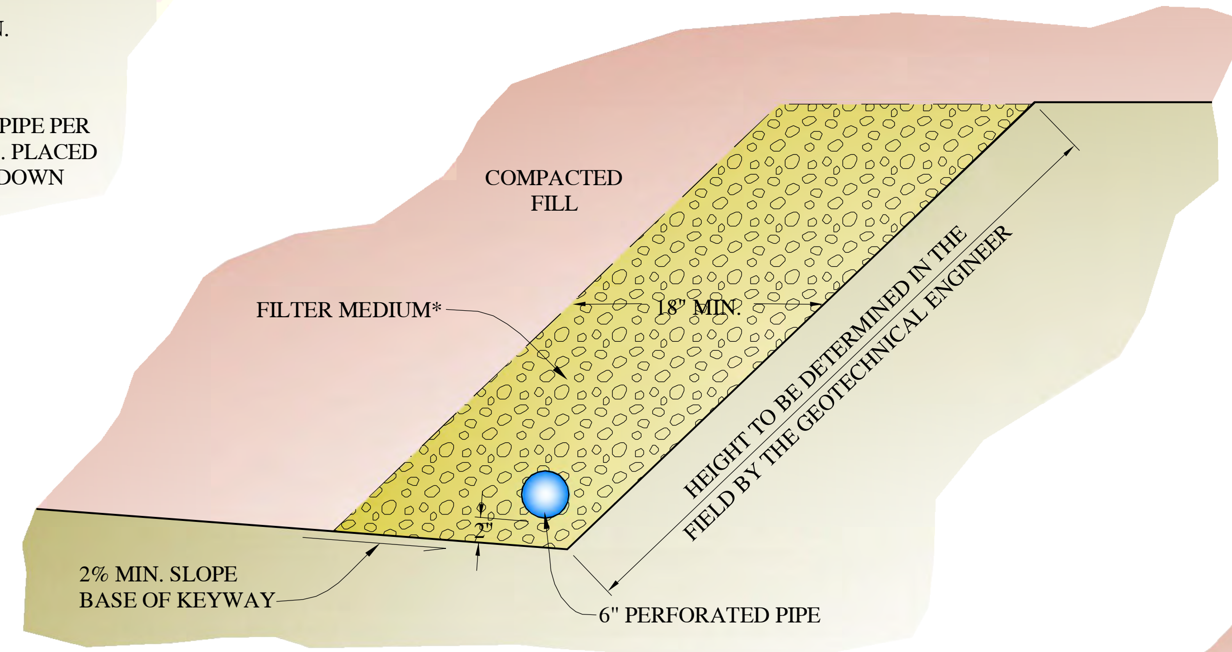
ORIGINAL FIGURE PRINTED IN COLOR



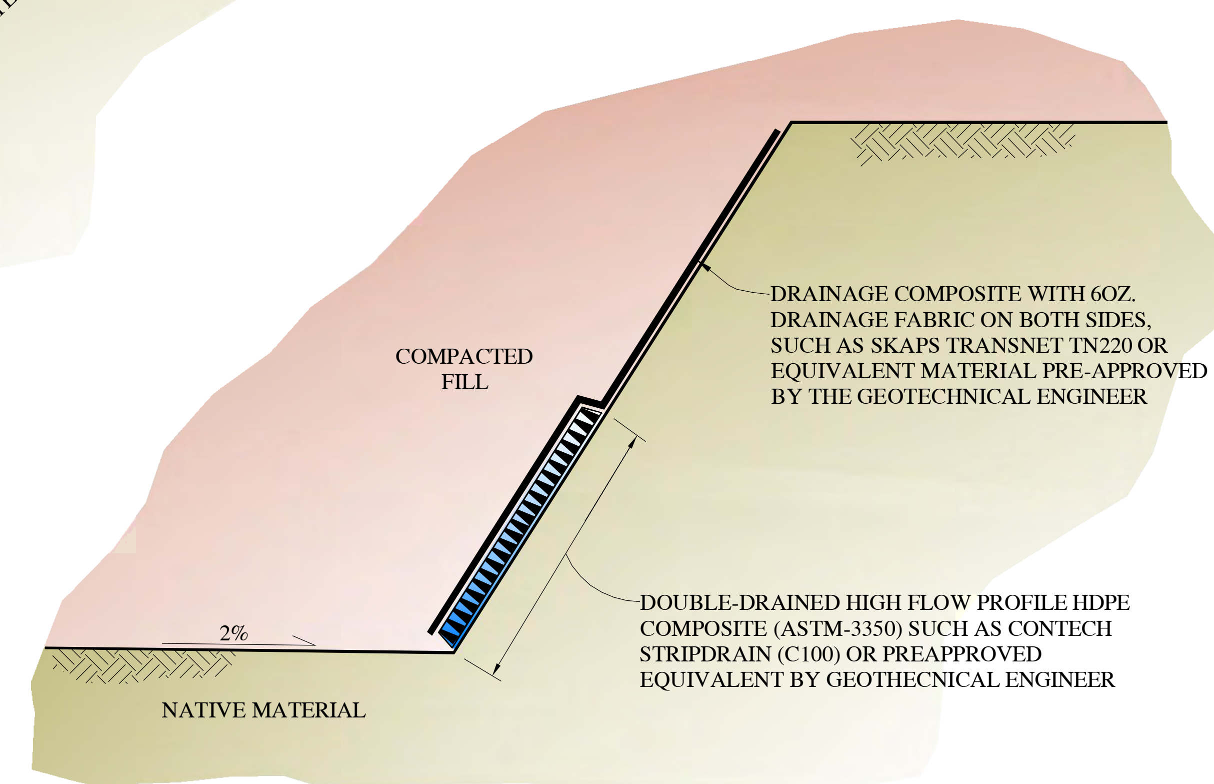
KEYWAY SUBDRAIN - OPTION 1



SWALE SUBDRAIN



KEYWAY SUBDRAIN - OPTION 2



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*

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TYPICAL SUBDRAIN DETAILS
THE TERRACES OF LAFAYETTE
LAFAYETTE, CALIFORNIA

PROJECT NO.: 9181.100.000	FIGURE NO. 7
SCALE: NO SCALE	
DRAWN BY: PC	CHECKED BY: PG

APPENDIX A

Boring Logs
Test Pit Logs
(ENGEO 2011)

APPENDIX A

LOG OF BORING B-1

ENGEO INCORPORATED

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		
	12	Brown mottled with dark gray, very stiff, with fine gravel, some manganese.									
40				16					27.7		2.5*
	13										
	14	Becomes stiff, few 1/16 to 1/8 inch sandstone fragments.		32							1.5*
	15										
50		Becomes very stiff, with subrounded to rounded sandstone fragments.		34							3.5*
		Bottom of boring at 51.5 feet, groundwater at 13 feet.									

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		Yellowish brown mottled with gray, with 1/8 to 1/4 inch sandstone fragments.		▽	34					26.5	100.4	3*
20	5, 6		Same as above.			18					28.2	95.9	2.5*
	7, 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												
						50/6"							
	2												
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 - GEOTECHNICAL 9181100000 GINT LOGS.GPJ ENGEO INC.GDT 8/18/11

ENGEO INCORPORATED

LOG OF BORING B-4

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.										

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

ENGEO INCORPORATED

LOG OF BORING B-6

Geotechnical Exploration
The Terraces of Lafayette
Lafayette, California
9181.100.000

DATE DRILLED:	6/15/2011	LOGGED / REVIEWED BY:	J. White / JBR
HOLE DEPTH:	Approx. 25½ ft.	DRILLING CONTRACTOR:	West Coast Exploration
HOLE DIAMETER:	4.0 in.	DRILLING METHOD:	Solid Flight Auger
SURF ELEV (msl):	Approx. 370 ft.	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.

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.

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.

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-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.

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-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.

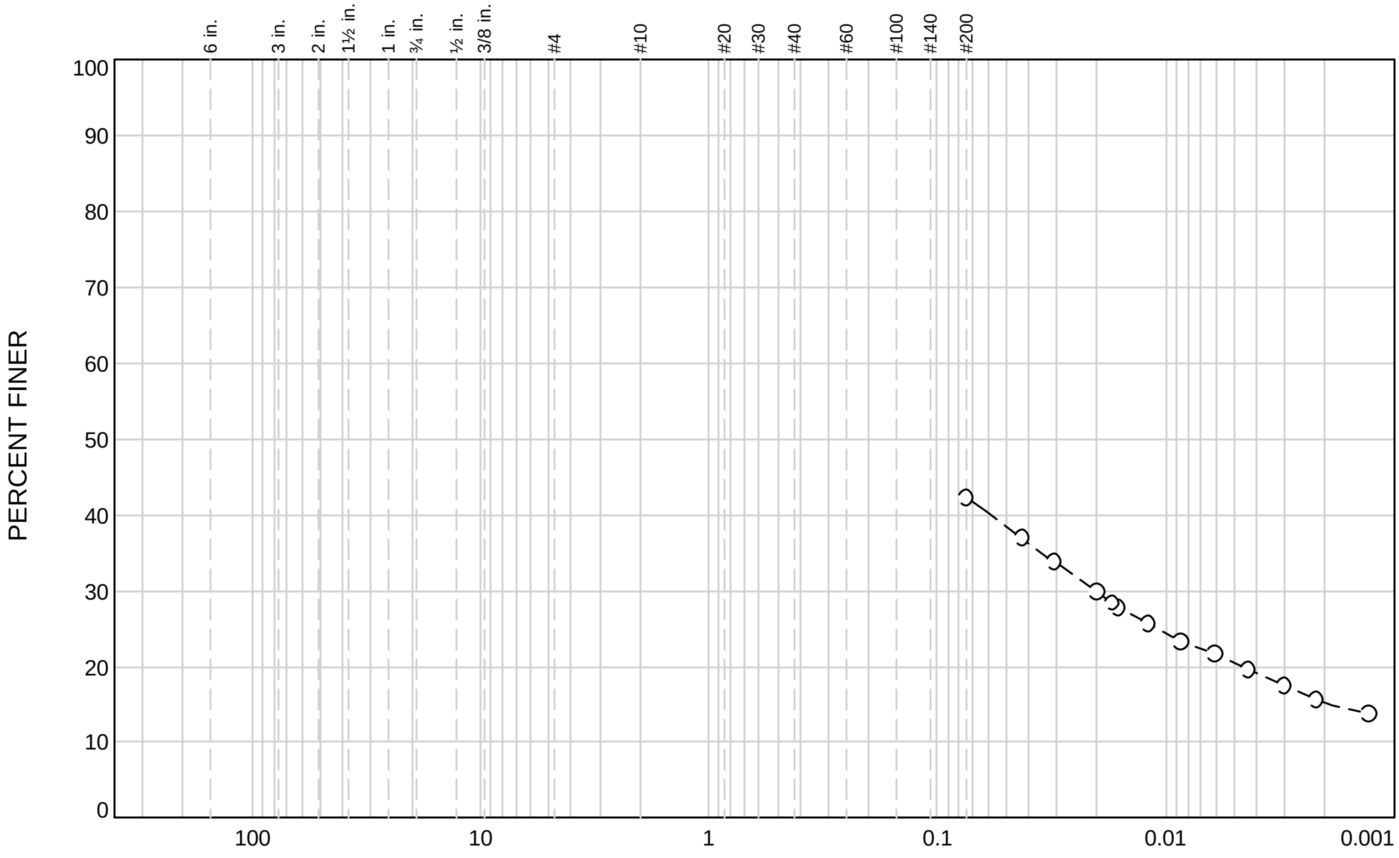
ENGEO INCORPORATED		TEST PIT LOG
Terraces at Lafayette Lafayette, California 9181.100.000		Logged By: J. White Logged Date: 6/1/11 to 6/2/11
Test Pit Number	Depth (Feet)	Description
	6 – 9	SANDSTONE, dark yellowish brown, weak closely fractured, thinly bedded, highly weathered, coarse grained. Bedding 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 2011)

APPENDIX B

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)

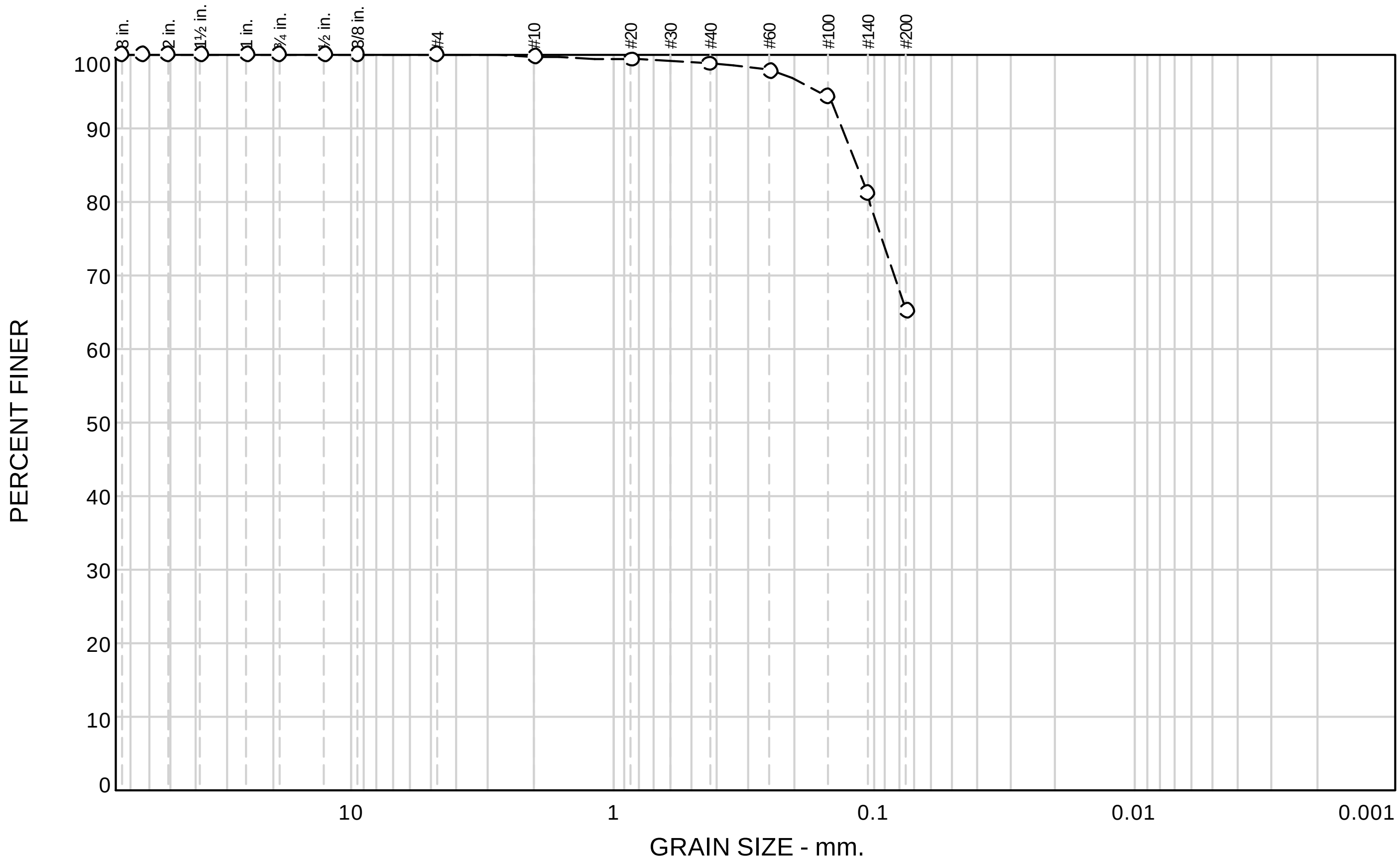
Remarks

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

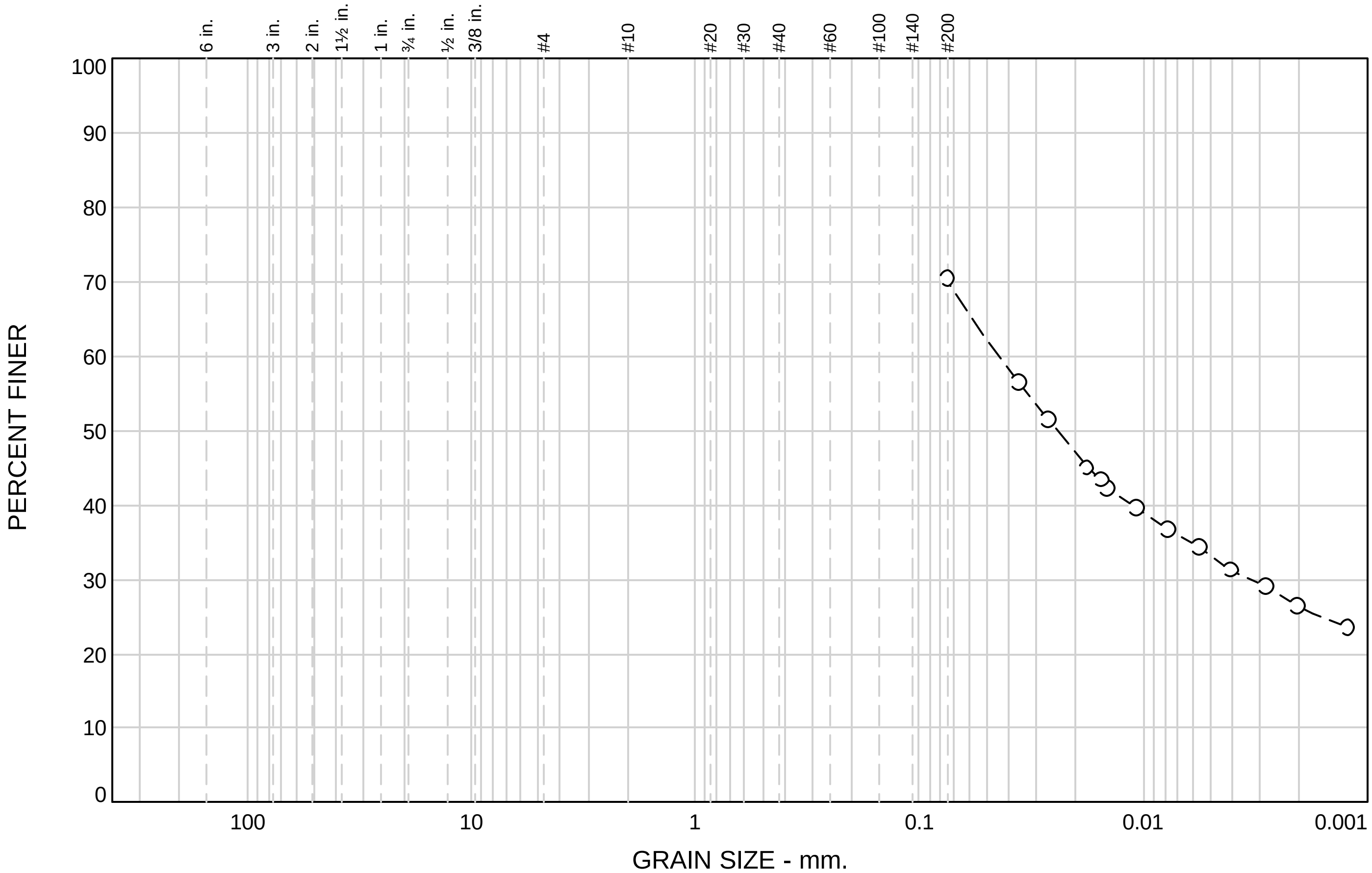
ENGEO INCORPORATED GEOTECHNICAL AND ENVIRONMENTAL CONSULTANTS MATERIALS TESTING

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

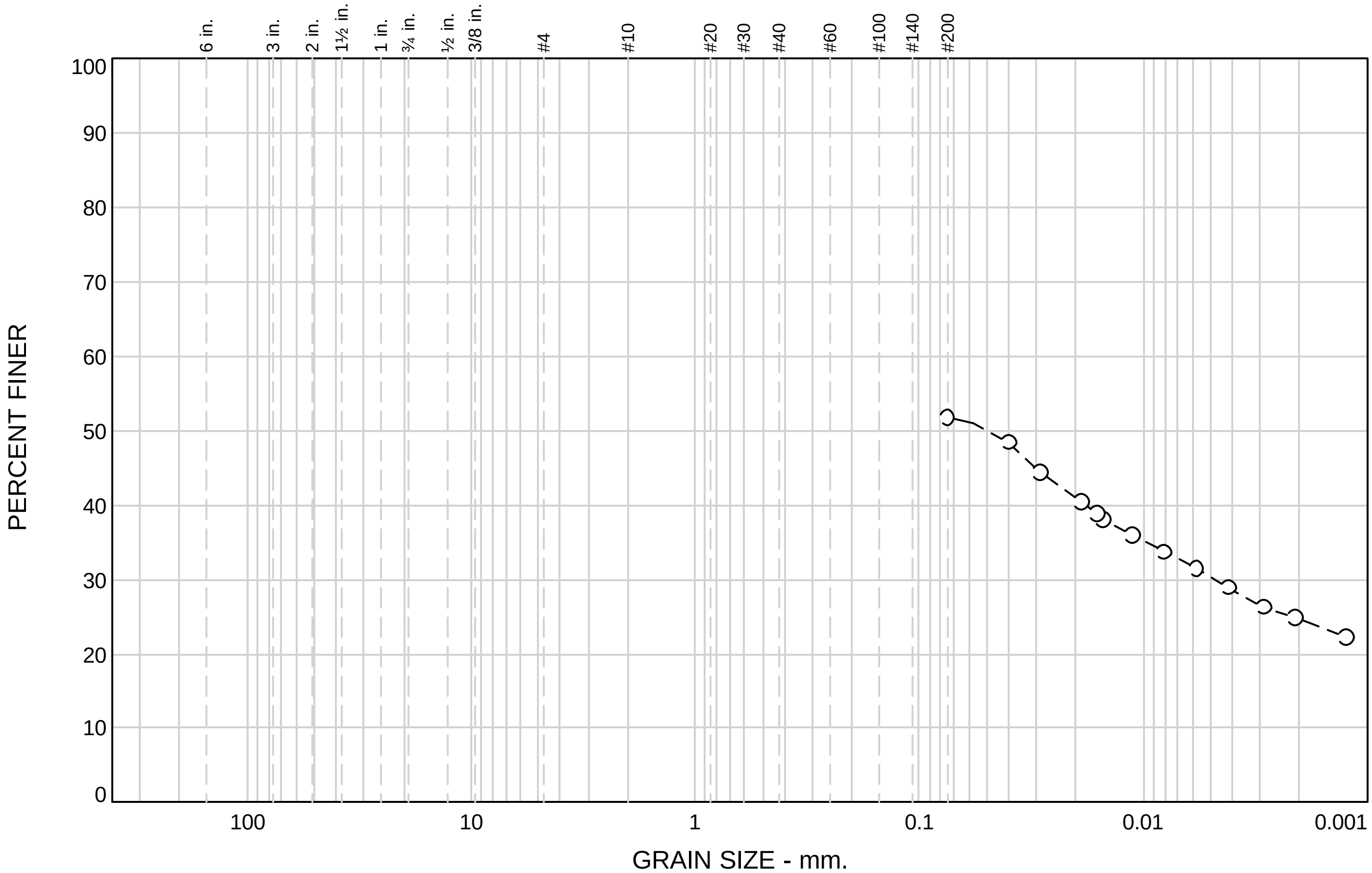
Sample No.: #2 @ 6' **Source of Sample:** **Date:** 02/16/11
Location: **Elev./Depth:**

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)

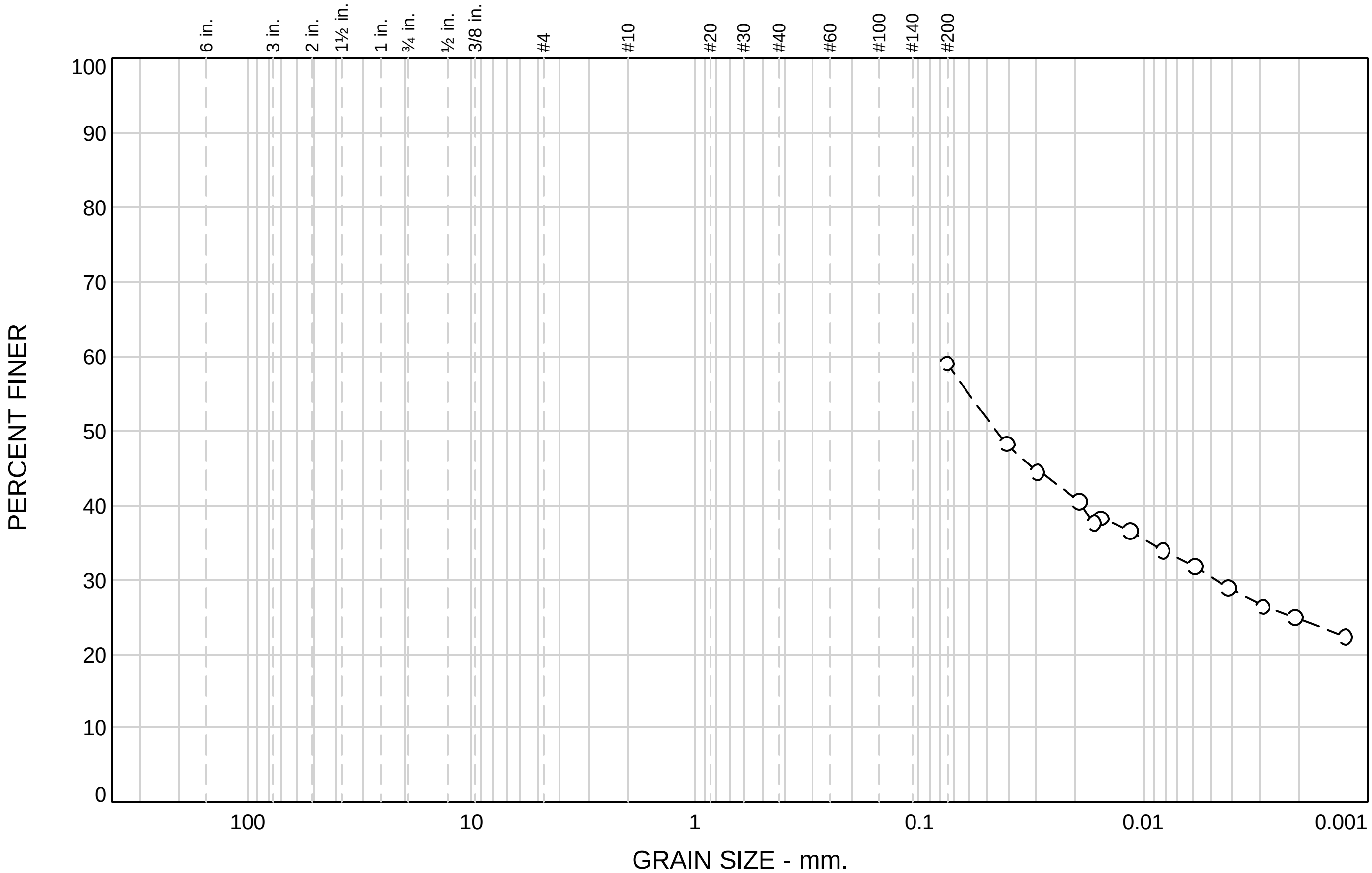
Remarks

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

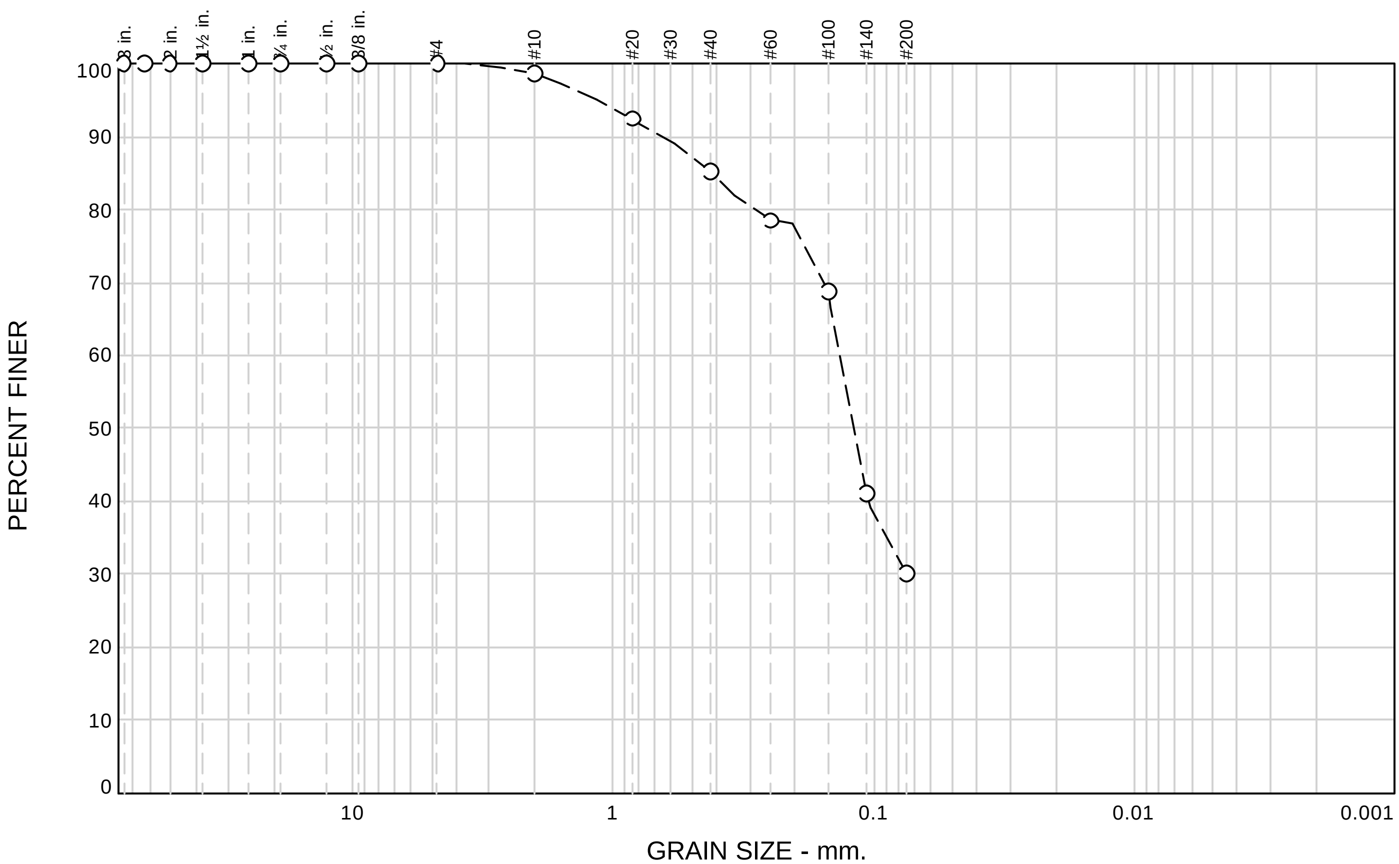
Sample No.: #4 @ 0.5' **Source of Sample:** **Date:** 02/16/11
Location: **Elev./Depth:**

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:**

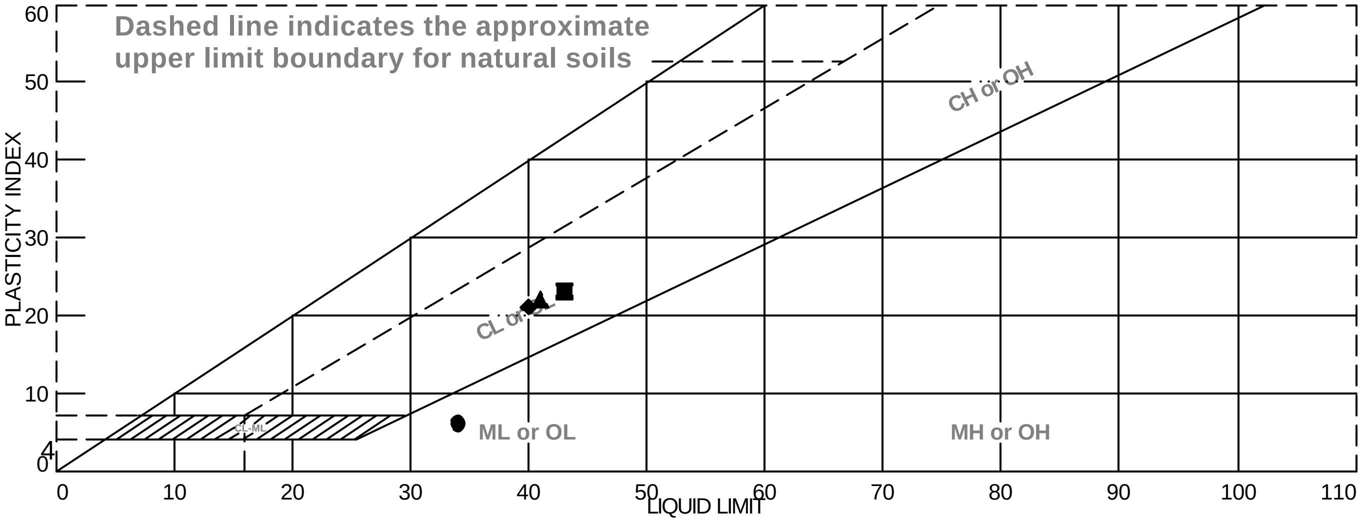
ENGEO INCORPORATED
GEOTECHNICAL AND ENVIRONMENTAL CONSULTANTS
MATERIALS TESTING

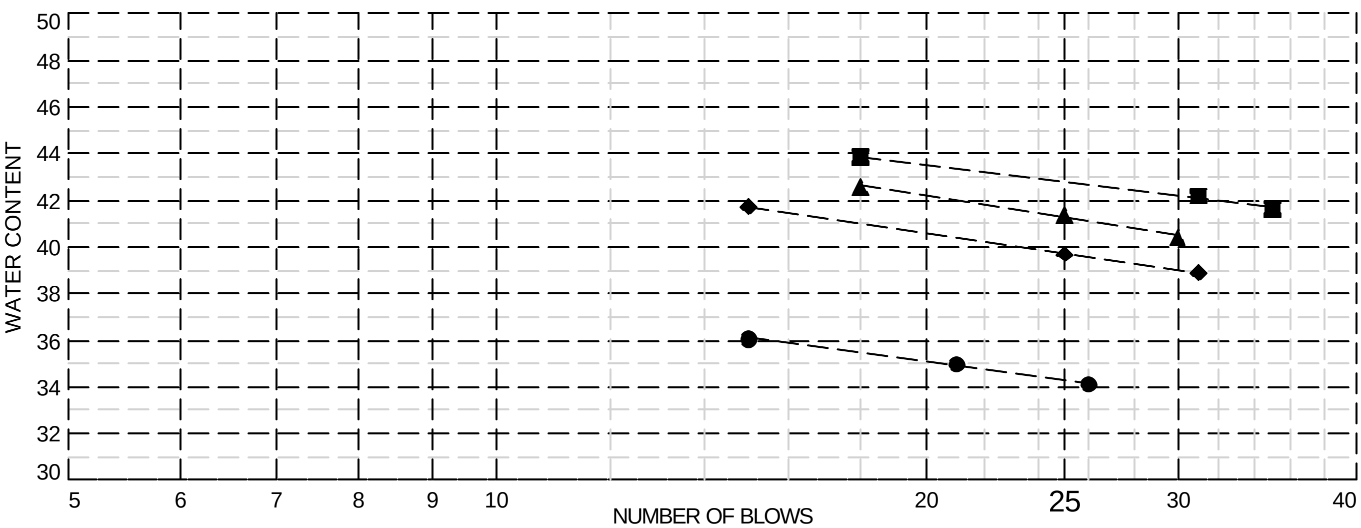
Client: O'Brien Land Company, LLC
Project: The Terraces of Lafayette

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:**

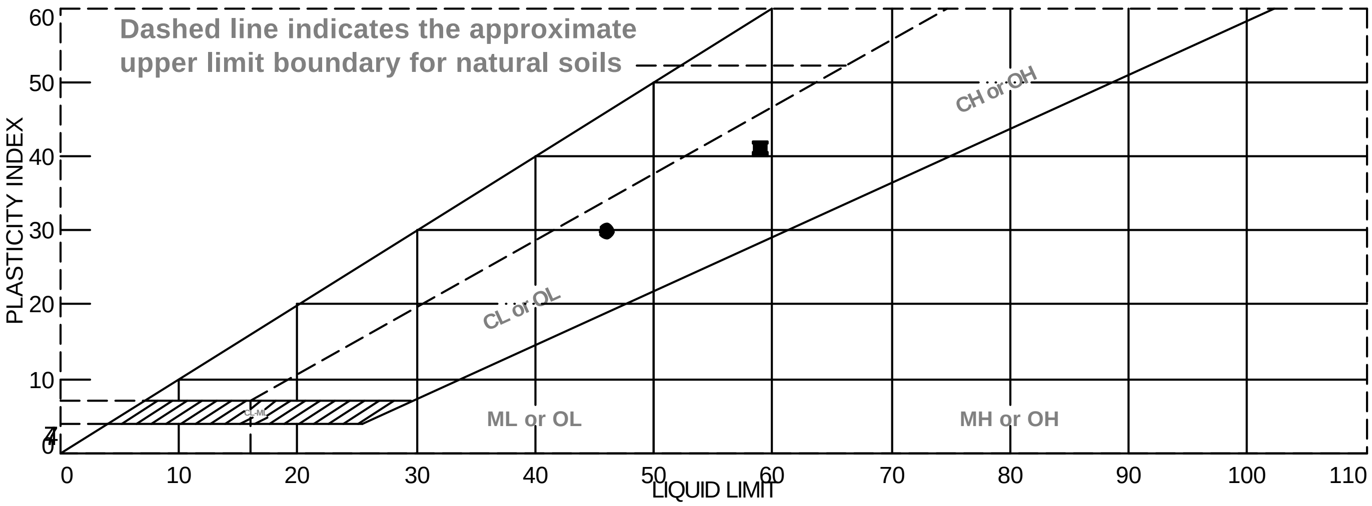
Project: Terraces of Lafayette

●**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

ENGEO INCORPORATED GEOTECHNICAL AND ENVIRONMENTAL CONSULTANTS MATERIALS TESTING

Remarks:

Plate

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

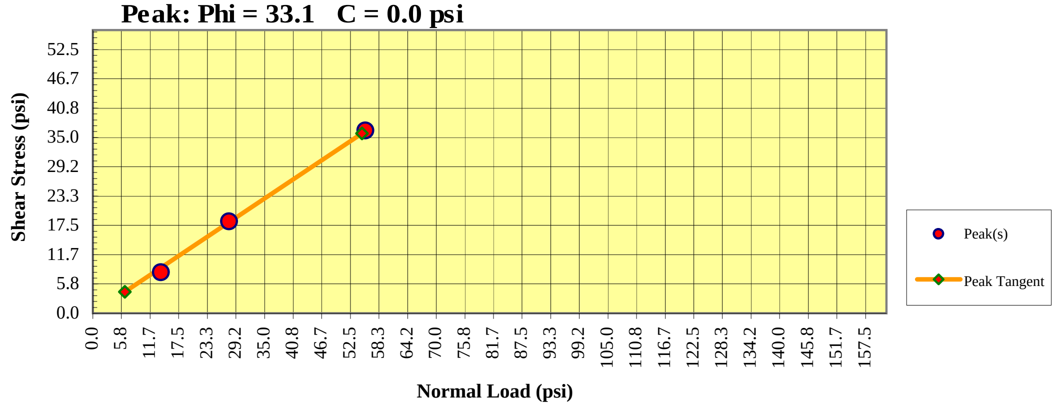
Direct Shear Test (ASTM D3080)

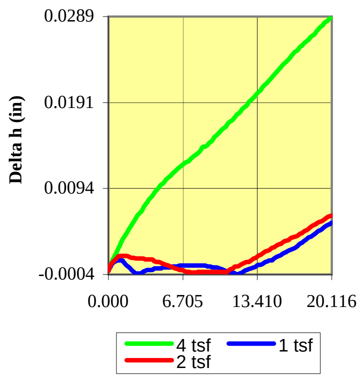
Tested By D. Seibold Date 7.6.11

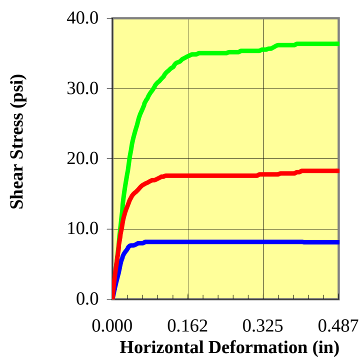
Checked By J. Fippin Date 7.6.11

Date 7.6.11

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

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)

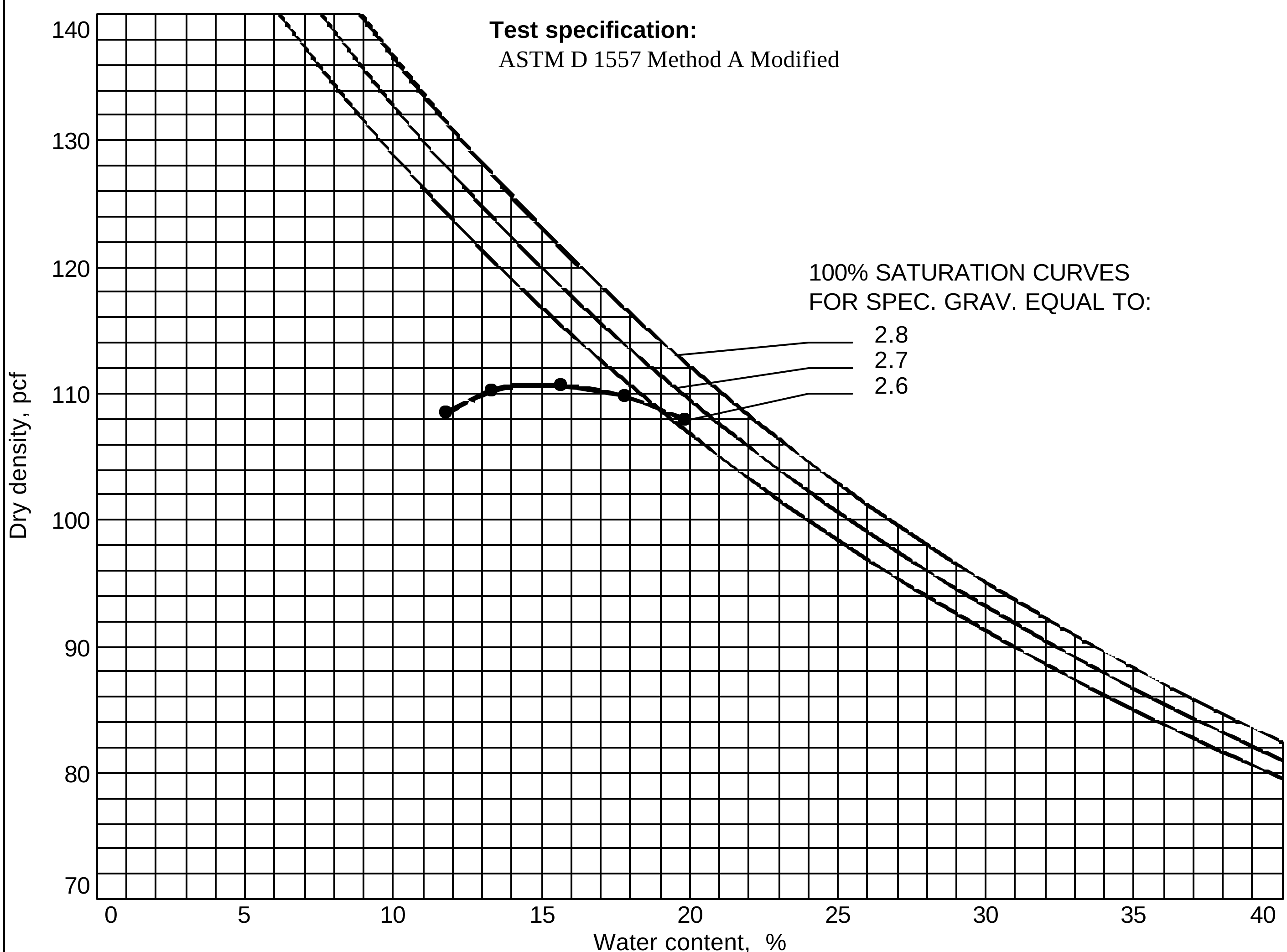
Nat. Moist. = **Sp.G. =**

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



ENGEO, INC.

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

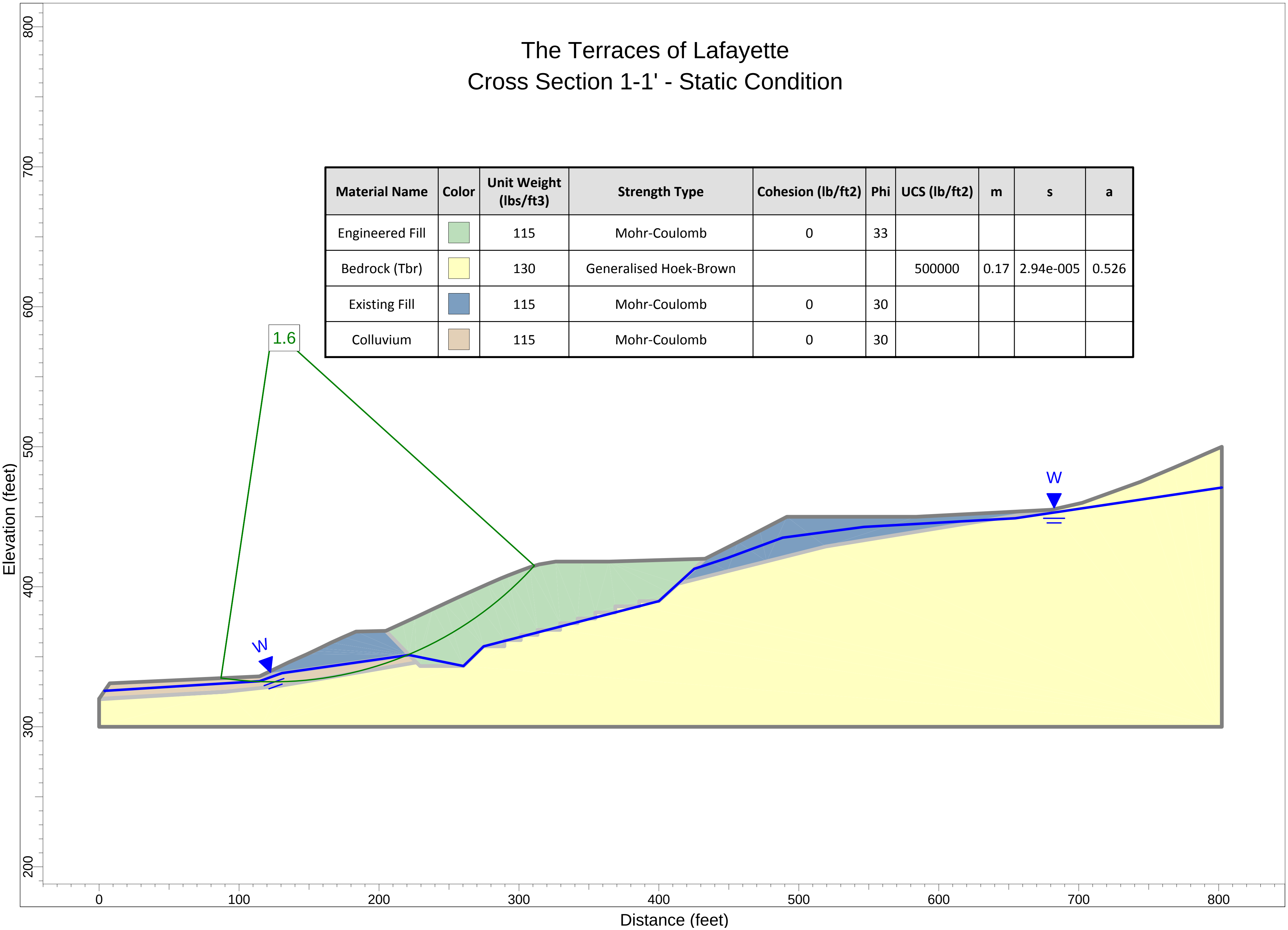
APPENDIX C

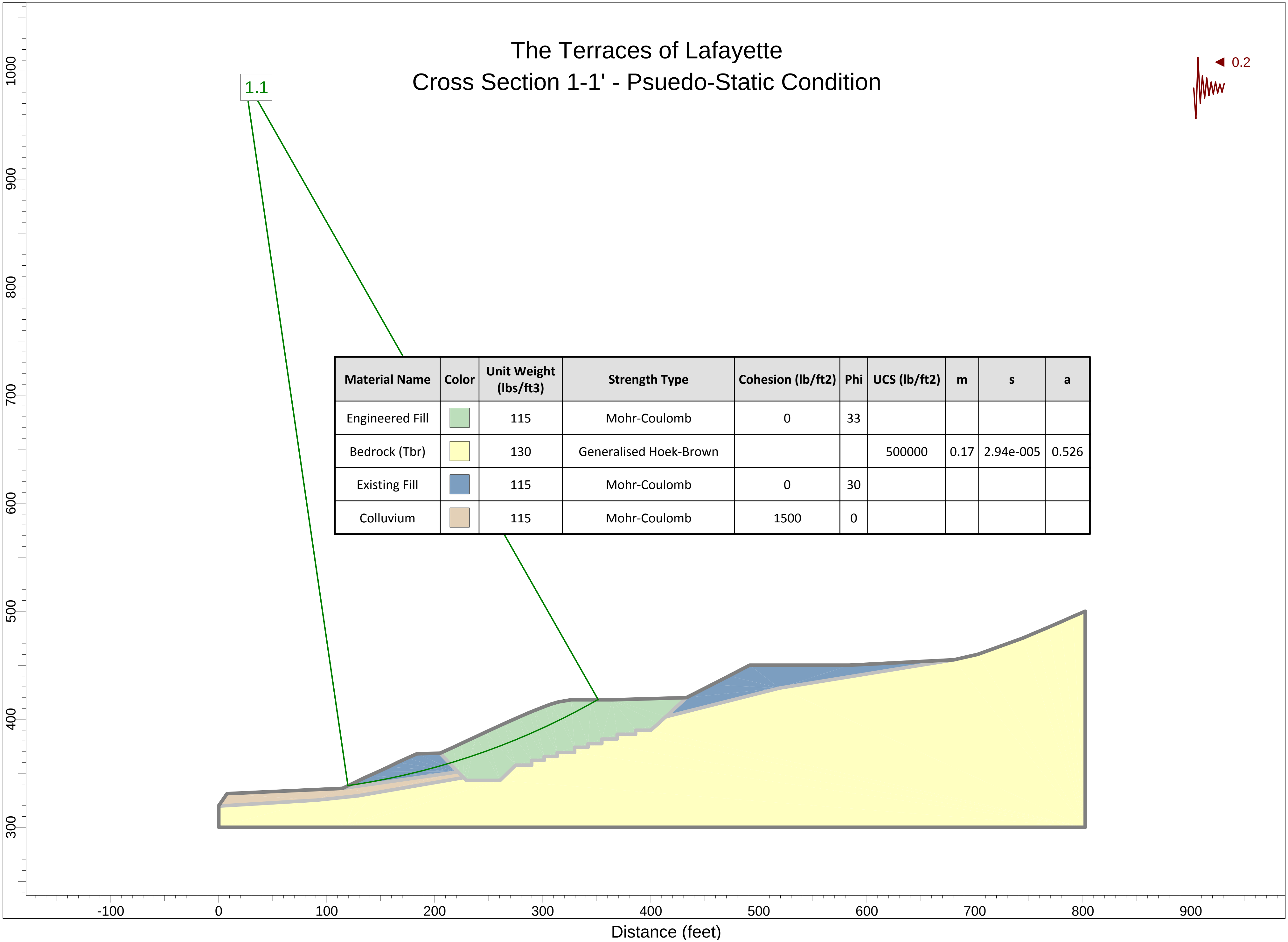
Slope Stability Analyses Results

A P P E N D I X C

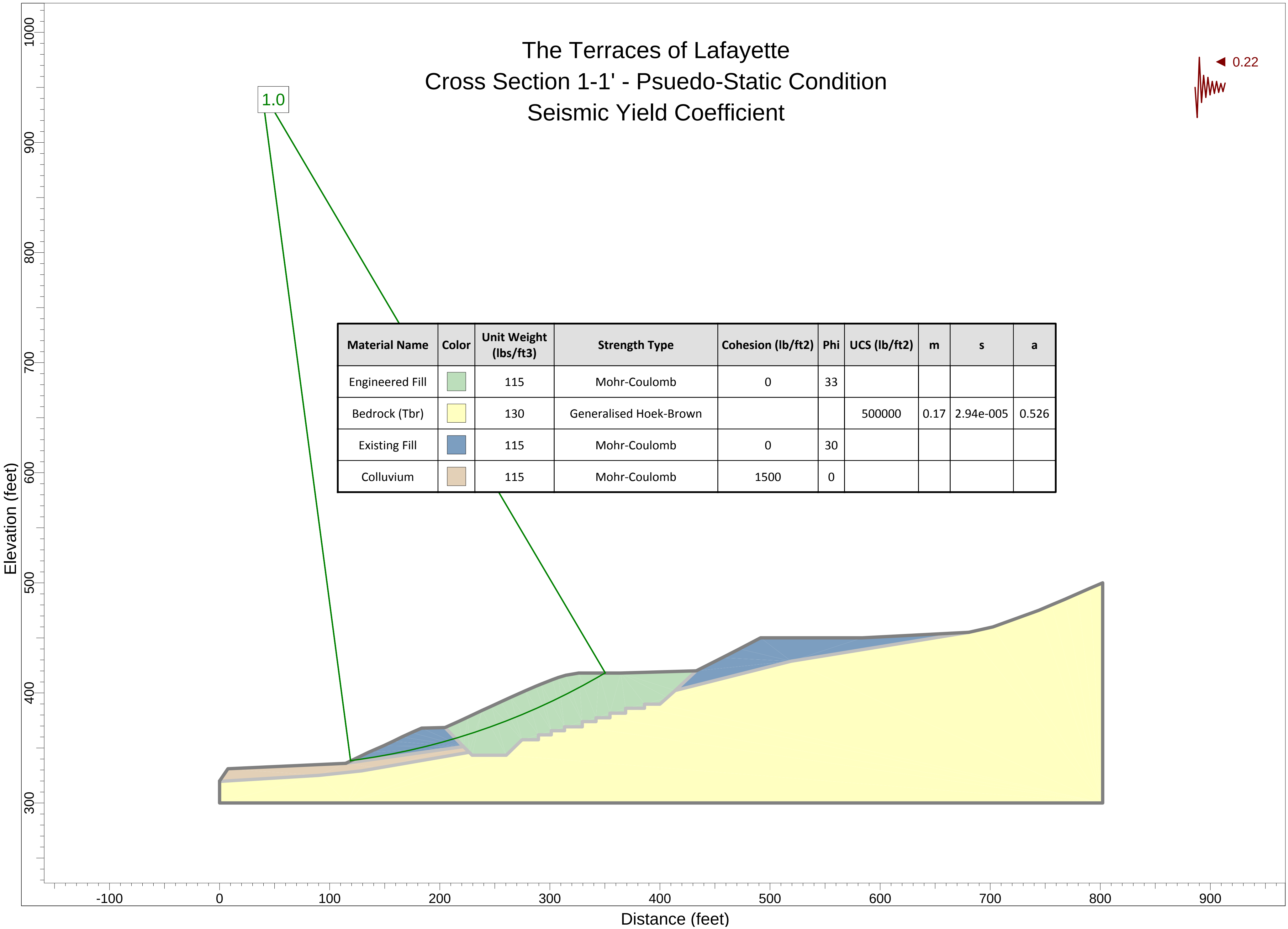
The Terraces of Lafayette
Cross Section 1-1' - Static Condition

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (lb/ft2)	Phi	UCS (lb/ft2)	m	s	a
Engineered Fill		115	Mohr-Coulomb	0	33				
Bedrock (Tbr)		130	Generalised Hoek-Brown			500000	0.17	2.94e-005	0.526
Existing Fill		115	Mohr-Coulomb	0	30				
Colluvium		115	Mohr-Coulomb	0	30				





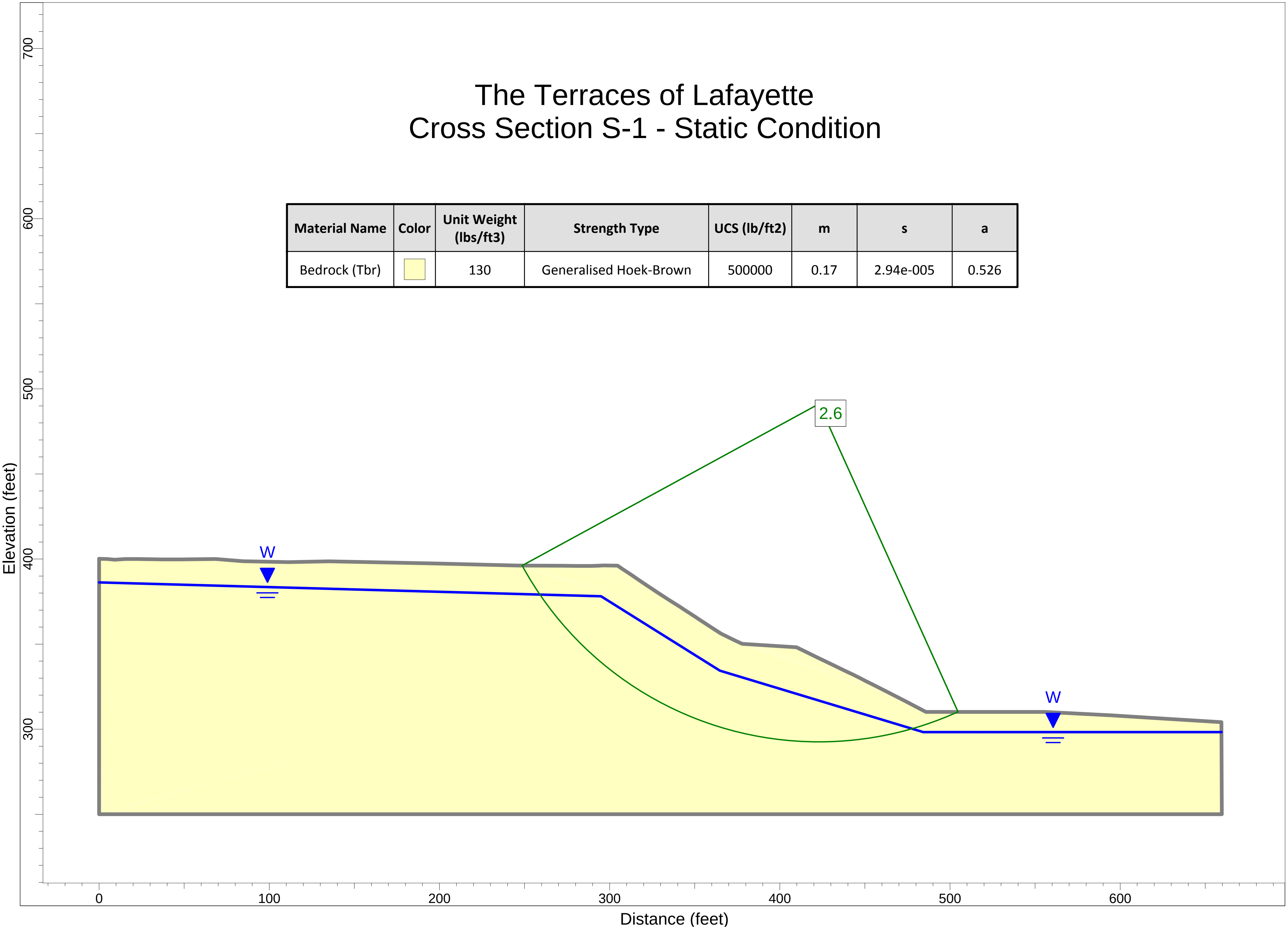
Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (lb/ft2)	Phi	UCS (lb/ft2)	m	s	a
Engineered Fill		115	Mohr-Coulomb	0	33				
Bedrock (Tbr)		130	Generalised Hoek-Brown			500000	0.17	2.94e-005	0.526
Existing Fill		115	Mohr-Coulomb	0	30				
Colluvium		115	Mohr-Coulomb	1500	0				



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (lb/ft2)	Phi	UCS (lb/ft2)	m	s	a
Engineered Fill		115	Mohr-Coulomb	0	33				
Bedrock (Tbr)		130	Generalised Hoek-Brown			500000	0.17	2.94e-005	0.526
Existing Fill		115	Mohr-Coulomb	0	30				
Colluvium		115	Mohr-Coulomb	1500	0				

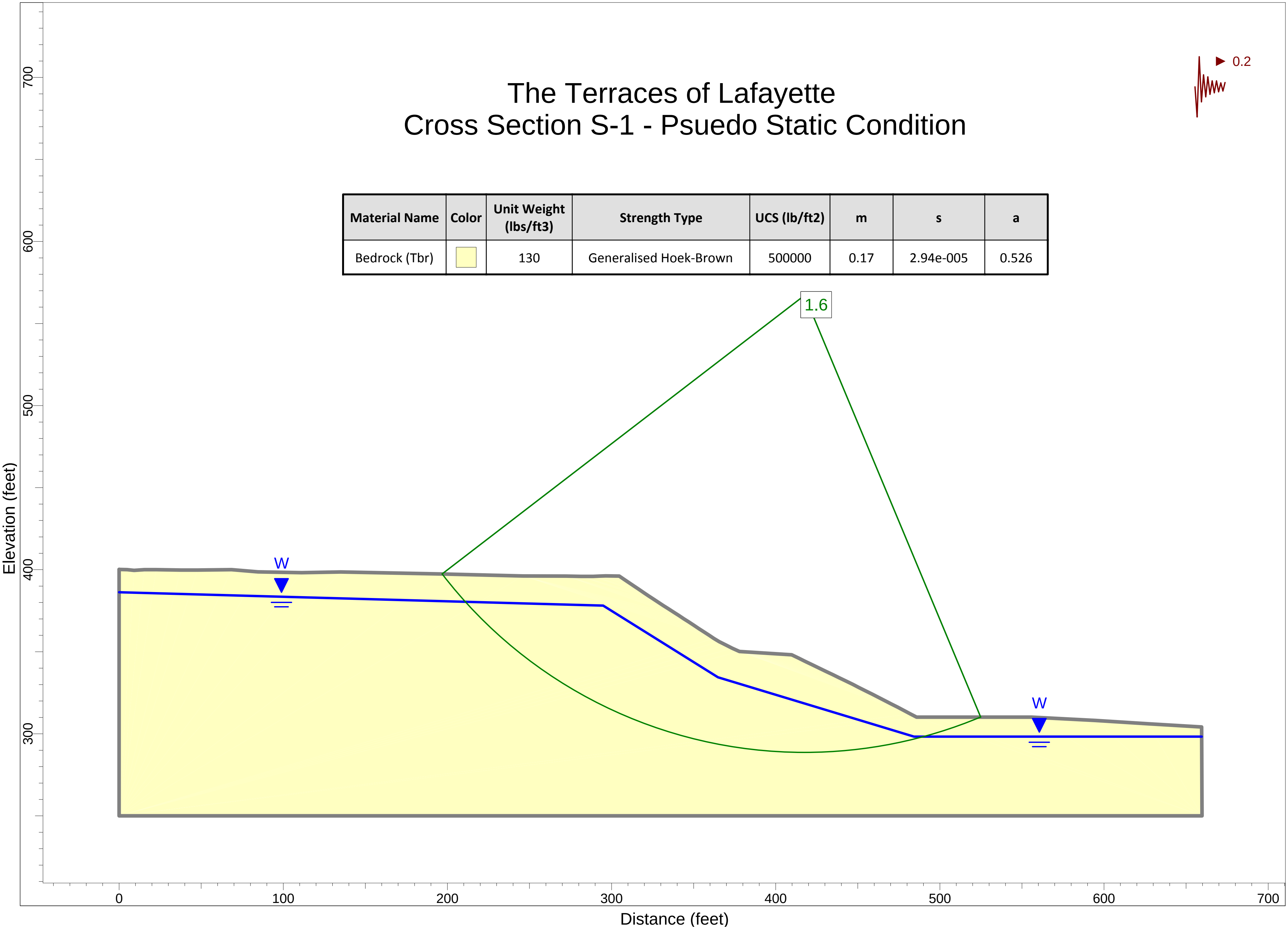
The Terraces of Lafayette
Cross Section S-1 - Static Condition

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	UCS (lb/ft2)	m	s	a
Bedrock (Tbr)		130	Generalised Hoek-Brown	500000	0.17	2.94e-005	0.526



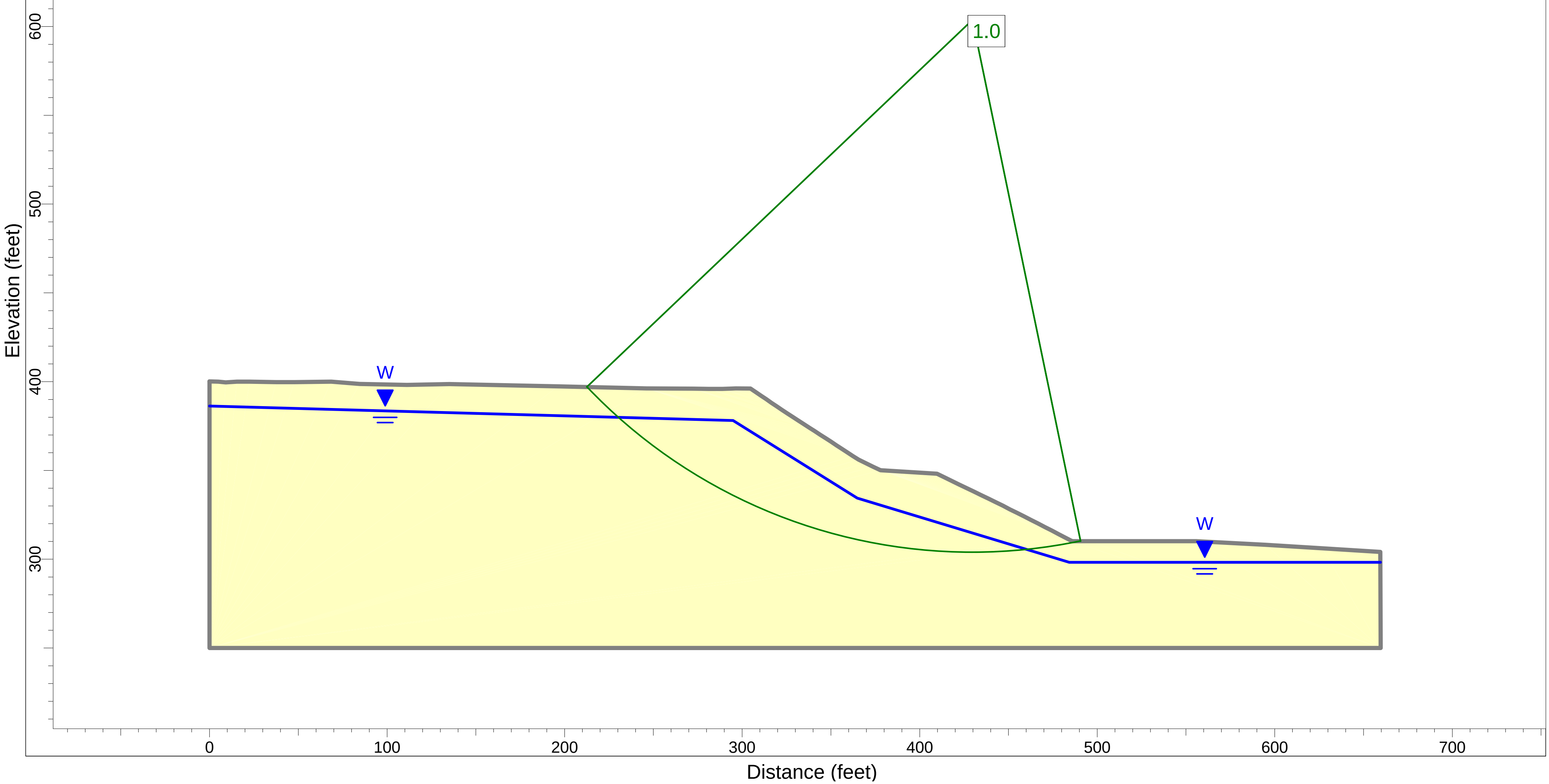
The Terraces of Lafayette
Cross Section S-1 - Psuedo Static Condition

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	UCS (lb/ft2)	m	s	a
Bedrock (Tbr)		130	Generalised Hoek-Brown	500000	0.17	2.94e-005	0.526



The Terraces of Lafayette
Cross-Section S-1 - Psuedo Static Condition
Seismic Yield Coefficient

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	UCS (lb/ft2)	m	s	a
Bedrock (Tbr)		130	Generalised Hoek-Brown	500000	0.17	2.94e-005	0.526



APPENDIX D

Guide Contract Specifications

APPENDIX D

GUIDE CONTRACT SPECIFICATIONS

PART I - EARTHWORK

PREFACE

These specifications are intended as a guide for the earthwork performed at the subject development project. If there is a conflict between these specifications (including the recommendations of the geotechnical report) and agency or code requirements, it should be brought to the attention of ENGEO and Owner prior to contract bidding.

PART 1 - GENERAL

1.01 WORK COVERED

A. Grading, excavating, filling and backfilling, including trenching and backfilling for utilities as necessary to complete the Project as indicated on the Drawings.

B. Subsurface drainage as indicated on the Drawings.

1.02 CODES AND STANDARDS

A. Excavating, trenching, filling, backfilling, and grading work shall meet the applicable requirements of the Uniform Building Code and the standards and ordinances of state and local governing authorities.

1.03 SUBSURFACE SOIL CONDITIONS

A. The Owners' Geotechnical Exploration report is available for inspection by bidder or Contractor. The Contractor shall refer to the findings and recommendations of the Geotechnical Exploration report in planning and executing his work.

1.04 DEFINITIONS

A. Fill: All soil, rock, or soil-rock materials placed to raise the grades of the site or to backfill excavations.

B. Backfill: All soil, rock or soil-rock material used to fill excavations and trenches.

C. On-Site Material: Soil and/or rock material which is obtained from the site.

D. Imported Material: Soil and/or rock material which is brought to the site from off-site areas.

E. Select Material: On-site and/or imported material which is approved by ENGEO as a specific-purpose fill.

F. Engineered Fill: Fill upon which ENGEO has made sufficient observations and tests to confirm that the fill has been placed and compacted in accordance with specifications and requirements.

G. Degree of Compaction or Relative Compaction: The ratio, expressed as a percentage, of the in-place dry density of the fill and backfill material as compacted in the field to the maximum dry density of the same material as determined by ASTM D-1557 or California 216 compaction test method.

H. Optimum Moisture: Water content, percentage by dry weight, corresponding to the maximum dry density as determined by ASTM D-1557.

I. ENGEO: The project geotechnical engineering consulting firm, its employees or its designated representatives.

J. Drawings: All documents, approved for construction, which describe the Work.

1.05 OBSERVATION AND TESTING

A. All site preparation, cutting and shaping, excavating, filling, and backfilling shall be carried out under the observation of ENGEO, employed and paid for by the Owners. ENGEO will perform appropriate field and laboratory tests to evaluate the suitability of fill material, the proper moisture content for compaction, and the degree of compaction achieved. Any fill that does not meet the specification requirements shall be removed and/or reworked until the requirements are satisfied.

B. Cutting and shaping, excavating, conditioning, filling, and compacting procedures require approval of ENGEO as they are performed. Any work found unsatisfactory or any work disturbed by subsequent operations before approval is granted shall be corrected in an approved manner as recommended by ENGEO.

C. Tests for compaction will be made in accordance with test procedures outlined in ASTM D-1557, as applicable. Field testing of soils or compacted fill shall conform with the applicable requirements of ASTM D-2922.

D. All authorized observation and testing will be paid for by the Owners.

1.06 SITE CONDITIONS

A. Excavating, filling, backfilling, and grading work shall not be performed during unfavorable weather conditions. When the work is interrupted by rain, excavating, filling, backfilling, and grading work shall not be resumed until the site and soil conditions are suitable.

B. Contractor shall take the necessary measures to prevent erosion of freshly filled, backfilled, and graded areas until such time as permanent drainage and erosion control measures have been installed.

PART 2 - PRODUCTS

2.01 GENERAL

A. Contractor shall furnish all materials, tools, equipment, facilities, and services as required for performing the required excavating, filling, backfilling, and grading work, and trenching and backfilling for utilities.

2.02 SOIL MATERIALS

A. Fill

1. Material to be used for engineered fill and backfill shall be free from organic matter and other deleterious substances, and of such quality that it will compact thoroughly without excessive voids when watered and rolled. Excavated on-site material will be considered suitable for engineered fill and backfill if it contains no more than 3 percent organic matter, is free of debris and other deleterious substances and conforms to the requirements specified above. Rocks of maximum dimension in excess of two-thirds of the lift thickness shall be removed from any fill material to the satisfaction of ENGEO.

2. Excavated earth material which is suitable for engineered fill or backfill, as determined by ENGEO, shall be conditioned for reuse and properly stockpiled as required for later filling and backfilling operations. Conditioning shall consist of spreading material in layers not to exceed 8 inches and raking free of debris and rubble. Rocks and aggregate exceeding the allowed largest dimension, and deleterious material shall be removed from the site and disposed off site in a legal manner.

3. ENGEO shall be immediately notified if potential hazardous materials or suspect soils exhibiting staining or odor are encountered. Work activities shall be discontinued within the area of potentially hazardous materials. ENGEO environmental personnel will conduct an assessment of the suspect hazardous material to determine the appropriate response and mitigation. Regulatory agencies may also be contacted to request concurrence and oversight. ENGEO will rely on the Owner, or a designated Owner's representative, to make necessary *notices to the appropriate regulatory agencies. The Owner may request ENGEO's*

assistance in notifying regulatory agencies, provided ENGEO receives Owner's written authorization to expand its scope of services.

4. ENGEO shall be notified at least 48 hours prior to the start of filling and backfilling operations so that it may evaluate samples of the material intended for use as fill and backfill. All materials to be used for filling and backfilling require the approval of ENGEO.

B. Import Material: Where conditions require the importation of fill material, the material shall be an inert, nonexpansive soil or soil-rock material free of organic matter and meeting the following requirements unless otherwise approved by ENGEO.

Gradation (ASTM D-421):	<u>Sieve Size</u>	<u>Percent Passing</u>
	2-inch	100
	#200	15 - 70
Plasticity (ASTM D-4318):	<u>Liquid Limit</u>	<u>Plasticity Index</u>
	< 30	< 12
Swell Potential (ASTM D-4546B):	<u>Percent Heave</u>	<u>Swell Pressure</u>
(at optimum moisture)	< 2 percent	< 300 psf
Resistance Value (ASTM D-2844):	Minimum 25	
Organic Content (ASTM D-2974):	Less than 2 percent	

A sample of the proposed import material should be submitted to ENGEO for evaluation prior to delivery at the site.

2.03 SAND

A. Sand for sand cushion under slabs and for bedding of pipe in utility trenches shall be a clean and graded, washed sand, free from clay or organic material, suitable for the intended purpose with 90 to 100 percent passing a No. 4 U.S. Standard Sieve, not more than 5 percent passing a No. 200 U.S. Standard Sieve, and generally conforming to ASTM C33 for fine aggregate.

2.04 AGGREGATE DRAINAGE FILL

A. Aggregate drainage fill under concrete slabs and paving shall consist of broken stone, crushed or uncrushed gravel, clean quarry waste, or a combination thereof. The aggregate shall be free from fines, vegetable matter, loam, volcanic tuff, and other deleterious substances. It shall be of such quality that the absorption of water in a saturated surface dry condition does not exceed 3 percent of the oven dry weight of the samples.

B. Aggregate drainage fill shall be of such size that the percentage composition by dry weight as determined by laboratory sieves (U. S. Series) will conform to the following grading:

Sieve Size	Percentage Passing Sieve
1½ inches	100
1 inch	90 - 100
#4	0 - 5

PART 3 - EXECUTION

3.01 STAKING AND GRADES

A. Contractor shall lay out all his work, establish all necessary markers, bench marks, grading stakes, and other stakes as required to achieve design grades.

3.02 EXISTING UTILITIES

A. Contractor shall verify the location and depth (elevation) of all existing utilities and services before performing any excavation work.

3.03 EXCAVATION

A. Contractor shall perform excavating as indicated and required for concrete footings, drilled piers, foundations, floor slabs, concrete walks, and site leveling and grading, and provide shoring, bracing, underpinning, cribbing, pumping, and planking as required. The bottoms of excavations shall be firm undisturbed earth, clean and free from loose material, debris, and foreign matter.

B. Excavations shall be kept free from water at all times. Adequate dewatering equipment shall be maintained at the site to handle emergency situations until concrete or backfill is placed.

C. Unauthorized excavations for footings shall be filled with concrete to required elevations, unless other methods of filling are authorized by ENGEO.

D. Excavated earth material which is suitable for engineered fill or backfill, as determined by ENGEO, shall be conditioned for reuse and properly stockpiled for later filling and backfilling operations as specified under Section 2.02, "Soil Materials."

E. Abandoned sewers, piping, and other utilities encountered during excavating shall be removed and the resulting excavations shall be backfilled with engineered fill as required by ENGEO.

F. Any active utility lines encountered shall be reported immediately to the Owner's Representative and authorities involved. The Owner and proper authorities shall be permitted free access to take the measures deemed necessary to repair, relocate, or remove the obstruction as determined by the responsible authority or Owner's Representative.

3.04 SUBGRADE PREPARATION

A. All brush and other rubbish, as well as trees and root systems not marked for saving, shall be removed from the site and legally disposed of.

B. Any existing structures, foundations, underground storage tanks, or debris must be removed from the site prior to any building, grading, or fill operations. Septic tanks, including all drain fields and other lines, if encountered, must be totally removed. The resulting depressions shall be properly prepared and filled to the satisfaction of ENGEO.

C. Vegetation and organic topsoil shall be removed from the surface upon which the fill is to be placed and either removed and legally disposed of or stockpiled for later use in approved landscape areas. The surface shall then be scarified to a depth of at least eight inches until the surface is free from ruts, hummocks, or other uneven features which would tend to prevent uniform compaction by the equipment to be used.

D. After the foundation for the fill has been cleared and scarified, it shall be made uniform and free from large clods. The proper moisture content must be obtained by adding water or aerating. The foundation for the fill shall be compacted at the proper moisture content to a relative compaction as specified herein.

3.05 ENGINEERED FILL

A. Select Material: Fill material shall be "Select" or "Imported Material" as previously specified.

B. Placing and Compacting: Engineered fill shall be constructed by approved and accepted methods. Fill material shall be spread in uniform lifts not exceeding 8 inches in uncompacted thickness. Each layer shall be spread evenly, and thoroughly blade-mixed to obtain uniformity of material. Fill material which does not contain sufficient moisture as specified by ENGEO shall be sprinkled with water; if it contains excess moisture it shall be aerated or blended with drier material to achieve the proper water content. Select material and water shall then be thoroughly mixed before being compacted.

C. Unless otherwise specified in the Geotechnical Exploration report, each layer of spread select material shall be compacted to at least 90 percent relative compaction at a moisture content of at least three percentage points above the optimum moisture content. Minimum compaction in all keyways shall be a minimum of 95 percent with a minimum moisture content of at least 1 percentage point above optimum.

D. Unless otherwise specified in the Geotechnical Exploration report or otherwise required by the local authorities, the upper 6 inches of engineered fill in areas to receive pavement shall be compacted to at least 95 percent relative compaction with a minimum moisture content of at least 3 percentage points above optimum.

E. Testing and Observation of Fill: The work shall consist of field observation and testing to determine that each layer has been compacted to the required density and that the required moisture is being obtained. Any layer or portion of a layer that does not attain the compaction required shall be reworked until the required density is obtained.

F. Compaction: Compaction shall be by sheepsfoot rollers, multiple-wheel steel or pneumatic-tired rollers or other types of acceptable compaction equipment. Rollers shall be of such design that they will be able to compact the fill to the specified compaction. Rolling shall be accomplished while the fill material is within the specified moisture content range. Rolling of each layer must be continuous so that the required compaction may be obtained uniformly throughout each layer.

G. Fill slopes shall be constructed by overfilling the design slopes and later cutting back the slopes to the design grades. No loose soil will be permitted on the faces of the finished slopes.

H. Strippings and topsoil shall be stockpiled as approved by Owner, then placed in accordance with ENGEO's recommendations to a minimum thickness of 6 inches and a maximum thickness of 12 inches over exposed open space cut slopes which are 3:1 or flatter, and track walked to the satisfaction of ENGEO.

I. Final Prepared Subgrade: Finish blading and smoothing shall be performed as necessary to produce the required density, with a uniform surface, smooth and true to grade.

3.06 BACKFILLING

A. Backfill shall not be placed against footings, building walls, or other structures until approved by ENGEO.

B. Backfill material shall be Select Material as specified for engineered fill.

C. Backfill shall be placed in 6-inch layers, leveled, rammed, and tamped in place. Each layer shall be compacted with suitable compaction equipment to 90 percent relative compaction at a moisture content of at least 3 percent above optimum.

3.07 TRENCHING AND BACKFILLING FOR UTILITIES

A. Trenching:

1. Trenching shall include the removal of material and obstructions, the installation and removal of sheeting and bracing and the control of water as necessary to provide the required utilities and services.

2. Trenches shall be excavated to the lines, grades, and dimensions indicated on the Drawings. Maximum allowable trench width shall be the outside diameter of the pipe plus 24 inches, inclusive of any trench bracing.

3. When the trench bottom is a soft or unstable material as determined by ENGEO, it shall be made firm and solid by removing said unstable material to a sufficient depth and replacing it with on-site material compacted to 90 percent minimum relative compaction.

4. Where water is encountered in the trench, the contractor must provide materials necessary to drain the water and stabilize the bed.

B. Backfilling:

1. Trenches must be backfilled within 2 days of excavation to minimize desiccation.

2. Bedding material shall be sand and shall not extend more than 6 inches above any utility lines.

3. Backfill material shall be select material.

4. Trenches shall be backfilled as indicated or required and compacted with suitable equipment to 90 percent minimum relative compaction at the required moisture content.

3.8 SAND CUSHION

A. A sand cushion shall be placed over the vapor retarder membrane under concrete slabs on grade. Sand cushion shall be placed in uniform thickness as indicated on the Drawings. Where not indicated, the thickness shall be 2 inches.

3.9 FINISH GRADING

A. All areas must be finish graded to elevations and grades indicated on the Drawings. In areas to receive topsoil and landscape planting, finish grading shall be performed to a uniform 6 inches below the grades and elevations indicated on the Drawings, and brought to final grade with topsoil.

3.10 DISPOSAL OF WASTE MATERIALS

A. Excess earth materials and debris shall be removed from the site and disposed of in a legal manner. Location of dump site and length of haul are the Contractor's responsibility.

PART II - GEOGRID SOIL REINFORCEMENT

1 DESCRIPTION:

Work shall consist of furnishing geogrid soil reinforcement for use in construction of reinforced soil slopes and retention systems.

2. GEOGRID MATERIAL:

2.1 The specific geogrid material shall be preapproved by ENGEO.

2.2 The geogrid shall be a regular network of integrally connected polymer tensile elements with aperture geometry sufficient to permit significant mechanical interlock with the surrounding soil or rock. The geogrid structure shall be dimensionally stable and able to retain its geometry under construction stresses and shall have high resistance to damage during construction, to ultraviolet degradation, and to all forms of chemical and biological degradation encountered in the soil being reinforced.

2.3 The geogrids shall have an Allowable Strength (T_a) and Pullout Resistance, for the soil type(s) indicated, as listed in Table I.

2.4 Certifications: The Contractor shall submit a manufacturer's certification that the geogrids supplied meet the respective index criteria set when geogrid was approved by ENGEO, measured in full accordance with all test methods and standards specified. In case of dispute over validity of values, the Contractor will supply test data from an ENGEO-approved laboratory to support the certified values submitted.

3. CONSTRUCTION:

3.1 Delivery, Storage, and Handling: Contractor shall check the geogrid upon delivery to ensure that the proper material has been received. During all periods of shipment and storage, the geogrid shall be protected from temperatures greater than 140 °F, mud, dirt, dust, and debris. Manufacturer's recommendations in regard to protection from direct sunlight must also be followed. At the time of installation, the geogrid will be rejected if it has defects, tears, punctures, flaws, deterioration, or damage incurred during manufacture, transportation, or storage. If approved by ENGEO, torn or punctured sections may be repaired by placing a patch over the damaged area. Any geogrid damaged during storage or installation shall be replaced by the Contractor at no additional cost to the owner.

3.2 On-Site Representative: Geogrid material suppliers shall provide a qualified and experienced representative on site at the initiation of the project, for a minimum of three days, to assist the Contractor and ENGEO personnel at the start of construction. If there is more than one slope on a project, this criterion will apply to construction of the initial slope only. The representative shall also be available on an as-needed basis, as requested by ENGEO, during construction of the remaining slope(s).

3.3 Geogrid reinforcement may be joined with mechanical connections or overlaps as recommended and approved by the Manufacturer. Joints shall not be placed within 6 feet of the slope face, within 4 feet below top of slope, nor horizontally or vertically adjacent to another joint.

3.4 Geogrid Placement: The geogrid reinforcement shall be installed in accordance with the manufacturer's recommendations. The geogrid reinforcement shall be placed within the layers of the compacted soil as shown on the plans or as directed.

The geogrid reinforcement shall be placed in continuous longitudinal strips in the direction of main reinforcement. However, if the Contractor is unable to complete a required length with a single continuous length of geogrid, a joint may be made with the Manufacturer's approval. Only one joint per length of geogrid shall be allowed. This joint shall be made for the full width of the strip by using a similar material with similar strength. Joints in geogrid reinforcement shall be pulled and held taut during fill placement.

Adjacent strips, in the case of 100 percent coverage in plan view, need not be overlapped. The minimum horizontal coverage is 50 percent, with horizontal spacings between reinforcement no greater than 40 inches. Horizontal coverage of less than 100 percent shall not be allowed unless specifically detailed in the construction drawings.

Adjacent rolls of geogrid reinforcement shall be overlapped or mechanically connected where exposed in a wrap around face system, as applicable.

The Contractor may place only that amount of geogrid reinforcement required for immediately pending work to prevent undue damage. After a layer of geogrid reinforcement has been placed, the next succeeding layer of soil shall be placed and compacted as appropriate. After the specified soil layer has been placed, the next geogrid reinforcement layer shall be installed. The process shall be repeated for each subsequent layer of geogrid reinforcement and soil.

Geogrid reinforcement shall be placed to lay flat and pulled tight prior to backfilling. After a layer of geogrid reinforcement has been placed, suitable means, such as pins or small piles of soil, shall be used to hold the geogrid reinforcement in position until the subsequent soil layer can be placed.

Under no circumstances shall a track-type vehicle be allowed on the geogrid reinforcement before at least six inches of soil have been placed. Turning of tracked vehicles should be kept to a minimum to prevent tracks from displacing the fill and the geogrid reinforcement. If approved by the Manufacturer, rubber-tired equipment may pass over the geosynthetic reinforcement at slow speeds, less than 10 mph. Sudden braking and sharp turning shall be avoided.

During construction, the surface of the fill should be kept approximately horizontal. Geogrid reinforcement shall be placed directly on the compacted horizontal fill surface. Geogrid reinforcements are to be placed within three inches of the design elevations and extend the length as shown on the elevation view unless otherwise directed by ENGEO. Correct orientation of the geogrid reinforcement shall be verified by ENGEO.

Table I
Allowable Geogrid Strength
With Various Soil Types
For Geosynthetic Reinforcement In
Mechanically Stabilized Earth Slopes

(Geogrid Pullout Resistance and Allowable Strengths vary with reinforced backfill used due to soil anchorage and site damage factors. Guidelines are provided below.)

SOIL TYPE	MINIMUM ALLOWABLE STRENGTH, T_a (lb/ft)*		
	GEOGRID Type I	GEOGRID Type II	GEOGRID Type III
A. Gravels, sandy gravels, and gravel-sand-silt mixtures (GW, GP, GC, GM & SP)**	2400	4800	7200
B. Well graded sands, gravelly sands, and sand-silt mixtures (SW & SM)**	2000	4000	6000
C. Silts, very fine sands, clayey sands and clayey silts (SC & ML)**	1000	2000	3000
D. Gravelly clays, sandy clays, silty clays, and lean clays (CL)**	1600	3200	4800

* All partial Factors of Safety for reduction of design strength are included in listed values. Additional factors of safety may be required to further reduce these design strengths based on site conditions.

** Unified Soil Classifications.

PART III - GEOTEXTILE SOIL REINFORCEMENT

1. DESCRIPTION:

   Work shall consist of furnishing geotextile soil reinforcement for use in construction of reinforced soil slopes.

2. GEOTEXTILE MATERIAL:

   2.1 The specific geotextile material and supplier shall be preapproved by ENGEO.

   2.2 The geotextile shall have a high tensile modulus and shall have high resistance to damage during construction, to ultraviolet degradation, and to all forms of chemical and biological degradation encountered in the soil being reinforced.

   2.3 The geotextiles shall have an Allowable Strength (Ta) and Pullout Resistance, for the soil type(s) indicated as listed in Table II.

   2.4 Certification: The Contractor shall submit a manufacturer's certification that the geotextiles supplied meet the respective index criteria set when geotextile was approved by ENGEO, measured in full accordance with all test methods and standards specified. In case of dispute over validity of values, the Contractor will supply the data from an ENGEO-approved laboratory to support the certified values submitted.

3. CONSTRUCTION:

   3.1 Delivery, Storage and Handling: Contractor shall check the geotextile upon delivery to ensure that the proper material has been received. During all periods of shipment and storage, the geotextile shall be protected from temperatures greater than 140 °F, mud, dirt, dust, and debris. Manufacturer's recommendations in regard to protection from direct sunlight must also be followed. At the time of installation, the geotextile will be rejected if it has defects, tears, punctures, flaws, deterioration, or damage incurred during manufacture, transportation, or storage. If approved by ENGEO, torn or punctured sections may be repaired by placing a patch over the damaged area. Any geotextile damaged during storage or installation shall be replaced by the Contractor at no additional cost to the owner.

   3.2 On-Site Representative: Geotextile material suppliers shall provide a qualified and experienced representative on site at the initiation of the project, for a minimum of three days, to assist the Contractor and ENGEO personnel at the start of construction. If there is more than one slope on a project, this criterion will apply to construction of the initial slope only. The representative shall also be available on an as-needed basis, as requested by ENGEO, during construction of the remaining slope(s).

3.3 Geotextile Placement: The geotextile reinforcement shall be installed in accordance with the manufacturer's recommendations. The geotextile reinforcement shall be placed within the layers of the compacted soil as shown on the plans or as directed.

The geotextile reinforcement shall be placed in continuous longitudinal strips in the direction of main reinforcement. Joints shall not be used with geotextiles.

Adjacent strips, in the case of 100 percent coverage in plan view, need not be overlapped. The minimum horizontal coverage is 50 percent, with horizontal spacings between reinforcement no greater than 40 inches. Horizontal coverage of less than 100 percent shall not be allowed unless specifically detailed in the construction drawings.

Adjacent rolls of geotextile reinforcement shall be overlapped or mechanically connected where exposed in a wrap around face system, as applicable.

The Contractor may place only that amount of geotextile reinforcement required for immediately pending work to prevent undue damage. After a layer of geotextile reinforcement has been placed, the succeeding layer of soil shall be placed and compacted as appropriate. After the specified soil layer has been placed, the next geotextile reinforcement layer shall be installed. The process shall be repeated for each subsequent layer of geotextile reinforcement and soil.

Geosynthetic reinforcement shall be placed to lay flat and be pulled tight prior to backfilling. After a layer of geotextile reinforcement has been placed, suitable means, such as pins or small piles of soil, shall be used to hold the geotextile reinforcement in position until the subsequent soil layer can be placed.

Under no circumstances shall a track-type vehicle be allowed on the geotextile reinforcement before at least six inches of soil has been placed. Turning of tracked vehicles should be kept to a minimum to prevent tracks from displacing the fill and the geotextile reinforcement. If approved by the Manufacturer, rubber-tired equipment may pass over the geotextile reinforcement as slow speeds, less than 10 mph. Sudden braking and sharp turning shall be avoided.

During construction, the surface of the fill should be kept approximately horizontal. Geotextile reinforcement shall be placed directly on the compacted horizontal fill surface. Geotextile reinforcements are to be placed within three inches of the design elevations and extend the length as shown on the elevation view unless otherwise directed by ENGEO. Correct orientation of the geotextile reinforcement shall be verified by ENGEO.

Table II
Allowable Geotextile Strength
With Various Soil Types
For Geosynthetic Reinforcement In
Mechanically Stabilized Earth Slopes

(Geotextile Pullout Resistance and Allowable Strengths vary with reinforced backfill used due to soil anchorage and site damage factors. Guidelines are provided below.)

SOIL TYPE	MINIMUM ALLOWABLE STRENGTH, Ta (lb/ft)*		
	GEOTEXTILE Type I	GEOTEXTILE Type II	GEOTEXTILE Type III
A. Gravels, sandy gravels, and gravel-sand-silt mixtures (GW, GP, GC, GM & SP)**	2400	4800	7200
B. Well graded sands, gravelly sands, and sand-silt mixtures (SW & SM)**	2000	4000	6000
C. Silts, very fine sands, clayey sands and clayey silts (SC & ML)**	1000	2000	3000
D. Gravelly clays, sandy clays, silty clays, and lean clays (CL)**	1600	3200	4800

*All partial Factors of Safety for reduction of design strength are included in listed values. Additional factors of safety may be required to further reduce these design strengths based on site conditions.
**Unified Soil Classifications.

PART IV - EROSION CONTROL MAT OR BLANKET

1. DESCRIPTION:

   Work shall consist of furnishing and placing a synthetic erosion control mat and/or degradable erosion control blanket for slope face protection and lining of runoff channels.

2. EROSION CONTROL MATERIALS:

   2.1 The specific erosion control material and supplier shall be pre-approved by ENGEO.

   2.2 Certification: The Contractor shall submit a manufacturer's certification that the erosion mat/blanket supplied meets the criteria specified when the material was approved by ENGEO. The manufacturer's certification shall include a submittal package of documented test results that confirm the property values. In case of a dispute over validity of values, the Contractor will supply property test data from an ENGEO-approved laboratory, to support the certified values submitted. Minimum average roll values, per ASTM D 4759, shall be used for conformance determinations.

3. CONSTRUCTION:

   3.1 Delivery, Storage, and Handling: Contractor shall check the erosion control material upon delivery to ensure that the proper material has been received. During all periods of shipment and storage, the erosion mat shall be protected from temperatures greater than 140 °F, mud, dirt, and debris. Manufacturer's recommendations in regard to protection from direct sunlight must also be followed. At the time of installation, the erosion mat/blanket shall be rejected if it has defects, tears, punctures, flaws, deterioration, or damage incurred during manufacture, transportation, or storage. If approved by ENGEO, torn or punctured sections may be removed by cutting OUT a section of the mat. The remaining ends should be overlapped and secured with ground anchors. Any erosion mat/blanket damaged during storage or installation shall be replaced by the Contractor at no additional cost to the Owner.

   3.2 On-Site Representative: Erosion control material suppliers shall provide a qualified and experienced representative on site, for a minimum of one day, to assist the Contractor and ENGEO personnel at the start of construction. If there is more than one slope on a project, this criteria will apply to construction of the initial slope only. The representative shall be available on an as-needed basis, as requested by ENGEO, during construction of the remaining slope(s).

   3.3 Placement: The erosion control material shall be placed and anchored on a smooth graded, firm surface approved by the Engineer. Anchoring terminal ends of the erosion control material shall be accomplished through use of key trenches. The material in the trenches shall be anchored to the soil on maximum 1½ foot centers. Topsoil, if required by construction drawings, placed over final grade prior to installation of the erosion control material shall be limited to a depth not exceeding 3 inches.

3.4 Erosion control material shall be anchored, overlapped, and otherwise constructed to ensure performance until vegetation is well established. Anchors shall be as designated on the construction drawings, with a minimum of 12 inches length, and shall be spaced as designated on the construction drawings, with a maximum spacing of 4 feet.

3.5 Soil Filling: If noted on the construction drawings, the erosion control mat shall be filled with a fine grained topsoil, as recommended by the manufacturer. Soil shall be lightly raked or brushed on/into the mat to fill the mat voids or to a maximum depth of 1 inch.

PART V - GEOSYNTHETIC DRAINAGE COMPOSITE

1. DESCRIPTION:

   Work shall consist of furnishing and placing a geosynthetic drainage system as a subsurface drainage medium for reinforced soil slopes.

2. DRAINAGE COMPOSITE MATERIALS:

   2.1 The specific drainage composite material and supplier shall be preapproved by ENGEO.

   2.2 The drain shall be of composite construction consisting of a supporting structure or drainage core material surrounded by a geotextile. The geotextile shall encapsulate the drainage core and prevent random soil intrusion into the drainage structure. The drainage core material shall consist of a three dimensional polymeric material with a structure that permits flow along the core laterally. The core structure shall also be constructed to permit flow regardless of the water inlet surface. The drainage core shall provide support to the geotextile. The fabric shall meet the minimum property requirements for filter fabric listed in Section 2.05C of the Guide Earthwork Specifications.

   2.3 A geotextile flap shall be provided along all drainage core edges. This flap shall be of sufficient width for sealing the geotextile to the adjacent drainage structure edge to prevent soil intrusion into the structure during and after installation. The geotextile shall cover the full length of the core.

   2.4 The geocomposite core shall be furnished with an approved method of constructing and connecting with outlet pipes or weepholes as shown on the plans. Any fittings shall allow entry of water from the core but prevent intrusion of backfill material into the core material.

   2.5 Certification and Acceptance: The Contractor shall submit a manufacturer's certification that the geosynthetic drainage composite meets the design properties and respective index criteria measured in full accordance with all test methods and standards specified. The manufacturer's certification shall include a submittal package of documented test results that confirm the design values. In case of dispute over validity of design values, the Contractor will supply design property test data from an ENGEO-approved laboratory, to support the certified values submitted. Minimum average roll values, per ASTM D 4759, shall be used for determining conformance.

3. CONSTRUCTION:

   3.1 Delivery, Storage, and Handling: Contractor shall check the geosynthetic drainage composite upon delivery to ensure that the proper material has been received. During all periods of shipment and storage, the geosynthetic drainage composite shall be protected from temperatures greater than 140 °F, mud, dirt, and debris. Manufacturer's recommendations in regards to protection from direct sunlight must also be followed. At the time of installation, the geosynthetic drainage composite shall be rejected if it has defects, tears, punctures, flaws, deterioration, or damage incurred during manufacture, transportation, or storage. If approved by ENGEO, torn or punctured sections may be removed or repaired. Any geosynthetic drainage composite damaged during storage or installation shall be replaced by the Contractor at no additional cost to the Owner.

3.2 On-Site Representative: Geosynthetic drainage composite material suppliers shall provide a qualified and experienced representative on site, for a minimum of one half day, to assist the Contractor and ENGEO personnel at the start of construction with directions on the use of drainage composite. If there is more than one application on a project, this criterion will apply to construction of the initial application only. The representative shall also be available on an as-needed basis, as requested by ENGEO, during construction of the remaining applications.

3.3 Placement: The soil surface against which the geosynthetic drainage composite is to be placed shall be free of debris and inordinate irregularities that will prevent intimate contact between the soil surface and the drain.

3.4 Seams: Edge seams shall be formed by utilizing the flap of the geotextile extending from the geocomposite's edge and lapping over the top of the fabric of the adjacent course. The fabric flap shall be securely fastened to the adjacent fabric by means of plastic tape or non-water-soluble construction adhesive, as recommended by the supplier. Where vertical splices are necessary at the end of a geocomposite roll or panel, an 8-inch-wide continuous strip of geotextile may be placed, centering over the seam and continuously fastened on both sides with plastic tape or non-water-soluble construction adhesive. As an alternative, rolls of geocomposite drain material may be joined together by turning back the fabric at the roll edges and interlocking the cuspidations approximately 2 inches. For overlapping in this manner, the fabric shall be lapped and tightly taped beyond the seam with tape or adhesive. Interlocking of the core shall always be made with the upstream edge on top in the direction of water flow. To prevent soil intrusion, all exposed edges of the geocomposite drainage core edge must be covered. Alternatively, a 12-inch-wide strip of fabric may be utilized in the same manner, fastening it to the exposed fabric 8 inches in from the edge and folding the remaining flap over the core edge.

3.5 Soil Fill Placement: Structural backfill shall be placed immediately over the geocomposite drain. Care shall be taken during the backfill operation not to damage the geotextile surface of the drain. Care shall also be taken to avoid excessive settlement of the backfill material. The geocomposite drain, once installed, shall not be exposed for more than seven days prior to backfilling.

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,

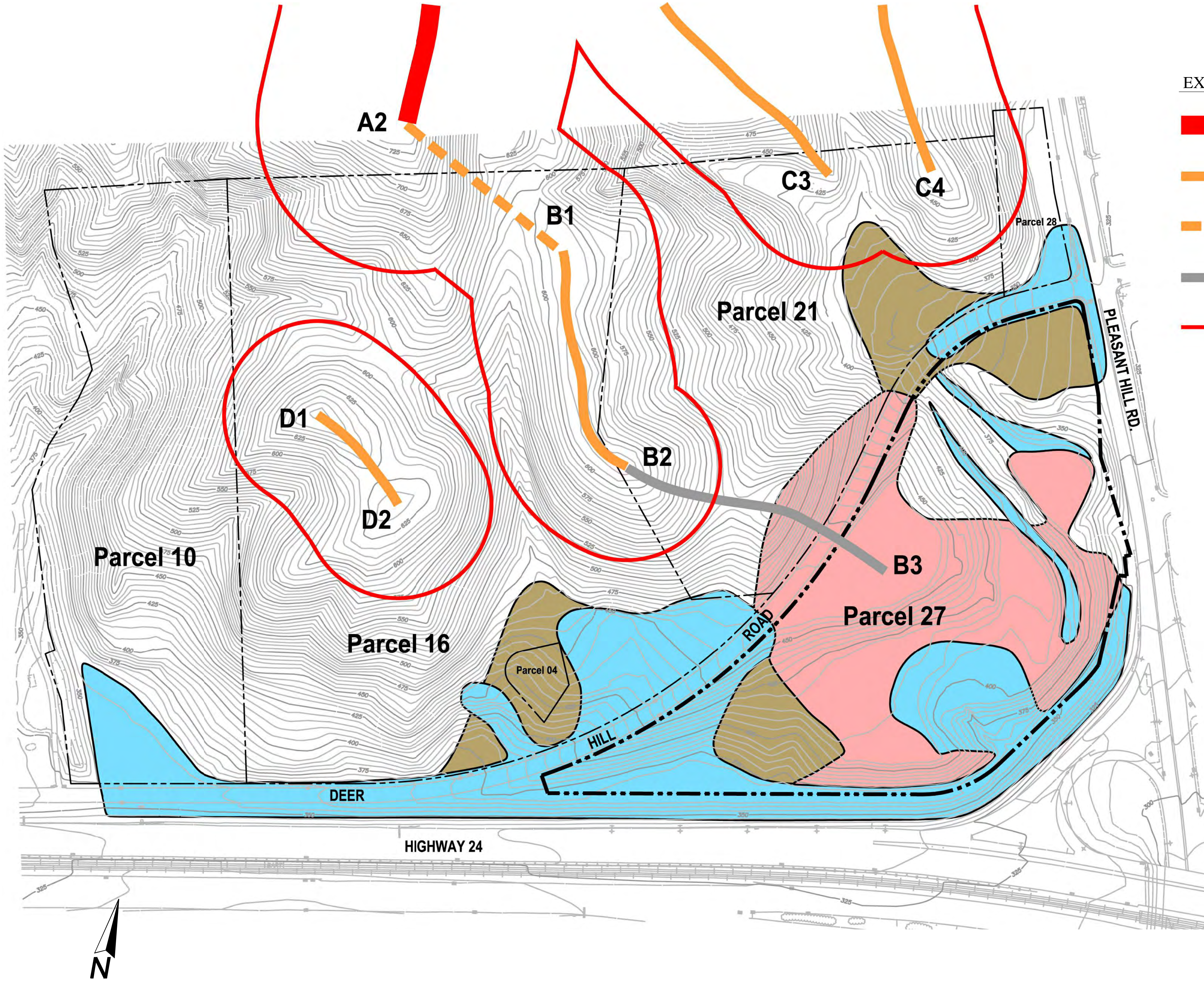
ENGEO Incorporated

Philip J. Stuecheli, CEG
Associate

Daniel S. Haynosch GE
Principal

Attachment: Figure 1

Cc: Norm Dyer, LCA



EXPLANATION

CLASS I RIDGE (400' SETBACK)

CLASS II RIDGE (250' SETBACK)

IMPLIED RIDGE (NO FEATURE)

NO RIDGE (HODM BASED ON OUTDATED TOPO)

RIDGE SETBACK

AREAS OF SIGNIFICANT CUT

AREAS OF SIGNIFICANT FILL

AREAS OF OTHER DISTURBANCE

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. 1
SCALE: AS SHOWN	
DRAWN BY: PC	CHECKED BY: PS

G:\Drafting\DRAFTING2_Dwg\9181\000\Letter\9181000100-1-DeerHillPlanningArea-0911.dwg Plot Date: 9-01-11 pcrockett

ORIGINAL FIGURE PRINTED IN COLOR