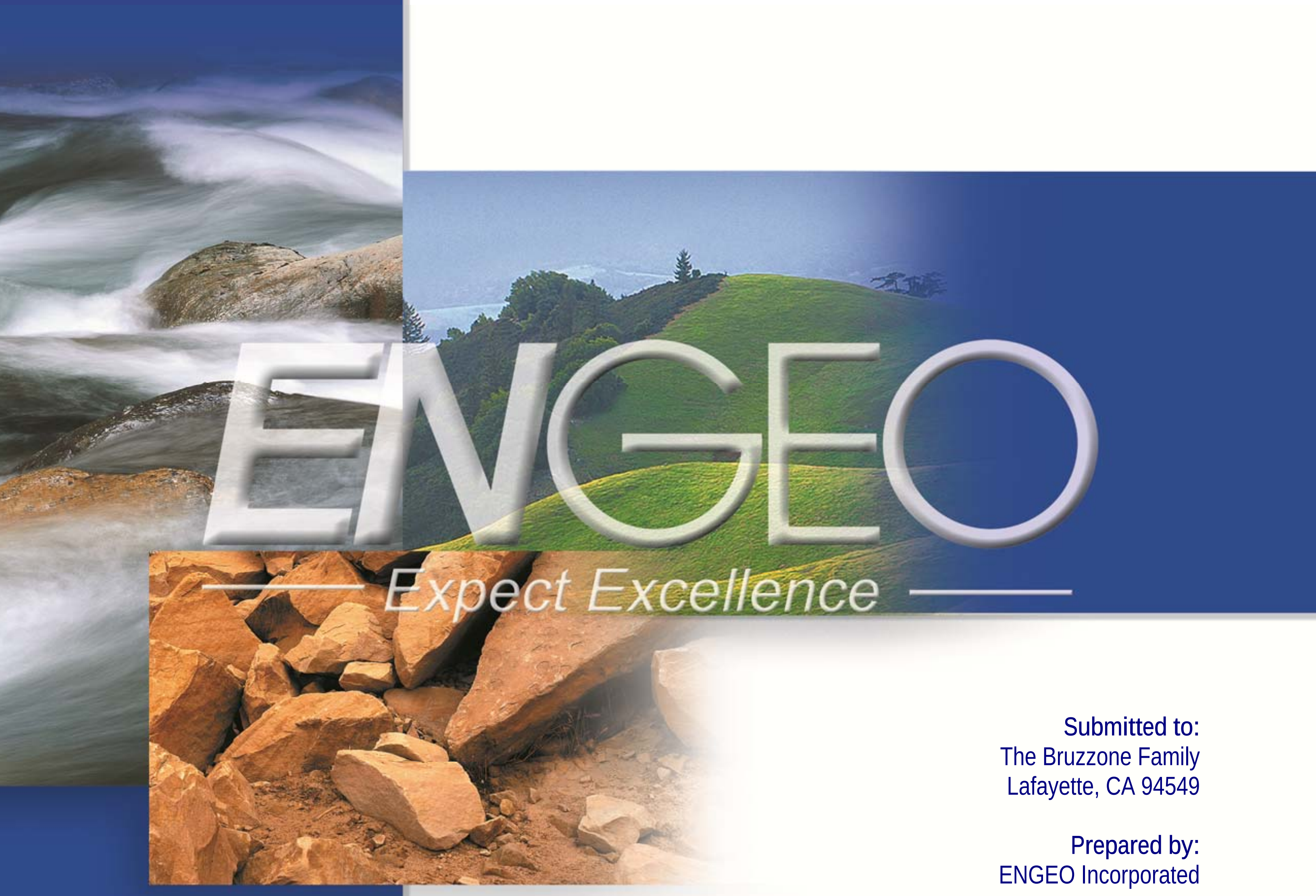
UPDATED PRELIMINARY GEOTECHNICAL REPORT

INDIAN VALLEY PROJECT
MORAGA, CALIFORNIA



Submitted to:
The Bruzzone Family
Lafayette, CA 94549

Prepared by:
ENGEO Incorporated

June 8, 2015
Revised June 24, 2015

Project No.
5900.200.000

- Expect Excellence -

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Expect Excellence

GEOTECHNICAL
ENVIRONMENTAL
WATER RESOURCES
CONSTRUCTION SERVICES

Project No.
5900.200.000

June 8, 2015
Revised June 24, 2015

The Bruzzone Family
c/o David Bruzzone
899 Hope Lane
Lafayette, CA 94549

Subject: Indian Valley Project
Moraga, California

UPDATED PRELIMINARY GEOTECHNICAL REPORT

Dear Mr. Bruzzone:

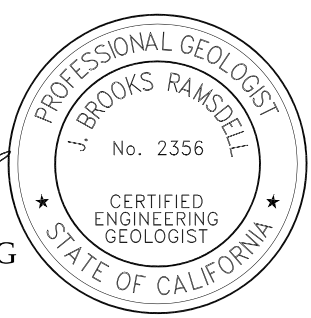
As requested, we prepared this updated preliminary geotechnical report including supplemental exploration to collect additional laboratory and field strength data for further characterization of the onsite soils.. This additional work serves to update the original report dated July 31, 2003. The accompanying report presents our field exploration and laboratory testing with our conclusions and recommendations regarding residential development at the site.

It is our opinion that the proposed development is feasible from a geotechnical standpoint, provided that appropriate mitigation of geologic hazards and design considerations as presented in this report are incorporated into the plans and implemented during construction for this project. It is recommended that an additional design level geotechnical exploration be performed as part of design development phase of the project.

We are pleased to provide our services to you on this project and look forward to consulting further with you and your design team.

Sincerely,

ENGEO Incorporated



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1.0 INTRODUCTION

1.1 PURPOSE AND SCOPE

We completed a preliminary geotechnical report for the project dated July 31, 2003. As discussed in our 2003 report, we identified several geotechnical considerations including expansive soils, slope stability (existing landslides, debris flow hazards, and creek bank stability), and seismic hazards. An important geotechnical consideration is slope stability with respect to Indian Creek, which is one of the primary focuses of our study. Since the time of our 2003 report, there have been changes to the development plan. In addition, the building code and standards for seismic slope stability analysis have changed significantly since then. The purpose of our study is to provide updated conclusions regarding the geotechnical considerations for the project and updated preliminary recommendations in support of project planning and preliminary design with consideration to these changes.

The scope of our services included the following:

- Review of available geologic maps; historical topographic maps; and available aerial photographs borrowed from PA Design Associates.
- Performance of four cone penetration test (CPT) soundings to a depth of approximately 78 feet below the ground surface.
- Performance of two auger borings to depths of up to 81 feet below the ground surface.
- Analysis of the geological and geotechnical data including slope stability analysis.
- Laboratory testing of select samples, including verification of field classifications; measurement of dry density and moisture content of selected samples; particle-size distribution; Plasticity Index, and strength.
- Preparation of this report which compiles both the current and previous data and summarizes our findings and preliminary geotechnical design recommendations.

This report was prepared for the exclusive use of the Bruzzone Family and their design team consultants. In the event that any changes are made in the character, design, or layout of the development, we should be contacted to review the conclusions and recommendations contained in this report to determine whether modifications to the report are necessary. Site Location and Description

The subject property comprises approximately 450 acres located southwest of Moraga Way and north of Canyon Road in the lower valley of Indian Creek approximately ½ mile upstream from the confluence with San Leandro Creek in Moraga, California, as shown on Figure 1.

Natural slope gradients range from very gently sloping on the alluvial deposits in the western portion of the site adjacent to Indian Creek, to as steep as 1:1 (horizontal:vertical) or nearly vertical in a few localized areas on the flanks of the northwest trending ridge line adjacent to the northeastern property boundary. Total relief on the site is approximately 555 feet, with site elevations ranging from approximately 515 feet above mean sea level at the base of the creek channel in southwest corner of the site to approximately 1070 feet at the highest location along the ridge line in the northern portion of the site.

The property can generally be characterized as a grass- and tree-covered northwest trending alluvial valley drained by Indian Creek. The valley margins are defined by Gudde Ridge on the northeast and an unnamed ridge to the southwest. The valley slopes southwest of Indian Creek are heavily vegetated with brush and trees. Drainage on the site is generally towards Indian Creek, which flows to the southeast. The site is generally undeveloped with the exception of a detached residence with associated maintenance buildings in the southwestern portion of the site. Based on review of historic aerial photos of various years dating back to 1939, the majority of the property has been used for cattle grazing as it is at the time of writing.

1.2 PROPOSED DEVELOPMENT

Based on plans prepared by PA Design Associates dated June 22, 2015, the proposed development will include 71 single-family residential lots, situated between Indian creek and the northeast ridgeline as shown on Figure 2. The main entry roadway is planned extending from Canyon Road north through the project development with branched interior roads and drives for lot access. The drawings show 3:1 (horizontal:vertical) fill slopes up to approximately 40 feet in height and 3:1 cut slopes up to approximately 100 feet in height. In addition, plans include several detention basins and bio-retention basins southwest of the main branch roadway and northeast of an emergency vehicle access road that parallels Indian Creek. Site improvements will also include underground utilities associated with the development.

1.3 PREVIOUS GEOTECHNICAL AND GEOLOGICAL STUDY

In 2003, we performed a preliminary geotechnical exploration at the site. Our previous exploration included a review of geologic literature and maps, examination of aerial photographs, geologic mapping, exploratory test pits and borings, laboratory testing of select samples, and preparation of a report dated July 31, 2003. As discussed above, we identified several geotechnical considerations including slope stability with respect to Indian Creek. Test pit logs, boring logs, and laboratory test results from this previous exploration are presented in Appendix C.

2.0 GEOLOGY AND SEISMICITY

2.1 GEOLOGIC SETTING

The Indian Valley site is located in the East Bay Hills between Gudde Ridge and Indian Creek. The East Bay Hills lie within the region of coastal California referred to by geologists as the Coast Ranges geomorphic province. The Coast Ranges have experienced a complex geological history characterized by Late Tertiary folding and faulting that has resulted in a series of northwest-trending mountain ranges and intervening valleys. The site is located within an uplifted range of hills locally referred to as the East Bay Hills block, bounded on the west by the active Hayward Fault and on the east by the active Calaveras Fault.

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

Based on geologic mapping by Graymer (1994) the majority of the site is underlain by units of the Orinda Formation and the Moraga Volcanics (Figure 3). Crane (1988) maps a northwestern trending, southwest-dipping thrust fault along the axis of Indian Creek concealed beneath the valley floor alluvium. This thrust fault is not considered to be active by the California State Geological Survey or by the United States Geological Survey. Several northeast to southeast tear faults are mapped along the ridgeline at the contact between Orinda Formation and Moraga Volcanics. A tear fault is as relatively small-scale local strike-slip fault associated with another larger-scale structure such as a thrust fault or a normal fault. These tear faults are the resultant of compression from the west during uplift of the East Bay Hills block and considered inactive and not laterally extensive across the site. The axis of a northwest-trending syncline is mapped approximately along the crest of Gudde Ridge (Crane 1988). In general, bedding at the site is mapped with a northwestern strike with dips ranging from about 45 to 86 degrees towards the northeast (Graymer, 1994; Crane, 1988; Radbruch, 1969). Figure 4 presents regional landslide mapping by Nilsen (1973).

2.1.2 Geologic Mapping

During our exploration, ENGEO geologists performed geologic mapping at the site. Below are descriptions of the geologic units observed during mapping and encountered during our explorations at the site as shown on Figure 2. Relatively thin fills associated with ranching activities have been placed in isolated portions of the site; however, these areas observed were too small to depict on the site plan included as Figure 2.

2.1.2.1 Landslide Debris (Qls)

As with most of the surrounding Moraga/Orinda Hills, landslides are a primary geotechnical consideration at the Indian Valley site. Landslide deposits identified during this study were mapped using stereo-paired aerial photographs and field checked during site reconnaissance and field explorations. Known and suspected landslides are shown on Figure 2. The types of landslides identified on this property include the following:

- Debris Flows: Debris flows are a type of landslide that can form during peak rainfall events when colluvium becomes saturated and fails, forming a fluid, mobile soil mass. Typically the formation and mobilization of debris flows is most likely on slopes that are inclined at 2:1(horizontal:vertical) or steeper, and where the colluvium has a relatively low clay content. Under these conditions, debris flows have been known to travel considerable distances from the source, sometimes entraining trees and boulders in their path. Several soil slide/debris flows as defined by Turner (1996) and potential debris flow source areas are present along the southwest flank of the steep-sided ridgeline northeast of the proposed development (Figure 2).

- Earthflows: Earthflows are a type of landslide characterized by mobilization as a viscous, slow-moving mass. Earthflows commonly move by a combination of semi-fluid flow and sliding along weak clay slip planes. Earthflows typically form when cohesive, clayey soils or weak bedrock become saturated and fail. Like debris flows, they commonly mobilize as a result of intense rains, but due to their high clay content they tend to move relatively slowly. Earthflows often accumulate as lobate masses of soil with complex internal shearing. A number of earthflows and earthflow complexes have been mapped at the site as shown on Figure 2. In general, these features occupy drainage swales on the steep-sided flanks of the ridges at the site.

- Deep-Seated Landslides: A number of possible deep-seated landslides have been identified on the ridge slopes southwest of Indian Creek (the side opposite the proposed development area), as shown of Figure 2. The landslides appear to be dormant, are heavily vegetated, and show no geomorphic evidence of recent activity.

2.1.2.2 Residual Soil and Colluvium (Qc)

The ground surface is typically mantled with 1 to 5 feet of residual soil formed from weathering and decomposition of the underlying bedrock and alluvium. Colluvium is a soil deposit formed from downslope movement and deposition of residual soil by such processes as slopewash, sloughing/shallow sliding, and creep.

The colluvium typically consists of silty clay with some sand and scattered rock fragments. The composition of the residual soils typically varies based on the underlying parent material. On the site, the weathering of the underlying soils and bedrock typically produces a silty clay soil with moderate to high shrink swell potential. Mappable deposits of colluvium (typically thicker than 5 feet) occur in swales and ravines and at the bases on many slopes on the property.

2.1.2.3 Alluvial/Colluvial Fan Deposits (Qafd)

The relatively low-lying landforms present along the northeast side of Indian Creek consist of a series of fan-like lobes originating within swales on the ridge flanks, and merging to the southwest adjacent to the incised creek channel. These landforms are interpreted to be alluvial/colluvial fans deposited by debris flows and stream flows originating on the steep ridge slopes. The average slope on the surface of these landforms is approximately 8:1 (horizontal:vertical). The fan deposits at the site are generally fine-grained comprising silty and sandy clay.

2.1.2.4 Bedrock Formations

Bedrock at the site includes the Upper Miocene-age terrestrial derived Orinda Formation and Upper Miocene-age Moraga Formation. In general, the Orinda formation bedrock consists of light brown, reddish brown, and gray, friable to moderately strong, massive to thinly interbedded sandstone, siltstone, claystone, and conglomerate. The Moraga Formation typically consists of reddish brown to gray on andesite, basalt, and volcanoclastic sandstone and generally makes up the higher elevation and steeper terrain at the site. Bedding structure generally strikes northwest and is dipping 23 degrees northeast in the northern portion of the site to 85 degrees northeast in the southern portion of the site.

2.2 FAULTING AND SEISMICITY

Because of the presence of nearby active faults, the Bay Area Region is considered seismically active. 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). Numerous small earthquakes occur every year in the region, and large (greater than moment magnitude 7) 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 and no Holocene active faults are known to pass through the project site, according to published geologic maps (Jennings, 2010; Graymer, 1994; Crane, 1988). Figure 5 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 Hayward Fault located approximately 3 miles southwest of the site. Other active faults located near the site include the Northern Calaveras Fault located approximately 5 miles to the northeast, Concord-Green Valley Fault located approximately 10 miles northeast of the site, and the San Andreas Fault located approximately 21 miles to the west. Many earthquakes of low magnitude occur every year throughout the region, most are concentrated along the San Andreas, Hayward, and Calaveras Faults.

The Uniform California Earthquake Rupture Forecast (UCERF3, 2014) evaluated the 30-year probability of a moment magnitude 6.7 or greater earthquake occurring on the known active fault systems in the Bay Area, including the Hayward Fault. The UCERF3 generated an overall probability of 72 percent for the Bay Area as whole.

3.0 FIELD EXPLORATION

The field exploration for this study was conducted from August 12 to 14, 2013, and consisted of advancing four CPTs and two hollow stem auger borings at the approximate locations shown on Figure 2. The locations of our subsurface borings and CPT's for this study were chosen specifically to characterize subsurface conditions along a critical cross section through the alluvial fan deposits at the project site (Figure 2). The field exploration locations were obtained by taping or pacing from existing features; therefore, they should be considered accurately located only to the degree implied by the method used.

The subsurface logs depict subsurface conditions at the time the exploration was conducted. Subsurface conditions at other locations may differ from conditions encountered 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.

3.1 CONE PENETRATION TEST PROBES

The cone penetration test (CPT) probes were advanced to depths of approximately 25 to 78 feet below ground surface (bgs) before meeting practical refusal (high tip resistance) or target depth.

The CPTs were performed in general accordance with ASTM test D-5778. The CPT logs and supporting empirical data are located in Appendix A.

3.2 AUGER TEST BORINGS

The test borings were drilled using a track-mounted drill rig equipped with hollow-stem augers and an automatic-trip safety hammer. The borings were advanced to depths ranging from approximately 74½ and 81 feet below ground surface. An ENGEO geologist logged the borings in the field and collected soil samples using either a 3-inch outside diameter (O.D.) California-type split-spoon sampler fitted with 6-inch-long stainless steel liners or a 2-inch outside diameter (O.D.) Standard Penetration Test split-spoon sampler. The samplers were driven with a 140-pound safety hammer falling a distance of 30 inches employing an automatic trip system.

We recorded the penetration of the samplers into the native materials as the number of blows needed to drive the sampler 18 inches in 6-inch increments. The boring logs record blow count results as the actual number of blows required for the last 1 foot of penetration; no conversion factors have been applied. When sampler driving was difficult, penetration was recorded as inches penetrated for 50 hammer blows. We used the field logs to develop the report boring logs, which are presented in Appendix A. Select samples were collected during drilling and transported back to our laboratory for testing.

3.3 LABORATORY TESTING

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

TABLE 3.3-1
Laboratory Testing

Soil Characteristic	Testing Method	Location of Results
Natural Unit Weight and Moisture Content	ASTM D-2216	Appendix A
Atterberg Limits	ASTM D-4318	Appendix A and B
Grain Size Distribution	ASTM D-422	Appendix A and B
Unconfined Compression	ASTM D-2166	Appendix B
Triaxial Compression – Consolidated Undrained	ASTM D-4767	Appendix B
Triaxial Compression – Unconsolidated Undrained	ASTM D-2850	Appendix B
Compaction Curve	ASTM D-1557	Appendix B

The laboratory test results are shown on the borelogs (Appendix A), with individual test results presented in Appendix B. Laboratory test results from our 2003 exploration are included in Appendix C.

3.4 SUBSURFACE CONDITIONS

Subsurface conditions encountered during the current exploration are generally consistent with those encountered during our 2003 exploration at the site. Borings in the fan deposits generally encountered fine-grained alluvial deposits to a depth up to approximately 72 feet. The thickness of the fan deposits generally increases near the center of the valley in the vicinity of Indian Creek and generally decreases in thickness upslope and away from the creek. The fan deposits encountered typically comprise medium stiff to hard lean clay with occasional lenses of medium dense clayey sand. We encountered weak and freshly weathered siltstone bedrock of the Orinda Formation underlying the fan deposits.

The boring, CPT, and test pit logs include specific subsurface conditions encountered at each exploration location. We include our exploration logs in Appendix A and the exploration logs from our 2003 exploration in Appendix C. The boring logs describe the soil type in general accordance with the Unified Soil Classification System (USCS) as well as color, moisture level, and consistency or density. The exploration logs depict the subsurface conditions encountered at the time of the exploration.

3.5 GROUNDWATER

Groundwater was encountered in Boring 2-B1 at a depth of approximately 40 feet and in Boring 2-B2 at approximately 49 feet, with perched groundwater encountered at 10 feet. Groundwater was observed during our 2003 field exploration at a depth of approximately 38 feet in Boring B-1, 19½ feet in Boring B-4, 18 feet in Boring B-5, and 23½ feet in Boring B-6. Several natural springs were observed at the project site. We observed a significant spring flowing with a flowrate of approximately estimated several hundred gallons of water per day; the location of this spring is shown on Figure 2 upslope of TP-1.

Fluctuations in groundwater levels occur seasonally and over a period of years because of variations in precipitation, temperature, irrigation, and other factors.

4.0 DISCUSSION AND CONCLUSIONS

Based on our explorations, laboratory test results and analysis, we conclude that the development of the proposed Indian Valley Project is feasible from a geotechnical standpoint. The recommendations included in this report, along with sound engineering practices, should be incorporated in the design and construction of the project. The site was evaluated with respect to known geologic and other hazards common to the greater San Francisco Bay Region. Geotechnical considerations for this project include expansive soils, slope stability (existing landslides, debris flow hazards, and creek bank stability), and seismic hazards. A primary geotechnical consideration is slope stability with respect to Indian Creek. . 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 EXPANSIVE SOIL

A significant geotechnical consideration is the expansive nature of the native soil across the proposed development area. The clayey soils encountered at the site have moderate to high expansion potential with Plasticity Indices (PI) that range between 17 and 39 over the project site. Based on laboratory testing the upper 15 feet generally yield higher PIs.

Expansive soils shrink and swell as a result of seasonal fluctuation in moisture content. This can 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 through proper foundation design.

Successful construction on expansive soils requires special attention during construction. It is imperative that exposed soils be kept moist by watering for several days before placement of concrete. It is extremely difficult to remoisturize clayey soils without excavation, moisture conditioning, and recompaction. Mitigation measures should include the prevention of moisture variation.

4.2 LANDSLIDES

As with most of the surrounding hillside developments, landslides are one of the primary geologic considerations at the Indian Valley site. Potentially unstable hillside deposits observed on the property include debris flows, earthflows, and earthflow complexes, Figure 2. Although slope instability can be a significant hazard, it can generally be mitigated through proper grading procedures. General mitigation measures include a combination of the following: removing landslide debris; replacing landslides with engineered fill; and providing toe buttresses, debris benches, deflection berms, debris catchment areas, and setback areas. Additional site-specific slope stability analyses should be performed during review of the final 40-scale grading plans. Landslide mitigation measures should be designed into the grading plans where improvements are planned immediately downslope of these hazards. The specific location, extent, and depth of the required landslide mitigation should be presented on the final grading plans.

4.3 SEISMIC HAZARDS

Potential seismic hazards resulting from a nearby moderate to major earthquake can generally be classified as primary and secondary. The primary effect is ground rupture, also called surface faulting. The common secondary seismic hazards include ground shaking, 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 are considered negligible at the site. The following sections present a discussion of these hazards as they apply to the site.

4.3.1 Ground Rupture

No known active faults have been mapped within the Indian Valley property. No evidence of Holocene active faulting was observed during our site reconnaissance and aerial photo review. Based on our previous study, field mapping, and review of aerial photographs, it is our opinion that fault-related ground rupture is unlikely at the subject property.

4.3.2 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 latest California Building Code (CBC) requirements, as a minimum.

4.3.2.1 California Building Code Seismic Design Parameters

The 2013 California Building Code (CBC) seismic design parameters in the table below are based on a Site Class D as determined in accordance with ASCE 7-10. The parameters below include design spectral response acceleration parameters based on the mapped Risk-Targeted Maximum Considered Earthquake (MCE_R) spectral response acceleration parameters as well as the Maximum Considered Earthquake geometric mean peak ground acceleration used for geotechnical evaluation.

TABLE 4.3.2.1-1
2013 CBC Seismic Design Parameters

Parameter	Design Value
Site Class	D
Mapped MCE_R Spectral Response Acceleration at Short Periods, S_S (g)	1.799
Mapped MCE_R Spectral Response Acceleration at 1-second Period, S_1 (g)	0.718
Site Coefficient, F_A	1.00
Site Coefficient, F_V	1.50
MCE_R Spectral Response Acceleration at Short Periods, S_{MS} (g)	1.799
MCE_R Spectral Response Acceleration at 1-second Period, S_{M1} (g)	1.077
Design Spectral Response Acceleration at Short Period, S_{DS} (g)	1.199
Design Spectral Response Acceleration at 1-second Period, S_{D1} (g)	0.718
Mapped MCE Geometric Mean Peak Ground Acceleration (g)	0.694
Site Coefficient, F_{PGA}	1.0
MCE Geometric Mean Peak Ground Acceleration, PGA_M (g)	0.694

4.3.3 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. The potential for the formation of these cracks is considered greater at contacts between deep alluvium and bedrock such as those at the margins of valley flood plains. Although the risk of ground lurching at the site is considered low, the risk of this hazard will be reduced through implementation of typical corrective grading measures.

4.3.4 Liquefaction and Cyclic Softening

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. In addition, sensitive high-plasticity fine-grained soils may be susceptible to significant strength loss (cyclic softening) as a result of significant cyclic loading, such as imposed by earthquakes. We summarize the results of our liquefaction analysis below.

We evaluated the liquefaction potential of the site soil with SPT data using methods published by Youd et al. (2001) and with CPT data using methods published by Robertson (2009). The Cyclic Stress Ratio (CSR) was estimated for a Peak Ground Acceleration (PGA_M) value of 0.694g, which is the mapped Maximum Considered Earthquake (MCE) Geometric Mean Peak Ground Acceleration based on the 2013 CBC for a Site Class D as discussed above. We also used a moment magnitude (M_w) of 7.3 in our analysis, which corresponds to the maximum magnitude for the Hayward-Rogers Creek fault based on the United States Geological Survey (USGS) national seismic hazard maps. Our analysis is based on the use of a groundwater depth of approximately 20 feet.

The effects of liquefaction as they relate to the proposed development include strength loss and seismic settlement. Based on the results of our SPT-based analysis, we estimate that relatively thin lenses (up to approximately 1½ feet thick) of medium dense sand encountered within the alluvium in Boring 2-B1 are potentially liquefiable. We did not encounter liquefiable sand layers in CPT-1 performed adjacent to Boring 2-B1, which indicates that the potentially liquefiable sands encountered in Boring 2-B1 are localized. Based on the results of our CPT-based analysis, we estimate that relatively thin layers (up to approximately ½ foot thick) of sand encountered within the alluvium in CPT-2 is potentially liquefiable. We did not encounter liquefiable sand layers in other CPTs performed at the site. This indicates that the potentially liquefiable sands encountered in Boring 2-B1 and CPT-2 are localized and not pervasive at the site. Given the discontinuous nature of the liquefiable sand encountered during our exploration, we expect that strength loss as a result of liquefaction would have a nominal impact on slope stability. In addition, based on the results of our analyses, we estimate less than 1 inch of seismic settlement resulting from liquefaction. We include preliminary recommendations below to address this effect.

The effects of cyclic softening as they relate to the proposed development include strength loss and seismic settlement. The results of our CPT-based analysis indicate that there are layers of clay that are subject to cyclic softening. However, based on the results of our laboratory testing, which indicate Plasticity Indices ranging from 17 to 33, and the generally stiff to hard consistency and low sensitivity of these clay layers, we consider the risk of cyclic softening and resulting significant strength loss or settlement to be low.

4.3.5 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 this site is considered low.

4.4 SLOPE STABILITY

4.4.1 Methods of Analysis

We performed two-dimensional limit-equilibrium slope stability analyses of a critical slope located in the northern portion of the development site (Cross Section A) with the computer slope stability software Slide Version 6.0 using Spencer's method (Spencer, 1967). Cross Section A was selected as a critical slope as it is an area of one of the larger and steeper alluvial fans where the tallest fill slope is planned. According to current grading plans, the proposed fill slope in this area consists of a 30-foot-high 3:1 (horizontal:vertical) slope approximately 220 feet from the existing flow line of the creek (PA Design Associates, 2015). Figure 2 shows the location of Cross Section A, which is included on Figure 9. A conservative groundwater table was assumed at roughly 25 feet below existing grade depending on location along the slope.

4.4.2 Estimation of Shear Strength

In order to estimate shear strengths for the alluvial soil, we performed consolidated-undrained triaxial compression and unconfined compression testing on select samples of alluvium; we used the undrained shear strengths from the results of this testing to estimate undrained shear strengths interpreted from the CPT data. For the purposes of slope stability analysis, we divided the alluvium into Upper and Lower Alluvium based on the strength data. In addition, for the evaluation of engineered fill strengths, we performed consolidated-undrained and unconsolidated-undrained triaxial compression testing on remolded soil samples of alluvium. We estimated the effective stress strength parameters for the alluvium based on the remolded consolidated-undrained triaxial compression testing. The remolded samples were compacted to 90 percent relative compaction at 2 percentage points above optimum moisture content. Bedrock material was modeled using equivalent Mohr-Coulomb strength parameters estimated from a Generalized Hoek-Brown shear-normal function. A summary of the shear strength parameters used in our slope stability analysis is provided in Table 4.4.2-1.

TABLE 4.4.2-1
Summary of Shear Strength Parameters

Material	Effective Stress (Drained) Strength Parameters		Total Stress (Undrained) Strength Parameters	
	Friction Angle (degrees)	Cohesion (psf)	Friction Angle (degrees)	Cohesion (psf)
Engineered Fill (Qef, proposed)	18	700	0	3500
Upper Alluvium - within upper 10 feet (Qal)	18	700	0	2500
Lower Alluvium – (Qal)	18	700	0	1500[1]
Bedrock (Tor)	0	5000	0	5000

1. Strength increase of 30 psf per foot of depth.

4.4.3 Results of Static Slope Stability Analysis

Appendix C shows the results of our static stability for Cross Section A. The results would indicate shallow failure surfaces with low factors of safety within the steep creek bank slopes, which would be expected given the relatively low strengths used to model the alluvium in this area. For the purposes of this study, we evaluated potential failure surfaces that would impact the site in our analyses, the results of which are summarized below in Table 4.4.3-1. The analyses indicate a factor of safety above commonly accepted criteria.

TABLE 4.4.3-1
Static Stability

Section	Calculated Static Factor of Safety
A	3.0

4.4.4 Seismic Slope Stability Analyses

Based on guidelines published by the California Geological Survey in Special Publication 117A, we used the screening analysis procedure developed by Stewart et al. (2003) to estimate a seismic coefficient for pseudo-static slope stability analysis that accounts for anticipated site seismicity for a given level of displacement (CGS, 2008). Based on this procedure, for our pseudo-static analysis we applied a factor of 0.4 to the Design Earthquake peak ground acceleration (0.48g) based on an earthquake Moment Magnitude of 7.3, for a threshold slope displacement of 15 centimeters (6 inches). Our pseudo-static analysis results in a factor of safety greater than 1.1, which indicates that the slope evaluated is unlikely to experience serious movement and damage under the design earthquake event.

4.5 SETTLEMENT OF FILL

Studies have shown that engineered fills in residential developments typically experience increases in moisture content after building construction due to increases in irrigation or natural infiltration and to alteration of drainage patterns. This process may take about 5 to 10 years after irrigation commences, or even more, before the fill becomes fully wetted. The wetting process can cause settlement or swell depending on soil type, compaction, moisture content, and overburden pressures (fill thickness). According to the preliminary plans, fills up to 40 feet in thickness are proposed.

We recommend that once 40-scale grading plans have been developed for the project, that swell/settlement potential tests be performed on soil and rock materials collected during design level investigation. Soil samples of the representative materials should be tested for swell/settlement potential by wetting and loading to the equivalent overburden pressures representing the proposed fill conditions. Test results should be used to establish compaction and moisture content requirements for placement of the site soils and bedrock materials.

4.6 CREEKBANK EROSION AND SLOUGHING

The creek bank paralleling the project site is steeply incised and several areas of minor sloughing and erosion were observed during our site reconnaissance. Care should be given when siting improvements in the vicinity of the top of creek channel banks. Mitigation measures to protect proposed improvements from potential creek bank failures may include Creek bank reconstruction with keyways/subdrainage and/or below grade retaining walls.

4.7 GROUNDWATER

Perched groundwater was encountered as shallow as 10 feet below existing grade at the time of our exploration. Groundwater was observed in our previous exploration as shallow as 18 feet deep, As a result, relatively shallow groundwater is present at lower elevations of the site at times during the year. Excavations to mitigate potential hazards or for planned cuts or utilities may encounter groundwater, depending upon the time of year of construction. Temporary construction dewatering may be necessary during grading.

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 lenses of less weathered bedrock. We anticipate that heavy duty excavators with should be capable of trenching the materials; however, in some instances significant difficulty may be encountered.

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

All grading and site development plans have been coordinated and should continue to be coordinated with the Engineering Geologist and the Geotechnical Engineer to modify the plans such that they mitigate known soil and geologic hazards. Detailed locations of keyways, subdrains, debris benches and subexcavation areas should be shown on the final grading plans upon their completion. Sequence of grading issues, such as placement of various cut materials in specific locations, should have also been evaluated during review of final 40-scale grading plans.

The Geotechnical Engineer or qualified representative should be present during all phases of grading operations to observe demolition, site preparation, grading operations, and subdrain placement. The Geotechnical Engineer should be notified a minimum of 72 hours prior to the commencement of any grading or stripping operations at the site. This is to provide time to coordinate the work with the Grading Contractor. After the grading operations commence, geologic observations of cut areas should be made at frequent intervals. This is advised so that revised geologic recommendations can be incorporated into updated grading plans as grading proceeds.

Ponding of storm water, other than within engineered detention basins, should not be permitted at the site, particularly during work stoppage for rainy weather. Before the grading is halted by rain, positive slopes should be provided to carry the surface runoff to storm drainage structures in a controlled manner to prevent erosion damage.

5.2 SELECTION OF MATERIALS

With the exception of some organically contaminated materials (soil which contains more than 3 percent organic content by weight), 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.

5.3 DEMOLITION STRIPPING AND REMOVAL OF WEAK AND COMPRESSIBLE SOILS

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.

No loose or uncontrolled backfilling of depressions resulting from demolition and stripping or other soil removal should be permitted. All exploratory geologic test pits excavated during site explorations are shown on Figure 2. It will be necessary to remove and recompact all loose soil within the test pits, where it will remain below final grades and is located within proposed improvement areas. Within the development areas, excavations resulting from demolition, clearing, and/or stripping which extend below final grades should be cleaned to firm undisturbed soil as determined by the Geotechnical Engineer's representative.

5.4 EXISTING FILLS

If existing fills are encountered during grading they should be treated as 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 FILL PLACEMENT

Overcompaction of expansive materials (PI over 12) may produce an undesirable environment for expansion in the zone of significant seasonal moisture variation; therefore, special requirements for compaction of expansive soils are necessary within the upper 5 feet in building areas. This recommendation is not to be interpreted as a requirement to remove and replace the top five feet within all lots, but is to be used when fill is placed within the top 5 feet of finished grade. The following compaction control requirements should be generally applied to engineered fills.

TABLE 5.5-1

Description	Materials	Minimum Relative Compaction (%)	Minimum Moisture Content (Percentage Points Above Optimum)
Within the upper 5 ft	Expansive	87 to 92	+5
	Non-expansive	90	+2
From 5 to 50 ft	Expansive	90	+4
	Non-expansive	95	+2

Maximum dry densities and moisture contents should be determined in accordance with ASTM D 1557, latest edition. Plasticity Index determinations, and possibly supplemental swell test data, should be made as a part of grading control. All fills should be placed in lifts not exceeding 12 inches or the depth of penetration of the compaction equipment used, whichever is less.

5.6 TOE KEYWAYS

After stripping, mass grading should begin with construction of keyways and subdrains. All fills should be adequately keyed into firm natural materials unaffected by shrinkage cracks. Keyways should be compacted in accordance with the specification presented above for fills greater than 50 feet deep.

Anticipated keyway sizes and locations should be determined based on the final grading plans by the Engineering Geologist. Typical minimum keyway sizes and subdrains are shown on Figure 6. The actual depth of the keyways will be determined in the field by the Geotechnical Engineer during grading. Filling above keyways should be benched into firm competent soil or bedrock and drained as appropriate. Unless otherwise recommended by the Geotechnical Engineer, benches should be constructed at vertical intervals of not less than 5 feet. 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.

5.7 SUBSURFACE DRAINAGE FACILITIES

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

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. 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.

5.8 DEBRIS BENCHES AND CATCHMENTS

Debris benches and catchments will be required along portions of the site where development cross drainages with steep upslope debris flow source areas as shown on Figure 2, and along steep cut and natural slopes.

5.8.1 Catchment Basins

Catchment basins will be required where development is proposed across the upper portions of existing natural drainage swale areas on the flanks of Gudde Ridge. These swale areas are filled with alluvial/colluvial fan deposits as identified on Figure 2. The alluvial/colluvial fans are interpreted to be soil deposits shed from the adjacent steep ridge flanks as debris flows. As described above debris flows can travel significant distances from the source area. Therefore, the upper limits of the development in swale areas will require provisions to intercept and retain debris shed from upslope areas. The size, location and configuration of required catchment basins should be designated as part of the future site planning process.

5.8.2 Slope Catchments

Debris benches with keyways will be required at the toes of cut or natural slopes. Exceptions to this requirement are areas which contain a debris catchment basin or drainage swale at least 100 feet wide between the slope and any development or at the toe of temporary slopes adjacent to roadways. A typical debris bench with keyway and subdrain is shown on Figure 6. The outboard side of the debris bench should be provided with a concrete V-ditch discharging into an approved outlet. The minimum debris bench width may be narrowed to 15 feet in areas where a roadway or open parking area is planned between the toe of cut slopes and the residential units. It should be noted, however, that if this stabilization scheme is employed, slope debris is likely to encroach at times onto paved areas. Based on the proposed development plan, it is anticipated that debris benches up to 30 feet wide will be required in several locations; behind lots 5, 12, 20 to 25, 33, 34, 43, 44, and behind and upslope of the grading limits for lots 58 to 63, upslope of Indian Ridge Court terminus, Indian Creek Court terminus, and Indian Ridge Lane. The size, location and configuration of required debris benches should be designated as part of the future site planning process.

5.8.3 Catchment Maintenance

All debris benches and catchment basins will require periodic maintenance consisting of the removal and disposal of accumulated slope detritus. Proper access should be provided for the heavy equipment which may be required for removal of slide debris from benches and paved areas.

All debris benches and buttress fills should be jointly designed by the Civil and Geotechnical Engineers to optimize stability, cut/fill balance, and drainage concerns. Recommendations for mass grading are generally applicable to landslide reconstruction and buttress fill installation.

5.9 GRADED SLOPES

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

TABLE 5.9-1
Slope Gradient Guidelines

Slope Height	Maximum Allowable Slope Inclination (horizontal:vertical)
Less than 8 feet	2:1
Greater than 8 feet	3:1

Where steeper slopes than those indicated above are desired, supplemental slope stabilization techniques (e.g. geogrid reinforcing) may be required. Erosion of graded slopes could be significant in areas where slopes are not properly vegetated or erosion control is not properly installed. Analysis and mitigation measures should be determined as necessary during the design level exploration. We provide the following preliminary recommendations regarding erosion control of graded slopes.

We recommend placing the topsoil strippings on graded slopes as an alternative to constructing slope drainage terraces. Site topsoil strippings should be placed over all open space cut and fill slopes immediately following grading and prior to the installation of erosion control measures. In our opinion, placing the site strippings on graded slopes reduces rainfall infiltration to natural levels, more actively promotes revegetation, enhances local slope stability, and provides a more natural slope appearance.

All cut slopes should be viewed by the Engineering Geologist during slope grading for adverse bedding, seepage, or bedrock conditions which may affect slope stability. In the event that adverse geologic conditions are detected during grading of the cut slopes, overexcavation and reconstruction of these slopes may be necessary. Track rolling to compact faces of slopes is not sufficient. Slopes should be overbuilt at least 2 feet and cut back to design grades.

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.

5.10 SLOPE STABILIZATION

Landslides or unstable hillsides, which pose a potential hazard to the proposed development, should be mitigated. Once final 40-scale grading plans are developed for the project, more detailed mitigation measures for the landslides should be developed by the geotechnical engineer or engineering geologist. Potential mitigation measures for landslides and slope instability at the project site include avoiding placement of structures in or downslope of slide areas, removing the landslide debris to bedrock and replacing it with engineered fill, buttressing the toes of landslides with engineered fill, and constructing keyways, debris benches, wide runnout or landslide catchment areas with surface and subsurface drainage. To reduce the potential for landslide danger, subsurface water flow and spring activity should be controlled in development areas through the use of subdrains.

It is important to note that to preserve the natural topography, wildlife habitat and vegetation of the site, stabilization of slide masses is planned only for slides that directly threaten the proposed improvements. Open-space slides that do not currently threaten the proposed improvements will not be repaired and may reactivate in the future.

5.11 SLOPE SETBACKS

Typical slope setbacks are variable depending on slope height and soil conditions. The recommended slope setbacks for habitable structures should be determined based on site specific slope stability analyses for static and seismic loading conditions. In general, where building pads are adjacent to uphill slopes, all permanent structures should be set back from the toe-of-slope a distance equal to one-half the vertical graded slope height or 15 feet, whichever is less. Where building pads are adjacent to downhill slopes, all permanent structures should generally be set back from the top-of-slope a distance equal to one-third the vertical graded slope height or 40 feet, whichever is less. We recommend that these be determined during design level exploration and analysis during future planning phases for the project.

5.12 CUT, FILL, AND CUT-FILL TRANSITION LOTS

Some single-family lots in this project will likely be entirely in cut or traversed by a cut/fill transition. It can be anticipated that significant variations in material properties may occur in areas of cut or cut/fill transition if not mitigated during site grading. It is our opinion that there is a potential for significant differential in swell characteristics across cut areas and cut/fill transitions. Such situations can be detrimental to building performance. Figure 8 represents the typical overexcavation recommended to mitigate the effects of differential materials located under a structure. We recommend that cut lots be overexcavated 2 feet, scarified 12 inches, and recompacted; cut/fill transition lots should be overexcavated 3 feet to provide a uniform thickness of engineered fill within the entire foundation area.

5.13 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 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.

5.14 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. The final grading plans should be submitted to the Geotechnical Engineer for review.

6.0 PRELIMINARY FOUNDATION RECOMMENDATIONS

A major consideration in foundation design for this project is the shrink-swell potential of site soils. The effects of expansion and shrinkage of soil could be minimized by the choice of a proper foundation system. In order to reduce the effects of the potentially expansive soils, the foundations should be sufficiently stiff to move as rigid units with minimum differential movements. Another consideration in foundation design is the liquefaction potential of site soils. As discussed above, based on the results of our preliminary analyses, we estimate less than 1 inch of seismic settlement resulting from liquefaction. The foundations should be designed to accommodate differential settlement as a result of liquefaction without collapse of the structure. In our opinion, post-tensioned mat foundations are appropriate to accommodate these foundation design considerations. Other foundation systems may be appropriate based on more specific site conditions such as sloping lots. Specific recommendations for these foundation systems should be developed following design level geotechnical exploration studies.

7.0 EXTERIOR SLABS-ON-GRADE

This section provides guidelines for secondary slabs such as porch slabs, exterior patio slabs, walkways, driveways, and steps. Secondary slabs-on-grade should be constructed structurally independent of the foundation system. This allows slab movement to occur with a minimum of foundation distress. Where slab-on-grade construction is anticipated, care must be exercised in attaining a near-saturation condition of the subgrade soil before concrete placement.

Slabs-on-grade should be designed specifically for their intended use and loading requirements. Some of the site soils have a high expansion potential; therefore, cracking of conventional slabs should be expected. As a minimum requirement, slabs-on-grade should be reinforced for control of cracking. Slab reinforcement should be designed by the Structural Engineer. In our experience, welded wire mesh is generally not sufficient to control slab cracking. Therefore, we recommend the Structural Engineer consider using a minimum of No. 3 bars for design of the slab reinforcement.

Slabs-on-grade should have a minimum thickness of 4 inches with a thickened edge extending at least 6 inches into compacted soil to minimize water infiltration. A 4-inch-thick layer of clean crushed rock or gravel should be placed under sidewalk and driveway slabs. As an alternative to providing a 6-inch-thick edge, a minimum 5½-inch-thick slab could be placed over 4 inches of clean crushed rock or gravel.

8.0 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 our guidance.

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.

9.0 CREEK BANK SETBACKS

As a planning guideline, we recommend setting back proposed improvements based on a projection of a 3:1 plane plus 10 feet from the flow line of the incised creek channel to the ground surface behind the top of bank. Alternatively, improvements proposed within the setback area could be protected from creek bank instability by construction of deep foundations or by the use of mechanically-stabilized earth (MSE) walls. Development of the final set-back line should be accomplished in consultation with the Civil Engineer.

10.0 EROSION CONTROL

In addition to vegetated cover, viable erosion mitigation measures may include concrete or asphalt lined drainage facilities on slopes graded steeper than 3:1 (horizontal:vertical). These measures are typically used on slopes with heights greater than 30 feet. The purpose of the drainage facilities is to intercept and divert the surface water runoff from the slopes and, combined with the 3:1 or flatter slopes, reduce runoff velocities, water infiltration, and sloughing or erosion of the slope surfaces.

Erosion of graded slopes can be mitigated by hydroseeding, landscaping, or placement of topsoil materials prior to the winter rains following rough grading. All landscaped slopes should be maintained in a vegetated state after project completion with drought tolerant vegetation requiring drip irrigation.

The tops of fill or cut slopes should be graded in such a way as to prevent water from flowing freely down the slopes. Due to the nature of the bedrock, slopes may experience severe erosion when grading is halted by heavy rain. Therefore, before work is stopped, a positive gradient away from the slopes should be provided to carry the surface runoff away from the slopes to areas where erosion can be controlled. It is vital that no completed slope be left standing through a winter season without erosion control measures having been provided.

11.0 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.

Our professional staff strive to perform our 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 our report. This document must not be subject to unauthorized reuse that is, reusing without our written authorization. Such authorization is essential because it requires us 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 our documents. Therefore, we must be engaged to prepare the necessary clarifications, adjustments, modifications or other changes before construction activities commence or further activity proceeds. If our scope of services does not include on-study area construction observation, or if other persons or entities are retained to provide such services, we 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.

SELECTED REFERENCES

American Society of Civil Engineers (2010). ASCE/SEI 7-10: Minimum Design Loads for Buildings and Other Structures.

California Geological Survey Special Publication 117A (2008). Guidelines for Evaluation and Mitigating Seismic Hazards in California.

Crane, R., 1988, Geologic Map of the Oakland East Quadrangle, Geology of the Mount Diablo Region, in Guidebook for 1988 Pacific Section NCGS Convention.

ENGEO (2003). Preliminary Geotechnical Exploration, Indian Valley Project, Moraga, California. July 31, 2003. Project No. 5900.1.001.01.

Google Earth (2012). Historic aerial photo imagery available for site spanning from the year 1939 to 2012.

Graymer, R.W., et al. (1999). Preliminary Geologic Map Emphasizing Bedrock Formations in Contra Costa County, California, USGS, Open File Report 94-82.

International Code Council (2012). California Building Code 2013.

Jennings, C. W. (2010). Fault Activity Map of California, California Division of Mines and Geology, Map No. 6.

Nilsen, T. H. (1975). Preliminary Photointerpretation Map of Landslide and Other Surficial Deposits, Oakland East Quadrangle, USGS Open-File Report MF-75-277-41.

Pacific Aerial Surveys (2002). Flight 141307, Line 33293, Frames I-1 through I-6, March 1, 2002.

P/A Design Associates (2015), Indian Valley Conceptual Development Plan, Town of Moraga, California, June 22, 2015.

Robertson, P.K. (2009). Performance based earthquake design using the CPT.

Witter C.W., Knudsen K.L., et al. (2006). Map of Quaternary Deposits and Liquefaction Susceptibility in the Central San Francisco Bay Region, USGS Open File Report 06-1037, 2006.

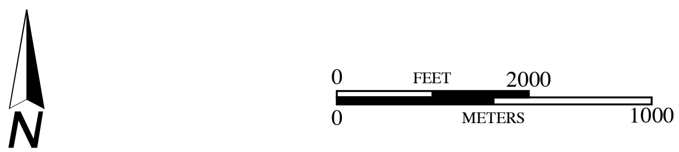
Youd et al. (2001). "Liquefaction resistance of soils." Summary Report from the 1996 NCEER and 1998 NCEER/NSF Wksps. on Evaluation of Liquefaction Resistance of Soils. ASCE J. of Geotech. and Geoenvironmental. Eng. 127 (10), 817-833.

SELECTED REFERENCES (Continued)

2007 Working Group on California Earthquake Probabilities (2008). The Uniform California Earthquake Rupture Forecast, Version 2 (UCERF 2): U.S. Geological Survey Open-File Report 2007-1437 and California Geological Survey Special Report 203 [http://pubs.usgs.gov/of/2007/1437/].

LIST OF FIGURES

FIGURES



BASE MAP SOURCE: GOOGLE EARTH PRO

ENGEO
Expect Excellence

VICINITY MAP
INDIAN VALLEY
MORAGA, CALIFORNIA

PROJECT NO.: 5900.200.000	FIGURE NO. 1
SCALE: AS SHOWN	
DRAWN BY: DLB	CHECKED BY: BR

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ORIGINAL FIGURE PRINTED IN COLOR

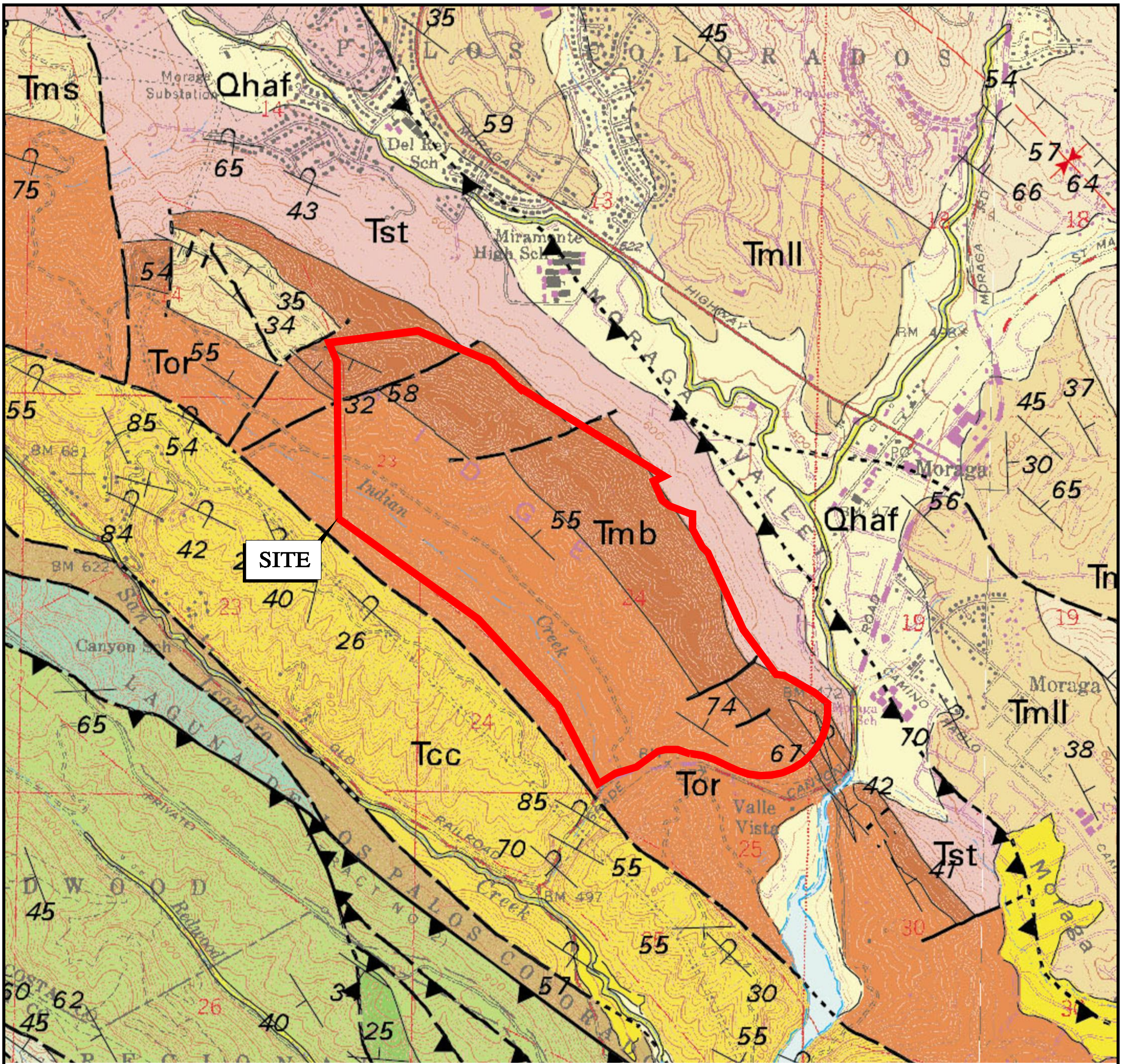
BASE PLAN SOURCE: PA DESIGN ASSOCIATES, 5/29/2015

ENGEO Expect Excellence

SITE PLAN
INDIAN VALLEY
MORAGA, CALIFORNIA

PROJECT NO.: 5900.200.000
SCALE: AS SHOWN
DRAWN BY: DLB
CHECKED BY: BR
FIGURE NO. 2

ORIGINAL FIGURE PRINTED IN COLOR



N

0 FEET 2000
0 METERS 1000

EXPLANATION

Qhaf	ALLUVIAL FAN DEPOSITS (HOLOCENE)
Tmll	LOWER MEMBER
Tst	SIESTA FORMATION
Tmb	MORAGA FORMATION
Tcc	CLAREMONT CHERT
Tms	INTERFLOW SEDIMENTARY ROCKS

BASE MAP SOURCE: GRAYMER, 2000

ENGEO
Expect Excellence

REGIONAL GEOLOGIC MAP
INDIAN VALLEY
MORAGA, CALIFORNIA

PROJECT NO.: 5900.200.000
SCALE: AS SHOWN
DRAWN BY: DLB
CHECKED BY: BR
FIGURE NO. 3

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ORIGINAL FIGURE PRINTED IN COLOR

EXPLANATION

LANDSLIDE DEPOSIT. ARROWS INDICATE GENERAL DIRECTION OF DOWNSLOPE MOVEMENT. QUERIED WHERE UNCERTAIN

Qal ALLUVIAL DEPOSIT

Qt ALLUVIAL TERRACE DEPOSIT. QUERIED WHERE UNCERTAIN

COLLUVIAL DEPOSIT AND/OR SMALL ALLUVIAL FAN DEPOSIT

Qaf ARTIFICIAL FILL

BEDROCK. QUERIED WHERE IDENTIFICATION UNCERTAIN

QUARRY OR GRAVEL PIT

N

0 FEET 2000

0 METERS 1000

BASE MAP SOURCE: NILSEN, 1975

ENGEO
Expect Excellence

REGIONAL LANDSLIDE MAP
INDIAN VALLEY
MORAGA, CALIFORNIA

PROJECT NO.:	5900.200.000
SCALE:	AS SHOWN
DRAWN BY: DLB	CHECKED BY: BR

FIGURE NO 4

G:\Drafting\DRAFTING2_Dwg\5900\200\PGEX\5900200000-PGEX-4-RegLandslideMap-0615.dwg Plot Date: 6-02-15 dborde

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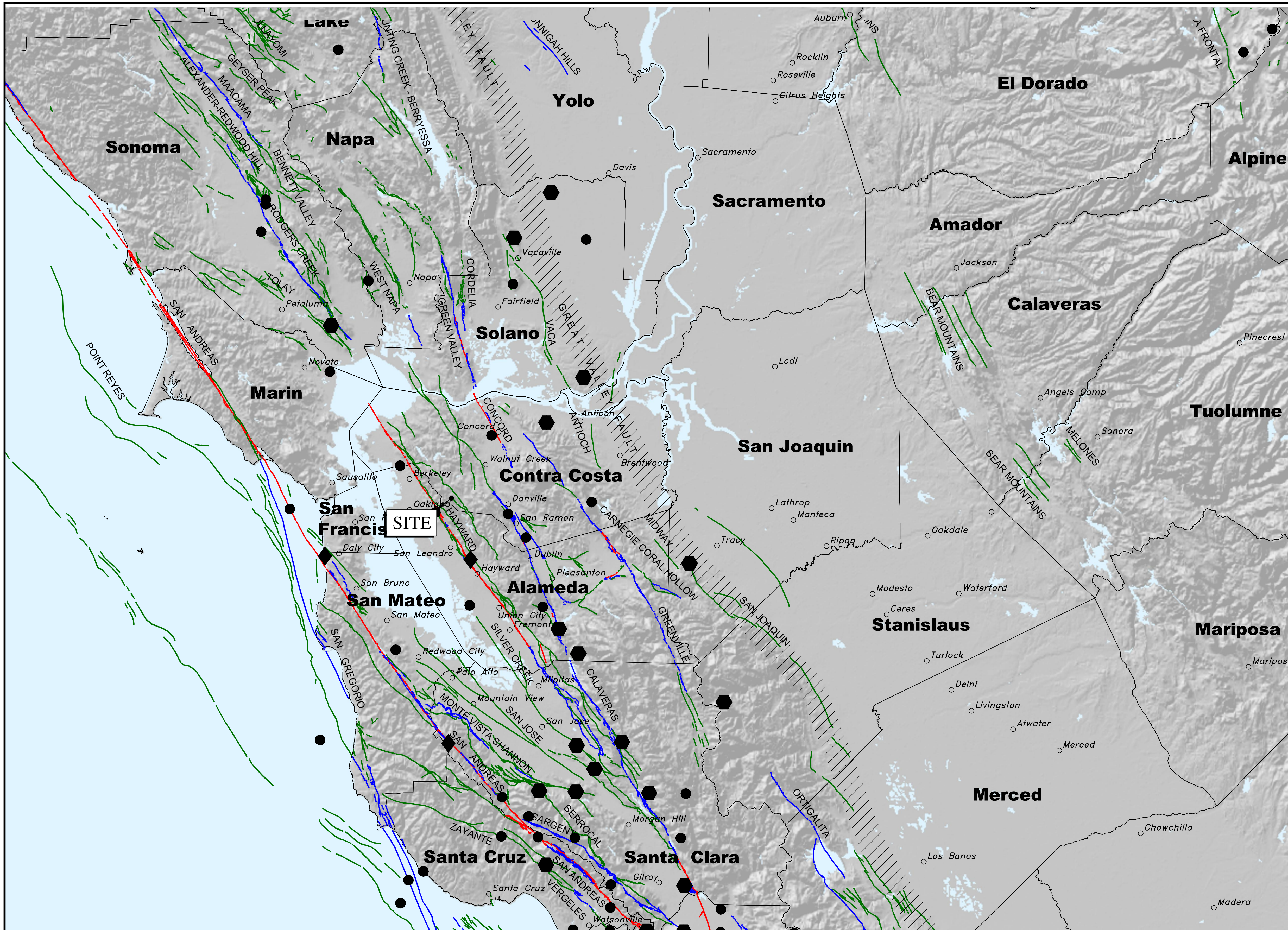

EXPLANATION

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

BASE MAP SOURCE:
HILLSHADE IMAGE BASED ON 1-ARC SECOND S.R.T.M. DATABASE
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U.S.G.S. HISTORIC EARTHQUAKE DATABASE (1800–2000)

ENGEO Expect Excellence

REGIONAL FAULTING AND SEISMICITY
INDIAN VALLEY
MORAGA, CALIFORNIA

PROJECT NO.:	5900.200.000
SCALE:	AS SHOWN
DRAWN BY: DLB	CHECKED BY: BR

FIGURE NO. 5

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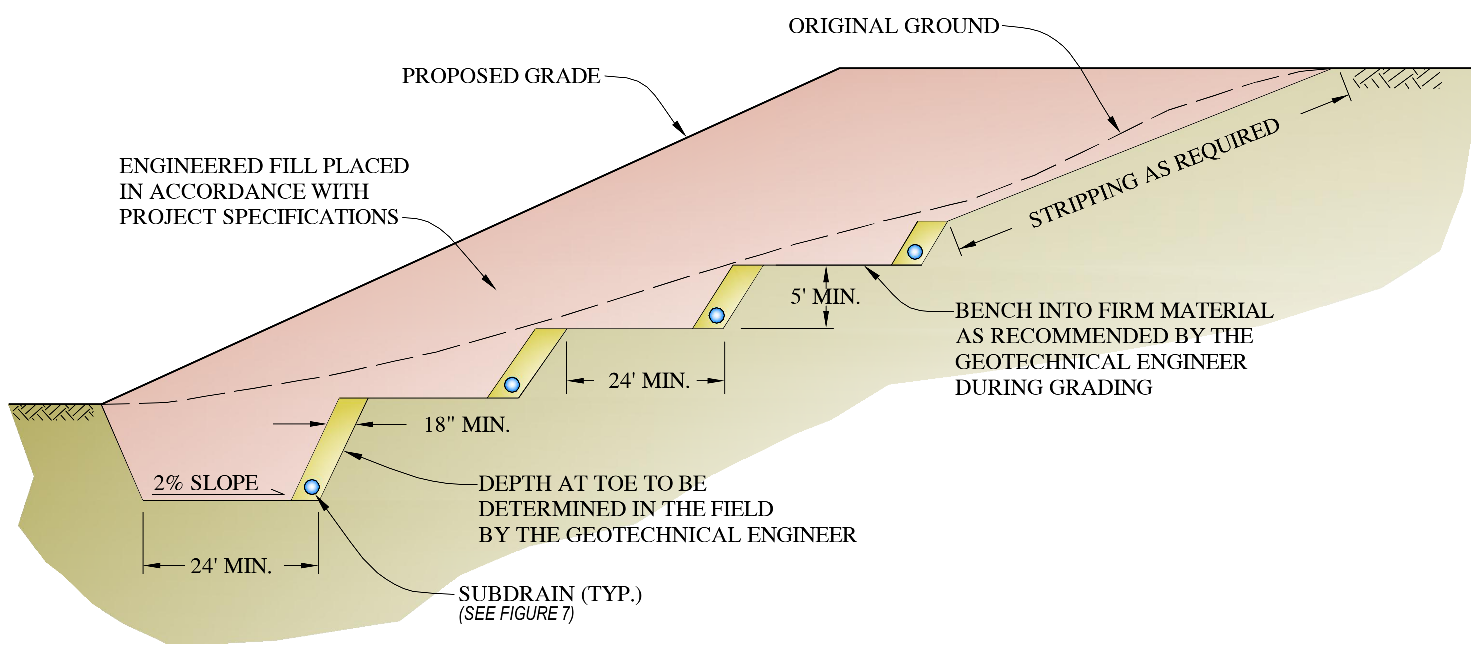
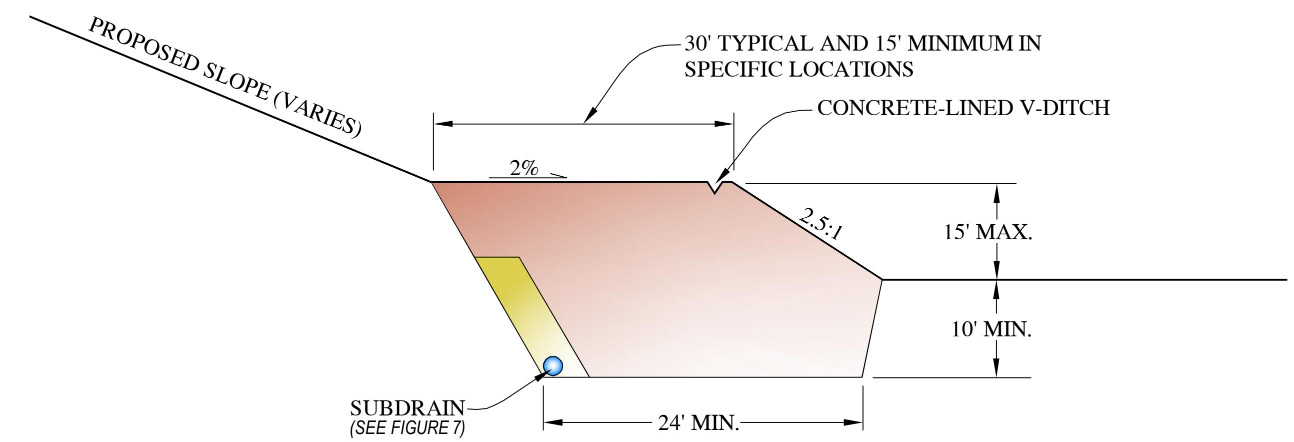
ORIGINAL FIGURE PRINTED IN COLOR

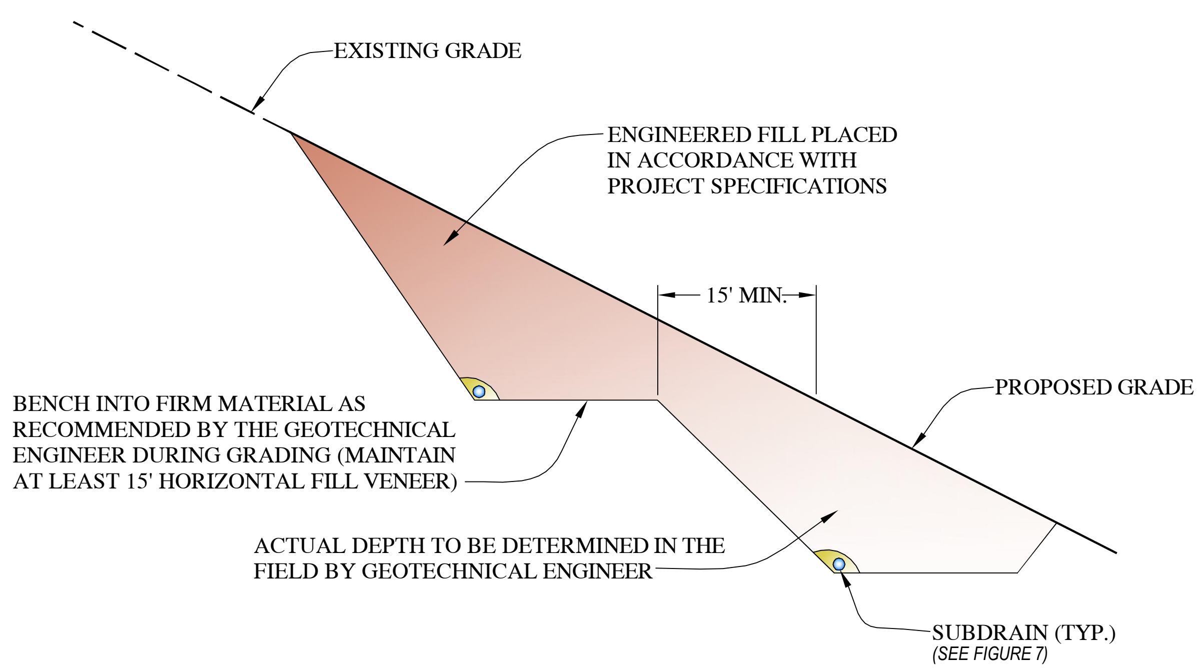


TYPICAL FILL SLOPE KEYWAY DETAIL



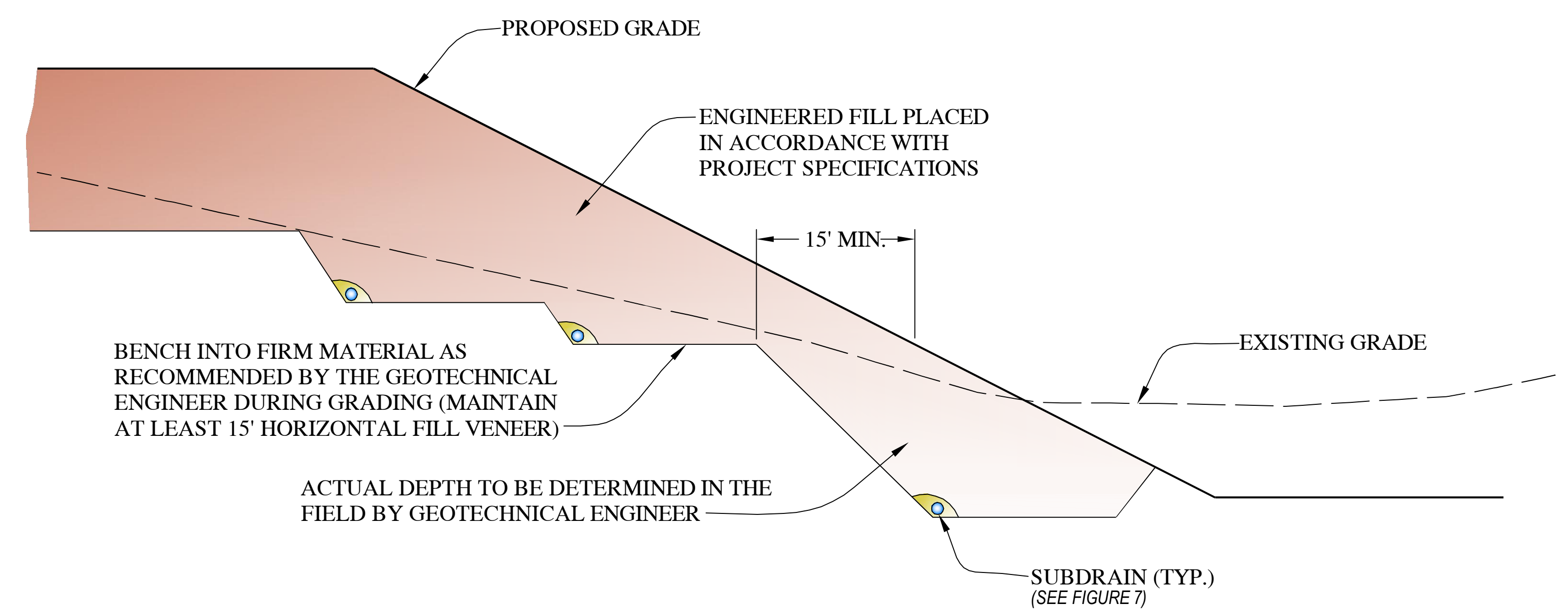
ACTUAL SIZE AND DEPTH OF KEYWAY TO BE DETERMINED IN THE FIELD BY THE GEOTECHNICAL ENGINEER

TYPICAL DEBRIS BENCH DETAIL



ALL SLOPES WILL BE CAPPED WITH APPROXIMATELY 6 INCHES OF ORGANICALLY-RICH SOIL, PLACED ON A ROUGHENED, MOISTURE CONDITIONED SURFACE

CUT SLOPE REBUILD



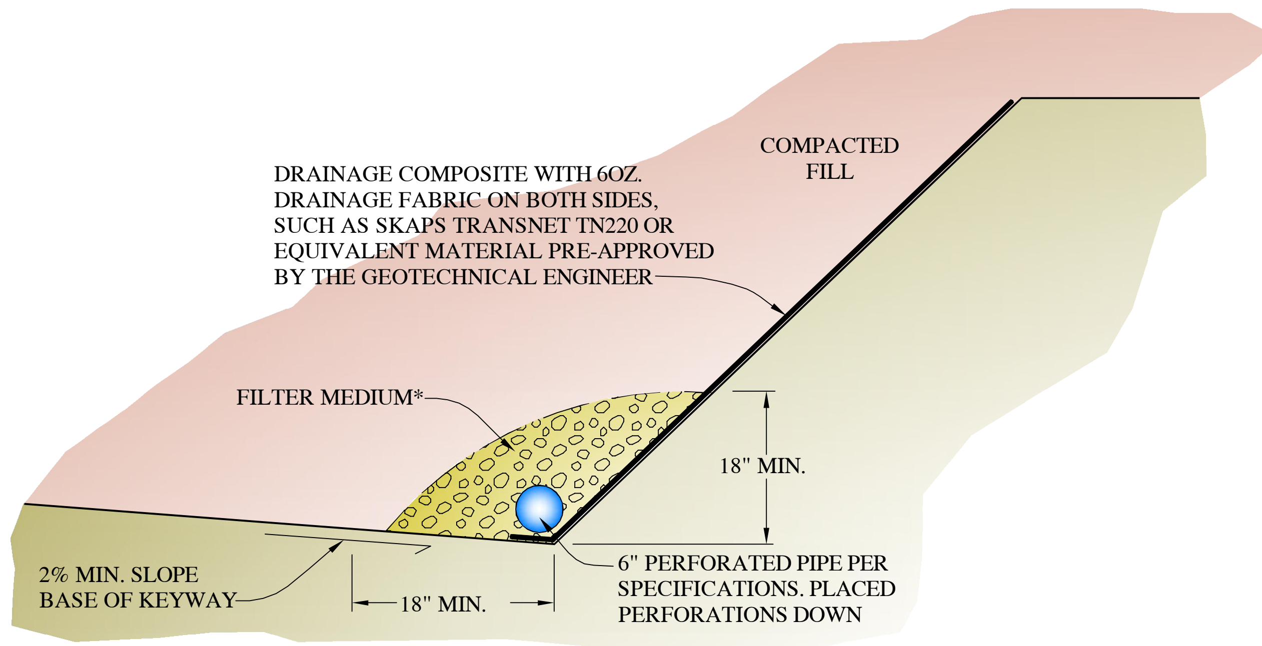
ALL SLOPES WILL BE CAPPED WITH APPROXIMATELY 6 INCHES OF ORGANICALLY-RICH SOIL, PLACED ON A ROUGHENED, MOISTURE CONDITIONED SURFACE

CUT-FILL TRANSITION SLOPE REBUILD

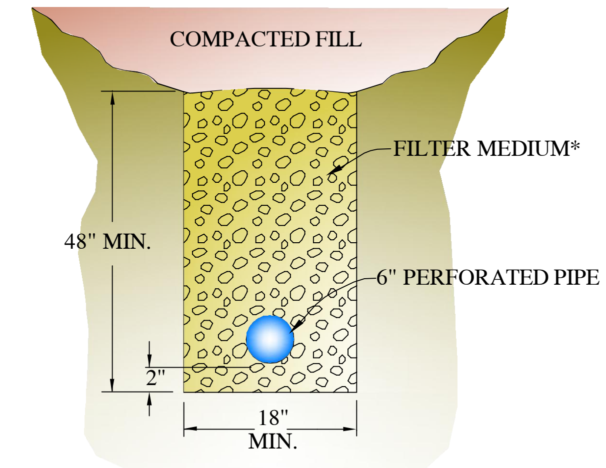
ENGEO
Expect Excellence

TYPICAL SLOPE REMEDIATION DETAILS
INDIAN VALLEY
MORAGA, CALIFORNIA

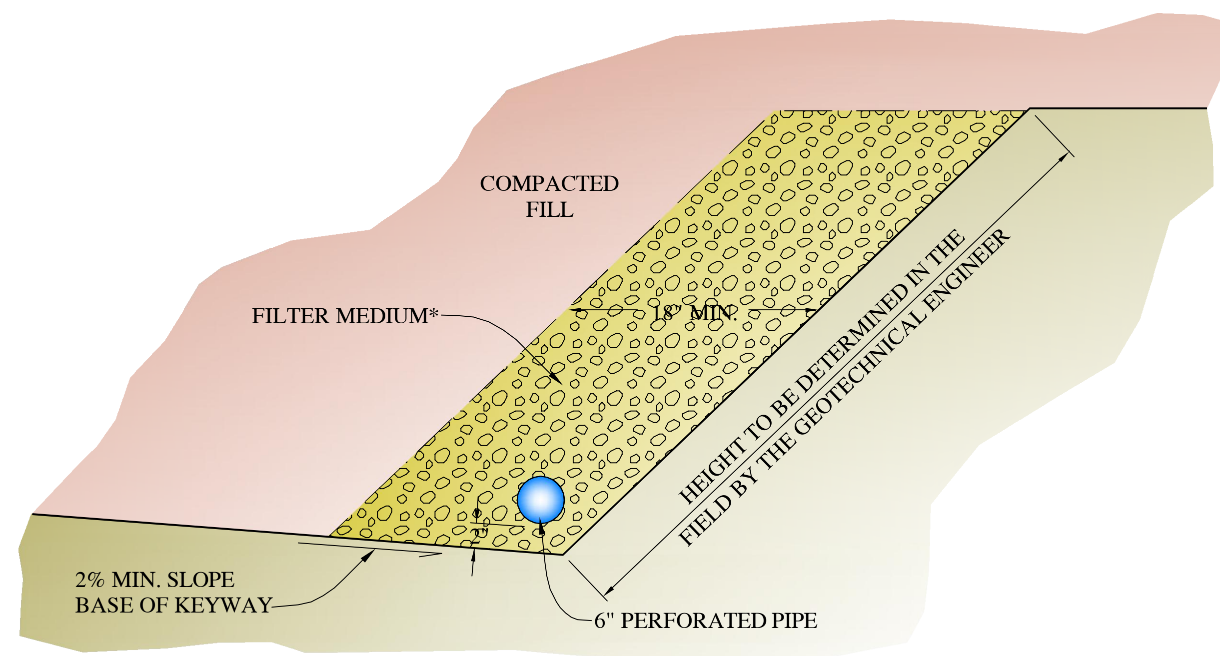
PROJECT NO.: 5900.200.000
SCALE: NO SCALE
DRAWN BY: DLB
CHECKED BY: BR
FIGURE NO. 6



KEYWAY SUBDRAIN - OPTION 1



SWALE SUBDRAIN



KEYWAY SUBDRAIN - OPTION 2

*FILTER MEDIUM

ALTERNATIVE A

CLASS 2 PERMEABLE MATERIAL

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

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

ALTERNATIVE B

CLEAN CRUSHED ROCK OR GRAVEL WRAPPED IN FILTER FABRIC

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

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

NOTES:

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

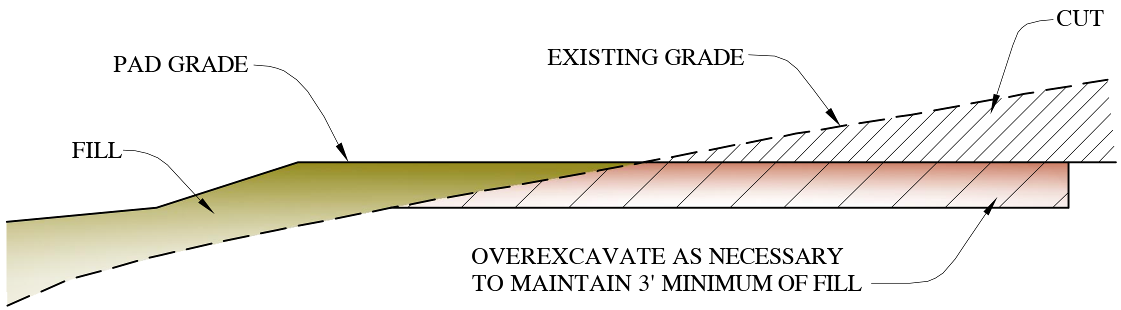
ENGEO
Expect Excellence

TYPICAL SUBDRAIN DETAILS
INDIAN VALLEY
MORAGA, CALIFORNIA

PROJECT NO.: 5900.200.000
SCALE: NO SCALE
DRAWN BY: DLB
CHECKED BY: BR
FIGURE NO. 7

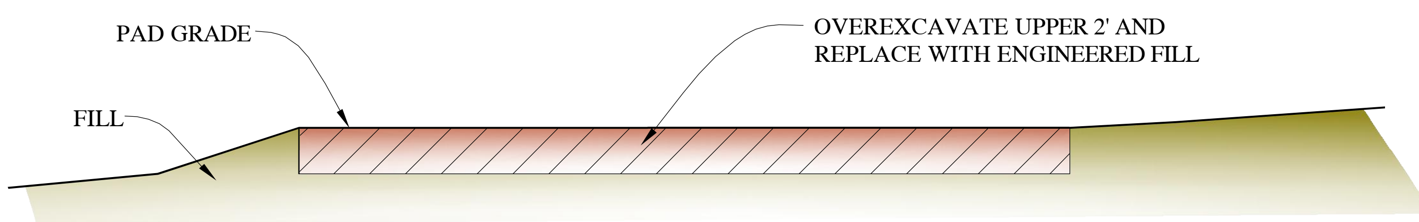
G:\Drafting\DRAFTING2_Dwg\5900\200\PGEX\5900200000-PGEX-7-SubdrainDetail-0615.dwg Plot Date: 6-02-15 dborde

G:\Drafting\DRAFTING2_Dwg\5900\200\PGEX\5900200000-PGEX-8-SurficialPadDetail-0615.dwg Plot Date:6-02-15 dborde



WHERE LOTS ARE PARTIALLY IN FILL, AND PARTIALLY IN CUT, THE CUT PORTION MUST BE OVEREXCAVATED AS SHOWN

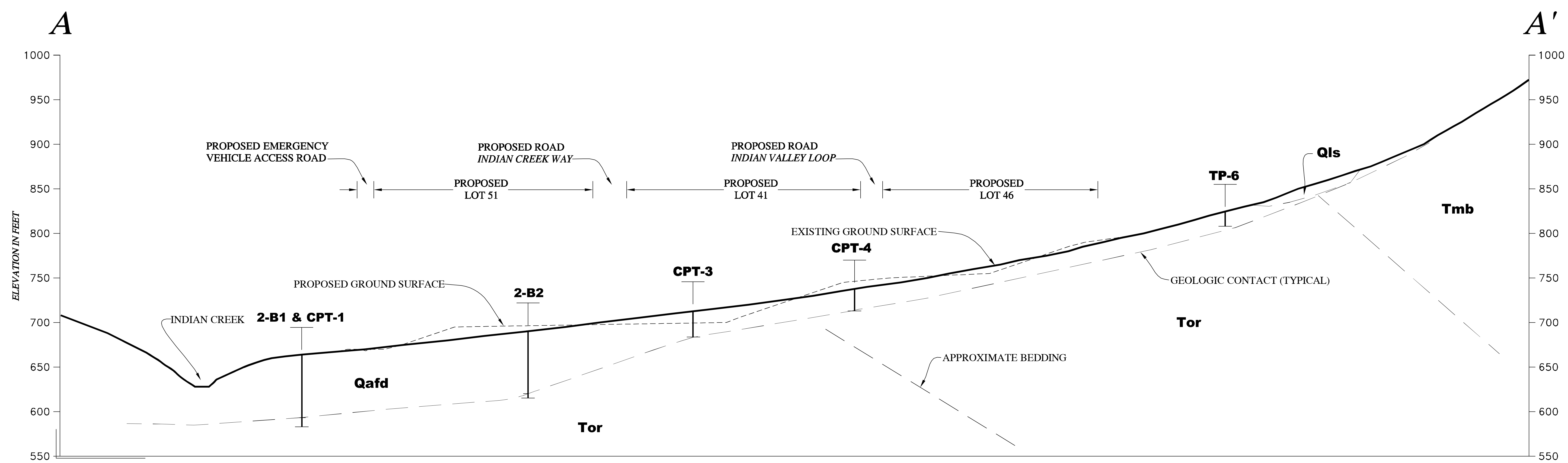
TYPICAL CUT- FILL TRANSITION PAD DETAIL

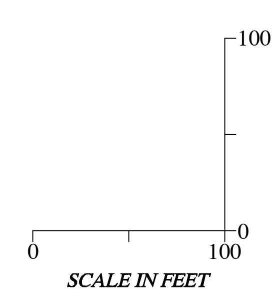


TYPICAL CUT PAD OVEREXCAVATION DETAIL

ENGEO Expect Excellence	SURFICIAL PAD TREATMENT DETAILS INDIAN VALLEY MORAGA, CALIFORNIA	PROJECT NO.: 5900.200.000 SCALE: NO SCALE DRAWN BY: DLB / CHECKED BY: BR	FIGURE NO. 8

ORIGINAL FIGURE PRINTED IN COLOR





EXPLANATION

ALL LOCATIONS ARE APPROXIMATE

Qafd	COLLUVIAL/ALLUVIAL FAN DEPOSITS
Tor	ORINDA FORMATION
Tmb	MORAGA BASALT
2B-2	BORING (ENGEO, 2013)
CPT-4	CONE PENETRATION TEST (ENGEO, 2013)
TP-6	TEST PIT (ENGEO, 2003)

ENGEO Expect Excellence	GEOLOGIC CROSS SECTION A-A' INDIAN VALLEY MORAGA, CALIFORNIA	PROJECT NO.: 5900.200.000 SCALE: AS SHOWN DRAWN BY: DLB CHECKED BY: BR	FIGURE NO. 9

APPENDIX A-

Exploratory Boring Logs and Cone Penetration Test Logs
(2013)

APPENDIX A

LOG OF BORING 2-B1

ENGEO INCORPORATED

Geotechnical Exploration
Indian Valley
Moraga, California
5900.200.000

DATE DRILLED: 8/12/2013
HOLE DEPTH: Approx. 81 ft.
HOLE DIAMETER: 4.0 in.
SURF ELEV (msl): Approx. 665 ft.

LOGGED / REVIEWED BY: J. White / JBR
DRILLING CONTRACTOR: Britton Exploration
DRILLING METHOD: Hollow Stem Auger
HAMMER TYPE: 140 lb. Auto Trip

DESCRIPTION	Blow Count/Foot	Atterberg Limits: Liquid Limit	Plastic Limit	Plasticity Index	Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Unconfined Strength (tsf) *field approx
FAT CLAY (CH), very dark brown, very stiff, moist, fine- to coarse-grained sand.	24	57	18	39		19.5	99.8	3.0*
	26							>4.5*
LEAN CLAY (CL), dark reddish brown, hard, moist, 5-10% sand and gravel.								
Very stiff, 5-10% sand.	14					19.6	106.8	1.93
Stiff, <5% sand.	18	32	17	15		22.2	107.6	2.0*
5-10% sand.	14							2.0*
LEAN CLAY (CL), dark reddish brown, stiff, moist, 5-10% sand.	15	36	19	17		27.8	95.9	1.33
LEAN CLAY with Sand (CL), very dark brown, very stiff, moist, fine-grained sand and gravel.	16							2.25*
Dark brown, <5% coarse gravel.	20					23.5	107.6	2.0*
CLAYEY SAND with Gravel (SC), very dark gray, medium dense, wet, fine- to coarse-grained sand and fine to coarse gravel.	15							
	10				32			
LEAN CLAY with Sand (CL), dark brown, stiff, moist, fine- to coarse-grained sand, <5% fine gravel.	13	35	17	18				1.75*

Depth in Feet: 10, 20, 30, 40, 50
Depth in Meters: 1, 2, 3, 4, 5, 6, 7, 8, 9, 10, 11, 12, 13, 14, 15

LOG - GEOTECHNICAL GINT LOGS.GPJ ENGEO INC.GDT 10/1/13

LOG OF BORING 2-B1

ENGEO INCORPORATED

Geotechnical Exploration
Indian Valley
Moraga, California
5900.200.000

DATE DRILLED: 8/12/2013
HOLE DEPTH: Approx. 81 ft.
HOLE DIAMETER: 4.0 in.
SURF ELEV (msl): Approx. 665 ft.

LOGGED / REVIEWED BY: J. White / JBR
DRILLING CONTRACTOR: Britton Exploration
DRILLING METHOD: Hollow Stem Auger
HAMMER TYPE: 140 lb. Auto Trip

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
	16		LEAN CLAY with Sand (CL), dark brown, stiff, moist, fine- to coarse-grained sand, <5% fine gravel.			26	32	15	17		19	112.4	1.75*
			SANDY LEAN CLAY with Gravel (CL), dark brown, stiff, moist, fine gravel.										
	17		SILTY SAND (SM), medium dense, moist.			24							1.75*
	18		SANDY LEAN CLAY with Gravel (CL), dark brown, stiff, moist, fine gravel.										
60	19		CLAYEY SAND (SC), dark grayish brown, medium dense, wet, fine- to coarse-grained sand, < 5% fine gravel.			24				44			
	20		SANDY LEAN CLAY (CL), dark grayish brown, stiff, wet, fine- to coarse-grained sand, <5% fine gravel.			16							
70	21		Very stiff, fine- to medium-grained sand.			14	31	14	17				3.5*
	22, 23		SILTSTONE, dark grayish brown and dark gray, weak, closely fractured, freshly weathered.			29							
80	24		Same as above.			50/4"							
			Bottom of boring at 81 feet, groundwater encountered at 40 feet.										

ENGEO INCORPORATED

LOG OF BORING 2-B2

Geotechnical Exploration
Indian Valley
Moraga, California
5900.200.000

DATE DRILLED: 8/13/2013
HOLE DEPTH: Approx. 74½ ft.
HOLE DIAMETER: 4.0 in.
SURF ELEV (msl): Approx. 690 ft.

LOGGED / REVIEWED BY: J. White / JBR
DRILLING CONTRACTOR: Britton Exploration
DRILLING METHOD: Hollow Stem Auger
HAMMER TYPE: 140 lb. Auto Trip

Depth in Feet: 10, 20, 30, 40, 50
Depth in Meters: 1, 2, 3, 4, 5, 6, 7, 8, 9, 10, 11, 12, 13, 14, 15

DESCRIPTION	Blow Count/Foot	Atterberg Limits: Liquid Limit	Plastic Limit	Plasticity Index	Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Unconfined Strength (tsf) *field approx
LEAN CLAY with Sand (CL), dark brown, hard, moist, 5% fine gravel, fine- to coarse-grained sand.	18							
Dark reddish brown, very stiff.	17	47	17	30		22.9	97.8	3.5*
SANDY LEAN CLAY (CL), dark reddish brown, medium stiff, wet, <5% fine gravel.	6							0.75*
Fine- to coarse-grained sand.	13					20.3	109.9	2.0*
LEAN CLAY with Sand (CL), dark reddish brown, very stiff, moist, <5% gravel.	21							2.5*
Dark brown, very stiff.	16	36	15	21		20.8	108.2	1.59
Same as above.	19							2.25*
SANDY LEAN CLAY (CL), reddish brown, very stiff, <5% fine gravel.	23					19.2	111.2	2.5*
With gravel, very stiff.	25					18.3	109.5	1.73
Fine gravel.	25							3.0*
LEAN CLAY with sand (CL), dark brown, very stiff, wet, <5% gravel, fine- to coarse-grained sand.	18							

ENGEO INCORPORATED

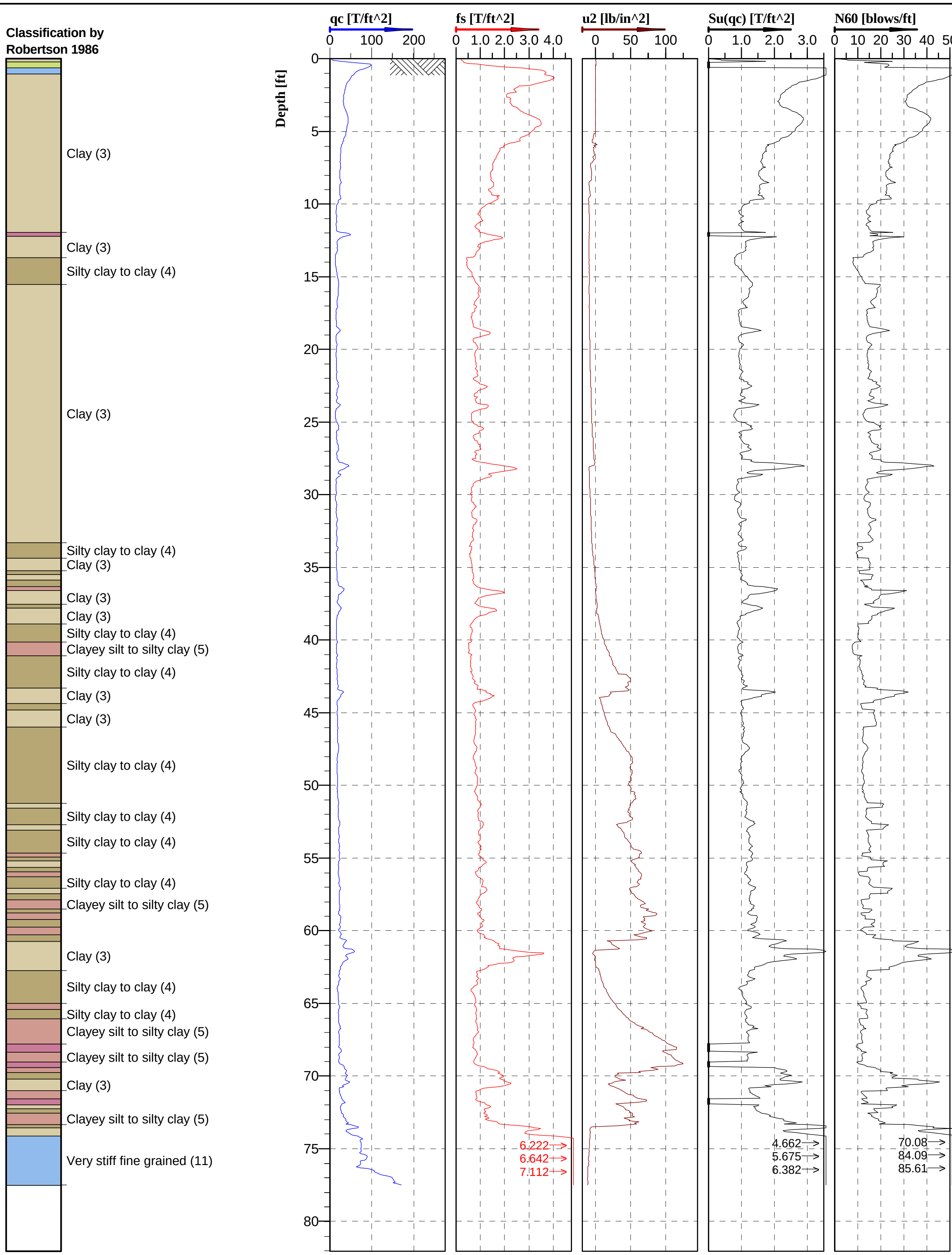
LOG OF BORING 2-B2

Geotechnical Exploration
Indian Valley
Moraga, California
5900.200.000

DATE DRILLED: 8/13/2013
HOLE DEPTH: Approx. 74½ ft.
HOLE DIAMETER: 4.0 in.
SURF ELEV (msl): Approx. 690 ft.

LOGGED / REVIEWED BY: J. White / JBR
DRILLING CONTRACTOR: Britton Exploration
DRILLING METHOD: Hollow Stem Auger
HAMMER TYPE: 140 lb. Auto Trip

Depth in Feet	Depth in Meters	DESCRIPTION	Blow Count/Foot	Atterberg Limits: Liquid Limit	Atterberg Limits: Plastic Limit	Atterberg Limits: Plasticity Index	Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Unconfined Strength (tsf) *field approx
	16	LEAN CLAY with Sand (CL), dark brown, hard, moist, 5% fine gravel, fine- to coarse-grained sand.		49	18	31		26.6	100	2.07
	17	Same as above.	17							2.5*
60	18	FAT CLAY (CH), dark brown, stiff, moist, <5% sand and gravel.	11	54	21	33		34.5	88.1	0.68
	19, 20	LEAN CLAY with Sand and Gravel (CL), dark brown, stiff, moist, fine- to coarse-gravel.	18							1.75*
70	21		28					20.4	109.5	2.5*
	22	SANDY LEAN CLAY with Gravel (CL), dark brown, very stiff, moist, fine gravel. Harder drilling at 71 feet.								
		SANDY SILTSTONE, dark brown, weak, very closely fractured, freshly weathered.	50/3"							
		Bottom of boring at 74.5 feet, perched groundwater encountered at 10 feet, groundwater encountered at 40 feet.								

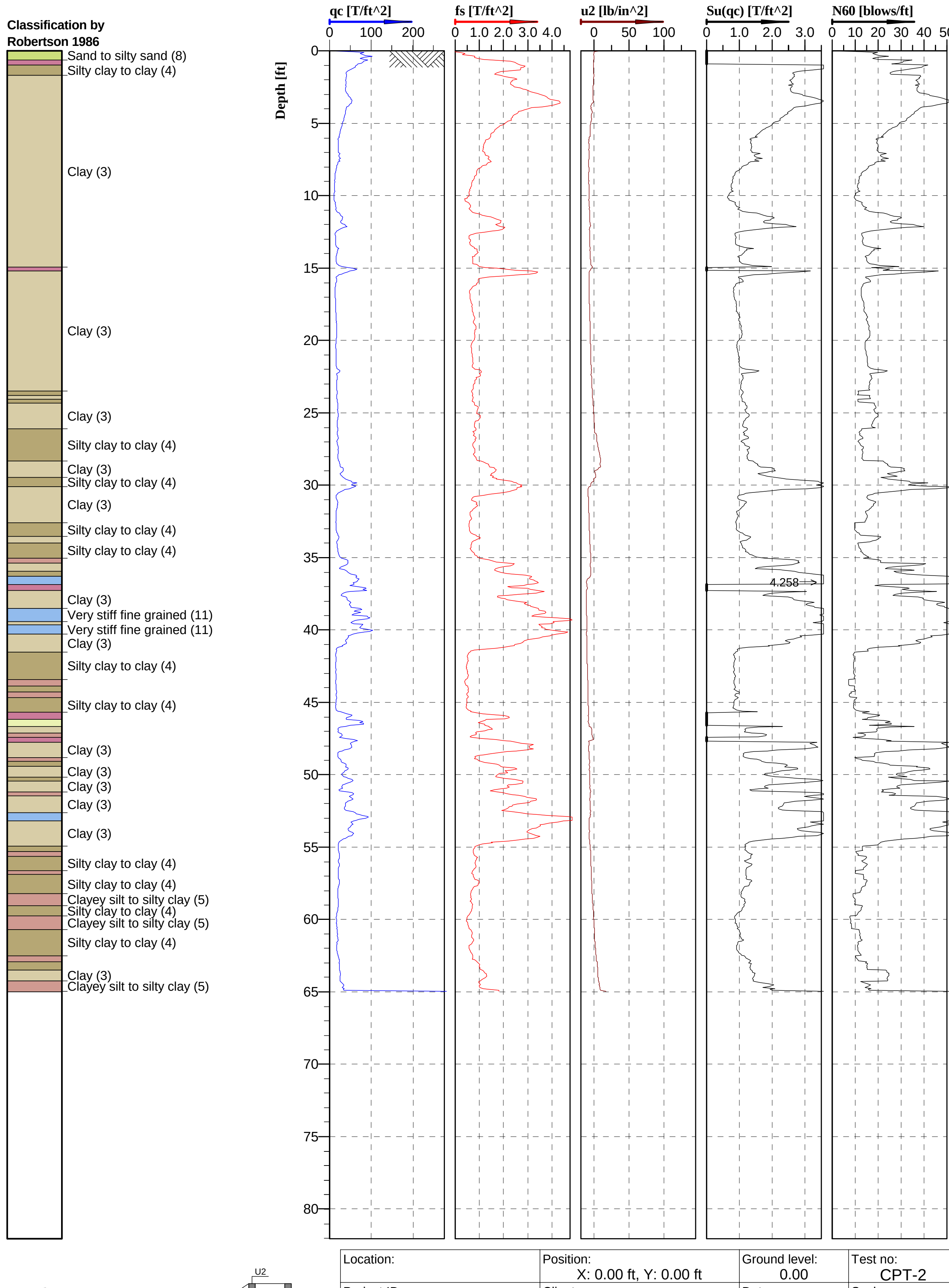


CALIFORNIA PUSH TECHNOLOGIES INCORPORATED

U2

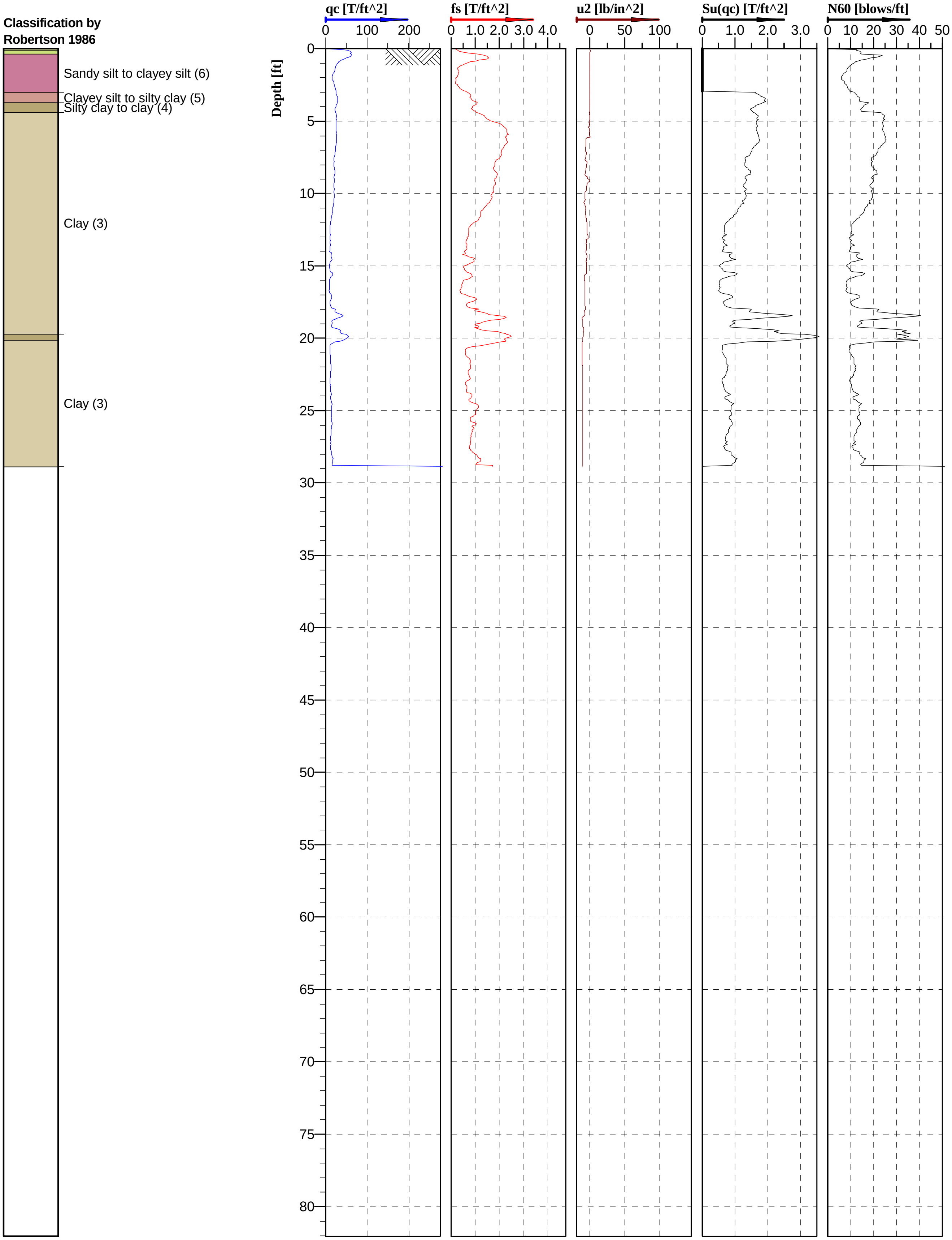
Cone No: 4141
Tip area [cm2]: 10
Sleeve area [cm2]: 150

Location: Moraga, California	Position:	Ground level:	Test no: CPT-1
Project ID: 5900.200.000	Client: Engeo	Date: 8/14/2013	Scale: 1 : 115
Project: Indian Valley		Page: 1/1	Fig:
		File: CPT-1.cpd	



Cone No: 4141
Tip area [cm2]: 10
Sleeve area [cm2]: 150

Location:	Position: X: 0.00 ft, Y: 0.00 ft	Ground level: 0.00	Test no: CPT-2
Project ID:	Client:	Date: 8/14/2013	Scale: 1 : 115
Project:		Page: 1/1	Fig:
		File: CPT-2.cpd	

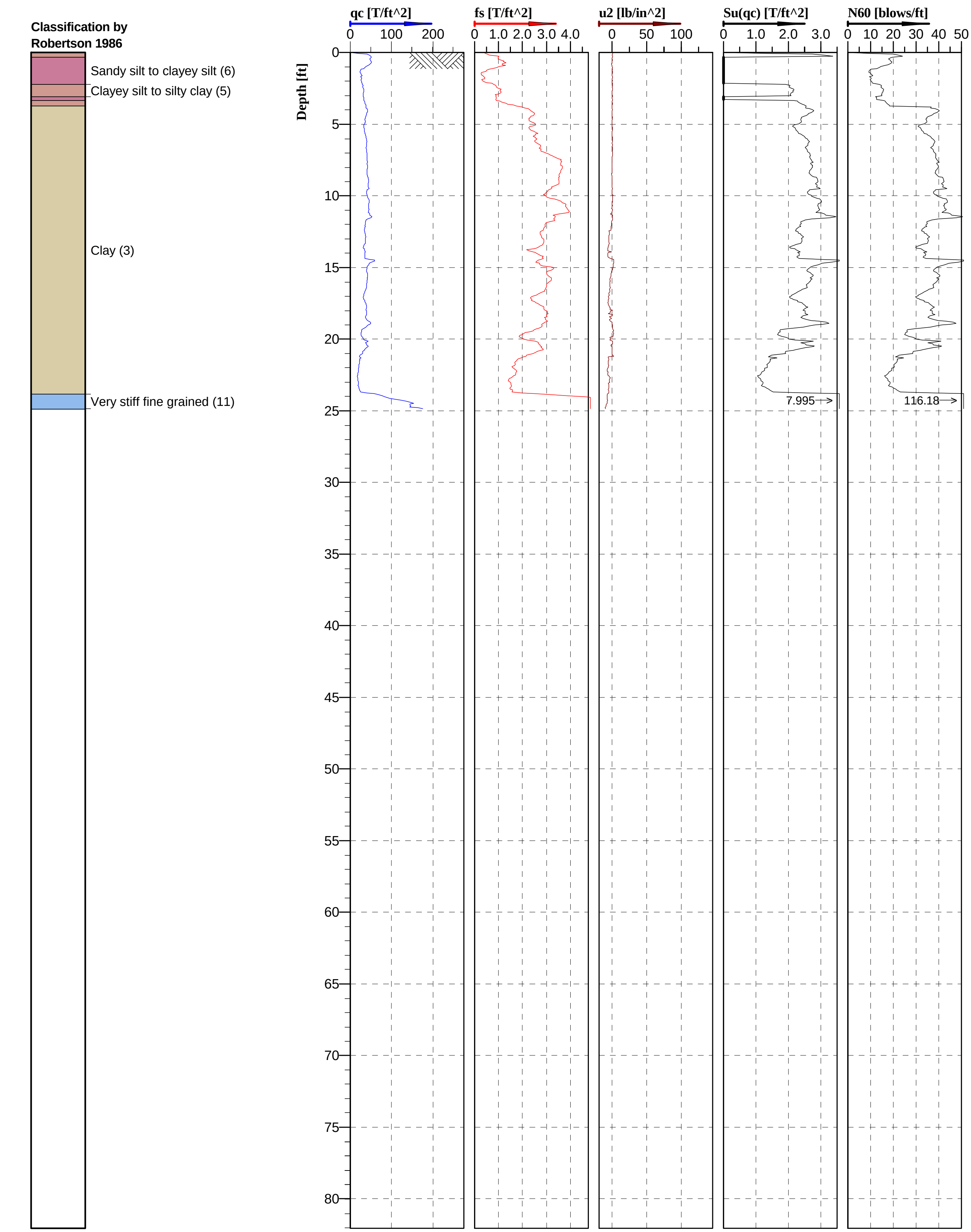


CALIFORNIA PUSH TECHNOLOGIES INCORPORATED

U2

Cone No: 4141
Tip area [cm2]: 10
Sleeve area [cm2]: 150

Location: Moraga, California	Position:	Ground level:	Test no: CPT-3
Project ID: 5900.200.000	Client: Engeo	Date: 8/14/2013	Scale: 1 : 115
Project: Indian Valley		Page: 1/1	Fig:
		File: CPT-3.cpd	



U2

Cone No: 4141
Tip area [cm2]: 10
Sleeve area [cm2]: 150

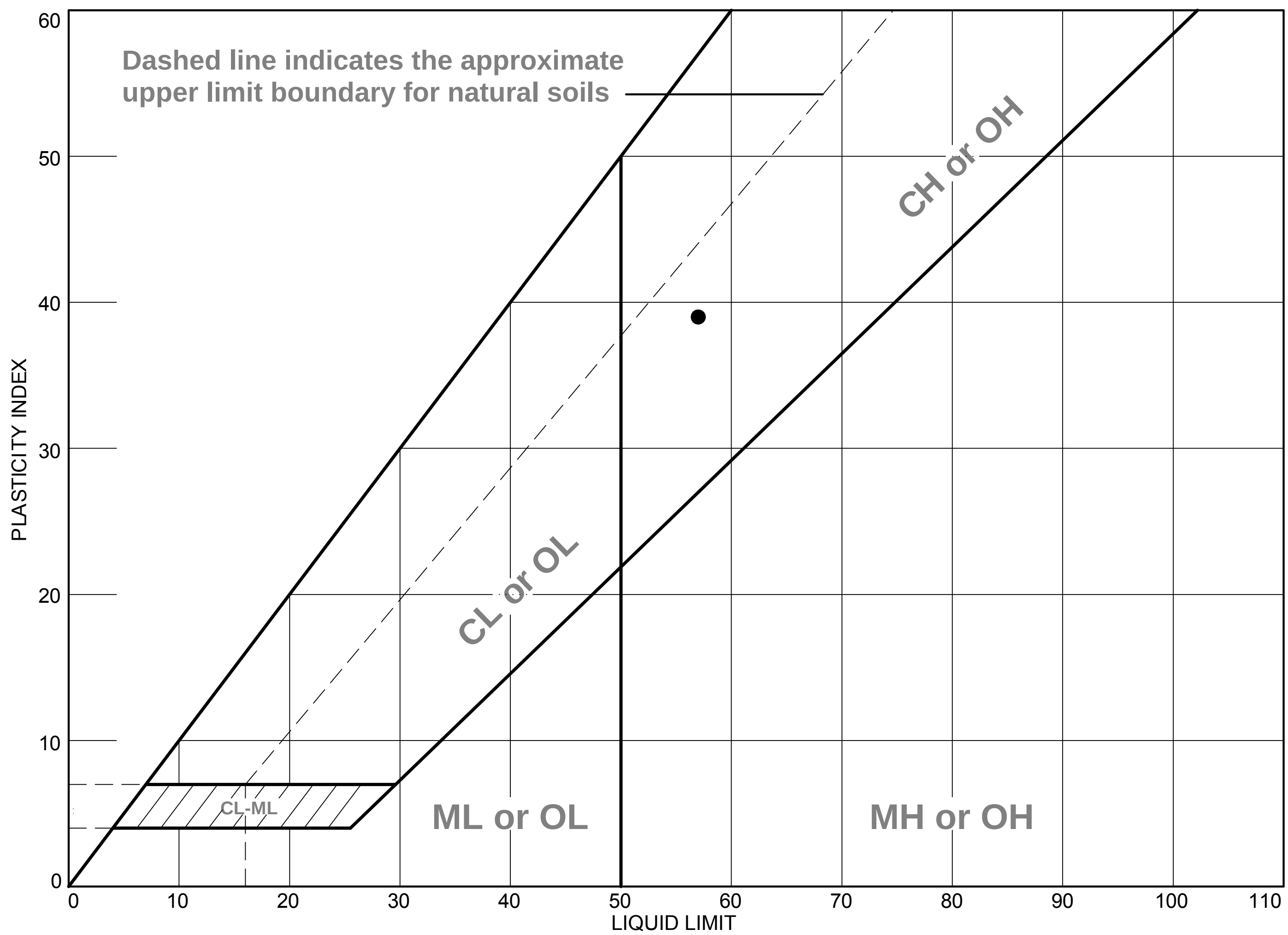
Location: Moraga, California	Position:	Ground level:	Test no: CPT-4
Project ID: 5900.200.000	Client: Engeo	Date: 8/14/2013	Scale: 1 : 115
Project: Indian Valley		Page: 1/1	Fig:
		File: CPT-4.cpd	

APPENDIX B

Laboratory Analysis (2013)

APPENDIX B

LIQUID AND PLASTIC LIMITS TEST REPORT



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	See exploration logs	57	18	39			

Project No. 5900.200.000 **Client:** The Bruzzone Family

Project: Indian Valley - Preliminary GEX update

● **Depth:** 3.0 feet **Sample Number:** 2-B1 @ 3

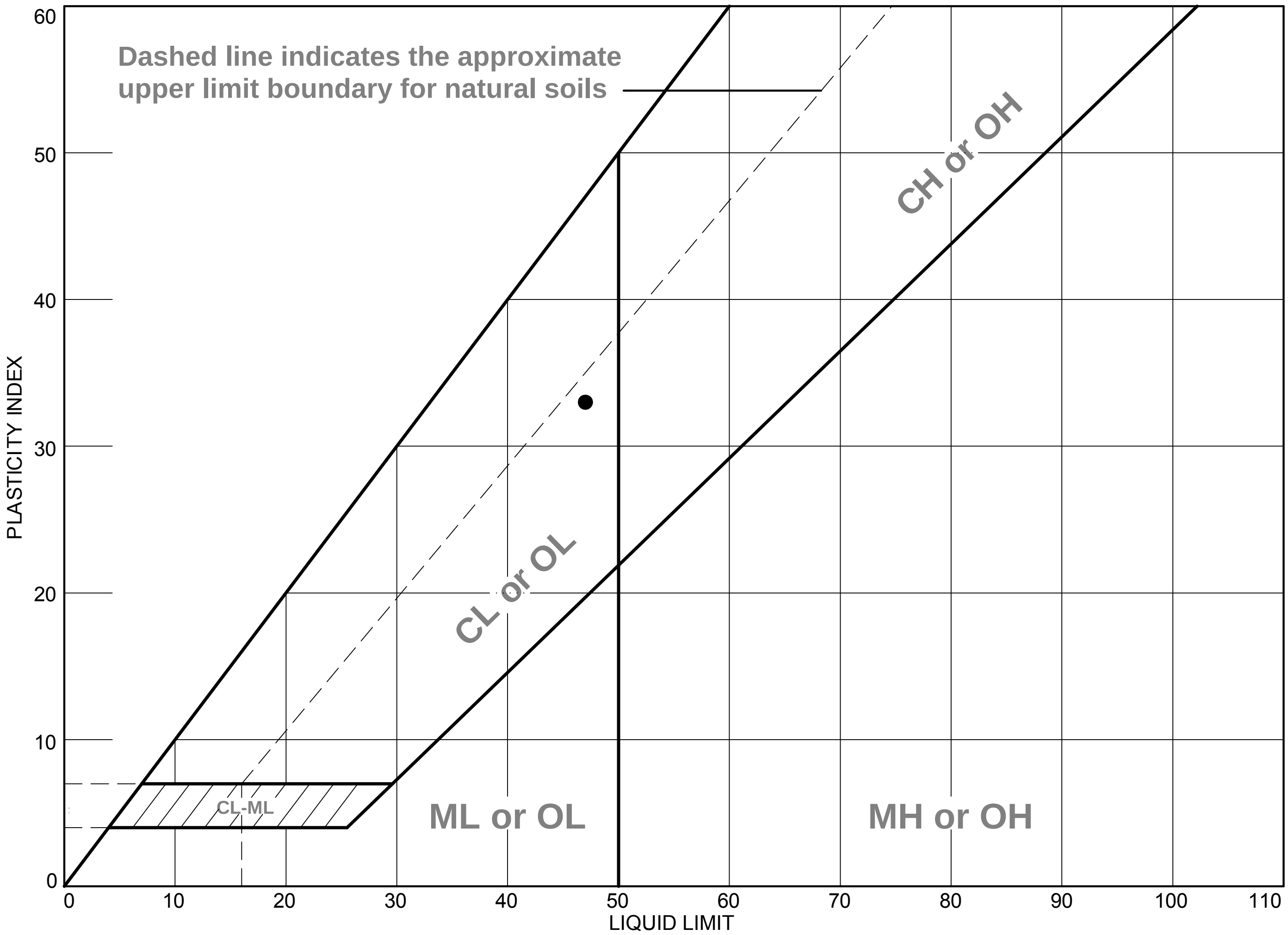
Remarks:

● PI: ASTM D4318

ENGEO INCORPORATED

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

LIQUID AND PLASTIC LIMITS TEST REPORT



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	See exploration logs	47	14	33	90.9	71.6	CL

Project No. 5900.200.000 **Client:** The Bruzzone Family

Project: Indian Valley - Preliminary GEX update

● **Depth:** 5.0-15.0 feet **Sample Number:** 2-B1 @ 5-15 (TxCU)

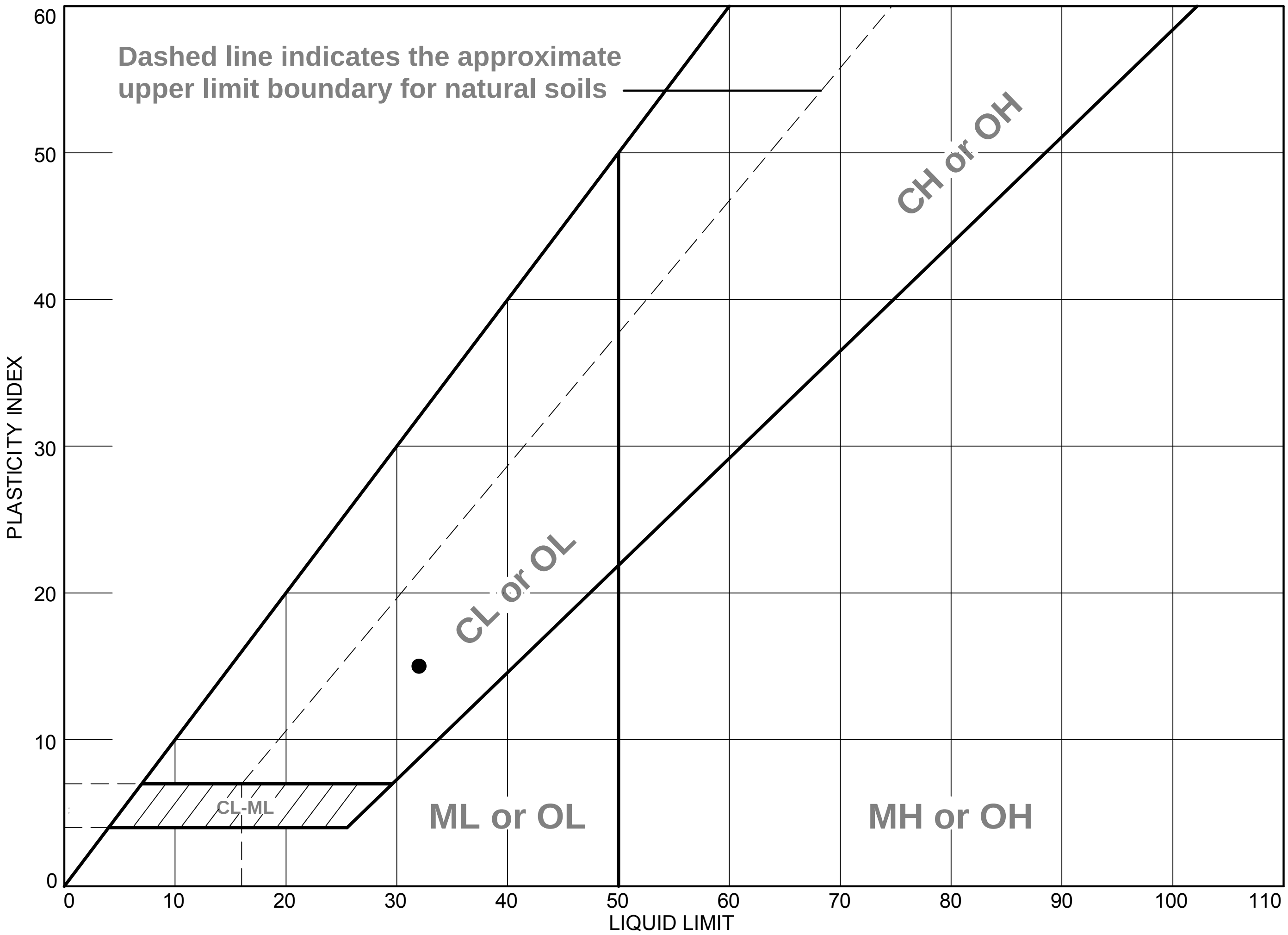
Remarks:

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

ENGEO INCORPORATED

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

LIQUID AND PLASTIC LIMITS TEST REPORT



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	See exploration logs	32	17	15			

Project No. 5900.200.000 **Client:** The Bruzzone Family
Project: Indian Valley - Preliminary GEX update

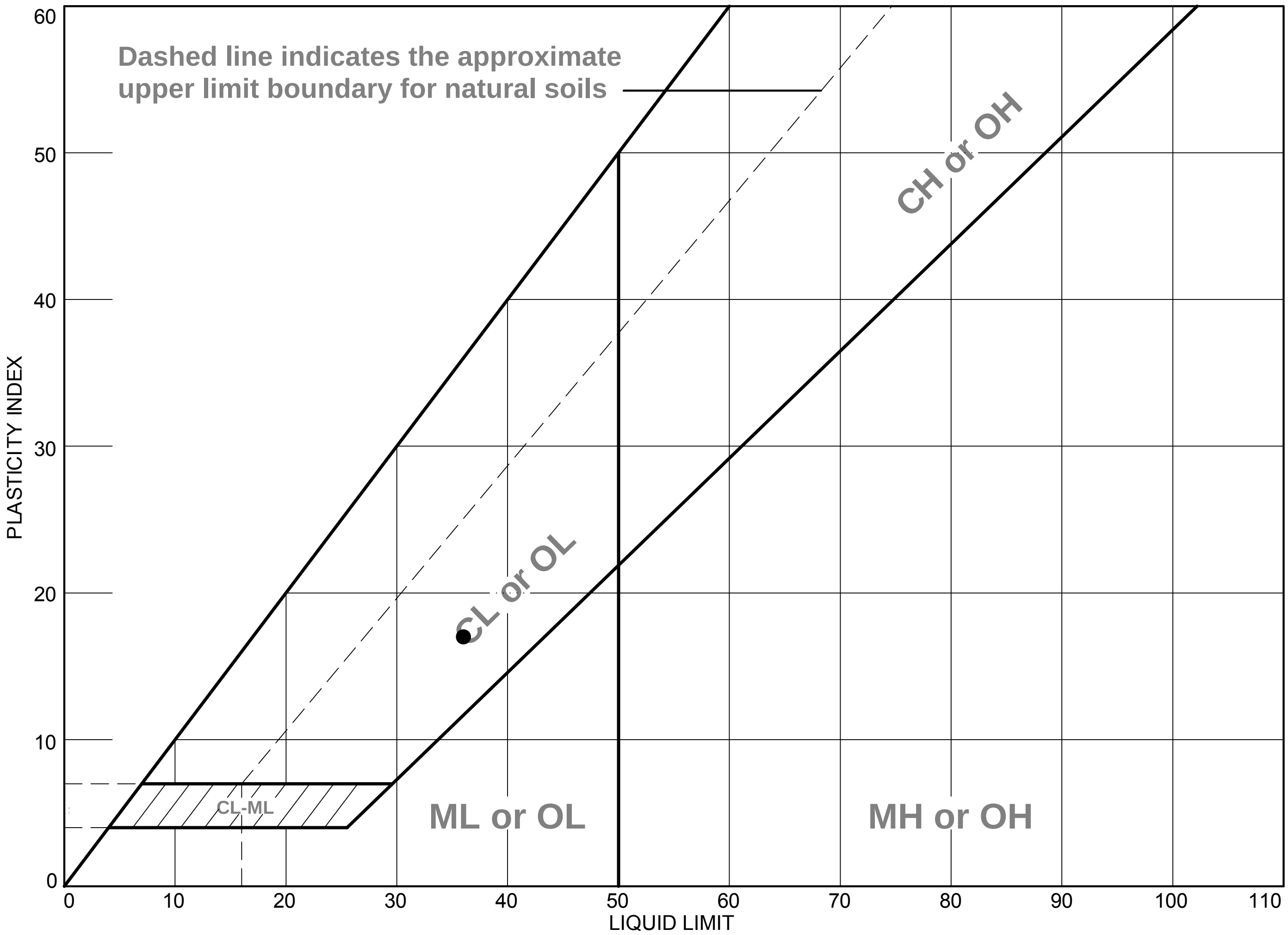
●**Depth:** 16.0 feet **Sample Number:** 2-B1 @ 16

Remarks:
●PI: ASTM D4316

ENGEO INCORPORATED

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

LIQUID AND PLASTIC LIMITS TEST REPORT



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	See exploration logs	36	19	17			

Project No. 5900.200.000 **Client:** The Bruzzone Family
Project: Indian Valley - Preliminary GEX update

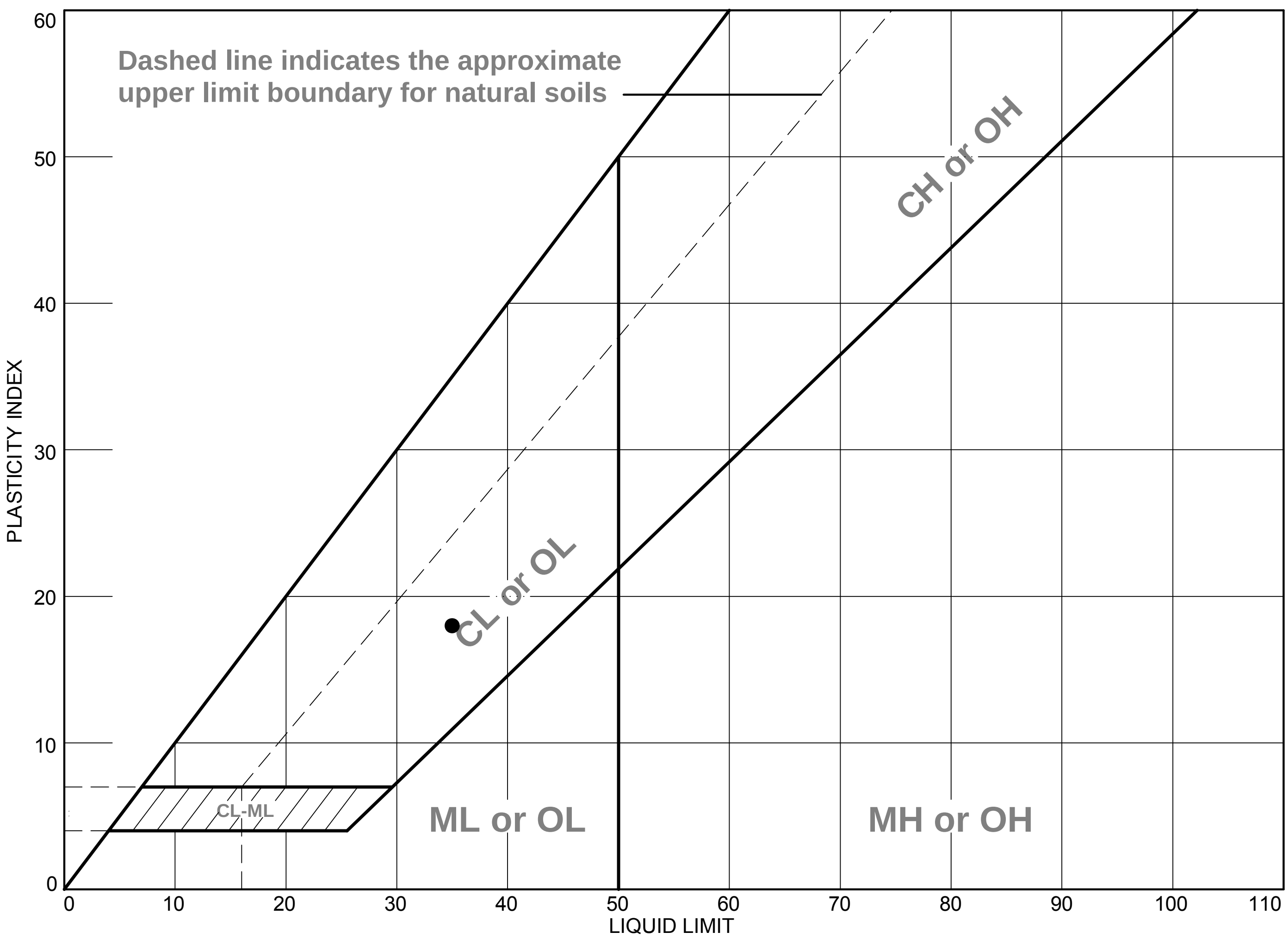
● **Depth:** 26.0 feet **Sample Number:** 2-B1 @ 26

Remarks:
● PI: ASTM D4318

ENGEO INCORPORATED

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

LIQUID AND PLASTIC LIMITS TEST REPORT



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	See exploration logs	35	17	18			

Project No. 5900.200.000 **Client:** The Bruzzone Family

Project: Indian Valley - Preliminary GEX update

● **Depth:** 46.0 feet **Sample Number:** 2-B1 @ 46

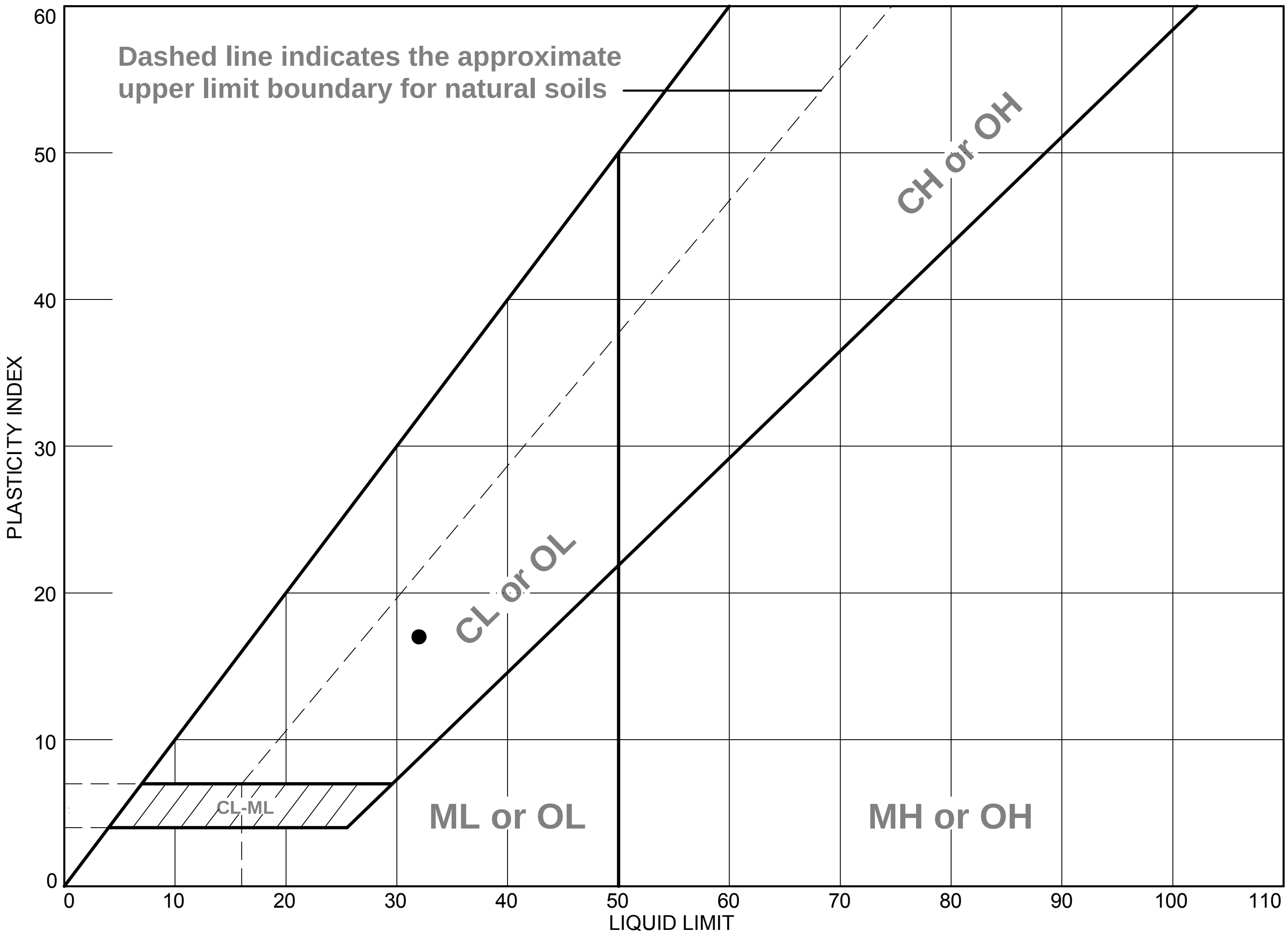
Remarks:

● PI: ASTM D4318

ENGEO INCORPORATED

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

LIQUID AND PLASTIC LIMITS TEST REPORT



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	See exploration logs	32	15	17	71.6	38.0	SC

Project No. 5900.200.000 **Client:** The Bruzzone Family

Project: Indian Valley - Preliminary GEX update

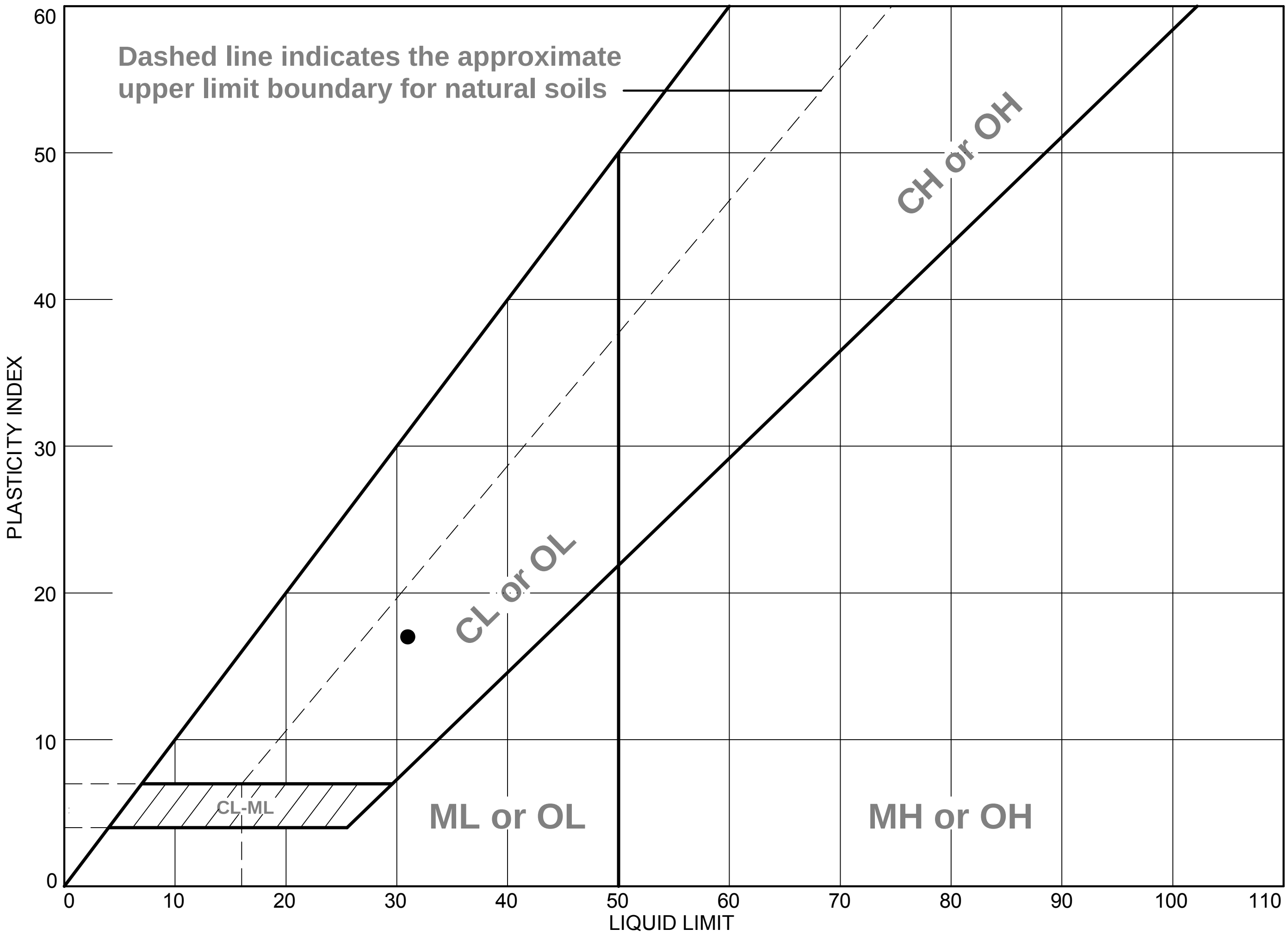
●**Depth:** 51.0 feet **Sample Number:** 2-B1 @ 51

Remarks:
●PI: ASTM D4318,
GS: ASTM D422,
USCS: ASTM D2487

ENGEO INCORPORATED

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

LIQUID AND PLASTIC LIMITS TEST REPORT



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	See exploration logs	31	14	17			

Project No. 5900.200.000 **Client:** The Bruzzone Family

Project: Indian Valley - Preliminary GEX update

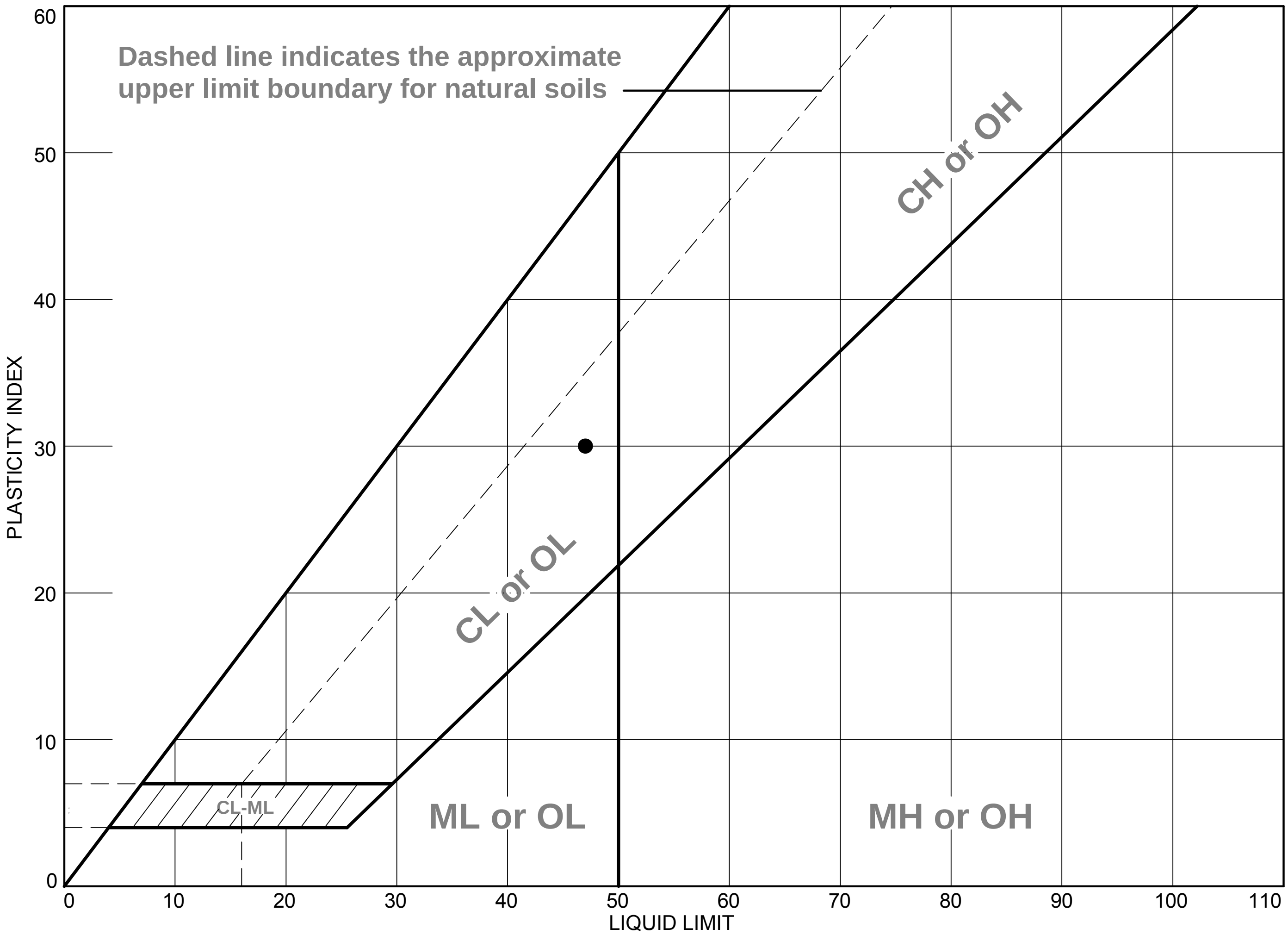
● **Depth:** 71.0 feet **Sample Number:** 2-B1 @ 71

Remarks:

● PI: ASTM D4318

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

LIQUID AND PLASTIC LIMITS TEST REPORT



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	See exploration logs	47	17	30			

Project No. 5900.200.000 **Client:** The Bruzzone Family
Project: Indian Valley - Preliminary GEX update

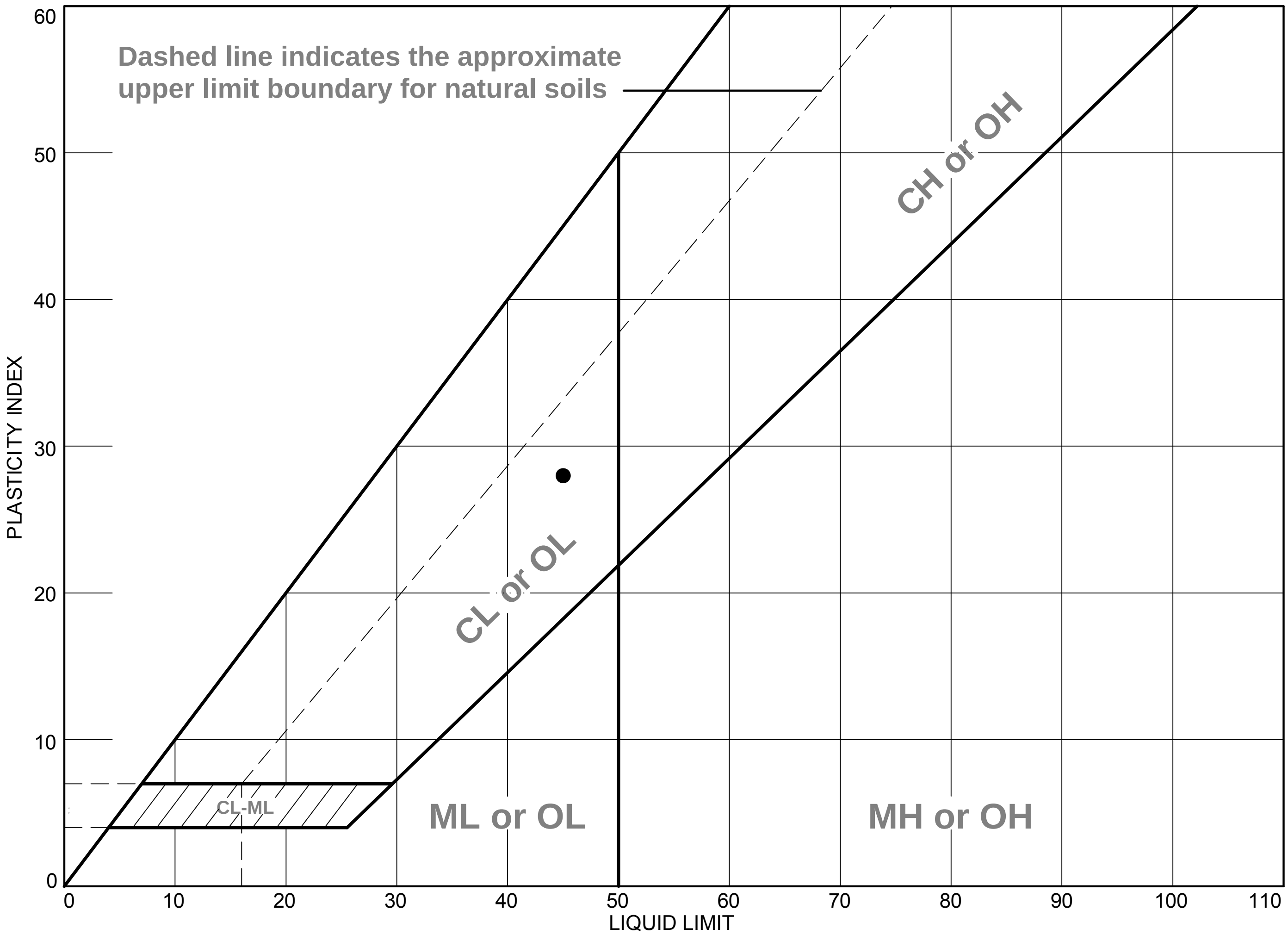
●**Depth:** 5.0 feet **Sample Number:** 2-B2 @ 5

Remarks:
●PI: ASTM D4318

ENGEO INCORPORATED

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

LIQUID AND PLASTIC LIMITS TEST REPORT



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	See exploration logs	45	17	28			

Project No. 5900.200.000 **Client:** The Bruzzone Family

Project: Indian Valley - Preliminary GEX update

● **Depth:** 5.0-15.0 feet **Sample Number:** 2-B2 @ 5-15 (TxUU)

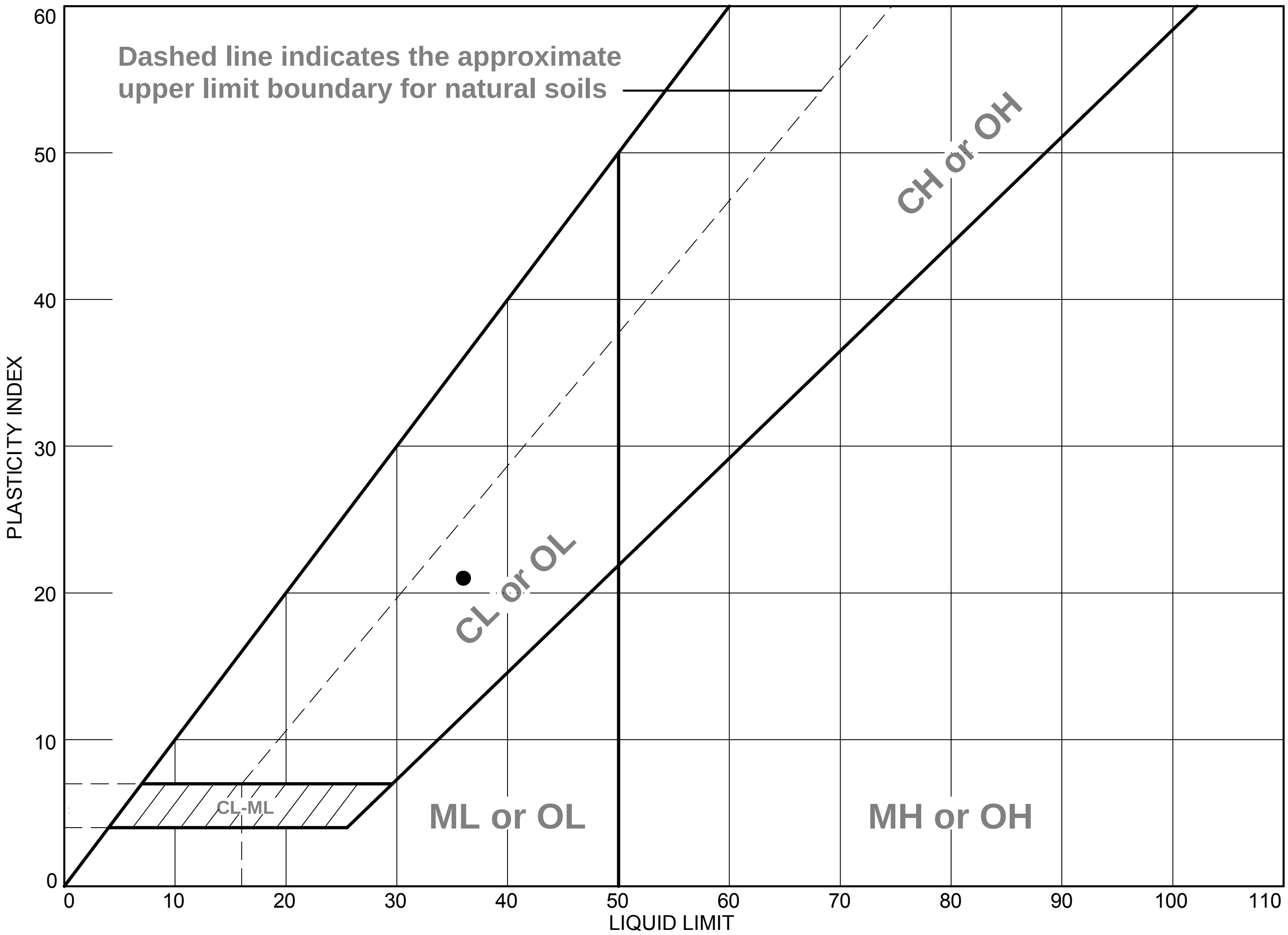
Remarks:

● PI: ASTM D4318

ENGEO INCORPORATED

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

LIQUID AND PLASTIC LIMITS TEST REPORT



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	See exploration logs	36	15	21			

Project No. 5900.200.000 **Client:** The Bruzzone Family

Project: Indian Valley - Preliminary GEX update

●**Depth:** 25.0 feet **Sample Number:** 2-B2 @ 25

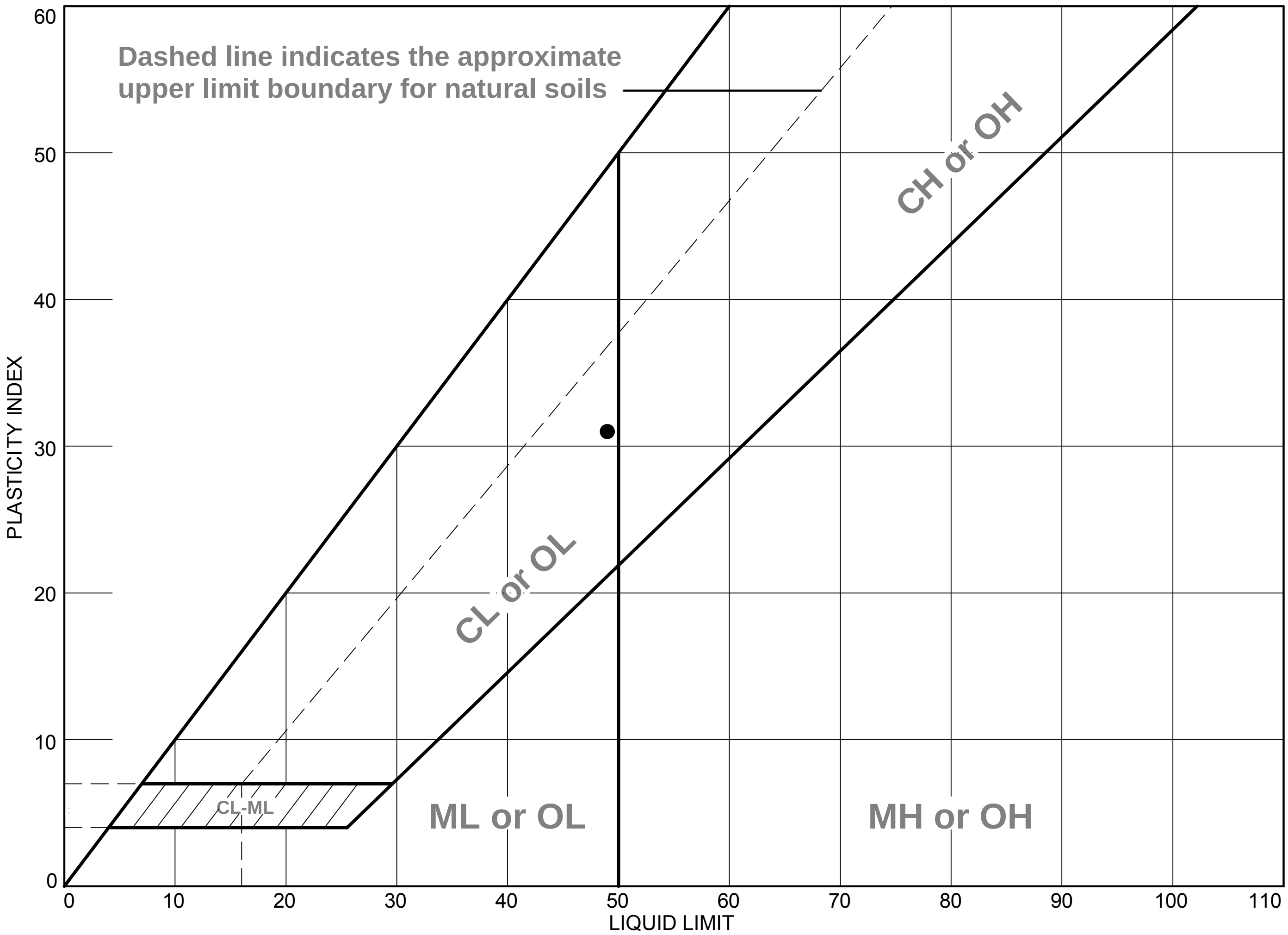
Remarks:

●PI: ASTM D4318

ENGEO INCORPORATED

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

LIQUID AND PLASTIC LIMITS TEST REPORT



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	See exploration logs	49	18	31			

Project No. 5900.200.000 **Client:** The Bruzzone Family

Project: Indian Valley - Preliminary GEX update

● **Depth:** 50.0 feet **Sample Number:** 2-B2 @ 50

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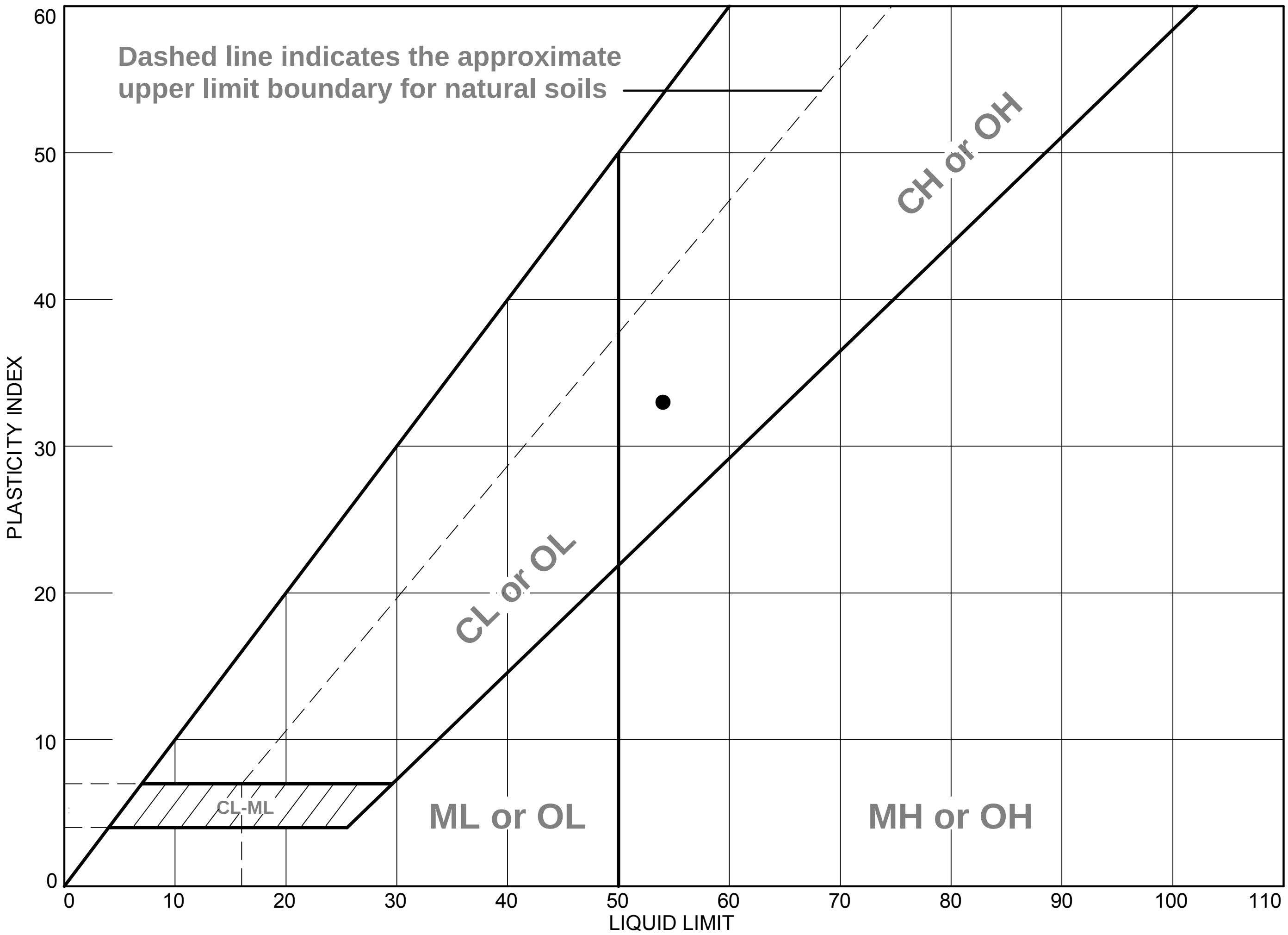
Remarks:

● PI: ASTM D4318

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

LIQUID AND PLASTIC LIMITS TEST REPORT



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	See exploration logs	54	21	33			

Project No. 5900.200.000 **Client:** The Bruzzone Family
Project: Indian Valley - Preliminary GEX update

● **Depth:** 60.0 feet **Sample Number:** 2-B2 @ 60

Remarks:
● PI: ASZTM D4318

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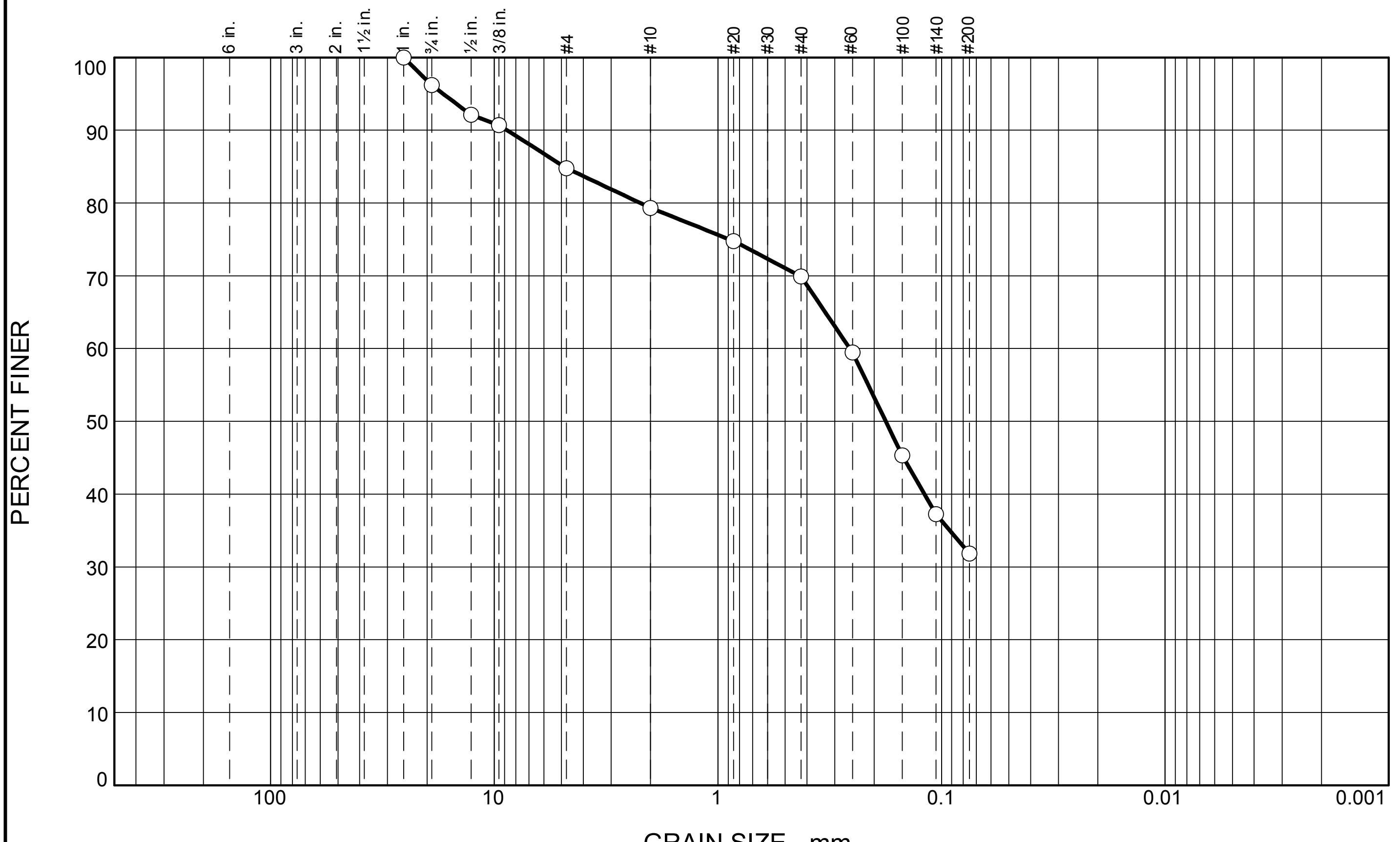
Tested By: GC **Checked By:** DS

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	3.8	11.4	5.5	9.4	38.1	31.8	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1	100.0		
3/4	96.2		
1/2	92.1		
3/8	90.7		
#4	84.8		
#10	79.3		
#20	74.8		
#40	69.9		
#60	59.5		
#100	45.3		
#140	37.2		
#200	31.8		

* (no specification provided)

Soil Description

See exploration logs

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 8.7743 D_{85}= 4.8784 D_{60}= 0.2567
D_{50}= 0.1775 D_{30}= D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= AASHTO=

Remarks

GS: ASTM D422

Sample Number: 2-B1 @ 42.5 **Depth:** 42.5 feet

Date: 8.23.13

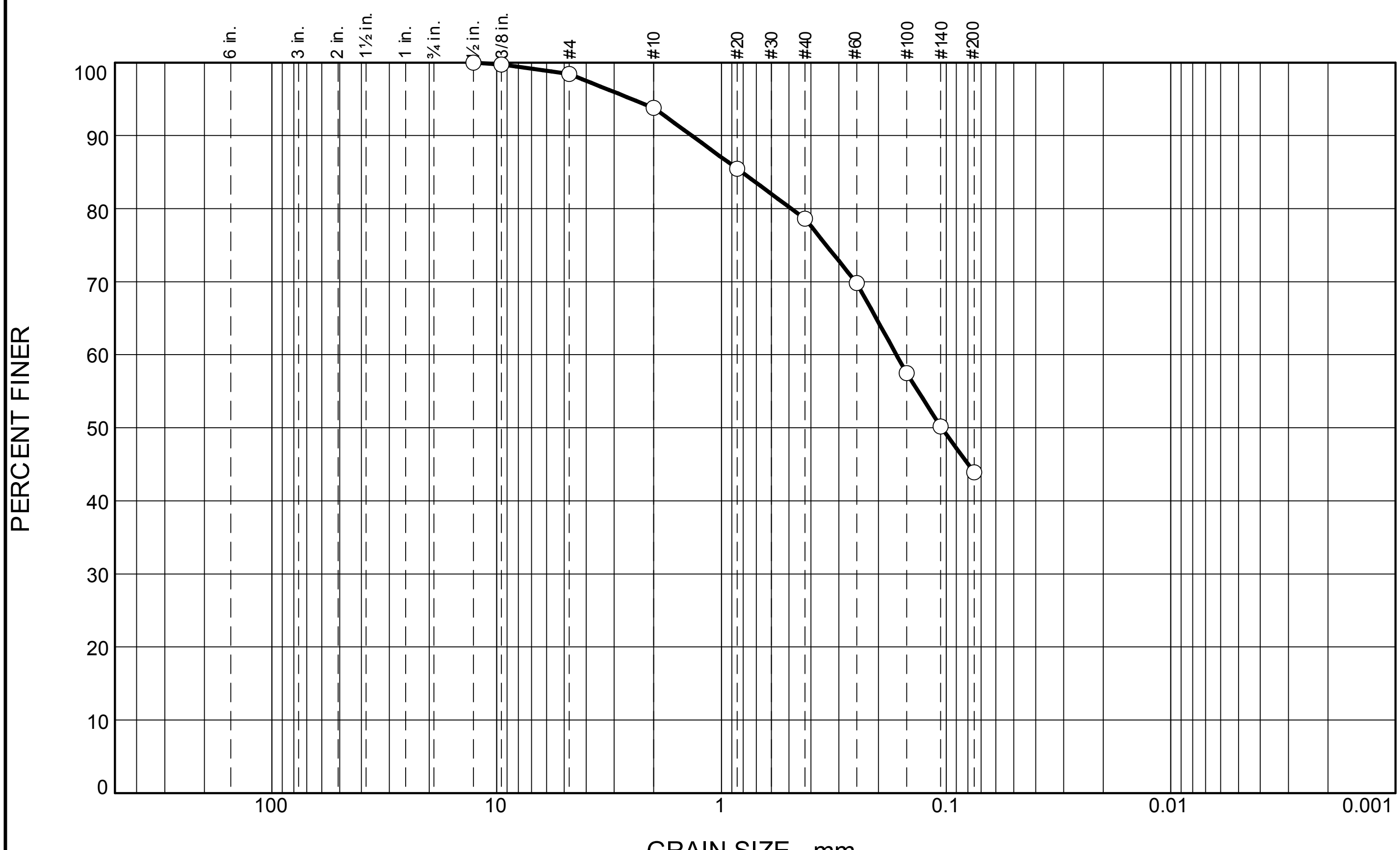
ENGEO INCORPORATED

Client: The Bruzzone Family
Project: Indian Valley - Preliminary GEX update

Project No: 5900.200.000

Tested By: TB **Checked By:** DS

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	1.6	4.6	15.2	34.7	43.9	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1/2	100.0		
3/8	99.7		
#4	98.4		
#10	93.8		
#20	85.4		
#40	78.6		
#60	69.8		
#100	57.5		
#140	50.2		
#200	43.9		

* (no specification provided)

Soil Description

See exploration logs

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 1.3564 D_{85}= 0.8122 D_{60}= 0.1665
D_{50}= 0.1049 D_{30}= D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= AASHTO=

Remarks

GS: ASTM D422

Sample Number: 2-B1 @ 61 **Depth:** 61.0 feet

Date: 8.23.13

ENGEO INCORPORATED

Client: The Bruzzone Family
Project: Indian Valley - Preliminary GEX update

Project No: 5900.200.000

Tested By: TB **Checked By:** DS

ENGEO Incorporated

Unconfined Compression Test Report (ASTM D2166)

Date 8.22.13

Checked By D. Seibold

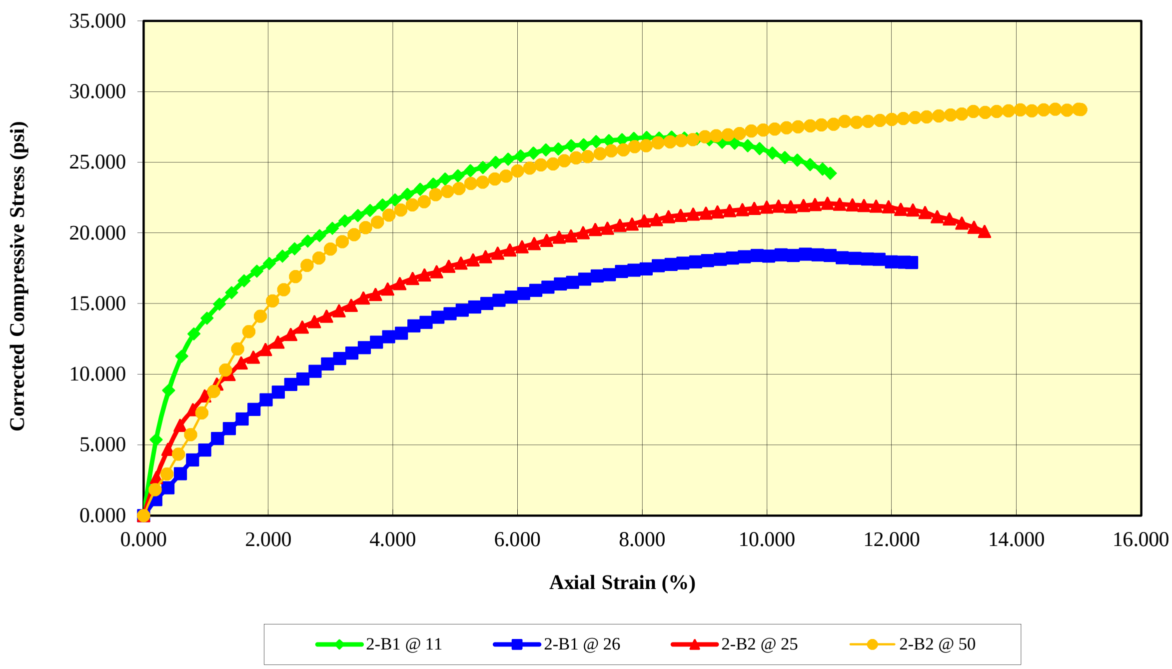
Date 8/22/2013

Computed By T. Borde

Date 8/21/2013

Tested By T. Borde

Compressive Stress Axial Strain Curve



Before Test	Specimen			
	2-B1 @ 11	2-B1 @ 26	2-B2 @ 25	2-B2 @ 50
Water Content (%)	19.61	27.82	20.77	26.60
Dry Density (pcf)	106.800	95.900	108.200	98.800
Saturation (%)	94.66	100.00	100.00	100.00
Void Ratio	0.55	0.73	0.53	0.67
Diameter (in)	2.388	2.420	2.413	2.427
Height (in)	5.000	5.130	5.150	5.380
Height-to-Diameter Ratio	2.094	2.120	2.134	2.217
Test Data	**2-B1 @ 11**	**2-B1 @ 26**	**2-B2 @ 25**	**2-B2 @ 50**
Unconfined Compressive Strength (psf)	3857.610	2664.504	3181.433	4142.283
Unconfined Compressive Strength (tsf)	1.929	1.332	1.591	2.071
Undrained Shear Strength (psf)	1928.805	1332.252	1590.716	2071.142
Strain at Failure (%)	8.472	10.616	10.967	14.622
Strain Rate (in./min.)	0.050000	0.050000	0.050000	0.050000

Project Information		Sample ID	Description
Project Number	5900.200.000	2-B1 @ 11	See exploration logs
Project Name	Indian Valley	2-B1 @ 26	See exploration logs
		2-B2 @ 25	See exploration logs
Test Date	8/21/2013	2-B2 @ 50	See exploration logs
Project Location	Moraga, CA		
Client	The Bruzzone Family		
Remarks			

Index Properties	2-B1 @ 11	2-B1 @ 26	2-B2 @ 25	2-B2 @ 50
Specific Gravity:	2.65	2.65	2.65	2.65
Liquid Limit:				
Plastic Limit:				

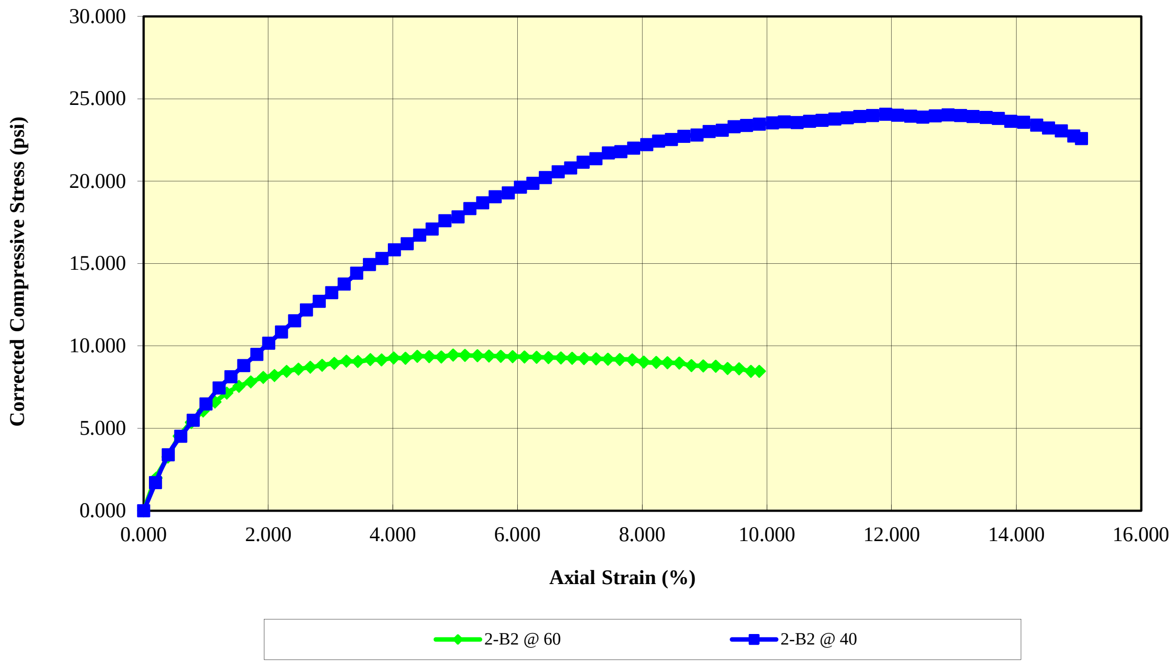
ENGEO Incorporated 2010 Crow Canyon Place Suite 250 San Ramon, CA 94583
Laboratory address: 2057 San Ramon Valley Blvd., San Ramon, CA 94583 (925) 837-2973

ENGEO Incorporated

Unconfined Compression Test Report (ASTM D2166)

Date 8.22.13
Checked By D. Seibold
Date 8/22/2013
Computed By T. Borde
Date 8/22/2013
Tested By T. Borde

Compressive Stress Axial Strain Curve



	Specimen			
Before Test	**2-B2 @ 60**	**2-B2 @ 40**		
Water Content (%)	34.46	18.32		
Dry Density (pcf)	88.100	109.500		
Saturation (%)	100.00	95.01		
Void Ratio	0.88	0.51		
Diameter (in)	2.417	2.416		
Height (in)	5.280	5.000		
Height-to-Diameter Ratio	2.185	2.070		
Test Data	**2-B2 @ 60**	**2-B2 @ 40**		
Unconfined Compressive Strength (psf)	1361.627	3465.093		
Unconfined Compressive Strength (tsf)	0.681	1.733		
Undrained Shear Strength (psf)	680.813	1732.547		
Strain at Failure (%)	4.966	11.901		
Strain Rate (in./min.)	0.050000	0.050000		

Project Information		**Sample ID**	**Description**
Project Number	5900.200.000	**2-B2 @ 60**	See exploration logs
Project Name	Indian Valley	**2-B2 @ 40**	See exploration logs
Test Date	8/22/2013		
Project Location	Moraga, CA		
Client	The Bruzzone Family		
Remarks			

Index Properties	**2-B2 @ 60**	**2-B2 @ 40**		
Specific Gravity:	2.65	2.65		
Liquid Limit:				
Plastic Limit:				

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Laboratory address: 2057 San Ramon Valley Blvd., San Ramon, CA 94583 (925) 837-2973

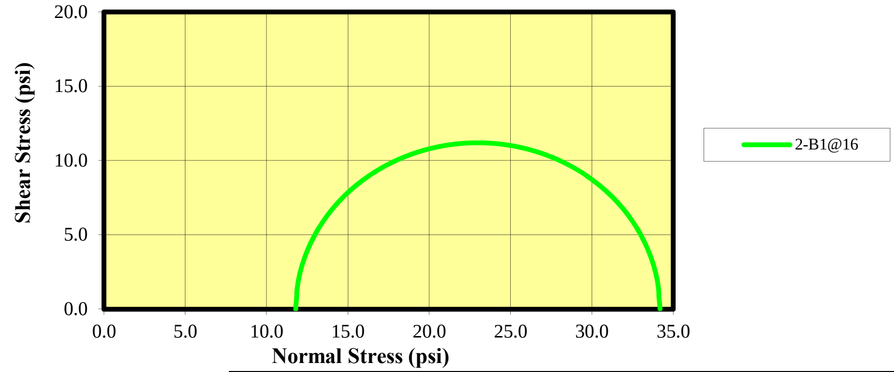
EN GEO

Consolidated Undrained Triaxial Test (ASTM D4767)

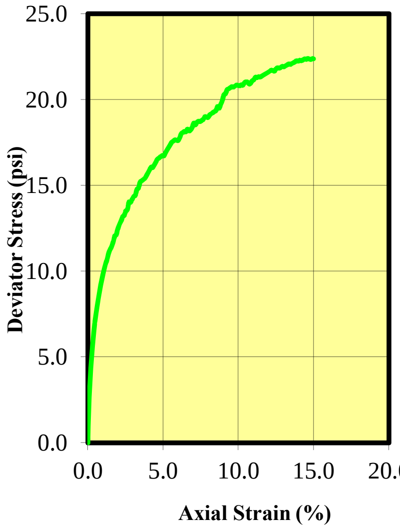
Checked By: G. Criste Date: 8.27.13

Tested By: D. Seibold Date: 8.27.13

Effective Stress at Maximum Deviator Stress Criterion



Deviator Stress Vs. Axial Strain



Initial	2-B1@16			
Height-to-Diameter Ratio	2.100			
Water Content (%)	22.2			
Dry Density (pcf)	107.6			
Saturation (%)	99.89			
Void Ratio	0.620			
Diameter (in)	2.376			
Height (in)	4.990			
Specific Gravity	2.798			
Liquid Limit				
Plastic Limit				
After Consolidation	**2-B1@16**			
B-Value	0.96			
Water Content (%)	22.48			
Dry Density (pcf)	109.08			
Saturation (%)	108.42			
Void Ratio	0.580			
Effective Stress (psi)	12.3			
Back Press. (psi)	50.9			
Rate of Strain	0.0002			
Height-to-Diameter Ratio	2.109			
After Shear	**2-B1@16**			
σ'1 at Failure (psi)	34.18			
σ'3 at Failure (psi)	11.78			
t_{50}	N/A			
t_{90}	174.1			

Maximum Deviator Stress Criterion	
C (psi)	0.0
C' (psi)	0.0
Ø (deg)	0.0
Ø' (deg)	0.0

Project:	Indian Valley - Preliminary GEX update
Location:	Moraga, California
Project Number:	5900.200.000
Boring Number:	B1
Sample Number:	Multiple
Depth:	16.0 feet
Sample Type:	Undisturbed
Description:	See exploration logs
Test Type	Consolidated Undrained
Remarks	

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

EN **GEO**

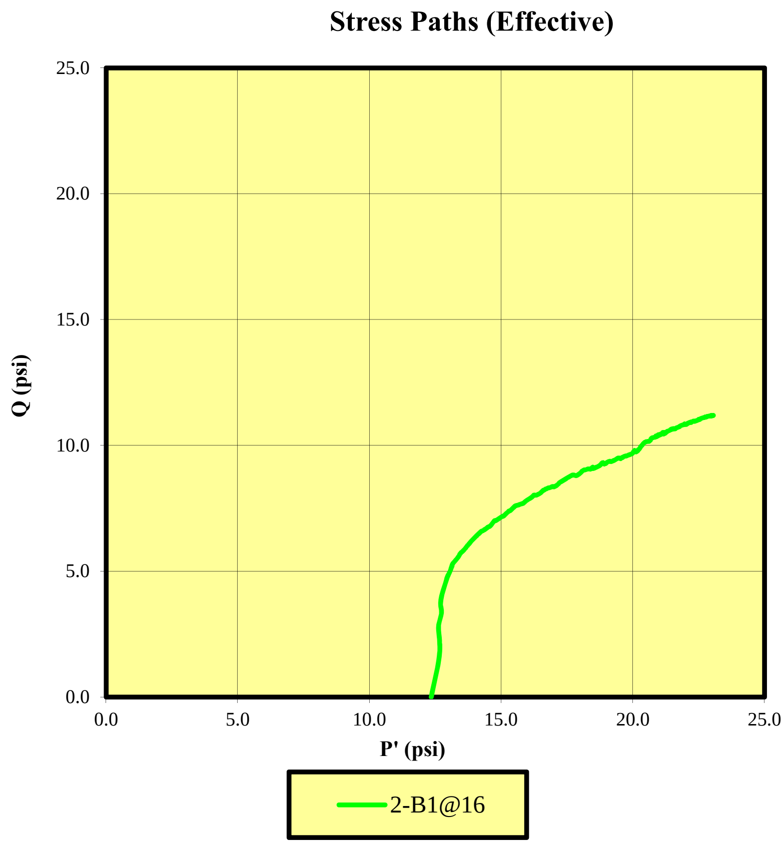
Consolidated Undrained Triaxial Test (ASTM D4767)

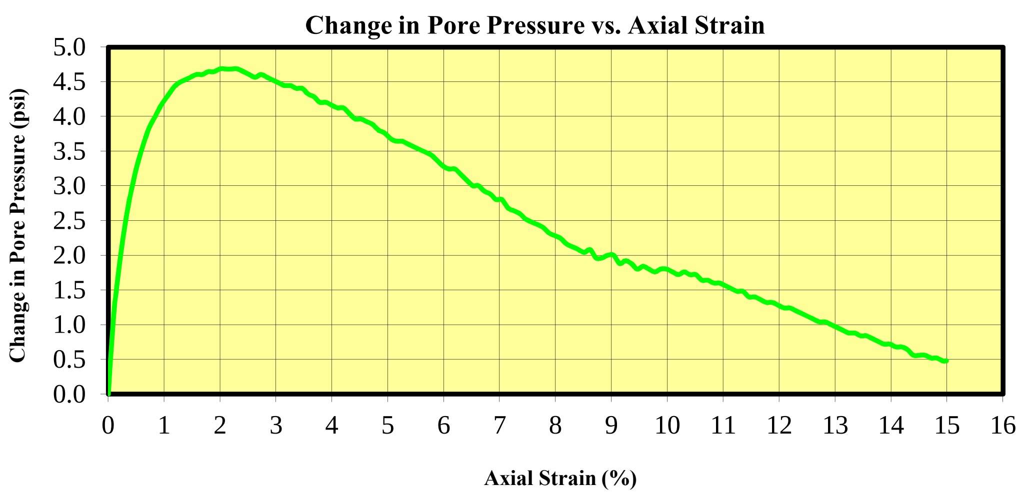
Date: 8.27.13

Checked By: G. Criste

Date: 8.27.13

Tested By: D. Seibold





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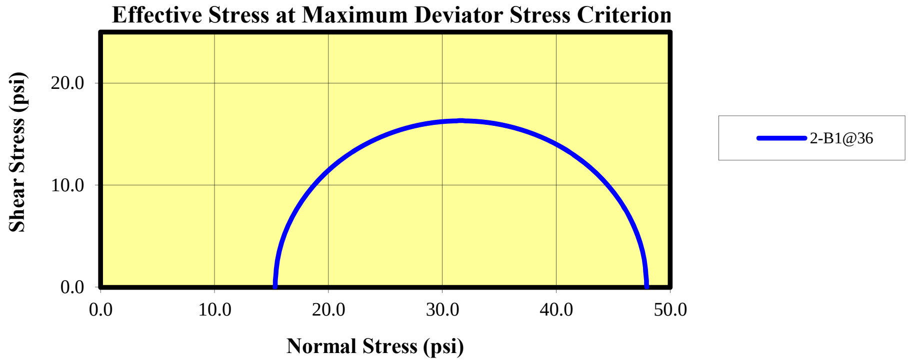
Consolidated Undrained Triaxial Test (ASTM D4767)

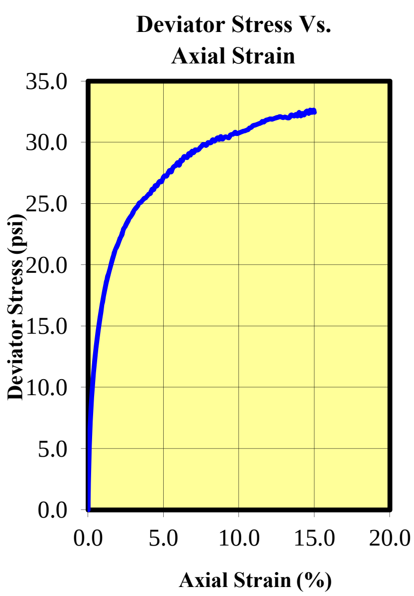
Date:

Checked By:

Date: 8.30.13

Tested By: D. Seibold





Initial	2-B1@36		
Height-to-Diameter Ratio	2.098		
Water Content (%)	23.5		
Dry Density (pcf)	107.6		
Saturation (%)	99.43		
Void Ratio	0.686		
Diameter (in)	2.379		
Height (in)	4.990		
Specific Gravity	2.910		
Liquid Limit	0		
Plastic Limit	0		
After Consolidation	**2-B1@36**		
B-Value	0.00		
Water Content (%)	21.97		
Dry Density (pcf)	111.22		
Saturation (%)	100.94		
Void Ratio	0.633		
Effective Stress (psi)	22.3		
Back Press. (psi)	49.7		
Rate of Strain	0.00028		
Height-to-Diameter Ratio	2.091		
After Shear	**2-B1@36**		
σ'1 at Failure (psi)	47.94		
σ'3 at Failure (psi)	15.30		
t_{50}	N/A		
t_{90}	129.4		

Maximum Deviator Stress Criterion	
C (psi)	0.0
C' (psi)	-2.7
Ø (deg)	0.0
Ø' (deg)	36.5

Project:	Indian Valley - Preliminary GEX update
Location:	Moraga, California
Project Number:	5900.200.000
Boring Number:	B1
Sample Number:	Multiple
Depth:	36.0 feet
Sample Type:	Undisturbed
Description:	See exploration logs
Test Type	Consolidated Undrained
Remarks	

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

ENGEO

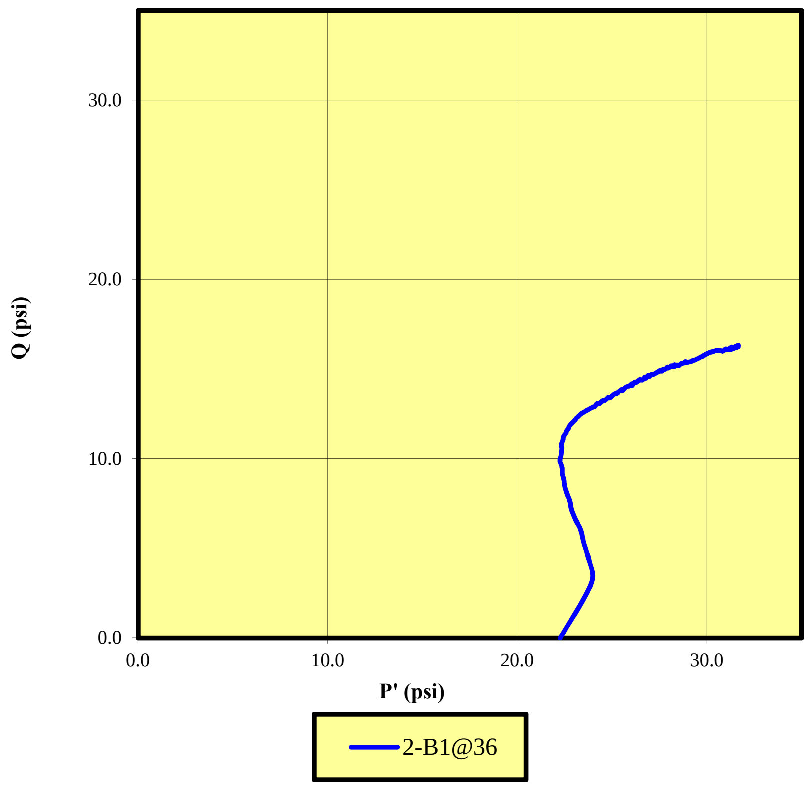
Consolidated Undrained Triaxial Test (ASTM D4767)

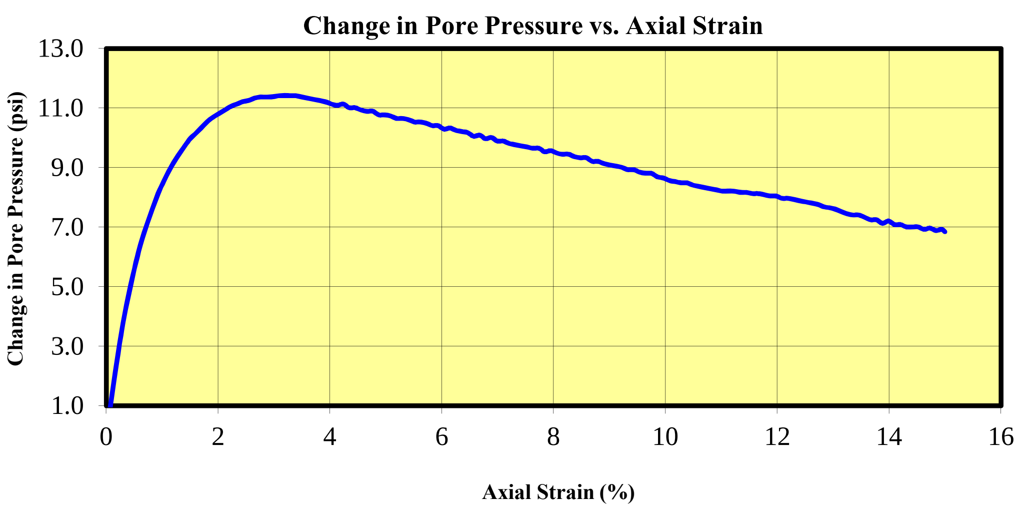
Date:

Checked By:

Date: 8.30.13

Tested By: D. Seibold





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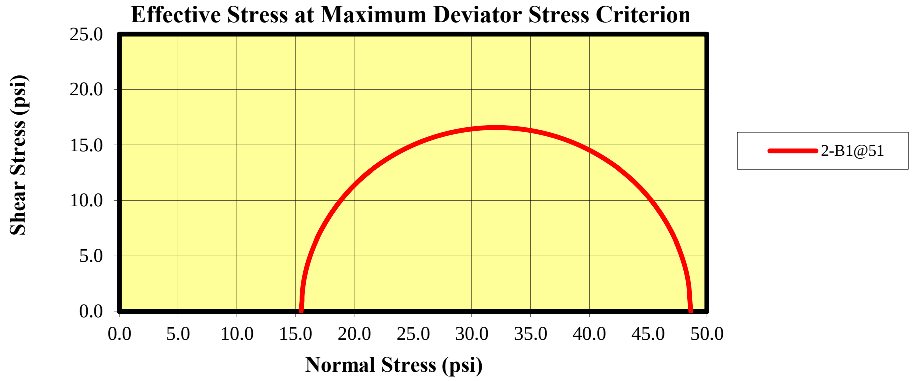
Consolidated Undrained Triaxial Test (ASTM D4767)

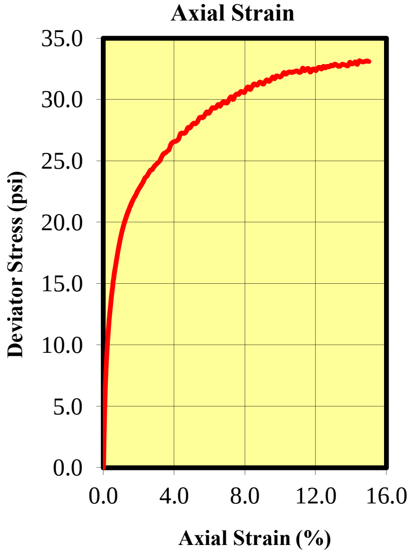
Date: 8.27.13 Checked By: G. Criste

Date: 8.27.13 Tested By: D. Seibold

Effective Stress at Maximum Deviator Stress Criterion



Deviator Stress Vs. Axial Strain



Initial			2-B1@51	
Height-to-Diameter Ratio			2.105	
Water Content (%)			19.0	
Dry Density (pcf)			112.4	
Saturation (%)			97.43	
Void Ratio			0.537	
Diameter (in)			2.380	
Height (in)			5.010	
Specific Gravity			2.772	
Liquid Limit				
Plastic Limit				
After Consolidation			**2-B1@51**	
B-Value			0.98	
Water Content (%)			18.19	
Dry Density (pcf)			115.83	
Saturation (%)			102.08	
Void Ratio			0.494	
Effective Stress (psi)			27.1	
Back Press. (psi)			52.6	
Rate of Strain			0.00042	
Height-to-Diameter Ratio			2.086	
After Shear			**2-B1@51**	
σ'1 at Failure (psi)			48.62	
σ'3 at Failure (psi)			15.46	
t_{50}			45.9	
t_{90}			86.7	

Maximum Deviator Stress Criterion	
C (psi)	0.0
C' (psi)	0.0
Ø (deg)	0.0
Ø' (deg)	0.0

Project:	Indian Valley - Preliminary GEX update
Location:	Moraga, California
Project Number:	5900.200.000
Boring Number:	B1
Sample Number:	Multiple
Depth:	51.0 feet
Sample Type:	Undisturbed
Description:	See exploration logs
Test Type	Consolidated Undrained
Remarks	

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

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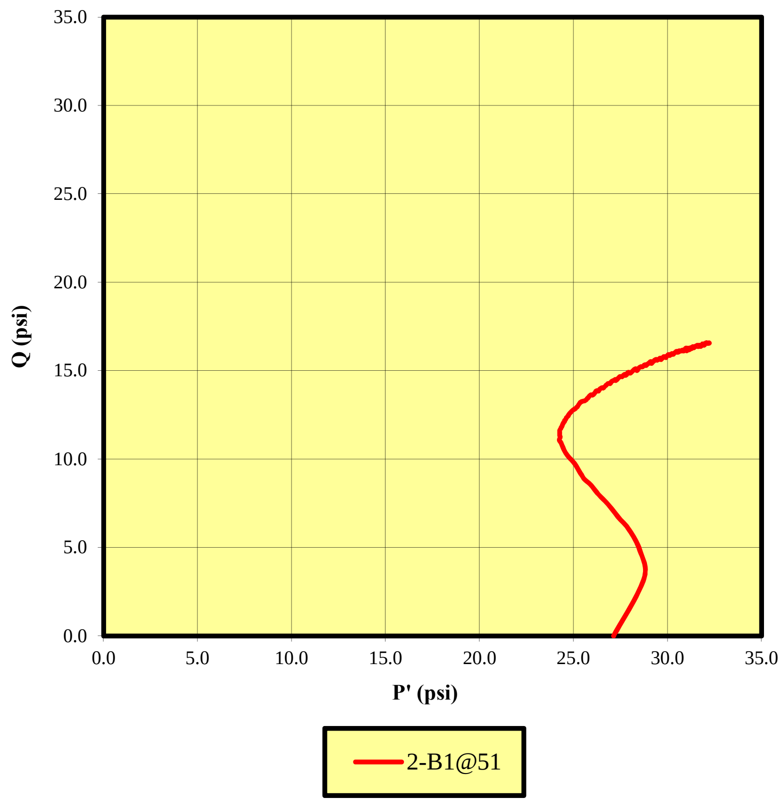
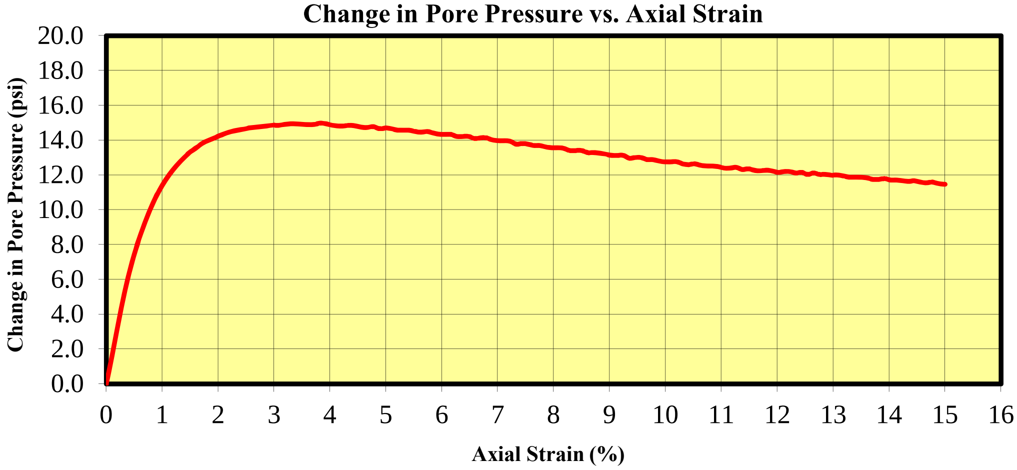
Consolidated Undrained Triaxial Test (ASTM D4767)

Date: 8.27.13

Checked By: G. Criste



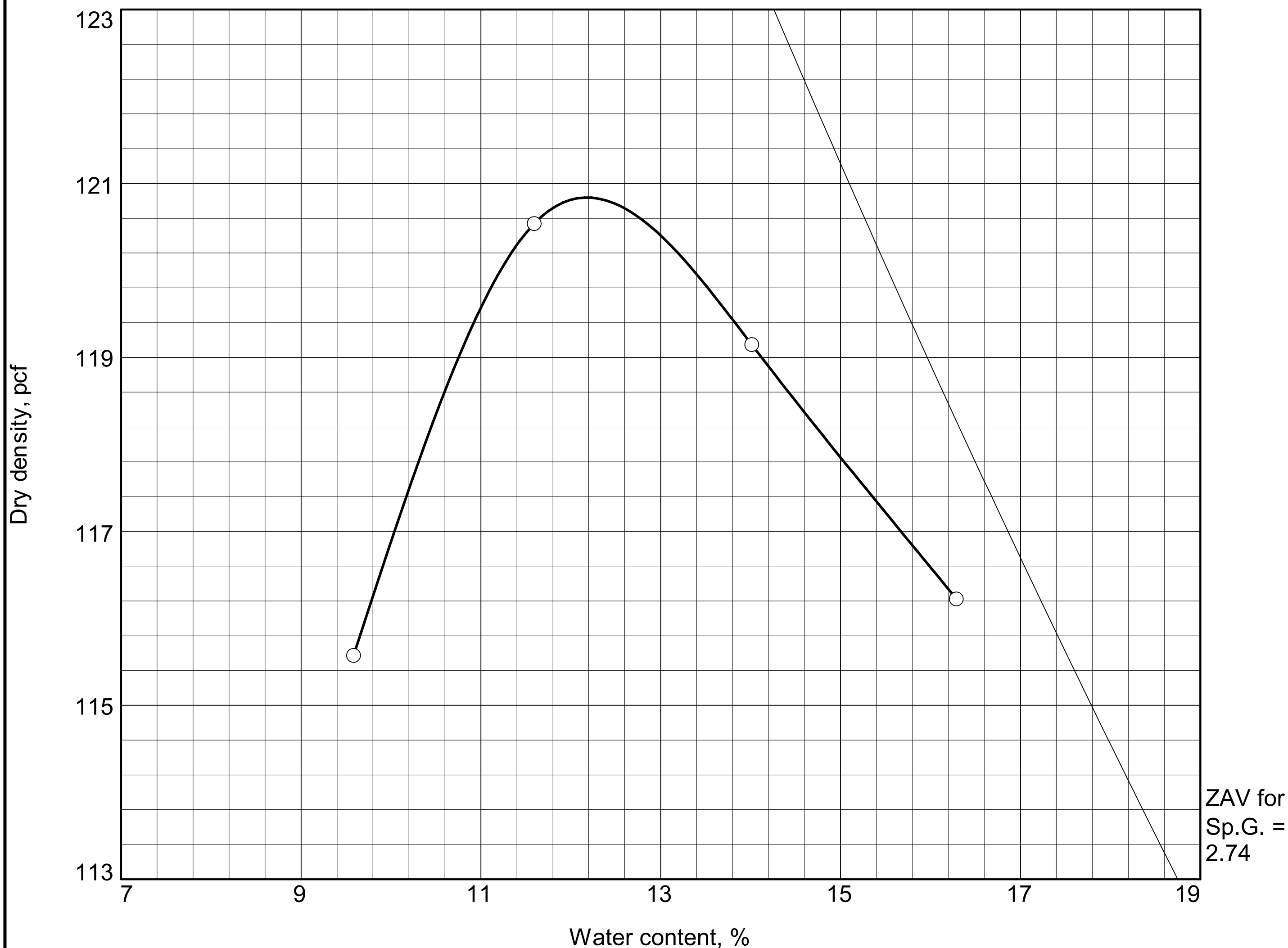
Change in Pore Pressure vs. Axial Strain



Date: 8.27.13

Tested By: D. Seibold

COMPACTION TEST REPORT For Curve No. 2-B1@5-15



Test specification: ASTM D 1557-07 Method B Modified

Elev/ Depth	Classification		Nat. Moist.	Sp.G.	LL	PI	% > 3/8 in.	% < No.200
	USCS	AASHTO						
5.0-15.0 feet								

TEST RESULTS	MATERIAL DESCRIPTION
Maximum dry density = 120.8 pcf Optimum moisture = 12.2 %	See exploration logs

Project No. 5900.200.000 **Client:** The Bruzzone Family
Project: Indian Valley - Preliminary GEX update
Date: 8.21.13
○ **Sample Number:** 2-B1 @ 5-15 (TxCU)

Remarks:

ENGEO INCORPORATED

Tested By: AW **Checked By:** DS

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Consolidated Undrained Triaxial Test (ASTM D4767)

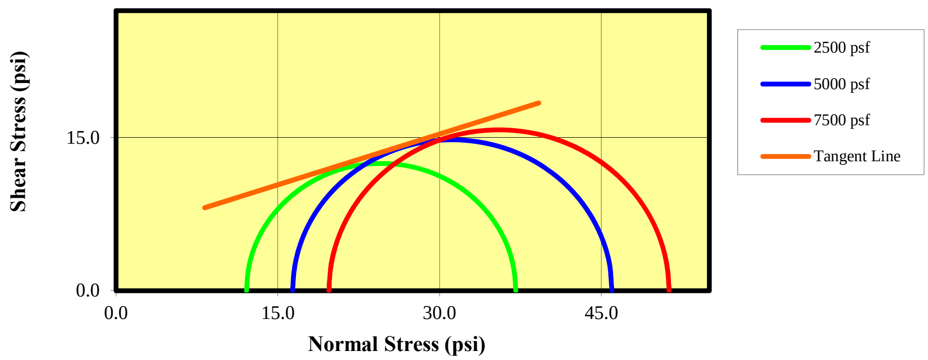
Date:

Checked By:

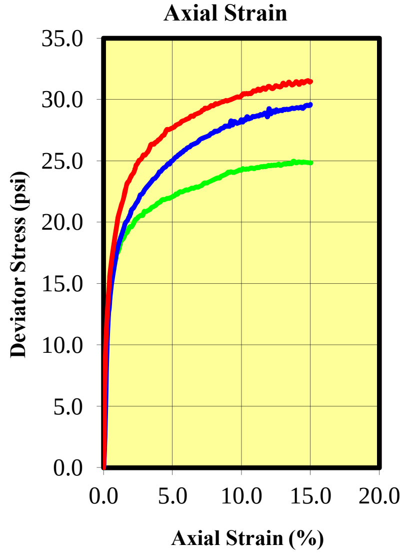
Date: 9.12.13

Tested By: D. Seibold

Effective Stress at Maximum Deviator Stress Criterion



Deviator Stress Vs. Axial Strain



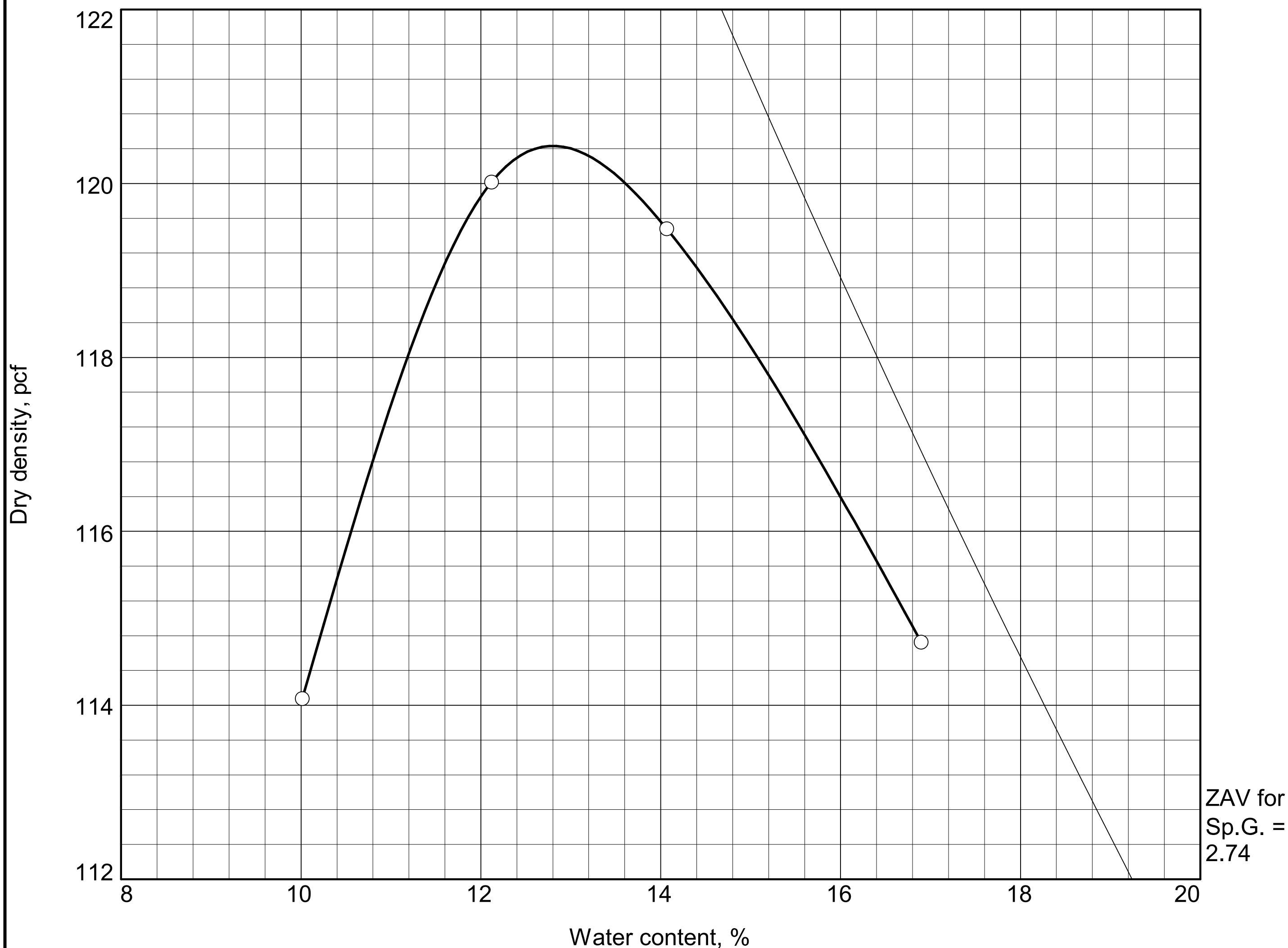
Initial	2500	5000	7500	
Height-to-Diameter Ratio	2.061	2.057	2.061	
Water Content (%)	15.6	15.6	15.6	
Dry Density (pcf)	106.9	107.2	106.9	
Saturation (%)	70.42	70.89	70.34	
Void Ratio	0.609	0.605	0.610	
Diameter (in)	2.426	2.426	2.426	
Height (in)	5.001	4.990	5.000	
Specific Gravity	2.761	2.761	2.761	
Liquid Limit	47	47	47	
Plastic Limit	14	14	14	
After Consolidation	**2500**	**5000**	**7500**	
B-Value	0.99	0.97	0.97	
Water Content (%)	22.22	21.85	21.21	
Dry Density (pcf)	103.40	106.48	109.44	
Saturation (%)	100.00	100.00	100.00	
Void Ratio	0.600	0.556	0.512	
Effective Stress (psi)	17.4	34.7	52.1	
Back Press. (psi)	51.8	45.0	49.5	
Rate of Strain	0.000725	0.000725	0.000725	
Height-to-Diameter Ratio	2.055	2.020	2.055	
After Shear	**2500**	**5000**	**7500**	
σ'1 at Failure (psi)	37.07	45.98	51.28	
σ'3 at Failure (psi)	12.09	16.36	19.73	
t_{50}	N/A	N/A	N/A	
t_{90}	3.36	10.43	24.37	

Maximum Deviator Stress Criterion	
C (psi)	10.2
C' (psi)	5.4
Ø (deg)	5.2
Ø' (deg)	18.4

Project:	Indian Valley - Preliminary GEX update
Location:	Moraga, California
Project Number:	5900.200.000
Boring Number:	2-B1
Sample Number:	2-B1@5-15
Depth:	5-15 feet
Sample Type:	Remolded
Description:	See exploration logs
Test Type	Consolidated Undrained
Remarks	

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

COMPACTION TEST REPORT For Curve No. 2-B2@5-15



Test specification: ASTM D 1557-07 Method B Modified

Elev/ Depth	Classification		Nat. Moist.	Sp.G.	LL	PI	% > 3/8 in.	% < No.200
	USCS	AASHTO						
5.0-15.0 feet								

TEST RESULTS	MATERIAL DESCRIPTION
Maximum dry density = 120.4 pcf Optimum moisture = 12.8 %	See exploration logs

Project No. 5900.200.000 **Client:** The Bruzzone Family

Project: Indian Valley - Preliminary GEX update

Date: 8.22.13

○ **Sample Number:** 2-B2 @ 5-15 (TxUU)

Remarks:

ENGEO INCORPORATED

Tested By: AW **Checked By:** DS

EN GEO

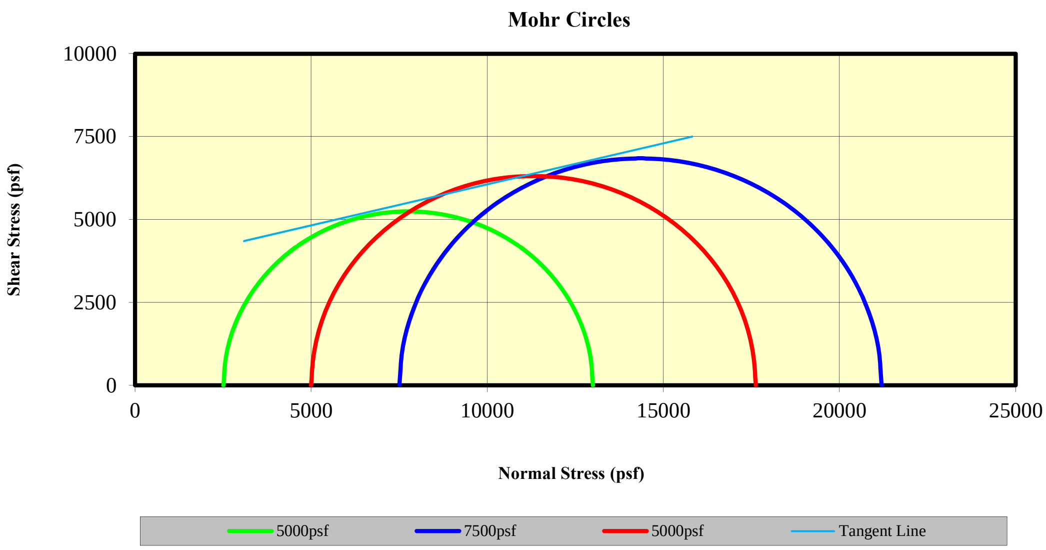
Unconsolidated Undrained Triaxial Test (ASTM D2850)

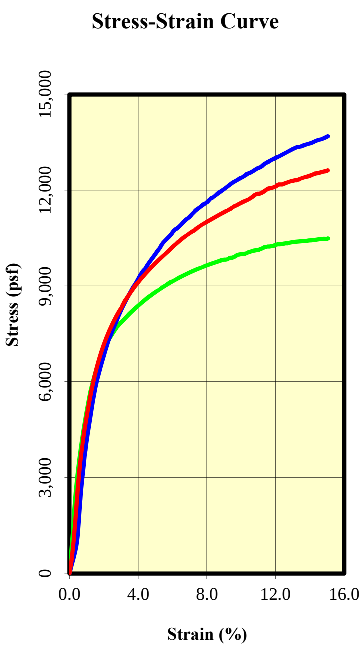
Date: 09/05/13

Checked By: D. seibold

Date: 09/05/13

Tested By: G. Criste





Before Test	Specimen 2500psf	7500psf	5000psf	
Water Content (%)	15.80	15.80	15.80	
Dry Density (pcf)	106.20	107.99	107.58	
Saturation (%)	75.04	78.71	77.86	
Void Ratio	0.56	0.53	0.54	
Diameter (in)	2.430	2.429	2.433	
Height (in)	5.027	4.971	4.896	
Liquid Limit	45.0	45.0	45.0	
Plastic Limit	17.0	17.0	17.0	
Specific Gravity	2.650	2.650	2.650	
Height-to-Diam. Ratio	2.069	2.047	2.012	
After Test	**2500psf**	**7500psf**	**5000psf**	
Water Content (%)	15.80	15.80	15.80	
Saturation (%)	75.04	78.71	77.86	
Strain Rate (in/min)	0.05	0.05	0.05	
Peak Deviator Stress (psf)	10491.3	13689.0	12622.4	
Axial Strain @ Failure (%)	15.056	15.041	15.032	
	Cell Pressure			
Cell (psf)	2505.6	7502.4	4996.8	
Back (psf)	n/a	n/a	n/a	
	Principle Stresses at Failure			
σ1 (psf)	12996.9	21191.4	17619.2	
σ3 (psf)	2505.6	7502.4	4996.8	

Mohr-Coulomb Parameters with a Non-zero Friction Angle (Ø≠0)			Cohesion at Failure with a Zero Friction Angle (Ø=0)			
Cohesion, c (psf)	3581.4		5245.6	6844.5	6311.2	
Friction Angle Ø	13.93		n/a	n/a	n/a	

Project Information			
Project Name:	Indian Valley - Preliminary GEX Update		
Project Number:	5900.200.000	Job Number:	5900.200.000
Location:		Boring Number:	2-B2
Client:	The Bruzzone Family	Sample Number:	2-B2@5.0-15.0 feet
Description:	See exploration logs		

APPENDIX C

Previous Exploration Logs and Laboratory Analysis (2003)

APPENDIX C

DATE OF BORING: May 28, 2003

SURFACE ELEVATION: Approx. 650 feet (198 meters)

DEPTH (FEET)	DEPTH (METERS)	SAMPLE NUMBER	DESCRIPTION	BLOWS/FT.	qu UNCON STRENGTH (TSF) *FIELD PENET. APPROX.	IN PLACE DRY UNIT WEIGHT (PCF)	IN PLACE MOIST. CONTENT % DRY WEIGHT
0							
	1	1-1	SILTY CLAY (CH), red brown, stiff, moist, some subrounded to subangular rock fragments.	18	1.25*		
5	2	1-2	Same as above.	16	1.0*	102	23.9
10	3	1-3	Same as above.	13	1.0*		
15	4, 5	1-4	SILTY CLAY (CH),red brown, stiff, moist, some subangular to subrounded rock fragments.	18	1.0*	110	20.3
20	6, 7	1-5	SILTY CLAY (CH), red brown, very stiff, moist, some subangular rock fragments.	22	2.20*		
25	8	1-6	SILTY CLAY (CH), dark red brown, very stiff, damp to moist, subangular rock fragments,	20	2.25*	111	18.5
30	9, 10	1-7	SILTY CLAY (CH), grayish brown, stiff to very stiff, moist, some subangular to subrounded rock fragments.	22	1.75*		
35							

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INDIAN VALLEY

LAFAYETTE, CALIFORNIA

BORING NO.: B-1

LOGGED BY: C. Tyler

PROJ. NO.: 5900.1.001.01

CHECKED BY PJS

FIGURE NO. A1

DEPTH (FEET)	DEPTH (METERS)	SAMPLE NUMBER	LOG, LOCATION AND TYPE OF SAMPLE	DATE OF BORING: May 28, 2003 SURFACE ELEVATION: Approx. 650 feet (198 meters) DESCRIPTION	BLOWS/FT.	qu UNCON STRENGTH (TSF) *FIELD PENET. APPROX.	IN PLACE DRY UNIT WEIGHT (PCF)	IN PLACE MOIST. CONTENT % DRY WEIGHT
35	11	1-8		SILTY CLAY (CH), grayish brown, moist, very stiff, trace subrounded rock fragments.	22	2.5*	109	20.5
40	12	1-9		SILTY CLAY (CH), dark grayish brown, very stiff, moist, trace oxidation.	29	1.75*		
45	13, 14	1-10		SILTY CLAY (CH), dark gray, very stiff, moist, some sand, some subrounded and angular rock fragments, some oxidation.	30	2.25*	105	21.4
50	15	1-11		SILTY CLAY (CH), olive brown, moist to damp, hard, some sand, trace subrounded rock fragments, some oxidation.	44	3.25*		
				Bottom of boring at approximately 50 feet. Groundwater encountered at 38 feet during drilling.				
55	16, 17							
60	18, 19							
65	20							
70	21							

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INDIAN VALLEY

LAFAYETTE, CALIFORNIA

BORING NO.: B-1

LOGGED BY: C. Tyler

PROJ. NO.: 5900.1.001.01

CHECKED BY PJS

FIGURE NO. A1

DATE OF BORING: May 29, 2003

SURFACE ELEVATION: Approx. 646 feet (197 meters)

DEPTH (FEET)	DEPTH (METERS)	SAMPLE NUMBER	LOG, LOCATION AND TYPE OF SAMPLE	DESCRIPTION	BLOWS/FT.	qu UNCON STRENGTH (TSF) *FIELD PENET. APPROX.	IN PLACE DRY UNIT WEIGHT (PCF)	IN PLACE MOIST. CONTENT % DRY WEIGHT
0				Seasonal grasses.				
	1	2-1		SILTY CLAY (CH), dark brown, very stiff, moist, trace rock fragments, trace organics.	22	1.75*	104	20.0
5	2	2-2		SILTY CLAY (CL), orange brown, very stiff to hard, moist, some angular to subrounded rock fragments, trace sand.	16	2.50		
10	3	2-3		Same as above, medium stiff.	12	0.75*	106	21.3
15	4, 5	2-4		SILTY CLAY with sand (CL), brown, very stiff, moist, trace rock fragments.	22	1.5*		
20	6	2-5		SILTY CLAY (CL), olive brown, moist, very stiff, trace subrounded rock fragments.	18	1.25*		
25	7, 8	2-6		SILTY CLAY (CL), olive gray, very stiff, moist, trace sand.	20	1.75	115	18.0
30	9	2-7		SILTY CLAY (CL), olive gray, very stiff, moist, some angular to rounded rock fragments.	20	2.0*		
	10			Bottom of boring at approximately 31 feet. Groundwater not encountered during drilling.				
35								

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INDIAN VALLEY

LAFAYETTE, CALIFORNIA

BORING NO.: B-2

LOGGED BY: C. Tyler

PROJ. NO.: 5900.1.001.01

CHECKED BY PJS

FIGURE NO. A2

ENGEO_BORELOG 590010101_INDIANVALLEY.GPJ 7/21/03

DATE OF BORING: May 29, 2003

SURFACE ELEVATION: Approx. 607 feet (185 meters)

DEPTH (FEET)	DEPTH (METERS)	SAMPLE NUMBER	LOG, LOCATION AND TYPE OF SAMPLE	DESCRIPTION	BLOWS/FT.	qu UNCON STRENGTH (TSF) *FIELD PENET. APPROX.	IN PLACE DRY UNIT WEIGHT (PCF)	IN PLACE MOIST. CONTENT % DRY WEIGHT
0								
	1							
5				SILTY CLAY with fine to medium sand (CH), dark yellowish brown, very stiff, very moist.		2.5*		
	2			SILTY CLAY (CL), reddish brown, stiff, very moist, very silty, with medium to coarse sand.		1.7*		
10	3			SANDY CLAY (CL), reddish brown, very stiff, very moist, very silty.				
	4							
15				SILTY CLAY (CL), reddish brown, stiff, with sand and trace gravel, (1 inch maximum dimension, subangular to subrounded).		0.5*		
	5					0.2*		
20	6			With GRAVEL (SC), reddish brown, very wet, possible water.		0.55*		
	7			CLAYEY GRAVEL (GC), reddish brown, wet, fine subangular.				
25	8							
						0.9		
30	9			SILTY CLAY (CL), gray, stiff, very moist, minor sand.				
				Same as above, stone fragments to $^1/_4$ inch maximum dimension common.				
	10					1.8*		
35				Trace black organics.				

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1971 - 2001 * 30 YEARS OF EXCELLENCE

INDIAN VALLEY

LAFAYETTE, CALIFORNIA

BORING NO.: B-3

LOGGED BY: K. Nowell

PROJ. NO.: 5900.1.001.01

CHECKED BY PJS

FIGURE NO. A3

DATE OF BORING: May 29, 2003

SURFACE ELEVATION: Approx. 607 feet (185 meters)

DEPTH (FEET)	DEPTH (METERS)	SAMPLE NUMBER	LOG, LOCATION AND TYPE OF SAMPLE	DESCRIPTION	BLOWS/FT.	qu UNCON STRENGTH (TSF) *FIELD PENET. APPROX.	IN PLACE DRY UNIT WEIGHT (PCF)	IN PLACE MOIST. CONTENT % DRY WEIGHT
35	11			SILTY CLAY with fine sand (CL), medium stiff, very moist.		0.8*		
40	12			SILTY CLAY (CH), grayish brown, stiff, moist, trace sand.		1.3*		
				SILTY CLAY with sand (CL), grayish brown, stiff, moist to very moist.				
	13			SILTY CLAY with trace sand (CH), gray, stiff, moist.		1.7*		
45	14			SILTY CLAY with sand (CL), dark gray, stiff, moist.		1.4		
				CLAYEY GRAVEL (GC), dark gray, very moist, gravels to $^{2}/_{3}$ inch maximum dimension, subangular.				
50	15			SILTY CLAY with sand (CL), dark gray, stiff, moist.		1.5*		
				Bottom of boring at approximately 50 feet, at 12:15. Groundwater not encountered during drilling.				
55	16, 17							
60	18, 19							
65	20							
70	21							

ENGEO INCORPORATED
1971 - 2001 * 30 YEARS OF EXCELLENCE

INDIAN VALLEY

LAFAYETTE, CALIFORNIA

BORING NO.: B-3

LOGGED BY: K. Nowell

PROJ. NO.: 5900.1.001.01

CHECKED BY PJS

FIGURE NO. A3

ENGEO_BORELOG 5900100101_INDIANVALLEY.GPJ 7/21/03

DATE OF BORING: May 30, 2003

SURFACE ELEVATION: Approx. 628 feet (191 meters)

DEPTH (FEET)	DEPTH (METERS)	SAMPLE NUMBER	DESCRIPTION	BLOWS/FT.	qu UNCON STRENGTH (TSF) *FIELD PENET. APPROX.	IN PLACE DRY UNIT WEIGHT (PCF)	IN PLACE MOIST. CONTENT % DRY WEIGHT
0			SILTY CLAY with sand (SC), dark grayish brown, moist, sand is predominantly fine grained.				
		4-1-2 4-1-1	SANDY CLAY (CL), reddish brown, very stiff, moist.	34	3.1*		
	1						
5		4-2-2 4-2-1	Same as above, minor coarse sand, very stiff, very moist.	20	2.1*	102	23.1
	2						
			With abundant-stone fragments to 1 1/2 inch maximum dimension, angular.				
10	3	4-3-2 4-3-1	Same as above, decreasing-stone fragments, loose, fine to coarse sand.	17	0.9*		
	4						
15		4-4		10	0.5*	103	23.1
	5						
20	6	4-5	▽ SANDY CLAY (CL), grayish brown, stiff, with stone fragments, to 1/4 inch maximum dimension common.	8	0.1*		
	7						
25		4-6		20	1.5*	1008	21.0
	8						
30	9	4-7-2 4-7-1	SILTY CLAY with sand (CH), grayish brown, stiff, very moist.	13	1.8*		
			Bottom of boring at approximately 30 feet, at 08:45. Groundwater not encountered during drilling.				
	10						
35							

ENGEO INCORPORATED

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INDIAN VALLEY

LAFAYETTE, CALIFORNIA

BORING NO.: B-4

LOGGED BY: K. Nowell

PROJ. NO.: 5900.1.001.01

CHECKED BY PJS

FIGURE NO. A4

DEPTH (FEET)	DEPTH (METERS)	SAMPLE NUMBER	LOG, LOCATION AND TYPE OF SAMPLE	DATE OF BORING: May 29, 2003 SURFACE ELEVATION: Approx. 572 feet (174 meters) DESCRIPTION	BLOWS/FT.	qu UNCON STRENGTH (TSF) *FIELD PENET. APPROX.	IN PLACE DRY UNIT WEIGHT (PCF)	IN PLACE MOIST. CONTENT % DRY WEIGHT
0				SILTY CLAY with sand (CL), dark grayish brown, stiff, very moist, trace gravel, 1 1/4 inch maximum dimension, subangular.		1.4*		
	1			SANDY CLAY (CL), dark yellowish brown, stiff to very stiff, very moist, with minor weathered sandstone fragments.				
5	2			Same as above, brown.				
10	3							
	4							
15	5			SILTY CLAY (CL), olive brown, very stiff, with siltstone fragments. ▽		0.8		
20	6			SILTSTONE, pale brown, highly weathered, crushed.				
	7			SILTSTONE, dark grayish brown, damp, highly weathered, crushed.				
25	8			SILTSTONE, dark gray, slightly weathered, friable, moderate to widely spaced fracturing. No recovery.				
30	9			Bottom of boring at approximately 30 feet. Groundwater encountered at 18 feet during drilling.				
	10							
35								

ENGEO INCORPORATED
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INDIAN VALLEY

LAFAYETTE, CALIFORNIA

BORING NO.: B-5

LOGGED BY: K. Nowell

PROJ. NO.: 5900.1.001.01

CHECKED BY PJS

FIGURE NO. A5

DATE OF BORING: May 30, 2003

SURFACE ELEVATION: Approx. 543 feet (166 meters)

DEPTH (FEET)	DEPTH (METERS)	SAMPLE NUMBER	DESCRIPTION	BLOWS/FT.	qu UNCON STRENGTH (TSF) *FIELD PENET. APPROX.	IN PLACE DRY UNIT WEIGHT (PCF)	IN PLACE MOIST. CONTENT % DRY WEIGHT
0			SANDY CLAY (CH), dark grayish brown, moist.				
	1	6-1-2 6-1-1	SANDY CLAY (CL), brown, very stiff to very stiff, moist, sand is predominantly fine grained.	26		105	20.2
5		6-2		11	3.2*		
	2		CLAYEY fine to medium SAND (SC), grayish brown, moist to very moist.				
		6-3-2 6-3-1	SILTY CLAY with sand (CL-CH), grayish brown, very stiff, moist to very moist.	22	3.6*		
10	3		Same as above, very stiff, moist.				
		6-4		22	2.5*	101	24.5
			Same as above, very moist, very soft.				
	4		CLAYEY SAND (SC), reddish brown, very moist, sand is fine to coarse.				
		6-5		17	0.1*		
15		6-6	SANDY CLAY to CLAYEY SAND (CL-SC), grayish brown, medium stiff, very moist, fine-grained sand.	7			
	5		Same as above, becoming CLAYEY SAND.				
20	6	6-7-2 6-7-1	CLAYEY GRAVEL (GC), grayish brown, very moist, gravels to 1 $^{1}/_{4}$ inch maximum dimension, subangular to angular.	21		100	25.2
			CLAYEY SAND (SC), grayish brown, very moist, sand is fine to medium grained.				
	7		SILTY CLAY (CH), grayish brown, stiff, very moist, trace fine sand.				
25		6-8		18	1.5*		
	8		Same as above, gray, stiff.				
30	9	6-9		15	1.7*	96	27.9
			Bottom of boring at approximately 30 feet, at 13:10. Groundwater encountered at 22 $^{3}/_{4}$ feet during drilling.				
	10						
35							

ENGEO INCORPORATED

1971 - 2001 * 30 YEARS OF EXCELLENCE

INDIAN VALLEY

LAFAYETTE, CALIFORNIA

BORING NO.: B-6

LOGGED BY: K. Nowell

PROJ. NO.: 5900.1.001.01

CHECKED BY PJS

FIGURE NO. A6

ENGEO_BORELOG 590010010l_INDIANVALLEY.GPJ 7/21/03

DATE OF BORING: May 30, 2003

SURFACE ELEVATION: Approx. 590 feet (180 meters)

DEPTH (FEET)	DEPTH (METERS)	SAMPLE NUMBER	DESCRIPTION	BLOWS/FT.	qu UNCON STRENGTH (TSF) *FIELD PENET. APPROX.	IN PLACE DRY UNIT WEIGHT (PCF)	IN PLACE MOIST. CONTENT % DRY WEIGHT
0			SANDY CLAY (CL), mottled brown/dark grayish brown, slightly moist, hard.				
		7-1		37	+4.5*		
	1						
5		7-2	SANDY CLAY (CL), grayish brown, stiff, sand is fine to coarse grained.	17		96	19.7
	2		Same as above, increase percent clay.				
			Same as above, becoming reddish brown.				
		7-3-2 7-3-1		25			
10	3	7-4		9			
			SILTY CLAY with sand (CL), reddish brown, medium stiff, moist.				
	4						
15		7-5-2 7-5-1	CLAYEY SAND (SC), reddish brown, very moist to wet, sand is predominantly medium to coarse grained.	15			
	5						
			SILTY CLAY with sand (CL-CH), grayish brown, stiff, trace fine subrounded gravel, minor black organic material.				
20	6	7-6-2 7-6-1		20		100	26.0
	7		SILTY CLAY (CL-CH), dark gray, stiff, very moist, with fine to medium sand.				
25		7-7	SANDY CLAY (CL), dark gray, very moist, stiff, fine- to medium-grained sand.	19	1.5*		
	8		SILTY CLAY (CL), dark gray, very moist.				
			Same as above, becoming dark grayish brown.				
			SILTY CLAY (CL), dark grayish brown, very moist, fine- to coarse-grained sand.				
30	9	7-8-2 7-8-1	SILTY CLAY with sand (CL), dark grayish brown, stiff, very moist, trace black (organic?) material.	19	1.7*	94	30.2
			Bottom of boring at approximately 30 feet, at 10:45. Groundwater not encountered during drilling.				
	10						
35							

ENGEO INCORPORATED
1971 - 2001 * 30 YEARS OF EXCELLENCE

INDIAN VALLEY

LAFAYETTE, CALIFORNIA

BORING NO.: B-7

LOGGED BY: K. Nowell

PROJ. NO.: 5900.1.001.01

CHECKED BY PJS

FIGURE NO. A7

<u>TEST PIT LOGS</u>

Test Pit Number	Depth (Feet)	Description
TP-1	0 – 2.5	SILTY CLAY, brown, medium stiff, moist.
	2.5 – 7	SANDSTONE, gray/green, moderately weathered, unfractured, friable to weak, medium to coarse-grained with fine gravel, angular to subrounded.
		Bottom of test pit at 7 feet. No free groundwater encountered.
TP-2	0 – 2.5	SILTY CLAY, dark brown, medium stiff, moist.
	2.5 – 10	SILTY CLAY, brown, medium stiff, very moist, sand to gravel-size lithic fragments (Qis).
	10 – 17	SILTY CLAY, red/brown, medium stiff to stiff, very moist, gravel to cobble-size fragments of volcanic rock (Qis).
		Bottom of test pit at 17 feet. No free groundwater encountered.
TP-3	0 – 2.5	SILTY CLAY, dark brown, medium stiff, moist, some small lithic fragments.
	2.5 –10	SILTY CLAY, dark brown, medium stiff, moist, with coarse gravel to fine boulder lithic fragments of blue/gray volcanic rocks (Qaft).
	10 – 17	SILTY CLAY, brown, mottled, medium stiff, moist, some sand-size lithics (Qaft).
		Bottom of test pt at 17 feet. No free groundwater encountered.

TEST PIT LOGS

Test Pit Number	Depth (Feet)	Description
TP-4	0 – 2.5	SILTY CLAY, dark brown, medium stiff, moist.
	2.5 – 6	SILTY CLAY, brown, stiff, moist, cobble to fine boulder volcanic fragments (Qafd). Sample TP-4-1 at 5 feet.
	6 – 17	SILTY CLAY, red/brown, medium stiff to stiff, very moist, gravel to cobble-size fragments of volcanic rock. Bottom of test pit at 17 feet. No free groundwater encountered.
TP-5	0 – 7	SILTY CLAY, red/ brown, medium stiff to stiff, very moist, gravel to cobble-size fragments of volcanic rock (Qc).
	7 – 8.5	BASALT, red/brown, gray on fresh surface, highly weathered, highly fractured, moderately strong to strong.
TP-6	0 – 2.5	SILTY CLAY, brown, medium stiff, moist.
	2.5- 10	SILTY CLAY, red/brown, medium stiff to stiff, very moist, gravel to boulder-size fragments of volcanic rock, boulders up to 1.5 feet to 3 feet (Qc).
	10 – 17	SILTY CLAY, red/brown, medium stiff, very moist, sand to gravel-size fragments (Qc). Bottom of test pit at 17 feet. No free groundwater encountered.

TEST PIT LOGS

Test Pit Number	Depth (Feet)	Description
TP-7	0 – 17	SILTY CLAY, brown, medium stiff, very moist, sand to gravel-size lithic fragments (Qafd).
		Bottom of test pit at 17 feet. No free groundwater encountered.
TP-8	0 – 1.5	SILTY CLAY, brown, stiff, moist.
	1.5- 7	INTERBEDDED SILTSTONE, SANDSTONE, and CLAYSTONE, light yellow-brown to red brown and blue gray, thinly bedded, highly weathered, highly fractured, friable to weak. BEDDING N25E 23SE.
		Bottom of test pit at 7 feet. No free groundwater encountered.
TP-9	0 – 2.5	SILTY CLAY, dark brown, medium stiff, moist.
	2.5 – 17	SILTY CLAY, brown, medium stiff, moist, small lithic fragments. Sample TP-9-1 at 3.5 feet.
		Bottom of test pit at 17 feet. No free groundwater encountered.
TP-10	0 – 4	SILTY CLAY, dark brown, medium stiff, moist, some sand to gravel lithic fragments. Sample TP-10-1 at 1 foot.
	4 – 17	SILTY CLAY, brown, medium stiff, moist to wet, soft to medium stiff, sand to fine cobble lithic fragments (Qafd).
		Bottom of test pit at 17 feet. No free groundwater encountered.

TEST PIT LOGS

Test Pit Number	Depth (Feet)	Description
TP-11	0 – 3	SILTY CLAY, dark brown, medium stiff, moist.
	3 – 18	SILTY CLAY, brown, medium stiff, moist to wet, sand to fine gravel sandstone lithic fragments, seep at 7.5 feet (Qafd).
		Bottom of test pit at 18 feet. No free groundwater encountered.
TP-12	0 – 5	SILTY CLAY, dark brown, medium stiff, moist, some small lithic fragments (Qafd).
	5 – 8	SILTY CLAY, brown, stiff, moist (Qc). Sample TP-12-1 at 7 feet.
	8 – 11	INTERBEDDED SANDSTONE, and CLAYSTONE, light brown and light gray, highly weathered, highly fractured, weak to moderately strong.
		Bottom of test pit at 11 feet. No free groundwater encountered.
TP-13	0 – 3.5	SILTY CLAY, dark brown, medium stiff, moist, some sand to cobble-size lithic fragments.
	3.5 – 16	SILTY CLAY, dark brown, medium stiff, moist, sand to cobble-size lithic fragments in layers sub-parallel to the slope, percentage of larger fragments and fragment size increases with depth (Qafd). Seep at 8 feet.
	16 – 18.5	SILTY CLAY, brown to gray, stiff, moist, sand to cobble-size lithic fragments, flat dull surface with rots at 18 feet (Qafd).
		Bottom of test pit at 18.5 feet. No free groundwater encountered.

TEST PIT LOGS

Test Pit Number	Depth (Feet)	Description
TP-14	0 – 2.5	SILTY CLAY, brown, medium stiff, moist.
	2.5 – 6.5	SILTSTONE, gray, red/brown staining on fractures, highly weathered, highly fractured, friable to weak, some carbonates in fractures.
		Bottom of test pit at 6.5 feet. No free groundwater encountered.
TP-15	0 – 2.5	SILTY CLAY, dark brown, soft to medium stiff, moist.
	2.5 – 17	SILTY CLAY, brown, medium stiff, moist, sand to cobble-size lithic fragments (Qafd).
		Bottom of test pit at 17 feet. No free groundwater encountered.
TP-16	0 – 18	Alternating 6-inch to 18-inch-thick layers sub-parallel to the slope of A) SILTY CLAY, dark red/brown, medium stiff, moist, abundant sand to gravel-size lithic fragments, few cobble fragments (Qafd). B) SILTY CLAY, red/brown, medium stiff, moist, abundant sand to coarse cobble lithic fragments.
		Bottom of test pit at 18 feet. No free groundwater encountered.
TP-17	0 – 10	SILTY CLAY, brown, medium stiff, moist, sand to gravel-size lithic fragments with few cobbles (Qafd).
	10 – 18	CLAYEY SILT, light red/brown, medium stiff, moist, abundant sand to cobble lithic fragments (Qafd).
		Bottom of test pit at 18 feet.

Test Pit Number	Depth (Feet)	Description
		No free groundwater encountered.
TP-18	0 – 2.5	SILTY CLAY, dark brown, medium stiff, moist, some sand to gravel-size lithic fragments (Qafd).
	2.5 – 17	SILTY CLAY, brown, medium stiff, moist, sand to fine cobble lithic fragments (Qafd).
		Bottom of test pit at 17 feet. No free groundwater encountered.

TEST PIT LOGS

Test Pit Number	Depth (Feet)	Description
TP-19	0 – 2	SILTY CLAY, dark brown, medium stiff, moist, high plasticity.
	2 – 4.5	SILTSTONE/CLAYSTONE, red/brown and gray, very highly weathered, highly fractured, friable.
	4.5 – 7	SILTSTONE/CLAYSTONE, red/brown and gray, very highly weathered, highly fractured, weak.
		Bottom of test pit at 7 feet. No free groundwater encountered.
TP-20	0 – 2.5	SILTY CLAY, dark brown, medium stiff, moist.
	2.5 – 8.5	SILTY CLAY, gray/brown, medium stiff, moist, sand to pebble-size lithic fragments, well rounded. Eight-foot to 8.5-foot increase in pebble content, matrix supported. Seep at 8 feet. Sample TP-20-1 at 6 feet.
		Bottom of test pit at 17 feet. No free groundwater encountered.
	8.5 – 11	SILTY CLAY, red/brown and gray, stiff, wet.
		Bottom of test pit at 11 feet. No free groundwater encountered.
TP-21	0 – 0.5	SANDY SILT, brown, medium stiff, moist.
	0.5 – 4	SANDSTONE, moderately weathered, moderately fractured, thinly bedded, weak to moderately strong. BEDDING-NO5W 52NE.

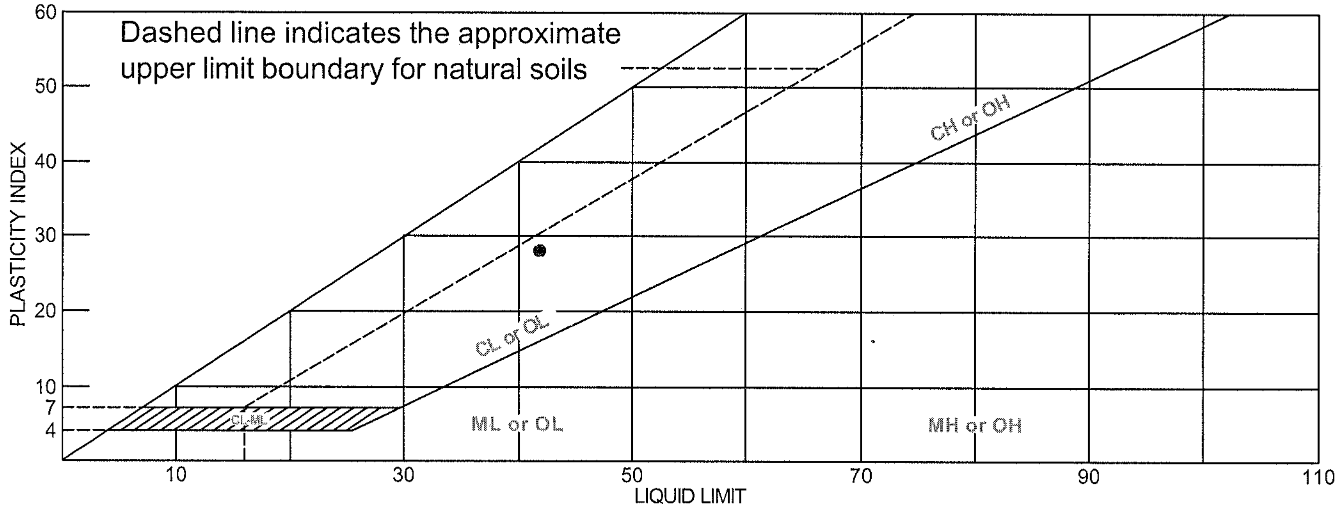
TEST PIT LOGS

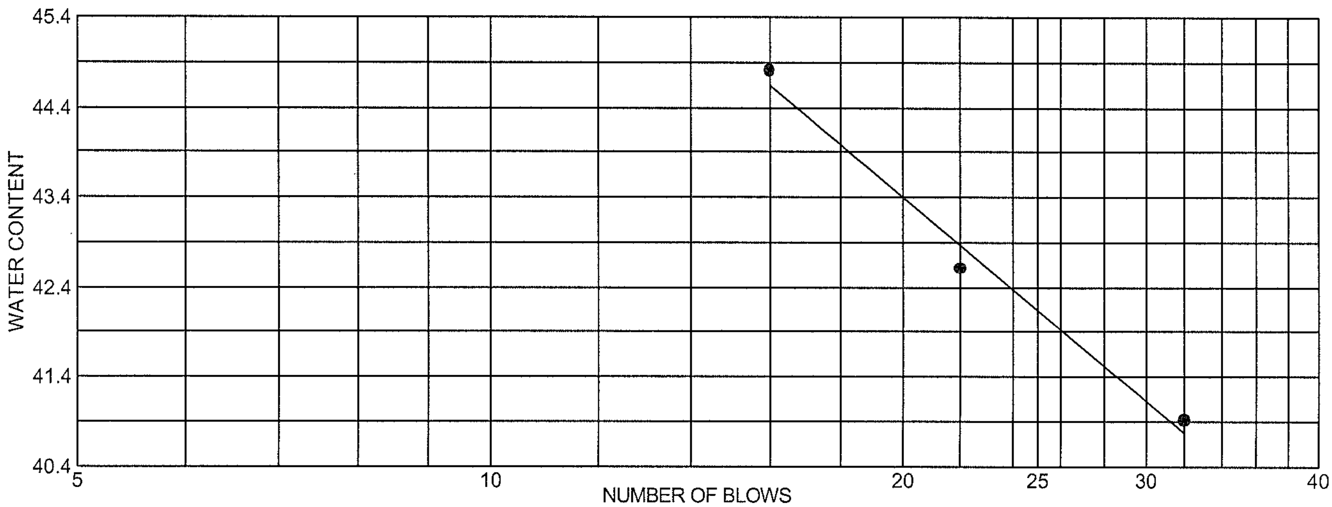
Test Pit Number	Depth (Feet)	Description
TP-22	0 – 3	SILTY CLAY, dark brown, stiff, moist.
	3 – 12	SILTY CLAY, brown, stiff, very moist, some shearing, few sand-size lithic fragment (Qls).
		Bottom of test pit at 12 feet. No free groundwater encountered.
TP-23	0 – 2.5	SILTY CLAY, dark brown, medium stiff, moist.
	2.5 – 11	SILTY CLAY, brown, stiff, very moist, few sand-size lithic fragments, no shearing (Qafd). Sample TP-23-1 at 6 feet.
		Bottom of test pit at 11 feet. No free groundwater encountered.
TP-24	0 – 4.5	SILTY CLAY, dark brown, stiff, moist, few sand to pebble-size lithic fragments.
	4.5 – 12	SILTY CLAY, brown, medium stiff, very moist, few sand to gravel-size lithic fragments, some shearing (Qls).
		Bottom of test pit at 12 feet. No free groundwater encountered.
TP-25	0 – 4	SILTY CLAY, dark brown, medium stiff, moist.
	4 – 11	SILTY CLAY, brown, stiff, very moist, few sand-size lithic fragments, no shearing (Qafd).
		Bottom of test pit at 11 feet. No free groundwater encountered.

TEST PIT LOGS

Test Pit Number	Depth (Feet)	Description
TP-26	0 – 2.5	SILTY CLAY, dark brown, medium stiff, moist.
	2.5 – 12	SILTY CLAY, brown, stiff, very moist, some sand to gravel-size lithic fragments, some shearing (Qafd).
		Bottom of test pit at 12 feet. No free groundwater encountered.
TP-27	0 – 3	SILTY CLAY, dark brown, medium stiff, moist.
	3 – 12	SILTY CLAY, brown, stiff, very moist, some sand to gravel-size lithic fragments, some shearing (Qafd). Sample TP-27-1 at 5 feet.
		Bottom of test pit at 12 feet. No free groundwater encountered.

LIQUID AND PLASTIC LIMITS TEST REPORT





	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	Dark brown sandy silty Clay	42	14	28		56.1	CL

Project No. 5900.1.001.01 **Client:**

Project: Indian Valley. Moraga, California

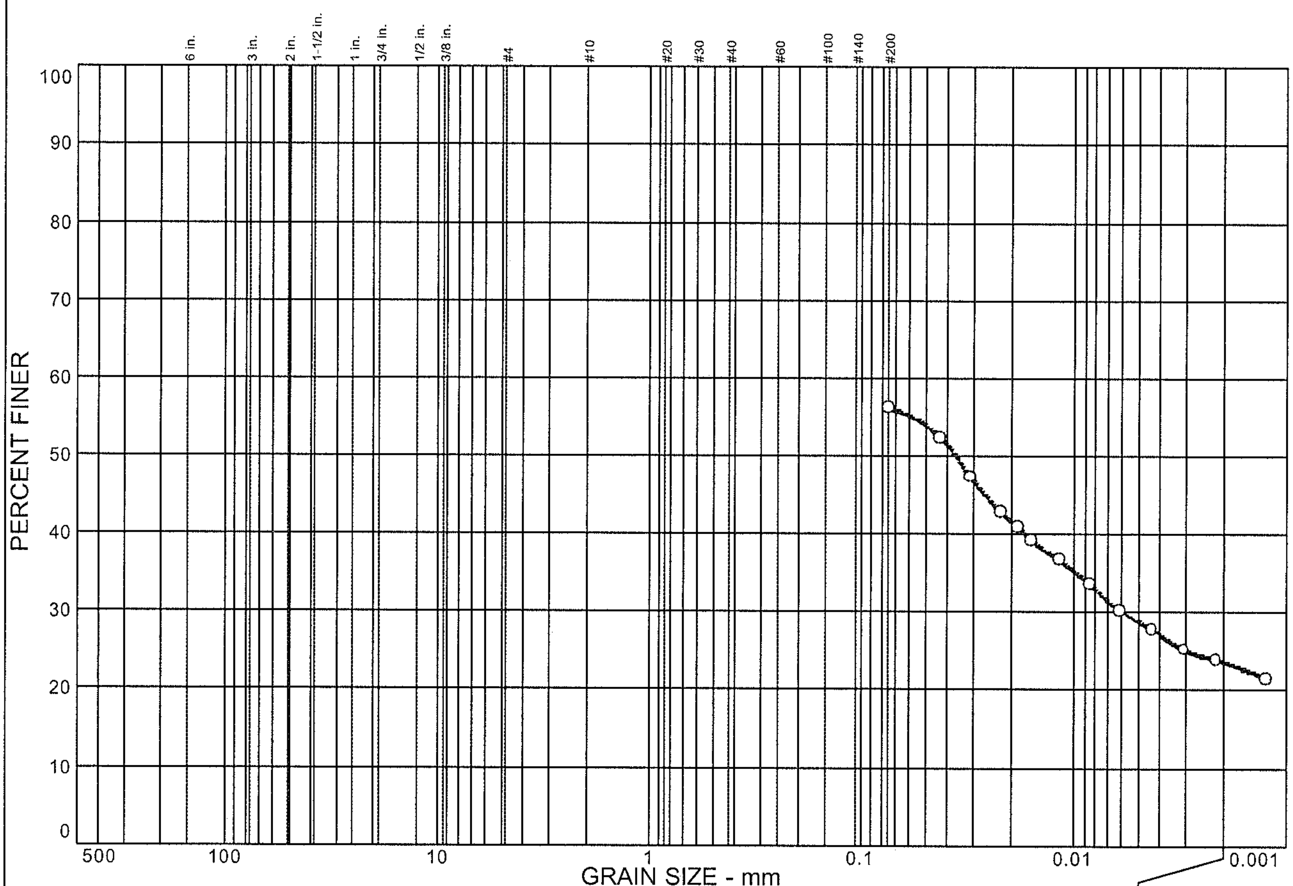
● **Source:** PI/Hy **Sample No.:** 1-1

Remarks:

● 1-1 (3 to 3.5 ft.)

ENGEO INCORPORATED — GEOTECHNICAL AND ENVIRONMENTAL CONSULTANTS MATERIALS TESTING

Particle Size Distribution Report



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
			32.5	23.6

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	56.1		

* (no specification provided)

Soil Description

Dark brown sandy silty Clay

Atterberg Limits

PL= 14 LL= 42 PI= 28

Coefficients

D_{85}= D_{60}= D_{50}= 0.0364
D_{30}= 0.0059 D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= CL AASHTO=

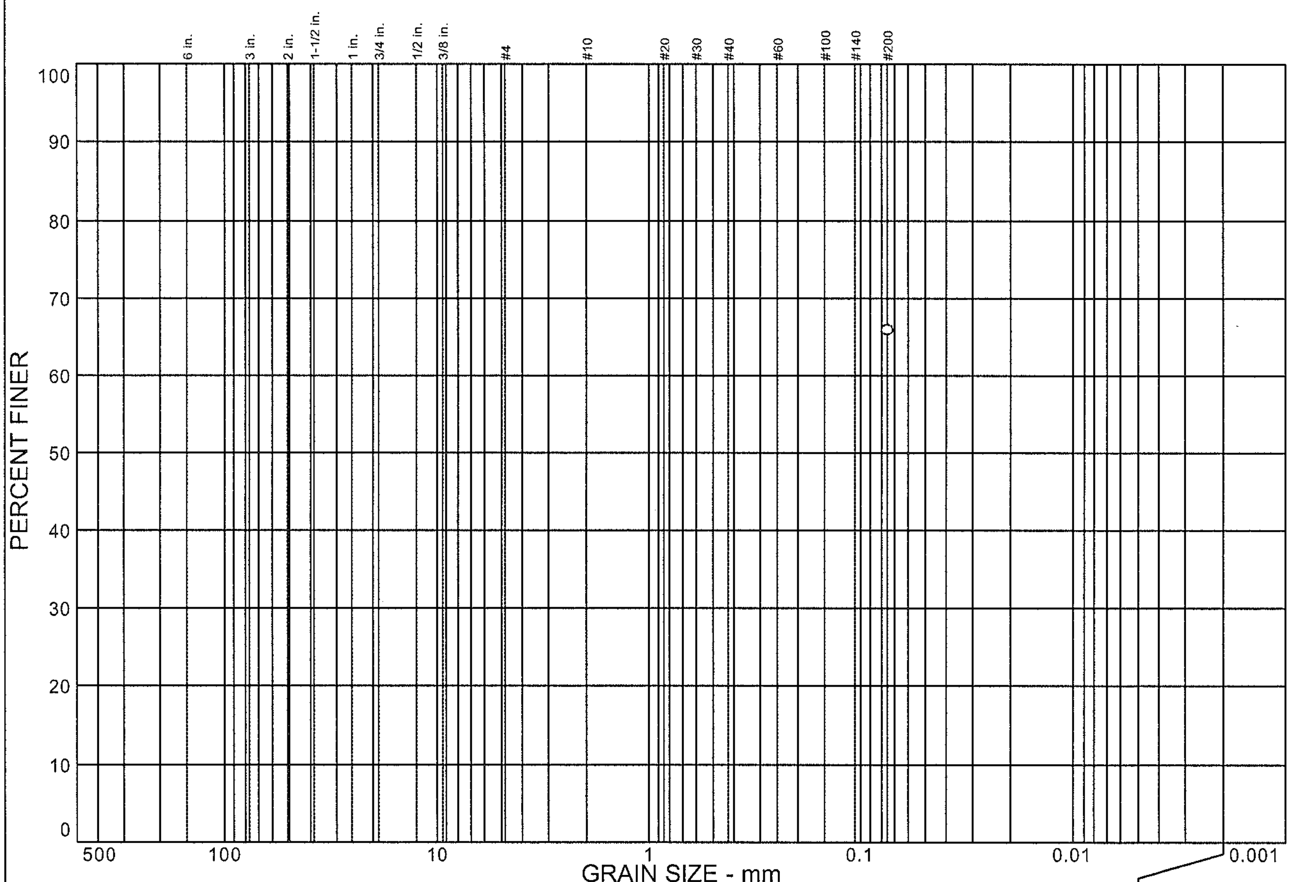
Remarks

Sample No.: 1-1 **Source of Sample:** PI/Hy **Date:** 06-07-03
Location: **Elev./Depth:** 3 to 3.5 ft.

ENGEO INCORPORATED GEOTECHNICAL AND ENVIRONMENTAL CONSULTANTS MATERIALS TESTING

Client:
Project: Indian Valley. Moraga, California
Project No: 5900.1.001.01

Particle Size Distribution Report



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
			65.8	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	65.8		

* (no specification provided)

Soil Description

Dark olive gray sandy silty Clay

Atterberg Limits

PL= LL= PI=

Coefficients

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

Classification

USCS= AASHTO=

Remarks

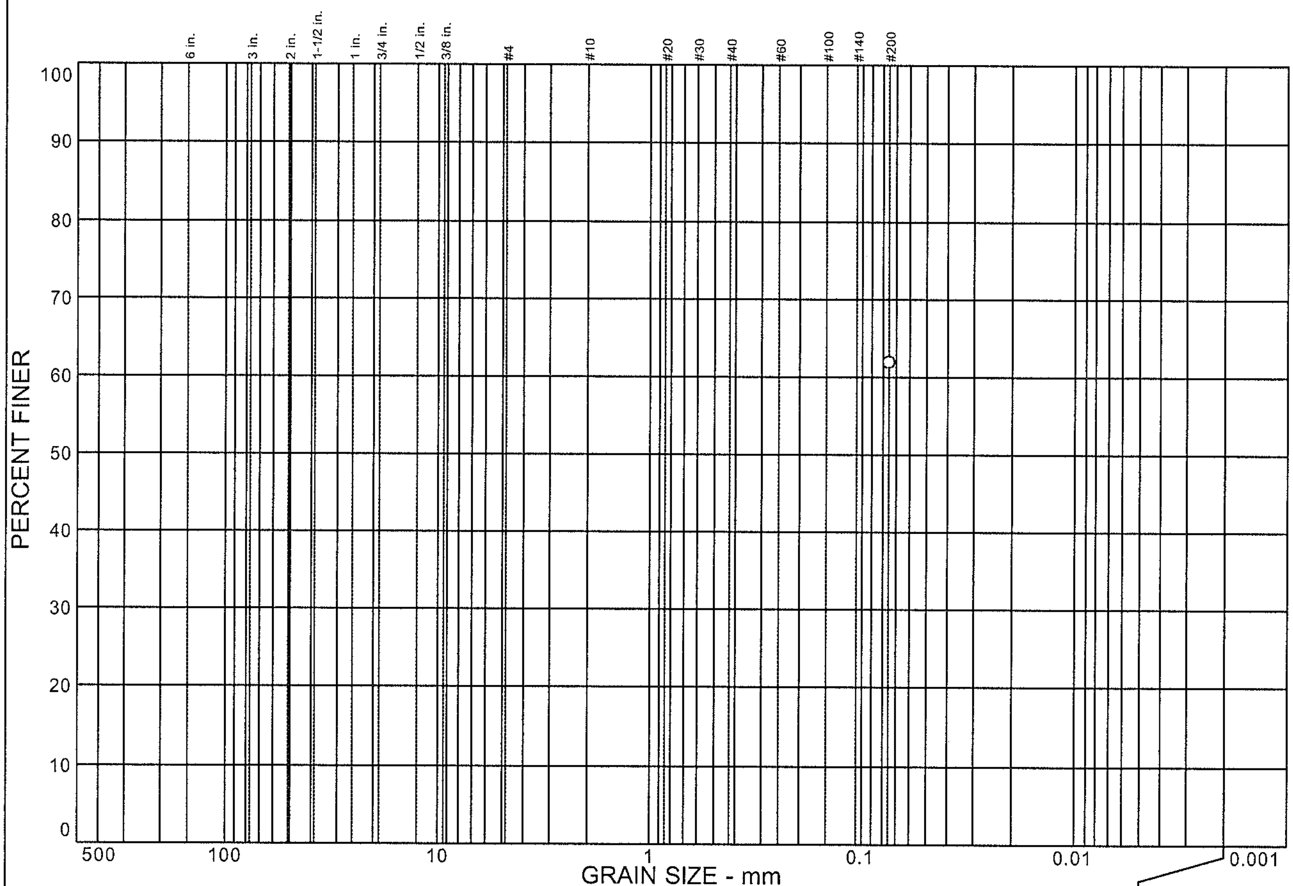
Sample No.: 7-7 **Source of Sample:** %200 **Date:** 06-07-03
Location: **Elev./Depth:** 24.5 to 25 ft.

ENGEO INCORPORATED GEOTECHNICAL AND ENVIRONMENTAL CONSULTANTS MATERIALS TESTING

Client:
Project: Indian Valley. Moraga, California

Project No: 5900.1.001.01

Particle Size Distribution Report



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
			61.7	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	61.7		

* (no specification provided)

Soil Description

Dark grayish brown sandy silty Clay

Atterberg Limits

PL= LL= PI=

Coefficients

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

Classification

USCS= CL AASHTO=

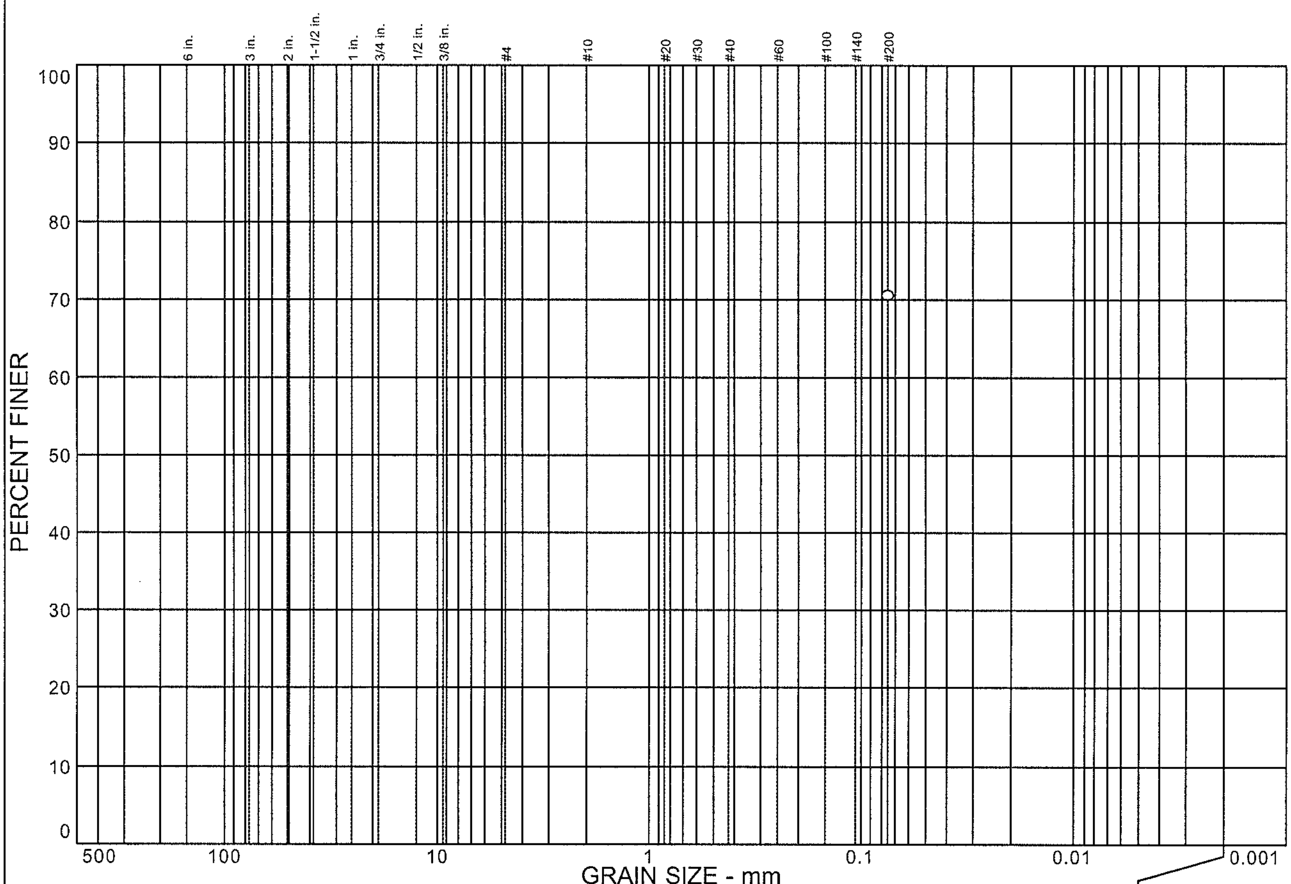
Remarks

Sample No.: 4-5 **Source of Sample:** %200 **Date:** 6-7-03
Location: **Elev./Depth:** 19.5 to 20 ft.

ENGEO INCORPORATED GEOTECHNICAL AND ENVIRONMENTAL CONSULTANTS MATERIALS TESTING

Client:
Project: Indian Valley. Moraga, California
Project No: 5900.1.001.01

Particle Size Distribution Report



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
			70.4	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	70.4		

* (no specification provided)

Soil Description

Dark yellowish brown sandy silty Clay

Atterberg Limits

PL= LL= PI=

Coefficients

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

Classification

USCS= CL AASHTO=

Remarks

Sample No.: 7-4 **Source of Sample:** %200 **Date:** 06-07-03
Location: **Elev./Depth:** 9.5 to 10 ft.

ENGEO INCORPORATED GEOTECHNICAL AND ENVIRONMENTAL CONSULTANTS MATERIALS TESTING

Client:
Project: Indian Valley. Moraga, California
Project No: 5900.1.001.01

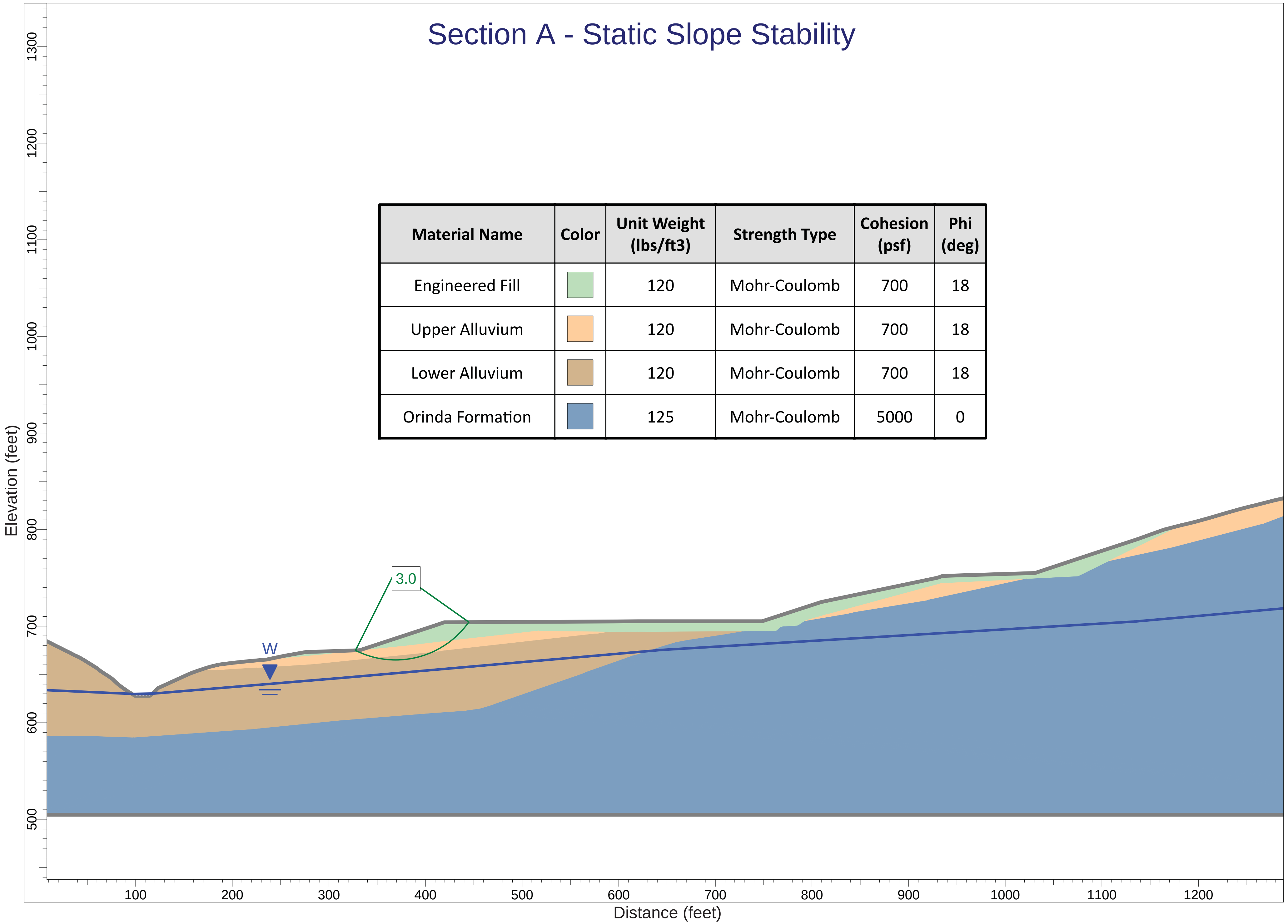
APPENDIX D

Slope Stability Analysis

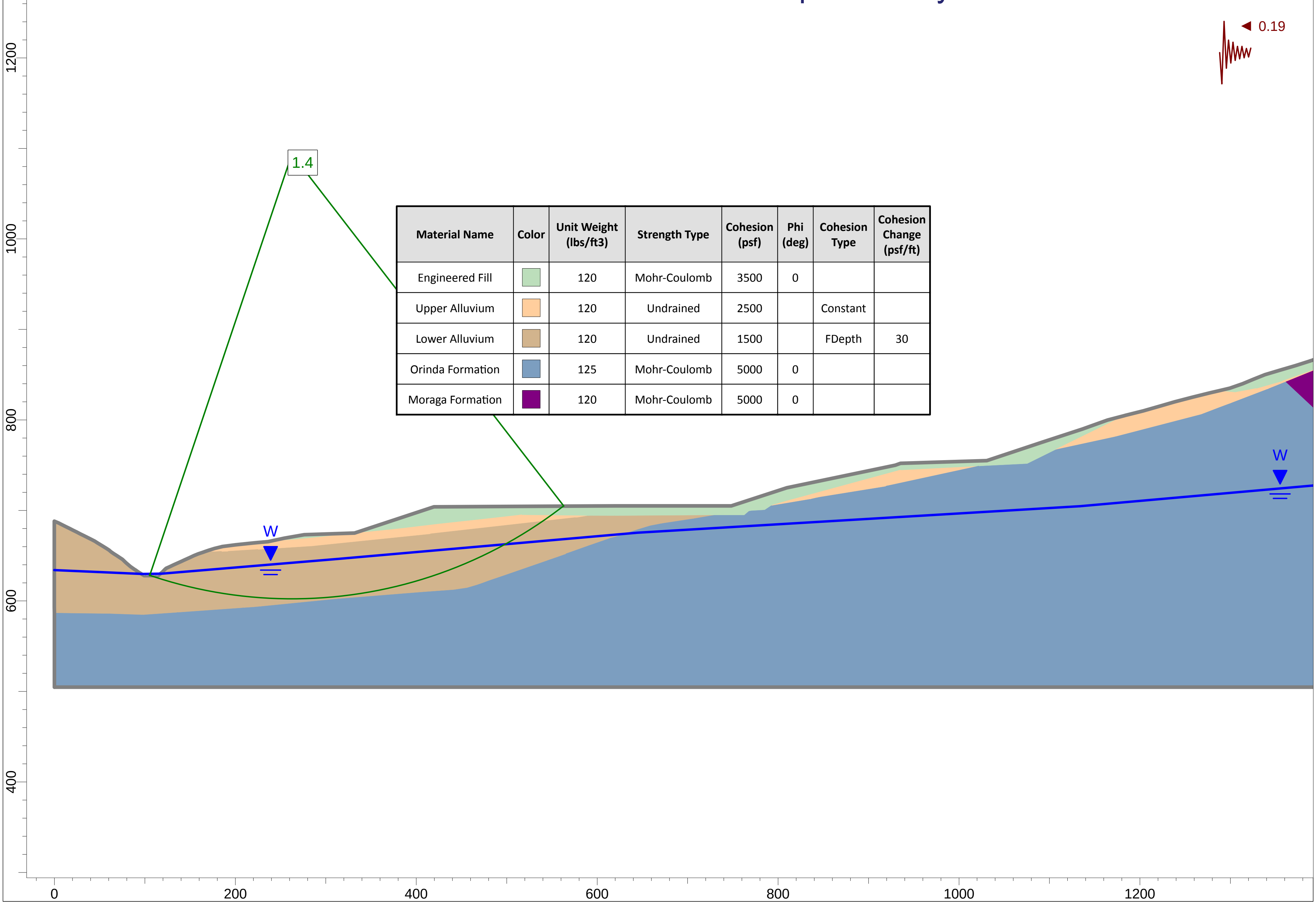
APPENDIX D

Section A - Static Slope Stability

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Engineered Fill		120	Mohr-Coulomb	700	18
Upper Alluvium		120	Mohr-Coulomb	700	18
Lower Alluvium		120	Mohr-Coulomb	700	18
Orinda Formation		125	Mohr-Coulomb	5000	0



Section A - Pseudo-Static Slope Stability



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Cohesion Type	Cohesion Change (psf/ft)
Engineered Fill		120	Mohr-Coulomb	3500	0		
Upper Alluvium		120	Undrained	2500		Constant	
Lower Alluvium		120	Undrained	1500		FDepth	30
Orinda Formation		125	Mohr-Coulomb	5000	0		
Moraga Formation		120	Mohr-Coulomb	5000	0		