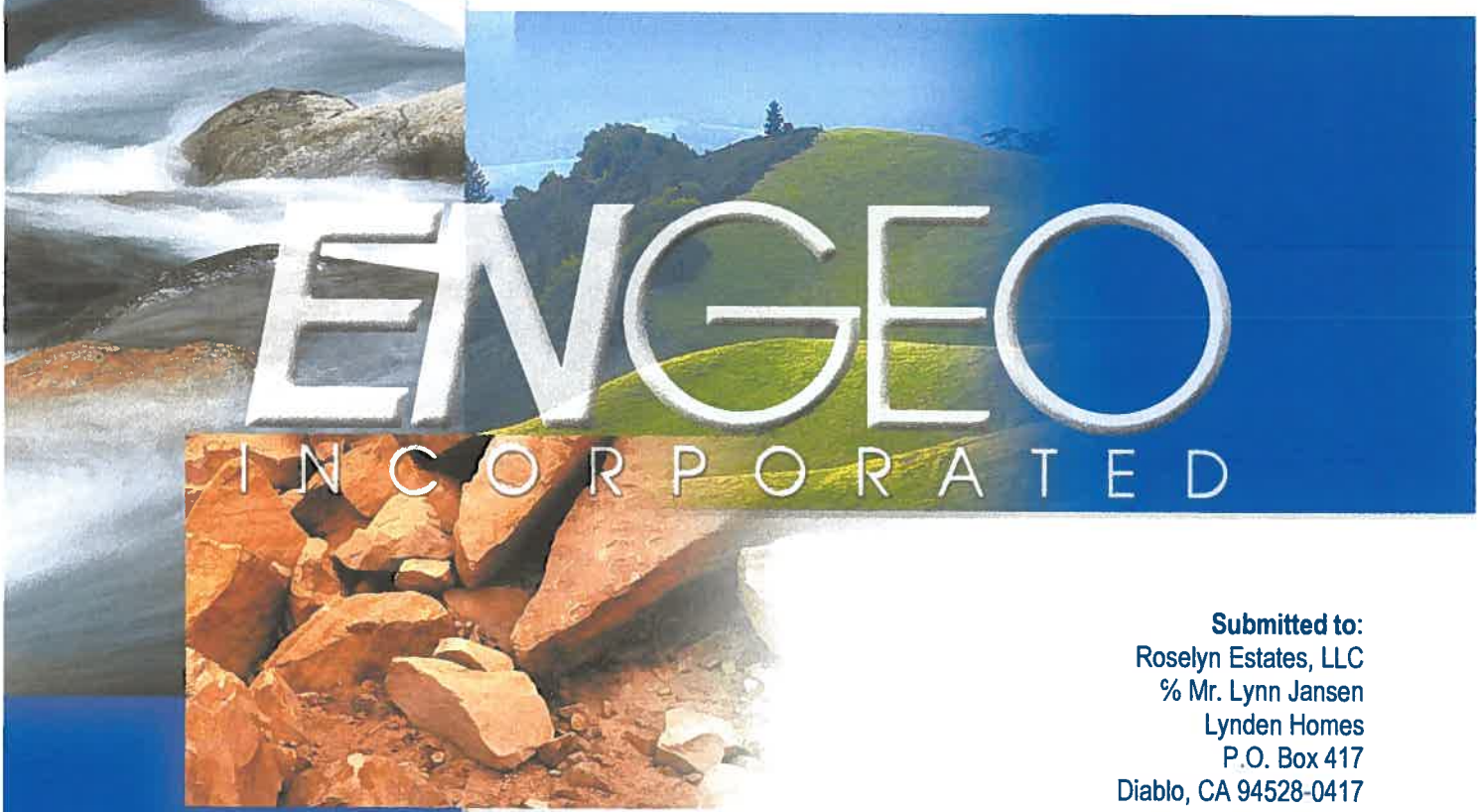
# GEOTECHNICAL EXPLORATION

## ROSELYN ESTATES II PROPERTY

### PLEASANTON, CALIFORNIA




**Submitted to:**
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**Prepared by:**
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**August 1, 2012**

**Project No:**
4425.000.000

**PUD-94**
**EXHIBIT B**

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Project No.
**4425.000.000**

August 1, 2012

Roselyn Estates, LLC
% Mr. Lynn Jansen
Lynden Homes
P.O. Box 417
Diablo, CA 94528-0417

Subject: Roselyn Estates II Property
1623 Cindy Way – Parcel B
Pleasanton, California

## GEOTECHNICAL EXPLORATION

Dear Mr. Jansen:

ENGEO prepared this geotechnical report for proposed residential development as outlined in our agreement dated November 3, 2011. We characterized the subsurface conditions at the site to provide the enclosed geotechnical recommendations for design.

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

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

Sincerely,

ENGEO Incorporated

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# TABLE OF CONTENTS

Letter of Transmittal

1.0 INTRODUCTION ..... 1
1.1 PURPOSE AND SCOPE ..... 1
1.2 PROJECT LOCATION ..... 1
1.3 PROPOSED DEVELOPEMENT ..... 2
1.4 PREVIOUS EXPLORATION ..... 2

2.0 FINDINGS ..... 2
2.1 FIELD EXPLORATION ..... 2
2.2 SITE BACKGROUND ..... 3
2.3 GEOLOGY AND SEISMICITY ..... 3
2.3.1 Regional and Site Geology ..... 3
2.3.2 Seismicity ..... 3
2.4 SURFACE CONDITIONS ..... 4
2.5 SUBSURFACE CONDITIONS ..... 4
2.6 GROUNDWATER CONDITIONS ..... 4
2.7 LABORATORY TESTING ..... 5
2.8 CREEK BANK STABILITY ANALYSES ..... 5
2.8.1 Method of Analysis ..... 5
2.8.2 Acceptable Factors of Safety ..... 6
2.8.3 Geometry and Idealized Soil Profiles ..... 6
2.8.3.1 Rapid Drawdown ..... 6
2.8.4 Results of Analyses ..... 7

3.0 CONCLUSIONS ..... 8
3.1 EXISTING FILL ..... 8
3.2 CREEK BANK STABILITY ..... 8
3.3 EXPANSIVE SOIL ..... 9
3.4 SEISMIC HAZARDS ..... 9
3.4.1 Ground Rupture ..... 9
3.4.2 Ground Shaking ..... 10
3.4.3 Liquefaction ..... 10
3.4.4 Lateral Spreading ..... 11
3.4.5 Seismic-Induced Settlement Analyses ..... 11
3.4.5.1 Liquefaction ..... 11
3.4.5.2 Dynamic Densification ..... 11
3.4.6 Ground Lurching ..... 12
3.4.7 Flooding ..... 12
3.5 SOIL CORROSION POTENTIAL ..... 12
3.6 2010 CBC SEISMIC DESIGN PARAMETERS ..... 14

4.0 CONSTRUCTION MONITORING ..... 14

5.0 SLOPE SETBACK RECOMMENDATIONS ..... 15
5.1 BIORENTENTION AREA ..... 15

## TABLE OF CONTENTS (Continued)

**6.0 EARTHWORK RECOMMENDATIONS ..... 15**
**6.1 EXISTING FILL REMOVAL ..... 16**
**6.2 GENERAL SITE CLEARING ..... 16**
**6.3 SURFICIAL SOIL REMOVAL ..... 16**
**6.4 ACCEPTABLE FILL ..... 17**
**6.5 FILL COMPACTION ..... 17**
6.5.1 Underground Utility Backfill ..... 18
6.5.1.1 General ..... 18
6.5.1.2 Structural Areas ..... 18
6.5.2 Landscape Fill ..... 19
**6.6 SLOPES ..... 19**
6.6.1 Gradients ..... 19
**6.7 SITE DRAINAGE ..... 19**
6.7.1 Surface Drainage ..... 19
6.7.2 Subsurface Drainage ..... 19
**6.8 LANDSCAPING CONSIDERATIONS ..... 19**

**7.0 FOUNDATION RECOMMENDATIONS ..... 20**
**7.1 POST-TENSIONED MAT FOUNDATIONS ..... 20**
7.1.1 Subgrade Treatment ..... 21
7.1.2 Slab Moisture Vapor Reduction ..... 21
7.1.3 Settlement ..... 21

**8.0 SLABS-ON-GRADE ..... 22**
**8.1 PRELIMINARY PAVEMENT DESIGN ..... 22**
**8.2 CUT-OFF CURBS ..... 23**
**8.3 RESIDENTIAL DRIVEWAYS/GARAGE SLABS ..... 23**

**9.0 LIMITATIONS AND UNIFORMITY OF CONDITIONS ..... 23**

**REFERENCES**

**FIGURES**
Figure 1 – Vicinity Map
Figure 2 – Site Plan
Figure 3 – Regional Geologic Map (Dibblee)
Figure 4 – Regional Faulting and Seismicity Map
Figure 5 – Seismic Hazard Map, Dublin Quadrangle, CGS 2008
Figure 6 – Typical Geogrid Reinforced Section

**APPENDIX A** – Exploration Logs
**APPENDIX B** – Laboratory Test Data
**APPENDIX C** – Slope Stability Analysis
**APPENDIX D** – Previous Exploration Boring Logs

# 1.0 INTRODUCTION

## 1.1 PURPOSE AND SCOPE

The purpose of this geotechnical report is to evaluate the geologic conditions of the site and to provide geotechnical conclusions and recommendations for the grading and foundation design for the proposed single-family home development at the Roslyn Estates II Property in Pleasanton, California.

The scope of our services included:

- Reviewing available literature, geologic maps, and previous geotechnical reports pertinent to the site.
- Drilling four exploratory borings at within the site.
- Sampling and laboratory testing of subsurface materials during exploration.
- Geologic mapping of the creek banks adjacent to the site.
- Analyzing the geotechnical field and laboratory data.
- Reporting our findings and recommendations.

This report was prepared for the exclusive use of Roselyn Estates LLC and its design team consultants. In the event that any changes are made in the character, design or layout of the development, the conclusions and recommendations contained in this report should be reviewed by ENGEO Incorporated to determine whether modifications to the report are necessary. This document may not be reproduced in whole or in part by any means whatsoever, nor may it be quoted or excerpted without the express written consent of ENGEO Incorporated.

## 1.2 PROJECT LOCATION

The Roselyn Estates II Property is located east of Cindy Way and Lynn Drive, north of Rose Avenue and the Alameda County Fairgrounds, and south of Arroyo Del Valle Creek as shown on the Vicinity Map, Figure 1. The property contained a residence with several detached buildings, a pool with associated flat work surrounded by a small orchard on the northwestern portion of the property. The eastern portion of the property is vacant. Recently a residence was moved from the center of the property to a lot with driveway access from Cindy Way. The residence utilized a well and septic system; to our knowledge they are still present but are no longer in use.

According to the Jones House Relocation Plan prepared by Debolt Civil Engineering dated February 15, 2011, the site topography is relatively level, ranging in elevation of approximately 335 to 338 feet above mean sea level (msl) (DeBolt, 2011). During construction of the foundation for the relocated home, material was borrowed from the center-west side of the

property creating a shallow depression in this area. The creek bank is approximately 30 feet high and with the upper approximately 15 feet at or near vertical at several locations. The creek bank and slopes contained a significant amount of accumulated debris, including metal siding and/or roofing panels, concrete, lumber and wood fencing, metal fencing, and dead vegetation.

### 1.3 PROPOSED DEVELOPMENT

According to the Possible Lot Configuration Plan developed by DeBolt Civil Engineering dated February 15, 2011, the property is to be subdivided into seven lots approximately 10,000 square feet in size. Each parcel will have a one-or two-story single-family home (Figure 2). Lynn Drive will be extended adjacent to Arroyo Del Valle Creek and connect to Calico Drive. The seven parcels will be accessed via a hammerhead on Lynn Drive. A 12-foot-wide bioretention area and 8-foot-wide trail are proposed between Lynn Drive and the existing creek bank. The current site plan depicts Lynn Drive and all non-habitable improvements setback from a 2:1 (horizontal:vertical) line of projection from the toe of the creek bank Habitable improvements are setback from a 3:1 line of projection plus 15 horizontal feet from the toe of the creek bank.

We anticipate that the site grading will consist of minor cutting and filling to establish relatively level building pads.

### 1.4 PREVIOUS EXPLORATION

ENGEO previously performed a geotechnical feasibility study dated February 18, 1998, on the adjacent properties on Cindy Way. In this study, we advanced four borings on January 16, 1998, to a depth of approximately 39.5 feet (see boring logs in Appendix D). The subsurface soils generally consisted of clayey silt to silty clay, intermixed with various percentages of sand and fine gravel. Thinly bedded layers of silty and clayey sand or fine gravel were also encountered in Borings 1, 3 and 4 at various elevations. Groundwater was encountered in Boring 1 at a depth of approximately 31 feet below the existing ground elevation.

## 2.0 FINDINGS

### 2.1 FIELD EXPLORATION

Our field exploration included drilling four borings on December 15 and 16, 2011. The approximate boring locations are shown on the Site Plan, Figure 2. Borings 1-B1 and 1-B2 were drilled to depths of approximately 51½ feet. Borings 1-B3 and 1-B4 were drilled to depths of approximately 21½ feet below existing grade. The exploratory borings were approximately located by visual sighting from existing features and the locations should be considered accurately located only to the degree implied by the method used.

Exploratory borings were drilled using a truck-mounted drill rig equipped with 8-inch-diameter hollow-stem augers for Borings 1-B1 and 1-B2 and 6-inch-diameter hollow stem augers for Borings 1-B3 and 1-B4. Soil samples recovered during drilling were retrieved using a 3-inch

outside-diameter (O.D.) California-type split-spoon sampler fitted with 6-inch-long brass liners, or a 2-inch O.D. Standard Penetration Test (SPT) split-spoon sampler. The split-spoon samplers were driven with a 140-pound auto-safety hammer falling a distance of 30 inches. The penetration of the sampler was field recorded as the number of blows needed to drive the sampler 18 inches in 6-inch increments. The boring logs show the number of blows required for the last one foot of penetration, and the blow counts have not been converted using any correction factors.

An ENGEO Geologist logged the boreholes in the field. The field logs were then used to develop the report boring logs in Appendix A. The logs depict subsurface conditions within the boring for the date of drilling; however, subsurface conditions may vary with time.

## 2.2 SITE BACKGROUND

Aerial photographs were studied in previous studies of the project site. The photographs revealed that a pervious residential structure was constructed prior to the 1950s. The area surrounding the residence consisted of walnut orchards until the late 1970s. In 2011, the residential structure was relocated approximately 20 feet north. The pool, old house foundation, and several out buildings were present at the time of our investigation. The location of the existing pool is shown on Figure 2.

## 2.3 GEOLOGY AND SEISMICITY

### 2.3.1 Regional and Site Geology

The site is located in the Coast Ranges physiomorphic province of California within the Amador Valley. The bedrock to the southeast of the site within the valley is mapped as Pliocene and Pleistocene-age Livermore Gravels, and to the west of the site within the Coast Ranges is mapped as late Cretaceous Panoche formation, by Graymer et al. (1996) and by Dibblee (2005), as shown in the Regional Geologic Map of Figure 3. Regional bedrock structure is mapped as striking to the northwest and dipping at inclinations of about 15 to 25 degrees to the northeast.

Dibblee (2005) has mapped the surficial deposits at the site as Quaternary-aged alluvium (Qa) as shown in Figure 3. Within the creek, Arroyo Del Valle, at the east edge of the property, a small area is mapped as Quaternary-aged alluvium of Arroyo Laguna (Qg). Generally, these soils are variable in nature and consist of interbedded clays, silts, and sands with some pebbly gravels. Nilson (1975) also characterizes the soil as alluvium (Qal), and according to the published open file maps, no landslide features are mapped within the site.

### 2.3.2 Seismicity

No active faults are known to pass through the project site, according to published maps (Jennings, 1994). The site is not located within a State of California Earthquake Fault Zone (CDMG 1982). The nearest active fault is the Calaveras fault, which is located approximately 1.2 miles west of the site. Due to the presence of active faults in the Northern California region,

the area is considered seismically active. Numerous earthquakes occur every year in the region, and large (>M7) earthquakes can be expected to occur in the future. As with all projects in the region, the development should take into account the potential for moderate to strong earthquake ground shaking.

## 2.4 SURFACE CONDITIONS

The site topography is relatively level ranging in elevation between approximately 335 to 338 feet above mean sea level (msl) (DeBolt 2011). At the time of our site visits for this study, tall grasses, bushes, trees, and minor shrinkage cracks were observed throughout the site. As previously stated, an existing pool and foundations associated with the previously relocated residential structure are located at the approximate center of the project site. The previous residential driveway asphalt concrete has been removed; however, the aggregate base is present.

## 2.5 SUBSURFACE CONDITIONS

The site soils consist of interlayered lean silt and lean clay with sand and gravel lenses. Generally, fine-grained materials above the groundwater level were very stiff to hard and below groundwater were medium stiff to very stiff. Granular materials were generally medium dense with loose zones and dense zones. Borings B-1 and B-2 encountered a two- to three-foot-thick layer of loose to medium dense silty sand at a depth of about 34 feet and 41 feet below the ground surface, respectively. The soil encountered in Boring B-3 was predominantly granular consisting of silty sand interlayered with lean silt and lean clay. The sand was loose to dense. Groundwater was encountered in Borings B-1 and B-2 at approximately 35 feet below ground surface. During relocation of the Jones House in the southwest corner of the project site, loose and compressible silt was encountered in the upper 4 to 5 feet of the building pad.

The near-surface soil has a Plasticity Index (PI) of between 4 and 7. The test results indicate the surficial fill has a low expansion potential. PI tests from previous studies in the area indicate the surrounding soils have a moderate expansion potential. Therefore, we anticipate some variation in the expansive characteristics of the site soils.

## 2.6 GROUNDWATER CONDITIONS

Groundwater was encountered at a depth of about 35 feet below grade at the time of drilling in Boring 1-B1 and 1-B2. The boreholes were backfilled on the day of drilling with cement grout materials approved by Zone 7 Water District. Borings 1-B3 and 1-B4 were backfilled with cuttings as directed by a Zone 7 Water District representative.

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

## 2.7 LABORATORY TESTING

Samples recovered during borehole drilling were tested to determine the following soil characteristics:

**TABLE 2.7-1**
Soil Characteristics

| Test | Designation | Location of Results |
|---|---|---|
| Natural Unit Weights and Moisture Contents | ASTM D-2216 | Appendix A |
| Atterberg Limits | ASTM D-4318 | Appendix B |
| Gradation | ASTM D-1140 | Appendix B |
| Hydrometer | ASTM D-422 | Appendix B |

The laboratory test results are shown on the boring logs (Appendix A). Results of individual laboratory tests are presented in Appendix B.

## 2.8 CREEK BANK STABILITY ANALYSES

The site is located adjacent to Arroyo Del Valle Creek. Sloughing of soils along portions of the creek bank have been observed and mapped (Figure 2). There are several locations of recent bank failures with near-vertical scarps up to approximately 15 feet in height. The failure debris is generally vegetated with berry bushes and vines which obscure the soil; therefore, it is difficult to observe the relative amount of actual vegetated soil vs. soil covered by foliage. There is also evidence of older bank failures which have established older vegetation including small trees. From the site plan, Figure 2, the northeast side of the property appears to have the greatest number of recent bank failures and undercut trees. The northwestern portion of the property, which is located on an inside bend of the creek appears to have a flatter, relatively more stable slope configuration. Several large diameter trees are growing on the creek bank; however, during our observation, we noted that many of the trees have exposed roots with substantial voids under the trees from creek undercutting or slope erosion. It appears that concrete was placed at the bottom of the creek bank in an effort to limit erosion in several areas. There is also an accumulation of concrete, building debris, and vegetation debris along the top edge of the slope.

### 2.8.1 Method of Analysis

Geotechnical evaluation of slope stability requires careful assessment of input parameters along with the selected method of analysis. While there are many different methods of stability analysis and numerous available computer programs, we selected the program SLIDE 6.0 produced by Roc Science. Limit equilibrium slope stability analyses was performed using Morgenstern and Price (1965) and Spencer (1967) method for slices. For our analyses, both undrained and drained strength parameters were used to determine the factor of safety for both long-term and short-term loading. A circular failure surface was used to determine the critical failure surface to recommend a structural setback.

In evaluating the stability of slopes under seismic conditions, we used a "pseudo-static" method of analysis as a screening analysis. SP117A states that slopes that have a pseudo-static factor of safety greater than 1.0 using a seismic coefficient derived from the screening analysis procedure of Stewart and others (2003) can be considered stable. The pseudo-static coefficient used was determined to be 0.42PGA (0.21g) based on 15 cm threshold of displacement as recommended by Stewart and others (2003). SP117A recommends as a general guideline that displacements of 0 to 15 cm are unlikely to correspond to serious landslide movement and damage.

### 2.8.2 Acceptable Factors of Safety

The minimum allowable factor of safety with respect to slope stability commonly ranges from 1.5 to 3 for static cases and 1 to 1.5 for seismic cases. Based on local geotechnical practice, in our opinion, we recommend that a static factor of safety of 1.5 and a pseudo-static factor of safety of 1.0 be considered adequate for the slope stability. We considered the various levels of conservatism involved in determining the engineering properties of the soil (density, shear strength, unit weight, permeability, etc...), the assumptions made in the method of analysis, nd potential variations in field conditions.

### 2.8.3 Geometry and Idealized Soil Profiles

Topographic data was obtained from the Possible Lot Configuration Plan for the Roselyn Estates II (Jones Property) dated February 15, 2011, developed by DeBolt Civil Engineering. Limited survey data was taken at the top and toe of the creek bank. The slope geometries depicted in cross-sections A-A' through D-D' included in Appendix C were modified from that shown on the plan by DeBolt based on our observations of the creek banks.

Prior to performing slope stability analyses, we evaluated the shear strength of the soil profile. To obtain shear strength values used in our analysis, we performed in-situ SPT tests and various laboratory tests. We reviewed the raw lab strength data and compared it with empirical correlations of SPT blow counts, plasticity index (PI), and soil type. Based on our data review, we developed the idealized soil profiles shown in Appendix C. The shear strength values were primarily derived from in-situ SPT data, correlations with PI and soil types using recommendations from Stark and Eid, 1997 and Bjerrum and Simons 1960, and unconfined compression strengths. With PI ranging between 6 and 15 for the fine- rained material, an effective stress internal angle of friction $\phi$' of between 27 and 31 degrees and a cohesion of 100 to 200 psf was estimated. Residual shear strengths for the potentially liquefiable material was determined using recommendations from Seed and Harder, 1990 and Mitchell and Soga 2005. The liquefied soil was estimated to have a residual undrained shear strength of approximately 400 psf.

#### 2.8.3.1 Rapid Drawdown

During rapid drawdown, the stabilizing effect of the water on the slope is lost, but the pore-water pressures within the embankment may remain high; as a result, the stability of the slope may be reduced. The saturation and dissipation of pore-water pressure in the slope is largely influenced

by the permeability of the soil. A detailed hydrologic study was performed in parallel with this geotechnical study. From this study, it is estimated that the flood elevation for the 100-year event will have a relatively short duration. Due to the creek bank consisting of predominantly stiff to hard fine-grained soil, and the short duration of the flood event, it is our opinion that the potential of the slope to saturate from a flooding event is low; therefore, rapid drawdown was not analyzed in this study.

### 2.8.4 Results of Analyses

We performed slope stability analysis for static conditions, pseudo-static conditions, and for the case where the slope is reinforced with geogrid. Reinforcement was modeled with two layers of Tensar UX1400HS geogrid 15 feet in length located 4 feet and 5 feet below the ground surface as shown in Figure 6. The geogrid is centered beneath the interception of the 2:1(horizontal:vertical) projection from the toe of the creek bank with the ground surface, or approximately 54 feet inward from the toe of the creek bank. The water elevation during static and pseudo-static conditions was modeled at elevation 308 feet, the toe elevation of the slope. The slope stability analysis indicates drained strength parameters govern the static stability of the creek. Seismic stability was analyzed using undrained strength parameters. Individual analyses are presented in Appendix C.

The results of our analyses are summarized in Tables 2.8.4-1 below.

**TABLE 2.8.4-1**
Summary of Slope Stability Analyses

| Section A to A' | | |
|---|---|---|
| Loading Condition | Calculated Factor of Safety | Acceptable Minimum Factor of Safety |
| Static (unreinforced) | 1.0 | 1.5 |
| Static (reinforced) | 1.6 | 1.5 |
| Pseudo-Static | 1.3 | 1.0 |
| **Section B to B'** | | |
| Loading Condition | Calculated Factor of Safety | Acceptable Minimum Factor of Safety |
| Static (unreinforced) | 1.4 | 1.5 |
| Static (reinforced) | 1.5 | 1.5 |
| Pseudo-Static | 1.4 | 1.0 |
| **Section C to C'** | | |
| Loading Condition | Calculated Factor of Safety | Acceptable Minimum Factor of Safety |
| Static (unreinforced) | 1.2 | 1.5 |
| Static (reinforced) | 1.6 | 1.5 |
| Pseudo-Static | 1.4 | 1.0 |

| Section D to D' | | |
|---|---|---|
| Loading Condition | Calculated Factor of Safety | Acceptable Minimum Factor of Safety |
| Static (unreinforced) | 1.5 | 1.5 |
| Static (reinforced) | 1.6 | 1.5 |
| Pseudo-Static | 1.1 | 1.0 |

The results of the analysis show that a 2:1 setback does not yield an acceptable minimum factor of safety without the use of geogrid reinforcement.

# 3.0 CONCLUSIONS

From a geotechnical engineering viewpoint, in our opinion, the site is suitable for the proposed development, provided the geotechnical recommendations in this report are properly incorporated into the design plans and specifications.

The primary geotechnical concerns that could affect development on the site are the presence of the existing pool and septic system, the existing fill, creek bank stability, expansive soil, and seismic settlement. We summarize our conclusions below.

## 3.1 EXISTING FILL

Boring B-3 encountered aggregate base fill associated with the previous residential driveway. It is anticipated that areas immediately surrounding the previous location of the residential house may be underlain by non-engineered fill.

Non-engineered fills can undergo excessive settlement, especially under new fill or building loads. Without proper documentation of existing fill placed on the site, we recommend complete removal and recompaction of the existing fill. There is also an existing leach field, pool, well, and other out buildings near the old residence. The leach field is not in use and should be properly abandoned. Any existing fill or improvements associated with the leach field, pool, well, or other out buildings should be treated as non-engineered material. These structures should be removed and differential fills resulting from subecxation should not exceed 5 feet. We present fill removal and replacement recommendations in Section 6.0 Earthwork Recommendations.

## 3.2 CREEK BANK STABILITY

As previously discussed, the existing creek bank shows evidence of ongoing erosion and localized slumping. Similar erosion and slumping should be expected in the future. In the section titled "Slope Setback Recommendations", we provide recommended setbacks to address the concern of erosion and slope instability.

## 3.3 EXPANSIVE SOIL

We encountered potentially expansive lean clay and silt near the surface of the site in our exploration borings. Our laboratory testing indicate that these soils exhibit low to moderate shrink/swell potential with variations in moisture content.

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

Post-tensioned mat foundations are the preferred foundation system for the residential structures. Design criteria for this foundation type are presented in Section 6.0.

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

We have also provided specific grading recommendations for compaction of clay soil at the site. The purpose of these recommendations is to reduce the swell potential of the clay by compacting the soil at a high moisture content and controlling the amount of compaction. Expansive soil mitigation recommendations are presented in Section 6.5 of this report.

## 3.4 SEISMIC HAZARDS

California Geologic Survey (CGS), 2008, maps this site within a Seismic Hazard Zone for areas that may be susceptible to liquefaction. Potential seismic hazards resulting from a nearby moderate to major earthquake can generally be classified as primary and secondary. The primary effect is ground rupture, also called surface faulting. The common secondary seismic hazards include ground shaking, and ground lurching. The following sections present a discussion of these hazards as they apply to the site. Based on topographic and lithologic data, the risk of regional subsidence or uplift, landslides, tsunamis, or seiches is considered low to negligible at the site.

### 3.4.1 Ground Rupture

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

### 3.4.2 Ground Shaking

An earthquake of moderate to high magnitude generated within the San Francisco Bay region could cause considerable ground shaking at the site, similar to that which has occurred in the past. To mitigate the shaking effects, all structures should be designed using sound engineering judgment and the 2010 California Building Code (CBC) requirements, as a minimum. Seismic design provisions of current building codes generally prescribe minimum lateral forces, applied statically to the structure, combined with the gravity forces of dead-and-live loads. The code-prescribed lateral forces are generally considered to be substantially smaller than the comparable forces that would be associated with a major earthquake. Therefore, structures should be able to: (1) resist minor earthquakes without damage, (2) resist moderate earthquakes without structural damage but with some nonstructural damage, and (3) resist major earthquakes without collapse but with some structural as well as nonstructural damage. Conformance to the current building code recommendations does not constitute any kind of guarantee that significant structural damage would not occur in the event of a maximum magnitude earthquake; however, it is reasonable to expect that a well-designed and well-constructed structure will not collapse or cause loss of life in a major earthquake (SEAOC, 1996).

### 3.4.3 Liquefaction

This site is mapped as potentially liquefiable by the 2008 CGS map. 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.

We evaluated the liquefaction potential of the subsurface soil by collecting soil samples using a Standard Penetration Test (SPT) split-spoon sampler. Layers of sandy silt and silty sand were encountered below the groundwater table 34 and 41 feet below the ground surface in Borings B-1 and B-2, respectively. We performed analyses of these layers of low-plasticity silts and sands and found that they have the potential to liquefy during a seismic event. In performing our analyses, we used methods published by Youd et al. (2001), Seed et al. (2003), and Idriss & Boulanger (2008) to evaluate the liquefaction potential of the granular material. To evaluate the liquefaction potential of the fine-grained material, we used the recommendation presented in Bray and Sancio (2006). The recommendations correlate liquefaction susceptibility of fine-grained soils to their Plasticity Index (PI), Liquid Limit (LL), and natural water content ($w_c$). Recorded SPT blow count resistance (N-value) was corrected for sampler and hammer type, overburden pressure, boring diameter, and fines content. We assumed a groundwater level 27 feet below existing ground, a peak ground acceleration of 0.5g and a $M_w$ of 6.8.

We evaluated the liquefaction potential for the soils encountered below the assumed water table. The results indicate that silty sand layers approximately 1 to 2 feet thick encountered at 34 feet below grade for Boring B-1 and 41 feet below grade for Boring B-2 are potentially liquefiable. In addition, low plasticity silt underlying the sand was found to be potentially liquefiable. The silt is estimated to have a plasticity index of approximately 6 to 7, a liquid limit of 26 to 27,

and a water content of 25 to 35%. Bray and Sancio (2006) found that soils with a PI<12 and a wc/LL > 0.85 were potentially liquefiable.

### 3.4.4 Lateral Spreading

Lateral spreading is a failure within a nearly horizontal soil zone (possibly due to liquefaction) that causes the overlying soil mass to move toward a free face or down a gentle slope. Generally, the effects of lateral spreading are most significant at the free face or the crest of a slope and diminish with distance from the slope.

As discussed previously, the Arroyo Del Valle Creek is located along the northern boundary of the project site. We analyzed the stability of the creek with residual shear strength of the liquefied soil as previously discussed. It is our opinion that the potential for lateral spreading at the project site is low.

### 3.4.5 Seismic-Induced Settlement Analyses

Seismic-induced settlement can be generally subdivided into two categories, settlement as a result of liquefaction of saturated or nearly saturated soils and dynamic densification of non-saturated soils.

#### 3.4.5.1 Liquefaction

Deformation of the ground surface is a common result of liquefaction. Vertical settlement may result from densification of the deposit or volume loss from venting to the ground surface. Densification occurs as excess pore pressures dissipate, resulting as vertical settlement at the ground surface.

We calculated potential settlement estimates based on the methods first proposed by Tokimatsu and Seed in 1987 and Ishihara and Yoshimine in 1992. Based on our liquefaction analysis, we estimate total theoretical liquefaction-induced volumetric strain settlements of approximately 1½ to 2 inches. The associated differential settlement is expected to be approximately ¾ to 1 inch.

#### 3.4.5.2 Dynamic Densification

Densification of loose granular soils above the groundwater could cause settlement of the ground surface due to earthquake-induced vibrations. Our exploration encountered loose and medium dense silty sand in Borings B-3 and B-4. The sandy layers ranged between 3 to 6 feet thick. We calculated potential settlement estimates based on the methods first published by Tokimatsu & Seed in 1984. Settlement estimates result in potential theoretical dynamic densification induced settlement of up to ¾ inch

### 3.4.6 Ground Lurching

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

### 3.4.7 Flooding

A detailed hydrologic study was performed in parallel with this study. The analysis indicates the potential flood elevation corresponding to a 100-year flood event to range between 322 to 327 feet elevation. The top of the creek bank ranges between 334 to337. Based on the site elevation, flooding is not expected at the subject site; however, the Civil Engineer should review pertinent information relating to possible flood levels for the subject site based on final pad elevations and provide appropriate design measures for development of the project, if necessary.

## 3.5 SOIL CORROSION POTENTIAL

During our field exploration, two additional near-surface soil samples were collected at the site for sulfate testing. The sulfate test results are included in Appendix B. According to the laboratory testing on the near-surface soils, the sulfate ion concentration to be 0.002% by weight.

The 2010 CBC references the 2008 American Concrete Institute Manual, ACI 318-08 (Chapter 4, Sections 4.2 and 4.3) for concrete requirements. ACI Tables 4.2.1 and 4.3.1 (Tables 3.5-1 and 3.5-2 below) provide the following sulfate exposure categories and classes and concrete requirements in contact with soil based upon the exposure risk.

**TABLE 3.5-1**
Sulfate Exposure Categories and Classes

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

**TABLE 3.5-2**
Requirements for Concrete by Exposure Class

| Exposure Class | Max w/cm | Min f'c (psi) | Cement Type | | | Calcium Chloride Admixture |
|---|---|---|---|---|---|---|
| | | | ASTM C150 | ASTM C595 | ASTM C1157 | |
| S0 | N/A | 2,500 | No Type restriction | No Type restriction | No Type restriction | No restriction |
| S1 | 0.5 | 4,000 | II†‡ | IP(MS), IS(<70) (MS) | MS | No restriction |
| S2 | 0.45 | 4,500 | V‡ | IP(HS) IS(<70) (HS) | HS | Not permitted |
| S3 | 0.45 | 4,500 | V + pozzolan or slag§ | IP(HS) + pozzolan or slag or IS(<70) (HS) + pozzolan or slag§ | HS + pozzolan or slag§ | Not permitted |

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

In accordance with the criteria presented in ACI 319-08, the test results are classified in the "Not Applicable" sulfate exposure class. Cement type and maximum water-cement ratio are not specified by the ACI for this class.

As minimum requirements, we recommend that Type II cement be used in foundation concrete for structures at the project site and concrete should incorporate a maximum water cement ratio of 0.5 and a minimum compressive strength of 3,000 psi. It should be noted, however, that the structural engineering design requirements for concrete might result in more stringent concrete specifications.

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

### 3.6 2010 CBC SEISMIC DESIGN PARAMETERS

We provide the 2010 California Building Code (CBC) seismic parameters in Table 3.6-1 below.

**TABLE 3.6-1**
2010 CBC Seismic Design Parameters

| Parameter | Design Value |
|---|---|
| Site Class | D |
| 0.2 second Spectral Response Acceleration, $S_S$ | 1.85 |
| 1.0 second Spectral Response Acceleration, $S_1$ | 0.68 |
| Site Coefficient, $F_A$ | 1.0 |
| Site Coefficient, $F_V$ | 1.5 |
| Maximum considered earthquake spectral response accelerations for short periods, $S_{MS}$ | 1.85 |
| Maximum considered earthquake spectral response accelerations for 1-second periods, $S_{MS}$ | 1.02 |
| Design spectral response acceleration at short periods, $S_{DS}$ | 1.23 |
| Design spectral response acceleration at 1-second periods, $S_{D1}$ | 0.68 |
| Long period transition-period, $T_L$ | 8 sec |

## 4.0 CONSTRUCTION MONITORING

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

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

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

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

# 5.0 SLOPE SETBACK RECOMMENDATIONS

Based on the stability analysis, the following setbacks are recommended:

- 3:1 (horizontal:vertical) line of projection from the toe of the creek bank to the top of the bank plus an additional horizontal distance of 15 feet for habitable structures.

- 2.5:1 line of projection from the toe of the creek bank to the top of the bank for non-habitable improvements, including the proposed Lynn Drive if no reinforcement is used.

- 2:1 line of projection from the toe of the creek bank to the top of the bank for non-habitable improvements, including the proposed Lynn Drive if two layers of Tensar Ux1400HS or approved equivalent geogrid reinforcement is placed as indicated in Figure 6. Fifteen foot long strips of Geogrid should be placed at depths of 4 feet and 5 feet below the ground surface centered at the ground projection as shown on Figure 6. The reinforcement should be placed such that the strength of the geogrid is gained perpendicular to the centerline of the creek.

The purpose of these setbacks is to address potential for instability and erosion of the creek banks. It is anticipated that surficial failures may adversely impact the area within the recommended setback zone. Maintenance and/or repair of the trail may be necessary over the long term.

## 5.1 BIORENTENTION AREA

As previously stated, a bioretention area is proposed between Lynn Drive and the top of the creek bank. Infiltration of water could adversely affect the stability of the creek bank. An impermeable membrane should underlay the bioretention area to limit water infiltration into the underlying soils.

# 6.0 EARTHWORK RECOMMENDATIONS

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

As used in this report, the term "moisture condition" refers to adjusting the moisture content of the soil by either drying if too wet or adding water if too dry.

We define "structural areas" in Section 4 of this report as any area sensitive to settlement of compacted soil. These areas include, but are not limited to building pads, sidewalks, pavement areas, and retaining walls.

## 6.1 EXISTING FILL REMOVAL

Remove all existing fill to competent native soil, as determined by ENGEO. As previously discussed, existing fill is anticipated in the immediate vicinity of the previous location of the residential structure. Fill is estimated to be generally less than 3 feet in thickness, except in the pool, leach field, and well improvements locations where the fill may be deeper.

## 6.2 GENERAL SITE CLEARING

Site development should commence with the removal of existing underground structures, such as buried utilities and their backfill, septic tanks, irrigation lines, and water well system if any exist, and removal of vegetation and debris, loose or compressible soils and any other deleterious materials. As previously discussed, foundations and the swimming pool associated with the previous location of the residential location were present during our field exploration. All previous material associated with the previous residence should be removed.

Tree roots should be removed to a depth of at least three feet below pad grades. Topsoil is estimated to be from two to four inches in thickness depending on location. The organically contaminated materials should not be used in proposed building pad or pavement areas. Subject to approval by th : Landscape Architect, strippings and organically contaminated soils can be used in landscape areas. Otherwise, such soils should be removed from the project site. Any topsoil that will be retained for future use in landscape areas should be stockpiled in areas where it will not interfere with grading operations.

A representative of ENGEO should determine the actual removal depth in the field based on conditions encountered during the site grading. Excavations resulting from demolition and stripping below design grades should be cleaned to a firm undisturbed, non-yielding soil surface as determined by the Geotechnical Engineer. Following clearing and grubbing, the exposed subgrade soils should be scarified, moisture-conditioned and backfilled the depressions with compacted engineered fill. The requirements for backfill materials and placement procedures are the same as those for engineered fill as described in the "Fill Placement" section.

## 6.3 SURFICIAL SOIL REMOVAL

Based on the soil data, loose and compressible materials in the southwest portion of the project site, approximately in the upper three to five feet, were observed in the adjacent sites. It is our anticipation that similar loose and compressible soil may be encountered in some areas of the project site.

We recommend building envelopes (including areas five feet beyond the building perimeters) and driveway pavement areas be re-worked to provide a minimum of two feet of engineered fill beneath improvements. This can be accomplished by overexcavating a minimum depth of one foot below existing grades to expose firm non-yielding soil. The upper 12 inches of the exposed excavation should be scarified and recompacted. Then, grades should be restored with engineered fill followed by placement of any additional fill to achieve finished pad grades per

specifications presented in the later section. Additionally, the areas of the pool and leach field should be overexcavated and backfilled with engineered fill. The subexcavation should be designed to limit differential fills beneath the building footprints to less than 5 feet.

The Geotechnical Engineer or qualified representative should determine the material removal depth in the field at the time of grading. Evaluation of unsuitable deposits should be performed during grading and may include sampling and laboratory analyses.

After the site has been properly cleared and stripped, and necessary excavations have been made, scarify the surface at least 12 inches, moisture condition, and compact in accordance with the recommendations presented below in the "Fill Placement" section, prior to replacing and recompacting overlying soils as engineered fill.

## 6.4 ACCEPTABLE FILL

Onsite soil material is suitable as fill material provided it is processed to remove concentrations of organic material, debris, and particles greater than 8 inches in maximum dimension.

Imported fill materials should meet the above requirements and have a plasticity index less than 12 and at least 20 percent passing the No. 200 sieve. Allow ENGEO to sample and test proposed imported fill materials at least 72 hours prior to delivery to the site.

## 6.5 FILL COMPACTION

The exposed non-yielding surface to receive fill should be scarified to a depth of 12 inches, moisture conditioned, and recompated to provide adequate bonding with the initial lift of fill. All fills should be placed in thin lifts. The lift thickness should not exceed 12 inches or the depth of penetration of the compaction equipment used, whichever is less.

The following compaction control requirements are generally applied to all fills:

| | |
|---|---|
| Required Moisture Content: | Not less than 3 percentage points above optimum moisture content. |
| Minimum Relative Compaction: | Not less than 90 percent. |

Since the compaction of expansive soils even up to 90 percent of the maximum dry unit weight may produce an undesirable environment for expansion in the zone of significant seasonal variation, special requirements for compaction of expansive soils are necessary within the upper layer of subgrade soil in building areas. For this condition, it is important to compact the soils such that their swell potential is reduced to acceptable values. The following specifications for compaction of potentially expansive soils within the building pad areas in the upper 3 feet for the fill lots and cut/fill transition lots, and the upper one foot for cut lots, are proposed:

| | |
|---|---|
| Required Moisture Content: | Not less than 4 percentage points above optimum moisture content. |
| Minimum Relative Compaction: | Not less than 87 percent and not more than 92 percent. Not less than 90 percent for the upper 6 inches of finish pavement subgrade. |

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

### 6.5.1 Underground Utility Backfill

#### 6.5.1.1 General

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

#### 6.5.1.2 Structural Areas

Place and compact trench backfill as follows:

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

2. Moisture condition trench backfill to 2 to 4 percent above the optimum moisture content. Moisture condition backfill outside the trench.

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

4. Compact fill to between 87 and 92 percent relative compaction (90 percent minimum relative compaction at depths of 3 feet or more below finish grades).

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

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

### 6.5.2 Landscape Fill

Process, place and compact fill in accordance with Section 6.5 Fill Compaction, except compact to at least 85 percent relative compaction (ASTM D1557).

## 6.6 SLOPES

### 6.6.1 Gradients

Construct final slope gradients to 2:1 (horizontal:vertical) or flatter. The contractor is responsible to construct temporary construction slopes in accordance with CALOSHA requirements.

## 6.7 SITE DRAINAGE

### 6.7.1 Surface Drainage

The project civil engineer is responsible for designing surface drainage improvements. With regard to geotechnical engineering issues, we provide the following minimum recommendation for surface drainage.

1. Slope pavement areas a minimum of 1 percent towards drop inlets or other surface drainage devices.

2. Slope finished grade away from building exteriors at a minimum of 3 percent for a distance of at least 7 feet.

3. Discharge roof down spouts into closed conduits and direct away from buildings to appropriate drainage devices.

### 6.7.2 Subsurface Drainage

Based on our site exploration and current grading concepts for the site, we do not anticipate that subdrainage systems will be necessary. We recommend that we review the site grading plans to further evaluate the need for subdrainage systems as well as observe the earthwork operations during site grading.

## 6.8 LANDSCAPING CONSIDERATIONS

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

- Selecting landscaping that requires little or no watering, especially within 3 feet of structures, slabs-on-grade, or pavements,

- Using low precipitation sprinkler heads,
- Regulating the amount of water distributed to lawn or planter areas by installing timers on the sprinkler system,
- Providing surface grades to drain rainfall or landscape watering to appropriate collection systems and away from structures, slabs-on-grade, or pavements,
- Preventing water from draining toward or ponding near building foundations, slabs-on-grade, or pavements, and
- Avoiding open planting areas within 3 feet of the building perimeter.

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

# 7.0 FOUNDATION RECOMMENDATIONS

We developed structural improvement recommendations using data obtained from our field exploration, laboratory test results, and engineering analysis.

## 7.1 POST-TENSIONED MAT FOUNDATIONS

We recommend that the proposed single-family structures be supported on post-tensioned (PT) mat foundations bearing on prepared natural soil or compacted fill.

We recommend that PT mats be at least 10 inches thick and have a thickened edge at least 2 inches greater than the mat thickness. The thickened edge should be at least 12 inches wide.

Design PT mats for an average allowable bearing pressure of 1,000 pounds per square foot (psf) for dead plus live loads, with maximum localized bearing pressures of 1,500 psf at column or wall loads. Allowable bearing pressures can be increased by one-third for all loads including wind or seismic. Design PT mats using the criteria presented in Table 7.1-1 below.

**TABLE 7.1-1**
Post-Tension Design Criteria

| Condition | Center Lift | Edge Lift |
|---|---|---|
| Edge Moisture Variation Distance, $e_m$ (ft.) | 7.7 | 4.0 |
| Differential Soil Movement, $y_m$ (in.) | 1.2 | 1.7 |

The above design criteria are based on the procedure presented by the Post-Tensioning Institute "Design of Post-Tensioned Slabs-on-Ground" Third Edition, including appropriate addenda (2004).

Underlay PT mats with a moisture reduction system as recommended below. In addition, moisture condition the pad subgrade to a moisture content at least 4 percentage points above optimum prior to foundation construction. The subgrade should not be allowed to dry prior to concrete placement. We also recommend that we be retained to observe the prepour moisture conditions to check that our report recommendations have been followed.

### 7.1.1 Subgrade Treatment

The subgrade material under structural mats should be uniform. The pad subgrade should be moisture conditioned to a moisture content of at least 3 percentage points above optimum. The subgrade should be thoroughly soaked prior to placing the concrete. The subgrade should not be allowed to dry prior to concrete placement.

### 7.1.2 Slab Moisture Vapor Reduction

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

1. Install a vapor retarder membrane directly beneath the slab. Seal the vapor retarder at all seams and pipe penetrations. Vapor retarders shall conform to Class A vapor retarder per ASTM E 1745-97 "Standard Specification for Plastic Water Vapor Retarders used in Contact with Soil or Granular Fill under Concrete Slabs".

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

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

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

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

### 7.1.3 Settlement

The estimated range of theoretical seismically induced settlement is discussed in Section 3.4.5. Based on our analysis, foundations should be designed to accommodate ¾ inch of seismically induced differential settlement over a distance of 30 feet.

# 8.0 SLABS-ON-GRADE

This section provides guidelines for secondary slabs such as 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 secondary slab-on-grade construction is anticipated, care must be exercised in attaining a near-saturation condition of the subgrade soil before concrete placement.

Secondary 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. Slabs-on-grade should be reinforced for control of cracking, and frequent control joints should be provided to control the cracking. Reinforcement should be designed by the structural engineer. In our experience, welded wire mesh may not be sufficient to control slab cracking. As a minimum, secondary slabs-on-grade should be reinforced with steel bars in lieu of wire mesh.

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

## 8.1 PRELIMINARY PAVEMENT DESIGN

Based on the soil boring data, a Resistance Value (R-Value) of 5 has been assumed in the preliminary pavement design. The following preliminary pavement sections have been determined for Traffic Indices of 5, 6 and 7, and an assumed R-value of 5, and in accordance to the design methods contained in Topic 608 of Caltrans Highway Design Manual.

**TABLE 8.1-1**
Recommended Asphalt Concrete Pavement Sections

| Traffic Index | Section | |
|---|---|---|
| | Asphalt Concrete (inches) | Class 2 Aggregate Base (inches) |
| 5 | 3.0 | 10.0 |
| 6 | 3.5 | 12.5 |
| 7 | 4.0 | 16.0 |

The Traffic Index should be determined by the Civil Engineer or appropriate public agency. These sections are for estimating purposes only. Actual sections to be used should be based on R-value tests performed on samples of actual subgrade materials recovered at the time of grading. Pavement construction and all materials should comply with the requirements of the Standard Specifications of the State of California Department of Transportation, Civil Engineer and the City of Pleasanton and the following minimum requirements.

- All pavement subgrades should be scarified to a depth of 12 inches below finished subgrade elevation, moisture conditioned to 4 percentage points above optimum, and compacted to at least 90 percent relative compaction and in accordance with the City of Pleasanton requirements.

- Subgrade soils should be in a stable, non-pumping condition at the time aggregate baserock materials are placed and compacted.

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

- Aggregate baserock materials should meet current Caltrans specifications for Class 2 aggregate baserock and should be compacted to at least 95 percent of maximum dry density at a minimum moisture content of at least optimum.

- Asphalt paving materials should meet current Caltrans specifications for asphalt co crete.

- All concrete curbs separating pavement and irrigated landscaped areas should extend into the subgrade and below the bottom of adjacent aggregate baserock materials.

### 8.2 CUT-OFF CURBS

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

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

### 8.3 RESIDENTIAL DRIVEWAYS/GARAGE SLABS

We were not retained to provide design recommendations for residential driveways or garage slabs. They should be designed to resist the anticipated traffic and structural loads, (and the effects of expansive soil movement.

## 9.0 LIMITATIONS AND UNIFORMITY OF CONDITIONS

This report presents geotechnical recommendations for design of the improvements discussed in Section 1.3 for the Roselyn Estates II Property project. If changes occur in the nature or design of the project, we should be allowed to review this report and provide additional recommendations, if any. It is the responsibility of the owner to transmit the information and recommendations of this report to the appropriate organizations or people involved in design of

the project, including but not limited to developers, owners, buyers, architects, engineers, and designers. The conclusions and recommendations contained in this report are solely professional opinions and are valid for a period of no more than 2 years from the date of report issuance.

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

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

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

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

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

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

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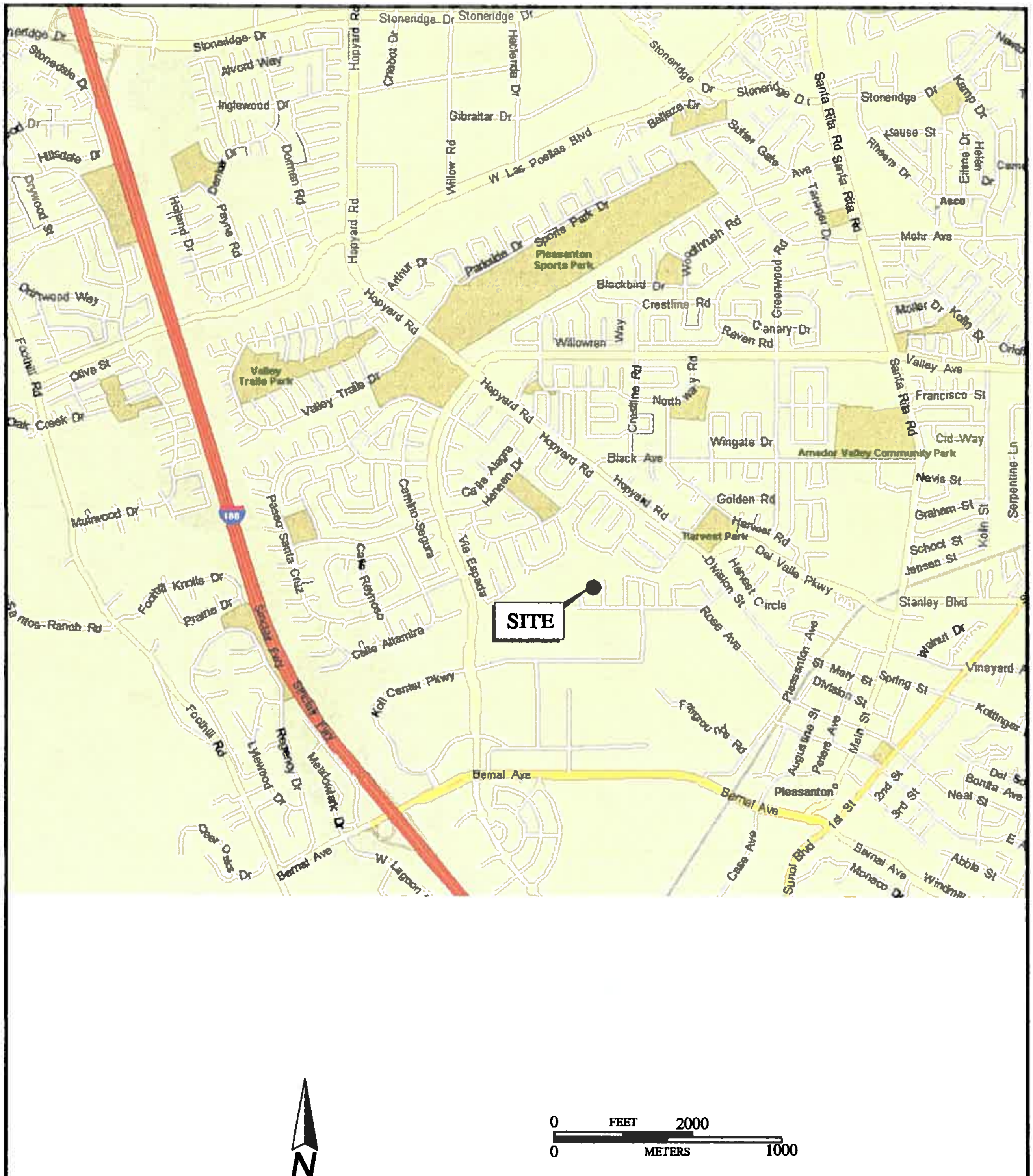
## FIGURES

**Figure 1 – Vicinity Map**
**Figure 2 – Site Plan**
**Figure 3 – Regional Geologic Map (Dibblee)**
**Figure 4 – Regional Faulting and Seismicity Map**
**Figure 5 – Seismic Hazard Map, Dublin Quadrange, CGS 2008**
**Figure 6 – Typical Geogrid Reinforced Section**

F I G U R E S




*BASE MAP SOURCE: MS STREETS AND TRIPS*

| ENGEO Expect Excellence | VICINITY MAP<br>ROSELYN ESTATES II PROPERTY<br>PLEASANTON, CALIFORNIA | PROJECT NO.: 4425.000.000 | | FIGURE NO. 1 |
|---|---|---|---|---|
| | | SCALE: AS SHOWN | | |
| | | DRAWN BY: SRP | CHECKED BY: DEB | |

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ORIGINAL FIGURE PRINTED IN COLOR

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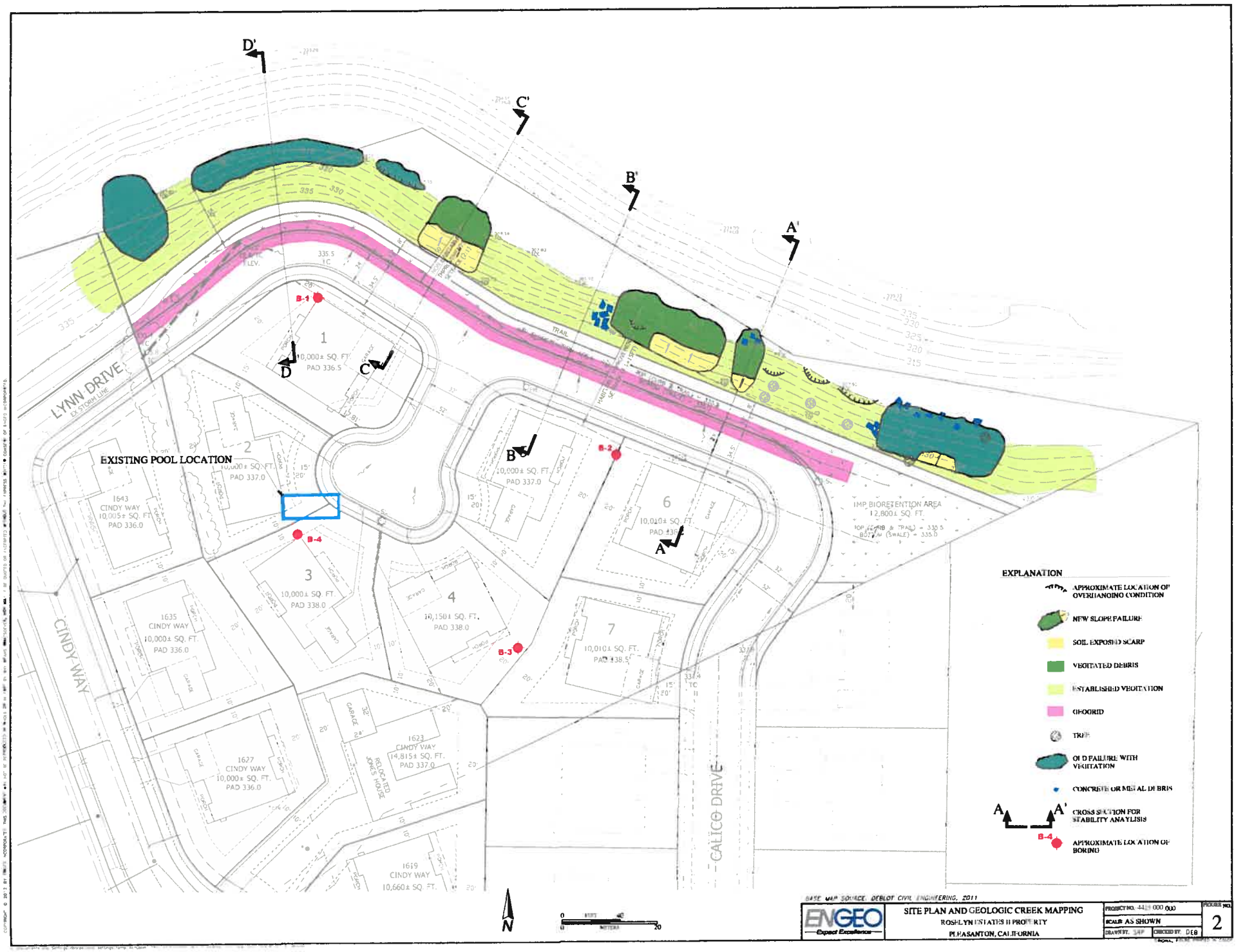


BASE MAP SOURCE: DEBLOT CIVIL ENGINEERING, 2011

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SITE PLAN AND GEOLOGIC CREEK MAPPING
ROSELYN ESTATES II PROPERTY
PLEASANTON, CALIFORNIA

PROJECT NO. 4415.000.000 | FIGURE NO. 2
SCALE AS SHOWN
DRAWN BY: SHP | CHECKED BY: DEB

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0 FEET 2000
0 METERS 1000

N

## EXPLANATION

BEDROCK CONTACT-DASHED WHERE GRADATIONAL OR APPROXIMATELY LOCATED

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

*STRIKE AND DIP OF STRATA*

INCLINED VERTICAL OVERTURNED

| | |
|---|---|
| Qg | ALLUVIAL GRAVEL |
| Qa | ALLUVIAL SAND AND GRAVEL |
| QTlg | LIVERMORE GRAVEL |
| Kp | CLAY SHALE (PANOCHE FORMATION) |
| Kps | SANDSTONE (PANOCHE FORMATION) |
| Kpc | CONGLOMERATE (PANOCHE FORMATION) |

*BASE MAP SOURCE: DIBBLEE, 2005*

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# REGIONAL GEOLOGIC MAP

ROSELYN ESTATES II PROPERTY

PLEASANTON, CALIFORNIA

PROJECT NO.: 4425.000.000 | FIGURE NO.: 3

SCALE: AS SHOWN

DRAWN BY: SRP | CHECKED BY: DEB

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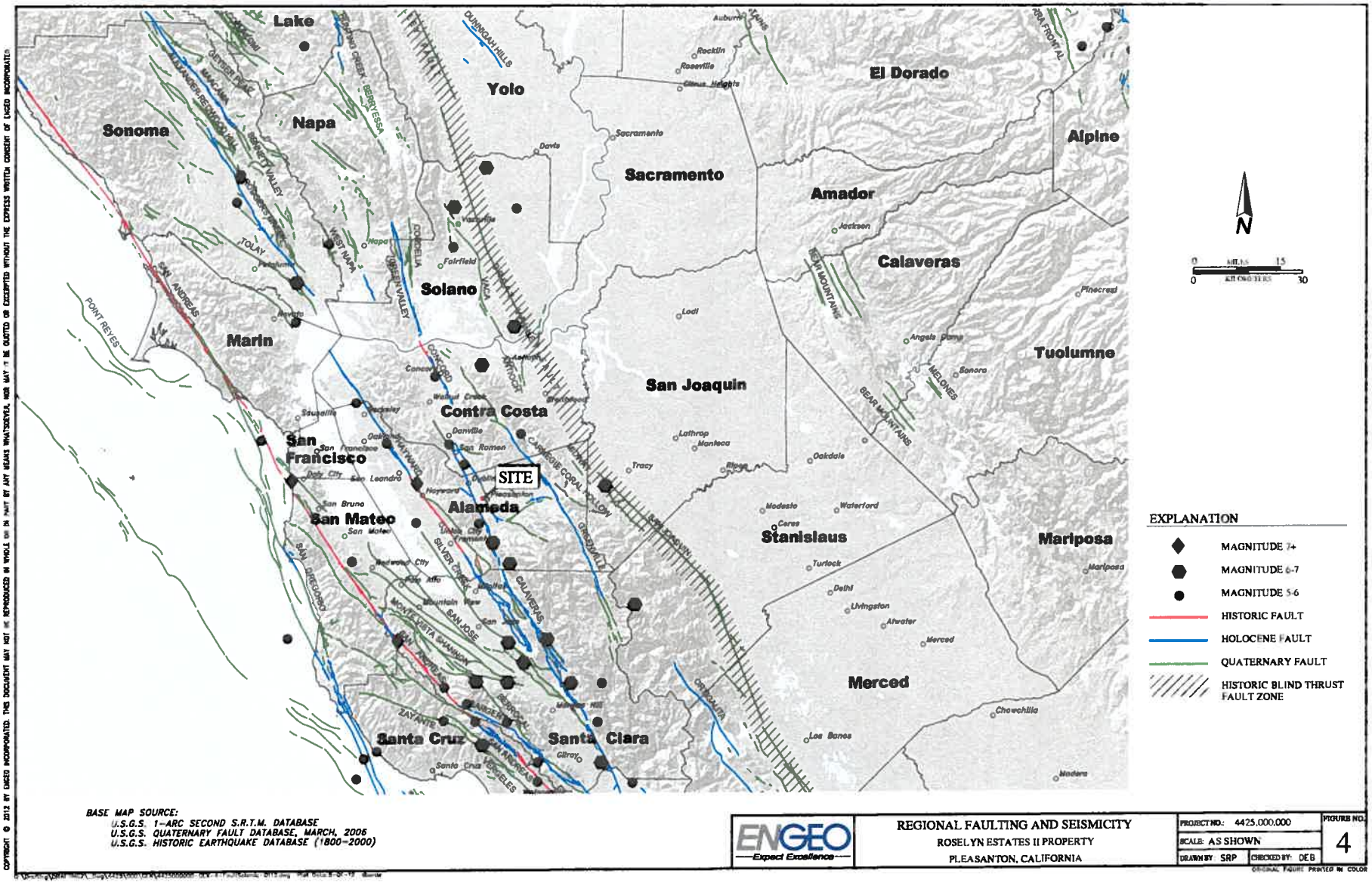
ORIGINAL FIGURE PRINTED IN COLOR




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

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REGIONAL FAULTING AND SEISMICITY
ROSELYN ESTATES II PROPERTY
PLEASANTON, CALIFORNIA

PROJECT NO.: 4425.000.000
SCALE: AS SHOWN
DRAWN BY: SRP
CHECKED BY: DEB

FIGURE NO. 4

ORIGINAL FIGURE PRINTED IN COLOR

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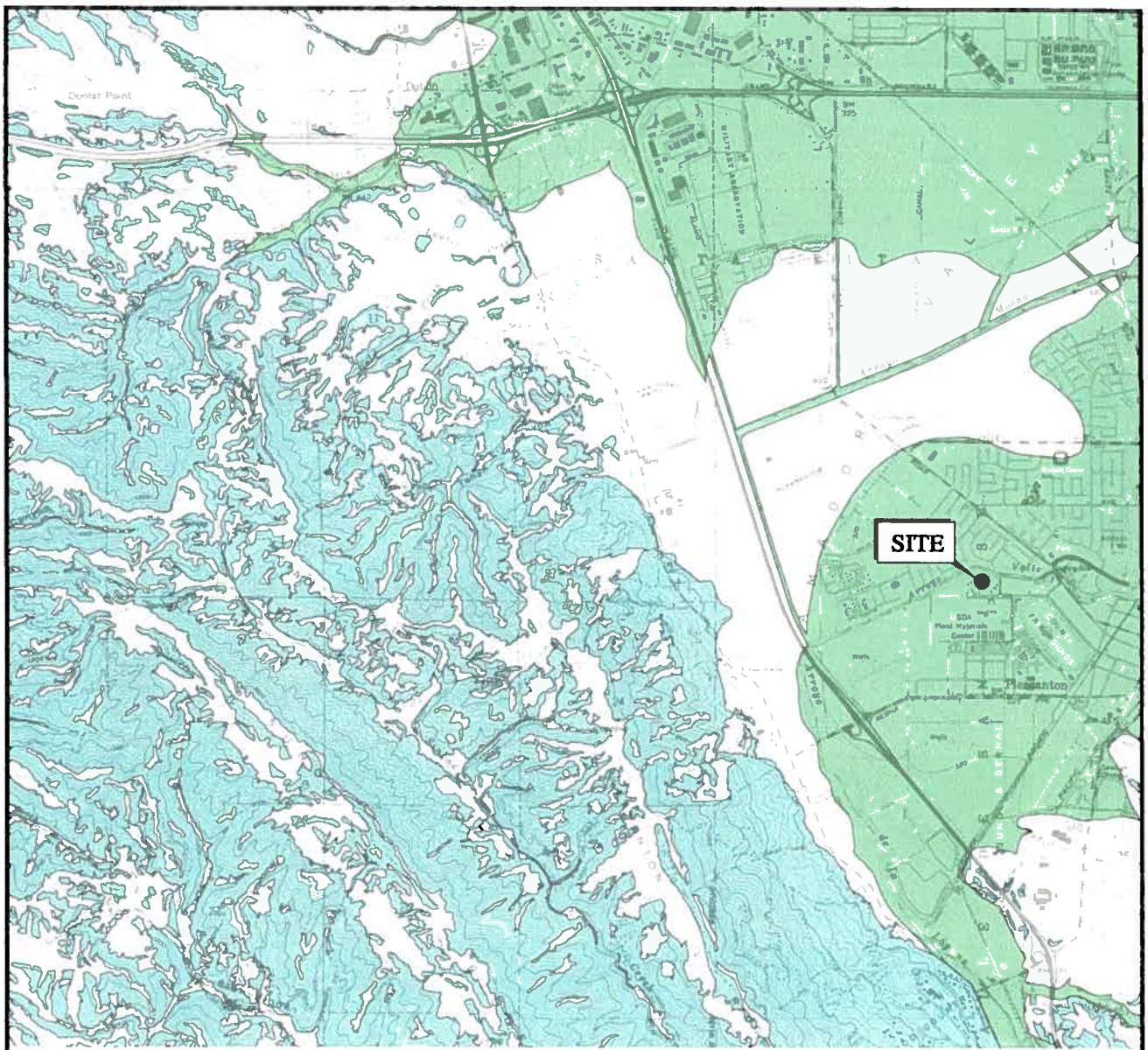


## EXPLANATION

### LIQUEFACTION

AREAS WHERE HISTORIC OCCURRENCE OF LIQUEFACTION, OR LOCAL GEOLOGICAL, GEOTECHNICAL AND GROUNDWATER CONDITIONS INDICATE A POTENTIAL FOR PERMANENT GROUND DISPLACEMENTS SUCH THAT MITIGATION AS DEFINED IN PUBLIC RESOURCES CODE SECTION 2693(c) WOULD BE REQUIRED

### EARTHQUAKE-INDUCED LANDSLIDES

AREAS WHERE PREVIOUS OCCURRENCE OF LANDSLIDE MOVEMENT, OR LOCAL TOPOGRAPHIC, GEOLOGICAL, GEOTECHNICAL AND SUBSURFACE WATER CONDITIONS INDICATE A POTENTIAL FOR PERMANENT GROUND DISPLACEMENTS SUCH THAT MITIGATION AS DEFINED IN PUBLIC RESOURCES CODE SECTION 2693(c) WOULD BE REQUIRED

N

0 FEET 4000

0 METERS 2000

*BASE MAP SOURCE: CALIFORNIA DEPARTMENT OF CONSERVATION, CALIFORNIA GEOLOGICAL SURVEY, 2008*

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# SEISMIC HAZARD ZONE MAP

ROSELYN ESTATES II PROPERTY

PLEASANTON, CALIFORNIA

| | | |
|---|---|---|
| PROJECT NO.: 4425.000.000 | | FIGURE NO. 5 |
| SCALE: AS SHOWN | | |
| DRAWN BY: SRP | CHECKED BY: DEB | |

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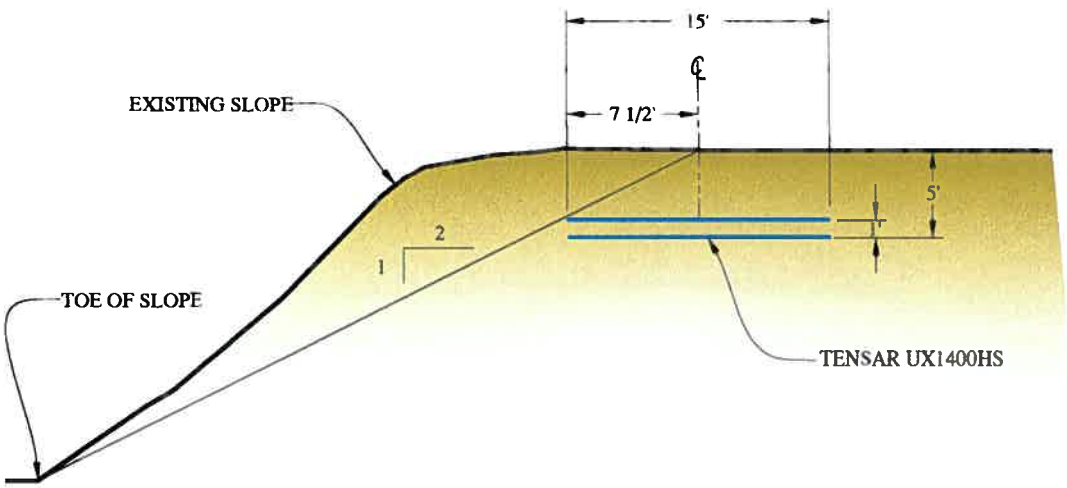


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# TYPICAL GEOGRID REINFORCED SECTION

ROSELYN ESTATES II PROPERTY

PLEASANTON, CALIFORNIA

| | | |
|---|---|---|
| PROJECT NO.: 4425.000.000 | | FIGURE NO. 6 |
| SCALE: NO SCALE | | |
| DRAWN BY: SRP | CHECKED BY: DEB | |

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# APPENDIX A

**Key to Boring Logs**
**Exploration Logs**

APPENDIX A

# KEY TO BORING LOGS

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

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

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

## GRAIN SIZES

| U.S. STANDARD SERIES SIEVE SIZE | | | | CLEAR SQUARE SIEVE OPENINGS | | | |
|---|---|---|---|---|---|---|---|
| 200 | 40 | 10 | 4 | 3/4" | 3" | 12" | |
| SILTS AND CLAYS | SAND | | | GRAVEL | | COBBLES | BOULDERS |
| | FINE | MEDIUM | COARSE | FINE | COARSE | | |

### RELATIVE DENSITY

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

### CONSISTENCY

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

### SAMPLER SYMBOLS

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

### MOISTURE CONDITION

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

### LINE TYPES

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

### GROUND-WATER SYMBOLS

- Groundwater level during drilling
- Stabilized groundwater level

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

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

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# LOG OF BORING B-1



Jones Propoerty (Roslyn Estates II)
Pleasanton, CA
4425.000.000

| | | | |
|---|---|---|---|
| Date Drilled | : 12/15/2011 | Logged/Reviewed By: | J.Botelho/ R.Skinner |
| Hole Depth (ft) | : 51 1/2 feet | Drilling Contractor | : Britton Exploration |
| Surface Elev (ft-msl) | : approximately 336 | Drilling Method | : Hollow Stem |
| Latitude (NAD83) | : 37.6675 | Hammer Type | : 140lb Auto Hammer |
| Longitude (NAD83) | : -121.8909 | Hole Diameter | : 9 inches |

| Depth in Feet | Surf. Elev. 336 | Sample Type | DESCRIPTION | Log Symbol | Water Level | Blow Count / Foot | Moisture Content (% dry weight) | Dry Unit Weight (pcf) | Unconfined Strength *field approx | Percent Passing #200 Sieve | Plasticity Index |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 0 | 336 | | Surface is tall grass | | | | | | | | |
| | | | Sandy SILT (ML), yellowish brown, dry to slightly moist, very stiff, few roots, fine-grained sand. | | | 5 | 5.2 | 91.0 | 3.25* | | |
| 5 | 331 | | | | | 13 | | | 3.25* | | |
| 10 | 326 | | Lean CLAY (CL), olive brown, hard, dry to slightly moist, contains silt, fine-grained sand, rootlets, slight porous texture. | | | 22 | 9.5 | 98.0 | 4.5+* | | |
| 15 | 321 | | Silty Clayey SAND (SC-SM), olive brown, medium dense, moist, increasing amount and size of sand with depth, gravels in shoe. | | | 32 | | | | 33.9 | 4 |
| 20 | 316 | | Lean CLAY (CL), olive brown mottled with yellowish red, hard, moist, dark reddish brown oxidation staining, contains silt, minor gray clay on soil partings. | | | 19 | 19.9 | 99.4 | 4.5+* | | |
| 25 | 311 | | Lean CLAY (CL), brown mottled with reddish brown, hard, moist, contains fine-grained sand and silt, abundant pores, some with gray clay coatings, orgaincs, dark reddish brown oxidation staining, granular texture. | | | 24 | | | 4.5+* | | |
| 30 | | | | | | | | | | | |

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# LOG OF BORING B-1



Jones Propoerty (Roslyn Estates II)
Pleasanton, CA
4425.000.000

| | | | |
|---|---|---|---|
| Date Drilled | : 12/15/2011 | Logged/Reviewed By: | J.Botelho/ R.Skinner |
| Hole Depth (ft) | : 51 1/2 feet | Drilling Contractor | : Britton Exploration |
| Surface Elev (ft-msl) | : approximately 336 | Drilling Method | : Hollow Stem |
| Latitude (NAD83) | : 37.6675 | Hammer Type | : 140lb Auto Hammer |
| Longitude (NAD83) | : -121.8909 | Hole Diameter | : 9 inches |

| Depth in Feet | Surf. Elev. 336 | Sample Type | DESCRIPTION | Log Symbol | Water Level | Blow Count / Foot | Moisture Content (% dry weight) | Dry Unit Weight (pcf) | Unconfined Strength *field approx | Percent Passing #200 Sieve | Plasticity Index |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 30 | 306 | | Lean CLAY with sand (CL), dark brown mottled with dark yellowish brown, medium stiff, wet (no free water) fine-grained sand, few coarse-grained sand, highly porous with abundant gray clay coatings, granular texture, minor organics. | | | 12 | 18.8 | 110.5 | 0.829 | 71.2 | 10 |
| | | | Silty SAND (SM), yellowish brown, medium dense, wet, fine-grained sand, reddish brown oxidation staining, organics. | | | | | | | | |
| 35 | 301 | | Sandy CLAY (CL), dark brown, medium stiff, wet, fine-grained sand. | | | 5 | | | 0.75* | 48.3 | 12 |
| | | | Silty SAND (SM) as above | | | | | | | | |
| | | | Sandy CLAY (CL), as above | | | 9 | | | | | |
| 40 | 296 | | Lean SILT with sand (ML), grayish brown mottled with light gray and olive brown, thin layering, very stiff, moist, fine-grained sand, minor reddish brown oxidation staining. | | | 11 | | | 2.75* | | |
| | | | Lean CLAY to Lean SILT (CL-ML), grayish brown mottled with gray and dark yellowish brown, very stiff, contains organics and dark reddish brown oxidation staining, light porous texture. | | | | | | | | |
| 45 | 291 | | | | | 10 | 25.0 | 108.2 | 4.5* | | 6 |
| | | | Lean SILT with sand (ML), brown mottled with dark reddish brown and gray, very stiff, moist, fine-grained sand, light porous texture, organics. | | | | | | | | |
| 50 | 286 | | | | | 13 | | | 3.0-3.25* | | |
| | | | Bottom of boring at approximately 51 1/2 feet. Groundwater encountered at approximately 35 feet. | | | | | | | | |
| 55 | 281 | | | | | | | | | | |
| 60 | | | | | | | | | | | |

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ENGEO
Expect Excellence

# LOG OF BORING B-2



Jones Propoerty (Roslyn Estates II)
Pleasanton, CA
4425.000.000

| | | | |
|---|---|---|---|
| Date Drilled | : 12/15/2011 | Logged/Reviewed By | : J.Botelho/ R.Skinner |
| Hole Depth (ft) | : 51 1/2 feet | Drilling Contractor | : Britton Exploration |
| Surface Elev (ft-msl) | : approximately 339 | Drilling Method | : Hollow Stem |
| Latitude (NAD83) | : 37.6673 | Hammer Type | : 140lb Auto Hammer |
| Longitude (NAD83) | : -121.8902 | Hole Diameter | : 9 inches |

| Depth in Feet | Surf. Elev. 339 | Sample Type | DESCRIPTION | Log Symbol | Water Level | Blow Count / Foot | Moisture Content (% dry weight) | Dry Unit Weight (pcf) | Unconfined Strength *field approx | Percent Passing #200 Sieve | Plasticity Index |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 0 | 339 | | Surface is short grass | | | | | | | | |
| | | | Lean SILT (ML), yellowish brown, hard, dry to slightly moist, contains rootlets and fine-grained sand, porous texture. | | | | | | 4.5+* | | |
| 5 | 334 | | as above | | | 10 | 6.3 | 83.5 | | | |
| 10 | 329 | | as above, some mottling with grayish brown, contains organics and carbonate, pores with gray coatings or carbonate. | | | 17 | | | | | |
| | | | Sandy lean CLAY (CL), dark brown mottled with dark yellowish brown, hard, slightly moist, porous, contains rootlets, fine-grained sand. | | | | | | | | |
| 15 | 324 | | Sandy lean CLAY (CL), light yellowish brown mottled with brown and olive gray, hard, slightly moist, fine-grained sand and some coarse-grained sand, porous, contains roots and rootlets. | | | 27 | 8.3 | 110.0 | 4.5+*<br>4.5+* | | |
| | | | Lean SILT (ML), light yellowish brown mottled with pale olive, hard, dry, pores with light clay coatings. | | | | | | | | |
| 20 | 319 | | Lean CLAY (CL), dark brown mottled with yellowish red, hard, dry to slightly moist, minor pores with dark gray clay coatings, contains silt, rootlets, and reddish yellow oxidation staining. | | | 22 | | | 4.5+*<br>4.5+* | 97.9 | 15 |
| 25 | 314 | | as above, siltier and lighter color | | | 19 | | | 4.5+* | | |
| | | | Sandy lean CLAY (CL), light yellowish brown mottled with olive gray, hard, slightly moist, fine-grained sand, minor pores, contains clay. | | | | | | | | |
| 30 | | | | | | | | | | | |

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# LOG OF BORING B-2



Jones Propoerty (Roslyn Estates II)
Pleasanton, CA
4425.000.000

| | | | |
|---|---|---|---|
| Date Drilled | : 12/15/2011 | Logged/Reviewed By: | J.Botelho/ R.Skinner |
| Hole Depth (ft) | : 51 1/2 feet | Drilling Contractor | : Britton Exploration |
| Surface Elev (ft-msl) | : approximately 339 | Drilling Method | : Hollow Stem |
| Latitude (NAD83) | : 37.6673 | Hammer Type | : 140lb Auto Hammer |
| Longitude (NAD83) | : -121.8902 | Hole Diameter | : 9 inches |

| Depth in Feet | Surf. Elev. 339 | Sample Type | DESCRIPTION | Log Symbol | Water Level | Blow Count / Foot | Moisture Content (% dry weight) | Dry Unit Weight (pcf) | Unconfined Strength *field approx | Percent Passing #200 Sieve | Plasticity Index |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 30 | 309 | | as above, dark yellowish brown mottled with yellowish red and gray, very stiff, moist, pores with gray clay coatings, gray along soil partings, dark reddish brown oxidation staining, few rootlets. | | | 13 | 15.6 | 115.6 | 3.25* | | 12 |
| 35 | 304 | | CLAY (CL), dark brown mottled with dark yellowish red, medium stiff, wet, pores with gray clay coatings, contains reddish yellow oxidation staining. | | | 10 | 18.7 | 108.3 | 0.687 | 48.3 | 9 |
| 40 | 299 | | as above, contains carbonates, very stiff | | | | | | | | |
| | | | Sandy lean SILT/Silty SAND (ML-SM), olive brown mottled with gray, stiff/loose, wet, fine-grained sand. | | | 9 | 22.0 | | 2.25* | | 1 |
| | | | SAND (SM), brown, loose, wet, fine-to medium-grained sand. | | | 11 | | | | | |
| | | | Sandy lean SILT (ML), olive brown some gray, medium stiff, wet, fine-grained sand, some interlayering of sandier zones. | | | | | | 0.75* | | |
| 45 | 294 | | Lean CLAY to Lean SILT (CL-ML), yellowish brown motteld with reddish brown and gray, stiff to very stiff, wet, contains clay and dark reddish brown oxidation staining, granular texture at bottom of sample. | | | 11 | 34.7 | 104.3 | 2.0* | | 7 |
| 50 | 289 | | as above, very stiff | | | 18 | | | 3.5* | | |
| | | | Bottom of boring at approximately 51 1/2 feet. Groundwater encountered at approximately 35 feet. | | | | | | | | |
| 55 | 284 | | | | | | | | | | |
| 60 | | | | | | | | | | | |

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# LOG OF BORING B-3



Jones PRopoerty (Roslyn Estates II)
Pleasanton, CA
4425.000.000

| | | | |
|---|---|---|---|
| Date Drilled | : 12/16/2011 | Logged/Reviewed By | : J.Botelho/ R.Skinner |
| Hole Depth (ft) | : 23 feet | Drilling Contractor | : Britton Exploration |
| Surface Elev (ft-msl) | : approximately 338 | Drilling Method | : Hollow Stem |
| Latitude (NAD83) | : 37.6668 | Hammer Type | : 140lb Auto Hammer |
| Longitude (NAD83) | : -121.8904 | Hole Diameter | : 6 inches |

| Depth in Feet | Surf. Elev. 338 | Sample Type | DESCRIPTION | Log Symbol | Water Level | Blow Count / Foot | Moisture Content (% dry weight) | Dry Unit Weight (pcf) | Unconfined Strength *field approx | Percent Passing #200 Sieve | Plasticity Index |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 0 | 338 | | Surface is soil, approximately 1 foot below original grade of old driveway. | | | | | | | | |
| | | | Lean SILT (ML), dark brown mottled with brown, very stiff, slightly moist, fine-grained sand, contains rootlets and carbonate around rootlets, yellowish brown oxidation staining, gravels to 1 1/2 feet (FILL). | | | 7 | | | 3.0* | | |
| | | | Silty SAND (SM), yellowish brown to dark yellowish brown with reddish yellow mottling, very loose to loose, moist, fine- to medium-grained sand some coarse-grained sand, contains roots. | | | 6 | | | | 18.3 | |
| 5 | 333 | | SAND (SM), grayish brown, loose, dry, medium-to coarse-grained sand. | | | | | | | | |
| | | | Silty SAND (SM) as above | | | 4 | | | | | |
| | | | Sandy lean SILT (ML), yellowish brown, stiff, slightly moist, fine-grained sand. | | | | | | | | |
| 10 | 328 | | SAND (SM), grayish brown, loose, slightly moist, medium-to coarse-grained sand, contains silt. | | | 13 | | | | 3.7 | |
| | | | Silty SAND with gravel (SM), pale yellow, dense, dry, angular gravel. | | | | | | | | |
| 15 | 323 | | SAND with gravel and silt (SM), grayish brown, dense, moist, subangular gravel, larger gravel than above. | | | 49 | | | | | |
| | | | Sandy CLAY (CL), dark brown, very stiff, coarse-grained sand, subangular. | | | | | | | | |
| 20 | 318 | | SAND (SW), yellowish red, medium dense, moist, medium- to coarse-grained sand, contains silt. | | | 33 | | | 3.75* | 10.0 | |
| | | | Gravelly SAND (SW), brownish gray, medium dense, sry to slightly moist, medium- to coarse-grained sand, subangular gravels, contains silt and yellowish red oxidation staining. Interlayered zones approxiamtely 3 inches to 5 inches of sand and gravel and sand no gravel. | | | 25 | | | | | |
| 25 | 313 | | Bottom of boring at 23 feet. No groundwater encountered. | | | | | | | | |
| 30 | | | | | | | | | | | |

02-03-2012 G:\Active Projects\4425\4425000000\GEX\Boring Logs\B-3.bor

ENGEO
Expect Excellence

# LOG OF BORING B-4



Jones PRopoerty (Roslyn Estates II)
Pleasanton, CA
4425.000.000

| | | | |
|---|---|---|---|
| Date Drilled | : 12/15/2011 | Logged/Reviewed By | : J.Botelho/ R.Skinner |
| Hole Depth (ft) | : 21 1/2 feet | Drilling Contractor | : Britton Exploration |
| Surface Elev (ft-msl) | : approximately 337 | Drilling Method | : Hollow Stem |
| Latitude (NAD83) | : 37.6671 | Hammer Type | : 140lb Auto Hammer |
| Longitude (NAD83) | : -121.8910 | Hole Diameter | : 6 inches |

| Depth in Feet | Surf. Elev. 337 | Sample Type | DESCRIPTION | Log Symbol | Water Level | Blow Count / Foot | Moisture Content (% dry weight) | Dry Unit Weight (pcf) | Unconfined Strength *field approx | Percent Passing #200 Sieve | Plasticity Index |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 0 | 337 | | Surface is short grass | | | | | | | | |
| | | | Lean CLAY to Lean SILT (CL-ML), dark yellowish brown mottled with olive brown, very stiff, slightly moist, porous, contains rootlets and fine-grained sand. | | | 8 | 7.8 | 84.2 | 3.5* | 71.2 | 5 |
| 5 | 332 | | Silty SAND (SM), dark yellowish brown, very loose, moist, fine- to medium-grained sand, porous, contains rootlets and black organics. | | | 5 | 7.7 | 87.1 | 2.75 | | |
| | | | Silty SAND (SM), dark yellowish brown to very loose, moist, fine- to medium-grained sand, contains pores, rootlets, and black organics. | | | | | | | | |
| 10 | 327 | | SILT with sand (ML), dark brown mottled with dark yellowish brown and some dark gray, hard, moist, fine-grained sand, some coarse-grained sand , porous with dark gray clay coatings, contains black organics. | | | 14 | | | 4.5+* | | |
| 15 | 322 | | Silty SAND (SM), yellowish brown mottled with gray, moist, medium dense, fine- to coarse-grained sand, reddish brown oxidation staining, contains organics and rootlets. | | | 20 | | | | 21.2 | |
| 20 | 317 | | Lean CLAY (CL), dark brown mottled with dark yellowish brown and dark olive gray, stiff to very stiff, moist to wet (some free water at top of sample), gray clay stringers, black oxidation staining and organics. | | | 17 | | | 1.5-2.25* | | |
| | | | Bottom of boring at approximately 21 1/2 feet. Perched groundwater encountered at approximately 20 feet. | | | | | | | | |
| 25 | 312 | | | | | | | | | | |
| 30 | | | | | | | | | | | |

02-03-2012 G:\Active Projects\4425\4425000000\GEX\Boring Logs\B-4.bor

# APPENDIX B
# LABORATORY TEST DATA

**Summary of Laboratory Results**
**Liquid and Plastic Limits Test Report (2 pages)**
**Unconfined Compression Test**
**Particle Size Distribution Report (9 pages)**
**Sulfate Test Report**

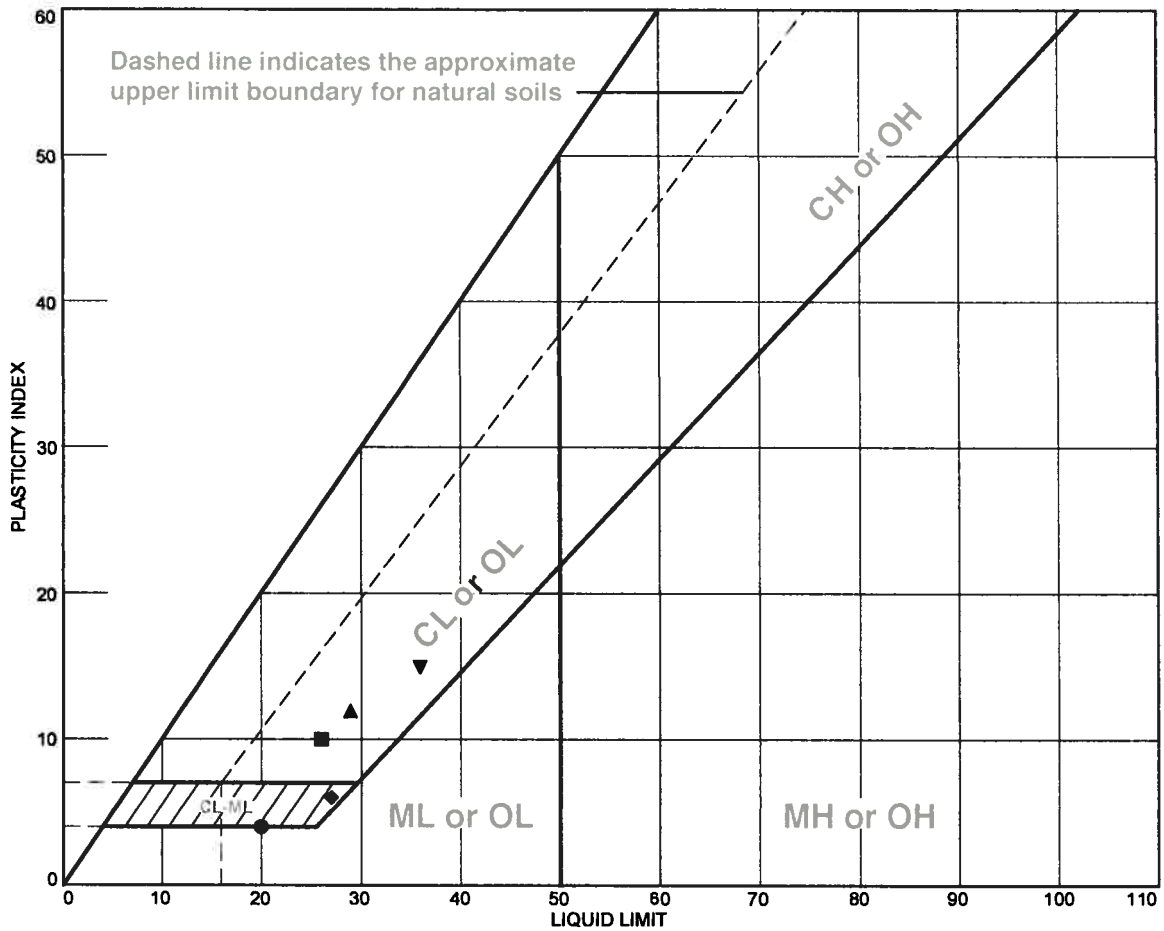
APPENDIX B

# Summary of Laboratory Testing

| Boring Number | Depth Feet | In-Place | | Atterberg Limits | | | Unconfined Compressive Strength tsf | Gradation (Percent Passing No. 200 Sieve) | Sulfate (% by weight) |
|---|---|---|---|---|---|---|---|---|---|
| | | Dry Unit Weight Pcf | Moisture Content % | Liquid Limit % | Plastic Limit % | Plasticity Index | | | |
| B-1 | 2 | 91.0 | 5.2 | | | | | | |
| B-1 | 11 | 98.0 | 9.5 | | | | | | |
| B-1 | 16 | | | 20 | 16 | 4 | | 33.9 | |
| B-1 | 21 | 99.4 | 19.9 | | | | | | |
| B-1 | 31 | 110.5 | 18.84 | 26 | 16 | 10 | 0.829 | 71.2 | |
| B-1 | 35 | | | | | | | 48.3 | |
| B-1 | 36 | | | 29 | 17 | 12 | | | |
| B-1 | 45.5 | | | 27 | 21 | 6 | | | |
| B-1 | 46 | 108.2 | 25.0 | | | | | | |
| B-2 | 6 | 83.5 | 6.3 | | | | | | |
| B-2 | 16 | 110.0 | 8.3 | | | | | | |
| B-2 | 21 | | | 36 | 21 | 15 | | 97.9 | |
| B-2 | 31 | 115.6 | 15.6 | 26 | 14 | 12 | | | |
| B-2 | 36 | 108.3 | 18.73 | 27 | 18 | 9 | 0.687 | | |
| B-2 | 41 | | 22.0 | 21 | 20 | 1 | | | |
| B-2 | 46 | 104.3 | 34.7 | 26 | 19 | 7 | | | |
| B-3 | 6 | | | | | | | 18.3 | |
| B-3 | 11 | | | | | | | 3.7 | |
| B-3 | 21.5 | | | | | | | 10.0 | |
| B-4 | 2 | 84.2 | 7.8 | 25 | 20 | 5 | | 71.2 | 0.002 |
| B-4 | 6 | 87.1 | 7.7 | | | | | | |
| B-4 | 16 | | | | | | | 21.2 | |

4425.000.000
February 3, 2012

# LIQUID AND PLASTIC LIMITS TEST REPORT




| | MATERIAL DESCRIPTION | LL | PL | PI | %<#40 | %<#200 | USCS |
|---|---|---|---|---|---|---|---|
| ● | See exploration logs. | 20 | 16 | 4 | 58.9 | 33.9 | SC-SM |
| ■ | See exploration logs. | 26 | 16 | 10 | 97.5 | 71.2 | CL |
| ▲ | See exploration logs. | 29 | 17 | 12 | | | CL |
| ◆ | See exploration logs. | 27 | 21 | 6 | | | CL-ML |
| ▼ | See exploration logs. | 36 | 21 | 15 | 100.0 | 97.9 | CL |

**Project No.** 4425.000.000 **Client:** Lynden Homes

**Project:** Jones Property - Pleasanton, CA

**Remarks:**

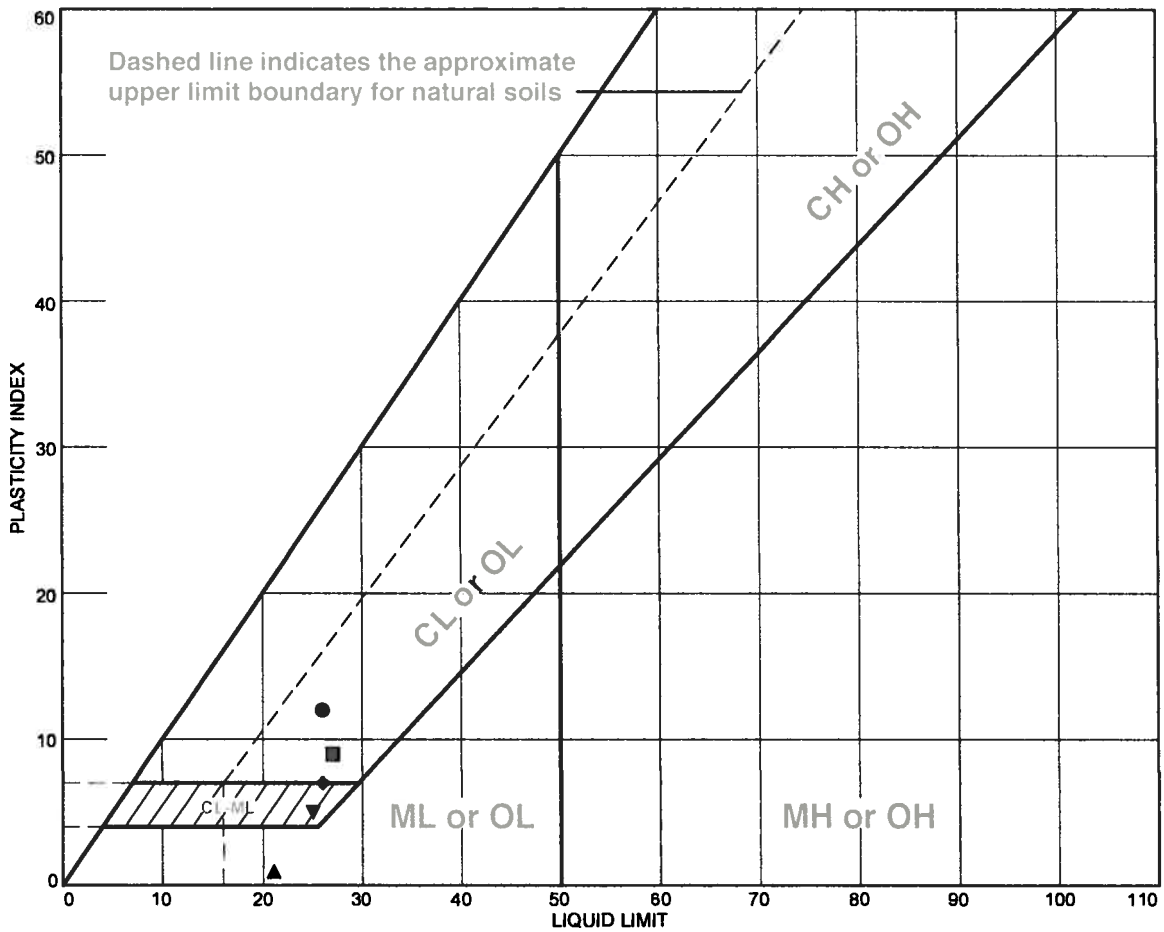
● **Depth:** 16 feet **Sample Number:** B-1 @ 16'
■ **Depth:** 31 feet **Sample Number:** B-1 @ 31'
▲ **Depth:** 36 feet **Sample Number:** B-1 @ 36'
◆ **Depth:** 45.5 feet **Sample Number:** B-1 @ 45.5'
▼ **Depth:** 21 feet **Sample Number:** B-2 @ 21'

ENGEO INCORPORATED

**Figure**

**Tested By:** ○ GC □ GC △ GC ◇ GC ▽ NN **Checked By:** DS

# LIQUID AND PLASTIC LIMITS TEST REPORT




| | MATERIAL DESCRIPTION | LL | PL | PI | %<#40 | %<#200 | USCS |
|---|---|---|---|---|---|---|---|
| ● | See exploration logs. | 26 | 14 | 12 | | | CL |
| ■ | See exploration logs. | 27 | 18 | 9 | | | CL |
| ▲ | See exploration logs. | 21 | 20 | 1 | | | ML-SM |
| ◆ | See exploration logs. | 26 | 19 | 7 | | | CL-ML |
| ▼ | See exploration logs. | 25 | 20 | 5 | 100.0 | 71.2 | CL-ML |

**Project No.** 4425.000.000 **Client:** Lynden Homes

**Project:** Jones Property - Pleasanton, CA

**Remarks:**

● **Depth:** 31 feet **Sample Number:** B-2 @ 31'
■ **Depth:** 36 feet **Sample Number:** B-2 @ 36'
▲ **Depth:** 41 feet **Sample Number:** B-2 @ 41'
◆ **Depth:** 46 feet **Sample Number:** B-2 @ 46'
▼ **Depth:** 2 feet **Sample Number:** B-4 @ 2'

ENGEO INCORPORATED

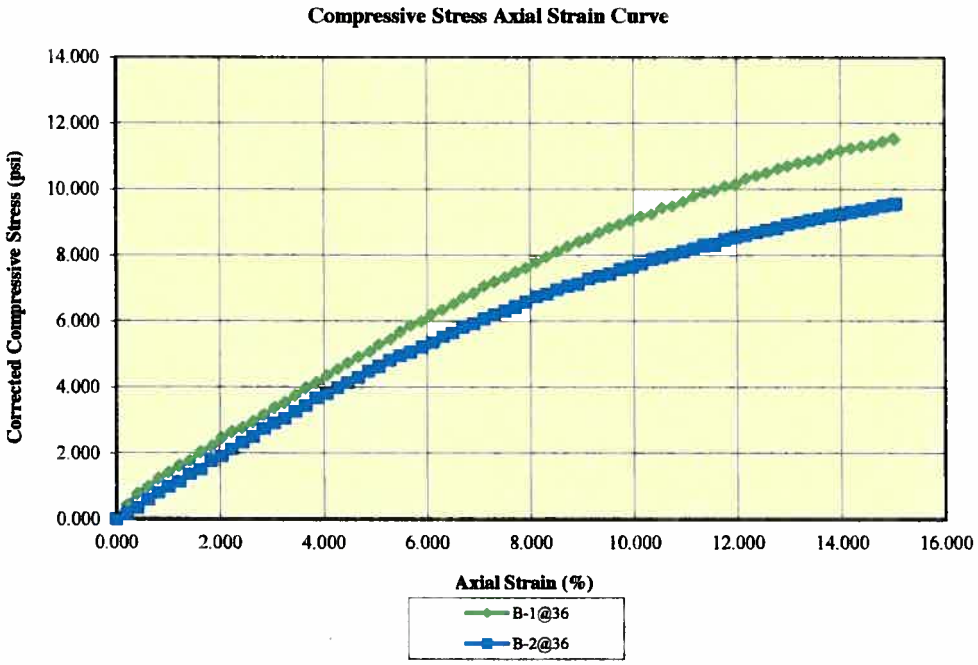
**Figure**

**Tested By:** ○ NN △ GC **Checked By:** DS

# ENGEO Incorporated

## Unconfined Compression Test Report (ASTM D2166)

Date 12/27
Checked By G. Criste
Date 12.27.11
Computed By D. Seibold
Date 12.27.11
Tested By D. Seibold

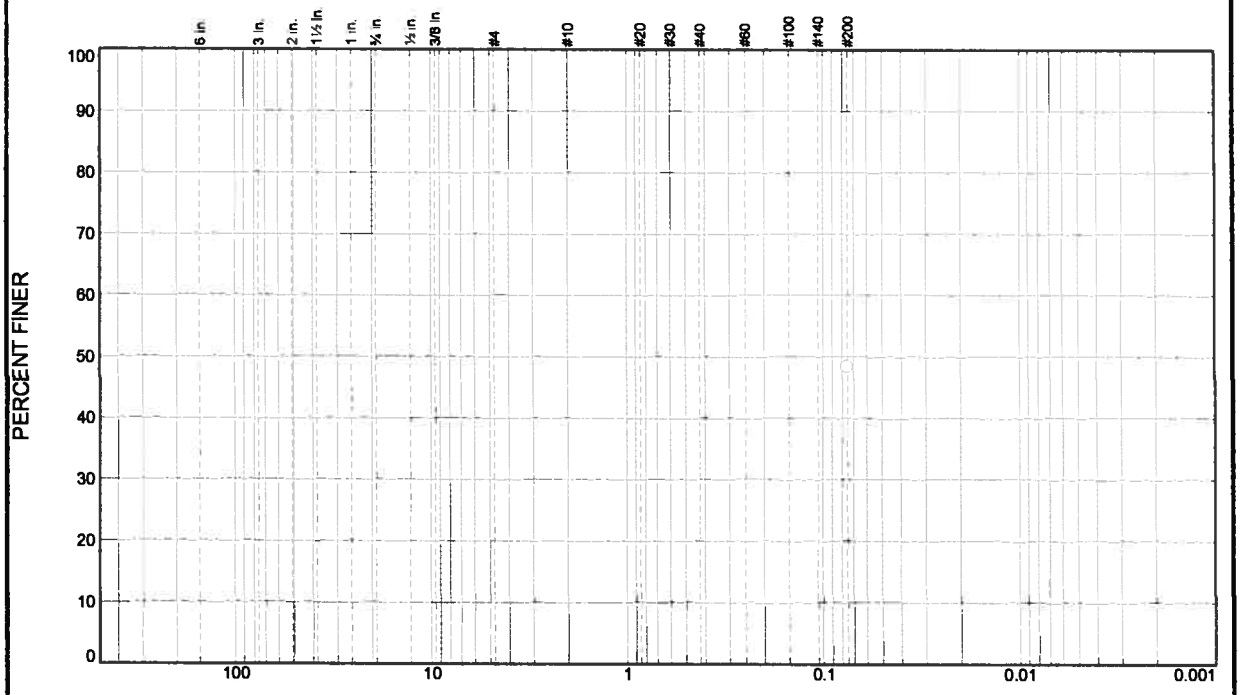



| Before Test | Specimen | | | |
|---|---|---|---|---|
| | B-1@31 | B-2@36 | | |
| Water Content (%) | 18.84 | 18.73 | | |
| Dry Density (pcf) | 110.500 | 108.300 | | |
| Saturation (%) | 100.50 | 94.06 | | |
| Void Ratio | 0.50 | 0.53 | | |
| Diameter (in) | 2.420 | 2.380 | | |
| Height (in) | 5.000 | 5.000 | | |
| **Test Data** | **B-1@31** | **B-2@36** | | |
| Unconfined Compressive Strength (psi) | 11.523 | 9.549 | | |
| Unconfined Compressive Strength (tsf) | 0.829 | 0.687 | | |
| Undrained Shear Strength (psi) | 5.761 | 4.775 | | |
| Rate of Strain (in/min) | 0.050000 | 0.050000 | | |
| Strain at Failure (%) | 15.02 | 15.06 | | |
| Description | | | | |

| Project Information | | Specimen Description | |
|---|---|---|---|
| Project Num | 4425000000 | B-1@31 | See exploration logs. |
| Project | Jones Property, Phase 002 | B-2@36 | See exploration logs. |
| Sampling Date | | | |
| Sample # | Various | | |
| Client | Lynden Homes | Test Variables | |
| | | Specific Gravity | 2.65 |
| | | Liquid Limit: | |
| | | Plastic Limit: | |

Remarks

# Particle Size Distribution Report




| % +3" | % Gravel | | % Sand | | | % Fines | |
|---|---|---|---|---|---|---|---|
| | Coarse | Fine | Coarse | Medium | Fine | Silt | Clay |
| | | | | | | | 48.3 |

| SIEVE SIZE | PERCENT FINER | SPEC.* PERCENT | PASS? (X=NO) |
|---|---|---|---|
| #200 | 48.3 | | |

* (no specification provided)

**Soil Description**

See exploration logs.

**Atterberg Limits**

PL= LL= PI=

**Coefficients**

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

**Classification**

USCS= AASHTO=

**Remarks**

**Sample Number:** B-1 @ 35' **Depth:** 35 feet

**Date:** 12/27/11

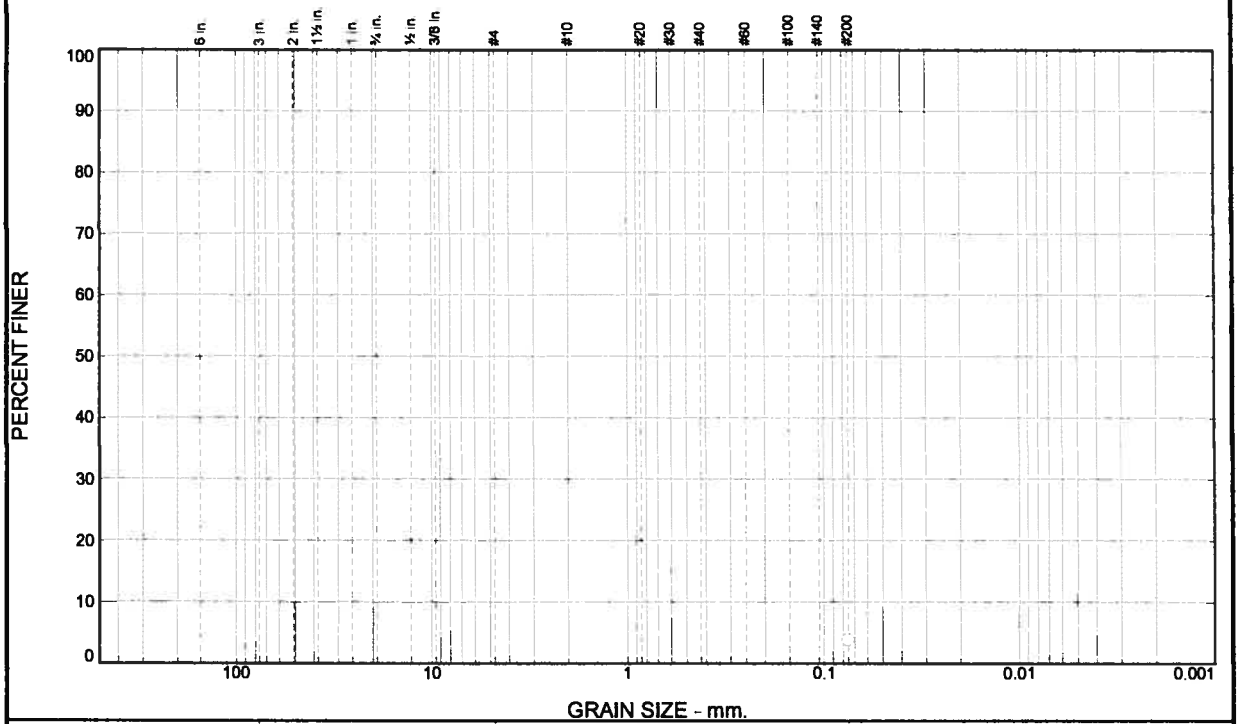
ENGEO INCORPORATED

**Client:** Lynden Homes
**Project:** Jones Property - Pleasanton, CA

**Project No:** 4425.000.000

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

# Particle Size Distribution Report




| % +3" | % Gravel | | % Sand | | | % Fines | |
|---|---|---|---|---|---|---|---|
| | Coarse | Fine | Coarse | Medium | Fine | Silt | Clay |
| | | | | | | | 3.7 |

| SIEVE SIZE | PERCENT FINER | SPEC.* PERCENT | PASS? (X=NO) |
|---|---|---|---|
| #200 | 3.7 | | |

* (no specification provided)

**Soil Description**

See exploration logs.

**Atterberg Limits**

PL= LL= PI=

**Coefficients**

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

**Classification**

USCS= AASHTO=

**Remarks**

**Sample Number:** B-3 @ 11' **Depth:** 11 feet

**Date:** 12/27/11

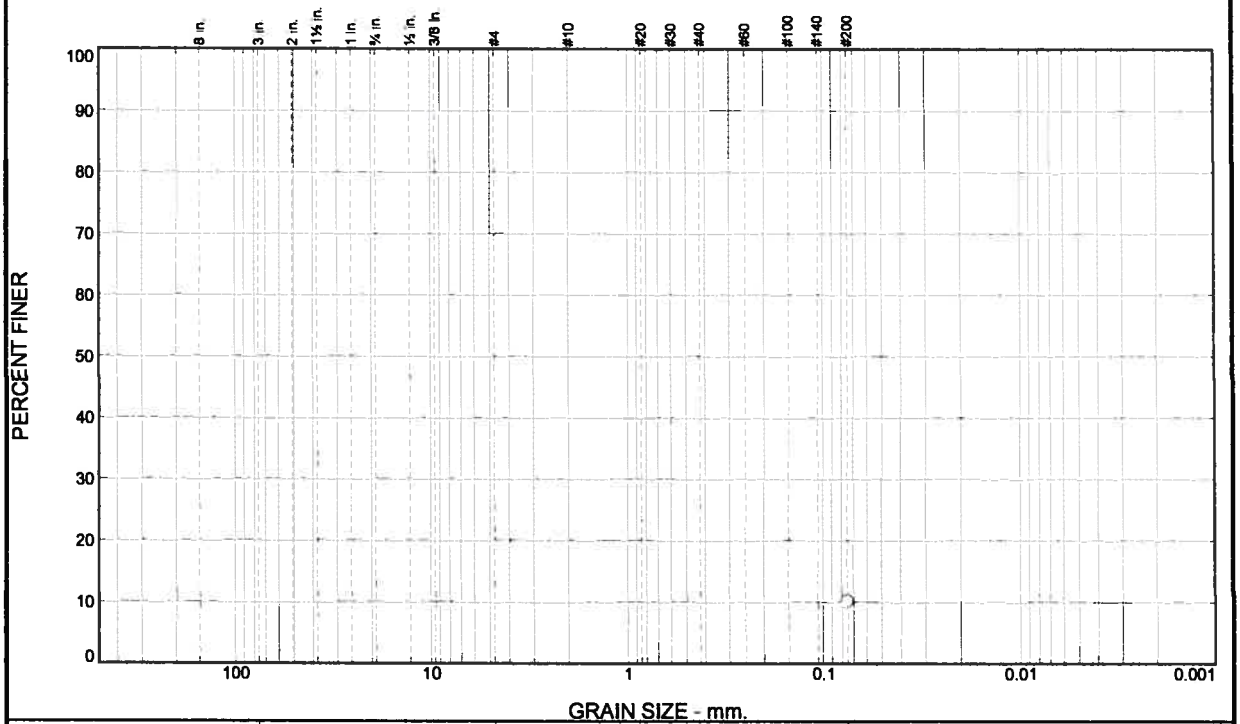
ENGEO INCORPORATED

**Client:** Lynden Homes
**Project:** Jones Property - Pleasanton, CA

**Project No:** 4425.000.000

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

# Particle Size Distribution Report




| % +3" | % Gravel | | % Sand | | | % Fines | |
|---|---|---|---|---|---|---|---|
| | Coarse | Fine | Coarse | Medium | Fine | Silt | Clay |
| | | | | | | | 10.0 |

| SIEVE SIZE | PERCENT FINER | SPEC.* PERCENT | PASS? (X=NO) |
|---|---|---|---|
| #200 | 10.0 | | |

* (no specification provided)

**Soil Description**

See exploration logs.

**Atterberg Limits**

PL= LL= PI=

**Coefficients**

$D_{90}$= $D_{85}$= $D_{60}$=
$D_{50}$= $D_{30}$= $D_{15}$=
$D_{10}$= 0.0750 $C_u$= $C_c$=

**Classification**

USCS= AASHTO=

**Remarks**

**Sample Number:** B-3 @ 21.5' **Depth:** 21.5 feet

**Date:** 12/27/11

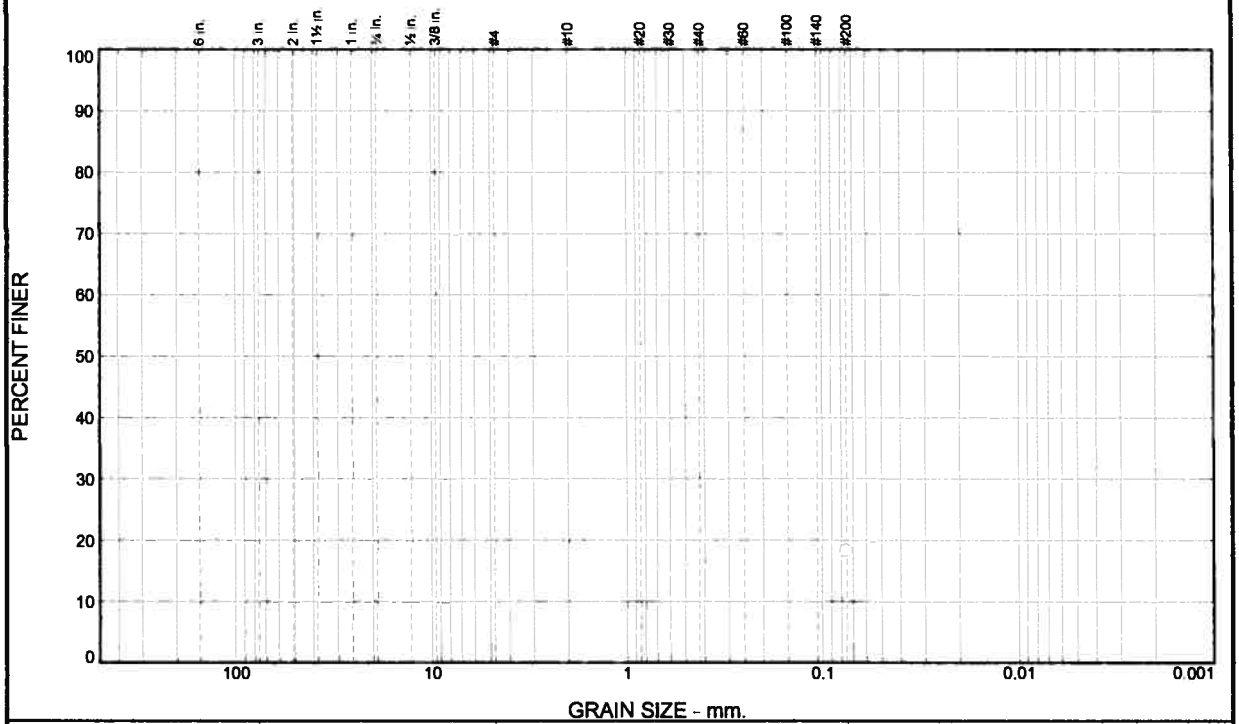
ENGEO INCORPORATED

**Client:** Lynden Homes
**Project:** Jones Property - Pleasanton, CA

**Project No:** 4425.000.000

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

# Particle Size Distribution Report




| % +3" | % Gravel | | % Sand | | | % Fines | |
|---|---|---|---|---|---|---|---|
| | Coarse | Fine | Coarse | Medium | Fine | Silt | Clay |
| | | | | | | | 18.3 |

| SIEVE SIZE | PERCENT FINER | SPEC.* PERCENT | PASS? (X=NO) |
|---|---|---|---|
| #200 | 18.3 | | |

* (no specification provided)

**Soil Description**

See exploration logs.

**Atterberg Limits**

PL= LL= PI=

**Coefficients**

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

**Classification**

USCS= AASHTO=

**Remarks**

**Sample Number:** B-3 @ 6' **Depth:** 6 feet

**Date:** 12/27/11

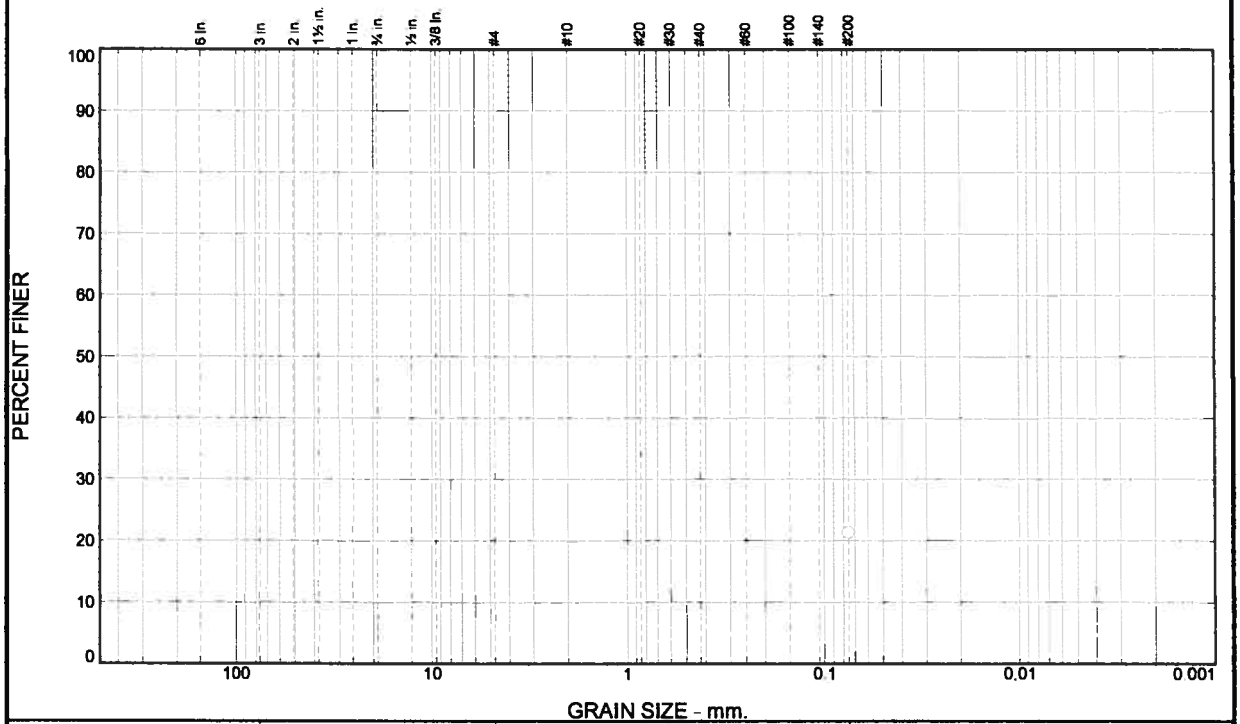
ENGEO INCORPORATED

**Client:** Lynden Homes
**Project:** Jones Property - Pleasanton, CA

**Project No:** 4425.000.000

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

# Particle Size Distribution Report




| % +3" | % Gravel | | % Sand | | | % Fines | |
|---|---|---|---|---|---|---|---|
| | Coarse | Fine | Coarse | Medium | Fine | Silt | Clay |
| | | | | | | | 21.2 |

| SIEVE SIZE | PERCENT FINER | SPEC.* PERCENT | PASS? (X=NO) |
|---|---|---|---|
| #200 | 21.2 | | |

* (no specification provided)

**Soil Description**

See exploration logs.

**Atterberg Limits**

PL= LL= PI=

**Coefficients**

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

**Classification**

USCS= AASHTO=

**Remarks**

**Sample Number:** B-4 @ 16' **Depth:** 16 feet

**Date:** 12/27/11

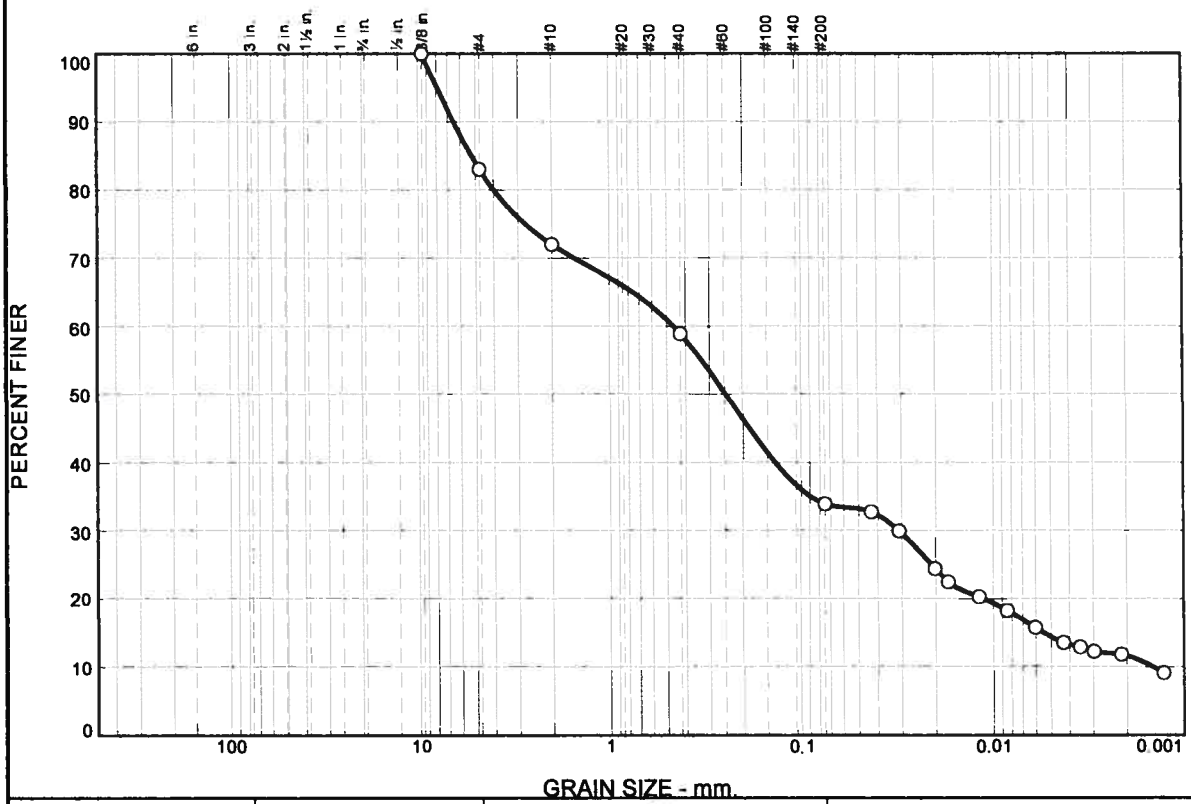
ENGEO INCORPORATED

**Client:** Lynden Homes
**Project:** Jones Property - Pleasanton, CA

**Project No:** 4425.000.000

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

# Particle Size Distribution Report




| % +3" | % Gravel | | % Sand | | | % Fines | |
|---|---|---|---|---|---|---|---|
| | Coarse | Fine | Coarse | Medium | Fine | Silt | Clay |
| 0.0 | 0.0 | 16.9 | 11.1 | 13.1 | 25.0 | 19.5 | 14.4 |

| SIEVE SIZE | PERCENT FINER | SPEC.* PERCENT | PASS? (X=NO) |
|---|---|---|---|
| 3/8 | 100.0 | | |
| #4 | 83.1 | | |
| #10 | 72.0 | | |
| #40 | 58.9 | | |
| #200 | 33.9 | | |
| 0.0431 mm. | 32.8 | | |
| 0.0309 mm. | 30.0 | | |
| 0.0201 mm. | 24.4 | | |
| 0.0172 mm. | 22.5 | | |
| 0.0119 mm. | 20.3 | | |
| 0.0084 mm. | 18.3 | | |
| 0.0060 mm. | 15.8 | | |
| 0.0043 mm. | 13.6 | | |
| 0.0035 mm. | 12.9 | | |
| 0.0030 mm. | 12.3 | | |
| 0.0021 mm. | 11.8 | | |
| 0.0013 mm. | 9.1 | | |

* (no specification provided)

**Soil Description**

See exploration logs.

**Atterberg Limits**

PL= 16 LL= 20 PI= 4

**Coefficients**

$D_{90}$= 6.4859 $D_{85}$= 5.2234 $D_{60}$= 0.4634
$D_{50}$= 0.2467 $D_{30}$= 0.0310 $D_{15}$= 0.0054
$D_{10}$= 0.0015 $C_u$= 318.03 $C_c$= 1.42

**Classification**

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

**Remarks**

Specific gravity = 2.879

**Sample Number:** B-1 @ 16' **Depth:** 16 feet

**Date:** 12/23/11

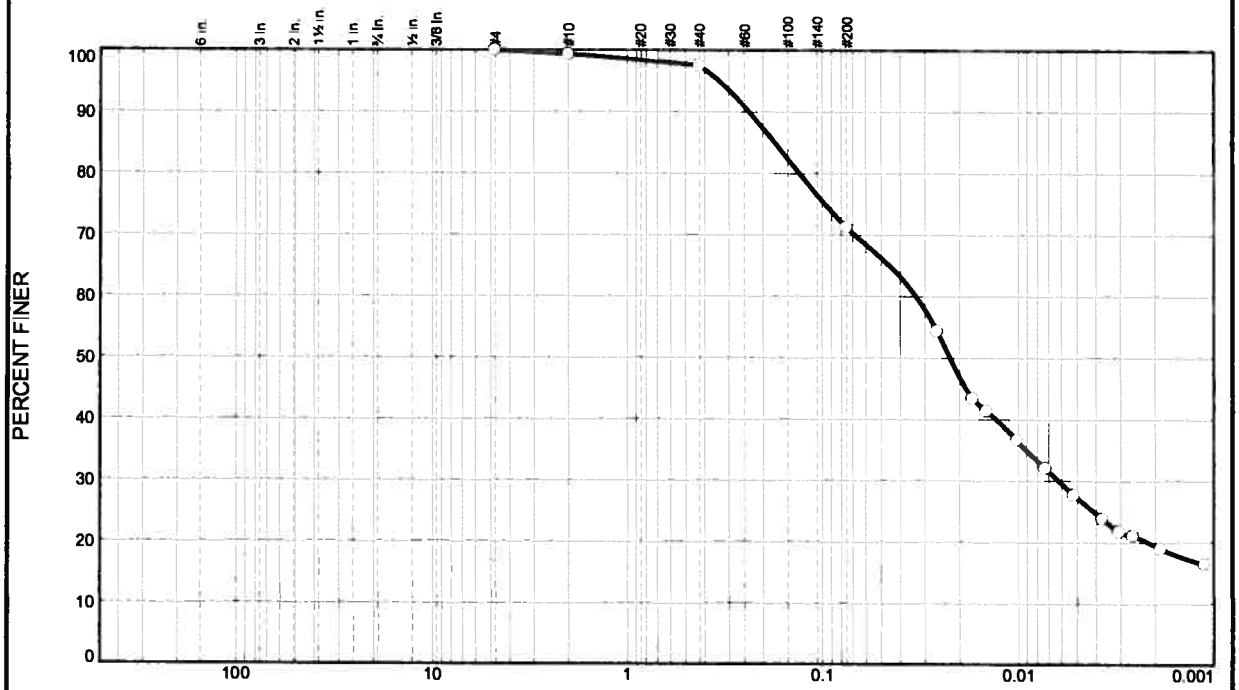
ENGEO INCORPORATED

**Client:** Lynden Homes
**Project:** Jones Property - Pleasanton, CA

**Project No:** 4425.000.000

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

# Particle Size Distribution Report




| % +3" | % Gravel | | % Sand | | | % Fines | |
|---|---|---|---|---|---|---|---|
| | Coarse | Fine | Coarse | Medium | Fine | Silt | Clay |
| 0.0 | 0.0 | 0.0 | 0.7 | 1.8 | 26.3 | 44.0 | 27.2 |

| SIEVE SIZE | PERCENT FINER | SPEC.* PERCENT | PASS? (X=NO) |
|---|---|---|---|
| #4 | 100.0 | | |
| #10 | 99.3 | | |
| #40 | 97.5 | | |
| #200 | 71.2 | | |
| 0.0260 mm. | 54.3 | | |
| 0.0172 mm. | 43.4 | | |
| 0.0146 mm. | 41.4 | | |
| 0.0101 mm. | 36.6 | | |
| 0.0073 mm. | 32.0 | | |
| 0.0052 mm. | 27.7 | | |
| 0.0037 mm. | 23.8 | | |
| 0.0031 mm. | 21.7 | | |
| 0.0026 mm. | 21.0 | | |
| 0.0019 mm. | 19.0 | | |
| 0.0011 mm. | 16.4 | | |

* (no specification provided)

**Soil Description**

See exploration logs.

**Atterberg Limits**

PL= 16 LL= 26 PI= 10

**Coefficients**

$D_{90}$= 0.2356 $D_{85}$= 0.1747 $D_{60}$= 0.0333
$D_{50}$= 0.0224 $D_{30}$= 0.0063 $D_{15}$=
$D_{10}$= $C_u$= $C_c$=

**Classification**

USCS= CL AASHTO= A-4(5)

**Remarks**

Specific gravity = 3.461

**Sample Number:** B-1 @ 31' **Depth:** 31 feet

**Date:** 12/23/11

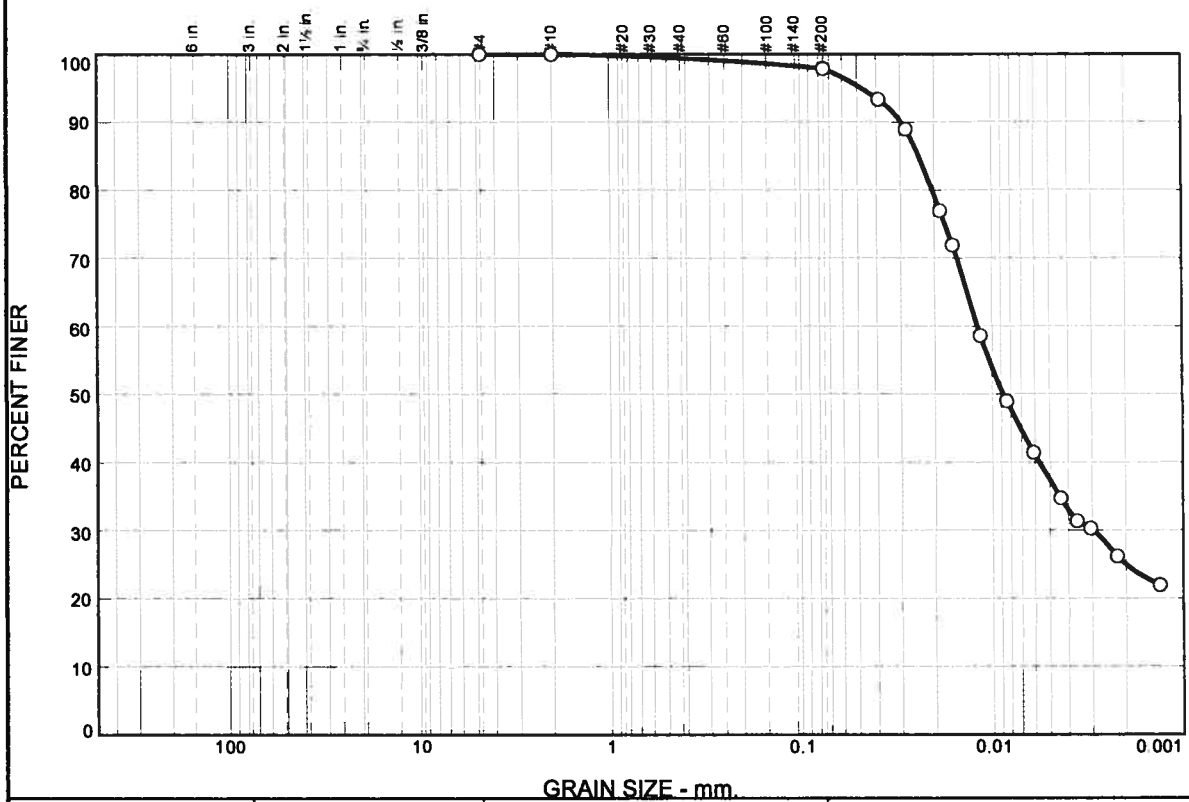
ENGEO INCORPORATED

**Client:** Lynden Homes
**Project:** Jones Property - Pleasanton, CA

**Project No:** 4425.000.000

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

# Particle Size Distribution Report




| % +3" | % Gravel | | % Sand | | | % Fines | |
|---|---|---|---|---|---|---|---|
| | Coarse | Fine | Coarse | Medium | Fine | Silt | Clay |
| 0.0 | 0.0 | 0.0 | 0.0 | 0.6 | 1.5 | 60.4 | 37.5 |

| SIEVE SIZE | PERCENT FINER | SPEC.* PERCENT | PASS? (X=NO) |
|---|---|---|---|
| #4 | 100.0 | | |
| #10 | 100.0 | | |
| #200 | 97.9 | | |
| 0.0387 mm. | 93.4 | | |
| 0.0279 mm. | 89.0 | | |
| 0.0186 mm. | 77.0 | | |
| 0.0160 mm. | 71.9 | | |
| 0.0115 mm. | 58.6 | | |
| 0.0084 mm. | 49.0 | | |
| 0.0061 mm. | 41.4 | | |
| 0.0044 mm. | 34.8 | | |
| 0.0036 mm. | 31.4 | | |
| 0.0031 mm. | 30.3 | | |
| 0.0022 mm. | 26.2 | | |
| 0.0013 mm. | 22.0 | | |

* (no specification provided)

**Soil Description**

See exploration logs.

**Atterberg Limits**

PL= 21 LL= 36 PI= 15

**Coefficients**

$D_{90}$= 0.0294 $D_{85}$= 0.0239 $D_{60}$= 0.0119
$D_{50}$= 0.0087 $D_{30}$= 0.0030 $D_{15}$=
$D_{10}$= $C_u$= $C_c$=

**Classification**

USCS= CL AASHTO= A-6(15)

**Remarks**

**Sample Number:** B-2 @ 21' **Depth:** 21 feet

**Date:** 12.27.11

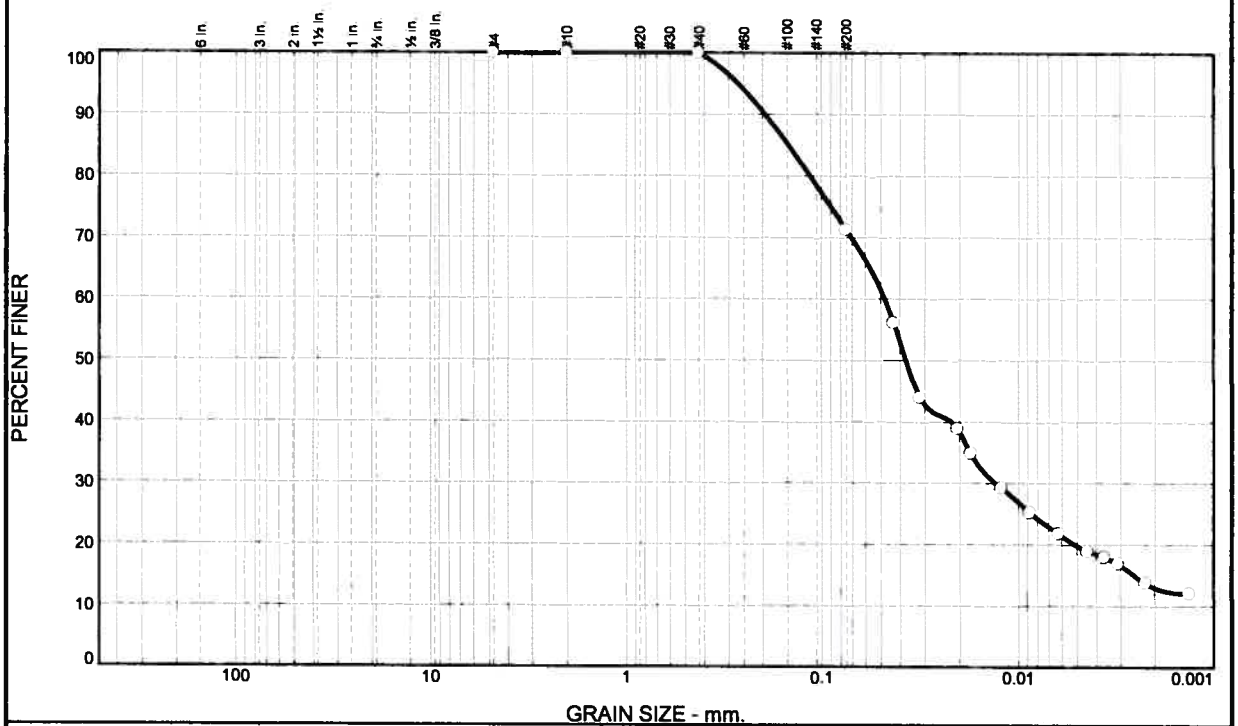
ENGEO INCORPORATED

**Client:** Lynden Homes
**Project:** Jones Property - Pleasanton, CA

**Project No:** 4425.000.000

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

# Particle Size Distribution Report




| % +3" | % Gravel | | % Sand | | | % Fines | |
|---|---|---|---|---|---|---|---|
| | Coarse | Fine | Coarse | Medium | Fine | Silt | Clay |
| 0.0 | 0.0 | 0.0 | 0.0 | 0.0 | 28.8 | 51.4 | 19.8 |

| SIEVE SIZE | PERCENT FINER | SPEC.* PERCENT | PASS? (X=NO) |
|---|---|---|---|
| #4 | 100.0 | | |
| #10 | 100.0 | | |
| #40 | 100.0 | | |
| #200 | 71.2 | | |
| 0.0430 mm. | 56.1 | | |
| 0.0318 mm. | 43.9 | | |
| 0.0205 mm. | 38.9 | | |
| 0.0175 mm. | 34.8 | | |
| 0.0122 mm. | 29.2 | | |
| 0.0087 mm. | 25.2 | | |
| 0.0062 mm. | 21.8 | | |
| 0.0044 mm. | 18.9 | | |
| 0.0036 mm. | 17.9 | | |
| 0.0031 mm. | 16.9 | | |
| 0.0022 mm. | 13.7 | | |
| 0.0013 mm. | 12.0 | | |

* (no specification provided)

**Soil Description**

See exploration logs.

**Atterberg Limits**

PL= 20 LL= 25 PI= 5

**Coefficients**

$D_{90}$= 0.1949 $D_{85}$= 0.1482 $D_{60}$= 0.0478
$D_{50}$= 0.0375 $D_{30}$= 0.0131 $D_{15}$= 0.0025
$D_{10}$= $C_u$= $C_c$=

**Classification**

USCS= CL-ML AASHTO= A-4(2)

**Remarks**

**Sample Number:** B-4 @ 2' **Depth:** 2 feet

**Date:** 12/23/11

ENGEO INCORPORATED

**Client:** Lynden Homes
**Project:** Jones Property - Pleasanton, CA

**Project No:** 4425.000.000

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

# *EN*GEO Incorporated

## SULFATE CONTENT RESULT(S)
### CALTRANS Test Method 417

**Project Name:** Jones Property
**Client:** Lynden Homes
**Tested by:** TB

**Project Number:** 4425.000.000

**Date:** December 23, 2011

| Sample Number | Sample Location | Matrix | Water Soluble Sulfate ($SO_4$) in Soil | |
|---|---|---|---|---|
| | | | mg/kg | % by Weight |
| 1 | B-4@2' | soil | 18 | 0.002 |
| | | | | |

# SULFATE CONTENT RESULT(S)

## CALTRANS Test Method 417

**Project Name:** Jones Property
**Client:** Lynden Homes
**Tested by:** TB

**Project Number:** 4425.000.000
**Date:** December 23, 2011

| Sample Number | Sample Location | Matrix | Water Soluble Sulfate ($SO_4$) in Soil | |
|---|---|---|---|---|
| | | | mg/kg | % by Weight |
| 1 | B-4@2' | soil | 18 | 0.002 |

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

ENGEO INCORPORATED

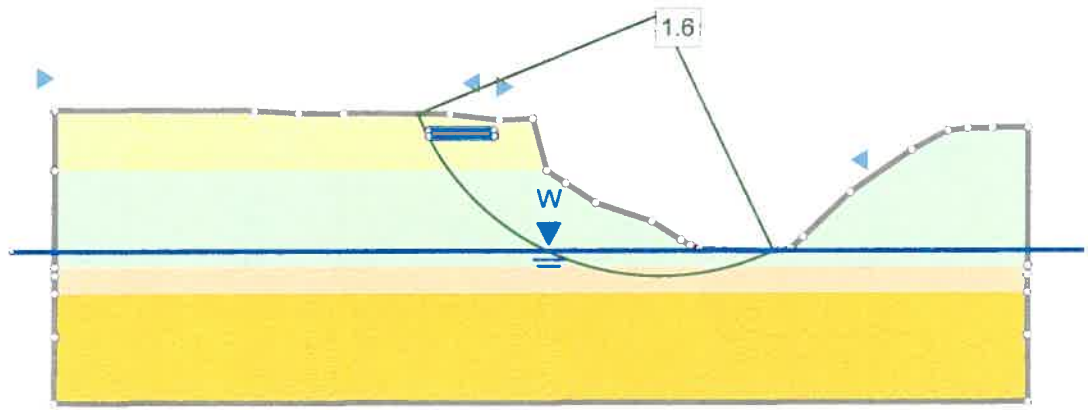
# APPENDIX C

## Slope Stability Analysis

APPENDIX C

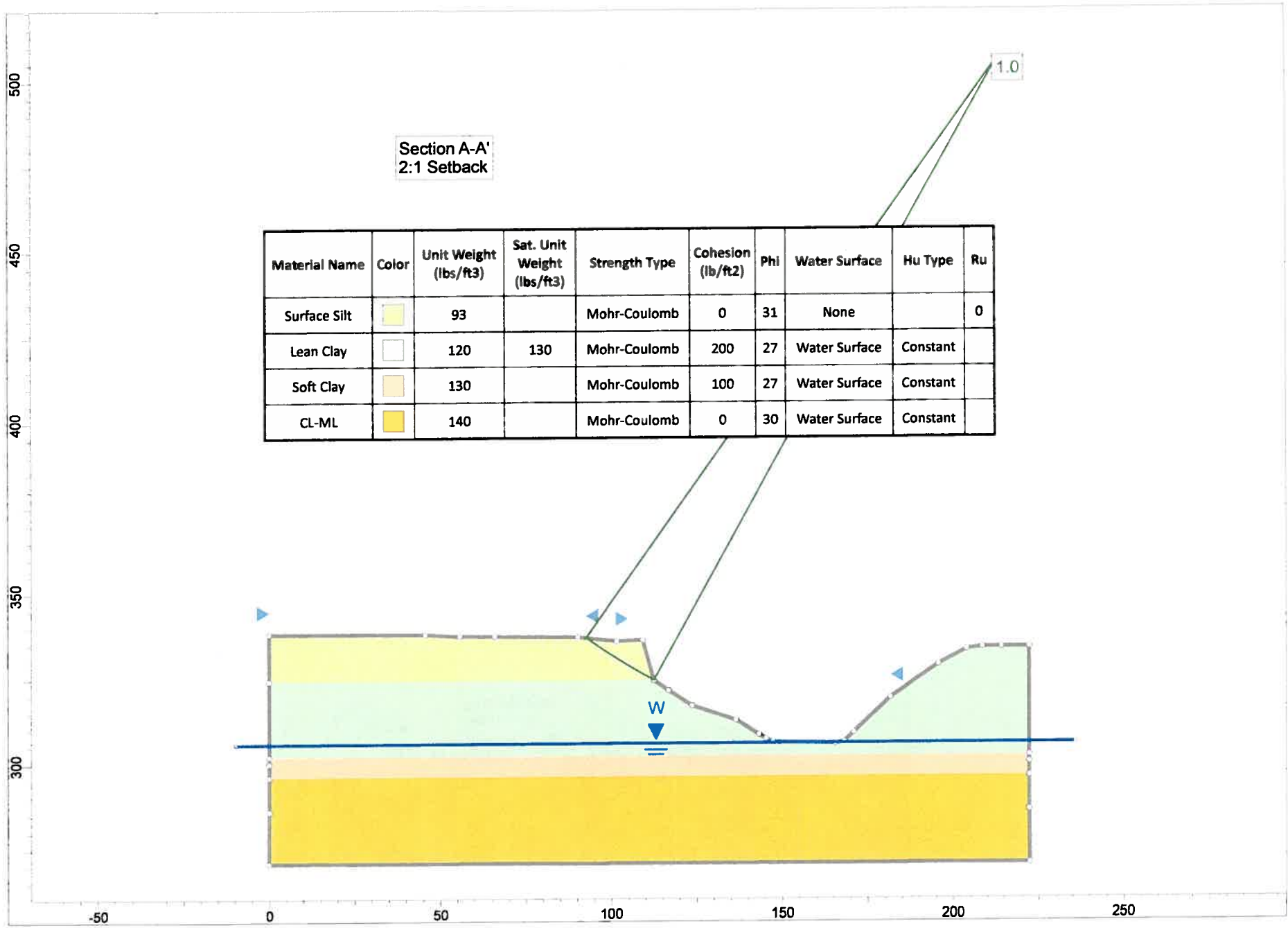
Section A-A'
2:1 Setback
Reinforced

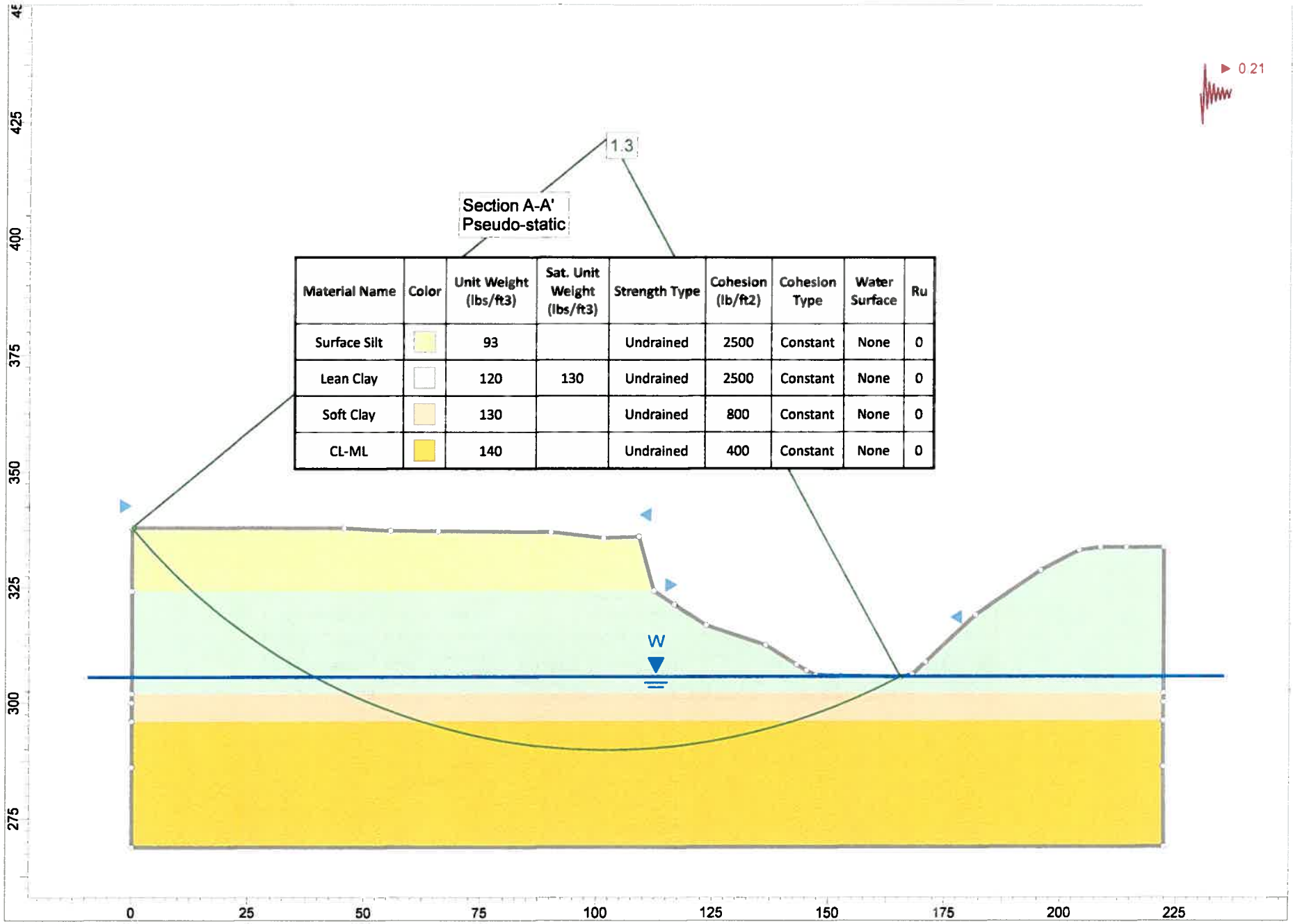
| Material Name | Color | Unit Weight (lbs/ft3) | Sat. Unit Weight (lbs/ft3) | Strength Type | Cohesion (lb/ft2) | Phi | Water Surface | Hu Type | Ru |
|---|---|---|---|---|---|---|---|---|---|
| Surface Silt | | 93 | | Mohr-Coulomb | 0 | 31 | None | | 0 |
| Lean Clay | | 120 | 130 | Mohr-Coulomb | 200 | 27 | Water Surface | Constant | |
| Soft Clay | | 130 | | Mohr-Coulomb | 100 | 27 | Water Surface | Constant | |
| CL-ML | | 140 | | Mohr-Coulomb | 0 | 30 | Water Surface | Constant | |



Section A-A'
2:1 Setback

| Material Name | Color | Unit Weight (lbs/ft3) | Sat. Unit Weight (lbs/ft3) | Strength Type | Cohesion (lb/ft2) | Phi | Water Surface | Hu Type | Ru |
|---|---|---|---|---|---|---|---|---|---|
| Surface Silt | | 93 | | Mohr-Coulomb | 0 | 31 | None | | 0 |
| Lean Clay | | 120 | 130 | Mohr-Coulomb | 200 | 27 | Water Surface | Constant | |
| Soft Clay | | 130 | | Mohr-Coulomb | 100 | 27 | Water Surface | Constant | |
| CL-ML | | 140 | | Mohr-Coulomb | 0 | 30 | Water Surface | Constant | |

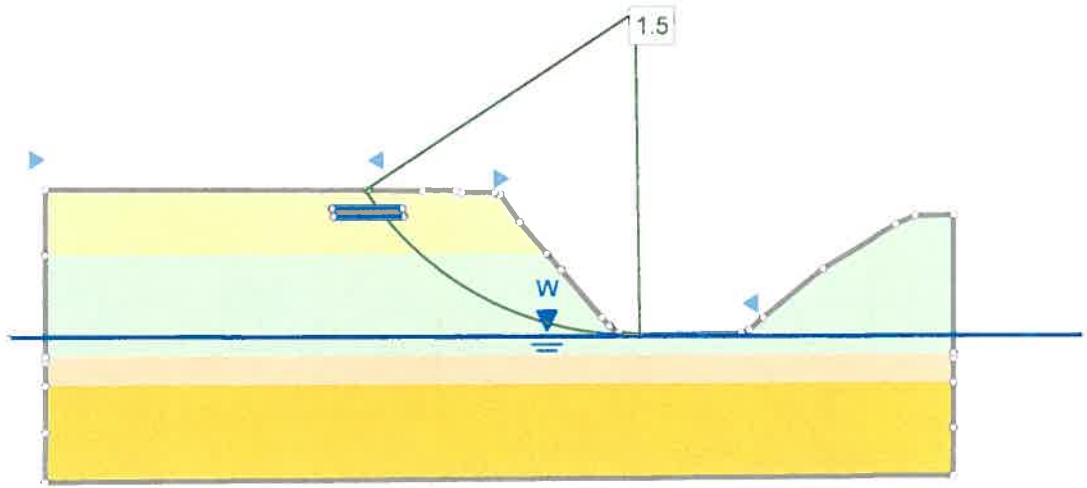


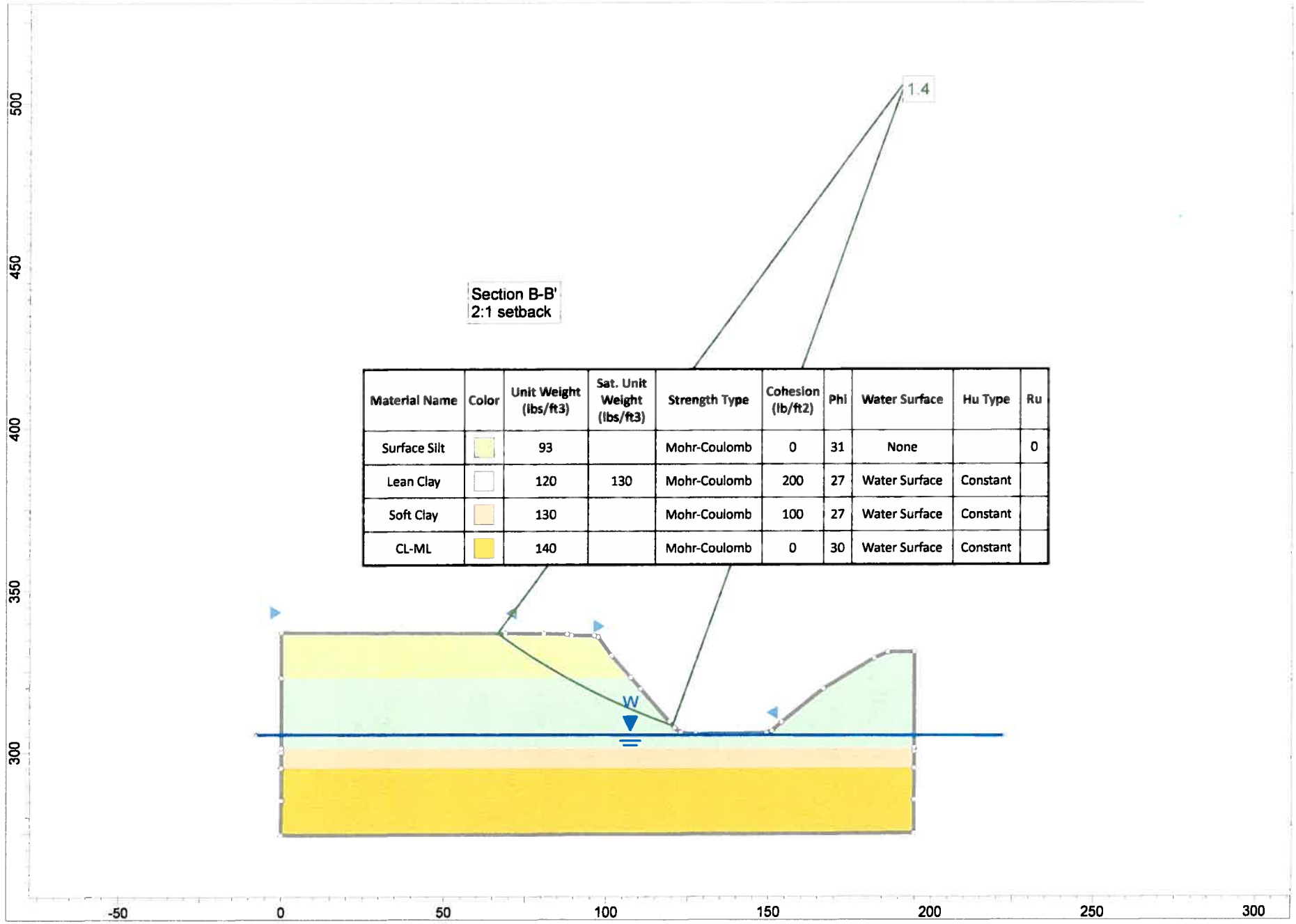



| Material Name | Color | Unit Weight (lbs/ft3) | Sat. Unit Weight (lbs/ft3) | Strength Type | Cohesion (lb/ft2) | Cohesion Type | Water Surface | Ru |
|---|---|---|---|---|---|---|---|---|
| Surface Silt | | 93 | | Undrained | 2500 | Constant | None | 0 |
| Lean Clay | | 120 | 130 | Undrained | 2500 | Constant | None | 0 |
| Soft Clay | | 130 | | Undrained | 800 | Constant | None | 0 |
| CL-ML | | 140 | | Undrained | 400 | Constant | None | 0 |

Section B-B'
2:1 Setback
Reinforced

| Material Name | Color | Unit Weight (lbs/ft3) | Sat. Unit Weight (lbs/ft3) | Strength Type | Cohesion (lb/ft2) | Phi | Water Surface | Hu Type | Ru |
|---|---|---|---|---|---|---|---|---|---|
| Surface Silt | | 93 | | Mohr-Coulomb | 0 | 31 | None | | 0 |
| Lean Clay | | 120 | 130 | Mohr-Coulomb | 200 | 27 | Water Surface | Constant | |
| Soft Clay | | 130 | | Mohr-Coulomb | 100 | 27 | Water Surface | Constant | |
| CL-ML | | 140 | | Mohr-Coulomb | 0 | 30 | Water Surface | Constant | |

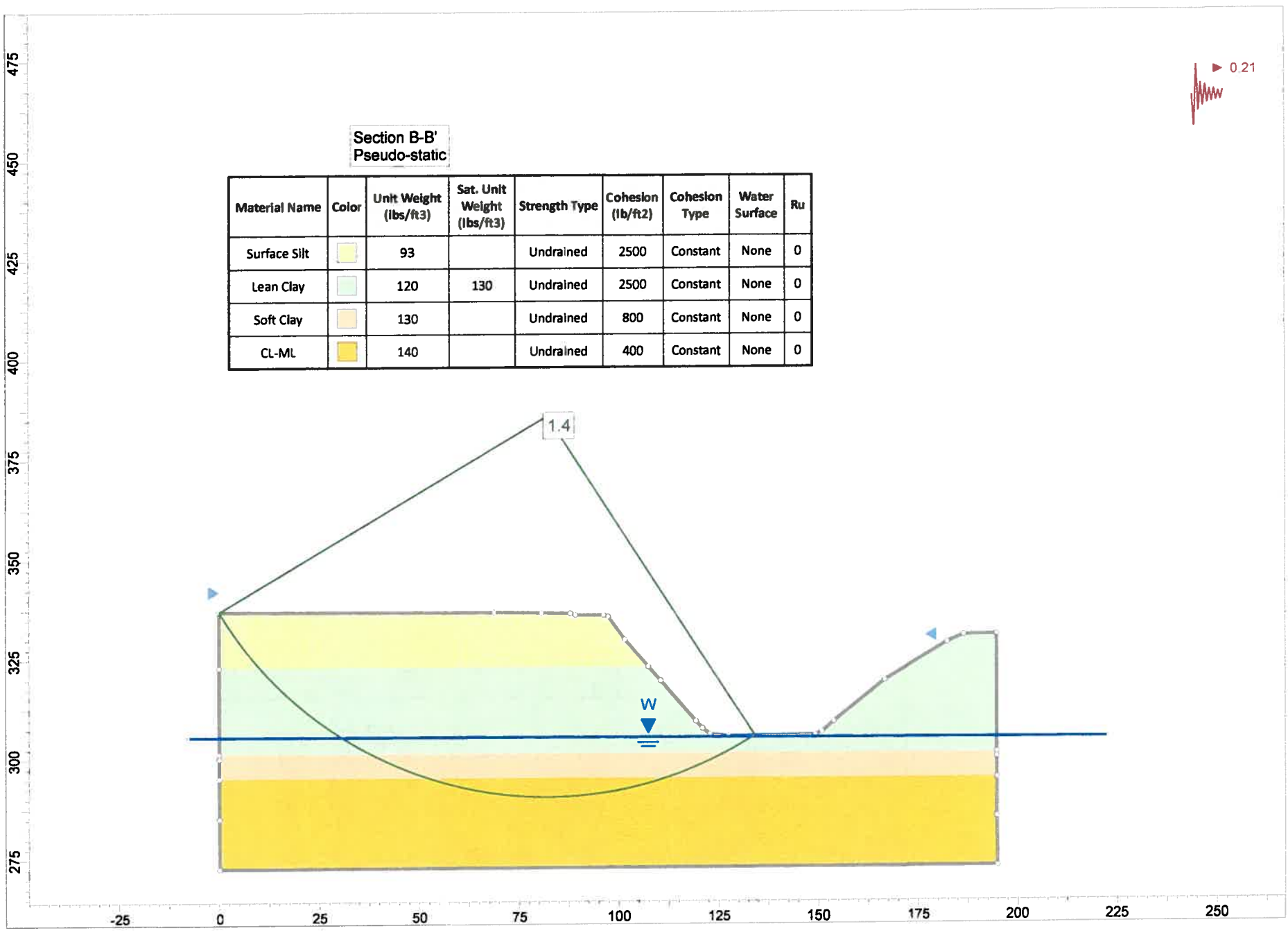





| Material Name | Color | Unit Weight (lbs/ft3) | Sat. Unit Weight (lbs/ft3) | Strength Type | Cohesion (lb/ft2) | Phi | Water Surface | Hu Type | Ru |
|---|---|---|---|---|---|---|---|---|---|
| Surface Silt | | 93 | | Mohr-Coulomb | 0 | 31 | None | | 0 |
| Lean Clay | | 120 | 130 | Mohr-Coulomb | 200 | 27 | Water Surface | Constant | |
| Soft Clay | | 130 | | Mohr-Coulomb | 100 | 27 | Water Surface | Constant | |
| CL-ML | | 140 | | Mohr-Coulomb | 0 | 30 | Water Surface | Constant | |

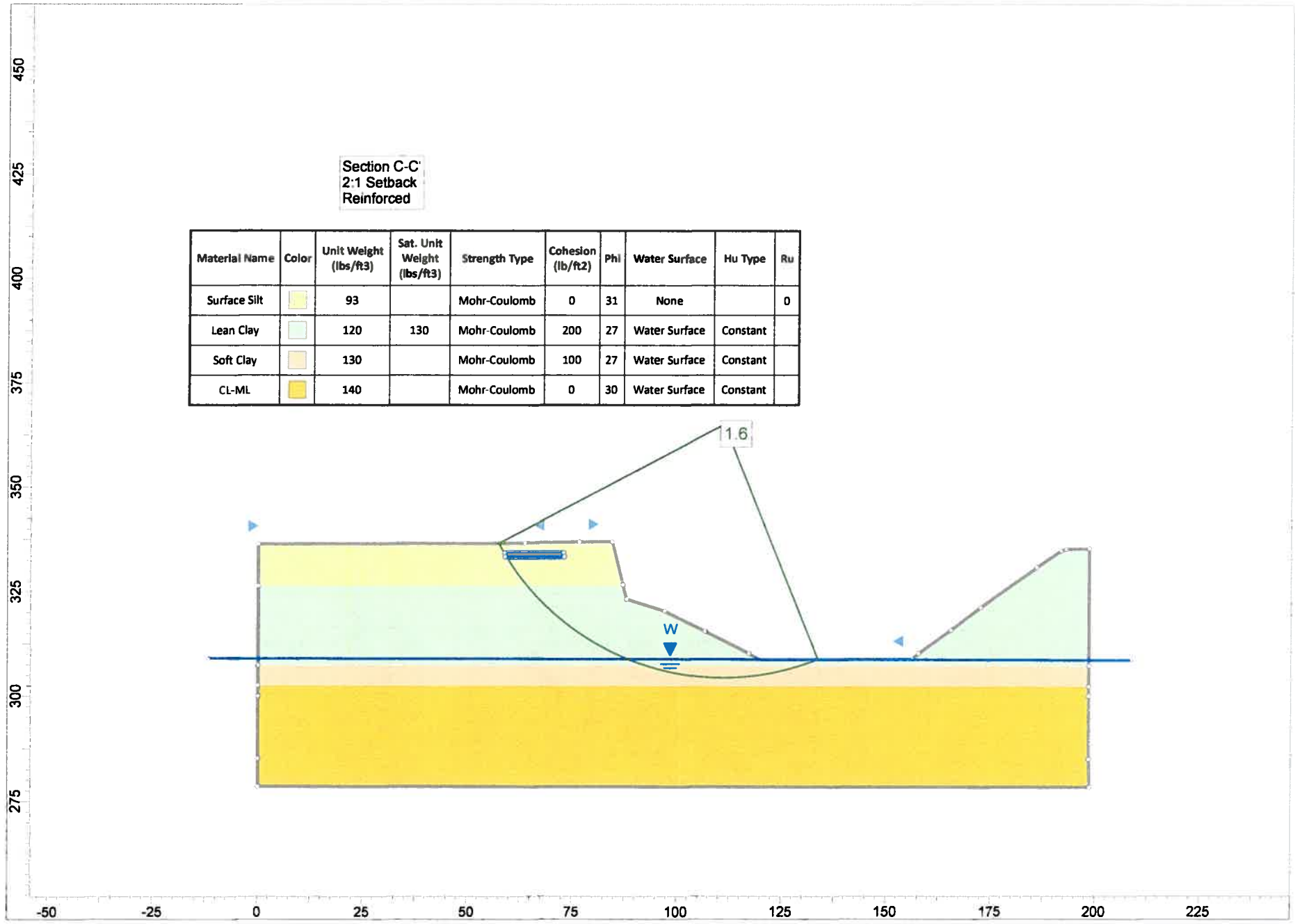
Section B-B'
Pseudo-static

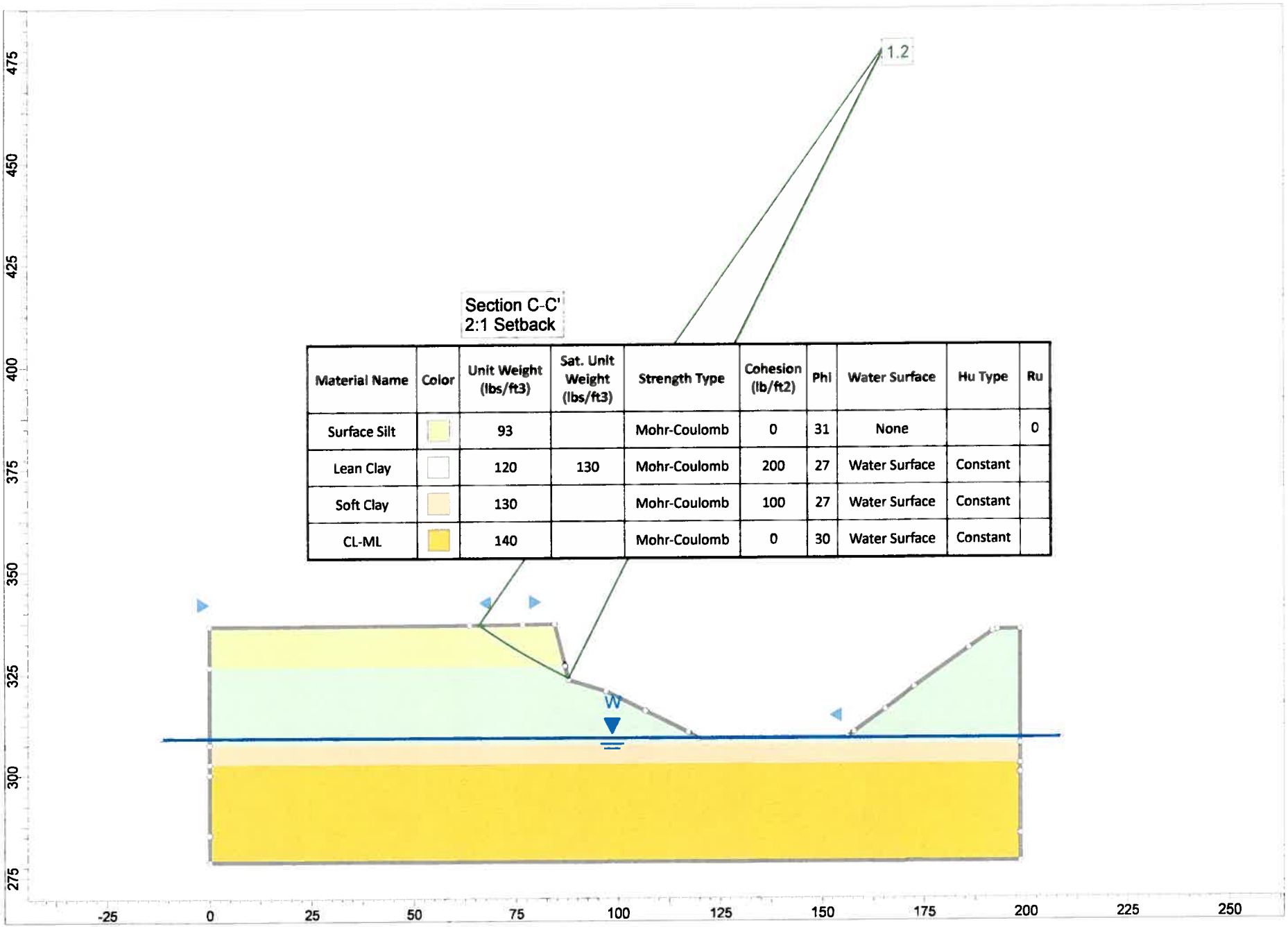
| Material Name | Color | Unit Weight (lbs/ft3) | Sat. Unit Weight (lbs/ft3) | Strength Type | Cohesion (lb/ft2) | Cohesion Type | Water Surface | Ru |
|---|---|---|---|---|---|---|---|---|
| Surface Silt | | 93 | | Undrained | 2500 | Constant | None | 0 |
| Lean Clay | | 120 | 130 | Undrained | 2500 | Constant | None | 0 |
| Soft Clay | | 130 | | Undrained | 800 | Constant | None | 0 |
| CL-ML | | 140 | | Undrained | 400 | Constant | None | 0 |



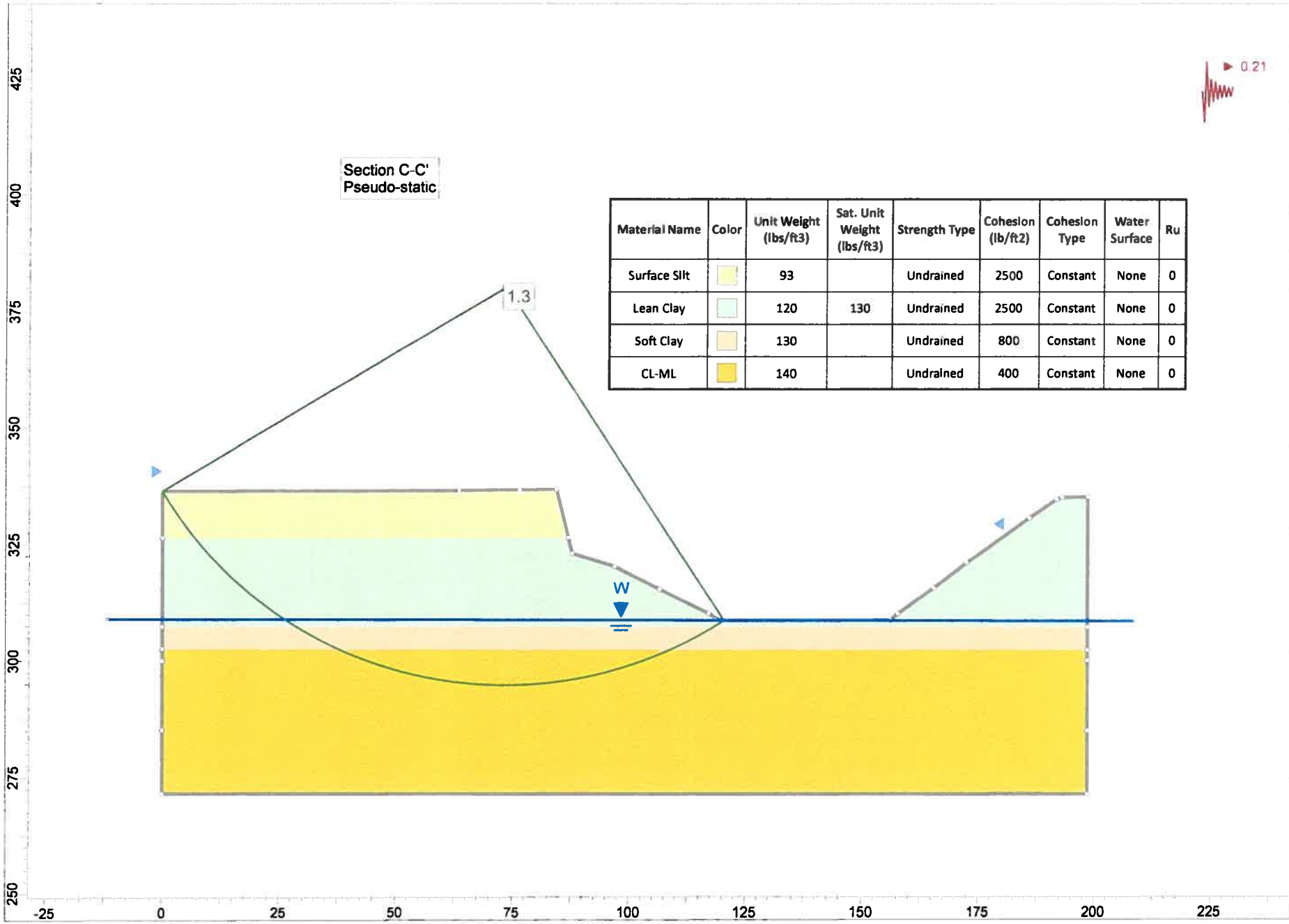
Section C-C'
2:1 Setback
Reinforced

| Material Name | Color | Unit Weight (lbs/ft3) | Sat. Unit Weight (lbs/ft3) | Strength Type | Cohesion (lb/ft2) | Phi | Water Surface | Hu Type | Ru |
|---|---|---|---|---|---|---|---|---|---|
| Surface Silt | | 93 | | Mohr-Coulomb | 0 | 31 | None | | 0 |
| Lean Clay | | 120 | 130 | Mohr-Coulomb | 200 | 27 | Water Surface | Constant | |
| Soft Clay | | 130 | | Mohr-Coulomb | 100 | 27 | Water Surface | Constant | |
| CL-ML | | 140 | | Mohr-Coulomb | 0 | 30 | Water Surface | Constant | |





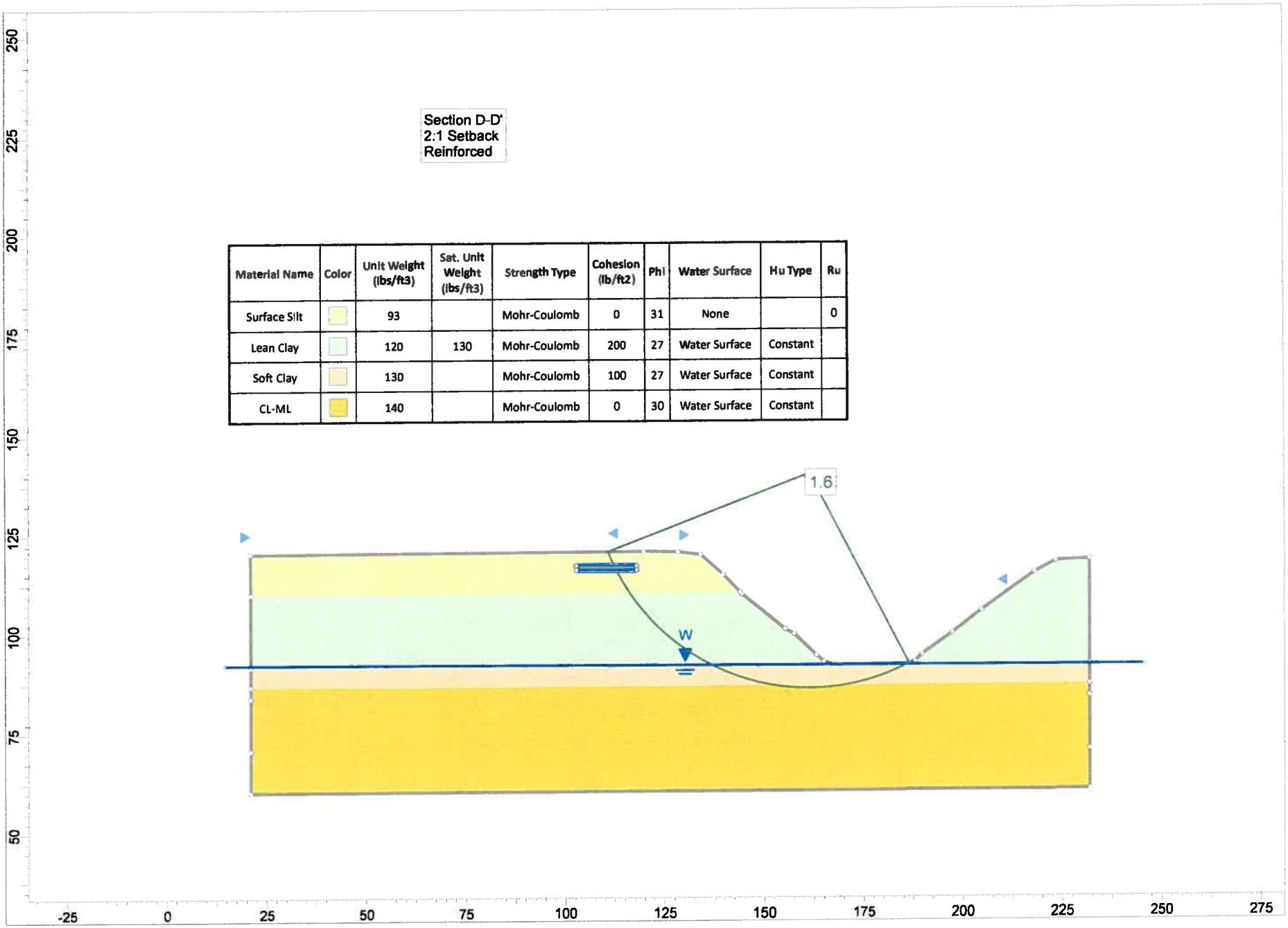

| Material Name | Color | Unit Weight (lbs/ft3) | Sat. Unit Weight (lbs/ft3) | Strength Type | Cohesion (lb/ft2) | Phi | Water Surface | Hu Type | Ru |
|---|---|---|---|---|---|---|---|---|---|
| Surface Silt | | 93 | | Mohr-Coulomb | 0 | 31 | None | | 0 |
| Lean Clay | | 120 | 130 | Mohr-Coulomb | 200 | 27 | Water Surface | Constant | |
| Soft Clay | | 130 | | Mohr-Coulomb | 100 | 27 | Water Surface | Constant | |
| CL-ML | | 140 | | Mohr-Coulomb | 0 | 30 | Water Surface | Constant | |




| Material Name | Color | Unit Weight (lbs/ft3) | Sat. Unit Weight (lbs/ft3) | Strength Type | Cohesion (lb/ft2) | Cohesion Type | Water Surface | Ru |
|---|---|---|---|---|---|---|---|---|
| Surface Silt | | 93 | | Undrained | 2500 | Constant | None | 0 |
| Lean Clay | | 120 | 130 | Undrained | 2500 | Constant | None | 0 |
| Soft Clay | | 130 | | Undrained | 800 | Constant | None | 0 |
| CL-ML | | 140 | | Undrained | 400 | Constant | None | 0 |

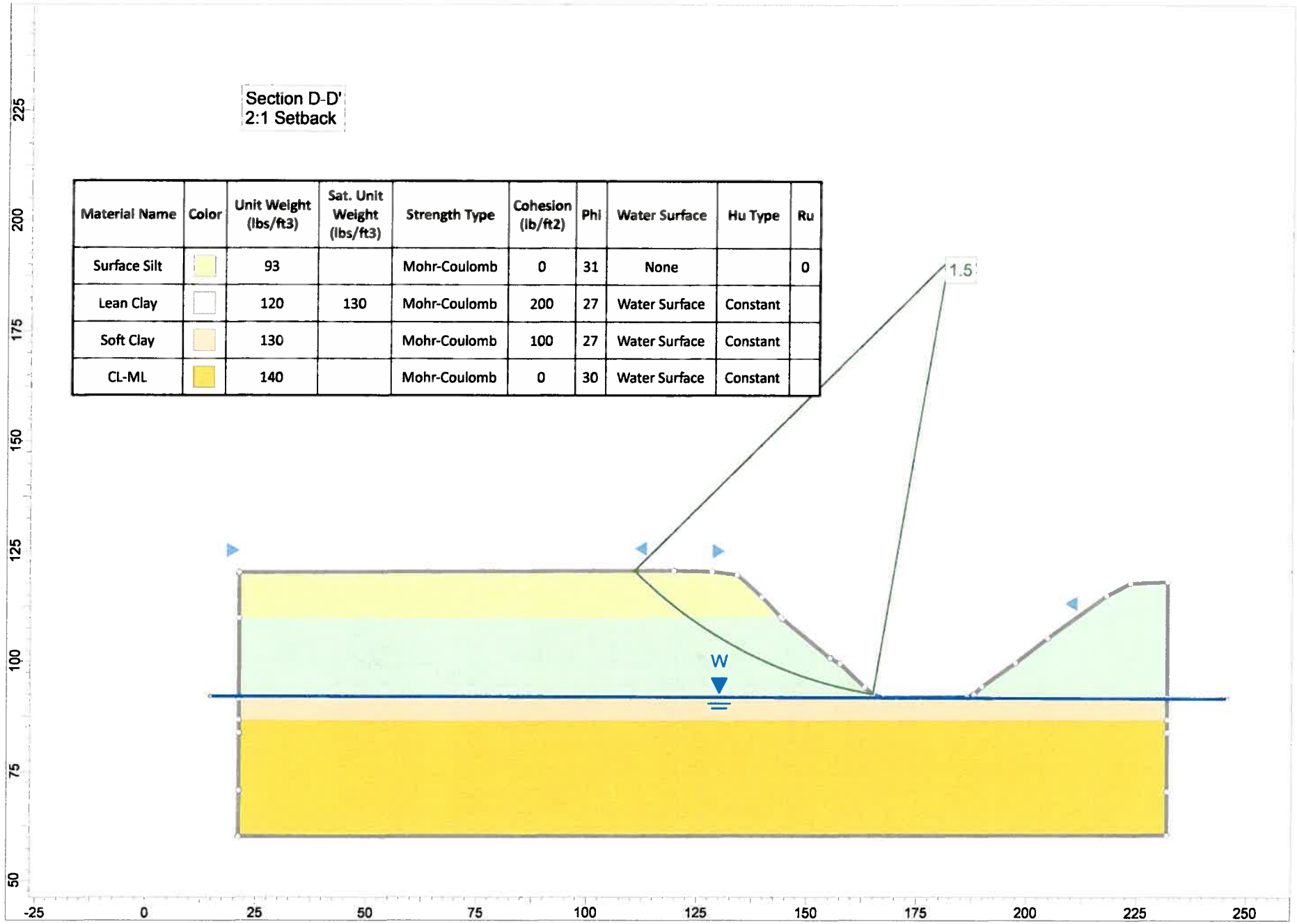
Section D-D'
2:1 Setback
Reinforced

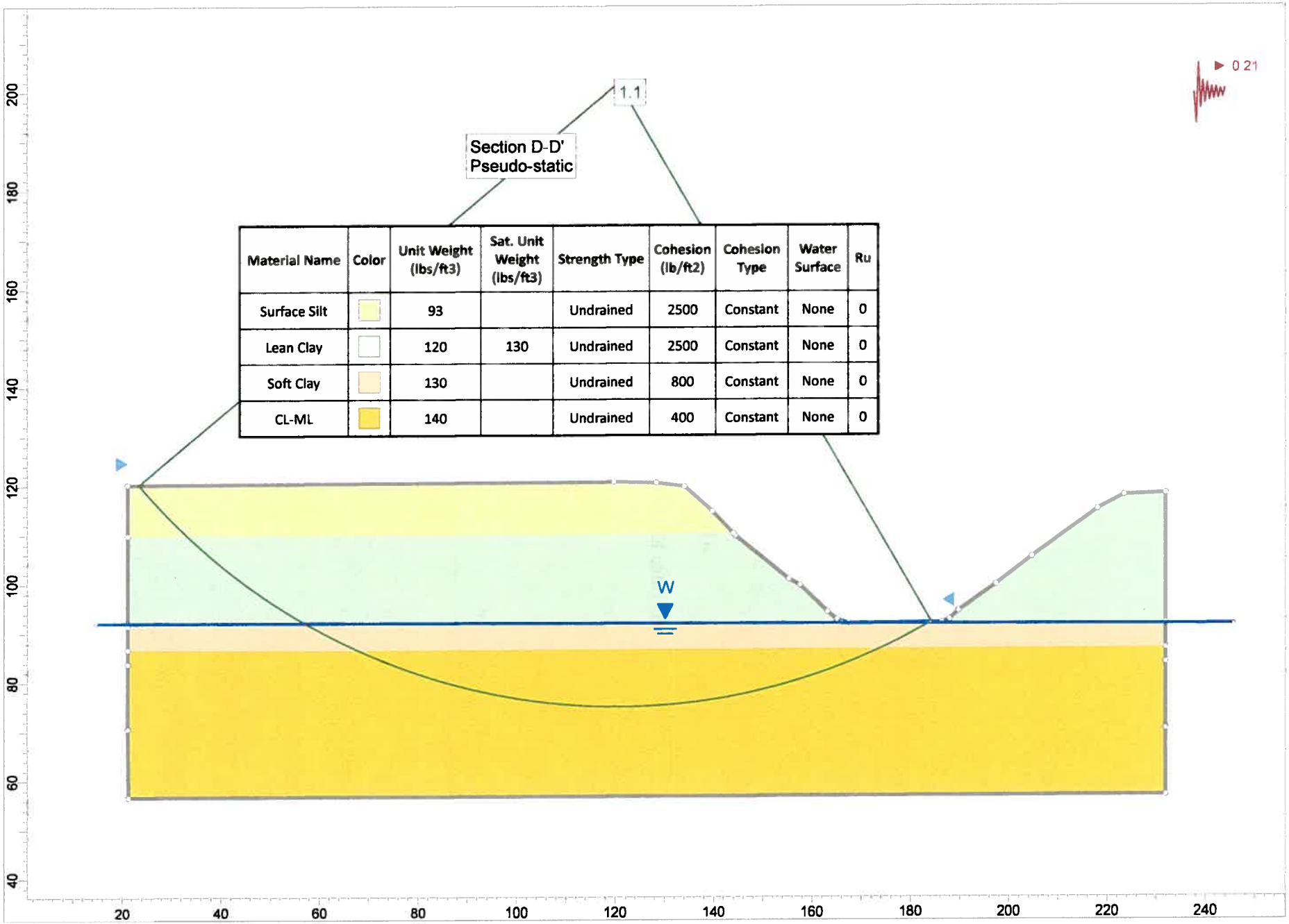
| Material Name | Color | Unit Weight (lbs/ft3) | Sat. Unit Weight (lbs/ft3) | Strength Type | Cohesion (lb/ft2) | Phi | Water Surface | Hu Type | Ru |
|---|---|---|---|---|---|---|---|---|---|
| Surface Silt | | 93 | | Mohr-Coulomb | 0 | 31 | None | | 0 |
| Lean Clay | | 120 | 130 | Mohr-Coulomb | 200 | 27 | Water Surface | Constant | |
| Soft Clay | | 130 | | Mohr-Coulomb | 100 | 27 | Water Surface | Constant | |
| CL-ML | | 140 | | Mohr-Coulomb | 0 | 30 | Water Surface | Constant | |



Section D-D'
2:1 Setback

| Material Name | Color | Unit Weight (lbs/ft3) | Sat. Unit Weight (lbs/ft3) | Strength Type | Cohesion (lb/ft2) | Phi | Water Surface | Hu Type | Ru |
|---|---|---|---|---|---|---|---|---|---|
| Surface Silt | | 93 | | Mohr-Coulomb | 0 | 31 | None | | 0 |
| Lean Clay | | 120 | 130 | Mohr-Coulomb | 200 | 27 | Water Surface | Constant | |
| Soft Clay | | 130 | | Mohr-Coulomb | 100 | 27 | Water Surface | Constant | |
| CL-ML | | 140 | | Mohr-Coulomb | 0 | 30 | Water Surface | Constant | |






| Material Name | Color | Unit Weight (lbs/ft3) | Sat. Unit Weight (lbs/ft3) | Strength Type | Cohesion (lb/ft2) | Cohesion Type | Water Surface | Ru |
|---|---|---|---|---|---|---|---|---|
| Surface Silt | | 93 | | Undrained | 2500 | Constant | None | 0 |
| Lean Clay | | 120 | 130 | Undrained | 2500 | Constant | None | 0 |
| Soft Clay | | 130 | | Undrained | 800 | Constant | None | 0 |
| CL-ML | | 140 | | Undrained | 400 | Constant | None | 0 |

# APPENDIX D

**Previous Exploration Boring Logs**

APPENDIX D

DATE OF BORING: January 16, 1998

SURFACE ELEVATION: Approx. 333.0 feet (101.5 meters)

| DEPTH (FEET) | DEPTH (METERS) | SAMPLE NUMBER | LOG, LOCATION AND TYPE OF SAMPLE | DESCRIPTION | N S.P.T. BLOWS/FT *MODIFIED FOR 3" O.D. SAMPLER | qu UNCON. COMP. STRENGTH (TSF) *FIELD PENET. APPROX. | IN PLACE DRY UNIT WEIGHT (PCF) | IN PLACE MOIST. CONTENT % DRY WEIGHT |
|---|---|---|---|---|---|---|---|---|
| 0 | | | | Plowed field over grayish brown clayey SILT, moist, soft. | | | | |
| | 1 | | | Olive brown and yellowish brown clayey SILT with fine sand, rare fine rootlets, very moist, soft. | | | | |
| 5 | | 1-1 | | | 2* | 4.0* | 94.4 | 20.7 |
| | 2 | | | | | | | |
| | | 1-2 | | Grayish brown with olive brown sandy silty CLAY, trace fine sand, some fine rootlets, rare iron oxide, moist, stiff. | 8* | 1.86 | 106.5 | 16.1 |
| 10 | 3 | | | | | | | |
| | 4 | | | Grayish brown with olive brown and yellowish brown sandy silty CLAY, some fine sand, rare fine subrounded gravel, slightly moist, very stiff. | | | | |
| | | 1-3 | | | 23* | 4.5+* | 123.7 | 12.3 |
| 15 | | | | Gravelly silty SAND lens. | | | | |
| | 5 | | | Grayish brown with olive brown and yellowish brown clayey SILT. | | | | |
| | | | | Olive brown with yellowish brown sandy clayey SILT, moist, very stiff. | | | | |
| | | 1-4 | | | 20* | | 103.7 | 19.5 |
| 20 | 6 | | | Olive brown with yellowish brown silty CLAY, trace fine sand, moist, very stiff. | | | | |
| | 7 | | | Approximate flow line of creek at depth of 23 feet. | | | | |
| | | 1-5 | | Olive brown with yellowish brown sandy clayey SILT, rare fine subrounded gravel, some carbonates and iron oxide, slightly moist, hard. | 31* | 4.5+* | 123.0 | 11.9 |
| 25 | 8 | | | | | | | |
| | 9 | 1-6 | | Grayish brown and olive brown with yellowish brown sandy silty CLAY, some fine sand, rare fine rootlets, moist, stiff. | 13* | 1.30 | 109.2 | 19.9 |
| 30 | | | | | | | | |

MET 4425 2/24/98

ENGEO INCORPORATED

1635/1777 ROSE AVENUE

PLEASANTON, CALIFORNIA

BORING NO.: B1

DATE: February 1998

PROJECT NO.: 4425.E1

CHECKED BY

FIGURE NO. 5

DATE OF BORING: January 16, 1998

SURFACE ELEVATION: Approx. 333.0 feet (101.5 meters)

| DEPTH (FEET) | DEPTH (METERS) | SAMPLE NUMBER | LOG, LOCATION AND TYPE OF SAMPLE | DESCRIPTION | N S.P.T. BLOWS/FT *MODIFIED FOR 3" O.D. SAMPLER | qu UNCON. COMP. STRENGTH (TSF) *FIELD PENET. APPROX. | IN PLACE DRY UNIT WEIGHT (PCF) | IN PLACE MOIST. CONTENT % DRY WEIGHT |
|---|---|---|---|---|---|---|---|---|
| | | | | Water level at time of drilling. | | | | |
| | 10 | | | Silty SAND lens, very moist. | | | | |
| | | 1-7 | | Grayish brown with yellowish brown and olive brown silty CLAY, some fine sand, very moist, stiff. | 9* | 1.0* | | 21.1 |
| 35 | 11 | | | | | | | |
| | | | | Silty SAND lens, very moist. | | | | |
| | 12 | 1-8 | | Olive brown with yellowish brown sandy silty CLAY, trace iron oxide, very moist, stiff. | 13 | | | 20.3 |
| 40 | | | | Bottom of boring at approximately 39.5 feet. | | | | |
| | 13 | | | | | | | |
| 45 | 14 | | | | | | | |
| | 15 | | | | | | | |
| 50 | | | | | | | | |
| | 16 | | | | | | | |
| 55 | 17 | | | | | | | |
| | 18 | | | | | | | |
| 60 | | | | | | | | |

ENGEO INCORPORATED

1635/1777 ROSE AVENUE

PLEASANTON, CALIFORNIA

BORING NO.: B1

DATE: February 1998

PROJECT NO.: 4425-E1

CHECKED BY

FIGURE NO. 5

MET 4425 2/24/98

DATE OF BORING: January 16, 1998

SURFACE ELEVATION: Approx. 334.0 feet (101.8 meters)

| DEPTH (FEET) | DEPTH (METERS) | SAMPLE NUMBER | LOG, LOCATION AND TYPE OF SAMPLE | DESCRIPTION | N S.P.T. BLOWS/FT *MODIFIED FOR 3" O.D. SAMPLER | qu UNCON. COMP. STRENGTH (TSF) *FIELD PENET. APPROX. | IN PLACE DRY UNIT WEIGHT (PCF) | IN PLACE MOIST. CONTENT % DRY WEIGHT |
|---|---|---|---|---|---|---|---|---|
| 0 | | | | Plowed field over olive brown clayey SILT, very moist, soft. | | | | |
| | 1 | 2-1 | | Olive brown with yellowish brown clayey SILT with fine sand, some rootlets, very moist, soft. | 2* | 2.25* | 94.9 | 22.4 |
| 5 | | | | | | | | |
| | 2 | 2-2 | | Light yellowish brown with olive brown clayey SILT, some fine sand, trace carbonates, slightly moist, stiff. | 10* | 4.5+* | 105.9 | 10.2 |
| 10 | 3 | | | | | | | |
| | 4 | 2-3 | | Olive brown with light yellowish brown clayey sandy SILT with fine sand, rare fine gravel, trace iron oxide and carbonates, slightly moist, stiff. | 14* | 4.0* | 111.4 | 17.6 |
| 15 | | | | | | | | |
| | 5 | 2-4 | | Light yellow brown with olive brown clayey SILT, some fine sand, trace iron oxide, slightly moist, very stiff. | 23* | 4.0* | 114.0 | 9.2 |
| | 6 | | | Bottom of boring at approximately 18.5 feet. Groundwater not encountered. | | | | |
| 20 | | | | | | | | |
| | 7 | | | | | | | |
| 25 | | | | | | | | |
| | 8 | | | | | | | |
| 30 | 9 | | | | | | | |

MET 4425 2/24/98

ENGEO INCORPORATED

1635/1777 ROSE AVENUE

PLEASANTON, CALIFORNIA

BORING NO.: B2

DATE: February 1998

PROJECT NO.: 4425-E1

CHECKED BY

FIGURE NO. 6

DATE OF BORING: January 16, 1998

SURFACE ELEVATION: Approx. 335.0 feet (102.1 meters)

| DEPTH (FEET) | DEPTH (METERS) | SAMPLE NUMBER | LOG, LOCATION AND TYPE OF SAMPLE | DESCRIPTION | N S.P.T. BLOWS/FT *MODIFIED FOR 3" O.D. SAMPLER | qu UNCON. COMP. STRENGTH (TSF) *FIELD PENET. APPROX. | IN PLACE DRY UNIT WEIGHT (PCF) | IN PLACE MOIST. CONTENT % DRY WEIGHT |
|---|---|---|---|---|---|---|---|---|
| 0 | | | | Plowed field over dark yellowish brown clayey SILT with fine sand, very moist, soft. | | | | |
| | 1 | | | Dark yellowish brown clayey SILT with fine sand, trace rootlets, moist, soft. | | | | |
| | | 3-1 | | | 3* | 2.25* | 101.6 | 19.7 |
| 5 | | | | | | | | |
| | 2 | | | | | | | |
| | | | | Olive brown with light yellowish brown clayey SILT with fine sand, moist, stiff. | | | | |
| | | 3-2 | | | 9* | 4.5+* | 105.3 | 18.6 |
| 10 | 3 | | | | | | | |
| | 4 | | | Olive brown with yellowish brown clayey SILT with fine sand, trace rootlets, moist, stiff. | | | | |
| | | 3-3 | | | 14* | 4.5* | 114.6 | 15.6 |
| 15 | 5 | | | | | | | |
| | | | | Gravelly SAND lens. | | | | |
| | | 3-4 | | Olive brown with yellowish brown silty CLAY, some fine sand, very moist, stiff. | 11* | 2.25* | 101.7 | 23.9 |
| 20 | 6 | | | Bottom of boring at approximately 19.5 feet. Ground water not encountered. | | | | |
| | 7 | | | | | | | |
| 25 | | | | | | | | |
| | 8 | | | | | | | |
| | 9 | | | | | | | |
| 30 | | | | | | | | |

MET 4425 2/27/98

ENGEO INCORPORATED

1635/1777 ROSE AVENUE

PLEASANTON, CALIFORNIA

BORING NO.: B3

DATE: February 1998

PROJECT NO.: 4425-E1

Checked by

FIGURE NO. 7

DATE OF BORING: January 16, 1998

SURFACE ELEVATION: Approx. 334.0 feet (101.8 meters)

| DEPTH (FEET) | DEPTH (METERS) | SAMPLE NUMBER | LOG, LOCATION AND TYPE OF SAMPLE | DESCRIPTION | N S.P.T. BLOWS/FT *MODIFIED FOR 3" O.D. SAMPLER | qu UNCON. COMP. STRENGTH (TSF) *FIELD PENET. APPROX. | IN PLACE DRY UNIT WEIGHT (PCF) | IN PLACE MOIST. CONTENT % DRY WEIGHT |
|---|---|---|---|---|---|---|---|---|
| 0 | | | | Native grass over grayish brown clayey SILT, very moist, soft. | | | | |
| | 1 | | | | | | | |
| 5 | | 4-1 | | Yellowish brown with olive brown clayey sandy SILT, rare fine rootlets, trace iron oxide, damp, soft to about 4' then medium stiff. | 7* | 4.5+* | | 4.4 |
| | 2 | | | | | | | |
| 10 | 3 | | | | | | | |
| | | 4-2 | | Olive brown and yellowish brown clayey SILT, some fine sand, rare fine rootlets, some carbonates, slightly damp, very stiff. | 16* | 4.5+* | 105.0 | |
| | 4 | | | | | | | |
| 15 | | | | | | | | |
| | 5 | 4-3 | | Olive brown clayey silty SAND, some subrounded gravel, trace iron oxide and carbonates, slightly moist, medium dense. | 21* | 4.5+* | 120.7 | 7.0 |
| | | | | Bottom of boring at approximately 16.5 feet. Groundwater not encountered. | | | | |
| 20 | 6 | | | | | | | |
| | 7 | | | | | | | |
| 25 | | | | | | | | |
| | 8 | | | | | | | |
| 30 | 9 | | | | | | | |

MET 4425 5/6/98

ENGEO INCORPORATED

1635/1777 ROSE AVENUE

PLEASANTON, CALIFORNIA

BORING NO.: B4

DATE: May 1998

PROJECT NO.: 4425.E1

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FIGURE NO. 8