APPENDIX C

Geotechnical Exploration

PENINSULA HEIGHTS
SAN MATEO, CALIFORNIA

GEOTECHNICAL EXPLORATION

SUBMITTED TO
Ms. Danielle Friend
Campus POP Investors, LLC
℅ Harvest Properties, Inc.
180 Grand Avenue
Oakland, CA 94610

PREPARED BY
ENGEO Incorporated

November 26, 2019
Latest Revision April 23, 2020

PROJECT NO.
16683.000.000

ENGEO
Expect Excellence

Project No.
16683.000.000

November 26, 2019
Latest Revision April 23, 2020

Ms. Danielle Friend
Campus POP Investors, LLC
% Harvest Properties, Inc.
180 Grand Avenue
Oakland, CA 94610

Subject: Peninsula Heights
San Mateo, California

GEOTECHNICAL EXPLORATION

Dear Ms. Friend:

We are pleased to present this geotechnical exploration report for the planned Peninsula Heights project located in San Mateo, California. We characterized the subsurface conditions at the site to provide the enclosed geotechnical recommendations for design.

From a geotechnical engineering viewpoint, the site is suitable for the proposed development provided the recommendations and guidelines in this report are implemented during project planning, design, and construction. The main geologic/geotechnical concerns at the site include the presence of artificial fill, slope stability, differential bedrock/soil foundation bearing conditions, and potential for strong earthquake generated ground motions. Our recommendations to address these concerns are presented in the accompanying report.

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

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

Sincerely,

ENGEO Incorporated

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1.0 INTRODUCTION

1.1 PURPOSE AND SCOPE

We prepared this geotechnical exploration report for the planned Peninsula Heights residential development in San Mateo, California, as outlined in our agreement dated October 3, 2019. For our use, we received various plans and CAD files prepared by BKF, transmitted electronically to us between October and November 2019.

Harvest Properties, Inc. (Campus POP Investors, LLC) authorized us to conduct the following scope of services:

- Historic map and aerial review
- Subsurface field exploration
- Laboratory testing
- Data analysis and conclusions
- Report preparation

This report was prepared for the exclusive use of our client and their consultants for design of this project. If any changes are made in the character, design or layout of the development, we must be contacted to review the conclusions and recommendations contained in this report to evaluate whether modifications are recommended. This document may not be reproduced in whole or in part by any means whatsoever, nor may it be quoted or excerpted without our express written consent.

1.2 SITE LOCATION AND DESCRIPTION

The site is located on Campus Drive, east of Highway 92 and north of West Hillsdale Boulevard in San Mateo, California (Figure 1). The site consists of two study areas located on opposite sides of Campus Drive, referred to as the “Northern Parcel” and “Southern Parcel” herein.

The Northern Parcel study area includes two individual terraces. It is currently occupied by two 3-story commercial office buildings with supporting paved parking areas. Each of the buildings is located towards the northern corner of the associated terrace. The upper terrace ranges in elevation between approximately 389 feet (San Mateo Datum) to the north and 367 feet to the south, and the lower terrace ranges in elevation between approximately 340 feet to the north and 300 feet to the south. The Northern Parcel is approximately 5 acres in area and is bordered by Live Oak Drive to the southwest, residential homes to the northwest, commercial properties to the northeast and east, and Campus Drive to the south. The two levels are separated by an approximately 30-foot-tall slope ranging from 2:1 (horizontal: vertical) to 2½:1.

The Southern Parcel study area also includes two relatively flat terraces. Three-story and two-story office buildings are located along Campus Drive at each of the levels with supporting paved parking areas. The upper terrace ranges from approximate Elevation 302 to 310 feet (southeast to northwest), and the lower terrace ranges from approximate Elevation 265 to 270 feet (south to north). At the southern boundary of the lower terrace, a downslope drops to an approximate Elevation of 240 feet. The Southern Parcel is approximately 8½ acres in area and is bordered by commercial properties to the north, Campus Drive to the east, and undeveloped open space sloping down to a drainage swale on the west and south. The two levels at the Southern Parcel are divided by approximately 30-foot-tall slopes ranging from 2:1 (horizontal: vertical) to 2½:1.

1.3 PROPOSED DEVELOPMENT

The project will include a mix of single-family and multi-family structures up to four stories in height, paved streets, sidewalks, underground utilities and bioretention areas. Additionally, various retaining walls up to 25 feet in height are planned. The Site Plan prepared by BKF, dated January 29, 2020, indicates site grading will include minor cuts and larger fills for construction of perimeter retaining walls up to 25 feet in height.

1.4 AERIAL PHOTOGRAPH AND TOPOGRAPHIC MAP REVIEW

To understand the site development history, we reviewed historic aerial photographs and topographic maps. Aerial photographs flown between 1941 and present day were available through UCSB Frame Finder and Google Earth. We also viewed historic topographic maps published back to 1947. The topographic map from 1947 and aerial photographs between 1941 and 1965 show significant differences to site topography compared to the current conditions. Most significantly, swales (that are presently filled) are visible along the southern and eastern boundaries of both the Northern Parcel and Southern Parcel. These differences in elevation contours are depicted in Figure 7.

Along the southern and eastern portions of the lower terrace in the Northern Parcel, where present elevations range between Elevations of 290 and 330 feet, historic 1947 topographic contours show Elevations between 275 and 325 feet. This difference suggests a maximum of 25 feet of artificial fill may have been previously placed.

In the Southern Parcel, where present Elevations range from 230 to 265 feet, historic 1947 topographic contours show Elevations between 210 and 250 feet. This difference suggests that up to 55 feet of artificial fill may have been previously placed in the deeper areas (within the panhandle of the Southern Parcel, where improvements are not currently planned). In the eastern portions of the parcel, where present Elevations range from Elevations of 275 to 310 feet, historic 1947 topographic contours show Elevations between 275 and 300 feet. This difference suggests a maximum of 25 feet of artificial fill may have been previously placed in the portion of the site where improvements are planned as part of the proposed development.

For both parcels, the upper terraces appear to be areas of bedrock cut, with cut slopes separating these areas from the lower terraces. The historic changes in topography suggest the lower terrace levels were constructed by placing fill sourced from cuts in the upper terraces.

Based on the aerial photographs reviewed, construction of the existing buildings in the Northern Parcel was completed by 1974, and grading activities at the Southern Parcel appear visible in the same photograph. Construction of the existing buildings in the Southern Parcel appears to have been completed by 1982. As discussed above, significant grading (consisting of both cut and fill) previously took place to form the current topography at the site.

2.0 GEOLOGIC CONDITIONS

2.1 REGIONAL GEOLOGY

The project site is located within the Coast Ranges Geomorphic Province of California. The Coast Ranges are characterized by a system of northwest-trending fault-bounded mountain ranges and intervening alluvial valleys. Bedrock in the Coast Ranges consists of igneous, metamorphic and sedimentary rocks that range in age from Jurassic to Pleistocene. The present physiography and geology of the Coast Ranges are the result of deformation and deposition along the tectonic boundary between the North American plate and the Pacific plate. Plate boundary fault movements are largely concentrated along the well-known fault zones, which in the area include the San Andreas, Hayward, and Calaveras faults, as well as other lesser-order faults.

2.2 SITE GEOLOGY

As shown in Figure 3, regional geologic mapping by Pampeyan (1994) indicates the site is underlain by sheared rock (mélange) of the Cretaceous and Jurassic Franciscan Complex (KJfsr). The Franciscan Complex makes up much of the basement rock of the Coast Ranges and consists of an assemblage of deformed and metamorphosed rock units. The Franciscan Complex bedrock in this area generally comprises graywacke, siltstone, and shale, substantial portions of which have been sheared (Brabb, 1998).

Deposits identified by Brabb (1998) as Pleistocene alluvial fan and fluvial deposits are mapped along the southern boundary of the Southern Parcel. In the same area, Pampeyan (1994) maps deposits of Holocene-age slope wash, ravine fill, and colluvium (Qsr). Both Brabb and Pampeyan map portions of both the Northern Parcel and Southern Parcel as underlain by artificial fill (Qaf), as shown in Figure 3.

2.3 REGIONAL FAULTING AND SEISMICITY

Northern California contains numerous active and potentially active earthquake faults. According to California Geologic Survey (CGS) Special Publication 42, an active fault is defined as one that has had surface displacement within Holocene time (the last 11,700 years) (CGS SP42, Revised 2018). The site is not located within a currently designated State of California Earthquake Fault Zone for active faults, and no known faults cross the site (CGS, 1974).

Numerous small earthquakes occur every year in the San Francisco Bay Region and larger earthquakes have been recorded and can be expected to occur in the future. Figure 6 shows the approximate locations of these faults and significant historic earthquakes recorded within the Greater Bay Area Region. The known nearby active faults within 20 miles of the site and their estimated maximum earthquake magnitudes are provided in the following table based on United States Geologic Survey (USGS) 2008 National Seismic Hazard Maps.

TABLE 2.3-1: Approximate Fault Distances and Locations Relative to Project Site

FAULT	DISTANCE (Miles)	LOCATION RELATIVE TO SITE	ESTIMATED MAXIMUM MAGNITUDE, M_W (Ellsworth)
San Andreas	1.9	West	8.1
Monte Vista-Shannon	8.0	South	6.5
San Gregorio	9.4	West	7.5

FAULT	DISTANCE (Miles)	LOCATION RELATIVE TO SITE	ESTIMATED MAXIMUM MAGNITUDE, M_W (Ellsworth)
Hayward-Rodgers Creek	16.4	East	7.3

Latitude: 37.537380°, Longitude: -122.326171°

The United States Geologic Survey evaluated the Bay Area seismicity through a study by the 2014 Working Group on California Earthquake Probabilities. The WGCEP estimated that there is a 22 percent probability that a moment magnitude (M_w) of 6.7 or greater earthquake will occur on the San Andreas Fault before 2043. The aggregate probability of a similarly sized earthquake in the San Francisco Bay Area was estimated to be 72 percent in the study.

3.0 FIELD EXPLORATION

3.1 TEST PITS

We excavated 11 exploratory test pits to depths of up to 6 feet below existing grade across the project site (Figure 2). The test pits were excavated using a rubber-track-mounted Yanmar Vio55 excavator. Our test pits, excavated at and near the slopes all terminated in bedrock. The fill deposits that we encountered in tests pits consisted predominately of angular gravels and gravelly silt. The bedrock encountered in the test pits consists of predominately of graywacke sandstone with occasional interbedded shale units. Bedding mapped at the site was primarily northwest striking and dipping towards the northeast (Figure 2). Dominant joint sets and discontinuities mapped at the site are predominately northeast striking and dipping towards the southeast. The test pit logs are provided in Appendix B.

3.2 BORINGS/CORINGS

Field exploration performed within this scope included drilling 12 hollow-stem auger borings and 2 continuous HQ cores at the approximate locations shown on the Site Plan, Figure 2. We performed our field exploration in October 2019. We performed the borings to depths between 3½ and 50½ feet below the ground surface. We established the exploration locations by visual sighting from existing features using a mobile GIS application. All current locations should be considered only as accurate as the methods used to determine them.

We performed borings and continuous HQ wireline coring using track-mounted rig. All borings utilized a 6-inch-diameter hollow-stem drilling method. An ENGEO representative logged the boreholes in the field and collected soil samples using a 2½-inch-inside-diameter California-type split-spoon sampler fitted with 6-inch-long stainless steel liners or a 2-inch-outside-diameter Standard Penetration Test (SPT) split-spoon sampler. We recorded the penetration of the samplers into underlying materials as the number of blows needed to drive the sampler 18 inches in 6-inch increments. The boring logs present blow count results as the actual number of blows required for the last 1 foot of penetration; we have applied no conversion factors. We drove the samplers with a 140-pound hammer falling a distance of 30 inches.

We performed HQ wireline coring for Borings 1-B01 and 1-B02. We obtained continuous rock cores below the top of bedrock to a depth of 25 feet. We examined material from the rock cores and logged our observations in the field. We collected representative samples from the continuous core for laboratory testing.

The field logs were then used to develop the report logs, presented in Appendix A. The logs depict subsurface conditions within the borings at the time of drilling; however, subsurface conditions may vary with time. Laboratory results are included in Appendix C.

3.3 LABORATORY TESTING

We performed the following laboratory tests on select samples recovered during boring operations.

TABLE 3.3-1: Laboratory Testing

SOIL CHARACTERISTIC	TESTING METHOD
Natural Unit Weight and Moisture Content	ASTM D7263, D2216
Atterberg Limits	ASTM D4318
Particle Size Distribution	ASTM D1140
Point Load Strength Index of Rock Core	ASTM D5731
Corrosivity Testing (Redox, pH, Resistivity, Chloride, Sulfate, Sulfide)	ASTM D1498, D4972, G57, D4327

Many of the laboratory test results are shown on the bore and core logs (Appendix A). Individual test results are located in Appendix C.

3.4 SUBSURFACE CONDITIONS

The historic grading activities at the site combined with steep terrains give rise to the existing subsurface conditions. Figure 2 (Site Plan) depicts our geologic map developed based on the previously discussed field exploration activities. Generalized geologic cross-section profiles are presented in Figures 2A and 2B. The main geologic units within the site boundary are summarized in the following sections.

3.4.1 Artificial Fill (Qaf)

As discussed in Section 1.4, artificial fill has been placed at the site from previous site activities. Areas of artificial fill were encountered along the southern and eastern portions of both the Northern Parcel and Southern Parcel. Within the planned future improvements, we anticipate up to approximately 25 feet of existing fill at the southeastern and eastern portions of both the Northern and Southern Parcels.

The fill encountered in our exploration generally consists of medium dense to dense clayey sand with intermittent layers of stiff sandy clay and silt with variable gravel content. Atterberg limits testing performed on six samples collected from existing artificial fill resulted in Plasticity Indices (PIs) ranging from 10 to 15, indicating low expansion potential. One sample collected approximately 3 feet below ground surface at Boring 1-B06 (located along the southeastern portion of the lower terrace in the Northern Parcel) yielded a PI of 26, indicating moderate to high expansion potential. A second sample tested from Boring 1-B06, collected at a depth of 16 feet yielded a PI of 15.

The laboratory testing to assess the expansive characteristics of artificial fill present on site (planned for re-use as engineered fill during site development) generally indicates low to moderate expansion potential with isolated pockets of high expansion potential material.

3.4.2 Franciscan Complex (KJfsr)

Bedrock, where encountered, generally consists of sandstone of the Cretaceous and Jurassic Franciscan Complex. Minor amounts of shale and siltstone occurring as thin interbeds were also encountered in subsurface explorations. The sandstone was generally olive brown, closely to very closely fractured and moderately weathered. An intact block of sandstone greater than 2 feet thick was encountered in one of the test pits excavated at the site. Intact blocks and fragments of sandstone encountered in test pits and cores range from moderately strong to very strong. Generally, the fractures were very closely to closely spaced, open, unhealed with iron and manganese oxide coatings. Orientation of fractures was highly variable. Due to very closely spaced discontinuities, the Rock Quality Designation (RQD) recorded in boreholes was zero. Bedding and discontinuity orientations collected in our test pits are shown on Figures 2, 2A and 2B.

We did not encounter ultramafic rock or serpentinite in our explorations or surface mapping. While the Franciscan Formation is known to be highly variable, the nearest mapped outcrops of serpentinite are located approximately 1½ miles from the site. Additionally, we did not encounter serpentinite in any of our boring or test pit locations. Therefore, the likelihood of encountering bedrock containing naturally occurring asbestos is low.

3.4.3 Residual Soil

In some areas, residual soil may be present at the site between the artificial fill and bedrock. Thin layers of residual soil encountered in Borings 1-B06 and 1-B08 consisted of sandy and clayey soil.

3.5 GROUNDWATER

Groundwater was not encountered in the borings and test pits during our field exploration. However, it is possible that groundwater at the site is transient with perched zones located above the bedrock resulting from runoffs from upland areas.

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

4.0 CONCLUSIONS

Based on our findings and results of engineering analyses, from a geologic and geotechnical standpoint, the study area is suitable for residential development provided the recommendations provided in this report and other sound engineering practices are incorporated in the design and construction of the project. We evaluated the site with respect to known geologic and other hazards common to the greater San Francisco Bay Region. The primary hazards and the risks associated with these hazards with respect to the planned development are: (1) seismic hazards; (2) slope stability; (3) artificial fill of variable density; and (4) bedrock excavation.

4.1 SEISMIC HAZARDS

Potential seismic hazards resulting from a nearby moderate to major earthquake can generally be classified as primary and secondary. The primary effect is ground rupture, also called surface faulting. The common secondary seismic hazards include ground shaking 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, soil liquefaction, lateral spreading, earthquake-induced landsliding, tsunamis, flooding or seiches is low to negligible at the site.

4.1.1 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, ground rupture is unlikely at the subject property.

4.1.2 Ground Shaking

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

4.1.3 Liquefaction

Soil liquefaction results from loss of strength during cyclic loading, such as imposed by earthquakes. Soil most susceptible to liquefaction is, loose, saturated, sand. Empirical evidence indicates soft, low-plasticity silt, and some low-plasticity clay are also potentially liquefiable.

The soil encountered in our exploration consists of medium dense to very dense clayey sand and gravel or stiff to hard clay above the groundwater table and bedrock that are not susceptible to liquefaction. The potential for liquefaction at the project site is therefore negligible.

4.1.4 Lateral Spreading

Lateral spreading involves lateral ground movements caused by seismic shaking. These lateral ground movements are often associated with a weakening or failure of an embankment or soil mass overlying a layer of liquefied or weak soil. Since the on-site soil is unlikely to liquefy, the potential for lateral spreading at this site is negligible.

4.1.5 Earthquake-Induced Landsliding

The site is mapped by USGS in an area susceptible to earthquake-induced landslides (Figure 5), however, based on our geologic mapping and observations of the existing bedrock slopes at the site, as well as proposed corrective grading, it is our opinion that the potential for earthquake induced landsliding impacts at the site is low. Further discussion on slope stability analysis is provided in Section 4.2 of this report.

4.1.6 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. The potential for the formation of these cracks is greater at contacts between deep alluvium and bedrock such as those at the margins of valley flood plains. The lack of deep alluvium makes the potential risk of this hazard negligible at site.

4.2 SLOPE STABILITY

4.2.1 Methods of Analysis

We performed limit equilibrium slope stability analyses of critical sections in the Northern and Southern Parcels using Spencer's Method (Spencer, 1967). The slope-stability analyses were performed on generalized cross sections within the Northern Parcel and Southern Parcel (Figures 2A through 2D). The conditions at each section are described below:

- Section A-A' shows interpretative grades and conditions at the Northern Parcel and includes the eastern area comprised of artificial fill, where a 22-foot-high mechanical stabilized earth (MSE) wall is planned.

- Section D-D' shows interpretative grades and conditions in the southern portion of existing artificial fill within the Southern Parcel, where a MSE wall ranging in height between 20 to 25 feet is planned.

- Sections F-F' and G-G' shows interpretative grades and conditions in the eastern portion of the Southern Parcel, across an area of "thicker fills" adjacent Campus Drive. A MSE wall ranging in height between 18 and 24 feet is proposed to be constructed along the eastern site boundary in this area. Sections F-F' and G-G' were selected to evaluate slope stability considering the critical conditions of deepest artificial fill depth and higher portion of the planned wall, respectively.

Our slope stability analysis considered geogrid reinforcement for MSE walls with minimum tensile strength of 3,200 pounds per foot. Where applicable, surcharge loading from the planned structures is included based on an average weight of 500 psf for the townhomes and 350 psf for single-family homes. Our sections incorporate proposed remedial grading measures and the implementation of the earthwork recommendations as presented in Section 5.0 of this report. With consideration to long-term conditions, we also incorporated a piezometric surface based on potential subdrainage in the vicinity of future keyways.

The analyses were performed using the computer-aided program SLIDE© (Version 7.0). We performed the analyses in general accordance with guidelines provided in the California Geological Survey's SP 117A (2008). A seismic coefficient of 0.35g was used to model slope stability under seismic shaking conditions using the simplified methods of Blake et al. (2002). These seismic coefficients were developed using a PGA_M of 1.09g as prescribed in the 2019 California Building Code, a Magnitude of 8.1, a distance of less than 10 km, and a threshold displacement of 15 centimeters.

4.2.2 Modeling Shear Strength Parameters

For the purposes of slope stability evaluation, we based our estimates of shear strength parameters for the soil material (existing artificial fill and proposed engineered fill) on field data, index properties on laboratory testing, and published correlations for estimating the shear strength based on blow counts. We assumed proposed engineered fill will comprise a mix of existing artificial fill (predominately clayey sand with gravel) that will be reused and material sourced from bedrock cuts. The Franciscan complex bedrock material was modeled using the Generalized Hoek-Brown strength function based on the results of point load testing performed on rock core and test pit samples collected during our exploration. The soil and rock parameters used in the slope stability analyses are summarized below.

TABLE 4.2.2-1: Summary of Shear Strength Parameters

MATERIAL	UNIT WEIGHT (PCF)	DRAINED STRENGTH PARAMETERS	
		COHESION (PSF)	FRICTION ANGLE (DEG)
Engineered Fill (Proposed)	125	300	33
Existing Artificial Fill (Qaf)	125	50	30

TABLE 4.2.2-2: Summary Generalized Hoek-Brown Parameters

MATERIAL	UCS (INTACT) (KSF)	GSI	INTACT ROCK CONSTANT	DISTURBANCE FACTOR
Franciscan Complex (KJfsr)	1000	20	18	0

4.2.3 Results of Slope Stability Analyses

Appendix E includes the results of our slope stability analyses for generalized Cross Sections A-A'. D-D', F-F' and G-G'. Based on the results of our analysis, to achieve a factor of safety for seismic conditions in accordance with commonly accepted criteria (minimum factor of safety of 1.0), we recommend incorporating the minimum geogrid lengths shown in Table 4.2.3-1 into MSE wall design for the walls planned along the southern and eastern boundaries of the Northern and Southern Parcels. Please note these minimum geogrid lengths are based on consideration of global stability only. To satisfy internal and external stability factors of safety, design of the MSE walls may require longer geogrid lengths than those listed in Table 4.2.3-1.

ENGEO should be retained to provide review of the design to ensure that the requirements to achieve the minimum global slope stability are achieved.

TABLE 4.2.3-1: Summary of Slope Stability Analyses

CROSS SECTION DESIGNATION	MINIMUM GEOGRID LENGTH (FEET)	FACTOR OF SAFETY	
		STATIC CASE	SEISMIC CASE
A-A'	30	1.7	1.0
D-D'	20	1.8	1.1
F-F'	25	2.0	1.0
G-G'	25	2.0	1.1

4.3 ROCK EXCAVATION AND SUITABILITY

The following is provided for informational purposes. A grading contractor should perform their own assessment of appropriate equipment during the bid process.

With the exception of the areas underlain by artificial fill (shown on Figure 2), the site is generally underlain by relatively shallow bedrock within 1 to 2 feet of the existing ground surface. Based on our subsurface data, geologic mapping and previous rippability analyses for Franciscan sandstone and graywacke at neighboring projects, the bedrock will likely require considerable ripping effort, and will generate oversized material (greater than six inches in diameter). Shear wave velocity obtained from seismic refraction at a neighboring project varies from less than 4,500 feet per second in the upper 10 to 15 feet, to approximately 7,000 to 7,500 feet per second below that depth.

Backhoes will likely experience extreme difficulty excavating the less weathered bedrock. The Yanmar Vio55 excavator used to excavate our test pits generally encountered very difficult excavation conditions and refusal within 4 feet into bedrock. We anticipate difficult ripping conditions requiring heavy-duty equipment such as D10 bulldozer or larger with single-tooth ripper shank. Excavators will encounter difficult excavation conditions in the areas of deepest cuts. Hoe-rams or other mechanical systems may be required.

The bedrock material that is excavated will require processing to generate material less than 6 inches in maximum dimension to be reused as fill on the project. Because of the expected high strength of some of this material, processing through a crusher or with hoe rams may be necessary to reduce the material to acceptable sizes for use as fill.

Please refer to Section 5.7 of this report for recommendations regarding excavating bedrock in areas to receive utilities during mass grading.

4.4 TEMPORARY STABILITY OF BEDROCK CUTS

As discussed above, the geologic structure within the bedrock exposed at the site may potentially give rise to adverse bedrock conditions that will likely daylight in steep temporary slopes and excavations. Excavation into the rock slope should be performed in stages and segments. Over-steepened cuts may trigger failures and the downslope movement of rocks should be expected during construction. Grading along the slope should be performed under the observation of our engineering geologist to minimize larger scale displacement of the slope.

4.5 ARTIFICIAL FILL

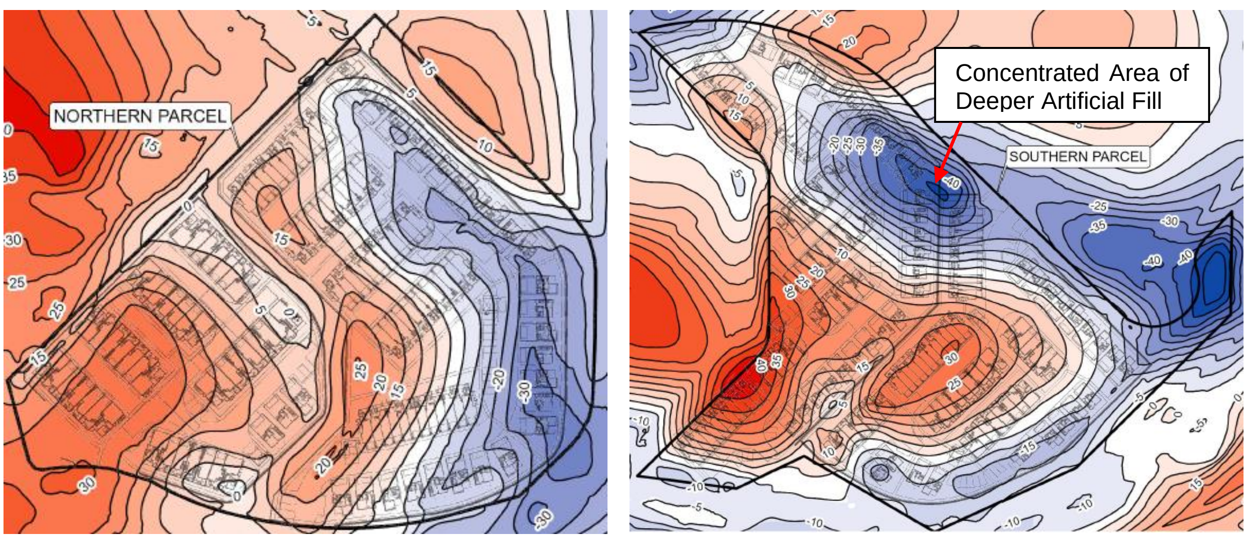
As discussed, our test pits and borings identified existing fill up to approximately 25 feet in thickness within the planned development area of the Northern and Southern Parcels. Artificial fill in the general area could extend to more than 50 feet below ground surface as indicated in Boring 1-B05, located in the landscaped area adjacent the existing cul-de-sac at the southern terminus of Campus Drive (the “panhandle”). The fill was placed between the late-1960s and the early-1980s. The city of San Mateo Building Department did not have records of fill placement. Based on our subsurface exploration, the fill generally consists of medium dense to dense clayey sand and stiff to hard sandy clay and silt with variable gravel content.

Based on the variable nature of artificial fill encountered and the inconsistency in density of the material with depth, we anticipate the artificial fill will potentially be susceptible to differential performance for building support if left unmitigated. Recommendations for treatment of existing artificial fill are provided in Section 5.2.

Although not anticipated, if encountered, construction debris and any other unsuitable material should be removed from the fill during processing and the fill may be used to backfill the excavation. Excavated fill material should be well mixed as part of processing prior to re-use in order to create a relatively homogenous material that will perform consistently.

Given that improvements are not currently planned within the “panhandle” of the Southern Parcel, where the deepest fill within the site boundary was encountered, removal of the deep fill in this portion of the site is not imperative. Due to the presence of the existing structure and nature of slopes in the southeastern portion of the Southern Parcel, the area located between Borings 1-B05 and 1-B14 was not explored. Based on review of the historic topographic maps discussed in Section 1.4, we estimate artificial fill thickness within the limits of proposed improvements generally up to approximately 25 feet, with the deepest portions concentrated along the perimeter where retaining walls are planned. A concentrated area of deeper fill (up to 40 feet) is also anticipated in the central eastern portion of the Southern Parcel as shown in Exhibit 4.5-1. Generally, excavation up to 15 feet below existing grades may be necessary to construct a toe keyway for future retaining walls (this will be deeper in the concentrated area identified in the Exhibit 4.5-1). Based on the results of our slope stability analysis, removal of existing artificial fill through excavation of a keyway constructed with temporary side slopes of 1:1 (horizontal:vertical) will result in satisfactory short-term stability conditions. However, it is important that the temporary 1:1 side slopes be observed during grading for evidence of instability by our Certified Engineering Geologist. If evidence of instability is observed, then it may be necessary to perform supplemental remedial measures or temporary structural shoring measures, as deemed appropriate.

EXHIBIT 4.5-1: Comparison of Existing and Historic Topography (Figure 7A)



4.6 EXPANSIVE SOIL

Our laboratory testing indicates that the artificial fill and bedrock at the site generally exhibit low to moderate shrink/swell potential with variations in moisture content. Shrink or swell due to seasonal changes in moisture and cause heaving and cracking of slabs-on-grade, pavements, and structures founded on shallow foundations. Conventional grading operations, incorporating fill placement specifications tailored to the expansive characteristics of the soil is a generally cost-effective measure to address the expansive potential of the foundation soils. We provide specific grading recommendations for compaction of the moderately expansive soil at the site.

4.7 2019 CBC SEISMIC DESIGN PARAMETERS

Based on the subsurface conditions encountered, and existing artificial fill is mitigated in accordance with our earthwork recommendations, we characterize the site as Site Class C in accordance with the 2019 CBC. We provide the 2019 CBC seismic design parameters in Table 4.7-1 below, which include design spectral response acceleration parameters based on the mapped Risk-Targeted Maximum Considered Earthquake (MCE_R) spectral response acceleration parameters.

TABLE 4.7-1: 2019 CBC Seismic Design Parameters

PARAMETER	VALUE
Site Class	C
Mapped MCE_R Spectral Response Acceleration at Short Periods, S_S (g)	2.11
Mapped MCE_R Spectral Response Acceleration at 1-second Period, S_1 (g)	0.88
Site Coefficient, F_A	1.2
Site Coefficient, F_V	1.4
MCE_R Spectral Response Acceleration at Short Periods, S_{MS} (g)	2.53
MCE_R Spectral Response Acceleration at 1-second Period, S_{M1} (g)	1.23
Design Spectral Response Acceleration at Short Periods, S_{DS} (g)	1.69
Design Spectral Response Acceleration at 1-second Period, S_{D1} (g)	0.82
Mapped MCE Geometric Mean (MCE_G) Peak Ground Acceleration, PGA (g)	0.91
Site Coefficient, F_{PGA}	1.2
MCE_G Peak Ground Acceleration adjusted for Site Class effects, PGA_M (g)	1.09
Long period transition-period, T_L	12

Latitude: 37.537380°, Longitude: -122.326171°

4.8 SOIL CORROSION POTENTIAL

One sample from each parcel was collected during our exploration and transported to CERCO Analytical for laboratory testing. The samples were tested for pH, resistivity, chloride ion, and sulfate concentrations. The test results provide an indication of the corrosion potential of the soil environment on buried concrete structures and metal pipes. The test results are summarized in the following table and are contained in full in the report prepared by CERCO Analytical (Appendix D).

TABLE 4.8-1: Soil Corrosivity Test Results

SAMPLE NUMBER AND DEPTH	REDOX (MV)	PH	RESISTIVITY (OHMS-CM)	CHLORIDE* (MG/KG)	SULFATE* (MG/KG)
1-B03 @ 4'-4.5'	200	8.35	3,400	N.D.	N.D.
1-B07 @ 3'-4.5'	240	8.22	2,600	N.D.	31

*Results reported on a wet weight basis
N.D. – None Detected

The resistivity measurements indicate the soil is moderately corrosive. As such, all buried iron, steel, cast iron, ductile iron, galvanized steel, and dielectric steel or iron should be properly protected against corrosion depending on the critical nature of the structure. We recommend a corrosion consultant provide specific design recommendations for any important buried metallic lines.

The sulfate ion concentrations reported as non-detect and 31 mg/kg of water-soluble sulfate (SO_4). The 2019 California Building Code (CBC) references the American Concrete Institute Manual, ACI 318-14 (Chapter 19) for concrete requirements. ACI Table 19.3.1.1 indicates the soil samples tested may be categorized as "S0" ('Not Applicable') sulfate exposure class. Considering a 'Not Applicable' sulfate exposure, there is no requirement for cement type or water cement ratio; however, a minimum concrete compressive strength of 2,500 psi is specified by the building code. For this sulfate range, we recommend a Type II cement and a concrete mix design for foundations and building slabs-on-grade that incorporate a maximum water-cement ratio of 0.50. It should be noted, however, that the structural engineering design requirements for concrete may result in more stringent concrete specifications.

Please also note that the testing was only performed on existing artificial site soils, not on bedrock or potential import material. Additional sulfate testing may be necessary once pad grading is complete to confirm the concrete type and strength recommended above.

5.0 EARTHWORK RECOMMENDATIONS

Supplemental recommendations may be necessary during the final grading plan review to refine the geotechnical remedial grading plans contained in this report. The final grading plans for the project should be reviewed by the Geotechnical Engineer.

The Geotechnical Engineer should be notified at least 48 hours prior to grading in order to coordinate with the grading contractor. Grading operations should be observed and tested by the Geotechnical Engineer.

5.1 DEMOLITION AND STRIPPING

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

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

5.2 EXISTING ARTIFICIAL FILL TREATMENT

Existing deposits of “man-made” artificial fills generally overlie bedrock in both parcels. Based on our review of historic topographic maps and the findings of our field exploration, it is anticipated that fill deposits range from approximately 1 foot up to 25 feet thick in future improvement areas; comparison of historic topographic maps suggest that fills are greater within the central-eastern portion of the Southern Parcel. In this area fills may extend up to 40 feet in depth below the existing site grade. Figure 2 displays the approximate lateral extent of existing fill at the site. Figure 7A presents the approximate fill depths (below existing grade) anticipated, based on comparison of present day and 1940s elevation contours along with information from our borings. We provide recommendations for addressing existing artificial fill at the site located within improvement areas.

5.2.1 Site Retaining Walls

Artificial fill should be removed to native material within keyway excavations supporting site retaining walls. We recommend removal of artificial fill encompassed within a 1:1 (horizontal: vertical) projection from keyway sidewalls (please refer to Section 5.9 for keyway construction recommendations). Temporary 1:1 slopes should be observed during excavation by our Certified Engineering Geologist for indications of instability. Artificial fill removal should be followed by processing and replacement of the existing fill in accordance with our fill placement recommendations.

5.2.2 Buildings

Existing artificial fill may be partially left in place beneath building footprint areas provided our foundation design and construction recommendations provided in Section 6.0 of this report are carefully implemented.

5.2.3 Other Improvements

For other structural areas (pavements, sidewalks, and other improvements) where artificial fill will be partially left in place, we recommend overexcavation, removal and replacement of existing artificial fills to at least 3 feet below bottom of improvement subgrade.

Our field representative should confirm that excavation bottoms (or benches) within existing artificial fill expose material that is stiff and free of deleterious matter.

5.3 EXPANSIVE SOIL MITIGATION

Atterberg limits testing performed on six samples collected from artificial fill present on site resulted in Plasticity Indices (PIs) ranging from 10 to 15, indicating low expansion potential. One sample collected approximately 3 feet below ground surface at Boring 1-B06 (located along the southeastern portion of the Northern Parcel) yielded a PI of 26, indicating moderate to high expansion potential.

To address potential pockets of material with high expansion potential, excavated fill material should be well mixed as part of processing prior to re-use in order to create a relatively homogenous material that will perform consistently. The mixed material that will be replaced as engineered fill is anticipated to have low to moderate expansion potential.

5.4 SELECTION OF MATERIALS

5.4.1 Soil/Rock

With the exception of organically contaminated materials (surficial soils which contains more than 3 percent organic content by weight), the site soils and bedrock derived materials are suitable for use as general fill. Other materials and debris, including trees with their root balls, should not be incorporated into the fill.

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

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

5.4.2 Reuse of On-Site Recycled Materials

If desired, the aggregate base from the existing pavement sections can be considered for use as recycled aggregate to replace some of the import aggregate base for pavements as well as for structural fill. The material will need to be broken down, but not pulverized, to have a maximum particle size less than 6 inches if used for fill and should conform to the gradations of aggregate base if used to substitute for roadway base.

5.5 DIFFERENTIAL FILL THICKNESS

For grading activities that create a differential fill thickness across individual building pads, mitigation to reduce the difference in fill thickness is necessary to reduce the differential performance of a shallow foundation system. We recommend a maximum differential fill thickness be kept to less than 15 feet across individual building pads to reduce the risk of excessive differential settlement. For a differential fill thickness exceeding 15 feet across an individual pad, we recommend performing subexcavation activities to bring this vertical distance to within the 15-foot tolerance and replacement of this material as engineered fill. The subexcavation area should include the entire structure footprint plus 5 feet beyond the edges of the building footprint.

5.6 ROADWAY AND UTILITY CORRIDOR EXCAVATION

Where bedrock is anticipated to be encountered within roadway and utility trench excavations, we recommend overexcavating the width of the roadway to at least the bottom of the deepest utility during mass grading to facilitate subsequent trenching for underground utilities installation. The overexcavation width should be between the proposed curbs.

5.7 FILL COMPACTION

5.7.1 Grading in Structural Areas

Areas to receive fill should be scarified to a minimum depth of 8 inches, moisture conditioned, and recompacted to provide adequate bonding with the initial lift of fill. Fills should be placed with a loose lift thickness no greater than 8 inches. The following compaction recommendations should be used for the placement and compaction of fills:

TABLE 5.7.1-1: Compaction and Moisture Content Requirements

DESCRIPTION	MATERIAL	RECOMMENDED RELATIVE COMPACTION (%)	MINIMUM MOISTURE CONTENT (PERCENTAGE POINTS ABOVE OPTIMUM)
General Fill	Site Soil	90	2
	Low-Expansive Import	90	1
Pavement Subgrade*	Site Soil	95	1
Pavement Aggregate Base	Class 2 Aggregate Base	95	0

*Upper 12 inches

Relative compaction refers to in-place dry density of the fill material expressed as a percentage of the maximum dry density as determined by ASTM D-1557. Optimum moisture is the moisture content corresponding to the maximum dry density.

5.7.2 Landscape Fill

In landscaping areas, the contractor should process, place, and compact fill in accordance with Section 5.7.1, except compact fill to at least 85 percent relative compaction.

5.7.3 Underground Utility Backfill

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

Utility trench backfill should conform to the recommendations in Section 5.7.1 and the requirements of the City of San Mateo. In general, uniformly graded gravel should not be used for pipe or trench zone backfill due to the potential for migration of soil into the relatively large void spaces present in this type of material and for movement of water along trenches backfilled with this type of material. If uniformly graded gravel is used, we recommend that it be encapsulated in 6-ounce filter fabric. Providing outlet locations into manholes or catch basins for water collected in granular trench backfill should also be considered.

Where utility trenches cross underneath structures, 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 structure. 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 a distance of at least 3 feet in each direction from the point where the utility crosses the structure perimeter.

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

5.7.4 Over-Optimum Soil Moisture Conditions

The contractor should anticipate encountering excessively over-optimum (wet) soil moisture conditions during winter or spring grading, or during or following periods of rain. Wet soil can make proper compaction difficult or impossible.

Wet soil conditions can be mitigated by:

1. Frequent spreading and mixing during warm dry weather.
2. Mixing with drier materials.
3. Mixing with a lime, lime-flyash, or cement product; or
4. Stabilizing with aggregate, geotextile stabilization fabric, or both.

Options 3 and 4 should be evaluated by ENGEO prior to implementation.

5.8 GRADED SLOPES

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. We recommend the following guidelines for cut and fill slope gradients:

TABLE 5.8-1: Slope Gradient Guidelines

SLOPE GRADIENT (HORIZONTAL:VERTICAL)	MAXIMUM CUT SLOPE HEIGHT (FEET)	MAXIMUM FILL SLOPE HEIGHT (FEET)
2:1	50	15
3:1	Greater than 50	Greater than 15

Where steeper slopes than those indicated above are planned, we recommend supplemental slope stabilization techniques, such as the use of geogrid reinforcement.

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

5.8.1 Slope Setbacks

Slope setbacks are intended to reduce the potential effects of long-term slope creep and possible earthquake-induced slope displacements on structures. The recommended slope setbacks for habitable structures are variable depending on slope height and soil conditions. For structures adjacent to slopes, we recommend a minimum setback of at least 10 feet or one-third of the slope height, whichever is greater, from the tops of slopes.

5.9 TOE KEYWAYS

To mitigate potential slope stability hazards, we recommend the construction of subdrained toe keyways at the toes of proposed mechanically stabilized earth (MSE) walls to be constructed in areas of existing artificial fill removal. Typical keyway designs consist of 24-foot-wide keyways constructed to a minimum depth of 5 feet into competent native or bedrock material, or extending below existing fill, alluvium, and/or colluvium and at least 3 feet into competent native materials, whichever is deeper.

Subsurface drainage systems should be installed within the keyways as recommended in a subsequent section. A typical keyway detail is presented on Figure 8. See Cross Sections on Figures 2A through 2D and 8 for further details regarding keyways.

Actual subsurface mitigation configurations (size and depths) should be shown on final 40-scale remedial grading plans. Fill should be adequately keyed/benched into competent material or bedrock materials, as determined by a geologist from our firm. The actual depth and location of the keyways, subexcavated benches, and locations of subdrainage may then be slightly modified in the field based on the actual field conditions and geometry exposed during grading.

5.10 SUBSURFACE DRAINAGE FACILITIES

We recommend subsurface drainage systems for keyways, and at the base of removal areas. Secondary bench subdrains may also be required, depending on the height of the fill slope and the slope of the underlying native terrain. In addition, observed seepage areas should be controlled in development areas through the use of subdrains. Positive fall of at least ½ (selectively) to 1 percent towards an approved outlet should also be provided for all subdrains.

A general keyway detail is presented on Figure 8. Subdrain systems should consist of a minimum 6-inch-diameter perforated pipe encased in Caltrans Class 2 permeable material, or crushed rock wrapped in filter fabric. The subdrain pipe and drainage blanket should meet the requirements contained in Sections 2.4 and 2.6/2.7 of the Supplemental Recommendations. As an alternative, prefabricated geocomposite drainage material could be considered in lieu of the granular medium above the subdrain zone. Prefabricated geocomposite drainage materials should follow the recommendations outlined in Section 2.8 of the Supplemental Recommendations.

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

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

5.11 SITE DRAINAGE

The project civil engineer is responsible for designing surface drainage improvements. With regard to geotechnical engineering issues, we recommend that finish grades be sloped away from buildings and pavements to the maximum extent practical to reduce the potentially damaging effects of expansive soil. The latest California Building Code Section 1804.4 specifies minimum slopes of 5 percent away from foundations. Where lot lines or surface improvements restrict meeting this slope requirement, we recommend that specific drainage requirements be developed. We recommend the following:

1. Discharge roof downspouts into closed conduits and direct away from foundations to appropriate drainage devices.

2. Consider the use of rear lot surface drainage collection systems to reduce overland surface drainage from back to front of lot.

3. Do not allow water to pond near foundations, pavements, or exterior flatwork.

6.0 FOUNDATION RECOMMENDATIONS

6.1 BUILDING PAD PREPARATION

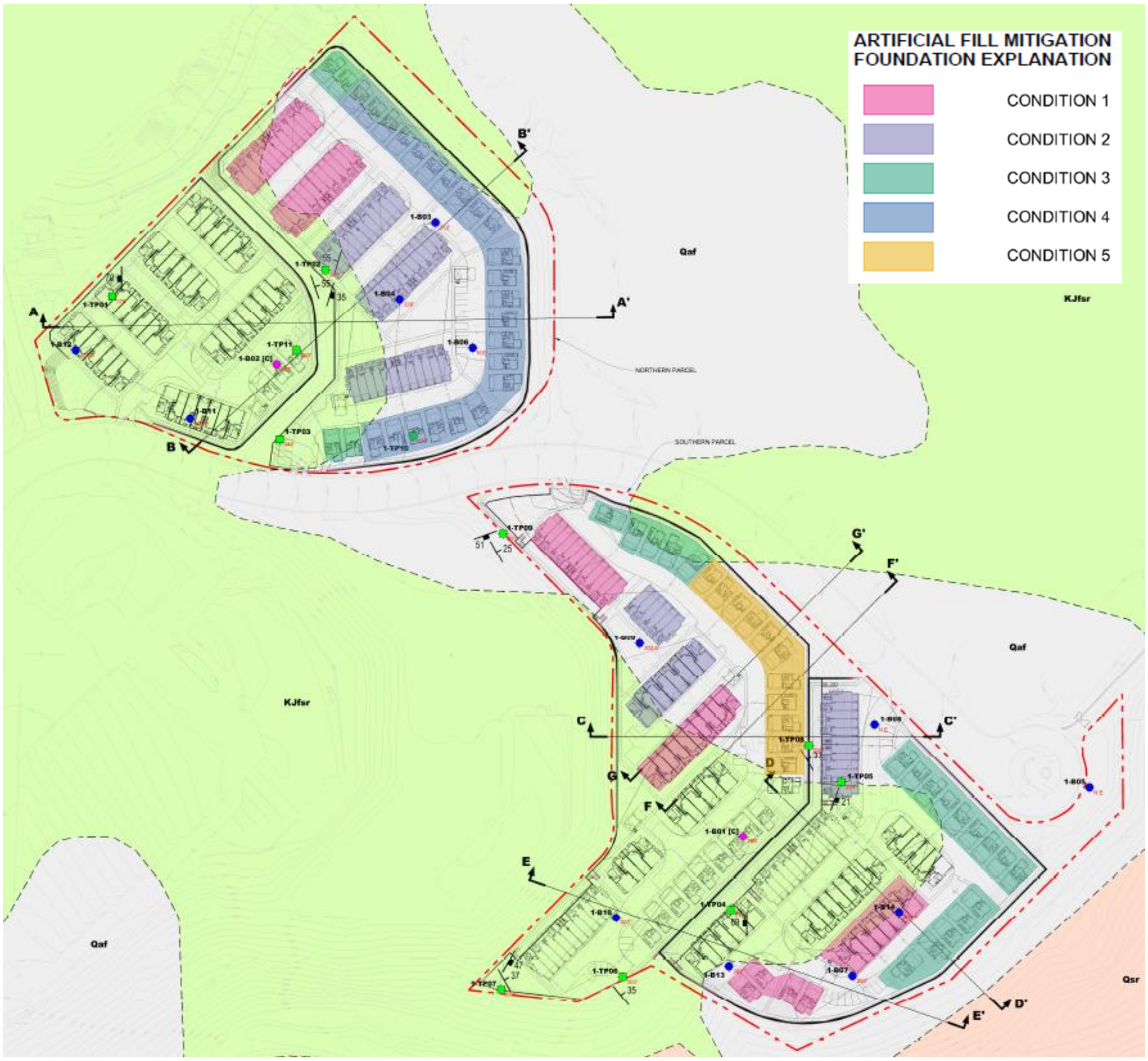
Building pads constructed over transitions between different soil and bedrock conditions may be subject to differential soil movements. To mitigate potentially differential movement, we recommend that all building pads be overexcavated and reconstructed to create uniform subgrade conditions. The following recommendations will provide a uniform, moisture conditioned state for foundation subgrade soil upon which foundation mats may be constructed. We provide recommendations for fill placement in Section 5.7.1 of this report.

Artificial fill depths up to 40 feet and differential fill thicknesses up to 25 feet (within a single building pad area) are anticipated for pads where existing artificial fill is present. The pads located within the "artificial fill zone" are identified in Exhibit 6.1-1, below. Constructing a thickened, uniform, engineered fill pad for foundation support is critical to achieving satisfactory performance of building foundations where existing artificial fill will be partially left in place.

For pads not requiring artificial fill mitigation, we recommend overexcavation of soil/rock beneath building pads to a minimum depth of 2 feet below finished pad grade and replacement of the overexcavated material with uniformly mixed compacted fill. The overexcavation should be performed over the entire flat pad area extending to a minimum of 5 feet beyond any building edge. For pads located within the artificial fill zone, we recommend overexcavation of soil/rock beneath building pads to a minimum depth of 5 feet below finished pad grade and replacement of the overexcavated material with uniformly mixed compacted fill. The overexcavation should be performed over the entire flat pad area extending to a minimum of 5 feet beyond any building edge.

Furthermore, for artificial fill zone pads identified as Condition 5 (Exhibit 6.1-1), where the deepest fill within improvement limits is anticipated, we recommend placement of reinforcement geogrid at the base of the engineered fill pad excavations. Geogrid should consist of a biaxial grid (such as BX1200 or approved equivalent). Geogrid should be installed in accordance with the manufacturer's specifications.

EXHIBIT 6.1-1: Artificial Fill Mitigation Zone Foundations



6.2 MAT DESIGN RECOMMENDATIONS

We recommend that the proposed single-family and multi-family residential structures be supported on structural reinforced mat foundations, such as conventional steel-reinforced or post-tensioned (PT) mat systems bearing on engineered fill.

Mats located within the artificial fill mitigation zone may be designed to impose a maximum average bearing pressure of 1,500 pounds per square foot (psf) for dead-plus-live loads with maximum localized bearing pressures of 2,000 psf at column or wall loads. Mats located on bedrock (outside the artificial fill mitigation zone) may be designed to impose a maximum average bearing pressure of 3,000 pounds psf. Allowable bearing pressures can be increased by one-third for wind or seismic loads.

Mat design may consider a modulus of subgrade reaction of 120 pounds per square inch per inch (psi/in). The foundation system used should be sufficiently stiff to move as rigid units with minimum differential movement. We recommend designing for a rigidity of 1/600. We recommend that the mats be designed for a center span criteria of 20 feet and an edge cantilever criteria of 6 feet.

Lateral loads may be resisted by friction along the base and by passive pressure along the sides of foundations. The passive pressure is based on an equivalent fluid pressure in pcf. We recommend an allowable passive lateral earth pressure of 300 pcf (neglecting the upper 12 inches of embedment for passive resistance). A coefficient of friction of 0.35 may be used considering sliding. The above allowable values include a factor of safety of 1.5. Passive lateral pressure should not be used for foundations on or above slopes.

Based upon the existing soil conditions, and the 2004 (Third Edition with 2008 errata) Post-Tensioning Institute, "Design of Post-Tensioned Slabs-On-Ground" manual, we recommend the following soil criteria for the post-tensioned (PT) mat foundations.

TABLE 6.2-1: Post-Tensioned Mat Design Recommendations

CONDITION	CENTER LIFT	EDGE LIFT
Edge Moisture Variation Distance, e_m (feet)	9.0	4.8
Differential Soil Movement, y_m (inches)	0.4	1.0

The Structural Engineer should determine the mat thickness using the geotechnical recommendations in this report; we defer to the professional judgment of the Structural Engineer on the necessary mat thickness. We recommend that mats have a thickened edge at least 2 inches greater than the mat thickness. We recommend that the thickened edge be at least 12 inches wide.

ENGEO should be retained to review the mat foundation design. Underlay mats with a moisture reduction system as recommended below.

6.3 SETTLEMENT CONSIDERATIONS

Foundation mats should be sufficiently rigid to accommodate potential differential settlement for buildings where artificial fill will be partially left in place. Foundation design for buildings located within the mitigation zone should consider the estimated differential settlements provided in Table 6.3-1, with condition designations presented in Exhibit 6.1-1.

The estimated differential settlements for design consider construction of a 5-foot-thick engineered fill pad. The differential settlement values should be assumed to act over a 30-foot distance.

TABLE 6.3-1: Estimated Differential Settlements for Mitigation Zone Foundations

FOUNDATION MITIGATION CONDITION	BUILDING TYPE	ESTIMATED DIFFERENTIAL SETTLEMENT
Condition 1	Multi-Family	1 inch
Condition 2	Multi-Family	1½ inches
Condition 3	Single-Family	1 inch
Condition 4	Single-Family	1½ inches
Condition 5	Single-Family	1¾ inches

6.4 SLAB MOISTURE VAPOR REDUCTION

When buildings are constructed with concrete slab-on-grade, water vapor from beneath the slab will migrate through the slab and into the building. This water vapor may be reduced but not stopped. Vapor transmission may 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 in accordance with 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.

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. If a sand or pea gravel layer atop the vapor retarder membrane is used in combination with a structural mat or post-tensioned mat, we recommend that the edges of the mat be thickened by the thickness of the granular layer to cutoff moisture transmission between the vapor retarder and the mat.

7.0 RETAINING WALLS

As mentioned in Section 1.3, retaining walls are proposed around the site to accommodate grade changes between the existing terrain and the proposed development. Retaining walls are generally planned at locations where previously graded fill slopes have been constructed along site perimeters. Historic documents indicate the existing fill slopes along site perimeters were constructed in the 1970s. Based on the findings of our exploration (which encountered variable artificial fill conditions), the existing graded slopes require retention or alternate mitigation to achieve stability under seismic conditions. Engineered retaining wall systems designed and constructed in accordance with our recommendations will have improved static and seismic stability compared to the existing slope conditions. Other relative benefits of incorporating retaining walls as part of site improvement include the following:

1. Reduced long-term maintenance compared to grades slopes, which require installation and maintenance of surface erosion control measures.
2. Reduced number of necessary surface drainage collection points. Surface drainage can be collected at top of walls and directed to appropriate outlet locations. Comparably, graded slopes may require more than one drainage interception point along the height of the slope to collect sheet flow.
3. Reduction for potential of pocket failures that can occur on graded slopes as a result of surface layer softening over time (due to environmental factors).

4. Reduced grading disruption zone. Graded slopes generally need to be constructed with a 2:1 (Horizontal:Vertical) gradient for long-term stability. This results in area of grading that disturbs a larger zone compared to grading for site retaining wall construction, which can be performed through construction of temporary 1:1 slopes. The larger area of grading can increase likelihood of destabilizing bedrock pockets (that wouldn't otherwise be exposed and disturbed).

Retaining wall systems with retained heights extending up to approximately 28 feet are planned. Walls proposed in cut areas should be constructed top-down so that the material at higher elevations is retained while the excavation operations continue. For these areas, we recommend either a soil nail wall or a soldier pile and lagging wall system be used. For walls which transition between cut and fill conditions (or where removal of artificial fill is anticipated in a portion of the wall), a soldier pile and lagging system is better suited. For walls planned in proposed fill areas, and areas where existing artificial fill will be removed, we recommend mechanically stabilized earth (MSE) walls. For structures adjacent to retaining walls, we recommend a minimum setback of at least 10 feet from the tops of walls.

7.1 SOIL NAIL WALLS

Where a permanent soil nail wall system is selected for site walls, we recommend the walls are designed and constructed in accordance with FHWA0-IF-03-017 – Geotechnical Engineering Circular No. 7. The following soil and rock parameters and factors of safety should be used in the design of soil nail walls for proposed walls in the northern and southern parcels. The actual bond strength should be confirmed by load testing during construction.

TABLE 7.1-1: Soil Nail Wall Design Parameters

SOIL MATERIALS	UNIT WEIGHT (pcf)	FRICTION ANGLE (degrees)	COHESION (psf)	ULTIMATE BOND STRESS (psi)
Franciscan Complex Bedrock	160	40	2500	30

In order to ensure that grout flows along the length of the soil nail and no voids form during grouting, soil nails should be designed and constructed with a minimum inclination of 10 degrees below horizontal. We recommend that a maximum horizontal spacing of 6 feet on center horizontally with a minimum length of 15 feet and an installation angle of 15 degrees from horizontal.

The ultimate bond strength parameters provided in Table 0-1 should be confirmed during construction by load testing performed by the design-build contractor and observed by ENGEO. Based on the results of the load testing, soil nail lengths may be adjusted.

In addition, Section 5.9 of FHSWA0-IF-03-017 includes recommended factors of safety for the allowable stress design method, some of which have been summarized below. The following minimum Factors of Safety (FS) should be considered for the soil nail wall design.

<u>Recommended Factors of Safety – Internal Stability</u>

- Pullout Resistance (Bond Strength): FS = 2.0 (temporary and permanent)
- Nail Bar Tensile Strength: FS = 1.8 (temporary and permanent)
- Pullout Resistance: FS = 1.5 (seismic)
- Nail Bar Tensile Strength: FS = 1.35 (seismic)

<u>Recommended Factors of Safety – External Stability</u>

- Global Stability (long-term): FS = 1.35 (temporary), 1.5 (permanent), and 1.1 (seismic)
- Global Stability (short-term): FS = 1.3 (temporary and permanent)
- Sliding: FS = 1.3 (temporary), 1.5 (permanent), and 1.1 (seismic)
- Bearing Capacity: FS = 2.5 (temporary), 3.0 (permanent), and 2.3 (seismic)

Shotcrete facing should have appropriate reinforcement steel designed to resist structural loads as well as stresses caused by regional temperature variations and concrete shrinkage. The shotcrete facing should be embedded at least 12 inches below grade along the bottom of the walls.

Drained soil parameters have been provided above, as all potential soil nail walls will be above the regional groundwater table; however, zones of groundwater seepage may be encountered. Accordingly, the wall should be provided with drainage facilities to prevent the build-up of hydrostatic pressures behind them. Wall drainage considerations are provided in Section 7.1.1.

Construction should be performed by a contractor experienced in soil nailing. The successful performance of soil-nailing wall systems is dependent on proper installation methods. The Geotechnical Engineer should perform full-time monitoring of nail installation and testing. The actual bond between the grout and the nail can vary significantly with the method of installation.

It is imperative that a comprehensive testing program be implemented to verify that the design loads can be attained. Load tests should be performed in accordance with FHWA0-IF-03-017. We recommend that at least two sacrificial nails per wall located at the discretion of ENGEO should be successfully tested prior to production nailing using the same equipment and methods to be used for production work. The verification test nails should have a minimum bond length of 10 feet and a minimum unbonded length of 10 feet and a minimum unbonded length of 5 feet. The test soil nail bars should be sized so that the test load does not exceed 80 percent of the yield or ultimate strength of the steel and should be loaded to 200 percent of the design load.

Five percent of the production nails should be proof tested to 150 percent of the design load. Production nails to be proof load tested should be selected by ENGEO. The proof load test soil nails should have a bonded length and unbonded free length, to be specified by ENGEO during construction. This will require close interaction between the geotechnical engineer and the contractor. Creep tests performed in accordance with FHWA0-IF-03-017 should be incorporated into verification and proof load testing. Upon completion of testing, the unbonded length should be backfilled with structural grout.

Zones of water seepage may also be expected. The contractor should be advised of the potential presence of these conditions and should be prepared to implement appropriate drilling methods. Holes should be drilled without a loss of ground, which may require casing or augercast installation methods, particularly in areas where groundwater seepage or highly weathered materials within fractures may be encountered. Holes should not be drilled with fluids or water. Nails should be installed and grouted immediately upon completion of drilling.

7.1.1 Soil Nail Wall Drainage Considerations

Soil nail walls should be designed with positive drainage away from the walls. In the event that positive drainage cannot be maintained, we recommend that a concrete drainage ditch or collection drain system be installed along the top of the slope protection system to divert water accumulated along this area. Finished grades along the bottom of the facing should allow for positive drainage away from the wall of at least 2 percent to a suitable drainage location.

Although the soil nail walls are anticipated to be constructed above the regional groundwater table, zones of free water seepage may be encountered. The walls should be provided with drainage facilities to prevent the build-up of hydrostatic pressures behind the walls. Wall drainage should be provided using prefabricated synthetic wall drain panels that are suitably attached to the soil/rock and hydraulically connected at the bottom of the wall to 4-inch-diameter perforated pipe (SDR 35 or approved equivalent) wrapped in filter fabric (8-ounce minimum). Geocomposite drainage boards should be installed behind the shotcrete facing extending up to within 1 foot of the top of the facing protection system. The drainage boards should be at least 12 inches wide and spaced no greater than 8 feet apart along the slope. The drainage boards should extend to a collection pipe (SDR 35 or approved equivalent) behind the shotcrete facing near the bottom of the facing protection to collect and allow discharge of accumulated water. Drainage should be collected by pipes and directed to an outlet.

7.2 SOLDIER PILE RETAINING WALLS

Soldier pile retaining walls may be designed using the lateral equivalent fluid pressures presented below, which do not include increases due to surcharge or hydrostatic pressures:

TABLE 7.2-1: Cantilever Lateral Earth Pressures

BACKFILL SLOPE CONDITIONS	EQUIVALENT FLUID PRESSURES (pcf)
Level	50
4:1	55
3:1	60
2:1	70

Passive pressures acting on retaining walls may be assumed as 300 pounds per cubic foot (pcf) for engineered fill and 450 pcf for foundations embedded into bedrock, provided that the area in front of the retaining wall is level for a distance of at least 10 feet or three times the depth of foundation and keyway, whichever is greater.

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

7.2.1 Soldier Pile Wall Drainage Considerations

The retaining walls should be provided with drainage facilities to prevent build-up of hydrostatic pressures behind them. Wall drainage may be provided using a 4-inch-diameter perforated pipe embedded in Class 2 permeable material or free-draining gravel surrounded by synthetic filter fabric. The width of the drain blanket should be at least 12 inches. The drain blanket should extend to about 1 foot below the finished grades. As an alternative, prefabricated synthetic wall drain panels can be used. The upper 1 foot of wall backfill should consist of clayey soil. Drainage should be collected into solid pipes and directed to an outlet approved by the Civil Engineer. Synthetic filter fabric should be preapproved by the Geotechnical Engineer prior to delivery.

7.3 MECHANICALLY STABILIZED EARTH (MSE) WALLS

We recommend MSE walls be founded on prepared subgrade in conformance with recommendations for fill placement provided in Section 5.7.1 of this report. In addition, we have recommend material from existing artificial fill mixed with bedrock derived material will be used as the foundation fill, retained soil, and reinforced fill soil for the MSE walls. Accordingly, the following soil material parameters should be incorporated in the MSE wall design.

TABLE 7.3-1: Soil Material Parameters

CONDITION	COHESION (c') (pcf)	FRICTION ANGLE (∅') (degrees)	UNIT WEIGHT (γ) (pcf)
Reinforced Fill	300	33	125
Retained Soil/Rock	300	33	125
Foundation Fill	300	33	125

The MSE design should incorporate the minimum embedment and grid length recommendations provided in Tables 7.3-2 and 7.3-3.

TABLE 7.3-2: Minimum Wall Embedment

FOREGROUND CONDITION	EMBEDMENT DEPTH (feet)*
Level	1
3:1	2
2:1	3

*Below lowest adjacent grade, does not include leveling pad

TABLE 7.3-3: Minimum Geogrid Length

FOREGROUND CONDITION	BACKGROUND CONDITION	MINIMUM GEOGRID LENGTH*
Level	Level	0.7*H
Level	2:1	H
2:1	2:1	1.4*H

*H = total height of MSE wall (exposed height plus embedment depth), does not include leveling pad

A global stability check should be performed once the MSE wall design is complete. The minimum geogrid length should be determined by either the MSE design or those required by the slope stability analysis.

Additionally, we recommend that the following minimum factors of safety be incorporated in the MSE wall design.

TABLE 7.3-4: External Stability

CONDITION	SAFETY FACTOR (STATIC/SEISMIC)
Sliding	1.5 / 1.1
Bearing Capacity	2.0 / 1.5
Overturning	2.0 / 1.5

TABLE 7.3-5: Internal Stability

CONDITION	SAFETY FACTOR (STATIC/SEISMIC)
Pull-out Resistance	1.5 / 1.1

The following general assumptions and design guidelines should also be incorporated into MSE wall design.

- Material generated from artificial fill removal and bedrock cuts may be used as the foundation soil, retained soil, and reinforced fill soil, provided it is well mixed and processed. Light equipment should be used during backfill compaction to reduce possible overstressing of the walls.
- Geogrid reinforcement should extend horizontally from the wall face into the backfill.
- If geogrid reinforcement extends under the building foundation, the upper layer of geogrid should extend over the length of the building footprint.
- Where landscaping is planned at the base of walls, wall embedment should be increased by a depth equal to the thickness of topsoil below finish grade, or 12 inches, whichever is greater, to account for the decreased resistance provided by topsoil.
- Where trees are planned near the base of MSE walls, a root barrier should be installed if root growth is anticipated to extend within the wall location.

7.3.1 MSE Wall Drainage Considerations

Construct a graded rock drain behind the MSE walls to reduce hydrostatic lateral forces. The drain shall consist of a minimum 12-inch-wide layer of Class 2 Permeable Filter Material (Caltrans Specification 68-2.02F) placed directly behind the wall. Extend rock drains from the wall base to within 12 inches of the top of the wall. Place a minimum of 4-inch-diameter perforated pipe (glued joints and end caps) at the base of the wall, inside the rock drain and fabric, with perforations placed down. Place pipe at a gradient at least 1 percent to direct water away from the wall by gravity to a drainage facility.

7.4 SURCHARGE LOADING FOR RETAINING WALLS

Surcharge loads from buildings, vehicles, hardscape, or paving should be included in the wall design if the surcharge loading is situated above a 1:1 line of projection extending up from the rear base edge of the bottom block. A minimum surcharge load equal to 150 psf should be considered for traffic loading, where applicable. The structural engineer should be consulted regarding building surcharge loading.

7.5 SEISMIC DESIGN FOR RETAINING WALLS

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

$$\Delta P = 15 \times H^2$$

H is the design height of the wall (in feet) and ΔP is the active incremental seismic force in pounds per foot of wall. This force has a horizontal direction and should be applied at 0.6 x H from the base of the wall.

8.0 PAVEMENT DESIGN

8.1 FLEXIBLE PAVEMENTS

Due to the clayey nature of surface soils across the site, we provide pavement design recommendations considering an R-value of 5. Using estimated traffic indices for various pavement loading requirements, we developed the following recommended pavement sections using Topic 633 of the Caltrans Highway Design Manual (including the asphalt factor of safety), presented in the table below.

TABLE 8.1-1: Recommended Asphalt Concrete Pavement Sections

TRAFFIC INDEX	PAVEMENT SECTION	
	ASPHALT CONCRETE (INCHES)	CLASS 2 AGGREGATE BASE (INCHES)
5	3	10
6	3½	13
7	4	16

The Civil Engineer should determine the appropriate traffic indices based on the estimated traffic loads and frequencies.

8.2 RIGID PAVEMENTS

Use concrete pavement sections to resist heavy loads and turning forces in areas such as fire lanes or trash enclosures. Final design of rigid pavement sections, and accompanying reinforcement, should be performed based on estimated traffic loads and frequencies. We recommend the following minimum design sections for rigid pavements:

- Use a minimum section of 6 inches of Portland Cement concrete over 8 inches of Caltrans Class 2 Aggregate Base.
- Concrete pavement should have a minimum 28-day compressive strength of 3,500 psi.
- Provide minimum control joint spacing in accordance with Portland Cement Association guidelines.

8.3 PAVEMENT SUBGRADE PREPARATION

Pavement subgrade preparation should comply with the following minimum requirements:

- All pavement subgrades should be scarified to a depth of 10 inches below finished subgrade elevation and compacted in accordance with Section 5.7.1. Pavement subgrades should also be prepared in accordance with City of San Mateo requirements if they are located in public streets.

- Subgrade soils should be in a stable, non-pumping condition at the time aggregate base rock materials are placed and compacted. Proof-rolling with a heavy wheel-loaded piece of construction equipment should be implemented. Yielding materials should be appropriately mitigated, with suitable mitigation measures developed in coordination with the client, contractor, and Geotechnical Engineer.
- Adequate provisions must be made such that the subgrade soils and aggregate base rock materials are not allowed to become saturated.
- Aggregate base materials should meet current Caltrans specifications for Class 2 Aggregate Base and should be compacted in accordance with Section 5.7.1. Proof-rolling with a heavy wheel-loaded piece of construction equipment should be implemented after placement and compaction of the aggregate base. Yielding materials should be appropriately mitigated, with suitable mitigation measures developed in coordination with the client, contractor, and Geotechnical Engineer.

8.4 CUT-OFF CURBS

Saturated pavement subgrade or aggregate base can cause premature failure or increased maintenance of asphalt concrete pavements. This condition often occurs where landscape areas directly abut and drain towards pavement. If it is desired to install pavement cutoff barriers, they should be placed 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 6 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, the cutoff barrier may be eliminated.

8.5 SECONDARY SLABS-ON-GRADE

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

Exterior slabs should slope away from the buildings to prevent water from flowing toward the foundations. Site soil should be moistened just prior to concrete placement.

We recommend that flatwork leading to a building entrance area be structurally independent of the building foundation to allow for differential movement between the flatwork and the building. Where smooth transition to provide access is necessary (ADA ramps), a hinge-slab should be designed to accommodate movements of approximately ½ inch. Flatwork should be reinforced to allow for the appropriate span in the event of settlement. Maintenance or replacement of entry slabs should also be expected following a seismic event as the ground settles at the perimeter of buildings.

9.0 STORMWATER BIORETENTION AREAS

Due to the consistency and fines content of near-surface site soil and bedrock, the site soil and bedrock are expected to have very low permeability value for stormwater infiltration. Infiltration tests should be performed to provide site specific values for design if on-site infiltration is desired.

The majority of the site includes USDA classified group C soils that have relatively slow infiltration rate when thoroughly wet. Therefore, best management practices should assume that very limited stormwater infiltration will occur at the site unless and engineered system is designed.

If bioretention areas are implemented, we recommend that, when practical, they be planned a minimum of 5 feet away from structural site improvements, such as buildings, streets, retaining walls, and sidewalks/driveways. When this is not practical, bioretention areas located within 5 feet of structural site improvements should:

1. Be constructed with structural side walls capable of withstanding the loads from the adjacent improvements.
2. Incorporate filter material compacted to between 85 and 90 percent relative compaction (ASTM D1557, latest edition) and a waterproofing system designed to reduce the potential for moisture transmission into the subgrade soil beneath the adjacent improvement.

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

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

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

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

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

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

9.1 LANDSCAPING CONSIDERATION

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

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

10.0 CONSTRUCTION MONITORING

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

1. Review the final grading and foundation plans and specifications prior to construction to evaluate whether our recommendations have been implemented, and to provide additional or modified recommendations, as needed. This also allows us to check if any changes have occurred in the nature, design or location of the proposed improvements and provides the opportunity to prepare a written response with updated recommendations.
2. Perform construction monitoring to check the validity of the assumptions we made to prepare this report. Earthwork operations should be performed under the observation of our representative to check that the site is properly prepared, the selected fill materials are satisfactory, and that placement and compaction of the fills has been performed in accordance with our recommendations and the project specifications. Sufficient notification to us prior to earthwork is important.

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

10.1 REMEDIAL GRADING PLANS

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

11.0 LIMITATIONS AND UNIFORMITY OF CONDITIONS

This report presents geotechnical recommendations for design of the development discussed in Section 1.3 for this report. 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; 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 are representative of the actual subsurface conditions across the site. Considering possible underground variability of soil 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, ENGEO must be notified 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, the proper regulatory officials must be notified immediately.

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

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

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

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FIGURES

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FEET

BASEMAP SOURCE: ESRI MAPPING SERVICE 2019

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VICINITY MAP
PENINSULA HEIGHTS
SAN MATEO, CALIFORNIA

PROJECT NO. : 16683.000.000	FIGURE NO. 1
SCALE: AS SHOWN	
DRAWN BY: JV	CHECKED BY: YZ

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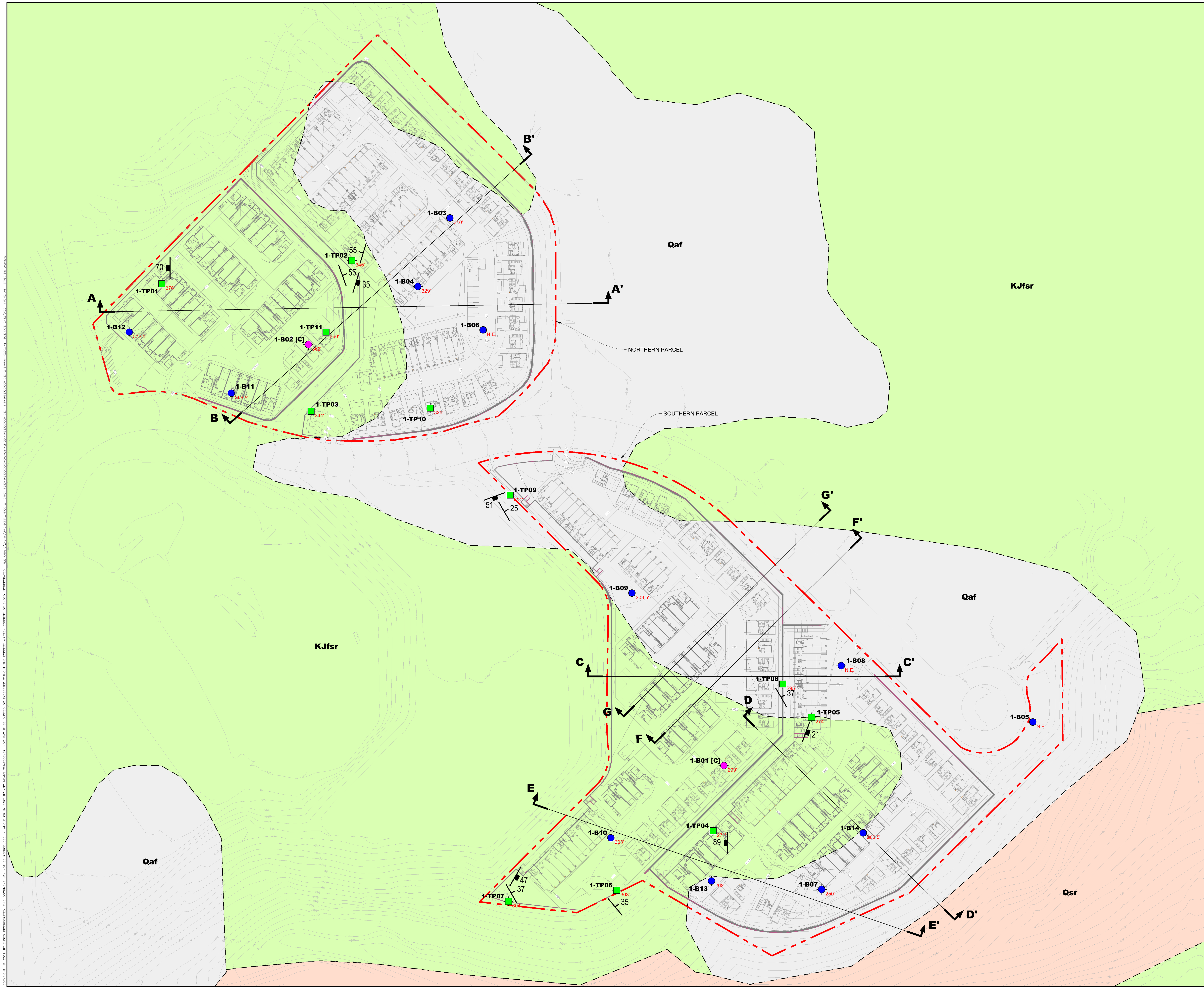

EXPLANATION

ALL LOCATIONS ARE APPROXIMATE

1-B02 [C]	ROCK CORES (ENGEO, 2019)
1-B14	SOIL BORINGS (ENGEO, 2019)
1-TP11	TEST PITS (ENGEO, 2019)
Qaf	EXISTING ARTIFICIAL FILL
Qsr	SLOPE WASH/RAVINE FILL
KJfsr	FRANCISCAN COMPLEX - PREDOMINANTLY GRAYWACKE WITH OCCASIONAL SHALE INTERBEDS
	GEOLOGIC CONTACT
F F'	CROSS SECTION LOCATION
51	STRIKE AND DIP OF JOINT
25	STRIKE AND DIP OF BEDDING
300'	APPROXIMATE BEDROCK ELEVATION SHOWN IN FEET

N

0 70 FEET

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SITE PLAN
PENINSULA HEIGHTS
SAN MATEO, CALIFORNIA

PROJECT NO.: 16683.000.000
SCALE: AS SHOWN
DRAWN BY: SPPE CHECKED BY: YZ
FIGURE NO. 2

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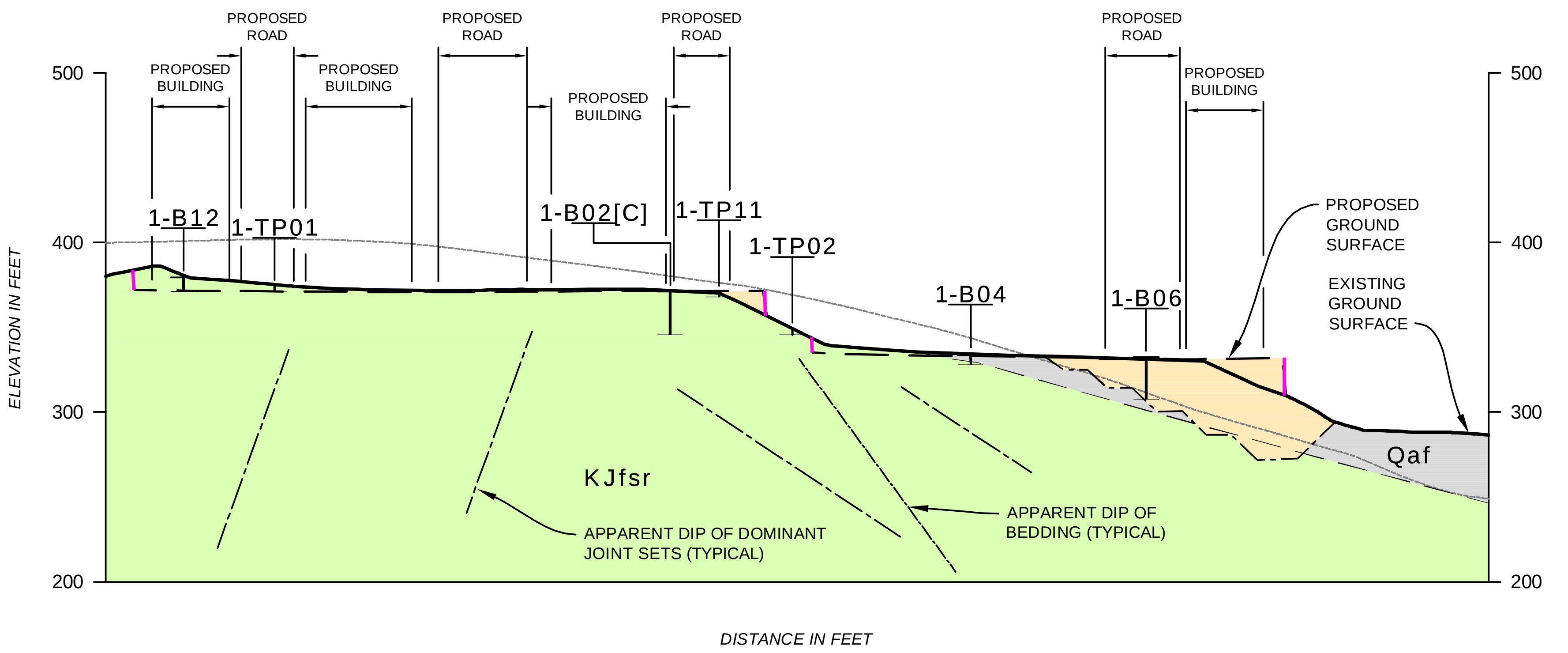

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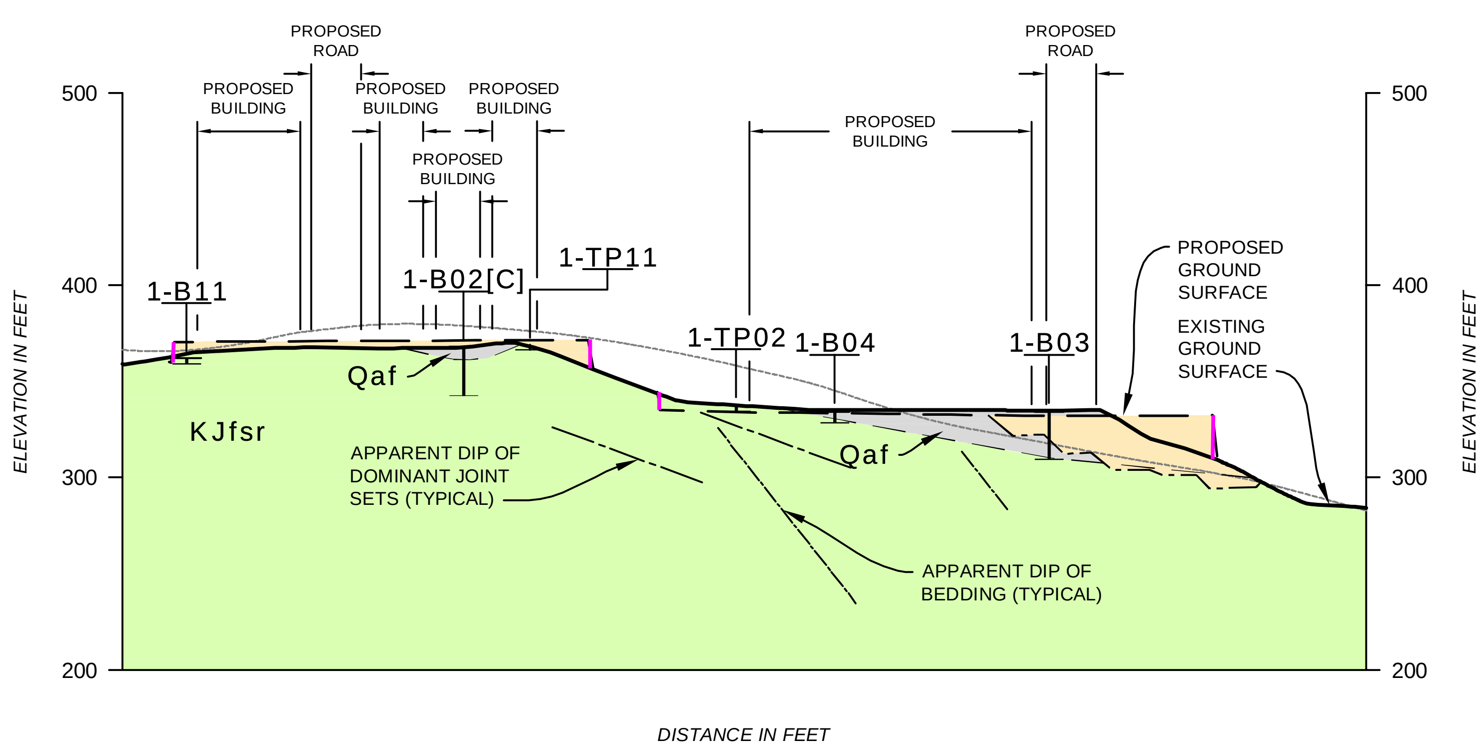

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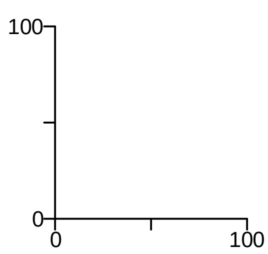

EXPLANATION

ALL LOCATIONS ARE APPROXIMATE

1-TP08	TEST PIT (ENGEO, 2019)		GEOLOGIC CONTACT	Qaf	EXISTING ARTIFICIAL FILL
1-B14	SOIL BORING (ENGEO, 2019)		ORIGINAL GROUND SURFACE (USGS 1947)	KJfsr	FRANCISCAN COMPLEX - PREDOMINANTLY GRAYWACKE WITH OCCASIONAL SHALE INTERBEDS
1-B01[C]	ROCK CORES (ENGEO, 2019)		PROPOSED CORRECTIVE GRADING		PROPOSED ENGINEERED FILL
			PROPOSED WALL		

NOTE: PROPOSED ENGINEERED FILL FOR BUILDING PADS NOT DEPICTED IN CROSS SECTIONS. REFER TO REPORT FOR BUILDING PAD PREPARATION RECOMMENDATIONS.

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CROSS SECTIONS
PENINSULA HEIGHTS
SAN MATEO, CALIFORNIA

PROJECT NO.: 16683.000.000
SCALE: AS SHOWN
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CHECKED BY: YZ
FIGURE NO. 2A

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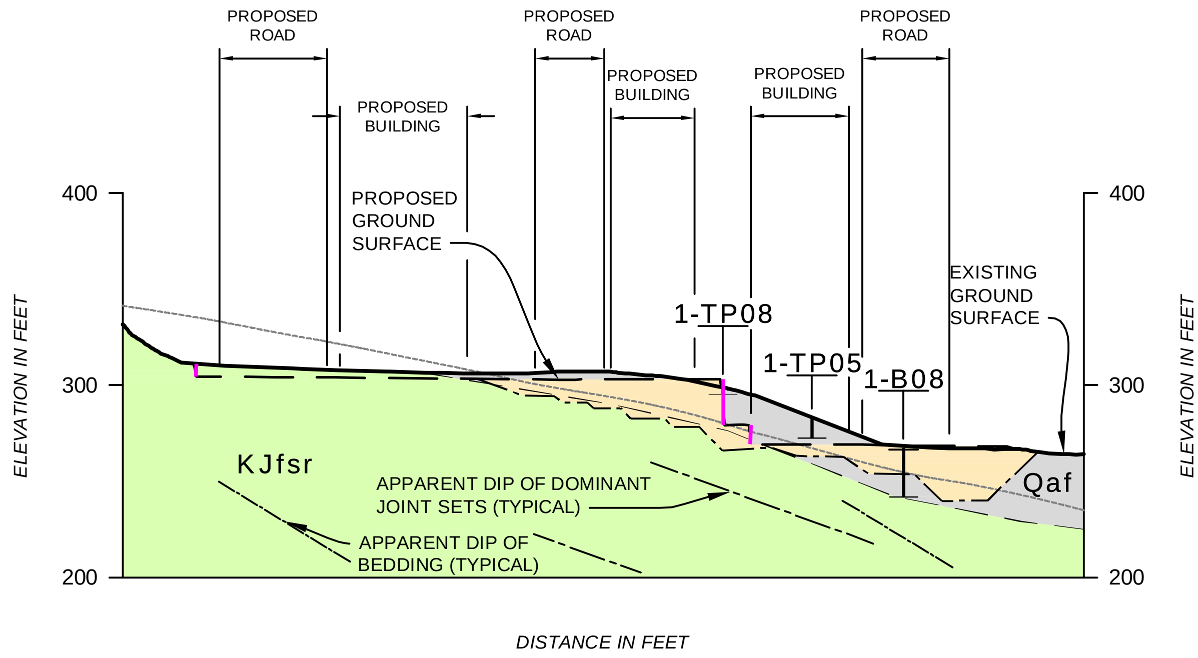

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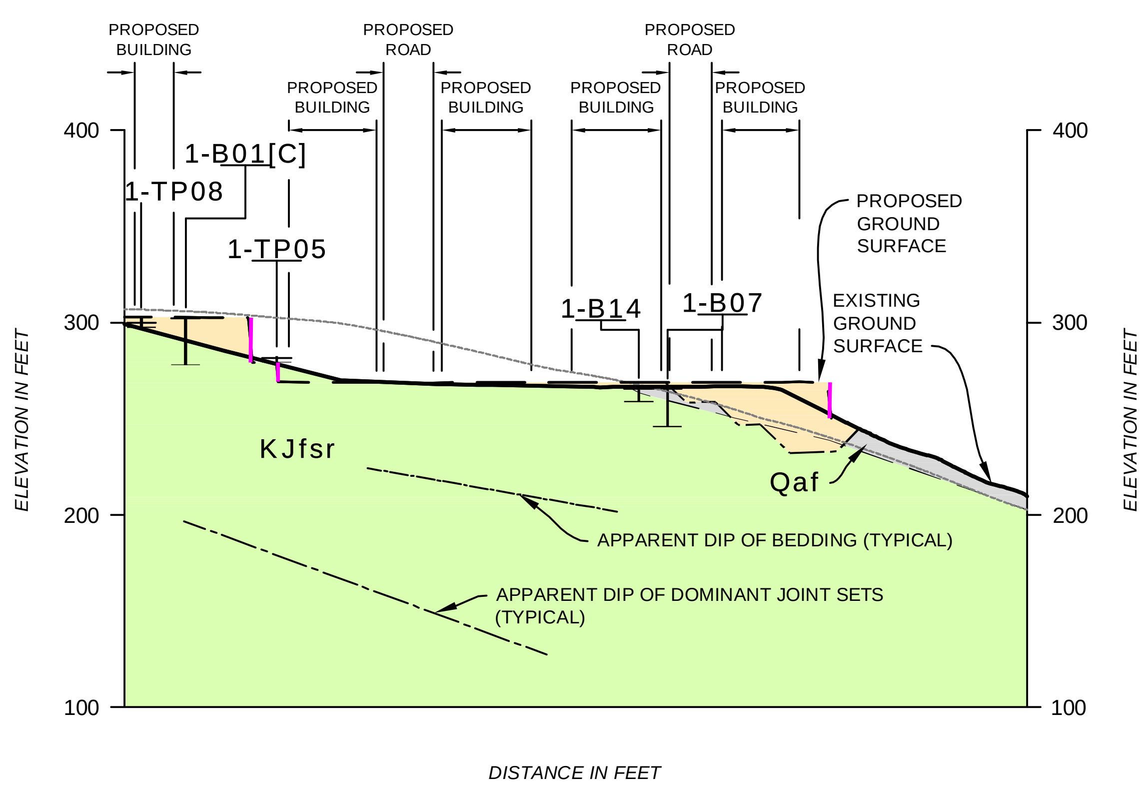

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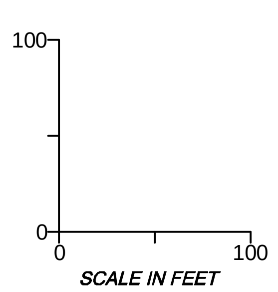

EXPLANATION

ALL LOCATIONS ARE APPROXIMATE

1-TP08	TEST PIT (ENGEO, 2019)
1-B14	SOIL BORING (ENGEO, 2019)
1-B01[C]	ROCK CORES (ENGEO, 2019)
	GEOLOGIC CONTACT
	ORIGINAL GROUND SURFACE (USGS 1947)
	PROPOSED CORRECTIVE GRADING
	PROPOSED WALL
Qaf	EXISTING ARTIFICIAL FILL
KJfsr	FRANCISCAN COMPLEX - PREDOMINANTLY GRAYWACKE WITH OCCASIONAL SHALE INTERBEDS
	PROPOSED ENGINEERED FILL

NOTE: PROPOSED ENGINEERED FILL FOR BUILDING PADS NOT DEPICTED IN CROSS SECTIONS. REFER TO REPORT FOR BUILDING PAD PREPARATION RECOMMENDATIONS.

ENGEO Expect Excellence	CROSS SECTIONS PENINSULA HEIGHTS SAN MATEO, CALIFORNIA	PROJECT NO.: 16683.000.000 SCALE: AS SHOWN DRAWN BY: SPPE CHECKED BY: YZ	FIGURE NO. 2B

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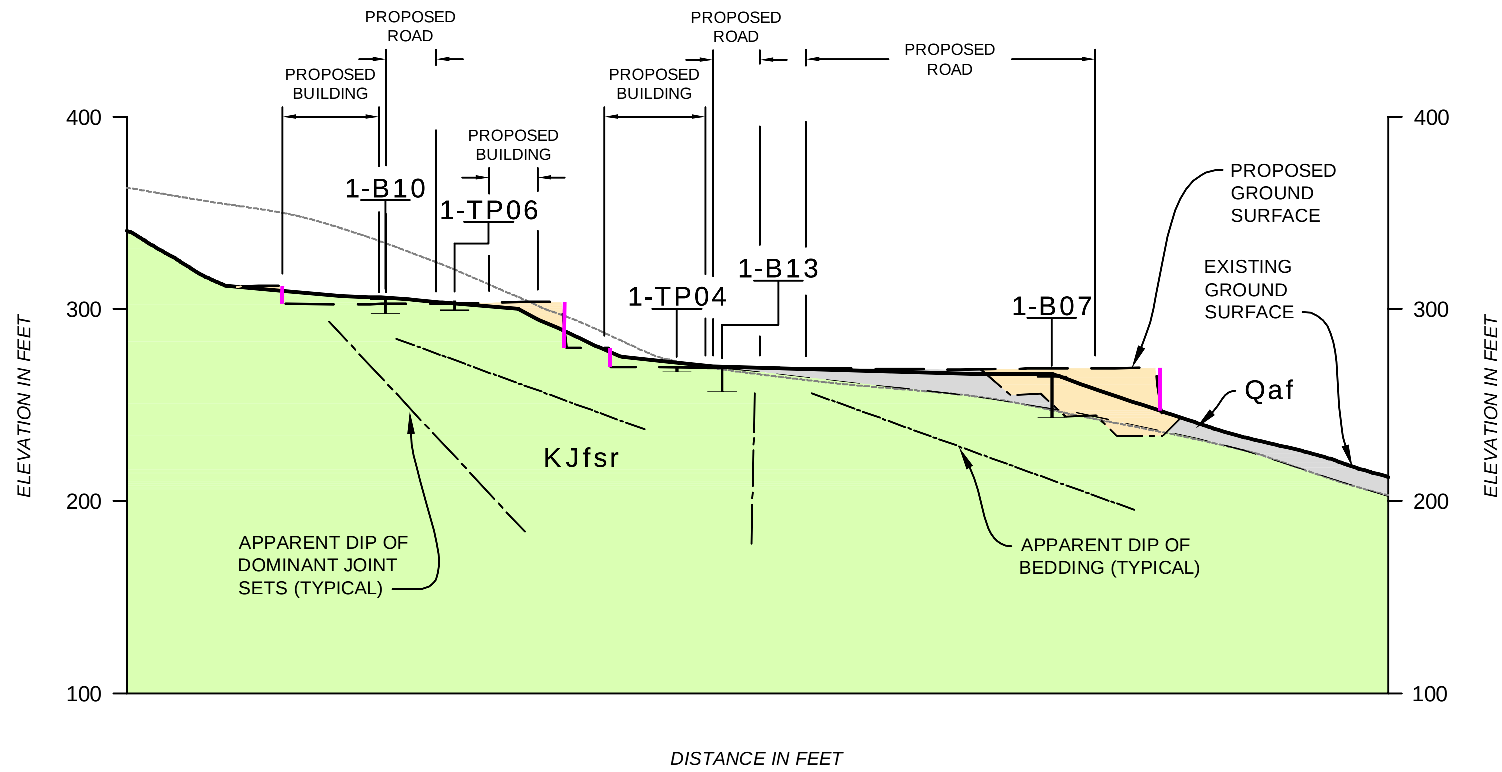

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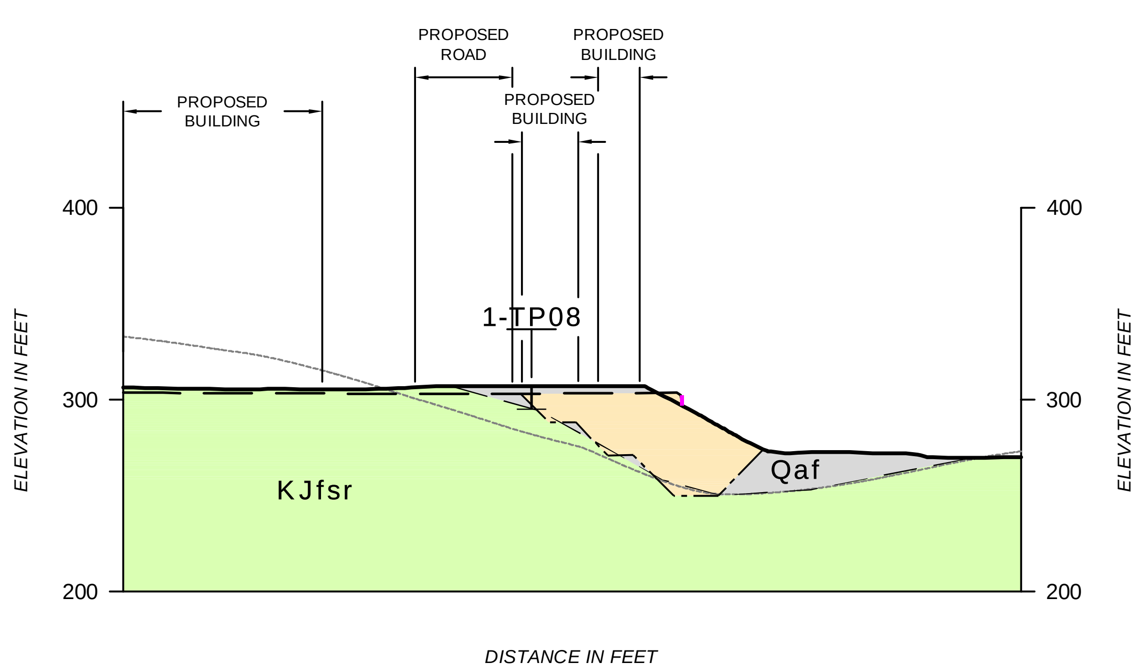

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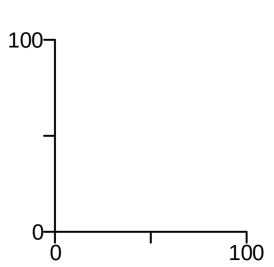

EXPLANATION

ALL LOCATIONS ARE APPROXIMATE

1-TP08	TEST PIT (ENGEO, 2019)		GEOLOGIC CONTACT	Qaf	EXISTING ARTIFICIAL FILL
			ORIGINAL GROUND SURFACE (USGS 1947)		
1-B14	SOIL BORING (ENGEO, 2019)		PROPOSED CORRECTIVE GRADING	KJfsr	FRANCISCAN COMPLEX - PREDOMINANTLY GRAYWACKE WITH OCCASIONAL SHALE INTERBEDS
1-B01[C]	ROCK CORES (ENGEO, 2019)		PROPOSED WALL		PROPOSED ENGINEERED FILL

NOTE: PROPOSED ENGINEERED FILL FOR BUILDING PADS NOT DEPICTED IN CROSS SECTIONS. REFER TO REPORT FOR BUILDING PAD PREPARATION RECOMMENDATIONS.

ENGEO Expect Excellence

CROSS SECTIONS
PENINSULA HEIGHTS
SAN MATEO, CALIFORNIA

PROJECT NO.: 16683.000.000
SCALE: AS SHOWN
DRAWN BY: SPPE CHECKED BY: YZ
FIGURE NO. 2C

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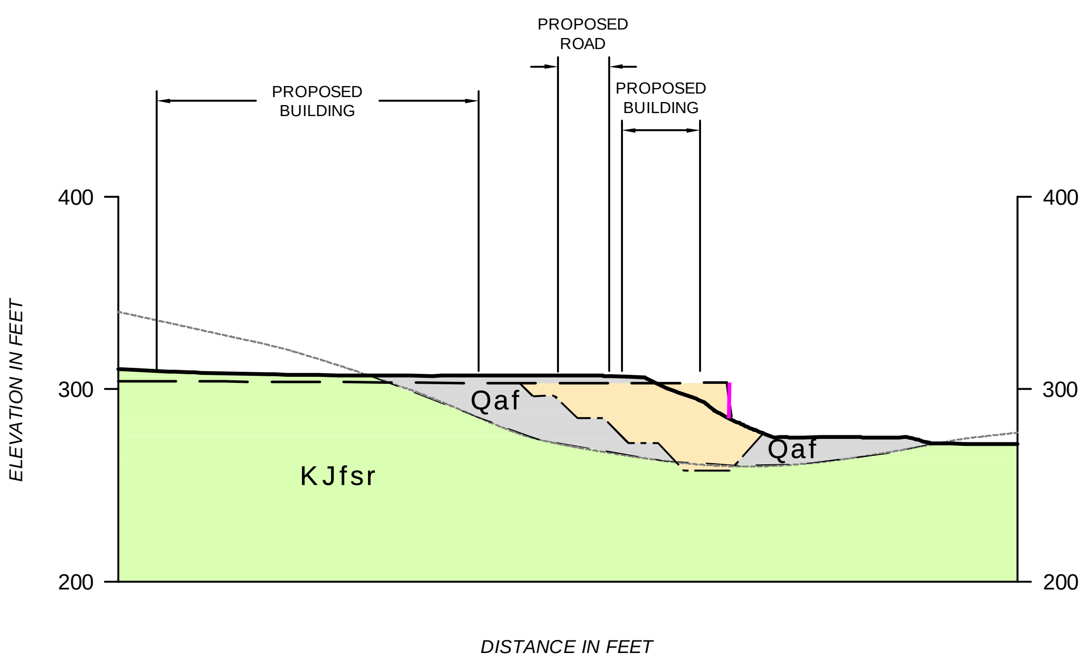

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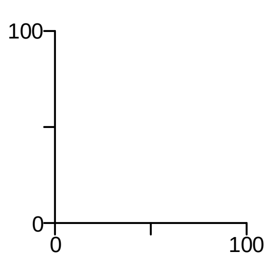


EXPLANATION

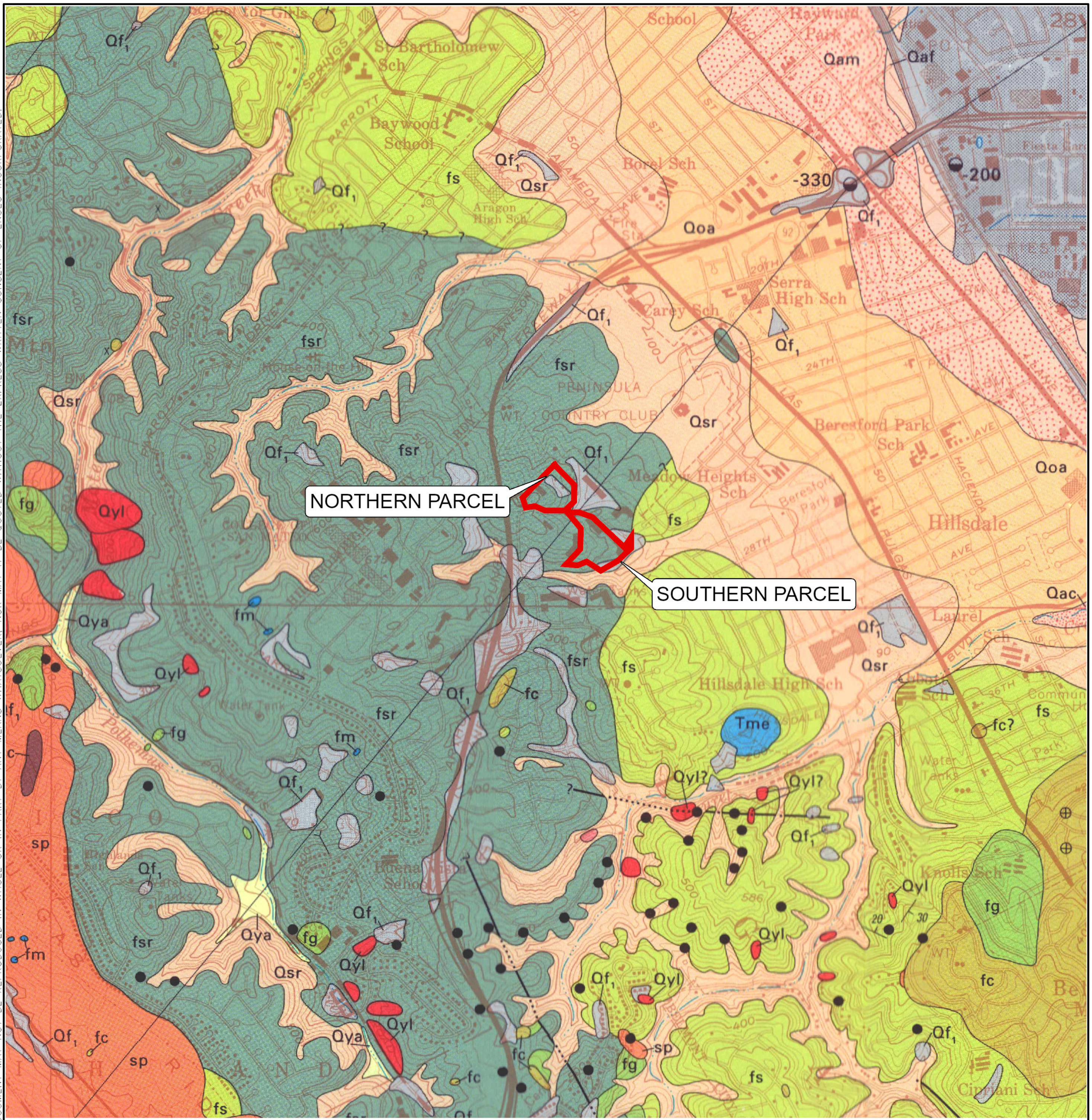
ALL LOCATIONS ARE APPROXIMATE

1-TP08	TEST PIT (ENGEO, 2019)	— — —	GEOLOGIC CONTACT	Qaf	EXISTING ARTIFICIAL FILL
1-B14	SOIL BORING (ENGEO, 2019)	·········	ORIGINAL GROUND SURFACE (USGS 1947)	KJfsr	FRANCISCAN COMPLEX - PREDOMINANTLY GRAYWACKE WITH OCCASIONAL SHALE INTERBEDS
		—·—·—	PROPOSED CORRECTIVE GRADING		PROPOSED ENGINEERED FILL
1-B01[C]	ROCK CORES (ENGEO, 2019)	|	PROPOSED WALL		

NOTE: PROPOSED ENGINEERED FILL FOR BUILDING PADS NOT DEPICTED IN CROSS SECTIONS. REFER TO REPORT FOR BUILDING PAD PREPARATION RECOMMENDATIONS.

ENGEO Expect Excellence	CROSS SECTIONS PENINSULA HEIGHTS SAN MATEO, CALIFORNIA	PROJECT NO.: 16683.000.000 SCALE: AS SHOWN DRAWN BY: SPPE CHECKED BY: YZ	FIGURE NO. 2D

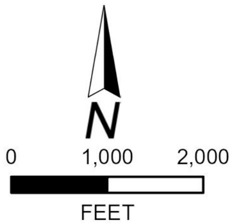
ORIGINAL FIGURE PRINTED IN COLOR



EXPLANATION

ALL LOCATIONS ARE APPROXIMATE

Qya	YOUNGER ALLUVIUM	Qaf	FINE-GRAINED ALLUVIUM (HOLOCENE)
Qf_1	ARTIFICIAL FILL (HOLOCENE) UNIT 1	fs	SANDSTONE
Qyl	YOUNGER LANDSLIDE DEPOSITS	fc	CHERT
Qsr	SLOPE WASH/RAVINE FILL	fg	GREENSTONE
Qac	COARSE-GRAINED ALLUVIUM	fsr	SHEARED ROCK
Qoa	FINE-GRAINED ALLUVIUM (HOLOCENE)	sp	SERPENTINITE



BASEMAP SOURCE: PAMPEYAN 1994

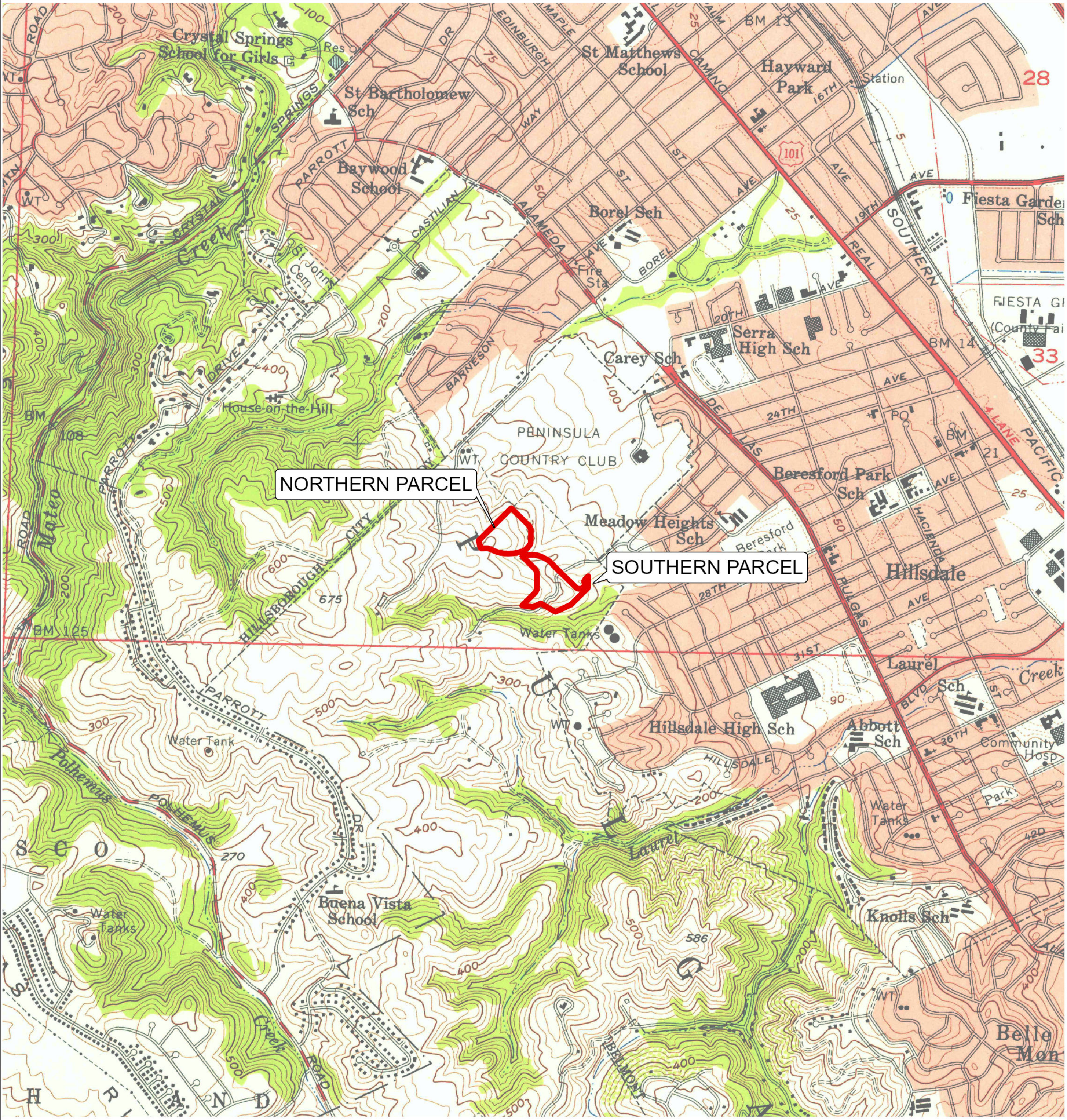
ENGEO
Expect Excellence

REGIONAL GEOLOGIC MAP
PENINSULA HEIGHTS
SAN MATEO, CALIFORNIA

PROJECT NO. : 16683.000.000
SCALE: AS SHOWN
DRAWN BY: JV
CHECKED BY: YZ
FIGURE NO. 3

PATH: G:\DRAFTING\PROJECTS_16000 TO 17999\16683\16683000000\GEOTECHNICAL\GEX\GIS\16683.APRX
USER: JVERGARA

ORIGINAL FIGURE PRINTED IN COLOR



0 1,000 2,000

FEET

N

BASEMAP SOURCE: USGS 1956

ENGEO
Expect Excellence

1956 TOPOGRAPHIC MAP

PENINSULA HEIGHTS

SAN MATEO, CALIFORNIA

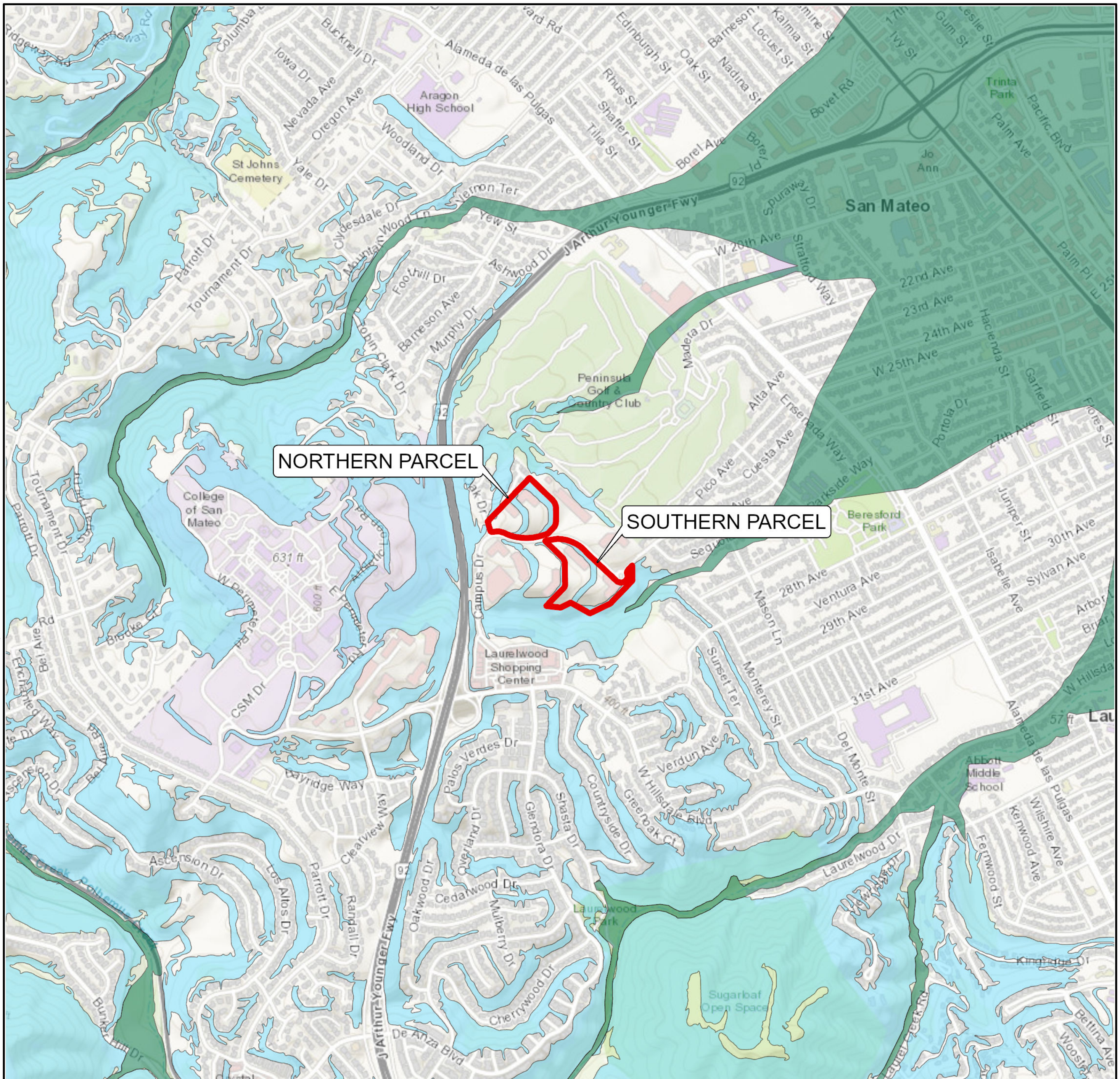
PROJECT NO. 16683.000.000	
SCALE: AS SHOWN	
DRAWN BY: JV	CHECKED BY: YZ

FIGURE NO.

4

PATH: G:\DRAFTING\PROJECTS_16000 TO 17999\16683\16683000000\GEOTECHNICAL\GEX\GIS\16683.APRX
USER: JVERGARA

ORIGINAL FIGURE PRINTED IN COLOR



EXPLANATION

ALL LOCATIONS ARE APPROXIMATE

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

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

N

0 1,000 2,000
FEET

BASEMAP SOURCE: ESRI MAPPING SERVICE
CALIFORNIA DEPARTMENT OF CONSERVATION, CALIFORNIA GEOLOGICAL SURVEY

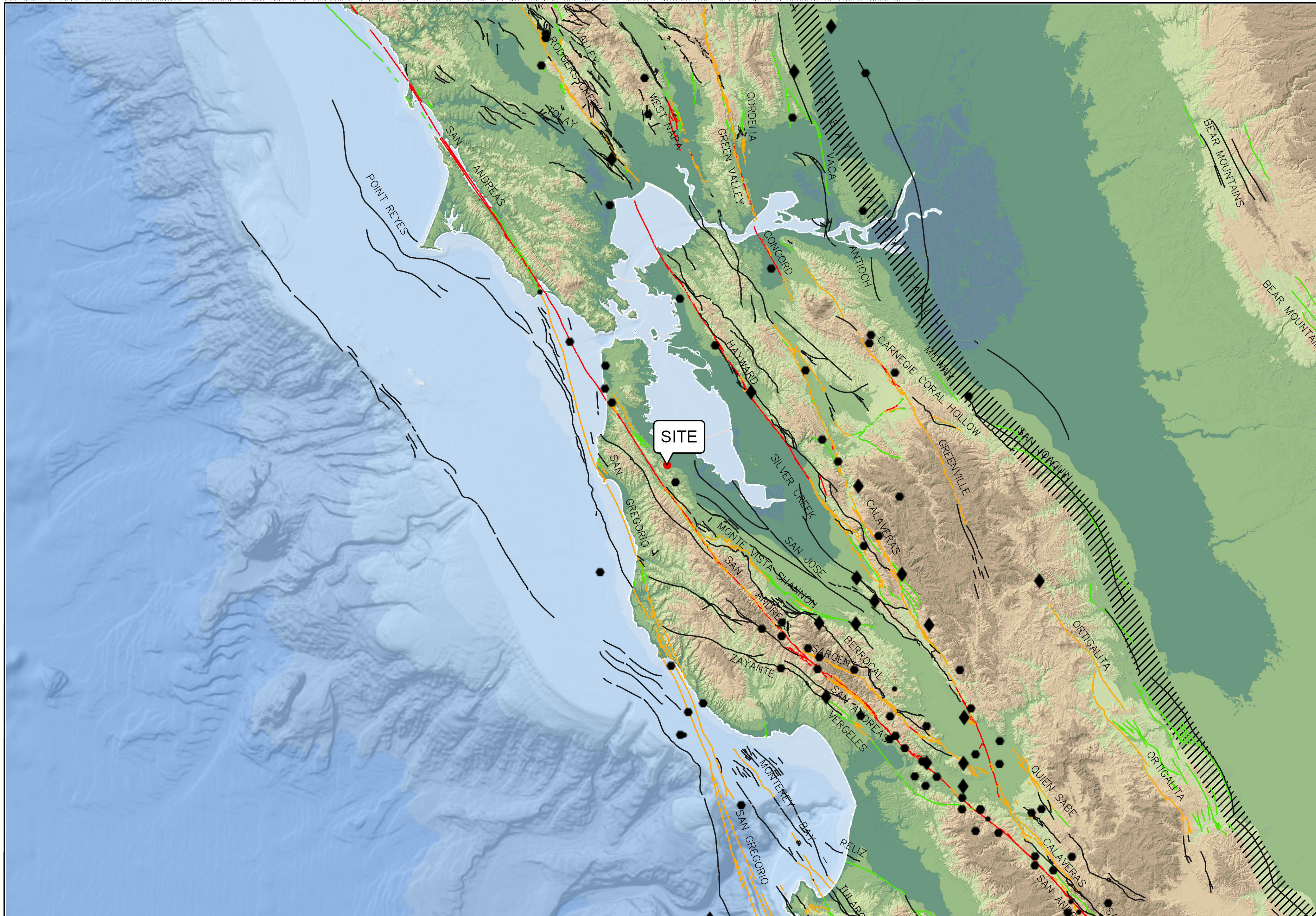
ENGEO
Expect Excellence

SEISMIC HAZARDS ZONE MAP
PENINSULA HEIGHTS
SAN MATEO, CALIFORNIA

PROJECT NO. : 16683.000.000	FIGURE NO. 5
SCALE: AS SHOWN	
DRAWN BY: JV	CHECKED BY: YZ

PATH: G:\DRAFTING\PROJECTS_16000 TO 17999\16683\16683000000\GEOTECHNICAL\GEX\GIS\16683.APRX
USER: JVERGARA

ORIGINAL FIGURE PRINTED IN COLOR



N
0 7.5 15
Miles

EXPLANATION

ALL LOCATIONS ARE APPROXIMATE

EARTHQUAKE

- MAGNITUDE 7+
- MAGNITUDE 6-7
- MAGNITUDE 5-6

USGS QUATERNARY FAULTS

- HISTORICAL
- LATEST QUATERNARY
- LATE QUATERNARY
- UNDIFFERENTIATED QUATERNARY
- HISTORIC BLIND THRUST FAULT ZONE

BASE MAP SOURCE
ESRI, GARMIN, GEBCO, NOAA NGDC, AND OTHER CONTRIBUTORS
COLOR HILLSHADE IMAGE BASED ON THE NATIONAL ELEVATION DATA SET (NED) AT 30 METER RESOLUTION
U.S.G.S. QUATERNARY FAULT DATABASE, 2018
U.S.G.S. HISTORIC EARTHQUAKE DATABASE (1800-PRESENT)

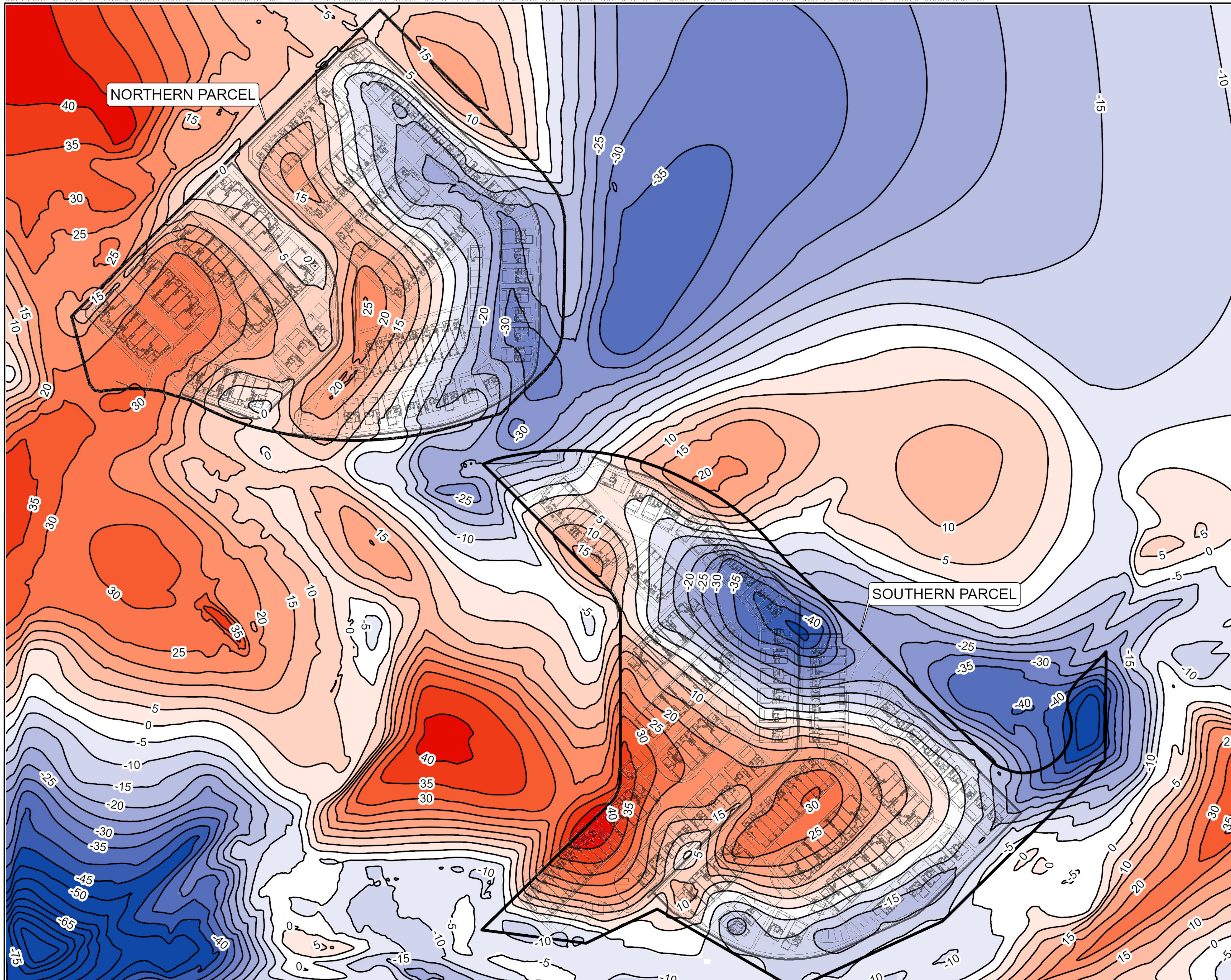
ENGEO
Expect Excellence

REGIONAL FAULTING AND SEISMICITY
PENINSULA HEIGHTS
SAN MATEO, CALIFORNIA

PROJECT NO.: 16683.000.000	FIGURE NO. 6
SCALE: AS SHOWN	
DRAWN BY: JV	CHECKED BY: YZ

PATH: G:\DRAFTING\PROJECTS_16000 TO 17999\16683\16683000000\GEOTECHNICAL\GEX\GIS\16683.APRX
USER: JVERGARA

ORIGINAL FIGURE PRINTED IN COLOR



EXPLANATION

APPROXIMATE ELEVATION DIFFERENCE IN FEET

≤ -45, ≤ -40, ≤ -35, ≤ -30, ≤ -25, ≤ -20, ≤ -15, ≤ -10, ≤ -5, ≤ 0, ≤ 5, ≤ 10, ≤ 15, ≤ 20, ≤ 25, ≤ 30, ≤ 35, ≤ 40, ≤ 45

BASEMAP SOURCE: ESRI MAPPING SERVICE 2017 AND BKF
CLASSIFICATION: USER DEFINED BREAKS

N

0 80 160
FEET

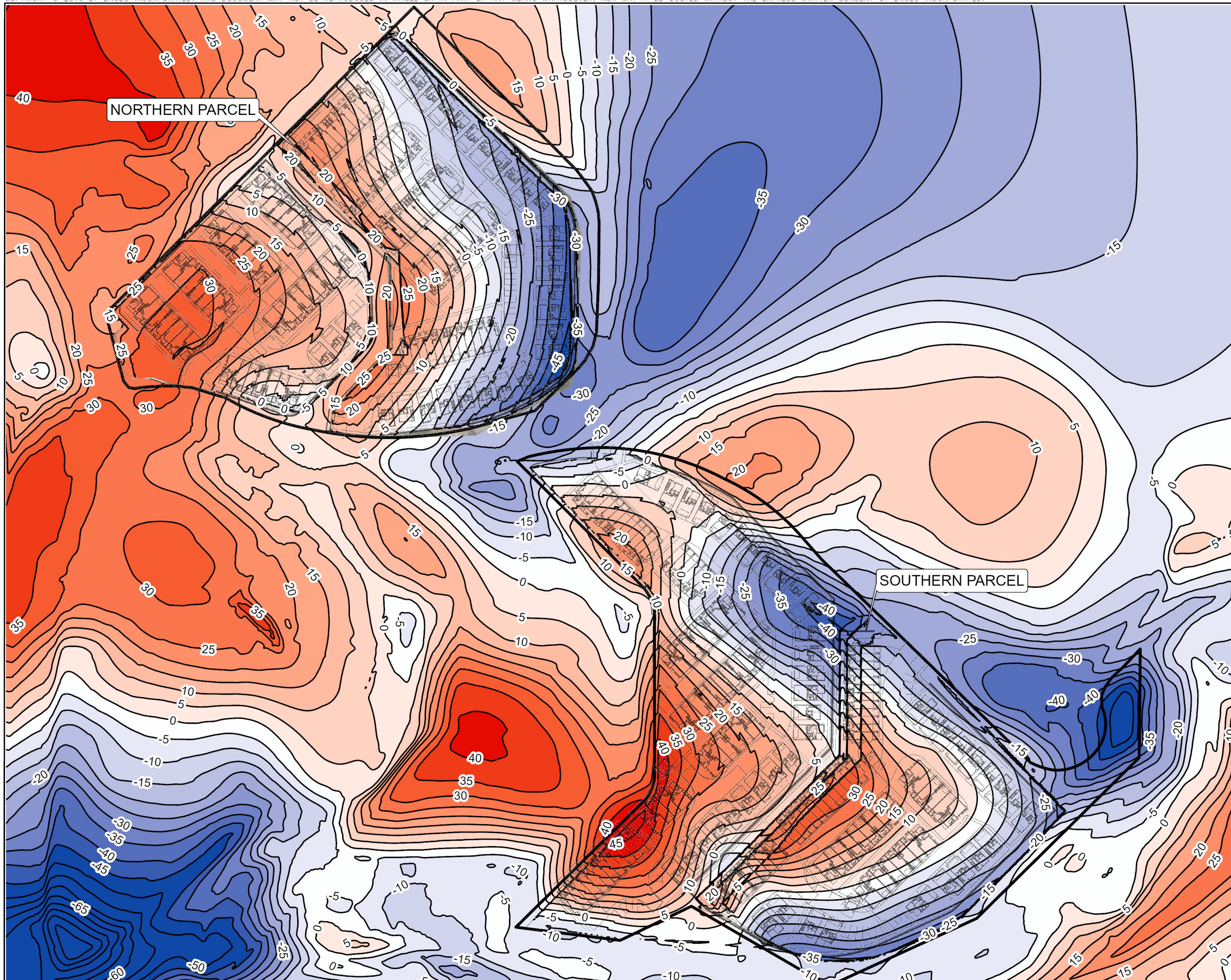
ENGEO
Expect Excellence

COMPARISON OF 1940'S AND PRESENT DAY TOPOGRAPHY
PENINSULA HEIGHTS
SAN MATEO, CALIFORNIA

PROJECT NO.: 16683.000.000	FIGURE NO. 7A
SCALE: AS SHOWN	
DRAWN BY: JV	CHECKED BY: YZ

PATH: G:\DRAFTING\PROJECTS_16000 TO 17999\16683\16683000000\GEOTECHNICAL\GEX\GIS\16683.APRX
USER: JVERGARA

ORIGINAL FIGURE PRINTED IN COLOR



EXPLANATION

APPROXIMATE ELEVATION DIFFERENCE IN FEET

≤-45	≤-10	≤25
≤-40	≤-5	≤30
≤-35	≤0	≤35
≤-30	≤5	≤40
≤-25	≤10	≤45
≤-20	≤15	
≤-15	≤20	

BASEMAP SOURCE: ESRI MAPPING SERVICE 2017 AND BKF
CLASSIFICATION: USER DEFINED BREAKS

N

0 80 160
FEET

ENGEO
Expect Excellence

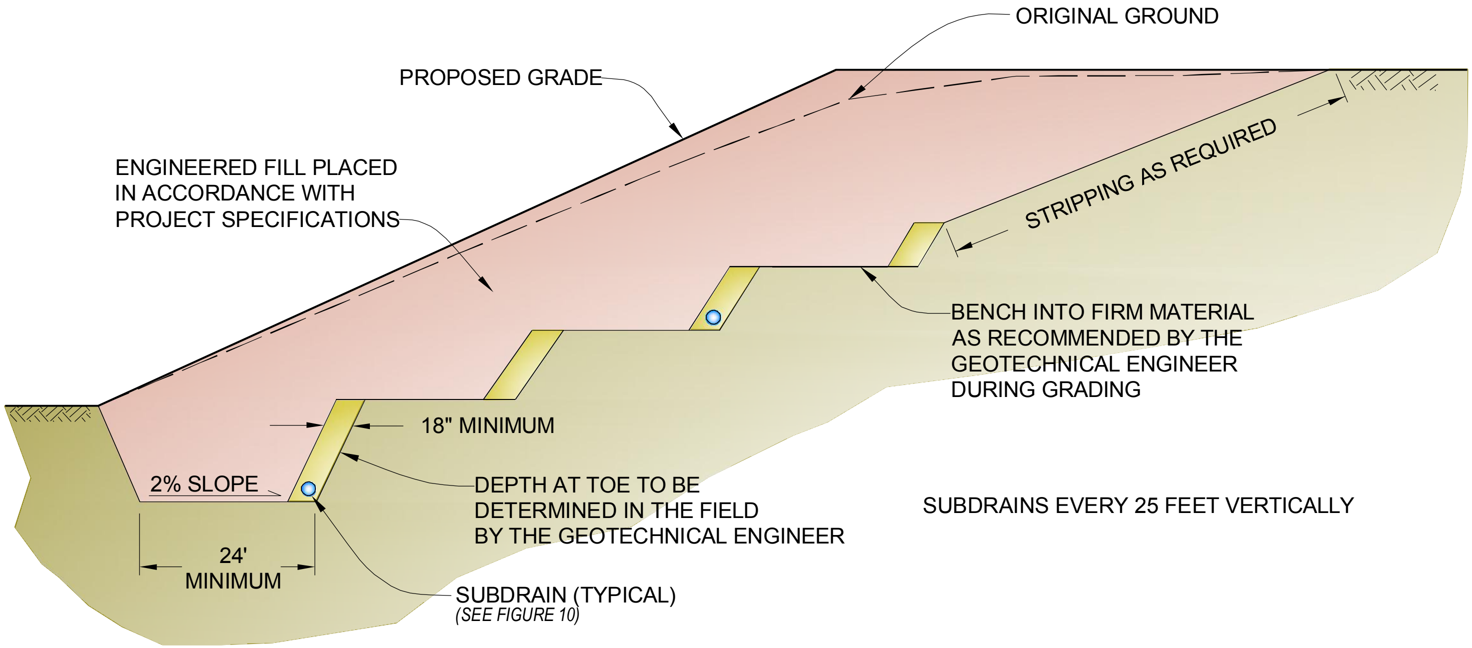
COMPARISON OF 1940'S AND PROPOSED TOPOGRAPHY
PENINSULA HEIGHTS
SAN MATEO, CALIFORNIA

PROJECT NO.: 16683.000.000
SCALE: AS SHOWN
DRAWN BY: JV
CHECKED BY: YZ
FIGURE NO.
7B

PATH: G:\DRAFTING\PROJECTS_16000 TO 17999\16683\16683000000\GEOTECHNICAL\GEX\GIS\16683.APRX
USER: JVERGARA

ORIGINAL FIGURE PRINTED IN COLOR

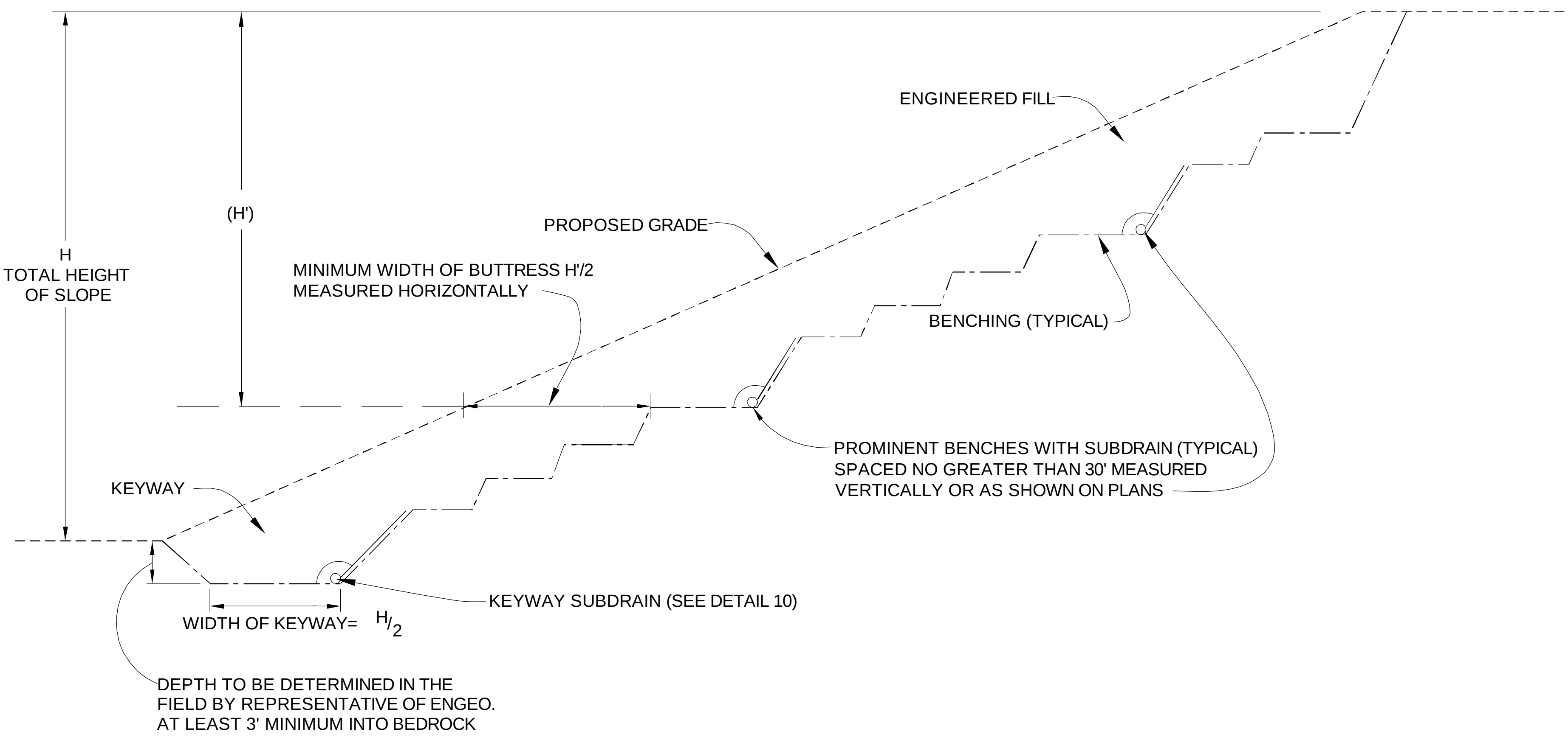
G:\Drafting\PROJECTS_16000 to 17999\16683\16683000000\Geotechnical\GEX\16683000000-GEX-1119\16683000000-GEX-6-8-Details-1119.dwg



ENGEO Expect Excellence	TYPICAL KEYWAY DETAIL PENINSULA HEIGHTS SAN MATEO, CALIFORNIA	PROJECT NO.: 16683.000.000 SCALE: NO SCALE DRAWN BY: SPPE CHECKED BY: YZ	FIGURE NO. 8

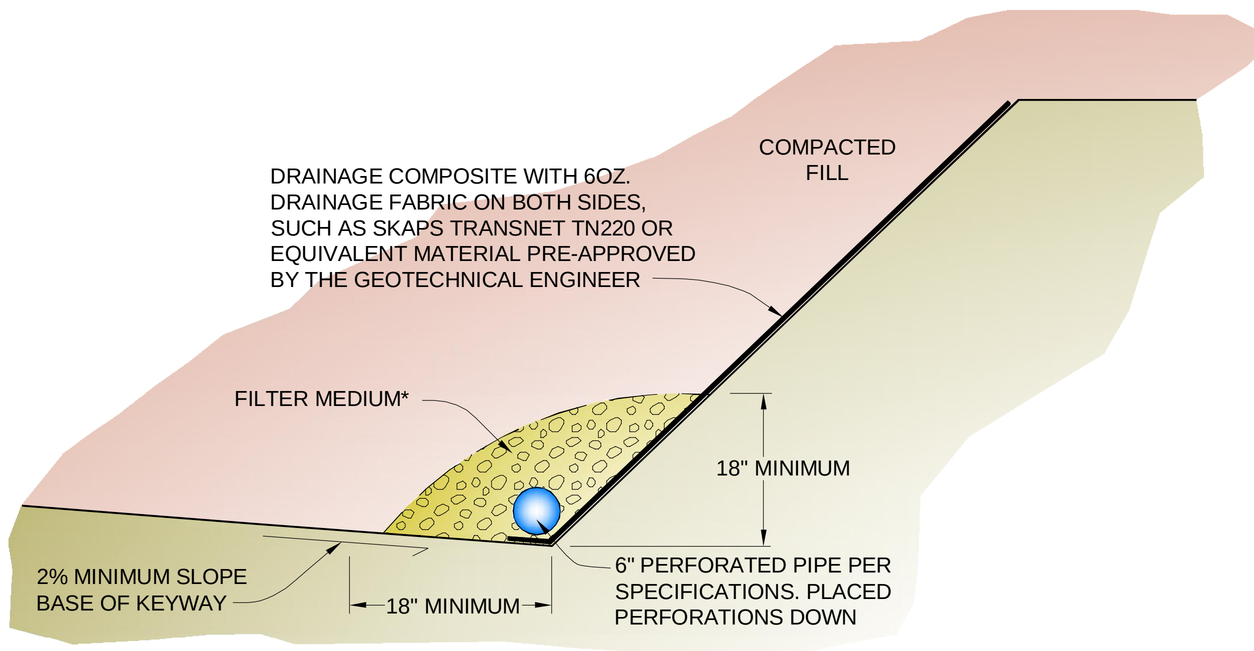
ORIGINAL FIGURE PRINTED IN COLOR

G:\Drafting\PROJECTS_16000 to 17999\16683\16683000000\Geotechnical\GEX\16683000000-GEX-1119\16683000000-GEX-6-8-Details-1119.dwg

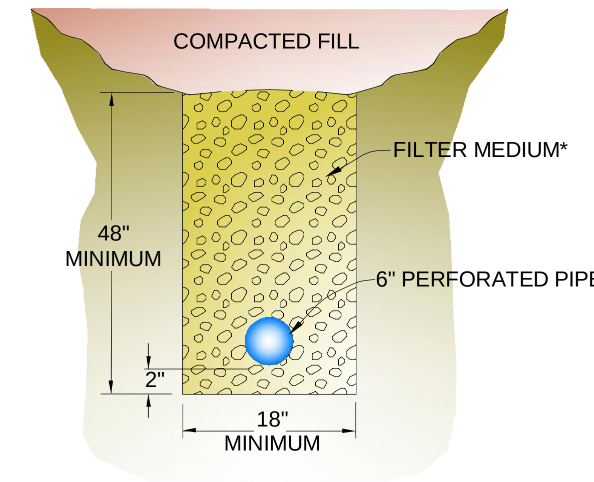


ENGEO Expect Excellence	TYPICAL CUT SLOPE DETAIL PENINSULA HEIGHTS SAN MATEO, CALIFORNIA	PROJECT NO.: 16683.000.000 SCALE: NO SCALE DRAWN BY: SPPE CHECKED BY: YZ	FIGURE NO. 9

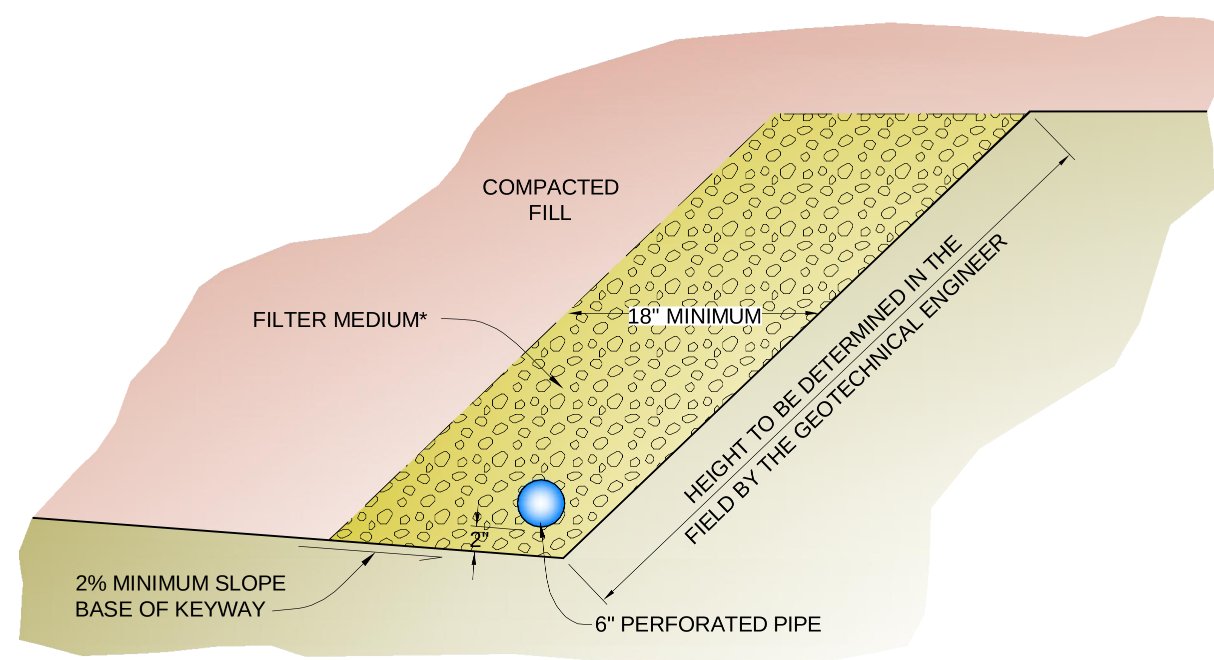
ORIGINAL FIGURE PRINTED IN COLOR



KEYWAY SUBDRAIN - OPTION 1



SWALE SUBDRAIN



KEYWAY SUBDRAIN - OPTION 2

*FILTER MEDIUM

ALTERNATIVE A

CLASS 2 PERMEABLE MATERIAL

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

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

ALTERNATIVE B

CLEAN CRUSHED ROCK OR GRAVEL WRAPPED IN FILTER FABRIC

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

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

NOTES:

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

ENGEO
Expect Excellence

TYPICAL SUBDRAIN DETAIL
PENINSULA HEIGHTS
SAN MATEO, CALIFORNIA

PROJECT NO.: 16683.000.000	FIGURE NO. 10
SCALE: NO SCALE	
DRAWN BY: SPPE	CHECKED BY: YZ

APPENDIX A

BORING AND CORE LOGS

CORELOG 1-B01

ENGEO Expect Excellence

LATITUDE: 37.5360696743692 LONGITUDE: -122.324660275624

Geotechnical Exploration
Peninsula Heights
San Mateo, CA
16683.000.000

DATE DRILLED:	10/21/2019
HOLE DEPTH:	Approx. 25 ft.
HOLE DIAMETER:	4.0 in.
SURF ELEV (San Mateo Datum):	Approx. 302 ft.
LOGGED / REVIEWED BY:	R. Ambrus / JBR
CORING CONTRACTOR:	Britton Exploration
CORING METHOD, DRILL BIT SIZE/TYPE:	Wireline Core, HQ
NO. OF CORE BOXES:	2

Run Number	Drill Rate (s/ft)	Run Length (ft) / Recovery (ft)	RQD	Relative Hardness (R0–R6) / Weathering Grade (RS, WC, WH, WM, WS, F)	Depth in Feet	Elevation in Feet	Graphic Log	DESCRIPTION	Discontinuities Remarks	Sample Type	In-Situ Testing	Notes
					0			2" AC over 6" AB	-			
								GRAYWACKE, light brown (7.5YR 6/4), [FRANCISCAN COMPLEX]				
						93						
											N=50/5"	
					5	94						
											N=50/3"	
						95						
1	7	1.5/3 (50%)	0	R4 / WM	10			Olive brown (2.5Y 4/3), strong (R4), very closely to closely fractured, massive, moderately weathered (WM), smooth joints with clay film	- Joint @ 40deg. - - Joint @ -85deg. - Joint @ -65deg.			
								Brecciated foliation, 1 inch thick	- Foliation @ 35deg.			
						96		Calcite veins up to 1/4 inch thick Very dark grayish brown (2.5Y 3/2)	- Vein @ 40deg. - Joint @ 40deg.			
									- Joint @ -65deg.			
2	9	4.5/5 (90%)	0	R4 / WM	15			With light greenish gray dark reddish brown (2.5YR 3/3), very strong (R5) Quartz nodules up to 1/4 inch thick	- Joint @ 40deg. - - Shear @ -35deg. - Joint @ -65deg.			
						97		Very close fracture spacing, Graywacke breccia within chert, some healed fractures Close to very close fracture spacing, quartz veins up to 1/8 inch thick	- Joint @ -65deg. - Joint @ 40deg. - Joint @ 40deg.			
						98			- Joint @ -65deg. - Foliation @ -35deg.			
					20							

CORELOG 1-B01

ENGEO
Expect Excellence

LATITUDE: 37.5360696743692 LONGITUDE: -122.324660275624

Geotechnical Exploration
Peninsula Heights
San Mateo, CA
16683.000.000

DATE DRILLED: 10/21/2019
HOLE DEPTH: Approx. 25 ft.
HOLE DIAMETER: 4.0 in.
SURF ELEV (San Mateo Datum): Approx. 302 ft.

LOGGED / REVIEWED BY: R. Ambrus / JBR
CORING CONTRACTOR: Britton Exploration
CORING METHOD, DRILL BIT SIZE/TYPE: Wireline Core, HQ
NO. OF CORE BOXES: 2

Run Number	Drill Rate (s/ft)	Run Length (ft) / Recovery (ft)	RQD	Relative Hardness (R0 R1 R2 R3 R4 R5 R6) / Weathering Grade (RS WC WH WM WS F)	Depth in Feet	Elevation in Feet	Graphic Log	DESCRIPTION	Discontinuities Remarks	Sample Type	In-Situ Testing	Notes
3	8.5	3.5/5 (70%)	0					Very close to crushed fracture spacing, many healed fractures, 1/64 inch marbling and 1/8 inch veinlets	- Joint @ 35deg. - - Joint @ -65deg. - Joint @ -20deg. - Joint @ -65deg. - Joint @ 35deg. - Joint @ 35deg.			
4	9	.5/2 (25%)	0		25				- - Joint @ 35deg. - Joint @ -65deg.			
								Set casing to 10 feet, HSA drilling to 10 feet, wireline coring to end of boring. End of boring at 25 feet. Groundwater not measured due to drilling method.				

R0 R1 R2 R3 R4 R5 R6
RS WC WH WM WS F

LOG-CORELOG_MASTER 1-B1 AND 1-B2 PH.GPJ ENGEO INC.GDT 11/25/19

CORELOG 1-B02

ENGEO Expect Excellence

LATITUDE: 37.5378897825537 LONGITUDE: -122.327007679493

Geotechnical Exploration
Peninsula Heights
San Mateo, CA
16683.000.000

DATE DRILLED:	10/22/2019
HOLE DEPTH:	Approx. 26½ ft.
HOLE DIAMETER:	4.0 in.
SURF ELEV (San Mateo Datum):	Approx. 369 ft.
LOGGED / REVIEWED BY:	R. Ambrus / JBR
CORING CONTRACTOR:	Britton Exploration
CORING METHOD, DRILL BIT SIZE/TYPE:	Wireline Core, HQ
NO. OF CORE BOXES:	2

Relative Hardness: R0 R1 R2 R3 R4 R5 R6
Weathering Grade: RS WC WH WM WS F

Run Number	Drill Rate (s/ft)	Run Length (ft) / Recovery (ft)	RQD	Depth in Feet	Elevation in Feet	DESCRIPTION	Discontinuities Remarks	In-Situ Testing	Notes
						4" AC over 6" AB			
					113	CLAYEY SAND WITH GRAVEL (SC), pale yellow, very dense, dry, fine gravel, [FILL]		N=44	FC=19%
				5	114				
					115	GRAYWACKE, grayish brown (10YR 5/2), [FRANCISCAN COMPLEX]		N=50/6"	
1	4	1.5/3 (50%)	0	10	116	Moderately strong (R3), crushed, massive, highly weathered (WH), core stones up to 1/2"	Joint @ -70deg. Joint @ 30deg.		
2	9	4.25/5 (85%)	0	15	117	Olive gray (5Y 4/2), strong (R4), very closely fractured, thin bedding, moderately weathered (WH)	Joint @ -70deg. Joint @ 35deg. Joint @ 30deg. Joint @ -70deg. Shear @ -10deg. Joint @ 10deg. Joint @ 30deg.		
				20	118	Dark olive gray / olive gray (5Y 3/2), strong (R4), very closely fractured, thick bedding, highly weathered (WH) to moderately weathered (WH)	Joint @ 30deg. Joint @ 10deg.		

CORELOG 1-B02

Expect Excellence

LATITUDE: 37.5378897825537 LONGITUDE: -122.327007679493

Geotechnical Exploration
Peninsula Heights
San Mateo, CA
16683.000.000

DATE DRILLED: 10/22/2019
HOLE DEPTH: Approx. 26½ ft.
HOLE DIAMETER: 4.0 in.
SURF ELEV (San Mateo Datum): Approx. 369 ft.

LOGGED / REVIEWED BY: R. Ambrus / JBR
CORING CONTRACTOR: Britton Exploration
CORING METHOD, DRILL BIT SIZE/TYPE: Wireline Core, HQ
NO. OF CORE BOXES: 2

Run Number	Drill Rate (s/ft)	Run Length (ft) / Recovery (ft)	RQD	Relative Hardness (R0 R1 R2 R3 R4 R5 R6) / Weathering Grade (RS WC WH WM WS F)	Depth in Feet	Elevation in Feet	Graphic Log	DESCRIPTION	Discontinuities Remarks	Sample Type	In-Situ Testing	Notes
3	5	5/5 (100%)	0					Moderately weathered (WH)	- Joint @ 35deg. - Joint @ -70deg. - - Joint @ -65deg. - Joint @ -20deg. - Joint @ -65deg. - Foliation @ -10deg.			
4	6	3.15/3.5 (90%)	0		25				- Joint @ -70deg. - Joint @ 30deg. - - Joint @ -70deg. - Joint @ -10deg. - Joint @ 30deg.			

Set casing to 10 feet, HSA drilling to 10 feet, wireline coring to end of boring.
End of boring at 26.5 feet. Groundwater not measured due to drilling method.

R0 R1 R2 R3 R4 R5 R6
RS WC WH WM WS F

LOG OF BORING 1-B03

ENGEO
Expect Excellence

LATITUDE: 37.5384602717152 LONGITUDE: -122.326236781716

Geotechnical Exploration
Peninsula Heights
San Mateo, CA
16683.000.000

DATE DRILLED: 10/22/2019
HOLE DEPTH: 25.5 ft.
HOLE DIAMETER: 6.0 in.
SURF ELEV (SAN MATEO DATUM): Approx. 335 ft.

LOGGED / REVIEWED BY: R. Ambrus / TPB
DRILLING CONTRACTOR: Britton Exploration
DRILLING METHOD: Hollow Stem Auger
HAMMER TYPE: 140 lb. Auto Trip

Depth in Feet	Elevation in Feet	DESCRIPTION	Blow Count/Foot	Atterberg Limits: Liquid Limit	Atterberg Limits: Plastic Limit	Atterberg Limits: Plasticity Index	Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Shear Strength (psf) *field approximation	Unconfined Strength (tsf) *field approximation	Strength Test Type
		4" AC over 6" AB										
		CLAYEY SAND (SC), brown, medium dense to dense, dry, fine to coarse gravel, trace white angular cobbles up to 1½" diameter [FILL]										
			36	31	17	14						
5	330											
		Rootlets	30									
		POORLY GRADED SAND WITH CLAY AND GRAVEL (SP), brown, medium dense to dense, moist, fine to coarse gravel [FILL]										
		CLAYEY SAND WITH GRAVEL (SC), brown, medium dense, moist, fine to coarse angular gravel [FILL]										
10	325		19									
15	320	Trace white angular cobbles up to 2" diameter.	29	29	16	13	24					
20	315	Trace white angular cobbles up to 4" diameter	16									
		POORLY GRADED SAND WITH CLAY (SP), yellow to pale yellow, very dense, moist, trace fine gravel [FILL]										
25	310		50/2"									
		GRAYWACKE pale yellow [BEDROCK]										
		Boring terminated at a depth of 25.5 feet below ground surface. Groundwater not encountered during drilling.										

LOG OF BORING 1-B04

ENGEO Expect Excellence

LATITUDE: 37.538155731406 LONGITUDE: -122.326407409326

Geotectechnical Exploration
Peninsula Heights
San Mateo, CA
16683.000.000

DATE DRILLED: 10/22/2019
HOLE DEPTH: 6.5 ft.
HOLE DIAMETER: 6.0 in.
SURF ELEV (SAN MATEO DATUM): Approx. 334½ ft.

LOGGED / REVIEWED BY: R. Ambrus / TPB
DRILLING CONTRACTOR: Britton Exploration
DRILLING METHOD: Hollow Stem Auger
HAMMER TYPE: 140 lb. Auto Trip

Depth in Feet	Elevation in Feet	Sample Type	DESCRIPTION	Log Symbol	Water Level	Blow Count/Foot	Atterberg Limits: Liquid Limit	Plastic Limit	Plasticity Index	Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Shear Strength (psf) *field approximation	Unconfined Strength (tsf) *field approximation	Strength Test Type
			4" AC over 6" AB												
			LEAN CLAY WITH SAND (CL), brown, stiff, moist, fine gravel, slow dilatancy [FILL]			14	31	17	14					1.5*	PP
5	330		CLAYEY SAND WITH GRAVEL (SC), brown, medium dense to dense, moist, fine gravel, trace white angular cobbles up to 2" diameter [FILL]												
			GRAYWACKE pale yellow to yellow [BEDROCK]			50/6"									
			Boring terminated at a depth of 6.5 feet below ground surface. Groundwater not encountered during drilling.												

LOG OF BORING 1-B05

LATITUDE: 37.5362878135828 LONGITUDE: -122.322954897605

Geotechnical Exploration
Peninsula Heights
San Mateo, CA
16683.000.000

DATE DRILLED: 10/23/2019
HOLE DEPTH: 50.5 ft.
HOLE DIAMETER: 6.0 in.
SURF ELEV (SAN MATEO DATUM): Approx. 258 ft.

LOGGED / REVIEWED BY: R. Ambrus / TPB
DRILLING CONTRACTOR: Britton Exploration
DRILLING METHOD: Hollow Stem Auger
HAMMER TYPE: 140 lb. Auto Trip

Depth in Feet	Elevation in Feet	DESCRIPTION	Blow Count/Foot	Atterberg Limits: Liquid Limit	Atterberg Limits: Plastic Limit	Atterberg Limits: Plasticity Index	Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Shear Strength (psf) *field approximation	Unconfined Strength (tsf) *field approximation	Strength Test Type
		CLAYEY SAND WITH GRAVEL (SC-CL), brown and gray, medium dense, moist, fine to coarse angular gravel [FILL]										
	255		16								4.5*	PP
5			12	27	17	10						
	250											
10		Brown mottled with gray	23								4.5*	PP
	245											
15		Grayish brown, rootlets	17									
	240	CLAYEY SAND WITH GRAVEL (SC), grayish brown, medium dense, moist, fine to coarse white angular gravel [FILL]										
20			21				27					
	235	SANDY CLAY (CL), dark bluish gray, stiff to medium stiff, moist, low plasticity [FILL]										
25			31								1.25*	PP
											>4.5*	PP

LOG OF BORING 1-B05

ENGEO Expect Excellence

LATITUDE: 37.5362878135828 LONGITUDE: -122.322954897605

Geotechnical Exploration
Peninsula Heights
San Mateo, CA
16683.000.000

DATE DRILLED: 10/23/2019
HOLE DEPTH: 50.5 ft.
HOLE DIAMETER: 6.0 in.
SURF ELEV (SAN MATEO DATUM): Approx. 258 ft.

LOGGED / REVIEWED BY: R. Ambrus / TPB
DRILLING CONTRACTOR: Britton Exploration
DRILLING METHOD: Hollow Stem Auger
HAMMER TYPE: 140 lb. Auto Trip

Depth in Feet	Elevation in Feet	Sample Type	DESCRIPTION	Log Symbol	Water Level	Blow Count/Foot	Atterberg Limits: Liquid Limit	Atterberg Limits: Plastic Limit	Atterberg Limits: Plasticity Index	Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Shear Strength (psf) *field approximation	Unconfined Strength (tsf) *field approximation	Strength Test Type
			CLAYEY SAND (SC-CL), brown and gray, medium dense, moist, fine to coarse white angular gravel [FILL]												
	230														
30						26									
	225														
35			Trace white angular cobbles up to 2" diameter			25									
	220		GRAVELLY LEAN CLAY WITH SAND (CL), brown to dark brown, stiff, moist, medium plasticity, fine to coarse gravel, trace white angular cobbles up to 2" diameter [FILL]												
40						24								2.0*	PP
	215		CLAYEY SAND WITH GRAVEL (SC-CL), brown, medium dense, wet, low plasticity, fine to coarse white angular gravel [FILL]			17									
45															
			Trace white angular cobbles up to 2" diameter			22									
	210														
50			Dark gray Brown			24									
			Boring terminated at a depth of 50.5 feet below ground surface. Groundwater not encountered during drilling.												

LOG - GEOTECHNICAL_SU+QU W/ ELEV ALL BORINGS.GPJ ENGEO INC.GDT 11/25/19

ENGEO

Expect Excellence

LOG OF BORING 1-B06

LATITUDE: 37.5379688347736 LONGITUDE: -122.326041624169

Geotechnical Exploration
Peninsula Heights
San Mateo, CA
16683.000.000

DATE DRILLED: 10/22/2019
HOLE DEPTH: 25.5 ft.
HOLE DIAMETER: 6.0 in.
SURF ELEV (SAN MATEO DATUM): Approx. 332 ft.

LOGGED / REVIEWED BY: R. Ambrus / TPB
DRILLING CONTRACTOR: Britton Exploration
DRILLING METHOD: Hollow Stem Auger
HAMMER TYPE: 140 lb. Auto Trip

Depth in Feet	Elevation in Feet	DESCRIPTION	Blow Count/Foot	Atterberg Limits: Liquid Limit	Atterberg Limits: Plastic Limit	Atterberg Limits: Plasticity Index	Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Shear Strength (psf) *field approximation	Unconfined Strength (tsf) *field approximation	Strength Test Type
0	332	3" AC over 6" AB										
	330	LEAN CLAY WITH SAND (CL), brown mottled with orange, stiff to very stiff, moist, slow dilatancy [FILL]	14	46	20	26				2100*	2.5*	PP+TV
5		LEAN SANDY CLAY (CL), brown mottled with orange, stiff to very stiff, moist, trace cobbles up to 2" in diameter	16									
	325	CLAYEY SAND WITH GRAVEL (SC), brown and dark brown, medium dense, moist, fine to coarse gravel, trace cobbles up to 2" in diameter [FILL]	21									
10	320	POORLY GRADED SAND WITH CLAY AND GRAVEL (SP), brown, medium dense, moist, fine to coarse gravel [FILL]	16									
15	315	SANDY CLAY WITH GRAVEL (CL), brown, stiff to very stiff, moist, fine to coarse white subangular gravel [FILL]	23	31	16	15						
20	310	GRAVELLY SAND WITH CLAY (SP), brown, very dense, moist, fine to coarse gravel, trace white angular cobbles up to 2" in diameter [FILL]	66									
25		Dense to very dense, 1" diameter inclusion of dark blue lean clay	47									
		Boring terminated at a depth of 25.5 feet below ground surface. Groundwater not encountered during drilling.										

LOG OF BORING 1-B07

ENGEO
Expect Excellence

LATITUDE: 37.5355321542991 LONGITUDE: -122.324104526057

Geotechnical Exploration
Peninsula Heights
San Mateo, CA
16683.000.000

DATE DRILLED: 10/23/2019
HOLE DEPTH: 20.5 ft.
HOLE DIAMETER: 6.0 in.
SURF ELEV (SAN MATEO DATUM): Approx. 265½ ft.

LOGGED / REVIEWED BY: R. Ambrus / TPB
DRILLING CONTRACTOR: Britton Exploration
DRILLING METHOD: Hollow Stem Auger
HAMMER TYPE: 140 lb. Auto Trip

Depth in Feet	Elevation in Feet	Sample Type	DESCRIPTION	Log Symbol	Water Level	Blow Count/Foot	Atterberg Limits: Liquid Limit	Atterberg Limits: Plastic Limit	Atterberg Limits: Plasticity Index	Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Shear Strength (psf) *field approximation	Unconfined Strength (tsf) *field approximation	Strength Test Type
	265		3" AC over 6" AB												
5	260		SANDY LEAN CLAY WITH GRAVEL (CL), pale yellow to light brown, hard, dry, low plasticity, rootlets, fine to coarse white angular gravel [FILL]			38									
			CLAYEY GRAVEL WITH SAND (GC), yellowish brown, medium dense, dry, fine to coarse gravel, trace white angular cobbles up to 2" diameter. [FILL]			64				19					
10	255		CLAYEY SAND WITH GRAVEL (SC), brown, very dense, moist, fine to coarse gravel, trace white angular cobbles up to 2" diameter. [FILL]			29	29	19	10	14					
			Yellowish brown, dry												
15	250		GRAYWACKE pale yellow [BEDROCK]			50/5"									
20	245					50/2"									
			Boring terminated at a depth of 20.5 feet below ground surface. Groundwater not encountered during drilling.												

LOG - GEOTECHNICAL_SU+QU W/ ELEV ALL BORINGS.GPJ ENGEO INC.GDT 11/25/19

LOG OF BORING 1-B08

ENGEO
Expect Excellence

LATITUDE: 37.536519349589 LONGITUDE: -122.324021440916

Geotechnical Exploration
Peninsula Heights
San Mateo, CA
16683.000.000

DATE DRILLED: 10/23/2019
HOLE DEPTH: 25.5 ft.
HOLE DIAMETER: 6.0 in.
SURF ELEV (SAN MATEO DATUM): Approx. 267 ft.

LOGGED / REVIEWED BY: R. Ambrus / TPB
DRILLING CONTRACTOR: Britton Exploration
DRILLING METHOD: Hollow Stem Auger
HAMMER TYPE: 140 lb. Auto Trip

Depth in Feet	Elevation in Feet	DESCRIPTION	Blow Count/Foot	Liquid Limit (Atterberg Limits)	Plastic Limit (Atterberg Limits)	Plasticity Index (Atterberg Limits)	Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Shear Strength (psf) *field approximation	Unconfined Strength (tsf) *field approximation	Strength Test Type
		4" AC over 6" AB										
	265	CLAYEY SAND WITH GRAVEL (SC), brown, dense to very dense, moist, fine to coarse white angular gravel [FILL]	43									
5		Dense, fine to coarse white angular gravel	39				21					
	260	SANDY LEAN CLAY WITH GRAVEL (CL), dark grayish brown, hard, moist, low plasticity, slow dilatancy, fine to coarse gravel [FILL]										
		CLAYEY SAND WITH GRAVEL (SC), brown, dense, moist, fine to coarse gravel [FILL]	41								>4.5*	PP
10	255											
15		Medium dense to dense, white angular gravel	30									
	250	Dark brown to brown, dense	49									
20		Grayish brown	32				35					
	245	SANDY LEAN CLAY (CL), dark brown with greenish yellow, medium stiff, moist, medium plasticity, slow dilatancy [NATIVE]	21								1.5*	PP
25		Boring terminated at a depth of 25.5 feet below ground surface. Groundwater not encountered during drilling.										

Log Symbol | Water Level | Sample Type

LOG OF BORING 1-B09

ENGEO
Expect Excellence

LATITUDE: 37.5368215159817 LONGITUDE: -122.325189203483

Geotechnical Exploration
Peninsula Heights
San Mateo, CA
16683.000.000

DATE DRILLED: 10/21/2019
HOLE DEPTH: 15 ft.
HOLE DIAMETER: 6.0 in.
SURF ELEV (SAN MATEO DATUM): Approx. 308 ft.

LOGGED / REVIEWED BY: R. Ambrus / TPB
DRILLING CONTRACTOR: Britton Exploration
DRILLING METHOD: Hollow Stem Auger
HAMMER TYPE: 140 lb. Auto Trip

Depth in Feet	Elevation in Feet	Sample Type	DESCRIPTION	Log Symbol	Water Level	Blow Count/Foot	Atterberg Limits: Liquid Limit	Atterberg Limits: Plastic Limit	Atterberg Limits: Plasticity Index	Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Shear Strength (psf) *field approximation	Unconfined Strength (tsf) *field approximation	Strength Test Type
			3" AC over 8" AB												
			CLAYEY SAND (SC), strong brown, very dense, moist, low plasticity, trace fine to coarse gravel [FILL]												
	305														
						50/3"	30	17	13	37					
5			GRAYWACKE pale yellow to light yellowish brown [BEDROCK]			50/6"									
	300														
						71									
10															
			Iron oxide staining			50/6"									
	295														
						50/6"									
15			Boring terminated at a depth of 15.0 feet below ground surface. Groundwater not encountered during drilling.												

LOG OF BORING 1-B10

ENGEO Expect Excellence

LATITUDE: 37.5357404802275 LONGITUDE: -122.325279084783

Geotechnical Exploration
Peninsula Heights
San Mateo, CA
16683.000.000

DATE DRILLED: 10/21/2019
HOLE DEPTH: 8 ft.
HOLE DIAMETER: 6.0 in.
SURF ELEV (SAN MATEO DATUM): Approx. 305 ft.

LOGGED / REVIEWED BY: R. Ambrus / TPB
DRILLING CONTRACTOR: Britton Exploration
DRILLING METHOD: Hollow Stem Auger
HAMMER TYPE: 140 lb. Auto Trip

Depth in Feet	Elevation in Feet	Sample Type	DESCRIPTION	Log Symbol	Water Level	Blow Count/Foot	Atterberg Limits: Liquid Limit	Plastic Limit	Plasticity Index	Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Shear Strength (psf) *field approximation	Unconfined Strength (tsf) *field approximation	Strength Test Type
			2" AC over 6" AB												
			GRAYWACKE pale yellow to yellow [BEDROCK]												
5	300					50/3"									
			Boring terminated at a depth of 8.0 feet below ground surface. Groundwater not encountered during drilling.												

LOG OF BORING 1-B11

ENGEO
Expect Excellence

LATITUDE: 37.5376691313522 LONGITUDE: -122.327429318234

Geotechnical Exploration
Peninsula Heights
San Mateo, CA
16683.000.000

DATE DRILLED: 10/21/2019
HOLE DEPTH: 3.5 ft.
HOLE DIAMETER: 6.0 in.
SURF ELEV (SAN MATEO DATUM): Approx. 368 ft.

LOGGED / REVIEWED BY: R. Ambrus / TPB
DRILLING CONTRACTOR: Britton Exploration
DRILLING METHOD: Hollow Stem Auger
HAMMER TYPE: 140 lb. Auto Trip

Depth in Feet	Elevation in Feet	Sample Type	DESCRIPTION	Log Symbol	Water Level	Blow Count/Foot	Atterberg Limits: Liquid Limit	Atterberg Limits: Plastic Limit	Atterberg Limits: Plasticity Index	Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Shear Strength (psf) *field approximation	Unconfined Strength (tsf) *field approximation	Strength Test Type
			4" AC over 7" AB												
	365		GRAYWACKE pale yellow to yellow [BEDROCK]			50/2.5"									
			Boring terminated at a depth of 3.5 feet below ground surface. Groundwater not encountered during drilling.												

LOG - GEOTECHNICAL_SU+QU W/ ELEV ALL BORINGS.GPJ ENGEO INC.GDT 11/25/19

LOG OF BORING 1-B12

ENGEO
Expect Excellence

LATITUDE: 37.5379301699504 LONGITUDE: -122.328000180117

Geotechnical Exploration
Peninsula Heights
San Mateo, CA
16683.000.000

DATE DRILLED: 10/21/2019
HOLE DEPTH: 5.5 ft.
HOLE DIAMETER: 6.0 in.
SURF ELEV (SAN MATEO DATUM): Approx. 376½ ft.

LOGGED / REVIEWED BY: R. Ambrus / TPB
DRILLING CONTRACTOR: Britton Exploration
DRILLING METHOD: Hollow Stem Auger
HAMMER TYPE: 140 lb. Auto Trip

Depth in Feet	Elevation in Feet	Sample Type	DESCRIPTION	Log Symbol	Water Level	Blow Count/Foot	Atterberg Limits: Liquid Limit	Atterberg Limits: Plastic Limit	Atterberg Limits: Plasticity Index	Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Shear Strength (psf) *field approximation	Unconfined Strength (tsf) *field approximation	Strength Test Type
			3" AC over 8" AB												
	375		POORLY GRADED SAND WITH CLAY AND GRAVEL (SP-SC), pale yellow and strong brown, very dense, dry, angular, fine gravel [FILL]			68				9					
5			GRAYWACKE pale yellow [FRANCISCAN]			50/6"									
			Boring terminated at a depth of 5.5 feet below ground surface. Groundwater not encountered during drilling.												

LOG OF BORING 1-B13

ENGEO
Expect Excellence

LATITUDE: 37.5355590521324 LONGITUDE: -122.324717803029

Geotechnical Exploration
Peninsula Heights
San Mateo, CA
16683.000.000

DATE DRILLED: 10/23/2019
HOLE DEPTH: 13 ft.
HOLE DIAMETER: 6.0 in.
SURF ELEV (SAN MATEO DATUM): Approx. 270 ft.

LOGGED / REVIEWED BY: R. Ambrus / TPB
DRILLING CONTRACTOR: Britton Exploration
DRILLING METHOD: Hollow Stem Auger
HAMMER TYPE: 140 lb. Auto Trip

Depth in Feet	Elevation in Feet	Sample Type	DESCRIPTION	Log Symbol	Water Level	Blow Count/Foot	Atterberg Limits: Liquid Limit	Atterberg Limits: Plastic Limit	Atterberg Limits: Plasticity Index	Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Shear Strength (psf) *field approximation	Unconfined Strength (tsf) *field approximation	Strength Test Type
			3" AC over 6" AB												
			CLAYEY SAND WITH GRAVEL (SC), yellowish brown, medium dense, slightly moist, fine to coarse gravel [FILL]			23									
5	265		Trace cobbles up to 2" diameter			22									
			GRAYWACKE pale yellow [BEDROCK]			50/1"									
10	260					50/0"									
			Boring terminated at a depth of 13.0 feet below ground surface. Groundwater not encountered during drilling.												

LOG - GEOTECHNICAL_SU+QU W/ ELEV ALL BORINGS.GPJ ENGEO INC.GDT 11/25/19

LOG OF BORING 1-B14

ENGEO
Expect Excellence

LATITUDE: 37.5357842914433 LONGITUDE: -122.323881661069

Geotechnical Exploration
Peninsula Heights
San Mateo, CA
16683.000.000

DATE DRILLED:	10/23/2019
HOLE DEPTH:	7.5 ft.
HOLE DIAMETER:	6.0 in.
SURF ELEV (SAN MATEO DATUM):	Approx. 266 ft.
LOGGED / REVIEWED BY:	R. Ambrus / TPB
DRILLING CONTRACTOR:	Britton Exploration
DRILLING METHOD:	Hollow Stem Auger
HAMMER TYPE:	140 lb. Auto Trip

Depth in Feet	Elevation in Feet	Sample Type	DESCRIPTION	Log Symbol	Water Level	Blow Count/Foot	Atterberg Limits: Liquid Limit	Atterberg Limits: Plastic Limit	Atterberg Limits: Plasticity Index	Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Shear Strength (psf) *field approximation	Unconfined Strength (tsf) *field approximation	Strength Test Type
			3" AC over 6" AB												
	265		CLAYEY SAND (SC), yellowish brown, very dense, slightly moist, trace fine to coarse gravel [FILL]												
			GRAYWACKE pale yellow [BEDROCK]			50/5"									
5	260					50/2"									
			Boring terminated at a depth of 7.5 feet below ground surface. Groundwater not encountered during drilling.												

APPENDIX B

TEST PIT LOGS

ENGEO Expect Excellence		**TEST PIT LOGS**
Peninsula Heights San Mateo, California 16683.000.000		Logged By: James Allen, PG Logged Date: October 15 and 16, 2019 Equipment: Yanmar Vi055 Mini Excavator, 2.5’ bucket
Test Pit Number	**Depth (feet)**	**Description**
1-TP01	0-4	GRAYWACKE, olive yellow (2.5 YR 6/6), very strong fragments to stronger at depth, closely fractured, tight fractures at depth where confined, fractured non-cemented, FeO2 and FeMn coated surfaces, open/unhealed/weak, moderately weathered, dry, very difficult excavating and refusal at 6-feet deep from slope [FRANCISCAN COMPLEX]. Discontinuity Pattern (joint set) in Sandstone: **Joints** NS 90° to 70°W Refusal at 4 feet.
1-TP02	0-0.8	CLAYEY SAND (SC), dark gray (10 YR 4/1), very stiff, moist [FILL-LANDSCAPING].
	0.8-3.25	GRAYWACKE AND SILTSTONE, light olive brown (2.5 YR 5/3) and dark gray (2.5Y 4/1), very weak and weathered in upper foot of saprolitic rock, grading to very strong fragments at depth, closely fractured with non-cemented, very tight fractures, shale/argillite is siliceous and has very strong zone although closely fractured and sheared [FRANCISCAN COMPLEX]. Discontinuity Pattern (joint sets and bedding) in Sandstone: **Bedding** **Joint** **Bedding** N20°W N17°E N17°E 55°NE 35°SE 55°NW Refusal/very difficult time consuming excavation at 3.25 feet.
1-TP03	0 - 0.25	GRAYWACKE, light yellowish brown [FRANCISCAN COMPLEX].
1-TP04	0 – 0.8	SILT (ML) with some gravels, reddish brown (5YR 5/3), very dense, dry, gravels are very hard sandstone 3-6” in diameter and angular [FILL].
	0.8 – 3.25	GRAYWACKE with some thin SHALE interbeds, olive brown (2.5 YR 4/3), very strong fragments, closely fractured, fractured open/unhealed/weak, dry very difficult excavating and refusal at 3.25-feet deep from slope [FRANCISCAN COMPLEX]. Joint Pattern in Sandstone: N/S 90° and 89°W Samples collected for Point Load testing. Refusal/very difficult time consuming excavation at 3.25 feet.
1-TP05	0-1	SILT (ML) with some gravels, reddish brown (5YR 5/3), very dense, dry, gravels are very hard sandstone 3-6” in diameter and angular [FILL].

ENGEO Expect Excellence		TEST PIT LOGS
Peninsula Heights San Mateo, California 16683.000.000		Logged By: James Allen, PG Logged Date: October 15 and 16, 2019 Equipment: Yanmar Vi055 Mini Excavator, 2.5' bucket
Test Pit Number	**Depth (feet)**	**Description**
	1-2.75	GRAYWACKE, with some thin SHALE interbeds, olive brown (2.5 YR 4/3), very strong fragments, closely fractured, fractured open/unhealed/weak, dry very difficult excavating and refusal at 2.5-feet deep from slope [FRANCISCAN COMPLEX]. Joint Pattern in Sandstone: N20ºE 21ºSE Refusal/very difficult time consuming excavation at 2.75 feet.
1-TP06	0-1.4	GRAVELS (GC) with some clay, very dark grayish brown (2.5Y 3/2), dense, dry [FILL].
	1.4-3.9	GRAYWACKE, light yellowish brown (2.5 YR 6/4), strong fragments, closely fractured, fractured non-cemented, FeO2 and FeMn coated surfaces, open/unhealed/weak, moderately weathered, dry very difficult excavating and refusal at 6-feet deep from slope [FRANCISCAN COMPLEX]. Discontinuity Pattern (joint sets and bedding) in Sandstone: **Bedding** N40ºW 35ºNE
	3.9 - 5	CLAYSTONE/ARGILLITE, dark gray (2.5Y 4/1), ranges extremely strong to very strong, closely tightly fractured [FRANCSICAN COMPLEX]. Refusal/very difficult time consuming excavation at 5 feet.
1-TP07	0-1.25	GRAVELLY SILT (ML), reddish brown (5YR 5/3), very dense, dry, gravels are very hard sandstone 3-6" in diameter and angular [FILL].
	1.25 - 4	GRAYWACKE AND SHALE, olive brown (2.5 YR 4/3), very strong fragments, closely to very closely fractured, fractured non-cemented, FeO2 and FeMn coated, open/unhealed/weak, dry very difficult excavating and refusal at 2.5-feet deep from slope, un-fractured sandstone block measuring >2 -feet long and 1.5 feet thick [FRANCISCAN COMPLEX]. Discontinuity Pattern (Bedding) in Sandstone: N30ºW N30ºW N30ºE 37ºNE 36ºNE 47ºSE Refusal/very difficult time consuming excavation at 4 feet.
1-TP08	0-3.2	SILT (ML) with some gravels, reddish brown (5YR 5/3), very dense, dry, gravels are very hard sandstone 3-6" in diameter and angular [FILL].
	3.2-5.5	GRAYWACKE AND SHALE, olive brown (2.5 YR 4/3), very strong fragments, closely to very closely fractured, fractured non-cemnted, FeO2 and FeMn coated,

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TEST PIT LOGS

Peninsula Heights
San Mateo, California
16683.000.000

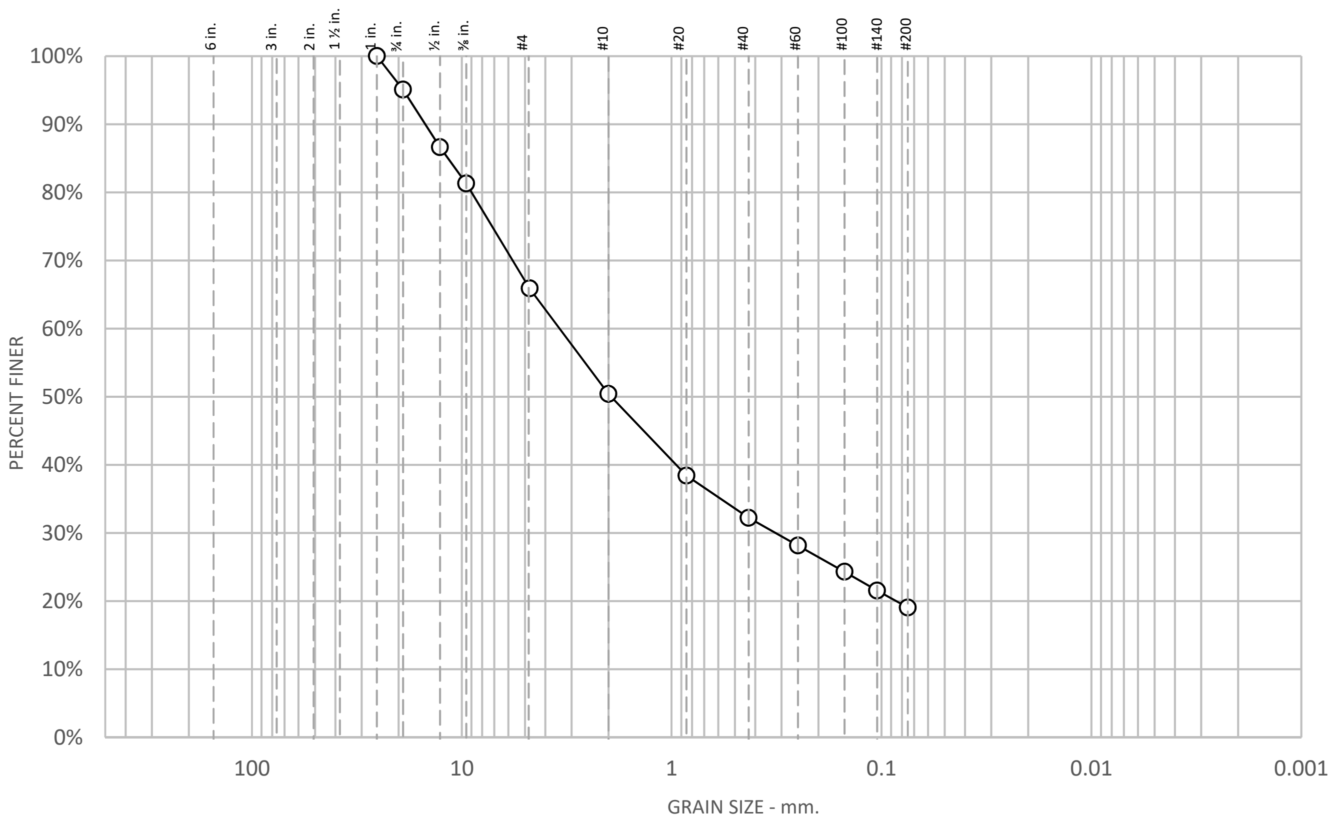
Logged By: James Allen, PG
Logged Date: October 15 and 16, 2019
Equipment: Yanmar Vi055 Mini Excavator, 2.5' bucket

Test Pit Number	Depth (feet)	Description
		open/unhealed/weak, dry very difficult excavating and refusal at 2.5-feet deep from slope, un-fractured sandstone block measuring >2 -feet long and 1.5 feet thick [FRANCISCAN COMPLEX]. <u>Discontinuity Pattern (bedding) in Sandstone:</u> N30°W 37°NE Refusal/very difficult time consuming excavation at 5.5 feet.
1-TP09	0-0.8 0.8-3.25	CLAYEY SILT (ML) with some gravels, light olive brown (2.5 YR 5/4), very hard/very dense, dry, [FILL]. GRAYWACKE, light yellowish brown (2.5 YR 6/4), strong fragments, closely fractured, fractured non-cemented, FeO2 and FeMn coated surfaces, open/unhealed/weak, moderately weathered, dry very difficult excavating and refusal at 6-feet deep from slope [FRANCISCAN COMPLEX]. Discontinuity Pattern (Joint sets and bedding) in Sandstone: **<u>Bedding Joints</u>** N30°E N70°E 25°NE 51°SE Refusal/very difficult time consuming excavation at 3.25 feet.
1-TP10	0-5 5-6	CLAYEY SILT (ML) with some gravels, light olive brown (2.5 YR 5/4), very hard/very dense, dry, [FILL]. GRAYWACKE, light yellowish brown (2.5 YR 6/4), strong fragments, closely fractured, fractured non-cemented, FeO2 and FeMn coated surfaces, open/unhealed/weak, moderately weathered, dry very difficult excavating and refusal at 6-feet deep from slope [FRANCISCAN COMPLEX]. Refusal/very difficult time consuming excavation at 6 feet.
1-TP11	0-1.5 1.5-2.5	Garden mulch and silty soils [FILL]. GRAYWACKE AND SHALE, light olive brown (2.5 YR 5/3) and dark gray (2.5Y 4/1), very strong fragments, closely fractured, fractures are non-cemented, very tight, shale/argillite is siliceous and has very strong zone although closely fractured and sheared [FRANCISCAN COMPLEX]. Refusal/very difficult time consuming excavation at 2.5 feet.

APPENDIX C

LABORATORY TEST RESULTS

Particle Size Distribution Report



% +75mm	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
	5.0	29.1	15.5	18.2	13.2	19.1	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1 in.	100.0		
¾ in.	95.1		
½ in.	86.6		
⅜ in.	81.3		
#4	65.9		
#10	50.4		
#20	38.4		
#40	32.2		
#60	28.1		
#100	24.3		
#140	21.5		
#200	19.1		

* (no specification provided)

Soil Description

See exploration logs

Atterberg Limits

PL = LL = PI =

Coefficients

D_{90} = 14.9314 mm D_{85} = 11.6327 mm D_{60} = 3.4170 mm

D_{50} = 1.9424 mm D_{30} = 0.3198 mm D_{15} =

D_{10} = C_u = C_c =

Classification

USCS =

Remarks

ASTM D6913, Method B

Sample Number: 1-B02 @ 4-5

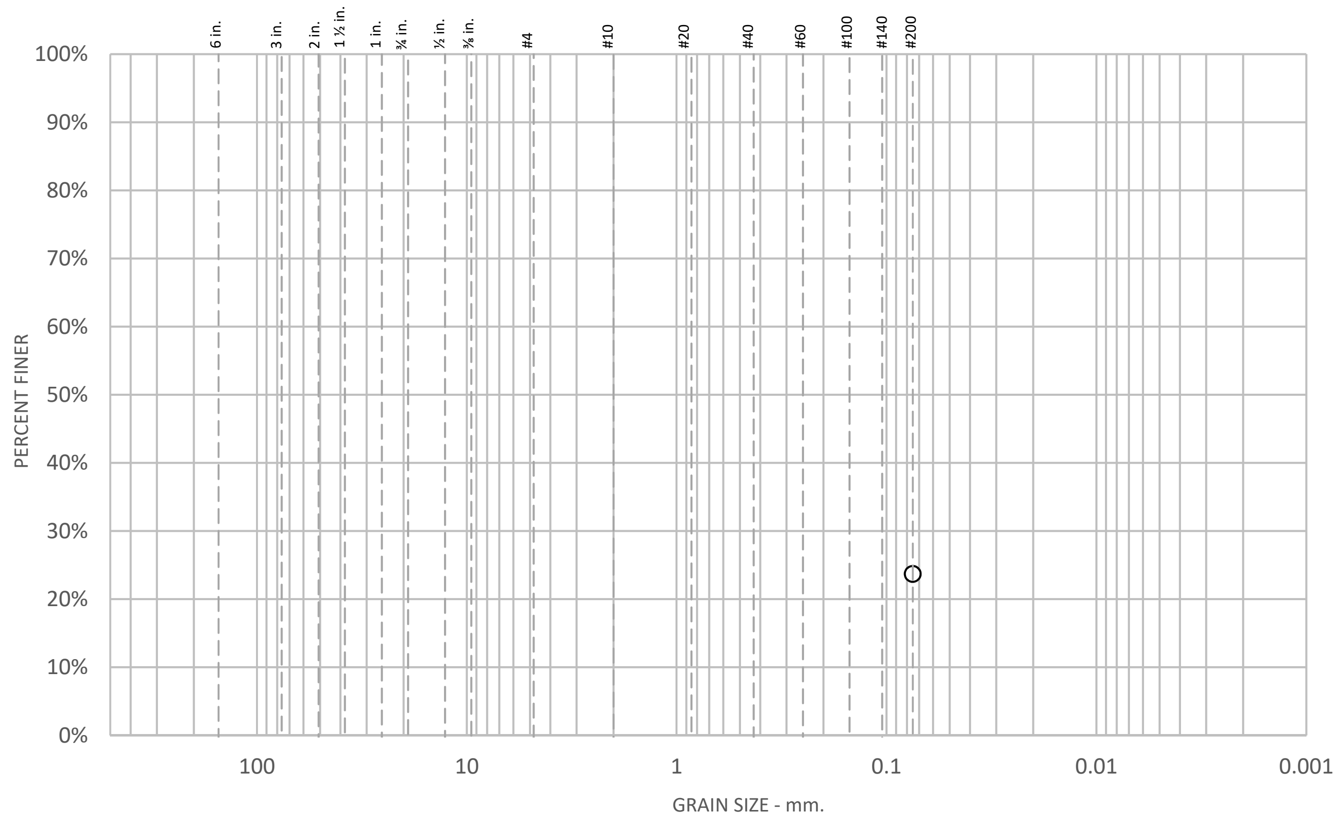
Client: Campus POP Investors, LLC
Project Number: 16683.000.000
Project: Peninsula Heights
Date: 11/11/2019
Project location: San Mateo, California

ENGEO
Expect Excellence

Tested By: C. Bruns **Checked By:** M. Quasem

Test Location: 3420 Fostoria Way, Suite E, Danville, CA 94526

Particle Size Distribution Report



% +75mm	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
						23.7	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	23.7		

* (no specification provided)

Soil Description

See exploration logs

Atterberg Limits

PL = 16 LL = 29 PI = 13

Coefficients

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

Classification

USCS =

Remarks

PI: ASTM D4318, Wet Method

ASTM D1140, Method B
Soak time = 180 min
Dry sample weight = 268.43 g

Sample Number: 1-B03 @ 14-15.5

Client: Campus POP Investors, LLC
Project: Peninsula Heights
Project location: San Mateo, California

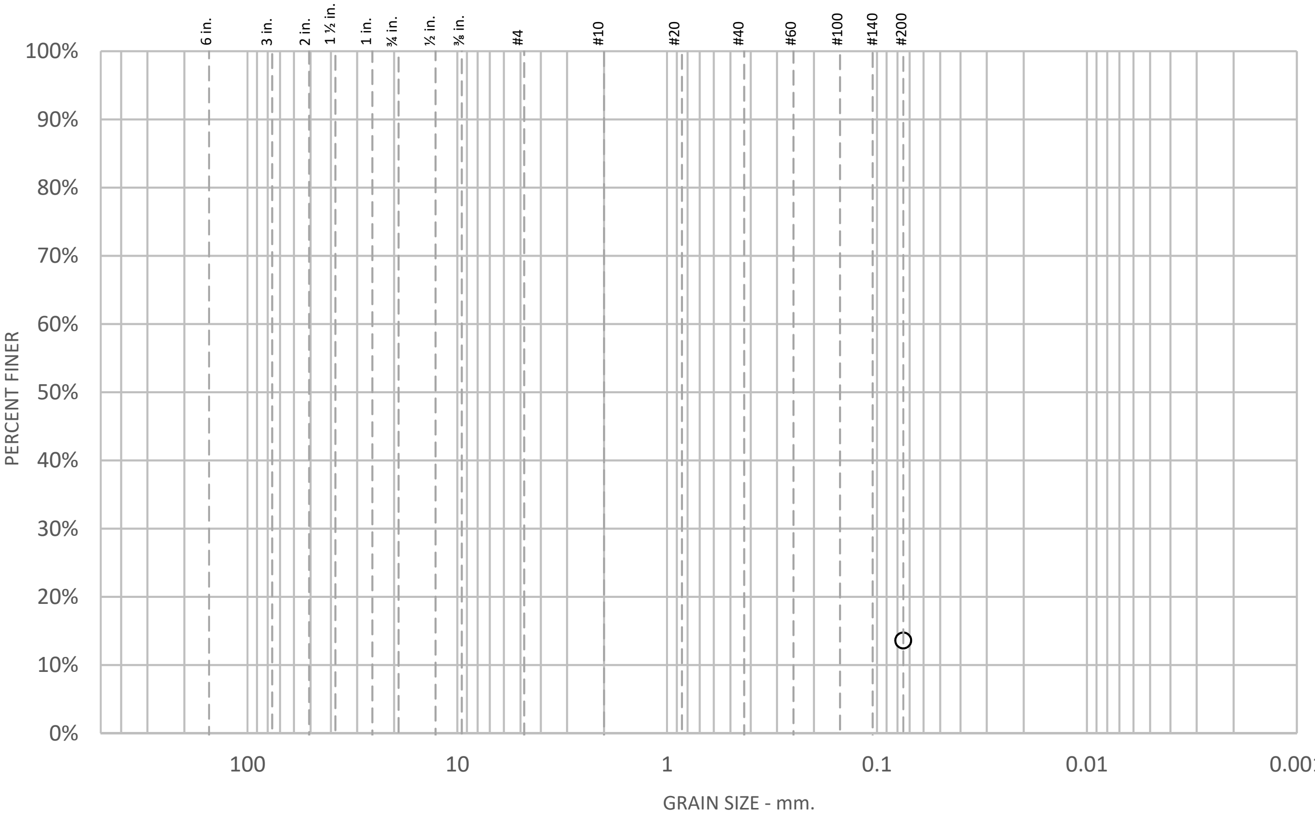
Project Number: 16683.000.000
Date: 11/11/2019

ENGEO
Expect Excellence

Tested By: C. Bruns **Checked By:** M. Quasem

Test Location: 3420 Fostoria Way, Suite E, Danville, CA 94526

Particle Size Distribution Report



% +75mm	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
						13.6	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	13.6		

* (no specification provided)

Soil Description

See exploration logs

Atterberg Limits

PL = 19 LL = 29 PI = 10

Coefficients

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

Classification

USCS =

Remarks

PI: ASTM D4318, Wet Method
ASTM D1140, Method B
Soak time = 180 min
Dry sample weight = 114.3 g

Sample Number: 1-B07 @ 10.5-11

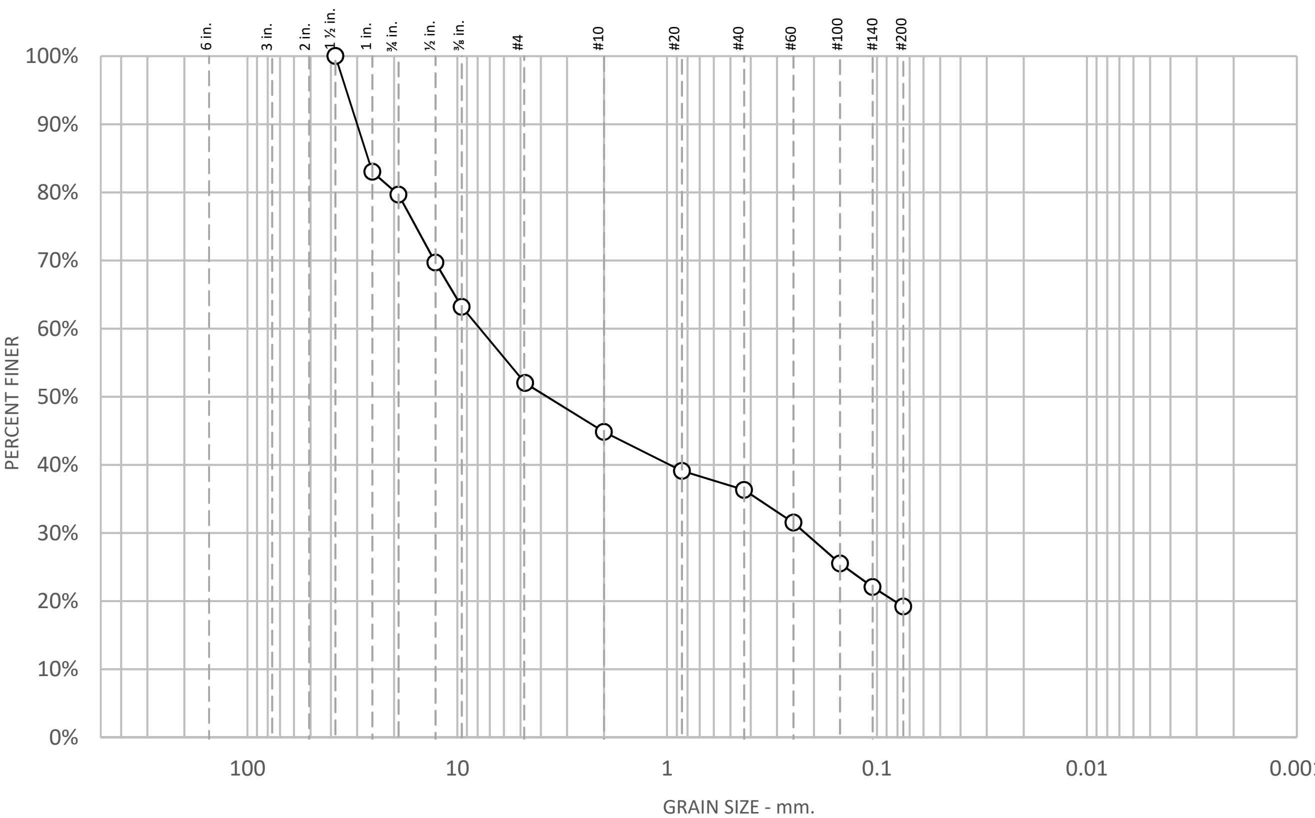
Client: Campus POP Investors, LLC
Project: Peninsula Heights
Project location: San Mateo, California
Project Number: 16683.000.000
Date: 11/11/2019

ENGEO Expect Excellence

Tested By: C. Bruns
Checked By: M. Quasem

Test Location: 3420 Fostoria Way, Suite E, Danville, CA 94526

Particle Size Distribution Report



% +75mm	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
	20.4	27.6	7.2	8.5	17.1	19.2	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1-½ in.	100.0		
1 in.	83.0		
¾ in.	79.7		
½ in.	69.7		
⅜ in.	63.2		
#4	52.0		
#10	44.8		
#20	39.1		
#40	36.3		
#60	31.5		
#100	25.5		
#140	22.1		
#200	19.2		

* (no specification provided)

Soil Description

See exploration logs

Atterberg Limits

PL = LL = PI =

Coefficients

D_{90} = 30.0138 mm D_{85} = 26.6390 mm D_{60} = 7.8115 mm
D_{50} = 3.7321 mm D_{30} = 0.2199 mm D_{15} =
D_{10} = C_u = C_c =

Classification

USCS =

Remarks

ASTM D6913, Method B

Sample Number: 1-B07 @ 6.5-7'

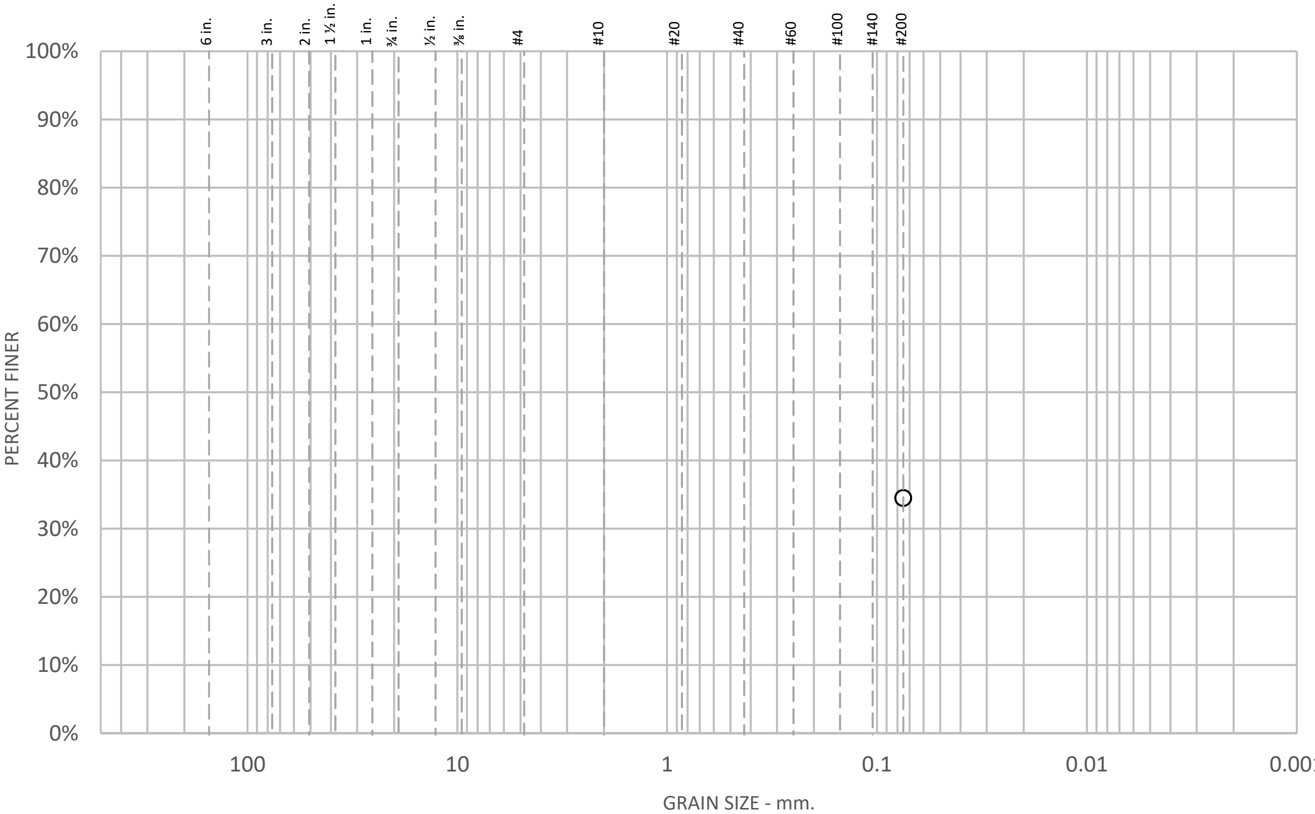
Client: Campus POP Investors, LLC
Project Number: 16683.000.000
Project: Peninsula Heights
Date: 11/11/2019
Project location: San Mateo, California

ENGEO
Expect Excellence

Tested By: C. Bruns
Checked By: M. Quasem

Test Location: 3420 Fostoria Way, Suite E, Danville, CA 94526

Particle Size Distribution Report



% +75mm	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
						34.5	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	34.5		

* (no specification provided)

Soil Description

See exploration logs

Atterberg Limits

PL = LL = PI =

Coefficients

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

Classification

USCS =

Remarks

ASTM D1140, Method B
Soak time = 180 min
Dry sample weight = 213.76 g

Sample Number: 1-B08 @ 20-21.5

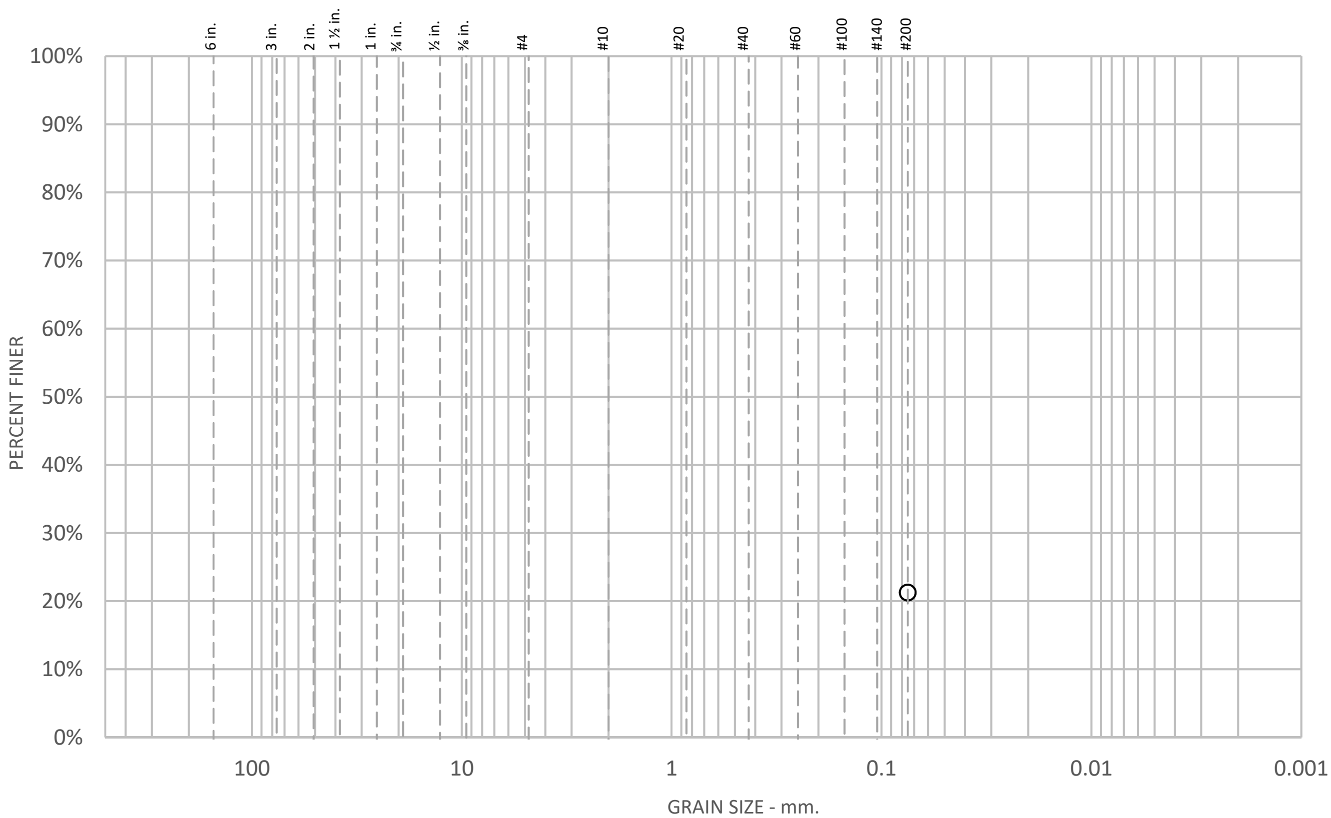
Client: Campus POP Investors, LLC
Project Number: 16683.000.000
Project: Peninsula Heights
Date: 11/11/2019
Project location: San Mateo, California

ENGEO Expect Excellence

Tested By: C. Bruns **Checked By:** M. Quasem

Test Location: 3420 Fostoria Way, Suite E, Danville, CA 94526

Particle Size Distribution Report



% +75mm	% 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 =

Remarks

ASTM D1140, Method B
Soak time = 180 min
Dry sample weight = 350.35 g

Sample Number: 1-B08 @ 3-3.5

Client: Campus POP Investors, LLC
Project: Peninsula Heights
Project location: San Mateo, California

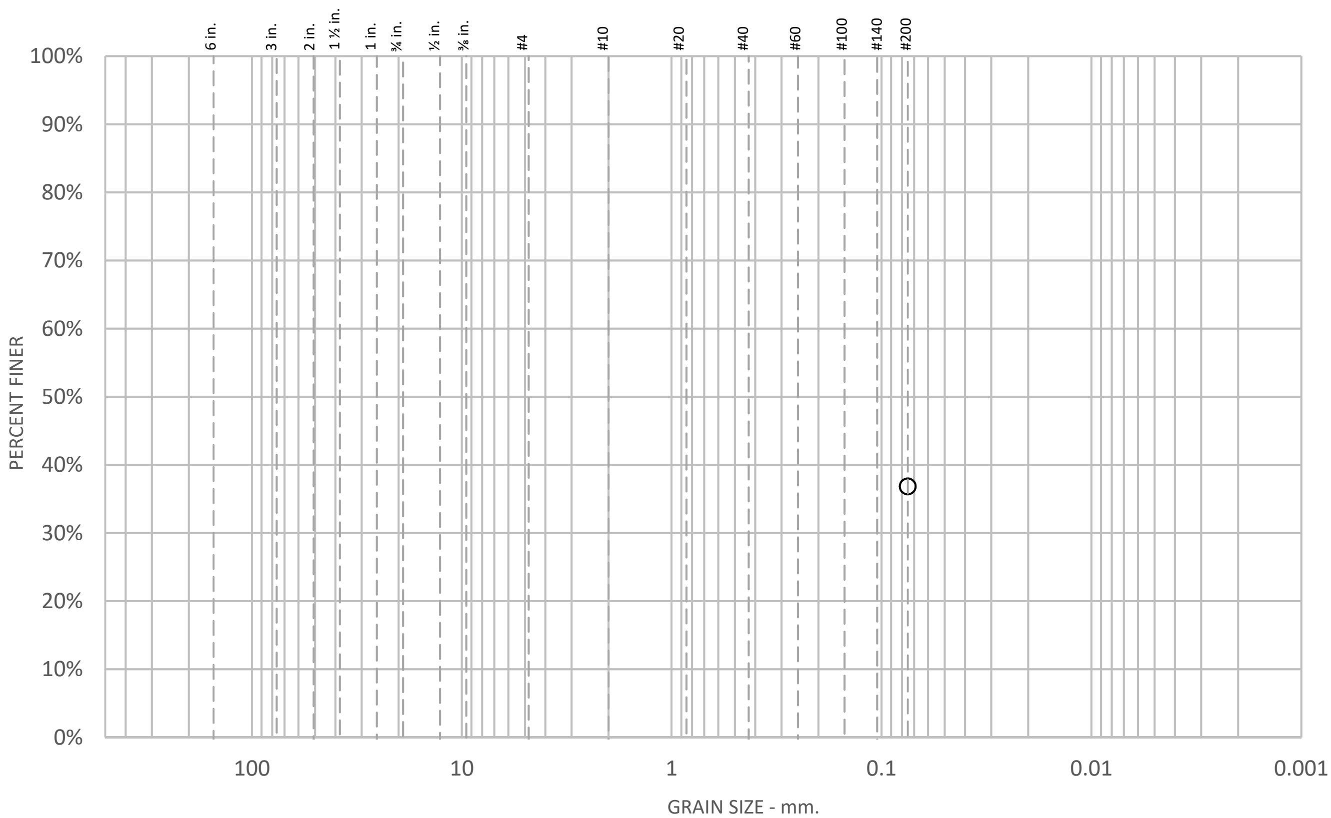
Project Number: 16683.000.000
Date: 11/11/2019

ENGEO Expect Excellence

Tested By: C. Bruns **Checked By:** M. Quasem

Test Location: 3420 Fostoria Way, Suite E, Danville, CA 94526

Particle Size Distribution Report



% +75mm	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
						36.8	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	36.8		

* (no specification provided)

Soil Description

See exploration logs

Atterberg Limits

PL = 17 LL = 30 PI = 13

Coefficients

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

Classification

USCS =

Remarks

PI: ASTM D4318, Wet Method
ASTM D1140, Method B
Soak time = 180 min
Dry sample weight = 156.87 g

Sample Number: 1-B09 @ 4-4.5

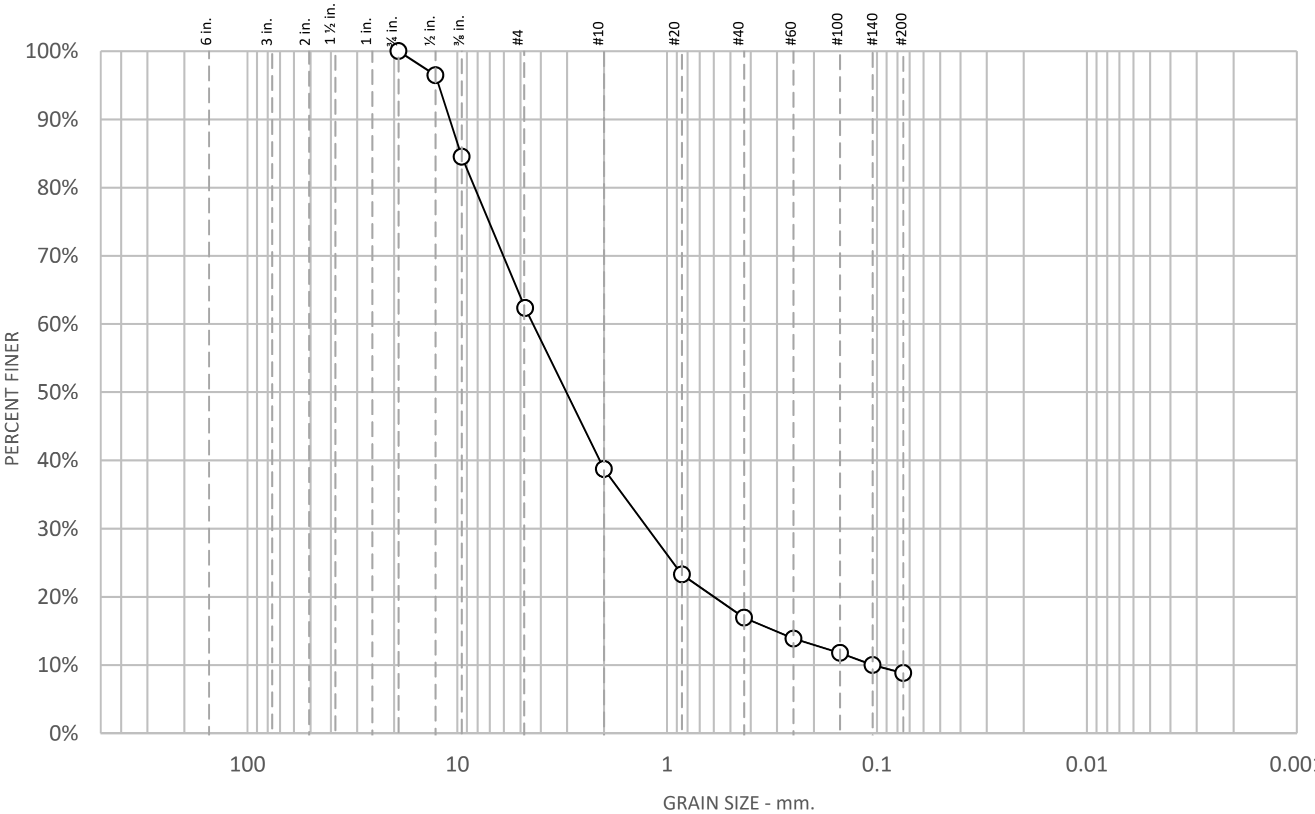
Client: Campus POP Investors, LLC
Project Number: 16683.000.000
Project: Peninsula Heights
Date: 11/11/2019
Project location: San Mateo, California

ENGEO Expect Excellence

Tested By: C. Bruns **Checked By:** M. Quasem

Test Location: 3420 Fostoria Way, Suite E, Danville, CA 94526

Particle Size Distribution Report



% +75mm	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
	0.0	37.6	23.6	21.8	8.1	8.8	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
¾ in.	100.0		
½ in.	96.5		
⅜ in.	84.5		
#4	62.3		
#10	38.7		
#20	23.3		
#40	16.9		
#60	13.9		
#100	11.8		
#140	10.0		
#200	8.8		

* (no specification provided)

Soil Description

See exploration logs

Atterberg Limits

PL = LL = PI =

Coefficients

D_{90} = 10.8675 mm D_{85} = 9.6352 mm D_{60} = 4.3606 mm
D_{50} = 3.0229 mm D_{30} = 1.2336 mm D_{15} = 0.3055 mm
D_{10} = 0.1050 mm C_u = 41.52 C_c = 3.32

Classification

USCS = SP

Remarks

ASTM D6913, Method B

Sample Number: 1-B12 @ 3-3.5

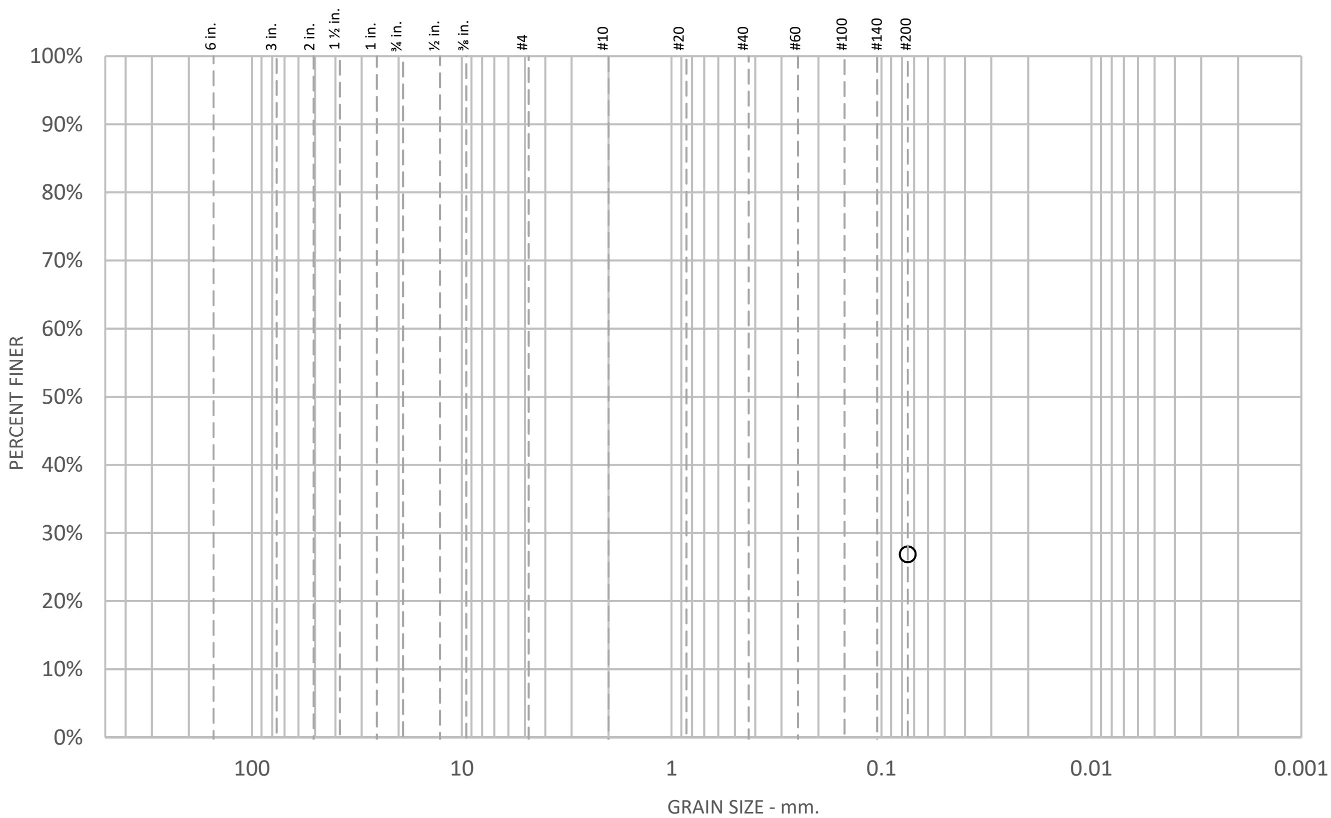
Client: Campus POP Investors, LLC **Project Number:** 16683.000.000
Project: Peninsula Heights **Date:** 11/11/2019
Project location: San Mateo, California

ENGEO Expect Excellence

Tested By: C. Bruns **Checked By:** M. Quasem

Test Location: 3420 Fostoria Way, Suite E, Danville, CA 94526

Particle Size Distribution Report



% +75mm	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
						26.8	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	26.8		

* (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 =

Remarks

ASTM D1140, Method B
Soak time = 180 min
Dry sample weight = 562.03 g

Sample Number: 1-B15 @ 19-20.5

Client: Campus POP Investors, LLC
Project: Peninsula Heights
Project location: San Mateo, California

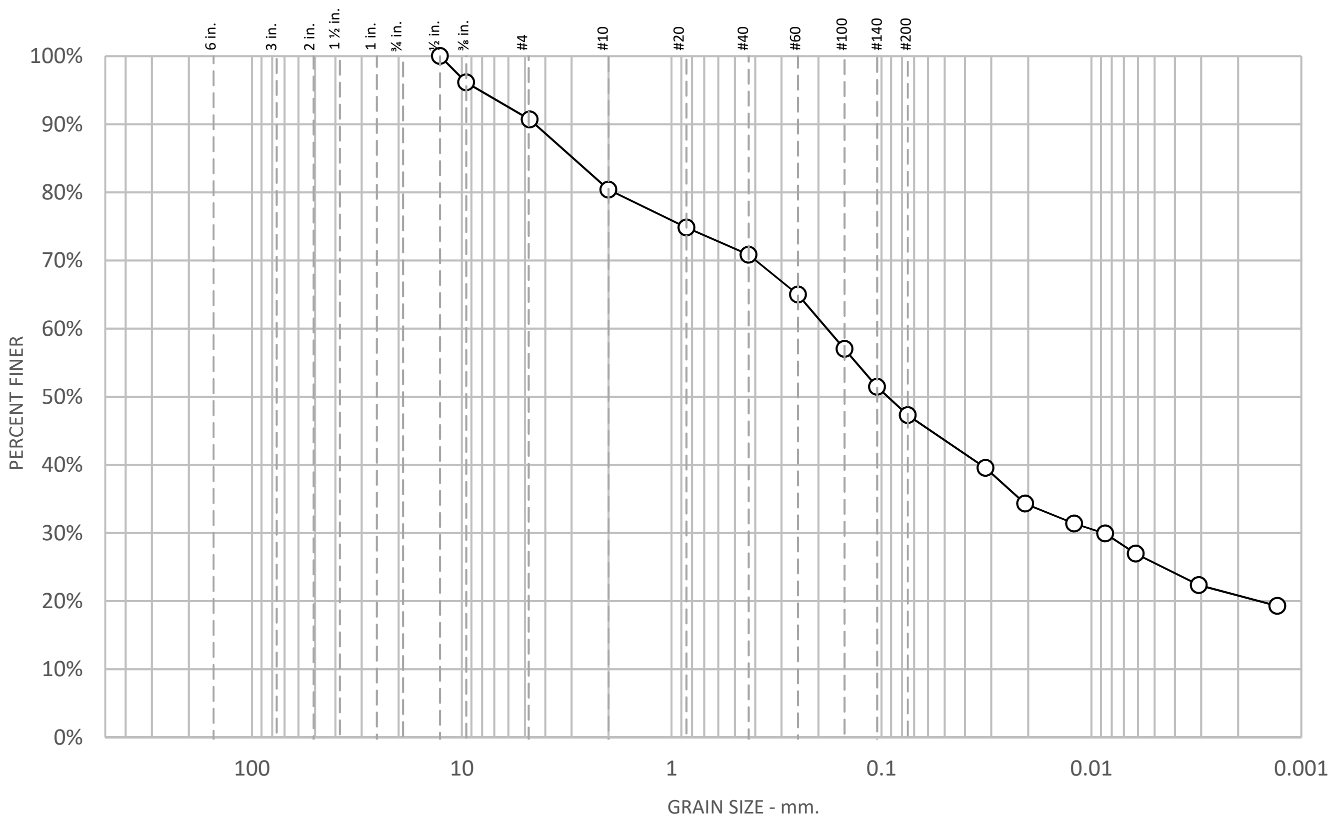
Project Number: 16683.000.000
Date: 11/11/2019

ENGEO Expect Excellence

Tested By: C. Bruns **Checked By:** M. Quasem

Test Location: 3420 Fostoria Way, Suite E, Danville, CA 94526

Particle Size Distribution Report



% +75mm	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
		9.3	10.3	9.6	23.5	26.5	20.8

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
½ in.	100.0		
⅜ in.	96.1		
#4	90.7		
#10	80.4		
#20	74.8		
#40	70.8		
#60	65.0		
#100	57.0		
#140	51.4		
#200	47.3		
0.0319 mm.	39.5		
0.0207 mm.	34.3		
0.0121 mm.	31.4		
0.0086 mm.	29.9		
0.0061 mm.	26.9		
0.0031 mm.	22.3		
0.0013 mm.	19.3		

* (no specification provided)

Soil Description

See exploration logs

Atterberg Limits

PL = LL = PI =

Coefficients

D_{90} = 4.4799 mm D_{85} = 2.9446 mm D_{60} = 0.1817 mm
D_{50} = 0.0934 mm D_{30} = 0.0088 mm D_{15} =
D_{10} = C_u = C_c =

Classification

USCS =

Remarks

GS: ASTM D422 ASTM D422
Silt/clay division of 0.002mm used

Sample Number: 1-B17 @ 6-6.5

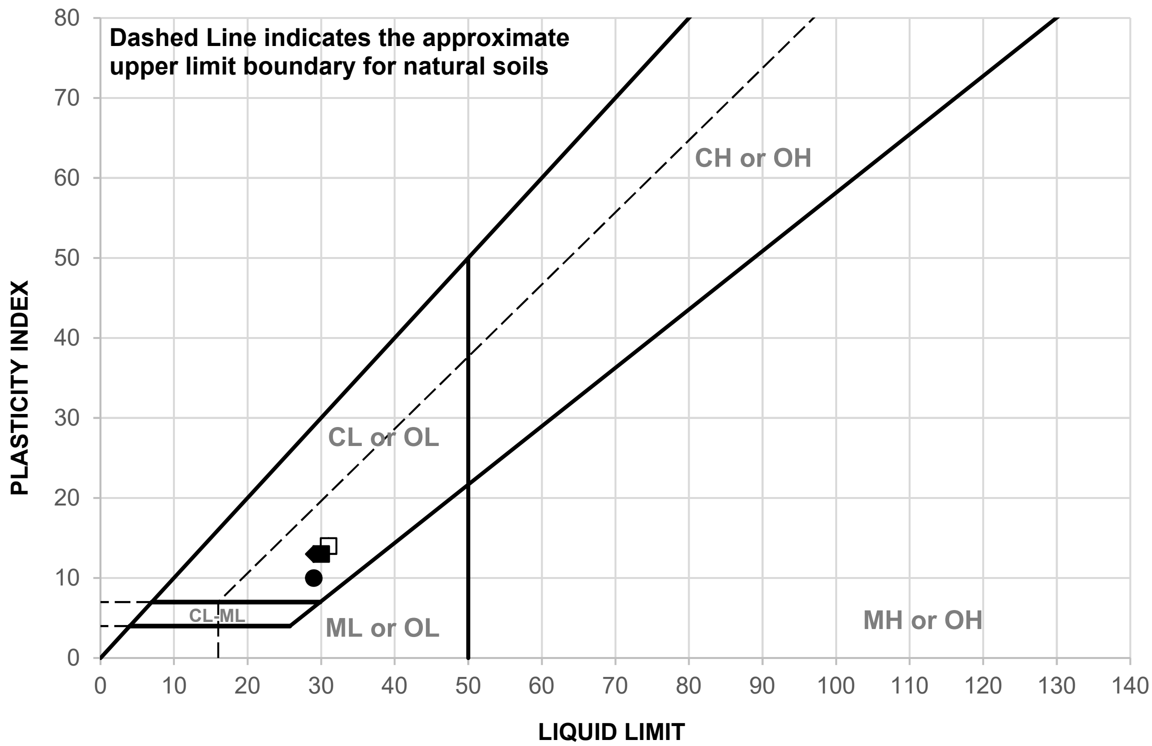
Client: Campus POP Investors, LLC **Project Number:** 16683.000.000
Project: Penninsula Heights **Date:** 11/13/2019
Project location: San Mateo, California

ENGEO Expect Excellence

Tested By: M. Quasem **Checked By:** W. Miller

Test Location: 3420 Fostoria Way, Suite E, Danville, CA 94526

LIQUID AND PLASTIC LIMITS TEST REPORT
ASTM D4318



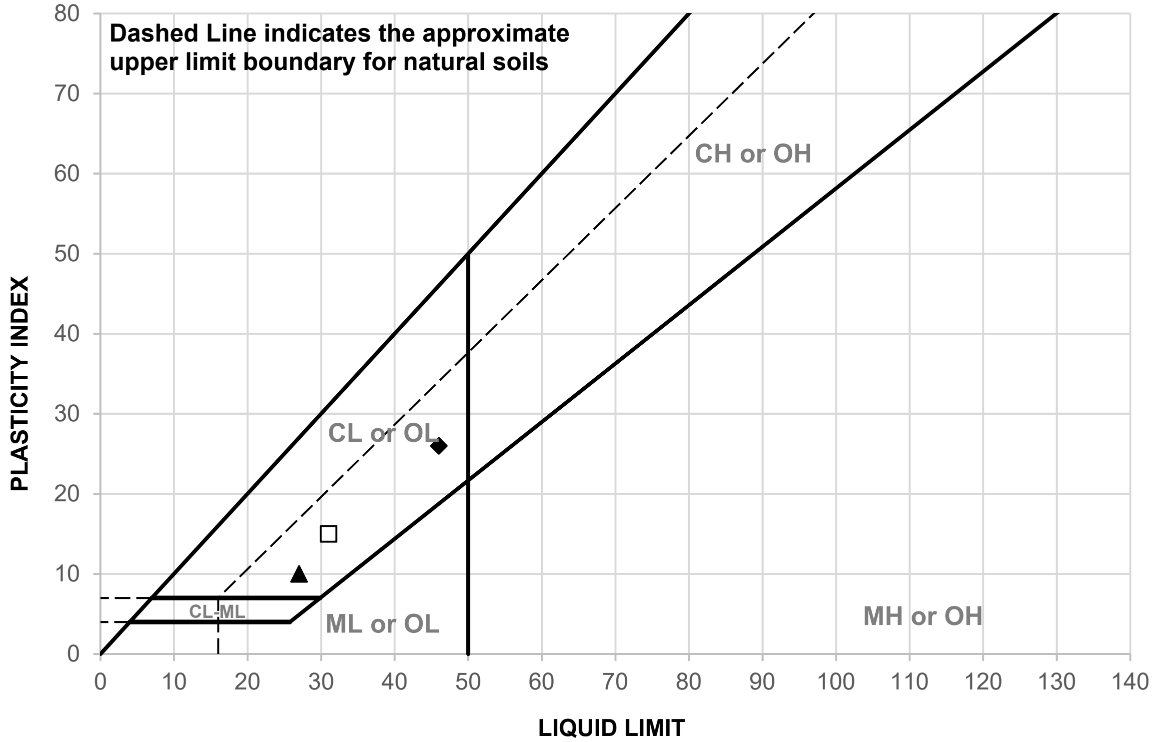
	SAMPLE ID	DEPTH	MATERIAL DESCRIPTION	LL	PL	PI
▲	1-B03	3.5-4	See exploration logs	31	17	14
◆	1-B03	14-15.5	See exploration logs	29	16	13
□	1-B04	3.5-4	See exploration logs	31	17	14
●	1-B07	10.5-11	See exploration logs	29	19	10
■	1-B09	4-4.5	See exploration logs	30	17	13

	SAMPLE ID	TEST METHOD	REMARKS
▲	1-B03	PI: ASTM D4318, Wet Method	
◆	1-B03	PI: ASTM D4318, Wet Method	
□	1-B04	PI: ASTM D4318, Wet Method	
●	1-B07	PI: ASTM D4318, Wet Method	
■	1-B09	PI: ASTM D4318, Wet Method	

ENGEO
Expect Excellence

CLIENT: Campus POP Investors, LLC
PROJECT NAME: Peninsula Heights
PROJECT NO: 16683.000.000
PROJECT LOCATION: San Mateo, California
REPORT DATE: 11/11/2019
TESTED BY: L. Santo Domingo
REVIEWED BY: M. Quasem

LIQUID AND PLASTIC LIMITS TEST REPORT
ASTM D4318



	SAMPLE ID	DEPTH	MATERIAL DESCRIPTION	LL	PL	PI
▲	1-B15	4-5.5	See exploration logs	27	17	10
◆	1-B17	3-3.5	See exploration logs	46	20	26
□	1-B17	16-17.5	See exploration logs	31	16	15

	SAMPLE ID	TEST METHOD	REMARKS
▲	1-B15	PI: ASTM D4318, Wet Method	
◆	1-B17	PI: ASTM D4318, Wet Method	
□	1-B17	PI: ASTM D4318, Wet Method	

ENGEO
Expect Excellence

CLIENT: Campus POP Investors, LLC
PROJECT NAME: Peninsula Heights
PROJECT NO: 16683.000.000
PROJECT LOCATION: San Mateo, California
REPORT DATE: 11/11/2019
TESTED BY: L. Santo Domingo
REVIEWED BY: M. Quasem

SUMMARY OF POINT LOAD TESTS
Peninsula Heights Project
16683.000.000
San Mateo, California

= input

Jack Piston Area = 1.5 in^2

Average UCS (psi) 9055

Exploration No.	Top Depth	Bottom Depth	Test Type Diam./Axial	Diameter	Width Axial Test	After Test D'	Change D	Equiv. Diam. D_e^2	Rupture Load	Rupture Load	Rupture Force	Uncorrected PLSI (Is)	Corrected PLSI (Is50)	Corrected PLSI (Is50)	Estimated Compressive Strength	Estimated Compressive Strength	Rock Type	Test Validation	Comments
-	(feet)	(feet)	-	(mm)	(in)	(mm)	(%)	(mm^2)	(psi)	(kPa)	(N)	(MPa)	(MPa)	(psi)	(MPa)	(psi)	-	V/I	
1-B01	15.0		D	21		20	5	441	726	5006	4841	10.98	7.43	1077	167.2	24244	Graywacke	I	Invalid test dimension, broke irreg. along weak discontinuity, rock frag test.
1-B01	23.0	25.0	D	22		21	5	484	654	4509	4361	9.01	6.23	903	140.1	20320	Graywacke	I	Invalid test dimension, broke irreg. along weak discontinuity, rock frag test.
1-B01	23.0	25.0	D	22		21	5	484	256	1765	1707	3.53	2.44	354	54.8	7954	Graywacke	I	Invalid test dimension, broke irreg. along weak discontinuity, rock frag test.
1-B02	13.0	18.0	D	33		32.5	2	1089	445	3068	2967	2.72	2.26	328	50.8	7375	Graywacke	I	Invalid test dimension, broke irreg. along weak discontinuity, rock frag test.
1-B02	23.0	26.0	D	60		57	5	3600	74	510	493	0.14	0.15	22	3.3	486	Graywacke	I	Compressed 3 mm prior to breaking. Indicator of cement and weathering. Invalid test dimension, broke irreg. along weak discontinuity, rock frag test.
1-B02	23.0	26.0	D	60		58	3	3600	65	448	433	0.12	0.13	19	2.9	426	Graywacke	I	Compressed 2 mm prior to breaking. Indicator of cement and weathering. Invalid test dimension, broke irreg. along weak discontinuity, rock frag test.
1-B02	23.0	26.0	D	60		59	2	3600	393	2710	2620	0.73	0.79	115	17.8	2578	Graywacke	I	Compressed 2 mm prior to breaking. Indicator of cement and weathering. Average with previous 2 tests. Invalid test dimension, broke irreg. along weak discontinuity, rock frag test.
1-TP04		3.3	D	35		35	0	1225	1068	7364	7121	5.81	4.95	718	111.4	16157	Graywacke	I	Invalid test dimension, broke irreg. along weak discontinuity, rock frag test.
1-TP04		3.3	D	26		26	0	676	270	1862	1800	2.66	1.98	288	44.6	6475	Graywacke	I	Invalid test dimension, broke irreg. along weak discontinuity, rock frag test.
1-TP04		3.3	D	34		34	0	1156	3254	22436	21696	18.77	15.78	2288	355.0	51490	Graywacke	I	Invalid test dimension, broke irreg. along weak discontinuity, rock frag test.
1-TP04		3.3	D	19		19	0	361	959	6612	6394	17.71	11.46	1662	257.8	37398	Graywacke	I	Invalid test dimension, broke irreg. along weak discontinuity, rock frag test.
1-TP04		3.3	D	29		29	0	841	470	3241	3134	3.73	2.92	423	65.6	9517	Graywacke	I	Invalid test dimension, broke irreg. along weak discontinuity, rock frag test.
1-TP04		3.3	D	22		22	0	484	765	5275	5101	10.54	7.28	1056	163.9	23769	Graywacke	I	Invalid test dimension, broke irreg. along weak discontinuity, rock frag test.

APPENDIX D

CORROSIVITY TEST RESULTS (CERCO ANALYTICAL)

California State Certified Laboratory No. 2153

CERCO analytical
1100 Willow Pass Court, Suite A
Concord, CA 94520-1006
925 462 2771 Fax. 925 462 2775
www.cercoanalytical.com

Client:	ENGEO Incorporated
Client's Project No.:	16683.000.000
Client's Project Name:	Peninsula Heights
Date Sampled:	10/22 & 23/19
Date Received:	11-Nov-19
Matrix:	Soil
Authorization:	Signed Chain of Custody

Date of Report: 19-Nov-2019

Job/Sample No.	Sample I.D.	Redox (mV)	pH	Conductivity (umhos/cm)*	Resistivity (100% Saturation) (ohms-cm)	Sulfide (mg/kg)*	Chloride (mg/kg)*	Sulfate (mg/kg)*
1911057-001	1-B03 @ 4-4.5'	200	8.35	-	3,400	-	N.D.	N.D.
1911057-002	1-B07 @ 3-4.5'	240	8.22	-	2,600	-	N.D.	31

	Redox	pH	Conductivity	Resistivity	Sulfide	Chloride	Sulfate
Method:	ASTM D1498	ASTM D4972	ASTM D1125M	ASTM G57	ASTM D4658M	ASTM D4327	ASTM D4327
Reporting Limit:	-	-	10	-	50	15	15
Date Analyzed:	12-Nov-2019	12-Nov-2019	-	18-Nov-2019	-	12-Nov-2019	12-Nov-2019

* Results Reported on "As Received" Basis

N.D. - None Detected

Cheryl McMillen
Laboratory Director

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

1911057

11521

CHAIN OF CUSTODY RECORD

PROJECT NUMBER: 16683.000.000

PROJECT NAME: Peninsula Heights

SAMPLED BY: (PRINT) Yanet Zepeda

PROJECT MANAGER: Yanet Zepeda 408-215-7009

ROUTING: E-MAIL vzepeda@engeo.com Hard Copy NA

	SAMPLE NUMBER	DATE	TIME	MATRIX	NUMBER OF CONTAINERS	CONTAINER SIZE	PRESERVATIVE	Redox	pH	Sulfate	Resistivity	Chloride				REMARKS REQUIRED DETECTION LIMITS
001	1-B03 @ 4-4.5	10/22/2019	—	Soil	1	Liner	N/A	X	X	X	X	X	X		Tube	ASTM Test Methods
002	1-B07 @ 3-4.5	10/23/2019	—	Soil	1	Baggie	N/A	X	X	X	X	X	X			ASTM Test Methods

RELINQUISHED BY: (SIGNATURE) — DATE/TIME 11/7 2:00 pm — RECEIVED BY: (SIGNATURE)

RELINQUISHED BY: (SIGNATURE) — DATE/TIME 11/11/19 1305 — RECEIVED BY: (SIGNATURE)

RELINQUISHED BY: (SIGNATURE) — DATE/TIME — RECEIVED FOR LABORATORY BY: (SIGNATURE) — DATE/TIME

RELINQUISHED BY: (SIGNATURE) — DATE/TIME 11/11 12:30 — RECEIVED BY: (SIGNATURE)

RELINQUISHED BY: (SIGNATURE) — DATE/TIME — RECEIVED BY: (SIGNATURE)

REMARKS

STANDARD TAT

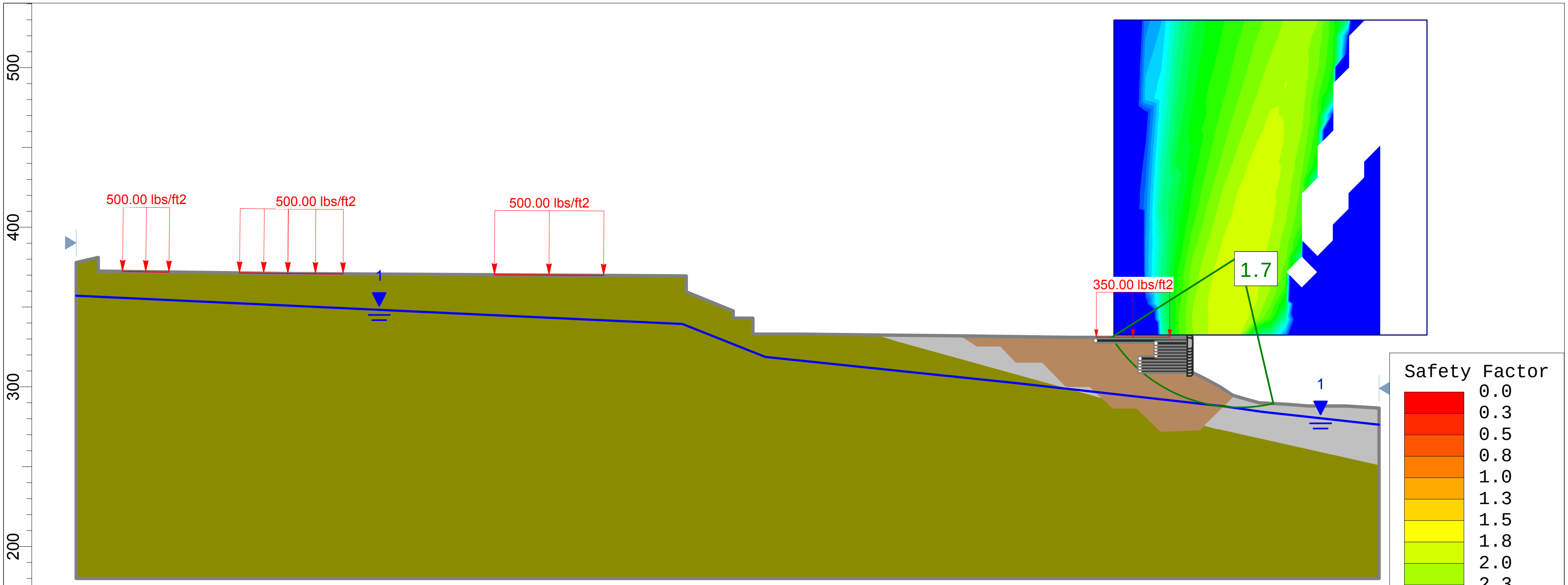
ENGEO INCORPORATED

6399 SAN IGNACIO AVENUE, SUITE 150
SAN JOSE, CALIFORNIA 95119
(408) 574-4900 FAX (888) 279-2698
WWW.ENGEO.COM

DISTRIBUTION: ORIGINAL ACCOMPANIES SHIPMENT; COPY TO PROJECT FIELD FILES

APPENDIX E

SLOPE STABILITY ANALYSES



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	UCS (psf)
Franciscan Complex (KJfsr)		160	Generalized Hoek-Brown			1e+06
Engineered Fill (Proposed)		125	Mohr-Coulomb	300	33	
Artificial Fill (Existing)		125	Mohr-Coulomb	50	30	

Support Name	Color	Force Application	Tensile Strength (lbs/ft)
Geogrid		Active (Method A)	3200

ENGEO
Expect Excellence

SLIDEINTERPRET 8.024

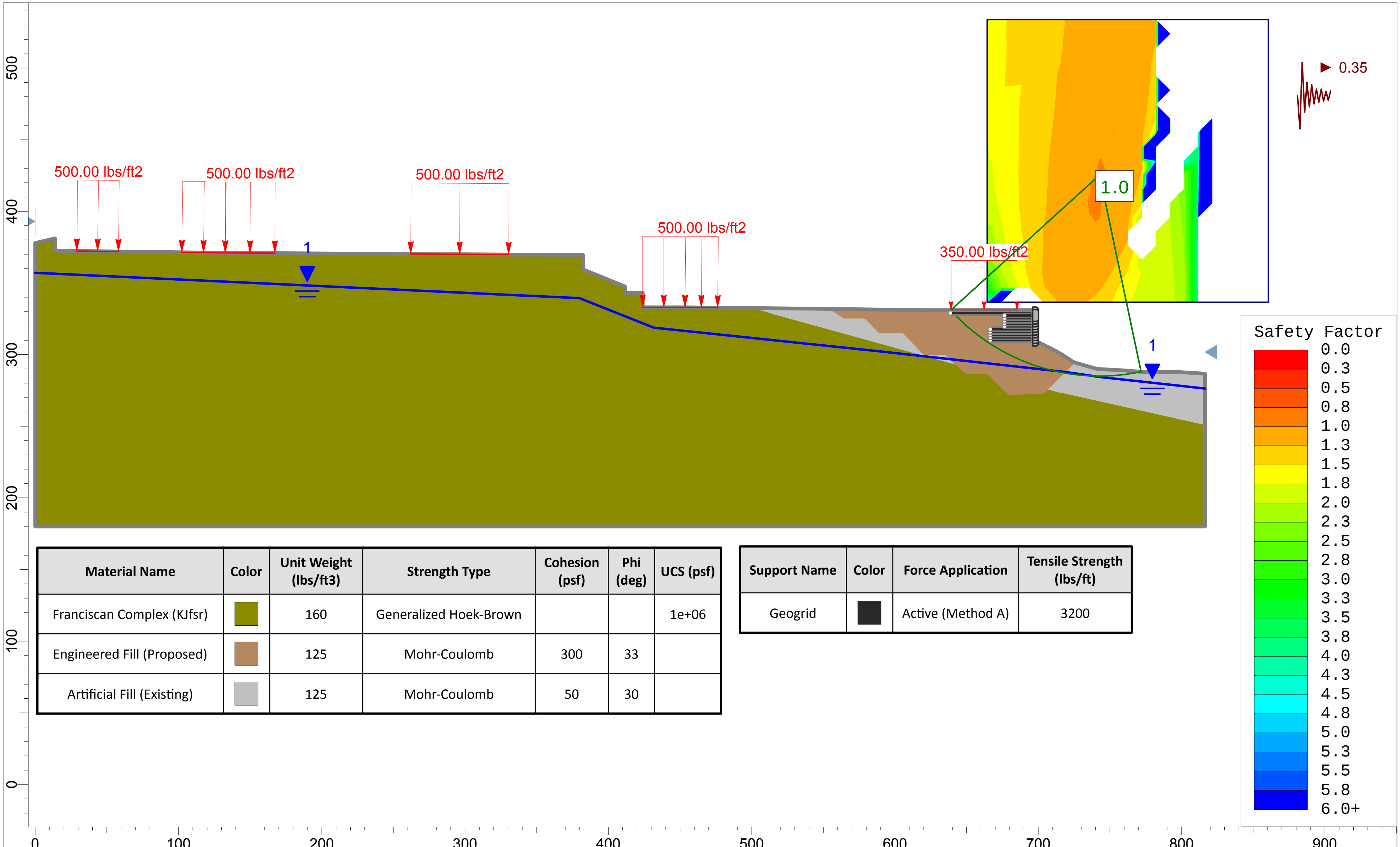
Project
Peninsula Heights - Residential Development

Slope Stability Analysis
Static Slope Stability - Section A-A'

Drawn By Y. Zepeda | *Scale* 1:1118 | *Company* ENGEO Incorporated

February 10, 2020

File Name 16683.000.000_Slope Stability Analysis_Revised 2020-01-17.slmd



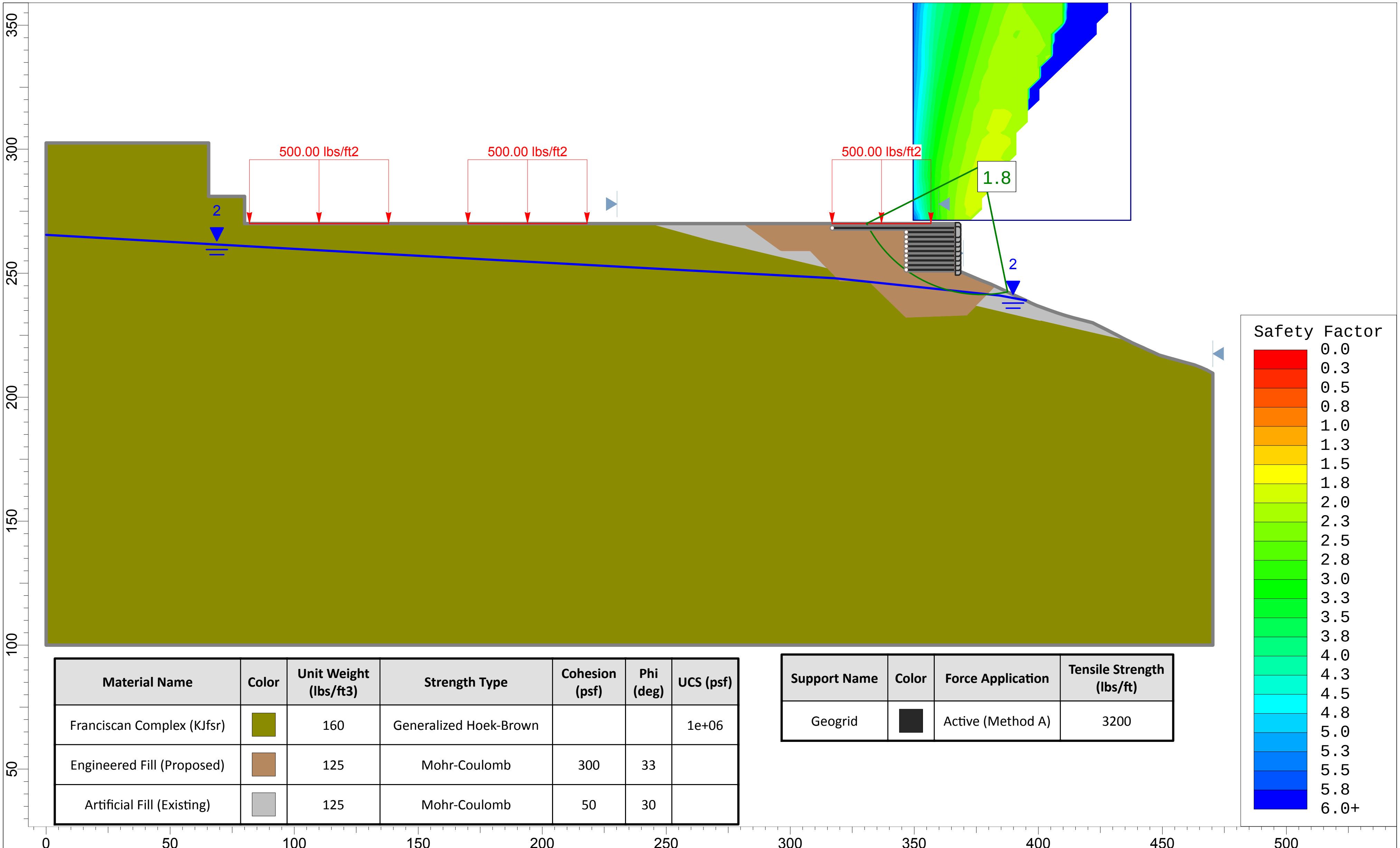
Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	UCS (psf)
Franciscan Complex (KJfsr)		160	Generalized Hoek-Brown			1e+06
Engineered Fill (Proposed)		125	Mohr-Coulomb	300	33	
Artificial Fill (Existing)		125	Mohr-Coulomb	50	30	

Support Name	Color	Force Application	Tensile Strength (lbs/ft)
Geogrid		Active (Method A)	3200

ENGEO
Expect Excellence

SLIDEINTERPRET 8.024

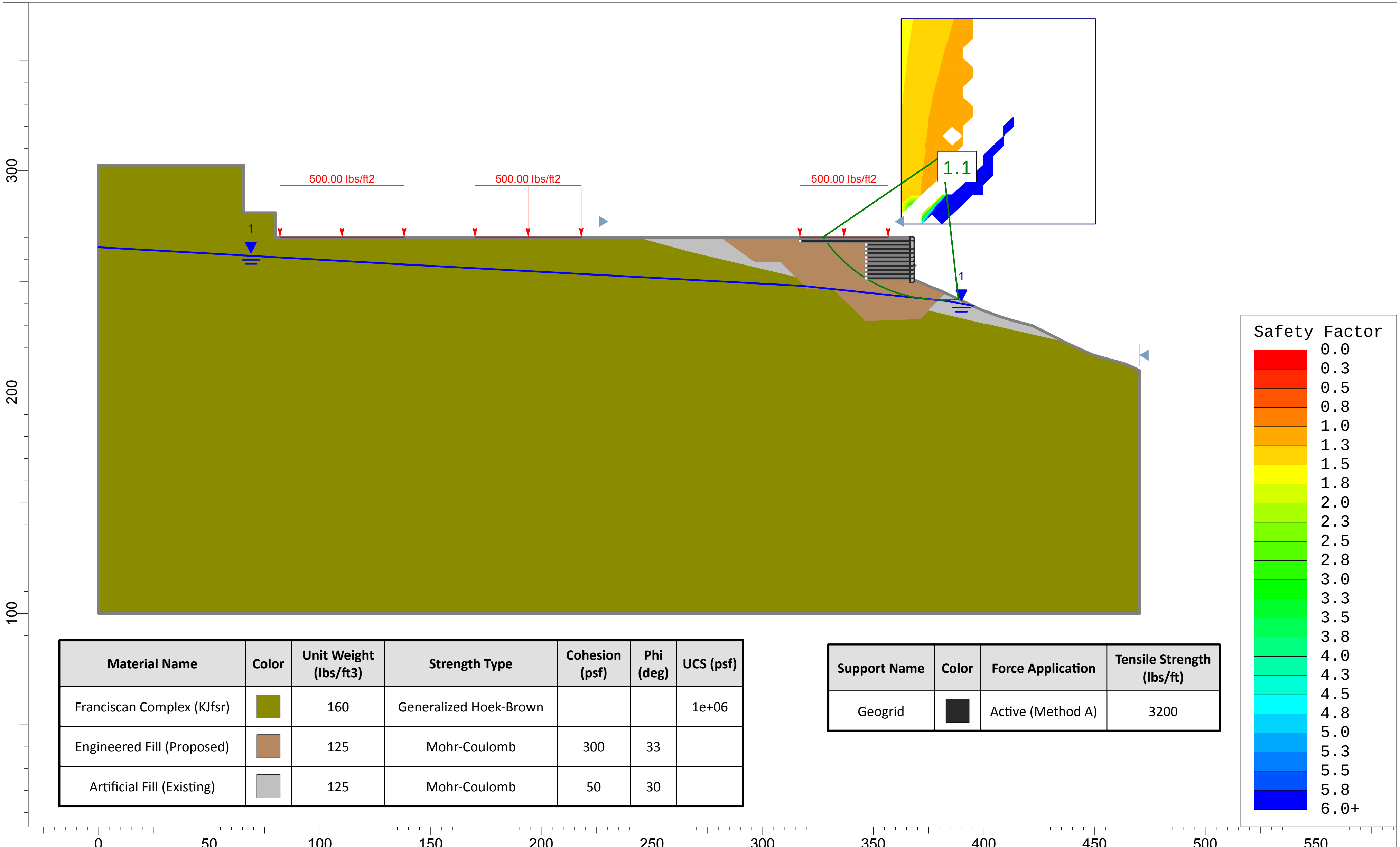
Project	Peninsula Heights - Residential Development
Slope Stability Analysis	Seismic Slope Stability - Section A-A'
Drawn By Y. Zepeda	*Scale* 1:1112
Company	ENGEO Incorporated
February 10, 2020	*File Name* 16683.000.000_Slope Stability Analysis_Revised 2020-01-17.slmd



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	UCS (psf)
Franciscan Complex (KJfsr)		160	Generalized Hoek-Brown			1e+06
Engineered Fill (Proposed)		125	Mohr-Coulomb	300	33	
Artificial Fill (Existing)		125	Mohr-Coulomb	50	30	

Support Name	Color	Force Application	Tensile Strength (lbs/ft)
Geogrid		Active (Method A)	3200

ENGEO Expect Excellence	*Project* Peninsula Heights - Residential Development	
	Slope Stability Analysis Static Slope Stability - Section D-D'	
	Drawn By Y. Zepeda	*Scale* 1:642	*Company* ENGEO Incorporated
SLIDEINTERPRET 8.024	February 10, 2020	*File Name* 16683.000.000_Slope Stability Analysis_Revised 2020-01-17.slmd



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	UCS (psf)
Franciscan Complex (KJfsr)		160	Generalized Hoek-Brown			1e+06
Engineered Fill (Proposed)		125	Mohr-Coulomb	300	33	
Artificial Fill (Existing)		125	Mohr-Coulomb	50	30	

Support Name	Color	Force Application	Tensile Strength (lbs/ft)
Geogrid		Active (Method A)	3200

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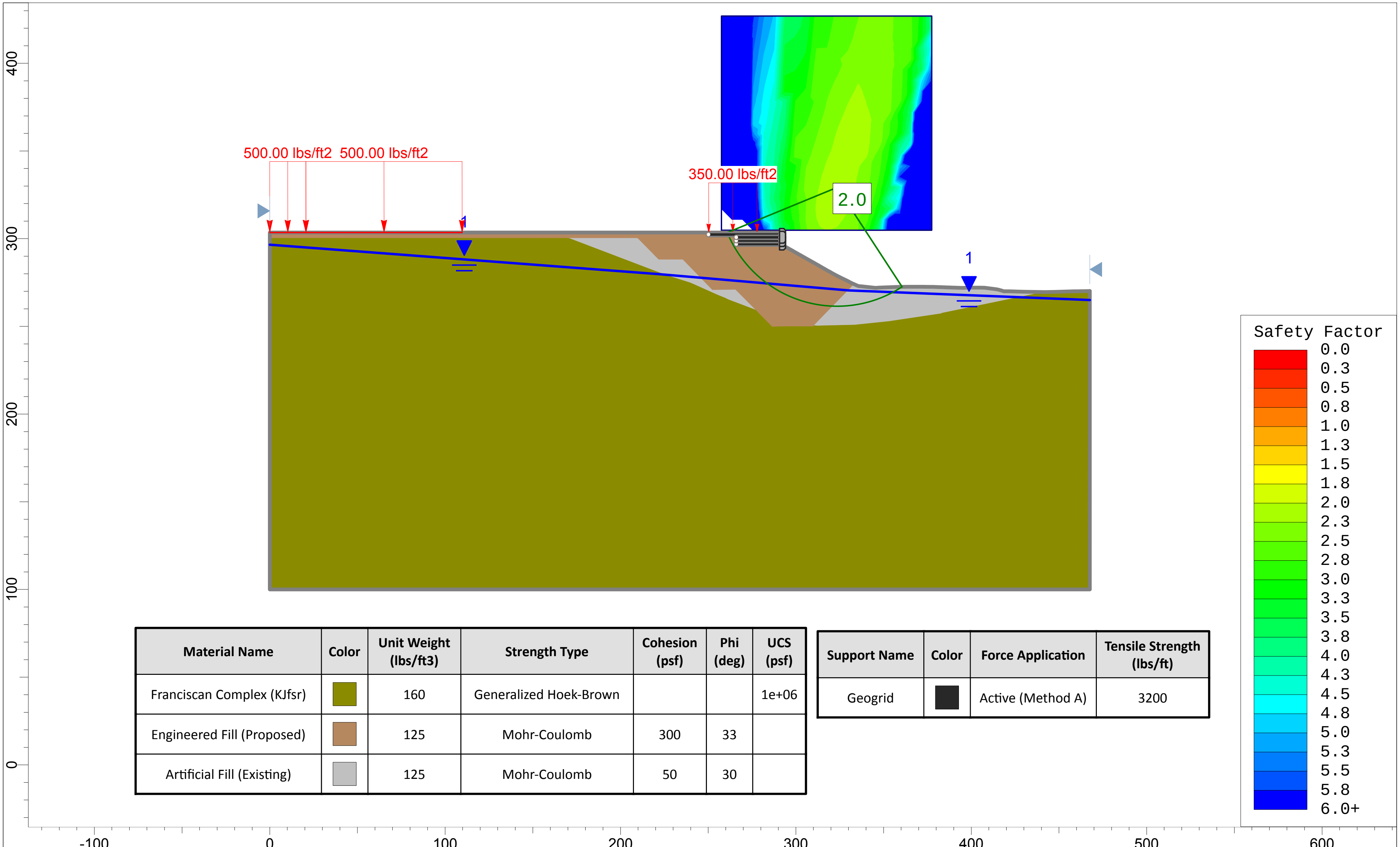
SLIDEINTERPRET 8.024

Project
Peninsula Heights - Residential Development

Slope Stability Analysis
Seismic Slope Stability - Section D-D'

Drawn By Y. Zepeda | *Scale* 1:720 | *Company* ENGEO Incorporated

February 10, 2020 | *File Name* 16683.000.000_Slope Stability Analysis_Revised 2020-01-17.slmd



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	UCS (psf)
Franciscan Complex (KJfsr)		160	Generalized Hoek-Brown			1e+06
Engineered Fill (Proposed)		125	Mohr-Coulomb	300	33	
Artificial Fill (Existing)		125	Mohr-Coulomb	50	30	

Support Name	Color	Force Application	Tensile Strength (lbs/ft)
Geogrid		Active (Method A)	3200

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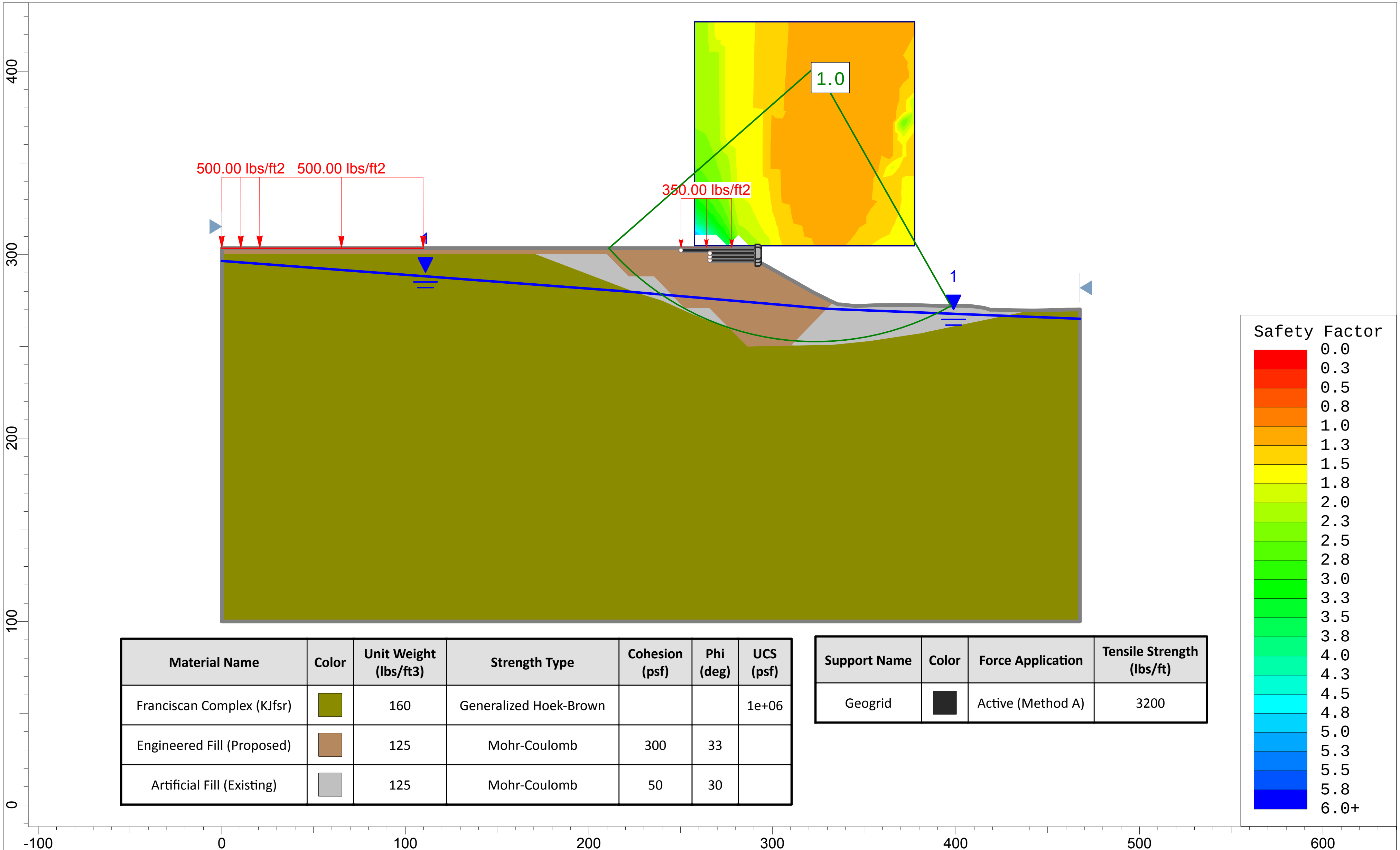
SLIDEINTERPRET 8.024

Project	Peninsula Heights - Residential Development		
Slope Stability Analysis	Static Slope Stability - Section F-F'		
Drawn By Y. Zepeda	*Scale* 1:908	*Company* ENGEO Incorporated	
February 10, 2020		*File Name* 16683.000.000_Slope Stability Analysis_Revised 2020-01-17.slmd	



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	UCS (psf)
Franciscan Complex (KJfsr)		160	Generalized Hoek-Brown			1e+06
Engineered Fill (Proposed)		125	Mohr-Coulomb	300	33	
Artificial Fill (Existing)		125	Mohr-Coulomb	50	30	

Support Name	Color	Force Application	Tensile Strength (lbs/ft)
Geogrid		Active (Method A)	3200

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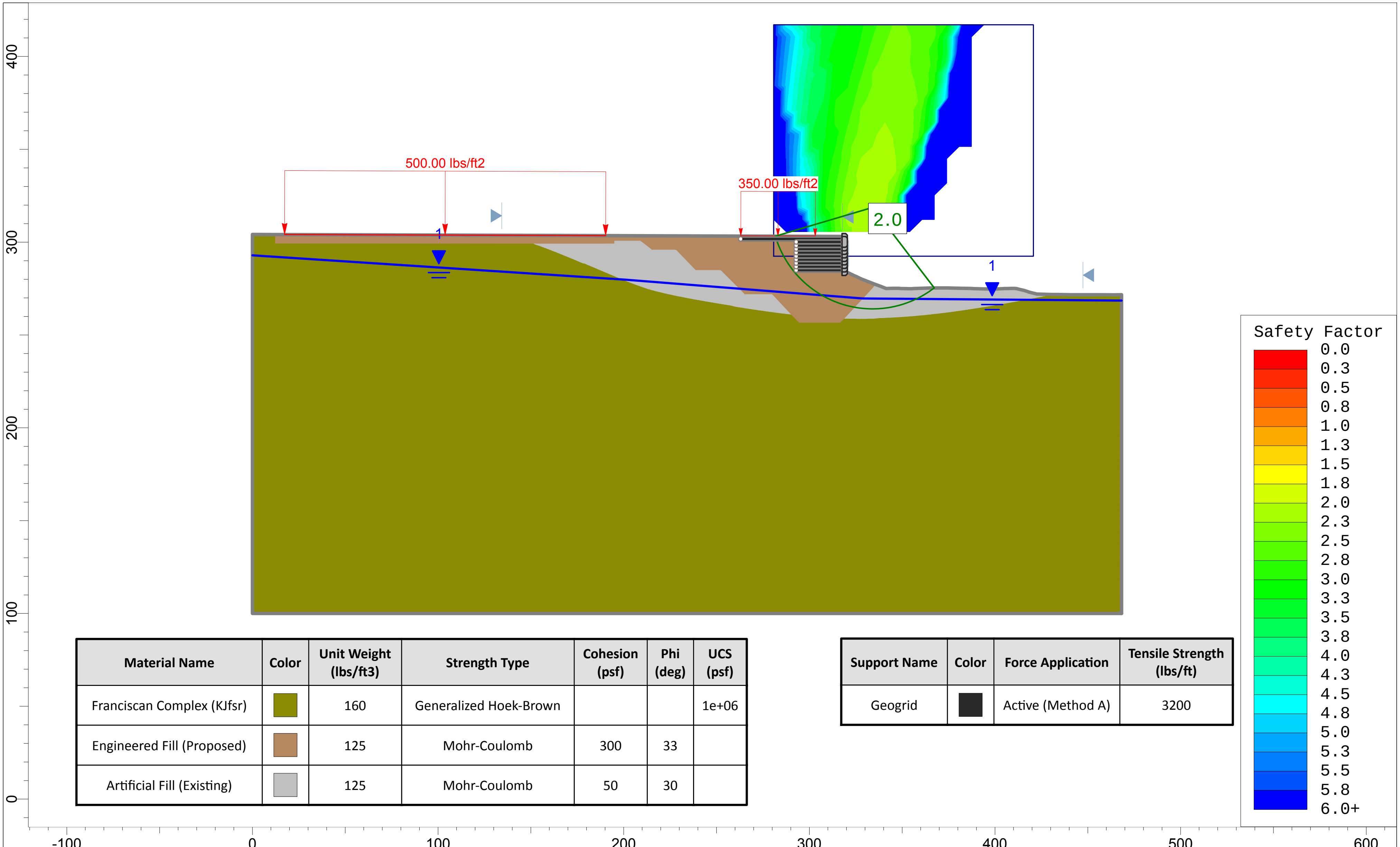
SLIDEINTERPRET 8.024

Project	Peninsula Heights - Residential Development
Slope Stability Analysis	Seismic Slope Stability - Section F-F'

Drawn By	Y. Zepeda	*Scale*	1:868	*Company*	ENGEO Incorporated
	February 10, 2020			*File Name*	16683.000.000_Slope Stability Analysis_Revised 2020-01-17.slmd



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	UCS (psf)
Franciscan Complex (KJfsr)		160	Generalized Hoek-Brown			1e+06
Engineered Fill (Proposed)		125	Mohr-Coulomb	300	33	
Artificial Fill (Existing)		125	Mohr-Coulomb	50	30	

Support Name	Color	Force Application	Tensile Strength (lbs/ft)
Geogrid		Active (Method A)	3200

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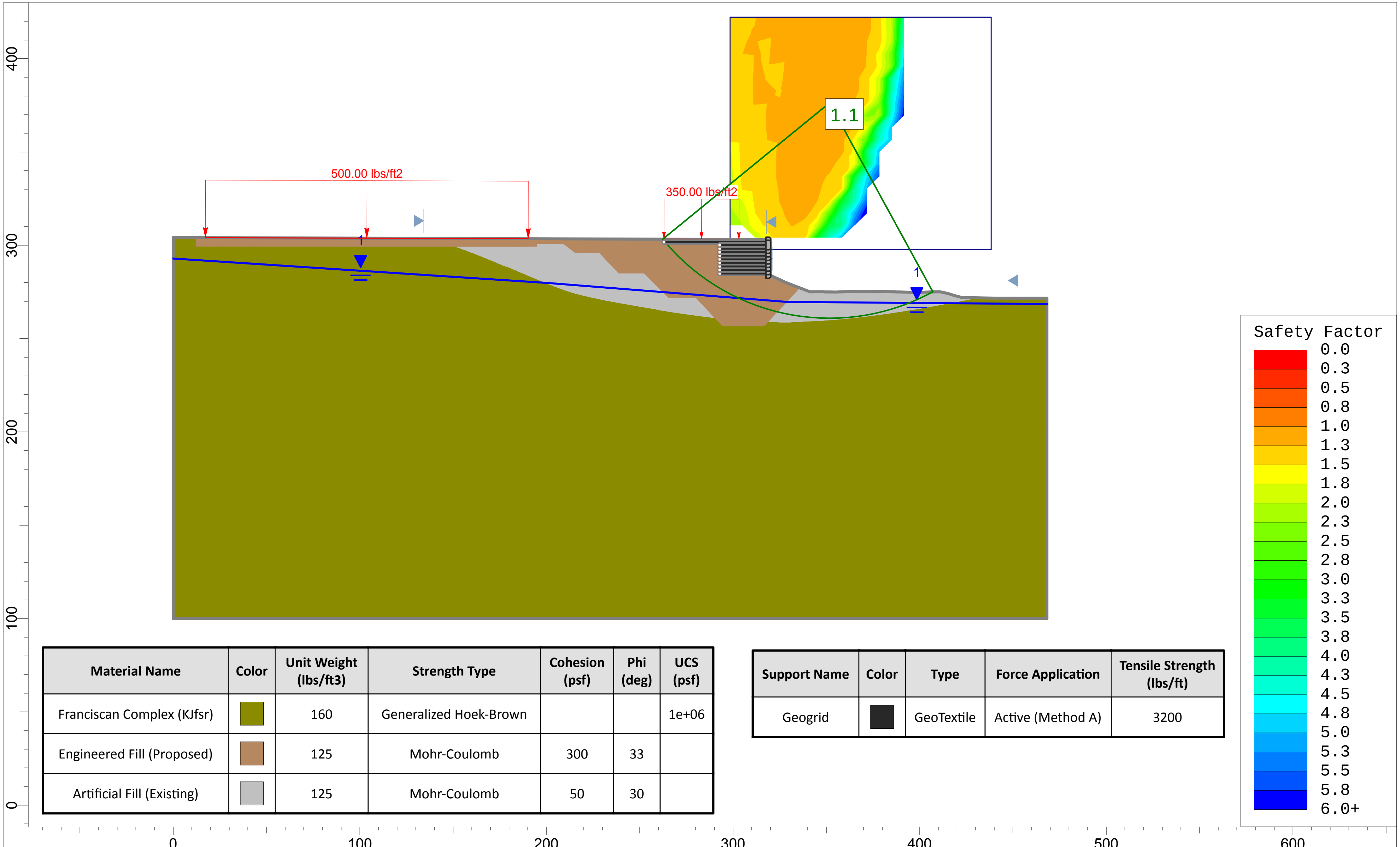
SLIDEINTERPRET 8.024

Project	Peninsula Heights - Residential Development
Slope Stability Analysis	Static Slope Stability - Section G-G'
Drawn By	Y. Zepeda
Scale	1:858
Company	ENGEO Incorporated
	February 10, 2020
File Name	16683.000.000_Slope Stability Analysis_Revised 2020-01-17.slmd



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	UCS (psf)
Franciscan Complex (KJfsr)		160	Generalized Hoek-Brown			1e+06
Engineered Fill (Proposed)		125	Mohr-Coulomb	300	33	
Artificial Fill (Existing)		125	Mohr-Coulomb	50	30	

Support Name	Color	Type	Force Application	Tensile Strength (lbs/ft)
Geogrid		GeoTextile	Active (Method A)	3200

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Project Peninsula Heights - Residential Development

Slope Stability Analysis Seismic Slope Stability - Section G-G'

Drawn By Y. Zepeda | *Scale* 1:854 | *Company* ENGEO Incorporated

February 10, 2020 | *File Name* 16683.000.000_Slope Stability Analysis_Revised 2020-01-17.slmd

SLIDEINTERPRET 8.024

APPENDIX F

SUPPLEMENTAL RECOMMENDATIONS

SUPPLEMENTAL RECOMMENDATIONS

Prepared by
ENGEO Incorporated

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TABLE OF CONTENTS

GENERAL INFORMATION

PREFACE

These supplemental recommendations are intended as a guide for earthwork and are in addition to any previous earthwork recommendations made by the Geotechnical Engineer. If there is a conflict between these supplemental recommendations and any previous recommendations, it should be immediately brought to the attention of ENGEO. Testing standards identified in this document shall be the most current revision (unless stated otherwise).

DEFINITIONS

BACKFILL	Soil, rock or soil-rock material used to fill excavations and trenches.
DRAWINGS	Documents approved for construction which describe the work.
THE GEOTECHNICAL ENGINEER	The project geotechnical engineering consulting firm, its employees, or its designated representatives.
ENGINEERED FILL	Fill upon which the Geotechnical Engineer has made sufficient observations and tests to confirm that the fill has been placed and compacted in accordance with geotechnical engineering recommendations.
FILL	Soil, rock, or soil-rock materials placed to raise the grades of the site or to backfill excavations.
IMPORTED MATERIAL	Soil and/or rock material which is brought to the site from offsite areas.
ONSITE MATERIAL	Soil and/or rock material which is obtained from the site.
OPTIMUM MOISTURE	Water content, percentage by dry weight, corresponding to the maximum dry density as determined by ASTM D-1557.
RELATIVE COMPACTION	The ratio, expressed as a percentage, of the in-place dry density of the fill or backfill material as compacted in the field to the maximum dry density of the same material as determined by ASTM D-1557.
SELECT MATERIAL	Onsite and/or imported material which is approved by the Geotechnical Engineer as a specific-purpose fill.

PART I - EARTHWORK

1.0 GENERAL

1.1 WORK COVERED

Supplemental recommendations for performing earthwork and grading. Activities include:

- ✓ Site Preparation and Demolition
- ✓ Excavation
- ✓ Grading
- ✓ Backfill of Excavations and Trenches
- ✓ Engineered Fill Placement, Moisture Conditioning, and Compaction

1.2 CODES AND STANDARDS

The contractor should perform their work complying with applicable occupational safety and health standards, rules, regulations, and orders. The Occupational Safety and Health Standards (OSHA) Board is the only agency authorized in the State to adopt and enforce occupational safety and health standards (Labor Code § 142 et seq.). The owner, their representative and contractor are responsible for site safety; ENGEO representatives are not responsible for site safety.

Excavating, trenching, filling, backfilling, shoring and grading work should meet the minimum requirements of the applicable Building Code, and the standards and ordinances of state and local governing authorities.

1.3 TESTING AND OBSERVATION

Site preparation, cutting and shaping, excavating, filling, and backfilling should be carried out under the testing and observation of ENGEO. ENGEO shall be retained to perform appropriate field and laboratory tests to check compliance with the recommendations. Any fill or backfill that does not meet the supplemental recommendations shall be removed and/or reworked, until the supplemental recommendations are satisfied.

Tests for compaction shall be made in accordance with test procedures outlined in ASTM D-1557, as applicable, unless other testing methods are deemed appropriate by ENGEO. These and other tests shall be performed in accordance with accepted testing procedures, subject to the engineering discretion of ENGEO.

2.0 MATERIALS

2.1 STANDARD

Materials, tools, equipment, facilities, and services as required for performing the required excavating, trenching, filling and backfilling should be furnished by the Contractor.

2.2 ENGINEERED FILL AND BACKFILL

Material to be used for engineered fill and backfill should be free from organic matter and other deleterious substances, and of such quality that it will compact thoroughly without excessive voids when watered and rolled.

Unless specified elsewhere by ENGEO, engineered fill and backfill shall be free of significant organics, or any other unsatisfactory material. In addition, engineered fill and backfill shall comply with the grading requirements shown in the following table:

TABLE 2.2-1: Engineered Fill and Backfill Requirements

US STANDARD SIEVE	PERCENTAGE PASSING
3"	100
No. 4	35–100
No. 30	20–100

Earth materials to be used as engineered fill and backfill shall be cleared of debris, rubble and deleterious matter. Rocks and aggregate exceeding the maximum allowable size shall be removed from the site. Rocks of maximum dimension in excess of two-thirds of the lift thickness shall be removed from any fill material to the satisfaction of ENGEO.

ENGEO shall be immediately notified if potential hazardous materials or suspect soils exhibiting staining or odor are encountered. Work activities shall be discontinued within the area of potentially hazardous materials. ENGEO shall be notified at least 72 hours prior to the start of filling and backfilling operations. Materials to be used for filling and backfilling shall be submitted to ENGEO no less than 10 days prior to intended delivery to the site. Unless specified elsewhere by ENGEO, where conditions require the importation of low expansive fill material, the material shall be an inert, low to non-expansive soil, or soil-rock material, free of organic matter and meeting the following requirements:

TABLE 2.2-2: Imported Fill Material Requirements

	SIEVE SIZE	PERCENT PASSING
GRADATION (ASTM D-421)	2-inch	100
	#200	15 - 70
PLASTICITY (ASTM D-4318)	Plasticity Index < 12	
ORGANIC CONTENT (ASTM D-2974)	Less than 3 percent	

A sample of the proposed import material should be submitted to ENGEO no less than 10 days prior to intended delivery to the site.

2.3 SUBDRAINS

A subdrain system is an underground network of piping used to remove water from areas that collect or retain surface water or subsurface water. Subsurface water is collected by allowing water into the pipe through perforations. Subdrain systems may drain and discharge to an appropriate outlet such as storm drain, natural swales or drainage, etc.. Details for subdrain systems may vary depending on many items, including but not limited to site conditions, soil types, subdrain spacing, depth of the pipe and pervious medium, as well as pipe diameter.

2.4 PIPE

Subdrain pipe shall conform with these supplemental recommendations unless specified elsewhere by ENGEO. Perforated pipe for various depths shall be manufactured in accordance with the following requirements:

TABLE 2.4-1: Perforated Pipe Requirements

PIPE TYPE	STANDARD	TYPICAL SIZES (INCHES)	PIPE STIFFNESS (PSI)
PIPE STIFFNESS ABOVE 200 PSI (BELOW 50 FEET OF FINISHED GRADE)			
ABS SDR 15.3		4 to 6	450
PVC Schedule 80	ASTM D1785	3 to 10	530
PIPE STIFFNESS BETWEEN 100 PSI AND 150 PSI (BETWEEN 15 AND 50 FEET OF FINISHED GRADE)			
ABS SDR 23.5	ASTM D2751	4 to 6	150
PVC SDR 23.5	ASTM D3034	4 to 6	153
PVC Schedule 40	ASTM D1785	3 to 10	135
ABS Schedule 40/DWV	ASTM D1527 & D2661	3 to 10	
PIPE STIFFNESS BETWEEN 45 PSI AND 50 PSI* (BETWEEN 0 TO 15 FEET OF FINISHED GRADE)			
PVC A-2000	ASTM F949	4 to 10	50
PVC SDR 35	ASTM D3034	4 to 8	46
ABS SDR 35	ASTM D2751	4 to 8	45
Corrugated PE	AASHTO M294 Type S	4 to 10	45

***Pipe with a stiffness less than 45 psi should not be used.**

Other pipes not listed in the table above shall be submitted for review by the Geotechnical Engineer not less 72 hours before proposed use.

2.5 OUTLETS AND RISERS

Subdrain outlets and risers must be fabricated from the same material as the subdrain pipe. Outlet and riser pipe and fittings must not be perforated. Covers must be fitted and bolted into the riser pipe or elbow. Covers must seat uniformly and not be subject to rocking.

2.6 PERMEABLE MATERIAL

Permeable material shall generally conform to Caltrans Standard Specification unless specified otherwise by ENGEO. Class 2 permeable material shall comply with the gradation requirements shown in the following table.

TABLE 2.6-1: Class 2 Permeable Material Grading Requirements

SIEVE SIZES	PERCENTAGE PASSING
1"	100
3/4"	90 to 100
3/8"	40 to 100
No. 4	25 to 40
No. 8	18 to 33
No. 30	5 to 15
No. 50	0 to 7
No. 200	0 to 3

2.7 FILTER FABRIC

Filter fabric shall meet the following Minimum Average Roll Values unless specified elsewhere by ENGEO.

Grab Strength (ASTM D-4632) .... 180 lbs
Mass per Unit Area (ASTM D-4751) ....6 oz/yd^2
Apparent Opening Size (ASTM D-4751) ....70-100 U.S. Std. Sieve
Flow Rate (ASTM D-4491)... 80 gal/min/ft^2
Puncture Strength (ASTM D-4833) .... 80 lbs

Areas to receive filter fabric must comply with the compaction and elevation tolerance specified for the material involved. Handle and place filter fabric under the manufacturer's instructions. Align and place filter fabric without wrinkles.

Overlap adjacent roll ends of filter fabric in accordance with manufacturer's recommendations. The preceding roll must overlap the following roll in the direction that the permeable material is being spread. Completely replace torn or punctured sections damaged during placement or repair by placing a piece of filter fabric that is large enough to cover the damaged area and comply with the overlap specified. Cover filter fabric with the thickness of overlying material shown within 72 hours of placing the fabric.

2.8 GEOCOMPOSITE DRAINAGE

Geocomposite drainage is a prefabricated material that includes filter fabric and plastic pipe. Filter fabric must be Class A. The drain shall be of composite construction consisting of a supporting structure or drainage core material surrounded by a geotextile. The geotextile shall encapsulate the drainage core and prevent random soil intrusion into the drainage structure. The drainage core material shall consist of a three-dimensional polymeric material with a structure that permits flow along the core laterally. The core structure shall also be constructed to permit flow regardless of the water inlet surface. The drainage core shall provide support to the geotextile.

A geotextile flap shall be provided along drainage core edges. This flap shall be of sufficient width for sealing the geotextile to the adjacent drainage structure edge to prevent soil intrusion into the structure during and after installation. The geotextile shall cover the full length of the core. The geocomposite core shall be furnished with an approved method of constructing and connecting with outlet pipes. If the fabric on the geocomposite drain is torn or punctured, replace the damaged section completely. The specific drainage composite material and supplier shall be preapproved by ENGEO.

The Contractor shall submit a manufacturer's certification that the geocomposite meets the design properties and respective index criteria measured in full accordance with applicable test methods. The manufacturer's certification shall include a submittal package of documented test results that confirm the design values. In case of dispute over validity of design values, the Contractor will supply design property test data from a laboratory approved by ENGEO, to support the certified values submitted.

Geocomposite material suppliers shall provide a qualified and experienced representative onsite to assist the Contractor and ENGEO at the start of construction with directions on the use of drainage composite. If there is more than one application on a project, this criterion will apply to construction of the initial application only. The representative shall also be available on an as-needed basis, as requested by ENGEO, during construction of the remaining applications. The soil surface against which the geocomposite is to be placed shall be free of debris and inordinate irregularities that will prevent intimate contact between the soil surface and the drain.

Edge seams shall be formed by utilizing the flap of the geotextile extending from the geocomposite's edge and lapping over the top of the fabric of the adjacent course. The fabric flap shall be securely fastened to the adjacent fabric by means of plastic tape or non-water-soluble construction adhesive, as recommended by the supplier. To prevent soil intrusion, exposed edges of the geocomposite drainage core edge must be covered.

Approved backfill shall be placed immediately over the geocomposite drain. Backfill operations should be performed to not damage the geotextile surface of the drain. Also during operations, avoid excessive settlement of the backfill material. The geocomposite drain, once installed, shall not be exposed for more than 7 days prior to backfilling.

PART II - GEOGRID SOIL REINFORCEMENT

Geogrid soil reinforcement (geogrid) shall be submitted to ENGEO and should be approved before use. The geogrid shall be a regular network of integrally connected polymer tensile elements with aperture geometry sufficient to permit significant mechanical interlock with the surrounding soil or rock. The geogrid structure shall be dimensionally stable and able to retain its geometry under construction stresses and shall have high resistance to damage during construction to ultraviolet degradation and to chemical and biological degradation encountered in the soil being reinforced. The geogrids shall have an Allowable Tensile Strength (T_a) and Pullout Resistance, for the soil type(s) as specified on design plans.

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

Geogrid material suppliers shall provide a qualified and experienced representative onsite at the initiation of the project, for a minimum of three days, to assist the Contractor and ENGEO personnel at the start of construction. If there is more than one slope on a project, this criterion will apply to construction of the initial slope only. The representative shall also be available on an as-needed basis, as requested by ENGEO, during construction of the remaining slope(s). Geogrid reinforcement may be joined with mechanical connections or overlaps as recommended and approved by the manufacturer. Joints shall not be placed within 6 feet of the slope face, within 4 feet below top of slope, nor horizontally or vertically adjacent to another joint.

The geogrid reinforcement shall be installed in accordance with the manufacturer's recommendations. The geogrid reinforcement shall be placed within the layers of the compacted soil as shown on the plans or as directed. The geogrid reinforcement shall be placed in continuous longitudinal strips in the direction of main reinforcement. However, if the Contractor is unable to complete a required length with a single continuous length of geogrid, a joint may be made with the manufacturer's approval. Only one joint per length of geogrid shall be allowed. This joint shall be made for the full width of the strip by using a similar material with similar strength. Joints in geogrid reinforcement shall be pulled and held taut during fill placement.

Adjacent strips, in the case of 100 percent coverage in plan view, need not be overlapped. The minimum horizontal coverage is 50 percent, with horizontal spacing between reinforcement no greater than 40 inches. Horizontal coverage of less than 100 percent shall not be allowed unless specifically detailed in the construction drawings. Adjacent rolls of geogrid reinforcement shall be overlapped or mechanically connected where exposed in a wrap around face system, as applicable.

The Contractor may place only that amount of geogrid reinforcement required for immediately pending work to prevent undue damage. After a layer of geogrid reinforcement has been placed, the next succeeding layer of soil shall be placed and compacted as appropriate. After the specified soil layer has been placed, the next geogrid reinforcement layer shall be installed. The process shall be repeated for each subsequent layer of geogrid reinforcement and soil. Geogrid reinforcement shall be placed to lay flat and pulled tight prior to backfilling. After a layer of geogrid reinforcement has been placed, suitable means, such as pins or small piles of soil, shall be used to hold the geogrid reinforcement in position until the subsequent soil layer can be placed.

Under no circumstances shall a track-type vehicle be allowed on the geogrid reinforcement before at least 6 inches of soil have been placed. Turning of tracked vehicles should be kept to a minimum to prevent tracks from displacing the fill and the geogrid reinforcement. If approved by the Manufacturer, rubber-tired equipment may pass over the geosynthetic reinforcement at slow speeds, less than 10 mph. Sudden braking and sharp turning shall be avoided. During construction, the surface of the fill should be kept approximately horizontal. Geogrid reinforcement shall be placed directly on the compacted horizontal fill surface. Geogrid reinforcements are to be placed as shown on plans, and oriented correctly.

PART III - GEOTEXTILE SOIL REINFORCEMENT

The specific geotextile material and supplier shall be preapproved by ENGEO. The contractor shall submit a manufacturer's certification that the geotextiles supplied meet the respective index criteria set when geotextile was approved by ENGEO, measured in full accordance with specified test methods and standards.

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

Geotextile material suppliers shall provide a qualified and experienced representative onsite at the initiation of the project to assist the Contractor and ENGEO personnel at the start of construction. The geotextile reinforcement shall be installed in accordance with the manufacturer's recommendations. The geotextile reinforcement shall be placed within the layers of the compacted soil as shown on the plans or as directed, secured with staples, pins, or small piles of backfill, placed without wrinkles, and aligned with the primary strength direction perpendicular to slope contours. Cover geotextile reinforcement with backfill within the same work shift. Place at least 6 inches of backfill on the geotextile reinforcement before operating or driving equipment or vehicles over it, except those used under the conditions specified below for spreading backfill.

Adjacent strips, in the case of 100 percent coverage in plan view, need not be overlapped. The minimum horizontal coverage is 50 percent, with horizontal spacing between reinforcement no greater than 40 inches. Horizontal coverage of less than 100 percent shall not be allowed unless specifically detailed in the construction drawings. Adjacent rolls of geotextile reinforcement shall be overlapped or mechanically connected where exposed in a wraparound face system, as applicable.

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

Geotextile reinforcement shall be placed to lay flat and be pulled tight prior to backfilling. After a layer of geotextile reinforcement has been placed, suitable means, such as pins or small piles of soil, shall be used to hold the geotextile reinforcement in position until the subsequent soil layer can be placed. Under no circumstances shall a track-type vehicle be allowed on the geotextile reinforcement before at least six inches of soil has been placed. Turning of tracked vehicles should be kept to a minimum to prevent tracks from displacing the fill and the geotextile reinforcement. If approved by the Manufacturer, rubber-tired equipment may pass over the geotextile reinforcement as slow speeds, less than 10 mph. Sudden braking and sharp turning shall be avoided.

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

Replace or repair any geotextile reinforcement damaged during construction. Grade and compact backfill to ensure the reinforcement remains taut. Geotextile soil reinforcement must be tested to the required design values using the following ASTM test methods.

TABLE III-1: Geotextile Soil Reinforcements

PROPERTY	TEST
Elongation at break, percent	ASTM D 4632
Grab breaking load, lb, 1-inch grip (min) in each direction	ASTM D 4632
Wide width tensile strength at 5 percent strain, lb/ft (min)	ASTM D 4595
Wide width tensile strength at ultimate strength, lb/ft (min)	ASTM D 4595
Tear strength, lb (min)	ASTM D 4533
Puncture strength, lb (min)	ASTM D 6241
Permittivity, sec^{-1} (min)	ASTM D 4491
Apparent opening size, inches (max)	ASTM D 4751
Ultraviolet resistance, percent (min) retained grab break load, 500 hours	ASTM D 4355

PART IV - EROSION CONTROL MAT

Work shall consist of furnishing and placing a synthetic erosion control mat and/or degradable erosion control blanket for slope face protection and lining of runoff channels. The specific erosion control material and supplier shall be pre-approved by ENGEO.

The Contractor shall submit a manufacturer's certification that the erosion mat/blanket supplied meets the criteria specified when the material was approved by ENGEO. The manufacturer's certification shall include a submittal package of documented test results that confirm the property values. Jute mesh shall consist of processed natural jute yarns woven into a matrix, and netting shall consist of coconut fiber woven into a matrix. Erosion control blankets shall be made of processed natural fibers that are mechanically, structurally, or chemically bound together to form a continuous matrix that is surrounded by two natural nets.

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

Erosion control material suppliers shall provide a qualified and experienced representative onsite, to assist the Contractor and ENGEO personnel at the start of construction. If there is more than one slope on a project, this criterion will apply to construction of the initial slope only. The representative shall be available on an as-needed basis, as requested by ENGEO, during construction of the remaining slope(s). The erosion control material shall be placed and anchored on a smooth graded, firm surface approved by the Engineer. Anchoring terminal ends of the erosion control material shall be accomplished through use of key trenches. The material in the trenches shall be anchored to the soil on maximum 1½ foot centers. Topsoil, if required by construction drawings, placed over final grade prior to installation of the erosion control material shall be limited to a depth not exceeding 3 inches.

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

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