

**GEOTECHNICAL DESIGN REPORT**

**SLIP-OUT REPAIR NEAR 1780 HIGGINS CANYON ROAD**

**CE&G DOCUMENT: 0207545.001**

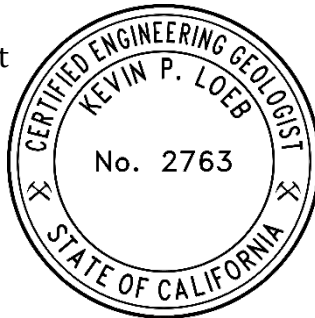
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## **1.0 INTRODUCTION**

### **1.1 GENERAL**

Cal Engineering & Geology, Inc. (CE&G), a division of Haley & Aldrich, has provided geotechnical engineering services for the slip-out repair near 1780 Higgins Canyon Road in Half Moon Bay, California. The work has been completed to provide recommended repair alternatives to stabilize the slope and restore the cross-sectional width of Higgins Canyon Road for the County of San Mateo (County).

### **1.2 PROJECT DESCRIPTION**

Due to heavy rain in late December 2022 through early January 2023, a landslide occurred near 1780 Higgins Canyon Road in unincorporated San Mateo County near Half Moon Bay, California (Figure 1). An adjacent emergency Rock Slope Protection (RSP) slope repair immediately east of the subject landslide, completed in April 2022 by San Mateo County, has also eroded due to recent rain events. These two failures have combined into one failure with a width of approximately 115 feet and extend downslope from the road surface approximately 28 feet. It is assumed that the landslides were due to the saturated slope materials in combination with erosion from elevated creek flows. The landslides have encroached onto Higgins Canyon Road, causing a roadside slip-out on the outboard shoulder. Photos of the project site and slip-out can be found in Appendix A. We understand that San Mateo County intends to restore the road and creek bank but has not identified a preferred repair option.

### **1.3 PURPOSE AND SCOPE OF SERVICES**

The investigation completed by CE&G was undertaken to assess the existing surface and subsurface conditions in the immediate vicinity of the proposed project and to develop geotechnical design recommendations for the proposed improvements.

The scope of work completed for the geotechnical investigation and report included the following:

1. Meetings and consultation with San Mateo County personnel and management of geotechnical explorations.
2. Performance of a design-level aerial LiDAR scan to develop a topographic base map utilizing an unmanned aerial system.

3. Completion of a desktop study to identify and evaluate relevant geologic and geotechnical information available for the site, including published geologic maps and unpublished geotechnical information in our files regarding the site and vicinity.
4. Geologic reconnaissance to observe and map current site conditions and to mark for USA (Underground Service Alert).
5. Subsurface exploration using a truck-mounted drill rig in accordance with a drilling permit facilitated by the County.
6. Laboratory testing to determine key engineering index properties of selected earth materials.
7. Engineering analysis to develop and evaluate alternative geotechnical approaches to restore the roadway embankment and develop parameters for the repair design.
8. Preparation of this geotechnical design report.

## **2.0 SITE DESCRIPTION**

The project site is located near 1780 Higgins Canyon Road in unincorporated San Mateo County near Half Moon Bay, California, on the western foothills of the Santa Cruz Mountains. The site is just east of a bridge that crosses Mills Creek. Higgins Canyon Road is a narrow two-lane paved roadway that is about 18 to 21 feet in width in the site vicinity and trends northwest to southeast. The subject slope sits on a southern creek bank along Mills Creek and is moderately vegetated with small to large trees and shrubs. Mills Creek at the project location is slightly horseshoe-shaped and comes downstream from the northeast and bends to the northwest at our project location, which is assumed to be contributing to the erosion at the toe of the subject slope. The road elevation ranges approximately between 200 and 202 feet above the main sea level. The creek bottom is approximately 174 feet above the main sea level.

Detailed descriptions of the site and road distress features are further described in the site reconnaissance section of this report (Section 4.1). Key site features are shown in Figure 2.

## 3.0 GEOLOGY

### 3.1 GEOLOGIC SETTING

The project site lies within the Coast Ranges geomorphic province of California. This province is characterized by northwest-southeast trending mountain ranges and intervening valleys, such as that occupied by San Francisco Bay and the Santa Clara Valley. The right-lateral strike-slip San Andreas fault system controls the northwest-southeast structural grain of the Coast Ranges and the Bay Area. The San Andreas fault system includes the Hayward-Rodgers Creek, Calaveras, Concord-Green Valley, and Greenville-Marsh Creek faults, among others, which have resulted in the uplift of the northwest-trending Diablo and Santa Cruz Mountain Ranges. The Santa Cruz Mountain Range makes up the majority of the San Francisco Peninsula, which is bounded by San Francisco Bay to the east and the Pacific Ocean to the west. The project site is located in the westernmost foothills of the Santa Cruz Mountain Range, approximately 2.9 miles inland from the Pacific Coast.

### 3.2 REGIONAL SETTING

The geologic setting is shown on the Regional Geology Map, Figure 3.

The general vicinity of the project site has been mapped several times, with geologic mapping having different emphases: Brabb and others (1998); Knudsen and others (2000); Graymer and others (2006); and Witter and others (2006).

According to Brabb and others (1998), the majority of the project site is mapped as being underlain by Holocene-aged alluvium, which is described as "unconsolidated gravel, sand, silt, and clay along streams." The east and west sides of the site are mapped as being underlain by Pleistocene-aged coarse-grained older alluvial fan and stream terrace deposits that generally consist of poorly consolidated gravel, sand, and silt (Brabb and others, 1998). Later mapping by Graymer and others (2006) agrees with mapping by Brabb and others (1998).

Bedrock consisting of Purisima Formation (Pliocene and Upper Miocene) likely underlies the above-described alluvial soils (Brabb and others, 1998). The Purisima Formation is generally described as gray and greenish-gray fine-grained sandstone, siltstone, and mudstone, but also includes some porcelaneous shale and mudstone, chert, silty mudstone, and volcanic ash (Brabb and others, 1998). Purisima Formation sedimentary beds locally strike northwest and dip to the southwest at approximately 41° (Brabb and others, 1998).

### 3.3 SEISMICITY

The project site is located within the greater San Francisco Bay Area, which is recognized as one of the more seismically active regions of California. The seismic activity in this region results from the complex movements along the transform boundary between the Pacific Plate and the North American Plate. Along this transform boundary, the Pacific Plate is slowly moving to the northwest relative to the more stable North American Plate at approximately 40 mm/yr in the Bay Area (Page, 1992). The differential movements between the two crustal plates caused the formation of a series of active fault systems within the transform boundary. The transform boundary between the two plates extends across a broad zone of the North American Plate, within which right-lateral strike-slip faulting predominates. In this broad transform boundary, the San Andreas fault accommodates less than half of the average total relative plate motion. Much of the remainder of the motion in the South Bay Area is distributed across faults such as the San Gregorio, Monte Vista-Shannon, Sargent, Hayward, Calaveras, Greenville, and Zayante-Vergeles fault zones.

Due to the site's location in the seismically active San Francisco Bay Area, it will likely experience strong ground shaking from a large (Moment Magnitude [Mw] 6.7) or greater earthquake along with one or more of the nearby active faults during the design lifetime of the project (WGCEP, 2003). It should be noted that the third Uniform California Earthquake Rupture Forecast (UCERF3) time-independent model supports a magnitude-dependent methodology that accounts for historic open intervals on faults without a date of last event constraint. The exact factors influencing differences between UCERF2 and UCERF3 vary throughout the region and depend on the evaluation of specific seismogenic sources. For example, with the 30 yr  $M \geq 6.7$  probabilities, the most significant changes from UCERF2 are a threefold increase on the Calaveras fault and a threefold decrease on the San Jacinto fault. The model also suggests that the average time between 6.7 Mw or larger events has increased. The UCERF3 model indicates that  $M \geq 6.7$  probabilities may not be representative of other hazard or loss measures, and the applicability of UCERF3 should be evaluated on a case-by-case basis if required during site-specific ground motion analyses or at the behest of the regulatory agencies (WGCEP, 2014).

Some contributors to seismic risk for the project include the Monte Vista/Shannon, San Andreas, Hayward, Calaveras, Sargent, Zayante-Vergeles, Greenville, and San Gregorio-Hosgri faults. A large-magnitude earthquake on any of these fault systems has the potential to cause significant ground shaking in the vicinity of the planned improvements. The intensity of ground shaking that is likely to occur in the area is generally dependent upon the magnitude of the earthquake and the distance to the epicenter.

Some relevant seismic sources in the San Francisco Bay area and their distances from the site are summarized in Table 3-1.

**Table 3-1. Distances to Selected Major Active Fault Surface Traces**

<b>Fault Name</b>	<b>Distance and Direction from Site to Surface Fault Traces</b>
San Gregorio	4.4 km southwest
Pilarcitos	4.7 km northeast
San Andreas	7.6 km northeast
Monte Vista-Shannon	18 km southeast
Hayward (southern segment)	38 km northeast
Calaveras	49 km northeast

### **3.4 GEOHAZARD MAPPING**

#### **3.4.1 Active Faults**

According to CGS (2018), a Holocene-active fault is defined as a fault that has had surface displacement within Holocene time (the last 11,700 years), and a pre-Holocene fault is defined as a fault whose recency of past movement is older than 11,700 years. The Alquist-Priolo Earthquake Fault Zoning Act only addresses the hazard of surface fault rupture for Holocene-active faults, although pre-Holocene-active faults may also have the potential for future surface fault rupture (CGS, 2018). The Alquist-Priolo Earthquake Fault Zoning Act's main purpose is to prevent the construction of buildings used for human occupancy on the surface trace of active faults. Before a new project is permitted, cities and counties require a geologic investigation to demonstrate that proposed buildings will not be constructed on active faults. According to CGS (2021), the site is not located within an Alquist-Priolo Earthquake Fault Zone.

According to the USGS Quaternary Fault and Fold Database (2017), there are no active faults mapped as crossing the project site (Figure 4).

#### **3.4.2 Liquefaction Hazards**

Soil liquefaction is a phenomenon in which saturated granular soils, and certain fine-grained soils, lose their strength due to the buildup of excess pore water pressure during cyclic loading, such as that induced by earthquakes. Soils most susceptible to liquefaction are saturated, clean, loose, fine-grained sands and non-plastic silts. Certain gravels, plastic silts, and clays are also susceptible to liquefaction. The primary factors affecting soil liquefaction include: 1) intensity and duration of seismic shaking; 2) soil type; 3) relative

density of granular soils; 4) moisture content and plasticity of fine-grained soils; 5) overburden pressure; and 6) depth to groundwater.

Witter and others (2006) have generated a map showing liquefaction susceptibility for the San Francisco Bay Area with a 5-class scale that includes very low (essentially in bedrock areas), low, moderate, high, and very high liquefaction susceptibility classes. Witter and others (2006) map the soil underlying the project site as having very low to low liquefaction susceptibility (Figure 5).

According to the Earthquake Zones of Required Investigation for the Half Moon Bay 7.5 Minute Quadrangle, the Mills Creek channel in the project area is within a liquefaction hazard zone. Higgins Canyon Road itself is not mapped within a liquefaction hazard zone.

According to the San Mateo County Hazard Mapping Tool, the project site is located within a very low to low susceptibility liquefaction zone.

### **3.4.3 Landslide Hazards**

A preliminary inventory map showing deep-seated landslides in the Half Moon Bay quadrangle was prepared by Brabb and others (2000) and does not show a mapped landslide within the project area.

According to the Earthquake Zones of Required Investigation for the Half Moon Bay 7.5 Minute Quadrangle, the project site is not within an area mapped as an earthquake-induced landslide hazard zone.

According to the San Mateo County Hazard Map, the project site is located within an area having high landslide susceptibility.

## **3.5 REGIONAL GROUNDWATER**

The California Department of Water Resources identifies the site as lying within the Half Moon Bay Terrace groundwater basin.

A map prepared by CGS (2021) showing the depth of historically high groundwater levels for the Half Moon Bay 7.5-minute quadrangle shows historic high groundwater levels of less than 10 feet along Mills Creek.

Groundwater within the site's hillslope areas is likely variable, with the water table commonly sloping downhill toward the closest drainage axis.

Site-specific groundwater data from our investigation is discussed in Section 4.3.3.

## 4.0 SITE INVESTIGATION

### 4.1 SITE RECONNAISSANCE

CE&G performed field reconnaissance of the site on December 22<sup>nd</sup>, 2022, and January 19<sup>th</sup> and March 6<sup>th</sup>, 2023, before and after drilling exploratory borings. The reconnaissance consisted of visually identifying key geologic and geomorphic features, photographic documentation of the project site, determining site access for drilling equipment, and identifying and marking boring locations for clearance by Underground Service Alert (USA). A private utility locator (GeoTech Utility Locating) was used to clear the exploration locations of existing utilities.

### 4.2 LIDAR SCAN AND SITE RECONNAISSANCE

A design-level aerial light detection and ranging (LiDAR) scan of the site was performed on February 17<sup>th</sup>, 2023, to develop a topographic basemap of the site utilizing an unmanned aerial system. An aerial image of the site was also collected during the drone flight and was used to produce an orthoimage. The topographic basemap and orthoimage were used for documenting surface features and geologic interpretations during our geologic reconnaissance on March 6<sup>th</sup>, 2023. The observed surface features and our geologic interpretations are presented in Figure 2. Some field observations made by the time of the final site reconnaissance visit are listed below. The below notes also reference site photographs, which are included in Appendix-A.

#### Site Observations:

- Higgins Canyon Road in the site vicinity is asphalt-paved and ranges from approximately 18 to 21 feet wide.
- There are two slip-out scarps on the downslope side of Higgins Canyon Road. The scarps were previously separated by a narrow knob of road fill (see Figure 2); however, the lower portion of the knob has since failed, and the two scarps are now connected as one large scarp.
- The width of the overall failed area extends about 85 feet along Higgins Canyon Road. This area is located along a sharp outside bend in Mills Creek, which appears to have eroded the base of the slope, resulting in at least four separate slip-out events.
- The scarp has encroached into Higgins Canyon Road by about 12 to 18 inches. Most of the slide mass has been washed away by Mills Creek, except for some boulder-

sized riprap near the toe of the scarp. The scarp exposes shoulder fill and underlying alluvial soils. Bedrock was not observed within the scarp.

- Two utility lines consisting of a one-inch and four-inch plastic pipe were exposed in the upper portions of the scarp. Based on nearby utility markings by USA, the utility lines may belong to AT&T.
- Longitudinal tension cracking is present along the roadway margin.

### 4.3 SUBSURFACE EXPLORATIONS

The subsurface investigation consisted of drilling three geotechnical borings (B-1, B-2, and B-3). The borings were drilled on January 25, 2023, by Britton Exploration, Inc., using a track-mounted Mobile CME-55 drill rig equipped with 6-inch diameter solid-flight augers.

Borings B-1 and B-2 were drilled and sampled to a total depth of 36 feet below ground surface (bgs), and Boring B-3 was drilled and sampled to a total depth of 40.5 feet bgs. Upon completion, the borings were backfilled with neat cement grout in accordance with San Mateo County permitting requirements. The surface of borings B-1, B-2, and B-3 were dyed black to match pre-existing conditions. Drilling spoils were thinly spread on-site in the vicinity of the boreholes.

#### 4.3.1 Logging and Sampling

The materials encountered in the borings were logged in the field by a CE&G project engineer. The soil was visually classified in the field, office, and laboratory according to the Unified Soil Classification System (USCS) in general accordance with ASTM D2487 and D2488.

During the drilling operations, soil samples were obtained using one of the following sampling methods:

- California Modified (CM) Sampler; 3.0-inch outer diameter (O.D.), 2.5-inch inner diameter (I.D.) (ASTM D1586)
- Standard Penetration Test (SPT) Split Spoon Sampler; 2.0-inch O.D., 1.375-inch I.D. (ASTM D1586)

The CM and SPT samplers were driven 18 inches (unless otherwise noted on the boring logs) with a 140-pound cable-drop hammer dropping 30 inches. The number of blows required to drive the SPT or CM samplers through each 6-inch interval was recorded for

each sample. The results are included in the boring logs in Appendix A. The blow counts included on the boring logs represent the field values and are uncorrected.

Soil samples obtained from the borings were packaged and sealed in the field to reduce the potential for moisture loss and disturbance. The samples were then taken to CE&G's laboratory in Hayward, California, and Cooper Testing Labs in Palo Alto, California, for testing and/or storage.

#### **4.3.2 Soil Conditions Encountered**

Subsurface soil conditions in the borings were generally consistent with regional geologic mapping. The following soil conditions were encountered beneath the surface:

##### Pavement

Approximately 4 to 8 inches of asphaltic concrete was encountered over another 5 inches of concrete at the surface of borings B-1, B-2, and B-3.

##### Artificial Fill

Fill was encountered in borings B-1, B-2, and B-3 to depths of approximately 5 to 10 feet bgs. The encountered fill generally consists of stiff to hard clay with mixtures of sand and gravel.

##### Quaternary Alluvium

Alluvial deposits, once deposited by Mills Creek, were encountered beneath the fill materials in each boring. The alluvium that was encountered to depths of approximately 30 to 35 feet generally consists of medium-dense to very dense gravels with mixtures of sands and clay, stiff clays, and very dense silts with mixtures of sands and gravels.

##### Purisima Formation Bedrock

Bedrock was encountered in Borings B-1 and B-2 at approximately 35 feet bgs and in Boring B-3 at approximately 30 feet bgs to the maximum depths explored. Bedrock consisted of soft to moderately hard sandy claystone, with moderately severe to moderate weathering. The bedrock is part of the Purisima Formation.

A detailed description of the encountered materials is included in the boring logs in Appendix A. Our interpretation of the soil profile below the project area is shown in Figure 7.

### **4.3.3 Groundwater Conditions Encountered**

Groundwater was encountered in Borings B-1 and B-2 at approximately 32 and 35 feet bgs, respectively. Groundwater was not encountered in Boring B-3. This observation was based on the retrieval of wet samples at depth and through measurement after equilibrium.

### **4.3.4 Geotechnical Laboratory Testing**

Testing was performed to obtain information concerning the qualitative and quantitative physical properties of the samples recovered during the subsurface exploration program. Tests were performed by Cooper Testing Labs in Palo Alto, California, and CE&G's laboratory in Hayward, California, in conformance with applicable ASTM standards. The following tests were performed:

- Moisture Content and Dry Unit Weight (ASTM D2216)
- Particle Size Analysis (ASTM D6913)
- Atterberg Limits (ASTM D4318; dry method)
- Triaxial Unconsolidated-Undrained (ASTM D2850)
- R-Value (ASTM D2844)
- Corrosion Caltrans Package includes:
  - Resistivity (Minimum) (Caltrans 643)
  - pH (Caltrans 643)
  - Chloride (Caltrans 422m)
  - Sulfate (Caltrans 417m)

Laboratory test results are shown on the boring logs presented in Appendix B and on laboratory test result sheets in Appendix C.

## 5.0 CONCLUSIONS AND DISCUSSION

### 5.1 GENERAL SUMMARY

In our judgment, high groundwater levels in combination with overly saturated roadway embankment fill soils and erosion at the toe of the slope by high flows in Mills Creek are likely the main contributing factors to the driving forces of the landslides. This judgment is based on observed seepage within the slide scarp areas downslope of the road and along the outboard edge of the road. Groundwater was also encountered in our borings at the time of drilling.

Based on the results of our investigation, we believe the site is geologically and geotechnically suitable for implementing a retaining wall repair. Geotechnical considerations to note during project design and construction are:

- Drillability and excavatability of encountered materials;
- Seismic design considerations for the project;
- Landsliding;
- Corrosion; and
- Maintaining proper surface and subsurface drainage.

Our evaluations and recommendations are based on the information obtained during this investigation. It is our opinion the site is geologically and geotechnically suitable for implementing a retaining wall repair. A discussion of our findings and recommendations for repairs to be considered for mitigating slope erosion are included in the following sections.

### 5.2 DRILLABILITY AND EXCAVATABILITY

Subsurface exploration was completed using 6-inch-diameter solid flight augers and encountered fill soil from the surface to depths of approximately 9 to 10 feet below the ground surface. Alluvial soil was encountered below the fill and extended to claystone bedrock that was encountered between 30 and 35 feet below the ground surface. Although auger refusal was not encountered, split-spoon drive sampling performed during the subsurface exploration operation encountered refusal (>50 blows per 6 inches) within the underlying bedrock in the borings. Based on the subsurface exploration, we anticipate conventional earthwork and excavation equipment may be used to construct and excavate the fill and native soils and upper bedrock consisting of very soft to medium hard sandy siltstone.

### 5.3 SEISMIC HAZARDS

Large magnitude earthquakes and strong ground shaking are likely to affect the project area within the design lifetime of the proposed improvements. Peak ground shaking parameters are presented below in Section 6.2 and should be considered in the design of the proposed improvements. Local ground-modifying effects of high-intensity ground shaking are considered secondary seismic effects. Our review of these processes is presented below.

- In our judgment, the potential for fault ground rupture or coseismal faulting to significantly affect the proposed improvements is low.
- In our judgment, the potential for ridgetop fissuring, ridgetop shattering, ridgetop spreading, or other seismically induced ground deformation to significantly affect the proposed improvements is low.
- In our judgment, the potential for soil liquefaction to significantly affect the proposed project is low due to the absence of loose to medium-dense granular soils below the groundwater table.

### 5.4 CREEK BANK EROSION

The banks of Mills Creek are generally over-steepened in the project area and are particularly susceptible to instability during the rainy season when flow velocity, depth, and volume are significantly greater. The creek conveys stormwater that can produce both high water levels and high-velocity flows during peak storm conditions. The creek geometry, along with the potential for stormwater events, makes the creek bank susceptible to erosion and undercutting under these conditions. Creek bank erosion (especially at the toe of the slope) is a primary cause of shallow slope failures and slumping, which leads to further erosion and a renewed cycle of slope degradation. Creek bank erosion should be considered in the primary design and construction of the project improvements.

### 5.5 GROUNDWATER AND CREEK FLOWS

Groundwater levels can fluctuate seasonally and/or over a period of years and could rise higher than the groundwater elevations encountered in the borings. During our investigation, groundwater was encountered below the elevation of the creek bottom, which suggests this section of the creek was recharging the local alluvial fan deposits at that time. Slope drainage should be incorporated into the repair of the slope to minimize hydrostatic pressures in the slope.

These conditions should be considered during the analysis, design, and construction of the permanent slope repair and, in particular, the design height of the erosion protection on the bank slope.

## **5.6 LANDSLIDING**

As previously described, no evidence of deep-seated landsliding was detected at the site. Relatively restricted shallow sloughing (landsliding) of alluvium and the uppermost, severely weathered bedrock appears to have been involved in the landslide. Such shallow instability was likely associated with the concentration of surface runoff from the roadway onto the slope below the roadway and the water level rising within the Mills Creek channel.

In our judgment, the potential for deep-seated landsliding (involving bedrock) to adversely affect the site improvements is low under static conditions and low to moderate under seismic conditions.

We also judge the potential for shallow-seated landsliding (under static and seismic conditions) to adversely affect the site improvements to be low, provided site improvements are appropriately designed and constructed and surface runoff is appropriately managed.

## **5.7 SLOPE STABILITY ANALYSIS**

Analysis has been performed to evaluate the existing slope and recommended repair alternative. Laboratory testing was performed on soil samples obtained from the borings to aid in the classification and estimation of the shear strength properties of the soils encountered. Slope stability analysis was performed using the computer program Slide2 by Rocscience (version 9.012) to assess the stability of the current slope and a general slope repair configuration. Cross-sections were developed from the CE&G LiDAR scan and our site observations. Characterization of the soil stratigraphy was determined from the borings performed at the site and subsequent laboratory testing. Soil strength parameters used in the analysis were estimated using laboratory shear strength test results and soil strength correlations based on the field data collected paired with slope stability back-analysis checks. The results of the analyses performed are presented in Appendix D, Slope Stability Analysis. For slope stability modeling, the soil material and strength properties upslope are estimated where no borings were drilled.

A back analysis was completed to assist in estimating the material properties of the encountered soil and bedrock to model the existing conditions. As observed in the field, the creek bank is exhibiting past material erosion and is not considered evidence of the overall

global stability of the slope. Riverbank erosion is mainly caused by high waters due to storms followed by a drawdown during the days after. Table 5-1 below shows the results of the back analysis.

**Table 5-1. Slope Stability Back Analysis**

Analysis Condition	Factor of Safety
Existing Model	0.99

A slope stability analysis featuring a generalized repair design configuration was completed to analyze the slope after it has been repaired, evaluating the high groundwater and seismic conditions of the slope. An additional repair design configuration with Rock Slope Protection (RSP) was also analyzed. Table 5-2 shows the results of the slope stability analysis with generalized repair design configurations.

**Table 5-2. Slope Stability Analysis of Generalized Repair Design Configuration**

Analysis Condition	Factor of Safety
Retaining Wall	1.20
Retaining Wall – Seismic	0.87
Wall w/ RSP	1.71
Wall w/ RSP – Seismic	1.09

In summary, we offer the following conclusions from the slope stability analysis:

1. Control of subsurface drainage is essential for the long-term stability of any repair.
2. A slope repair increases the factor of safety of the slope during high groundwater conditions, particularly with subsurface drainage incorporated into the repair.

If it is determined that a retaining wall will be constructed on the slope, it should be anchored into bedrock.

Section 5.10 below provides discussions of the slope repair alternatives.

## 5.8 LIQUEFACTION

The soils along the creek channel are within a liquefaction hazard zone mapped by the State of California. To address this, we evaluated the above-noted criteria required for

liquefaction to occur. Based on a review of historic groundwater levels, the highest groundwater level recorded in the site vicinity is approximately 32 to 35 feet below the road's ground surface. The soils encountered below 32 feet include dense to very dense granular soils and/or bedrock (minimum blow count = 40 blows per foot, uncorrected), which are not considered susceptible to liquefaction. In summary, based on subsurface information collected during this investigation, we judge the potential for significant settlement to occur as a result of liquefaction at this site to be low because the groundwater level is generally low, the granular soils below the site are generally too dense to liquefy.

Seismic densification is the densification of unsaturated, loose to medium-dense granular soils due to strong vibrations such as that resulting from earthquake shaking. There is a potential for seismic densification of on-site native loose to medium-dense granular soils. We estimated approximately ½- to 1-inch of settlement within the granular soils comprising the creek bank may result from strong ground shaking during a major earthquake.

## 5.9 CORROSION

Corrosion testing, in general accordance with Caltrans methods, was performed on one soil sample from boring B-1. Testing results are presented in the following table:

**Table 5-3. Corrosion Testing Results**

Boring (depth in feet)	Resistivity (Ohm-cm)	Chloride (mg/kg)	Sulfate (mg/kg)	pH
B-1 (1.5)	1586	94	109	7.1

Caltrans Corrosion Guidelines, May 2021, identify a site as corrosive for structural elements if one or more of the following conditions exist:

- Chloride concentration is 500 ppm or greater;
- Sulfate concentration is 1,500 ppm or greater;
- pH is 5.5 or less.

A minimum resistivity value for soil and/or water less than 1500 ohm-cm indicates the presence of high quantities of soluble salts and a higher susceptibility to being corrosive. Based on the results of the laboratory testing performed, the tested soil sample is not considered to be corrosive to metals or concrete based on the Caltrans criteria listed above.

According to ACI 318 Section 4.3, Table 4.3.1:

- Sulfate concentration below 0.10 percent by weight (1,000 ppm) is negligible (no restrictions on concrete type)
- Water-soluble chloride content of less than 500 ppm is generally considered non-corrosive to concrete.

Based on the results of the laboratory testing performed, the soil sample tested is considered not corrosive to concrete.

Corrosion results should be considered preliminary and are an indicator of potential soil corrosivity for the sample tested. Other soils found on-site may be more, less, or of similar corrosive nature. Our scope of services does not include corrosion engineering; therefore, a detailed analysis of the corrosion tests is not included.

## **5.10 REPAIR ALTERNATIVES**

We have considered several conceptual alternatives to repair the slope. For a permanent repair to be implemented, several factors specific to the site will have an impact on the development and implementation of the permanent repair, including:

- Lateral extent to be repaired,
- Hydraulic design conditions,
- Shoring of existing roadway,
- Limited right-of-way
- Overhead clearance and utilities,
- Environmental requirements for permitting,
- Planning and coordination of construction staging areas, and
- Potential disturbance to trees in the project area.

We considered repair alternatives such as earth repairs and retaining walls.

### **5.10.1 Earth Repairs**

Earth repairs may require an excavation encroaching into the pavement and may require the road to be closed for the duration of the construction. Earth repairs would require exporting all excavated material and importing engineered fill to replace the material for

the entire length of the repair. An earth repair would allow for the placement of subdrains and reinforced geogrid to stabilize the slope. This repair option would allow for the placement of erosion protection measures at the toe of the slope, such as vegetated slope protection (VSP) consisting of planted rip rap, that intends to decrease the flow velocity of the creek.

### **5.10.2 Soldier Pile and Lagging Wall with Tiebacks**

A soldier pile and lagging wall with tiebacks may require significantly less encroachment on the road, allowing vehicles to pass by during construction. Tiebacks drilled and placed along the wall may encroach onto nearby easements, depending on the length and configuration of the tiebacks. A tieback wall would allow for the placement of erosion protection measures, such as VSP (planted rip rap), that intends to decrease the flow velocity of the creek. A tieback wall would allow for the placement of subdrains behind the wall to remove the buildup of hydrostatic pressures.

Additionally, we recommend the final design of the riverbank repair consider potential changes in river flows resulting from climate change, if applicable.

## **5.11 RECOMMENDED ALTERNATIVE**

Based on our preliminary engineering evaluation, we recommend a soldier pile and lagging wall with tiebacks and VSP down to the creek bottom for the following reasons:

1. Trees removed as part of site clearing can be recycled as log deflectors or root wads placed within the VSP at the slope bottom for erosion control,
2. VSP provides both resistance to higher flow velocity/shear stresses typically observed in sharp channel meanders and can accommodate vegetation growth,
3. Reduced excavation depths required and allows for more efficient shoring designs,
4. Confine limits of native soil replacement with VSP to areas most probable to be subject to high shear stresses that cause erosion and scour.

Construction duration and phasing will largely be determined by the contractor's proposed approach and schedule. Overhead utilities above the channel bank, as well as subsurface utilities beneath Higgins Canyon Road, must be considered for each of the alternatives.

## 5.12 GEOTECHNICAL CONSIDERATIONS

Some geotechnical issues that will affect the design and construction of the proposed creek slope bank repair are as follows:

- **Temporary Excavation Configuration** – Based on the County's limited right-of-way, steep temporary construction slopes would be required as part of the creek bank repair.
- **Creek Bank Erosion** – Flows in the creek may undermine the toe of the slope, requiring erosion control to maintain the stability of the permanently repaired slope.
- **Drillability** – Subsurface exploration was completed using solid-flight augers and did not encounter auger refusal to the maximum depths explored. Based on the subsurface exploration, we anticipate that an appropriately sized drill rig equipped with rock bits will be able to drill through the soil and bedrock underlying the project site.
- **Retaining Wall** –Based on the site geometry, depth to competent materials, and proposed retaining wall layout, the anticipated wall heights are achievable through a tieback wall approach. Advantages to a tieback wall include shorter steel and smaller steel section, whereas disadvantages include a more complex design, more steps in the construction, and the need to install tiebacks. Recommendations have been provided in Section 6 for the tieback retaining wall.
- **Surface Water Drainage** – Surface water runoff should be collected from the roadway above and discharged in an appropriate energy dissipater away from the slide area below the proposed repair. Surface drainage improvements should be designed to adequately collect and accommodate the volume of water that reach these drainages.

## **6.0 DESIGN AND CONSTRUCTION RECOMMENDATIONS**

### **6.1 EXCAVATIONS**

Excavations for the project are anticipated to be up to but not limited to 25 feet deep for the construction of the repair alternatives. We anticipate that an appropriately sized backhoe or excavator can excavate the soil underlying the project site. The narrow road may limit excavation equipment size and laydown areas along the road. Excavations for the repair are anticipated to encounter stiff clayey fill soils overlying medium dense to very dense granular soils.

Temporary excavation slopes should not be steeper than 1.5H:1V and should conform to the requirements of Occupational Safety and Health Administration (OSHA) requirements, and the State of California Department of Transportation, Trenching and Shoring Manual for earthen slopes. The stability and safety of excavations, braced or unbraced, are the contractor's responsibility.

CE&G should be retained to observe subsurface conditions at the time of excavation to confirm assumed conditions and provide revised recommendations, if necessary.

### **6.2 EARTHWORK**

Grading required for the project will include excavation to develop temporary site access and create a bench for drilling and construction of the soldier piles and tiebacks. This bench will also be needed for the placement of excavated material as engineered fill in order to approximately restore the original grade at the site. Minor grading could also be required to modify or construct drainage facilities and to distribute excess excavated material on-site, as appropriate.

Before the commencement of the grading operation, the site should be cleared for the buried or overhead utilities. Care should be taken not to damage any utilities present. This should be done in coordination with utility providers. The site should be grubbed of existing vegetation and any large fallen trees. All existing debris should be removed from the site, including but not limited to existing pavements and buried pipes within the work area. Before engineered fill is placed, loose soil and vegetation should be removed from the areas to receive fill. All depressions created by tree and stump removal should be excavated to firm soil or bedrock prior to placement of fill.

### **6.2.1 Engineered Fill Placement and Compaction**

On-site soils that will likely be excavated as part of the retaining wall construction may be used as general engineered fill. Fill should be placed and compacted to a relative compaction of 90 percent ASTM D-1557 latest edition. Fill materials shall be spread evenly and compacted in uniform lifts not exceeding 8 inches in uncompacted thickness. Fill materials not meeting the specified relative compaction shall be ripped, moisture conditioned, and reworked until the required relative compaction and moisture content are attained.

### **6.2.2 Select Import Backfill**

All imported fill must be reviewed and approved by the geotechnical engineer before importation to the site. A minimum of five days will be required to evaluate and test the suitability of all proposed imported materials. All select import backfill materials should meet the following criteria:

The import materials shall be non-expansive and have a Plasticity Index of less than 12 percent and a Liquid Limit of 30 percent or less with a minimum friction angle of 34 degrees. The import material shall not contain rocks or lumps larger than 6 inches in greatest dimension and should not contain more than 15 percent of the material larger than 3 inches. These materials shall be free of organic debris or contaminated materials.

Imported fill materials should be placed and compacted to a minimum of 90 percent relative compaction at a moisture content of at least 2 percent over optimum as determined by the ASTM D-1557 (latest revision) test procedure. Fill material in the upper 24 inches of the pavement subgrade shall be compacted to a minimum of 95 percent relative compaction.

### **6.2.3 Wet Weather Construction**

We recommend that earthwork not be performed during wet weather seasons. If site grading and construction are to be performed during rainy periods, the owner and contractors should be fully aware of the potential impact of wet weather. Rainstorms could cause unstable excavations, delays to construction, and damage to previously completed work by saturating compacted fills or subgrades or flooding excavations.

Earthwork during rainy months will require extra effort and caution from the contractors. The contractor should be responsible for protecting his work to avoid damage by rainwater. Standing pools of water should be pumped out immediately. The project construction bid documents should address construction during wet weather conditions.

We recommend that the contractor submit a wet weather construction plan outlining procedures they will employ to protect their work and minimize damage to their work by rainstorms.

### 6.3 SEISMIC DESIGN PARAMETERS

Due to the site’s proximity to the numerous active fault systems which traverse the greater San Francisco Bay Area, the project site will likely be subjected to the effects of a major earthquake during the design life of the proposed improvements. The effects are likely to consist of significant ground accelerations. These ground-type movements may cause damage to the proposed improvements. Therefore, we recommend that, at minimum, the structural systems for the proposed improvements be designed per the requirements of Chapter 16 of the 2019 California Building Code and ASCE 7-16, Supplement 3, for Site Class D type soils and Risk Category 1, and latitude 37.445993, longitude -121.404572. The design parameters yielded an MCE PGA of 0.873g. The ASCE 7-16 seismic design parameters for the site are included in Table 6-1. 2019 CBC Seismic Design Parameters .

**Table 6-1. 2019 CBC Seismic Design Parameters**

Item	Design Value
Site Soil Class Definition	D
S <sub>s</sub> – 0.2 Second Spectral Response Acceleration	1.899
S <sub>1</sub> – 1.0 Second Spectral Response Acceleration	0.718
F <sub>a</sub> – Values of Site Coefficient	1.0
F <sub>v</sub> – Value of Site Coefficient	1.7*
S <sub>DS</sub> – Designed Spectral Response Acceleration for Short Periods	1.266
S <sub>D1</sub> – Designed Spectral Response Acceleration for 1-Sec Periods	1.221
S <sub>MS</sub> – MCER Spectral Response Acceleration Parameter (g)	1.899
S <sub>M1</sub> – MCER Spectral Response Acceleration Parameter (g)	1.7x0.718x1.5=1.831**
PGA	0.794
PGA <sub>M</sub>	0.873

\* - This value shall only be used for the calculation of T<sub>s</sub> and the parameters S<sub>M1</sub> and S<sub>D1</sub> before they are increased by 50%

\*\* - In accordance with the EXCEPTION to ASCE 7-16, Section 11.4.8, the value of the parameter S<sub>M1</sub> determined by Eq. (11.4-2) has been increased by 50% for all applications of S<sub>M1</sub> to eliminate the need for performing a ground motion hazard analysis.

As a result of earthquake shaking, the soil behind the retaining walls will exert an additional horizontal force on the walls. We recommend using an additional seismic equivalent fluid pressure (EFP) of 39 PCF to model the earthquake-induced force on the walls. The seismic equivalent fluid pressure was selected based on the design response spectrum peak ground acceleration (PGA), which is 2/3 of the Maximum Considered Earthquake (MCE) PGA, determined using the United States Geologic Survey (USGS) Earthquake Hazards Program website (USGS, 2015) for Site Class D type soils. Using methods published by Sitar and Agusti 2013 in their paper for *Seismic Earth Pressures on Retaining Structures in Cohesive Soils*, a seismic equivalent fluid pressure equal to the following was used for a cantilever (level) retaining wall.

$$\text{EFP} = 25 \text{ PCF}$$

Based on load factors of 1.2 used for earth pressures and 1.0 for seismic, the seismic EFP was reduced by 1.2.

#### **6.4 RETAINING WALLS**

To stabilize the existing slope, we recommend constructing a soldier pile and concrete lagging retaining wall with tieback anchors. Based on the results of our subsurface exploration, the depth to the top of bedrock along the retaining wall alignment extends to approximately 30 feet below the anticipated top of the wall, and the existing slope below the wall is generally 1H:1V in inclination. Retaining walls with heights of 20 feet and with the geometry that exists along the proposed alignment typically require deep piers with larger structural elements for a cantilever wall or require the use of tiebacks in order to scale down the size and depth of the soldier piles. The end piers for the wall may be designed as cantilever structural elements, and the interior piers should be designed with tiebacks incorporated into the structure. Recommendations for this wall type are included in the following sections.

It should be noted that the suitability for repair using a geogrid-reinforced slope or a geogrid-reinforced wall was also considered. These approaches were eliminated as likely economically infeasible due to the difficult construction access on the steep site slopes for an earthwork solution and the long distance down the slope that would be required to start a fill slope as well as the large volume of required excavation and the space required for soil stockpiling and processing, as well as the likely need to work within the creek channel.

### 6.4.1 Lateral Earth Pressures

Static lateral earth pressure will be imposed on all shored excavations. Table 6-2 summarizes the lateral earth pressures recommended for use in the design of unbraced temporary shoring. Active pressure should be assumed for conditions where the top of the wall is free to deflect up to ½ inch. Passive pressure should be ignored for a depth of 24 inches in unpaved areas and may be utilized to resist overturning and sliding. Where structures will be located below groundwater, hydrostatic pressures are already considered in the passive lateral earth pressure values shown in Table 6-2.

**Table 6-2 - Lateral Earth Pressures - Shoring**

Pressure Type	Above Creek Bed		Below Creek Bed	
	Above Groundwater Level (Equiv. Fluid Pressure)	Below Groundwater Level (Equiv. Fluid Pressure + Hydrostatic)	Above Groundwater Level (Equiv. Fluid Pressure)	Below Groundwater Level (Equiv. Fluid Pressure + Hydrostatic)
Active	50 pcf	90 pcf	35 pcf	90 pcf
At-Rest	70 pcf	100 pcf	50 pcf	90 pcf
Passive	355 pcf <sup>1</sup>	245 pcf <sup>1</sup>	540 pcf	345 pcf

<sup>1</sup> where slope (1.5H:1V or flatter) exists in front of the shoring wall, reduce to 120 PCF. Increase to table value where the horizontal distance from the wall to the slope face is 10 feet or greater.

The design of unbraced shoring will likely be controlled by deflections. Lateral deflection at the top of the shoring shall not be greater than ½-inch.

If the temporary shoring will be braced, rectangular, or trapezoidal loading diagrams such as those recommended by Terzaghi & Peck, Tschebortarioff, and others (Caltrans Trenching and Shoring Manual and FHWA GEC No. 4) should be used. These methods generally correlate the earth pressure load to a percentage of the unit weight of the soil times the height of the excavation. The method and loading should be determined by the contractor and provided to the Engineer for review.

If a rectangular or trapezoidal loading is used, the native sandy deposits can be assumed to have a uniform lateral load of 30H psf for the full height (H) (in feet) of the excavation shoring plus a lateral fluid pressure of 62.4 PCF (unit weight of water) starting at the design groundwater elevation to account for groundwater. It is recommended that the contractor's shoring design engineer evaluate high and low groundwater cases to confirm which case governs the design. Shoring for excavations that penetrate below an imaginary

plane having an inclination of 1½:1 (horizontal to vertical) extending down from the bottom edge of the foundations must be designed for foundation surcharge pressures and must limit deflections to less than ½ inch. The shoring design should consider surcharge loading from traffic on the adjacent areas and construction equipment adjacent to excavations.

#### 6.4.2 Tieback Retaining Wall

The tieback soldier pile retaining wall should be designed for unrestrained (active) conditions using the following:

- The planned wall alignment is not yet known. However, based on the topography and soil conditions, we estimate the wall height to be approximately 18 feet. For walls over about 12 feet in height, tiebacks will likely be needed.
- The first design loading condition should be for a cantilever wall extending to the excavation limits for the tiebacks. For the design of the cantilever section, we recommend an active equivalent fluid pressure of 50 pcf and a passive equivalent fluid pressure of 360 pcf to a depth of 9 feet and an active equivalent fluid pressure of 40 pcf and a passive equivalent fluid pressure of 420 pcf between 9 and 18 feet. The passive pressure should be taken on two pile diameters and begin 5 feet below the bench excavation for the installation of the tiebacks. The active equivalent fluid pressure should be assumed to act continuously along the wall. This loading condition is temporary for construction purposes and should have a target factor of safety of 1.2.
- A passive equivalent fluid pressure of 540 psf/ft starting 3 feet below the bottom of the creek bed acting over two pier diameters;
- A seismic equivalent fluid pressure of 25 pcf acting over the full height of the retaining wall. Seismic loading should be applied in addition to the above active equivalent fluid pressure ignoring traffic live load.
- For the tieback loading condition, a trapezoidal-shaped load distribution based upon the apparent earth pressure diagram on page 51 of the FHWA manual (Figure 24 – included in Appendix E). The FHWA diagram shows the load distribution to the anchors based on the anchor locations.
- Determination of the tieback force and soldier pile maximum moment should be based upon comparing the requirements from both temporary cantilever loading and final tieback loading conditions. This requirement is necessary since the

requirements will vary at each stage, and the pile and tieback must be designed to handle both cases.

- Tieback rods should be a minimum of 1-inch diameter, ASTM A722, Grade 150, Class I double corrosion protected, or equivalent;
- The tieback should be locked off at 100 percent of the design load;
- For preliminary tieback design, tiebacks should be drilled at an inclination of 15 to 20 degrees below the horizontal and have an unbounded zone of 10 feet;
- Ultimate ground-grout bond strength of 110 psi is recommended for preliminary tieback design. This bond strength assumes the tieback is pressure grouted. For gravity-grouted anchors, the average ultimate bond stress is significantly lower at 20 psi.
- The tieback should be designed with a post-grout tube in the event secondary grouting is determined to be necessary;
- Since the construction methods used to install tiebacks can dramatically influence the capacity of the anchors, the final tieback design and length of the bonded zone shall be the responsibility of the contractor to achieve the design capacity. Anchors may use secondary grouting techniques;
- Proof and performance testing should be performed to a maximum load of 1.33 times the dead load. At least one anchor shall be performance tested. Anchor acceptance should conform to the criteria included in the FHWA manual for creep and apparent free length;
- Minimum pile diameter of 30 inches;
- Minimum pile spacing of three diameters on center;
- Minimum pile depth of 10 feet below the creek bottom;
- Minimum tieback anchor diameter of 6 inches;

The active and seismic equivalent fluid pressures assume the retaining wall will be backfilled using on-site materials excavated during soldier pile drilling operations or select import backfill with a minimum friction angle of 34 degrees and as outlined in Section 6.1.

#### **6.4.3 Retaining Wall Drainage**

The above equivalent fluid pressures for a tieback retaining wall assume fully drained conditions behind the soldier pile wall. Therefore, the wall should be provided with a full-

height back wall drainage consisting of a 12-inch-wide layer of Caltrans Class 2 permeable material that stops 12 inches below the ground surface. Native clayey soil or aggregate base and asphalt pavement should be used for the upper foot of the wall backfill and should cap the drainage material. A clean coarse gravel or drain rock may be used as an alternative to the Class 2 Permeable drainage material. If coarse gravel or drain rock is selected as a drainage material, it should be separated from all adjacent soil by an engineering filter fabric such as Mirafi 140N or a similar geotextile. Enough space should be provided between the laggings to allow seepage through the face of the wall.

In lieu of the above-mentioned drain rock, a prefabricated drainage composite such as "CCW MiraDRAIN 6000XL" or equivalent may be used for drainage behind the retaining walls. This drainage composite should be installed in accordance with the manufacturer's recommendations on the back of the tieback wall at least 1 foot below the ground surface and should be wrapped around a drainage pipe at the base of the wall.

#### **6.4.4 Construction Considerations**

The contractor is expected to provide suitable equipment to drill soldier piles. It is recommended that the contractor fully evaluate considerations such as the use of additional specialized equipment during the bidding process.

The bottoms of soldier piles should be dry and free of loose cuttings and debris before installing the steel beams and concrete. This shall be done to satisfy the engineer or geologist from Cal Engineering & Geology, Inc., who observes the drilling operations. The concrete should be placed carefully in the drilled holes so that over-pouring of the piles (mushrooming at the top) does not occur and the concrete does not have a free fall drop over 4 feet.

Free groundwater was encountered at 32 to 35 feet depths during the exploratory drilling. The drilling contractor should be prepared to drill and place steel and concrete for the piles on the same day. Under no circumstances shall water be allowed to remain in a drilled pile hole overnight. Should this occur, it will be necessary for the contractor to enlarge the hole to a wider diameter and/or a greater depth to the satisfaction of the engineer or geologist from our office who is observing the drilling operation.

#### **6.5 VEGETATED SLOPE PROTECTION (VSP)**

Unprotected steep slopes are prone to future localized slumping and shallow slope failures. Remediation to reduce the potential for future slope erosion and shallow failures include flattening and/or armoring the existing slopes. We recommend that the toe of the creek

bank slopes be armored with planted riprap or a similar form of erosion protection to minimize erosion. Slope armoring, such as the placement of planted riprap slope protection at the toe of the slopes, will reduce the potential for toe erosion and progressive slope instability.

## **6.6 SURFACE DRAINAGE**

Surface drainage along the roadway is to be considered by the Design Team and incorporated into the project plans where appropriate. Collected surface water from the roadway should be conveyed by a pipe to a discharge point below active sliding or gullying, and appropriate energy dissipaters should be constructed at the outlet points to reduce the potential for future slope instability or erosion/gullying.

## **6.7 SOIL OR BEDROCK CORROSION POTENTIAL**

The corrosion potential of the on-site soil and bedrock materials was tested as part of this investigation. A sample 1.5 feet deep from boring B-1 was chosen to have resistivity and chloride and sulfate tests at Cooper Testing Laboratory. Following the standard stated in California Test 643, the resistivity of the sample was determined, and the sample was classified as one with low corrosion potential. However, since the site is adjacent to a creek, and due to the resistivity value of the tested sample being near the threshold value for corrosive soils, the County should use a coating for all steel beams and use Class 1 corrosion protection for tiebacks. If the County has previous experience with the corrosivity of the on-site soils and import material, and/or additional corrosion testing is completed, these recommendations can be modified accordingly.

## **7.0 LIMITATIONS**

The conclusions and recommendations presented in this report are based on the information provided regarding the planned construction and the results of the geologic mapping, subsurface exploration, and testing, combined with interpolation of the subsurface conditions between boring locations. Site conditions described in the text of this report are those existing at the time of our last field reconnaissance and are not necessarily representative of the site conditions at other times or locations. This information notwithstanding, the nature and extent of subsurface variations between borings may not become evident until construction. If variations are encountered during construction, Cal Engineering & Geology, Inc. should be notified promptly so that conditions can be reviewed and recommendations reconsidered, as appropriate.

It is the County's responsibility to ensure that the recommendations contained in this report are carried out during the construction phases of the project. This report was prepared based on preliminary design information provided, which is subject to change during the design process. At approximately the 90 percent design level, Cal Engineering & Geology, Inc. should review the design assumptions made in this report and prepare addenda or memoranda as appropriate. Any modifications included in these addenda or memoranda should be carefully reviewed by the project designers to make sure that any conclusions or recommendations that are modified are accounted for in the final design of the project.

The findings of this report should be considered valid for a period of three years unless the conditions of the site change. After a period of three years, CE&G should be contacted to review the site conditions and prepare a letter regarding the applicability of this report.

This report presents the results of a geotechnical and geologic investigation only and should not be construed as an environmental audit or study. The evaluation or identification of the potential presence of hazardous materials at the site was not requested and was beyond the scope of this investigation and report.

The conclusions and recommendations contained in this report are valid only for the project described in this report. We have employed accepted geotechnical engineering procedures, and our professional opinions and conclusions are made in accordance with generally accepted geotechnical engineering principles and practices. This standard is in lieu of all other warranties, either expressed or implied.

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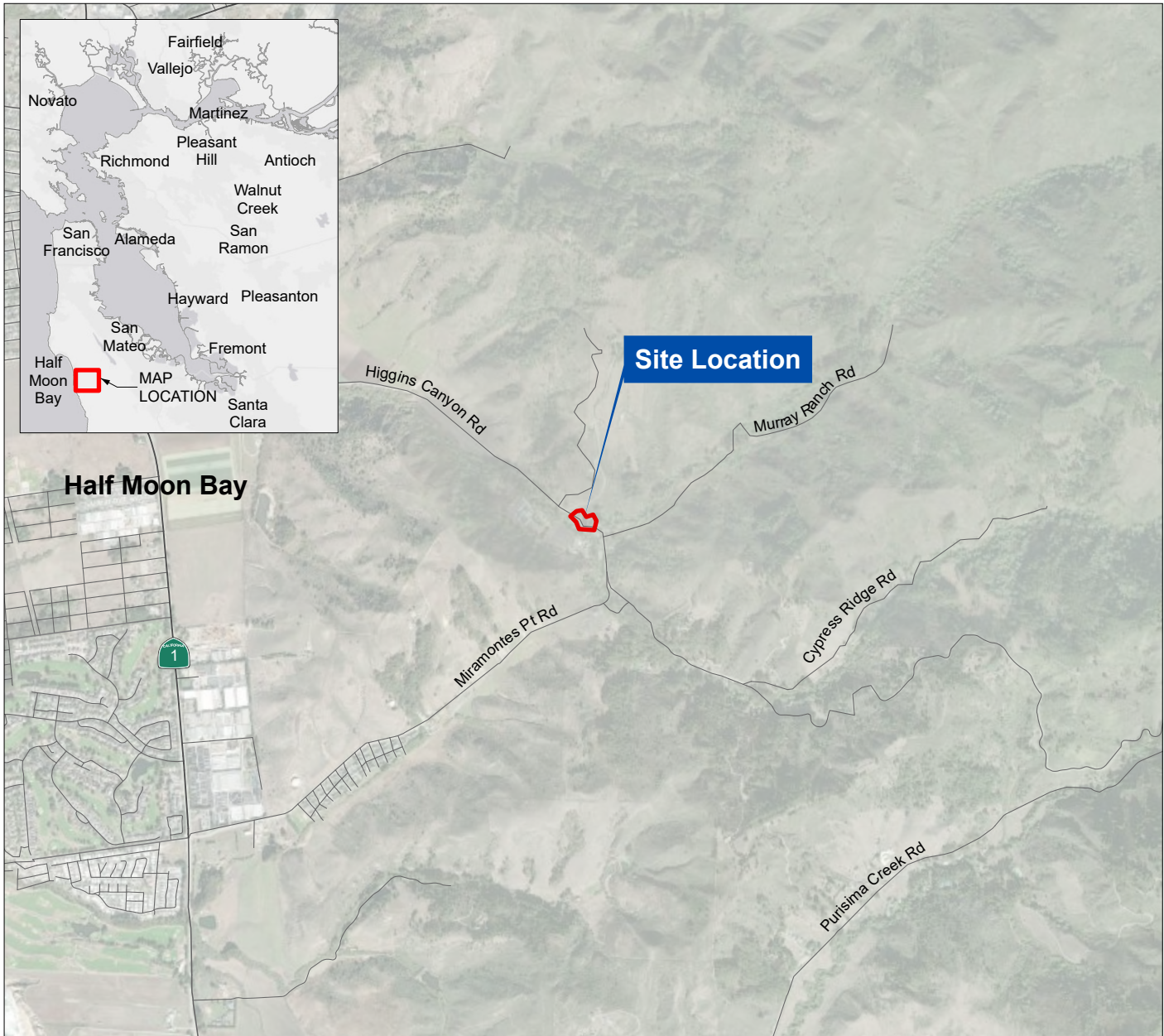
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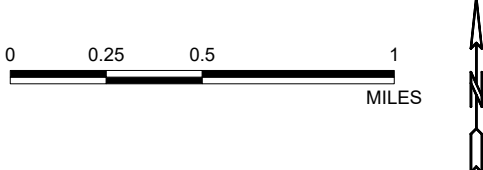
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## **FIGURES**



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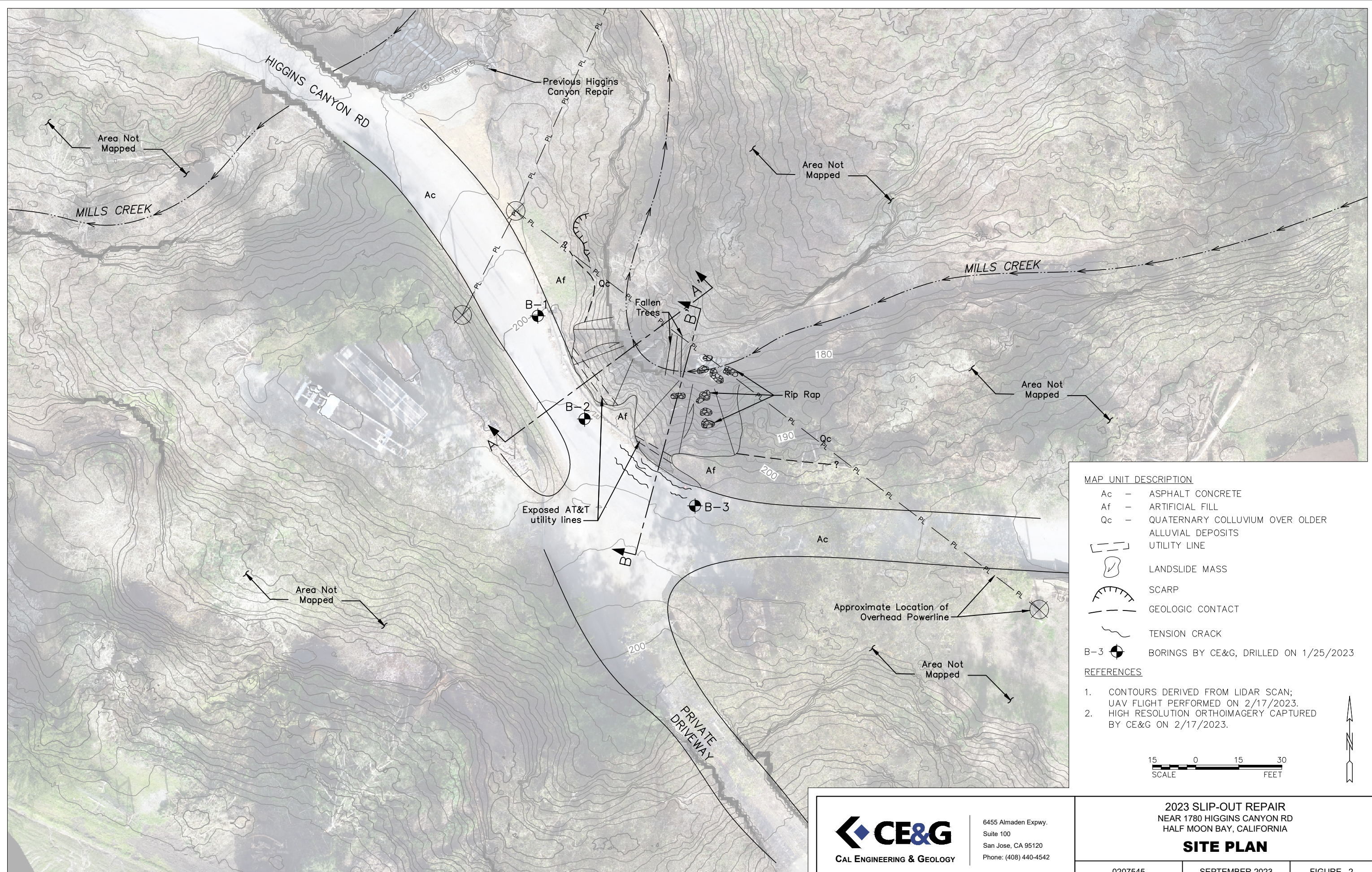
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1780 HIGGINS CANYON ROAD SLOPE STABILIZATION PROJECT  
1780 HIGGINS CANYON ROAD  
HALF MOON BAY, CALIFORNIA

**SITE LOCATION MAP**

0207545	SEPTEMBER 2023	FIGURE 1
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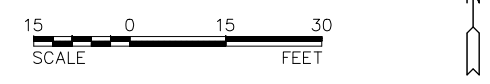
\\holtydrich\share\granite\2022\0207545-SonMateoCo-1780HigginsCynRd\AutoCAD\Figures\Fig2-StePlan.dwg 9-08-23 08:26:43 AM slamothe



**MAP UNIT DESCRIPTION**

Ac	-	ASPHALT CONCRETE
Af	-	ARTIFICIAL FILL
Qc	-	QUATERNARY COLLUVIUM OVER OLDER ALLUVIAL DEPOSITS
[Symbol]	-	UTILITY LINE
[Symbol]	-	LANDSLIDE MASS
[Symbol]	-	SCARP
[Symbol]	-	GEOLOGIC CONTACT
[Symbol]	-	TENSION CRACK
B-3	⊗	BORINGS BY CE&G, DRILLED ON 1/25/2023

- REFERENCES**
1. CONTOURS DERIVED FROM LIDAR SCAN; UAV FLIGHT PERFORMED ON 2/17/2023.
  2. HIGH RESOLUTION ORTHOIMAGERY CAPTURED BY CE&G ON 2/17/2023.



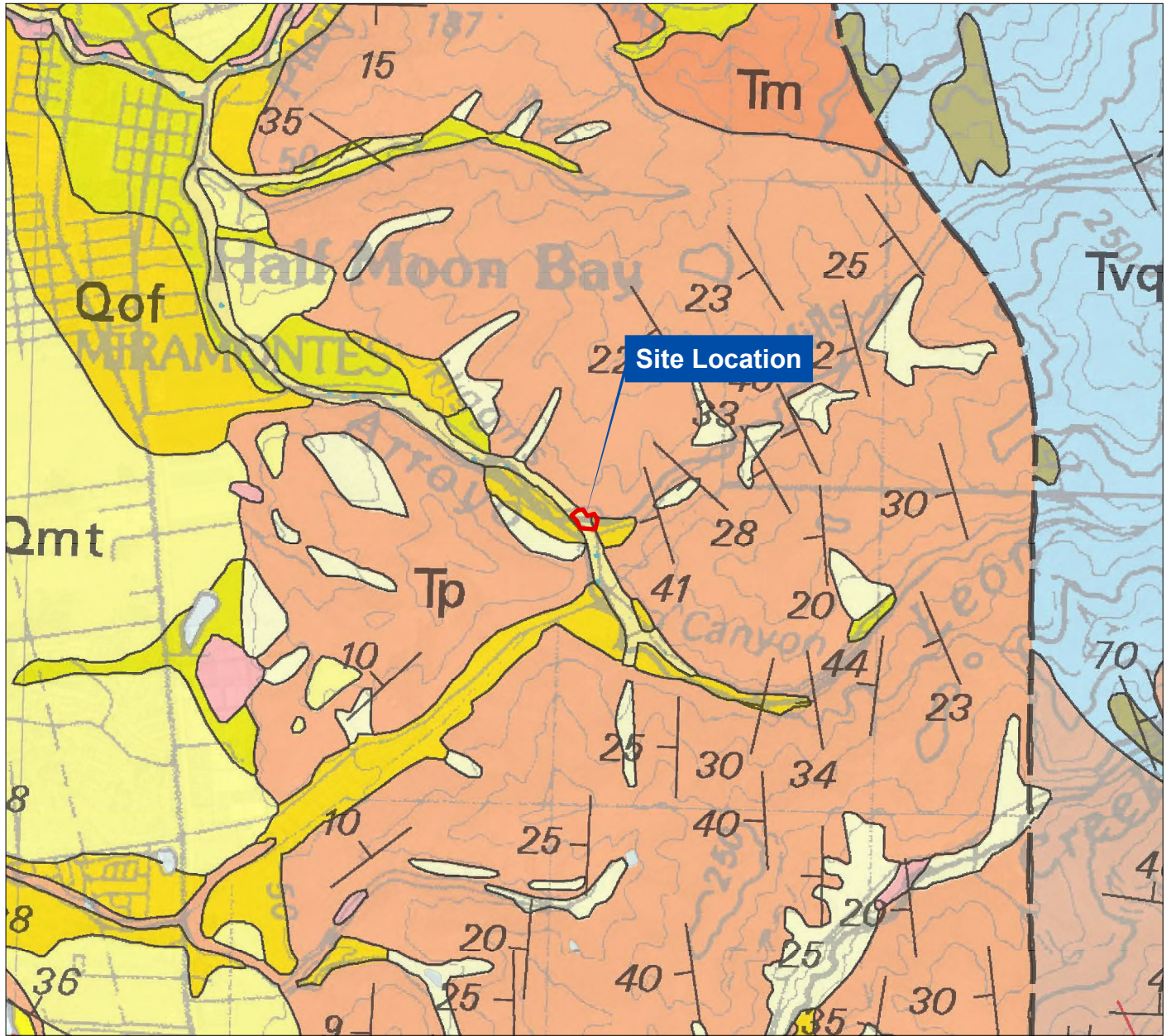
**CE&G**  
**CAL ENGINEERING & GEOLOGY**

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 San Jose, CA 95120  
 Phone: (408) 440-4542

**2023 SLIP-OUT REPAIR**  
 NEAR 1780 HIGGINS CANYON RD  
 HALF MOON BAY, CALIFORNIA

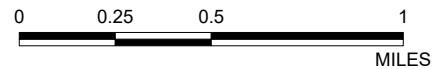
**SITE PLAN**

0207545	SEPTEMBER 2023	FIGURE 2
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**BASEMAP REFERENCE**

1. REGIONAL GEOLOGY FROM BRABB ET AL., 2000.



**MAP UNIT DESCRIPTION**

<p><b>Qyf</b> YOUNGER (INNER) ALLUVIAL FAN DEPOSITS (HOLOCENE)</p> <p><b>Qyfo</b> YOUNGER (OUTER) ALLUVIAL FAN DEPOSITS (HOLOCENE)</p> <p><b>Qcl</b> COLLUVIUM (HOLOCENE)</p> <p><b>Qs</b> SAND DUNE AND BEACH DEPOSITS (HOLOCENE)</p> <p><b>Qal</b> ALLUVIUM (HOLOCENE)</p> <p><b>Qof</b> COARSE-GRAINED OLDER ALLUVIAL FAN AND STREAM TERRACE DEPOSITS (PLEISTOCENE)</p>	<p><b>Qmt</b> MARINE TERRACE DEPOSITS (PLEISTOCENE)</p> <p><b>Tp</b> PURISIMA FORMATION (PLIOCENE AND UPPER MIOCENE)</p> <p><b>Tm</b> MONTEREY FORMATION (MIDDLE MIOCENE)</p> <p><b>Tla</b> LAMBERT SHALE (OLIGOCENE AND LOWER MIOCENE)</p> <p><b>Tmb</b> MINDEGO BASALT AND RELATED VOLCANIC ROCKS (MIOCENE AND/OR OLIGOCENE)</p> <p><b>Tvq</b> VAQUEROS SANDSTONE (LOWER MIOCENE AND OLIGOCENE)</p>
--	---

STRIKE & DIP



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1780 HIGGINS CANYON ROAD SLOPE STABILIZATION PROJECT  
1780 HIGGINS CANYON ROAD  
HALF MOON BAY, CALIFORNIA

**REGIONAL GEOLOGY MAP**

0207545

SEPTEMBER 2023

FIGURE 3



**BASEMAP REFERENCE**

1. FAULT LOCATIONS FROM US GEOLOGICAL SURVEY QUATERNARY FAULTS AND FOLDS DATABASE, ACCESSED ONLINE ON 30 JULY 2021.



**MAP UNIT DESCRIPTION**

- Historical (<150 years), well constrained location
- - - Historical (<150 years), moderately constrained location
- Historical (<150 years), inferred location
- Latest Quaternary (<15,000 years), well constrained location
- - - Latest Quaternary (<15,000 years), moderately constrained location
- Latest Quaternary (<15,000 years), inferred location
- Late Quaternary (<130,000 years), well constrained location
- - - Late Quaternary (<130,000 years), moderately constrained location
- Late Quaternary (<130,000 years), inferred location
- Undifferentiated Quaternary (<1.6 million years), well constrained location
- - - Undifferentiated Quaternary (<1.6 million years), moderately constrained location
- undifferentiated Quaternary (<1.6 million years), inferred location

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1780 HIGGINS CANYON ROAD SLOPE STABILIZATION PROJECT  
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**FAULT ACTIVITY MAP**

0207545

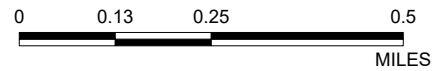
SEPTEMBER 2023

FIGURE 4



**BASEMAP REFERENCE**

1. HALF MOON BAY QUADRANGLE LANDSLIDE INVENTORY DATA FROM BRABB, 2000.



**MAP UNIT DESCRIPTION**

- D DEFINITE LANDSLIDE DEPOSIT
- P PROBABLE LANDSLIDE DEPOSIT
- ? QUESTIONABLE LANDSLIDE DEPOSIT
- UA UNATTRIBUTED LANDSLIDE DEPOSIT

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1780 HIGGINS CANYON ROAD SLOPE STABILIZATION PROJECT  
1780 HIGGINS CANYON ROAD  
HALF MOON BAY, CALIFORNIA

**LANDSLIDE INVENTORY**

0207545

SEPTEMBER 2023

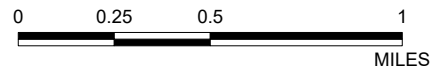
FIGURE 5

\\haleyaldrich\share\granite\2022\0207545-SanMateoCo-1780-HigginsCynRdRetWall\GIS\ArcGIS\221440-HigginsCynRdRetWall.aprx: 9/12/2023; slsmothe



**BASEMAP REFERENCE**

- 1. RELATIVE LIQUEFACTION SUSCEPTIBILITY FROM WITTER (2006).



**MAP UNIT DESCRIPTION**

- VERY HIGH
- MODERATE
- LOW
- VERY LOW



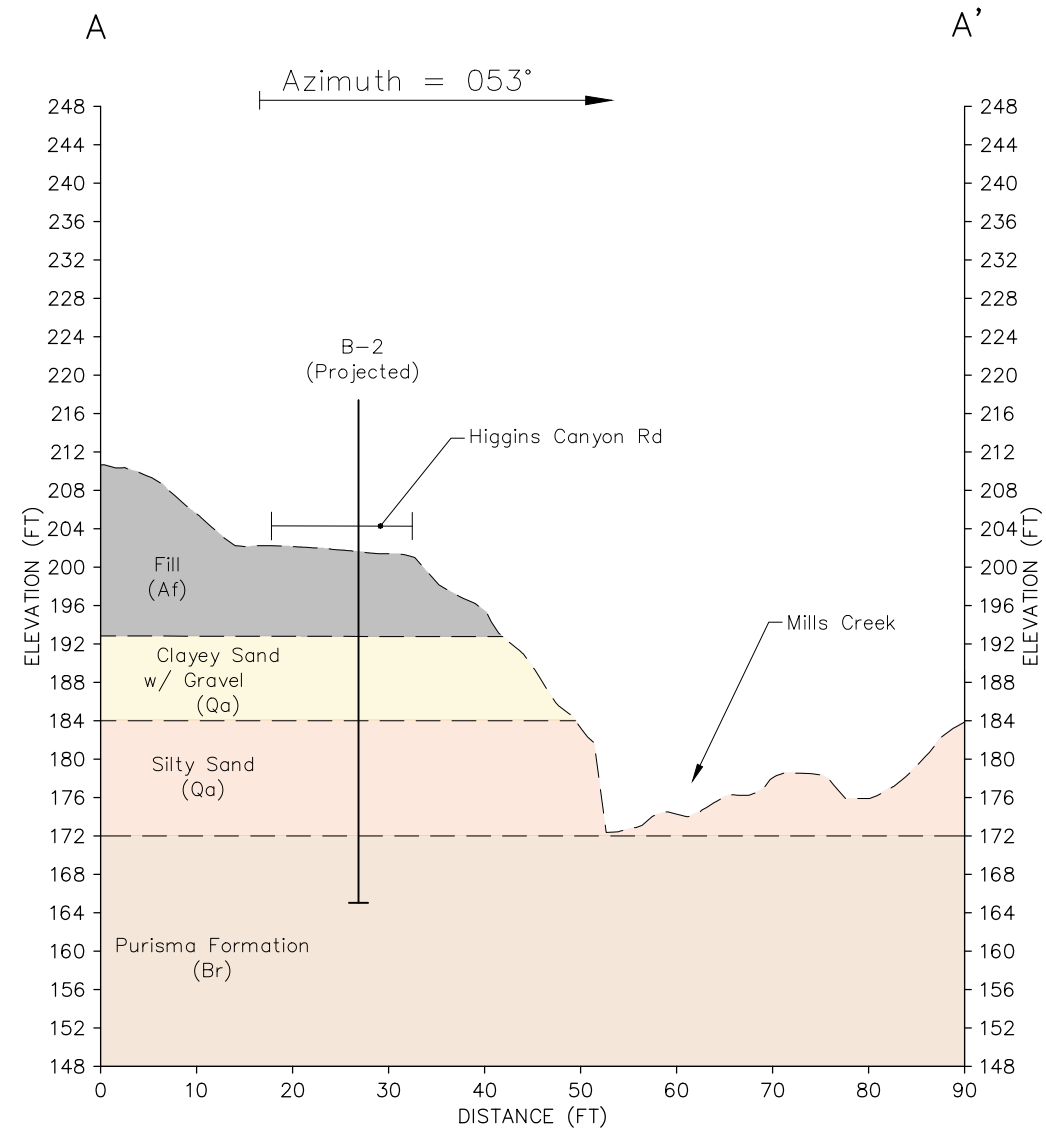
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 HALF MOON BAY, CALIFORNIA

**LIQUEFACTION SUSCEPTIBILITY MAP**

0207545	SEPTEMBER 2023	FIGURE 6
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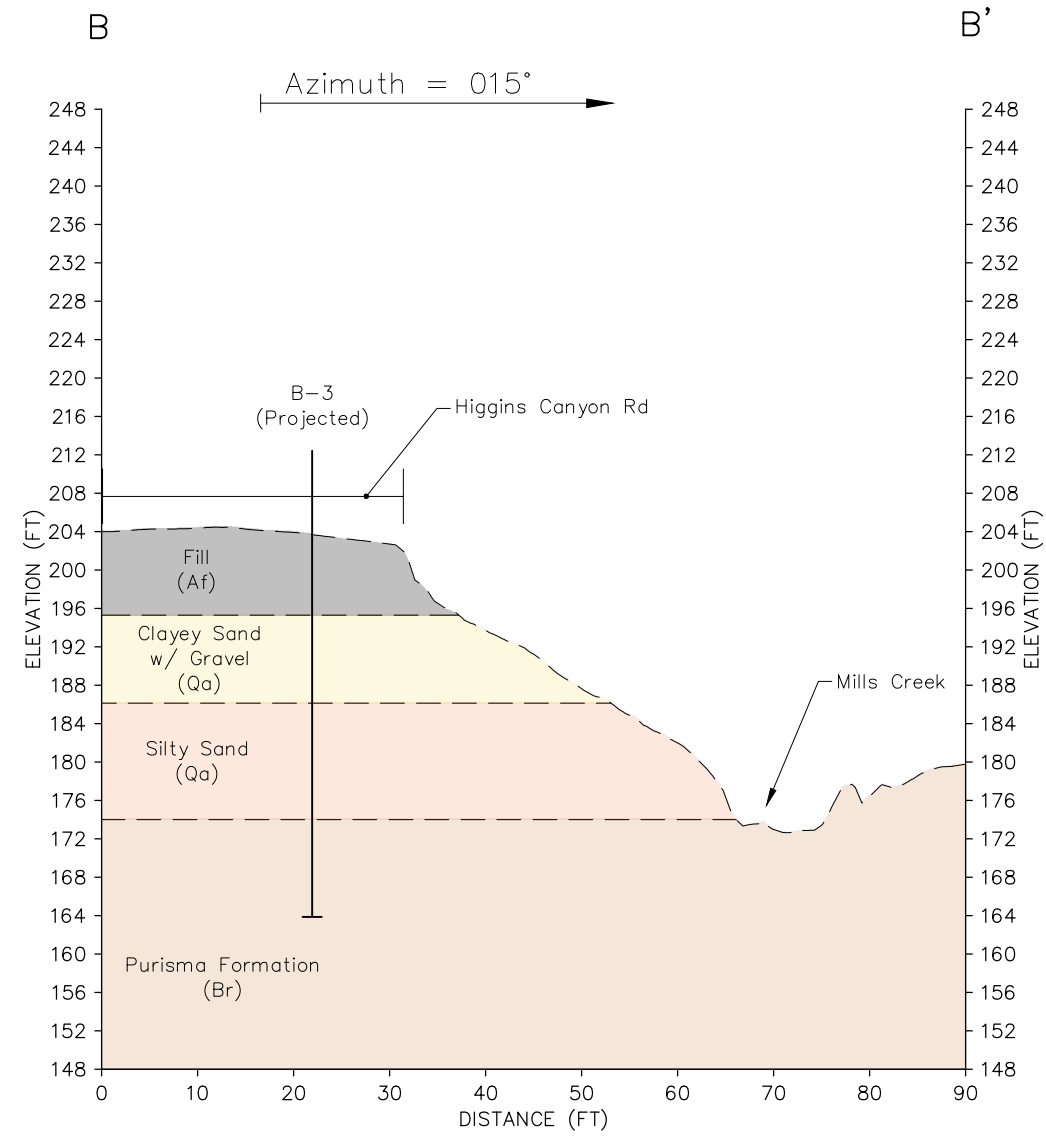
\\holydridch.com\share\granite\2022\0207545-SanMateoCo-1780HigginsCynRdRefWall\AutoCAD\Figures\Fig2-StePlan.dwg 9-15-23 04:25:34 PM kdrozynska



CROSS SECTION A-A'

**LEGEND**

Af	-	Artificial Fill
Qa	-	Quaternary Alluvium
Br	-	Bedrock



CROSS SECTION B-B'

- REFERENCES**
- EXISTING SURFACE PROFILE DERIVED FROM LIDAR SCAN; UAV FLIGHT PERFORMED ON 2/17/2023.



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**2023 SLIP-OUT REPAIR**  
 NEAR 1780 HIGGINS CANYON RD  
 HALF MOON BAY, CALIFORNIA

**CROSS SECTIONS**

0207545	SEPTEMBER 2023	FIGURE 7
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## **Appendix A. Site Photos**



Photo 1: Northwest-facing view of the slip-out area. The photo was taken on March 8<sup>th</sup>, 2023.



Photo 2: West-facing view of the slip-out area. The photo was taken on March 8<sup>th</sup>, 2023.



**Photo 3: West-facing view of the slip-out area. Note the two exposed utility lines in the scarp area (see yellow arrow). The photo was taken on March 8<sup>th</sup>, 2023.**



**Photo 4: East-facing view of the slip-out area. Note the two exposed utility lines in the scarp area. The photo was taken on March 8<sup>th</sup>, 2023.**



**Photo 5: Northeast-facing view of the slip-out area. Note the exposed utility line (see yellow arrow). Also note exposed riprap in slope fill (orange arrow). The photo was taken on March 8<sup>th</sup>, 2023.**



**Photo 6: Southeast-facing view of the slip-out area. The photo was taken on March 8<sup>th</sup>, 2023.**



Photo 7: East-facing view of the slip-out area. The photo was taken on March 8<sup>th</sup>, 2023.



Photo 8: Northwest-facing view of the slip-out area. The photo was taken on March 8<sup>th</sup>, 2023.

## **Appendix B. Boring Logs**

# UNIFIED SOIL CLASSIFICATION SYSTEM (ASTM D-2487)

Field Identification		Group Symbols	Typical Names	Laboratory Classification Criteria			
<b>Coarse-Grained Soils</b> More than 50% of material is retained on the No. 200 sieve.	<b>Gravels</b> More than 50% coarse fraction retained on the No. 4 sieve	Clean Gravels	<b>GW</b>	Well-graded gravels, gravel-sand mixtures, little or no fines	<b>CLASSIFICATION OF GRAVELS &amp; SANDS WITH 5% TO 12% FINES REQUIRES DUAL SYMBOLS</b> Gravel/Silty Gravel Gravel/Clayey Gravel Sand/Silty Sand Sand/Clayey Sand	$C_u = D_{60} \div D_{10} \geq 4$ and $C_c = (D_{30})^2 \div (D_{10} \times D_{60}) \geq 1 \text{ \& } \leq 3$	
		< 5% Fines	<b>GP</b>	Poorly graded gravels, gravel-sand mixtures, little or no fines		$C_u = D_{60} \div D_{10} < 4$ and/or $C_c = (D_{30})^2 \div (D_{10} \times D_{60}) < 1 \text{ \& } > 3$	
		Gravels with Fines	<b>GM</b>	Silty gravels, poorly graded gravel-sand-silt mixtures		Fines classify as ML or MH	If fines classify as CL-ML, use dual symbol GC/GM
			<b>GC</b>	Clayey gravels, poorly graded gravel-sand-clay mixtures		Fines classify as CL or CH	
	<b>Sands</b> More than 50% coarse fraction passes the No. 4 sieve	Clean Sands	<b>SW</b>	Well-graded sands, gravelly sands, little or no fines		$C_u = D_{60} \div D_{10} \geq 6$ and $C_c = (D_{30})^2 \div (D_{10} \times D_{60}) \geq 1 \text{ \& } \leq 3$	
		< 5% Fines	<b>SP</b>	Poorly graded sands, gravelly sands, little or no fines		$C_u = D_{60} \div D_{10} < 6$ and/or $C_c = (D_{30})^2 \div (D_{10} \times D_{60}) < 1 \text{ \& } > 3$	
		Sands with Fines	<b>SM</b>	Silty sands, poorly graded sand-silt mixtures		Fines classify as ML or MH	If fines classify as CL-ML, use dual symbol SC/SM
			<b>SC</b>	Clayey sands, poorly graded sand-clay mixtures		Fines classify as CL or CH	

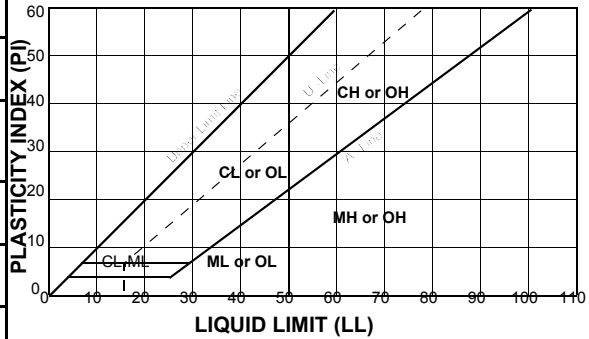
**Identification Procedures on Percentage Passing the No. 40 Sieve**

<b>Fine-Grained Soils</b> More than 50% of material passes the No. 200 sieve.	<b>Silts &amp; Clays</b> Liquid Limit less than 50%	<b>ML</b>	Inorganic silts, very fine sands, rock flour, silty or clayey fine sands with slight plasticity
		<b>CL</b>	Inorganic clays of low to medium plasticity, gravelly, sandy, and/or silty clays, lean clays
		<b>OL</b>	Organic silts, organic silty clays of low plasticity
	<b>Silts &amp; Clays</b> Liquid Limit greater than 50%	<b>MH</b>	Inorganic silts, micaceous or diatomaceous fine sandy/silty soil, elastic silts
		<b>CH</b>	Inorganic clays of high plasticity, fat clays
		<b>OH</b>	Organic clays of medium to high plasticity
<b>HIGHLY ORGANIC SOILS</b>		<b>PT</b>	Peat and other highly organic soils

### PLASTICITY CHART

**For Classification of Fine-Grained Soils and Fine-Grained Fraction of Coarse-Grained Soils**

Equation of "A"-Line:  $PI = 4 @ LL = 4 \text{ to } 25.5$ , then  $PI = 0.73 \times (LL - 20)$   
 Equation of "U"-Line:  $LL = 16 @ PI = 0 \text{ to } 7$ , then  $PI = 0.9 \times (LL - 8)$



### KEY TO SAMPLER TYPES AND OTHER LOG SYMBOLS

- CS** California Standard Sampler
- CM** California Modified Sampler
- SPT** Standard Penetration Test Sampler
- SHL** Shelby Tube Sampler
- BU** Bulk Sample
- LL** Liquid Limit of Sample (ASTM D-4318)
- PI** Plasticity Index of Sample (ASTM D-4318)
- Q<sub>u</sub>** Unconfined Compression Test (ASTM D-2166)

- Depth at which Groundwater was Encountered During Drilling
- Depth at which Groundwater was Measured After Drilling
- PP** Pocket Penetrometer Test
- PTV** Pocket Torvane Test
- #200** % of Material Passing the No. 200 Sieve Test (ASTM D-1140)
- PSA** Particle-Size Analysis (ASTM D-422 & D-1140)
- C** Consolidation Test (ASTM D-2435)
- TXUU** Unconsolidated Undrained Compression Test (ASTM D-2850)

### KEY TO SAMPLE INTERVALS

- Length of Sampler Interval with a CS Sampler
- Length of Sampler Interval with a CM Sampler
- Length of Sampler Interval with a SPT Sampler
- Length of Sampler Interval with a SHL Sampler

- Bulk Sample Recovered for Interval Shown (i.e., cuttings)
- Length of Coring Run with Core Barrel Type Sampler
- NR** No Sample Recovered for Interval Shown

## Rock Hardness Descriptions

<b>Very Hard</b>	Cannot be scratched with knife or sharp pick. Breaking of hand specimen requires several hard blows of geologist's pick.
<b>Hard</b>	Can be scratched with knife or pick only with difficulty. Hard blow of hammer required to detach hand specimen.
<b>Moderately Hard</b>	Can be scratched with knife or pick. Gouges or grooves to 1/4-inch deep can be excavated by hard blow of geologist's pick. Hand specimens can be detached by moderate blow.
<b>Medium</b>	Can be grooved or gouged 1/16-inch deep by firm pressure of knife or pick point. Can be excavated in small chips to pieces about 1-inch maximum size by hard blows of the point of a geologist's pick.
<b>Soft</b>	Can be gouged or grooved readily with knife or pick point. Can be excavated in chips to pieces several inches in size by moderate blows of a pick point. Small tin pieces can be broken by finger pressure.
<b>Very Soft</b>	Can be carved with knife. Can be excavated readily with point of pick. Pieces 1-inch or more in thickness can be broken with finger pressure. Can be scratched readily by fingernail.

## Bedding Thickness & Joint/Fracture Spacing Descriptions

Centimeters	Inches	Bedding	Joints/Fractures
< 2	< ¾	Laminated	Extremely Close
2-5	¾-2	Very Thin	Very Close
5-30	2-12	Thin	Close
30-90	12-36	Medium	Moderate
90-300	36-120	Thick	Wide
> 300	> 120	Very Thick	Very Wide

## Rock Weathering Descriptions

<b>Fresh</b>	Rock fresh, crystals bright, few joints may show slight staining. Rock rings under hammer if crystalline.
<b>Very Slight</b>	Rock generally fresh, joints may show thin clay coatings, crystals in broken face show bright. Rock rings under hammer if crystalline.
<b>Slight</b>	Rock generally fresh, joints stained, and discoloration extends into rock up to 1 inch. Joints may contain clay. In granitoid rocks some occasional feldspar crystals are dulled and discolored. Crystalline rocks ring under hammer.
<b>Moderate</b>	Significant portions of rock show discoloration and weathering effects. In granitoid rocks, most feldspars are dull and discolored; some show clayey. Rock has dull sound under hammer and shows significant loss of strength as compared with fresh rock.
<b>Moderately Severe</b>	All rock except quartz discolored or stained. In granitoid rocks, all feldspars dull and discolored and majority show kaolinization. Rock shows severe loss of strength and can be excavated with geologist's pick. Rock goes "clunk" when struck.
<b>Severe</b>	All rock except quartz discolored or stained. Rock "fabric" clear and evident, but reduced in strength to strong soil. In granitoid rocks, all feldspars kaolinized to some extent. Some fragments of strong rock usually left.
<b>Very Severe</b>	All rock except quartz discolored or stained. Rock "fabric" discernible. But mass effectively reduced to "soil" with only fragments of strong rock remaining.
<b>Complete</b>	Rock reduced to "soil." Rock "fabric" not discernible or discernible only in small scattered locations. Quartz may be present as dikes or stringers.

The above Bedrock Characteristics are based on the ASCE Manual No. 56, "Subsurface Investigation For Design And Construction Of Foundations Of Buildings," 1976.










CLIENT San Mateo County

PROJECT NAME 1780 Higgins Canyon Road Slope Stabilization



PROJECT NUMBER 0207545

PROJECT LOCATION San Mateo County

**LITHOLOGIC SYMBOLS**  
*(Unified Soil Classification System)*

-  ASPHALT: Asphalt
-  BEDROCK: Bedrock
-  CL: USCS Low Plasticity Clay
-  GC: USCS Clayey Gravel
-  ML: USCS Silt
-  SC: USCS Clayey Sand
-  SM: USCS Silty Sand
-  SP: USCS Poorly-graded Sand
-  SP-SM: USCS Poorly-graded Sand with Silt

**SAMPLER SYMBOLS**

-  California Modified Sampler
-  Standard Penetration Test

**WELL CONSTRUCTION SYMBOLS**

**ABBREVIATIONS**


- |                                      |   |
|--------------------------------------|---|
| LL - LIQUID LIMIT (%)                | TV - TORVANE                                    |
| PI - PLASTIC INDEX (%)               | PID - PHOTOIONIZATION DETECTOR                  |
| W - MOISTURE CONTENT (%)             | UC - UNCONFINED COMPRESSION                     |
| DD - DRY DENSITY (PCF)               | ppm - PARTS PER MILLION                         |
| NP - NON PLASTIC                     | ∇ - Water Level at Time Drilling, or as Shown   |
| -200 - PERCENT PASSING NO. 200 SIEVE | ▼ - Water Level at End of Drilling, or as Shown |
| PP - POCKET PENETROMETER (TSF)       | ∇ - Water Level After 24 Hours, or as Shown     |

**CLIENT** San Mateo County **PROJECT NAME** 1780 Higgins Canyon Road Slope Stabilization  
**PROJECT NUMBER** 0207545 **PROJECT LOCATION** San Mateo County  
**DATE STARTED** 1/25/2023 **COMPLETED** 1/25/2023 **GROUND ELEVATION** 200 ft **DATUM** NAVD88 **HOLE SIZE** 6 in.  
**DRILLING CONTRACTOR** Britton Exploration **COORDINATES: LATITUDE** 37.445933 **LONGITUDE** -122.404442  
**DRILLING RIG/METHOD** CME-55/6-in. Solid Flight Auger **GROUNDWATER AT TIME OF DRILLING** 28.0 ft / Elev 172.0 ft  
**LOGGED BY** C. Rodil **CHECKED BY** K. Loeb **GROUNDWATER AT END OF DRILLING** --- Not Measured  
**HAMMER TYPE** 140 lb hammer with 30 in. cathead **GROUNDWATER AFTER DRILLING** 32.0 ft / Elev 168.0 ft

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE	BLOW COUNTS (FIELD VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS			FINES CONTENT (%)	
								LIQUID LIMIT (%)	PLASTIC LIMIT (%)	PLASTICITY INDEX (%)		
0		Approximately 4" Asphaltic Concrete over 4.5" Aggregate Base										
5		Lean CLAY (CL): very dark grayish brown (10YR 3/2), moist, stiff, low plasticity, trace fine gravel [FILL] Corrosion test @ 1.5 ft Lean CLAY w/ Sand (CL): dark yellowish brown (10YR 4/6), dry, medium stiff, low plasticity, fine to medium sand, trace fine gravel	CM	7-9-8	3.5 3.5	105	21					
			SPT	5-4-3								
10		Lean CLAY (CL): yellowish brown (10YR 5/4), moist, very stiff, low plasticity  -some gravel	CM	6-10-17	3.5 3.5	105	21					
			SPT	4-8-13								
15		Clayey SAND w/ Gravel (SC): light yellowish brown (10YR 6/4) to yellowish brown (10YR 5/6), moist, medium dense, fine to medium sand, fine gravel up to 0.5" [Alluvium]  -gravel up to 1.5"	CM	6-12-17	3.5 3.5	105	16					
			CM	11-22-33								
20		Lean CLAY w/ Sand (CL): olive yellow (2.5Y 6/6), moist, stiff to very stiff, low plasticity, fine to medium sand, increases in sand with depth TXUU test @ 21.0 ft	CM	8-11-13	3.25	94	30	46	19	27	85	
			CM	20-19-32								
25		Poorly Graded SAND (SP): reddish yellow (7.5Y 6/6), moist, dense, few fine angular gravel up to 0.75"  Poorly Graded SAND w/ Silt (SP-SM): yellowish brown (10YR 5/4), moist, dense, fine to medium sand, some silt, some organics -rig chatter	CM	20-19-32	3.25	94	30	46	19	27	85	
			CM	13-24-31								
30		Poorly Graded SAND w/ Gravel (SP): greenish gray (GLEYS 1 5/1), moist to wet, dense, fine to medium sand, few coarse sand, angular gravel up to 0.75"	CM	13-24-31	3.25	94	30	46	19	27	85	
			CM	13-24-31								
35												

(Continued Next Page)

**CLIENT** San Mateo County **PROJECT NAME** 1780 Higgins Canyon Road Slope Stabilization
**PROJECT NUMBER** 0207545 **PROJECT LOCATION** San Mateo County

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE	BLOW COUNTS (FIELD VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS			FINES CONTENT (%)
								LIQUID LIMIT (%)	PLASTIC LIMIT (%)	PLASTICITY INDEX (%)	
35			CM	15-50/3"							

Sandy CLAYSTONE (Bedrock): greenish gray (GLEY 1 5/1), dry, soft to medium hardness, moderately severe to moderate weathering [Purisima Formation]

Bottom of borehole at 36.0 ft. Borehole backfilled with neat cement grout.

<b>CLIENT</b> <u>San Mateo County</u>	<b>PROJECT NAME</b> <u>1780 Higgins Canyon Road Slope Stabilization</u>
<b>PROJECT NUMBER</b> <u>0207545</u>	<b>PROJECT LOCATION</b> <u>San Mateo County</u>
<b>DATE STARTED</b> <u>1/25/2023</u> <b>COMPLETED</b> <u>1/25/2023</u>	<b>GROUND ELEVATION</b> <u>202 ft</u> <b>DATUM</b> <u>NAVD88</u> <b>HOLE SIZE</b> <u>6 in.</u>
<b>DRILLING CONTRACTOR</b> <u>Britton Exploration</u>	<b>COORDINATES: LATITUDE</b> <u>37.446008</u> <b>LONGITUDE</b> <u>-122.404583</u>
<b>DRILLING RIG/METHOD</b> <u>CME-55/6-in. Solid Flight Auger</u>	<b>GROUNDWATER AT TIME OF DRILLING</b> <u>---</u>
<b>LOGGED BY</b> <u>C. Rodil</u> <b>CHECKED BY</b> <u>K. Loeb</u>	<b>GROUNDWATER AT END OF DRILLING</b> <u>--- Not Measured</u>
<b>HAMMER TYPE</b> <u>140 lb hammer with 30 in. cathead</u>	<b>GROUNDWATER AFTER DRILLING</b> <u>--- Not Measured</u>

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE	BLOW COUNTS (FIELD VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS			FINES CONTENT (%)
								LIQUID LIMIT (%)	PLASTIC LIMIT (%)	PLASTICITY INDEX (%)	
0		Approximately 8" Asphaltic Concrete over 4" Aggregate Base									
5	[Diagonal Hatching]	Lean CLAY w/ Sand (CL): dark yellowish brown (10YR 3/4), dry, very stiff to hard, fine to medium sand, few fine subrounded gravel up to 0.75" [FILL] R-Value test @ 1 - 5 ft -becomes yellowish brown (10YR 5/4)	CM	8-7-10	4.0	100	25				
			SPT	4-6-7	4.5						
10	[Diagonal Hatching]	-decrease in sand and gravel, becomes moist	CM	5-11-15	3.25	102	16				33
			SPT	4-6-9	4.5						
15	[Diagonal Hatching]	Clayey SAND w/ Gravel (SC): brown (10YR 4/3) to yellowish brown (10YR 5/4), moist, medium dense, fine to coarse sand, gravel up to 2.0" [Alluvium]	CM	6-11-13	1.5	94	29	47	18	29	
					1.25						
20	[Diagonal Hatching]	Lean CLAY (CL): light yellowish brown (2.5Y 6/4), moist, stiff, low to medium plasticity, trace fine sand TXUU test @ 16.0 ft -becomes dark grayish brown (2.5Y 4/2), increase in plasticity	CM	2-4-7	1.5	98	21				54
					1.25						
25	[Diagonal Hatching]	Clayey GRAVEL w/ Sand (GC): yellowish brown (10YR 5/6) to dark yellowish brown (10YR 3/4), moist, very dense, fine to medium sand, gravel up to 0.75"	CM	13-28-43		98	21				54
30	[Diagonal Hatching]	Sandy SILT w/ Gravel (ML): greenish gray (GLEYS 1 5/1), moist, very dense, fine to medium sand, few coarse sand, few fine gravel up to 0.75"	CM	13-28-50		98	21				54
35											


(Continued Next Page)

**CLIENT** San Mateo County

**PROJECT NAME** 1780 Higgins Canyon Road Slope Stabilization

**PROJECT NUMBER** 0207545

**PROJECT LOCATION** San Mateo County


DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE	BLOW COUNTS (FIELD VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS			FINES CONTENT (%)
								LIQUID LIMIT (%)	PLASTIC LIMIT (%)	PLASTICITY INDEX (%)	
35		Sandy CLAYSTONE (Bedrock): greenish gray (GLEY 1 5/1), moist, soft to medium hardness, moderately severe weathering [Purisima Formation]	CM	28-50/4"							
Bottom of borehole at 36.5 ft. Borehole backfilled with neat cement grout.											

**CLIENT** San Mateo County **PROJECT NAME** 1780 Higgins Canyon Road Slope Stabilization  
**PROJECT NUMBER** 0207545 **PROJECT LOCATION** San Mateo County  
**DATE STARTED** 1/25/2023 **COMPLETED** 1/25/2023 **GROUND ELEVATION** 200 ft **DATUM** NAVD88 **HOLE SIZE** 6 in.  
**DRILLING CONTRACTOR** Britton Exploration **COORDINATES: LATITUDE** 37.446136 **LONGITUDE** -122.40468  
**DRILLING RIG/METHOD** CME-55/6-in. Solid Flight Auger **GROUNDWATER AT TIME OF DRILLING** --- Not Encountered  
**LOGGED BY** C. Rodil **CHECKED BY** K. Loeb **GROUNDWATER AT END OF DRILLING** --- Not Encountered  
**HAMMER TYPE** 140 lb hammer with 30 in. cathead **GROUNDWATER AFTER DRILLING** --- Not Encountered

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE	BLOW COUNTS (FIELD VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS			FINES CONTENT (%)
								LIQUID LIMIT (%)	PLASTIC LIMIT (%)	PLASTICITY INDEX (%)	
0		Approximately 4" Asphaltic Concrete over 5" Aggregate Base									
		Lean CLAY (CL): light olive brown (2.5Y 5/4), moist, stiff, low plasticity [FILL]	CM	3-5-10	2.0						
		-trace fine gravel	SPT	3-5-6	2.0						
5											
		TXUU test @ 6.0 ft	CM	7-8-11	3.25						
		-becomes sandy, fine to medium sand, increase in gravel	SPT	7-8-10	2.5	110	18	34	15	19	
		-rig chatter									
10		Clayey GRAVEL w/ Sand (GC): brownish yellow (10YR 6/6) to yellowish brown (10YR 5/6), moist, dense, fine to coarse sand, subrounded gravel up to 1.5"	CM	10-15-28		110	18				
15		-becomes very dense	CM	14-22-38		104	21				
20		Silty SAND (SM): light yellowish brown (2.5Y 6/4), moist, very dense, fine to medium sand, few fine gravel	CM	20-44-50		104	20				
25		Silty SAND w/ Gravel (SM): greenish gray (GLEYS 1 5/1) and dark yellowish brown (10YR 4/4), moist, very dense, fine to coarse sand, angular gravel up to 0.75"	CM	22-40-50/4"							19
		-becomes very dark grayish brown (10YR 3/2)									
30		Sandy CLAYSTONE (Bedrock): greenish gray (GLEYS 1 5/1), dry, medium to moderately hard, moderately severe to moderate weathering [Purisima Formation]	CM	50/5"							
35											

(Continued Next Page)

**CLIENT** San Mateo County **PROJECT NAME** 1780 Higgins Canyon Road Slope Stabilization
**PROJECT NUMBER** 0207545 **PROJECT LOCATION** San Mateo County

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE	BLOW COUNTS (FIELD VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS			FINES CONTENT (%)
								LIQUID LIMIT (%)	PLASTIC LIMIT (%)	PLASTICITY INDEX (%)	
35											
		Sandy CLAYSTONE (Bedrock): greenish gray (GLEY 1 5/1), dry, medium to moderately hard, moderately severe to moderate weathering [Purisima Formation] <i>(continued)</i>	CM	50/5"		100	19				
40			CM	50/4"							

Bottom of borehole at 40.5 ft. Borehole backfilled with neat cement grout.

## **Appendix C. Laboratory Testing**



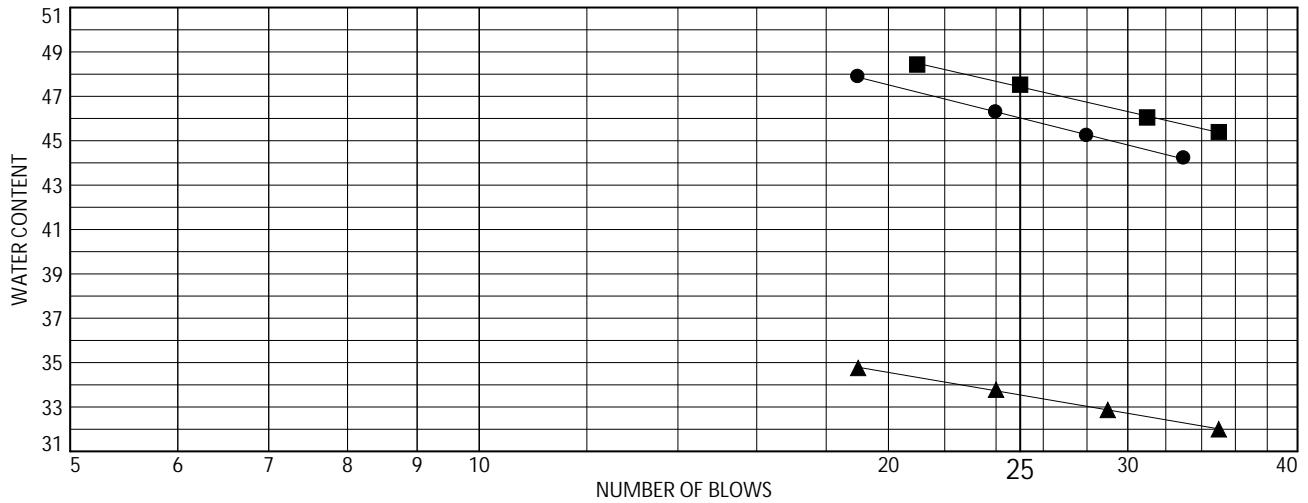
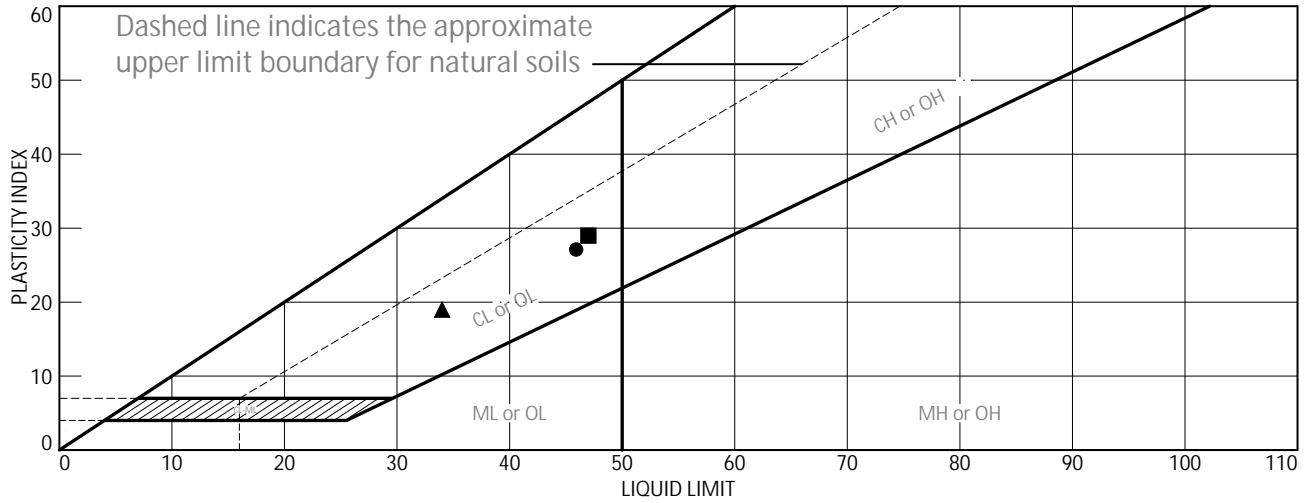
# SUMMARY OF LABORATORY RESULTS

CLIENT San Mateo County PROJECT NAME 1780 Higgins Canyon Road Slope Stabilization  
 PROJECT NUMBER 0207545 PROJECT LOCATION San Mateo County

Borehole	Depth	Date Tested	Liquid Limit	Plastic Limit	Plasticity Index	Maximum Screen Size (mm)	%<#200 Sieve	Classification	Water Content (%)	Dry Density (pcf)	Saturation (%)	Void Ratio
B-1	6.0	2/14/2023							21.3	104.8		
B-1	11.0	2/14/2023							15.9	105.5		
B-1	16.0	2/14/2023							12.2	117.8		
B-1	21.0	2/16/2023	46	19	27				29.5	93.7		
B-1	21.5	2/22/2023				4.75	85					
B-1	31.0	2/14/2023							15.8	111.9		
B-2	6.0	2/14/2023							24.9	100.0		
B-2	10.5	2/14/2023							16.2	102.2		
B-2	11.0	2/14/2023				19	33					
B-2	16.0	2/16/2023	47	18	29				29.0	94.0		
B-2	26.0	2/14/2023				25	54		21.3	98.5		
B-3	6.0	2/16/2023	34	15	19				18.4	110.4		
B-3	11.0	2/14/2023							18.0	110.4		
B-3	16.0	2/14/2023							21.0	104.4		
B-3	21.0	2/14/2023							20.3	104.3		
B-3	26.0	2/14/2023				19	19					
B-3	35.0	2/14/2023							18.9	100.3		



# LIQUID AND PLASTIC LIMITS TEST REPORT (ASTM D4318)



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	Yellowish Brown Lean CLAY w/ Sand	46	19	27			
■	Light Olive Brown Lean CLAY w/ Sand	47	18	29			
▲	Dark Yellowish Brown Lean CLAY w/ Sand, trace Gravel	34	15	19			

Project No. 471-404      Client: Cal Engineering & Geology  
 Project: 1780 Higgins Canyon Road - 0207545

● Source of Sample: B-1      Depth: 21.0'      Sample Number: 1-13  
 ■ Source of Sample: B-2      Depth: 16.0'      Sample Number: 2-10  
 ▲ Source of Sample: B-3      Depth: 6.0'      Sample Number: 3-5

Remarks:

## COOPER TESTING LABORATORY

Figure



## #200 Sieve Wash Analysis ASTM D 1140

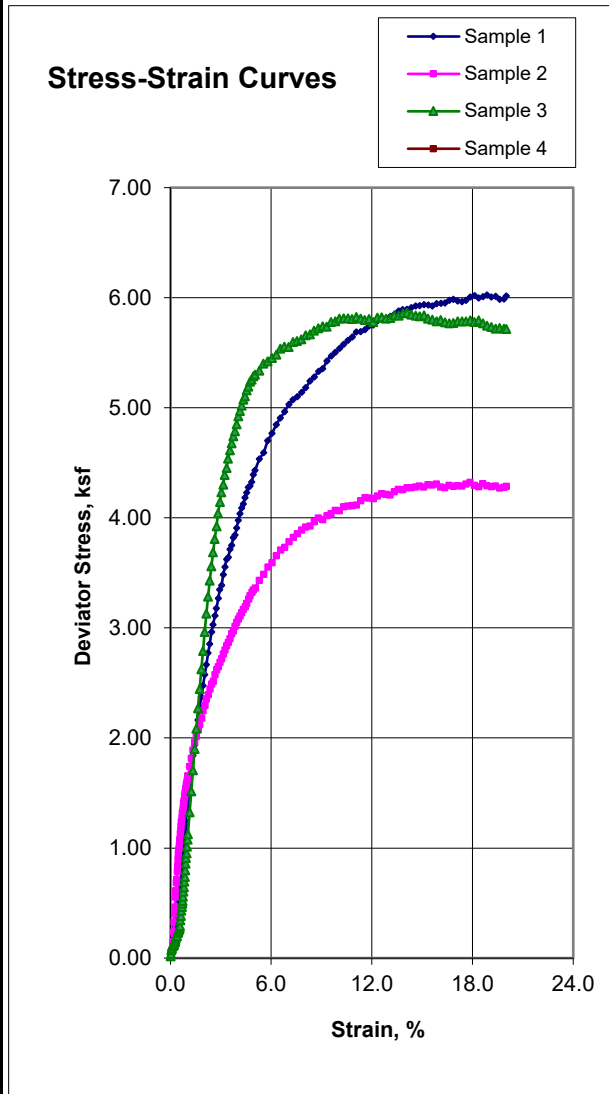
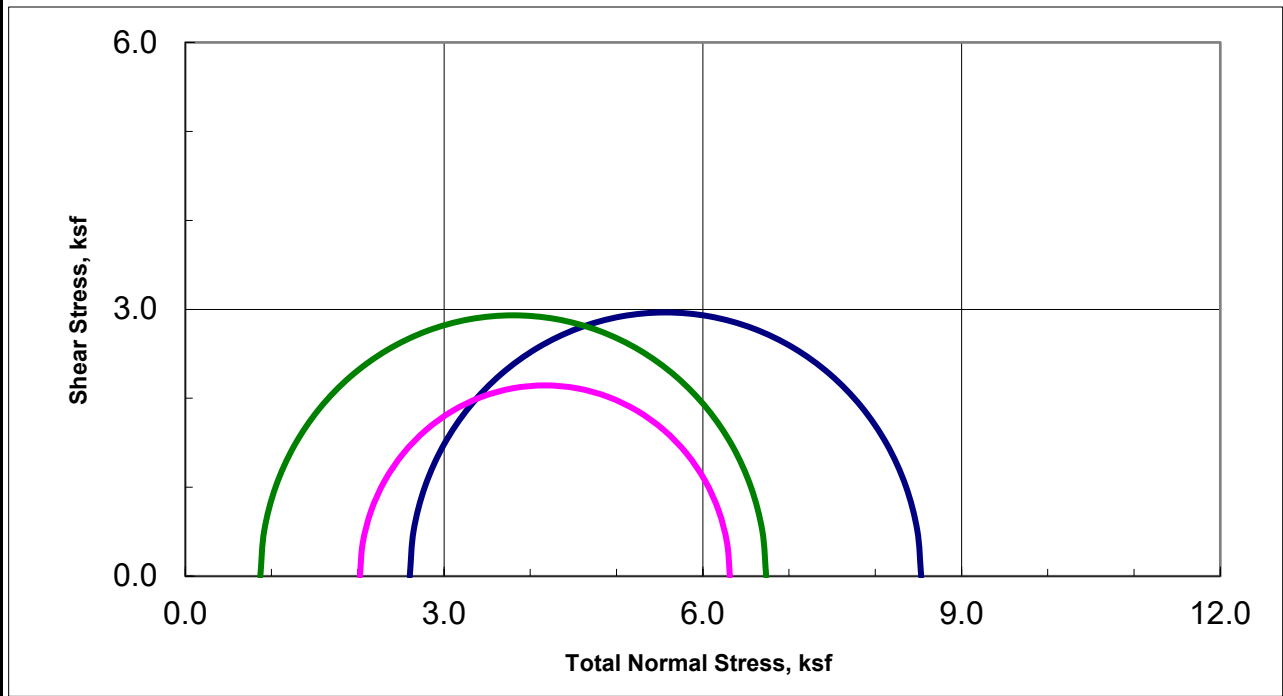
Job No.: 471-404	Project No.: 0207545	Run By: MD
Client: Cal Engineering & Geology	Date: 2/22/2023	Checked By: DC
Project: 1780 Higgins Canyon Road		

<b>Boring:</b>	B-1						
<b>Sample:</b>	1-13						
<b>Depth, ft.:</b>	21.0						
<b>Soil Type:</b>	Yellowish Brown Lean CLAY w/ Sand						
<b>Wt of Dish &amp; Dry Soil, gm</b>	407.9						
<b>Weight of Dish, gm</b>	172.4						
<b>Weight of Dry Soil, gm</b>	235.5						
<b>Wt. Ret. on #4 Sieve, gm</b>	0.0						
<b>Wt. Ret. on #200 Sieve, gm</b>	36.3						
<b>% Gravel</b>	<b>0.0</b>						
<b>% Sand</b>	<b>15.4</b>						
<b>% Silt &amp; Clay</b>	<b>84.6</b>						

Remarks: As an added benefit to our clients, the gravel fraction may be included in this report. Whether or not it is included is dependent upon both the technician's time available and if there is a significant enough amount of gravel. The gravel is always included in the percent retained on the #200 sieve but may not be weighed separately to determine the percentage, especially if there is only a trace amount, (5% or less).



Unconsolidated-Undrained Triaxial Test  
 ASTM D2850



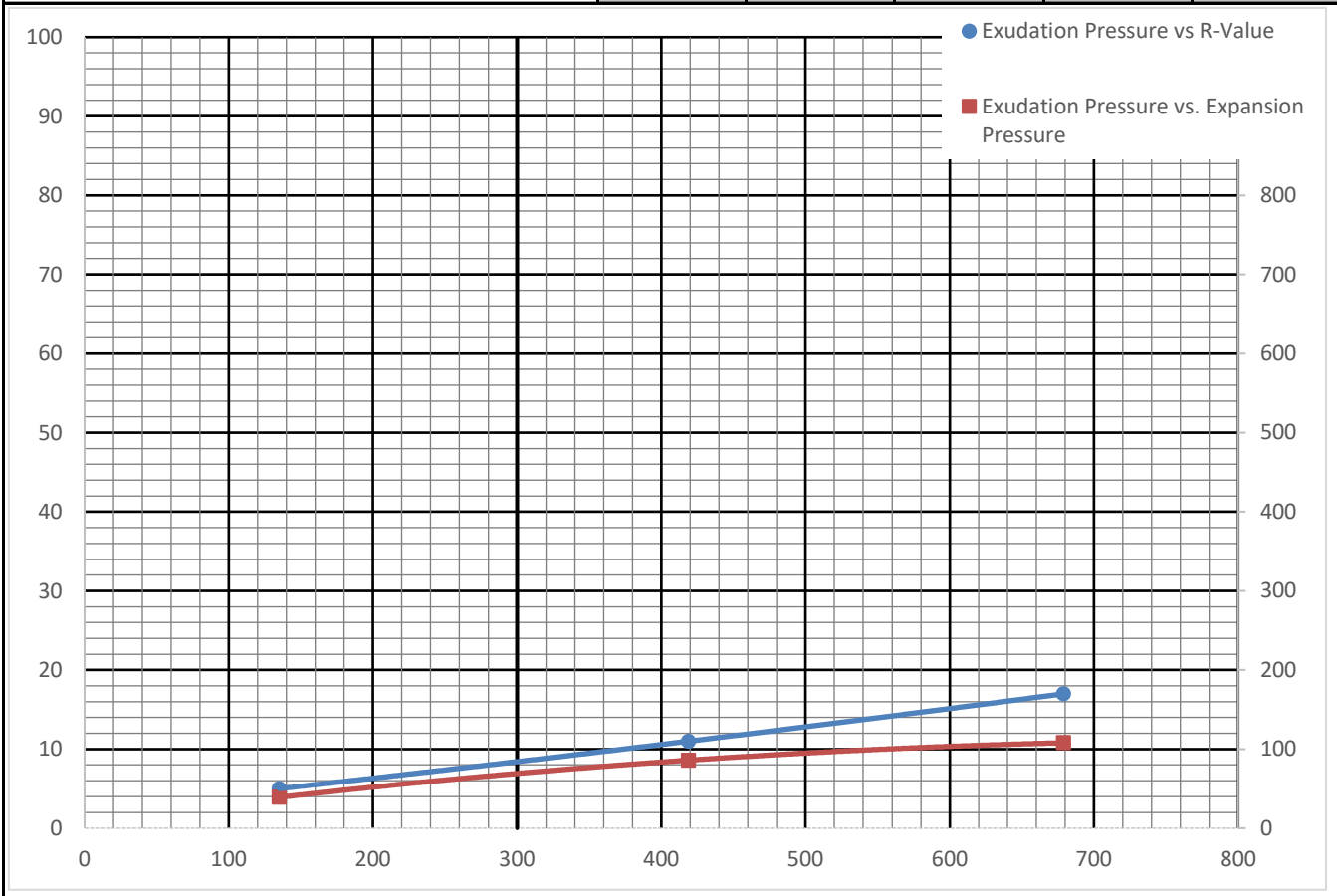
Sample Data				
	1	2	3	4
Moisture %	29.5	29.0	18.4	
Dry Den,pcf	93.7	94.0	110.4	
Void Ratio	0.798	0.793	0.526	
Saturation %	99.8	98.7	94.6	
Height in	5.02	5.00	5.00	
Diameter in	2.41	2.39	2.41	
Cell psi	18.1	14.1	6.0	
Strain %	15.00	15.00	14.10	
Deviator, ksf	5.929	4.290	5.865	
Rate %/min	1.00	1.00	1.00	
in/min	0.050	0.050	0.050	
Job No.:	471-404			
Client:	Cal Engineering & Geology			
Project:	0207545			
Boring:	B-1	B-2	B-3	
Sample:	1-13	2-10	3-5	
Depth ft:	21.0	16.0	6.0	
Visual Soil Description				
Sample #				
1	Yellowish Brown Lean CLAY w/ Sand			
2	Light Olive Brown Lean CLAY w/ Sand			
3	Dark Yellowish Brown Lean CLAY w/ Sand, trace Gravel			
4				
Remarks:				

Note: Strengths are picked at the peak deviator stress or 15% strain which ever occurs first per ASTM D2850.



# R-Value CTM 301

CTL Job No.:	471-404	Boring:	B-2	Reduced By:	RU
Client:	Cal Engineering & Geology	Sample:	Bulk	Checked By:	PJ
Project Number:	207545	Depth:	1-5'	Date:	2/21/2023
Project Name:	1780 Higgins Canyon Road			<b>R-Value</b>	<b>8</b>
Soil Description:	Olive Brown Sandy CLAY				
Remarks:				<b>Expansion Pressure</b>	<b>65</b>
Specimen Designation	A	B	C	D	E
Compactor Foot Pressure (psi)	80	40	30		
Exudation Pressure (psi)	679	419	135		
Exudation Load (lbf)	8533	5265	1696		
Height After Compaction (in)	2.41	2.40	2.48		
Expansion Pressure (psf)	108	86	39		
Stabilometer @ 2000	120	134	148		
Turns Displacement	3.87	3.74	4.18		
R-value	18	11	5		
Corrected R-Value	17	11	5		
Moisture Content (%)	18.0	21.3	25.6		
Wet Density (pcf)	128.4	124.8	121.3		
Dry Density (pcf)	108.8	102.9	96.6		



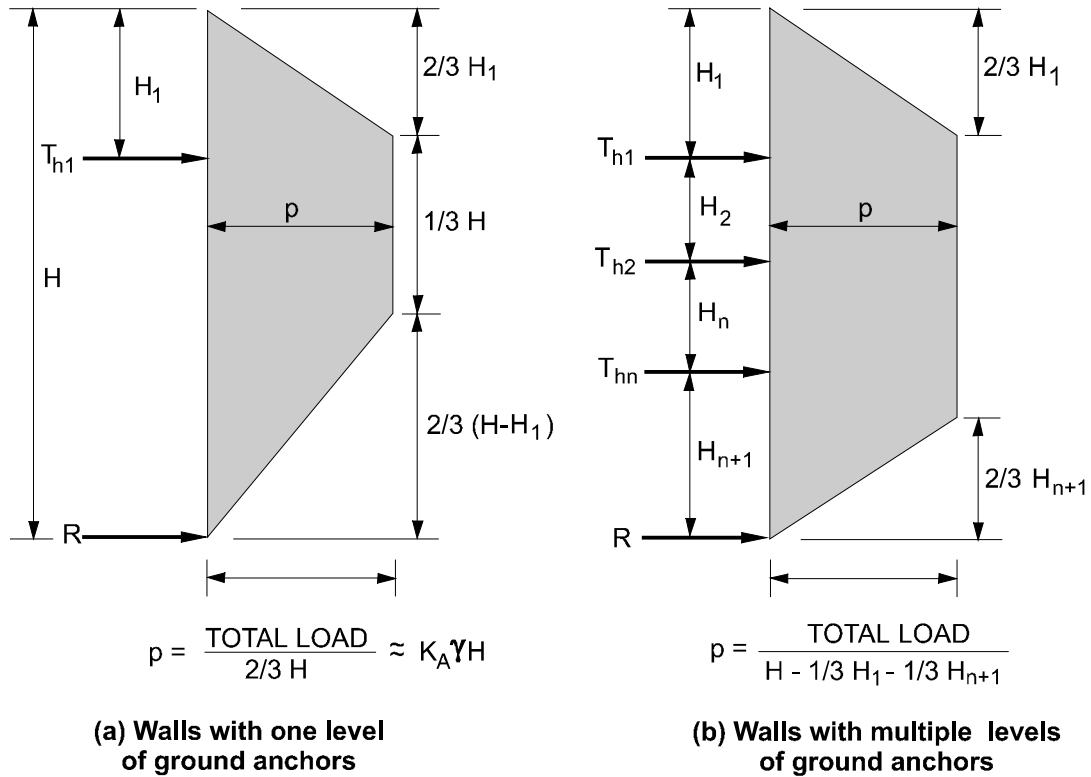


## **Appendix D. Calculations**

$$p = 0.65 K_A \gamma H$$

(Equation 10b)

where  $\phi'$  is the effective stress friction angle of the sand. Using this value of lateral earth pressure, the total lateral earth load from the rectangular apparent earth pressure diagram (figure 23a) for sands is  $0.65 K_a \gamma H^2$ . The recommended apparent earth pressure envelope for single level anchored walls and walls with two or more levels of ground anchors is trapezoidal and is shown in figure 24.



$H_1$  = Distance from ground surface to uppermost ground anchor

$H_{n+1}$  = Distance from base of excavation to lowermost ground anchor

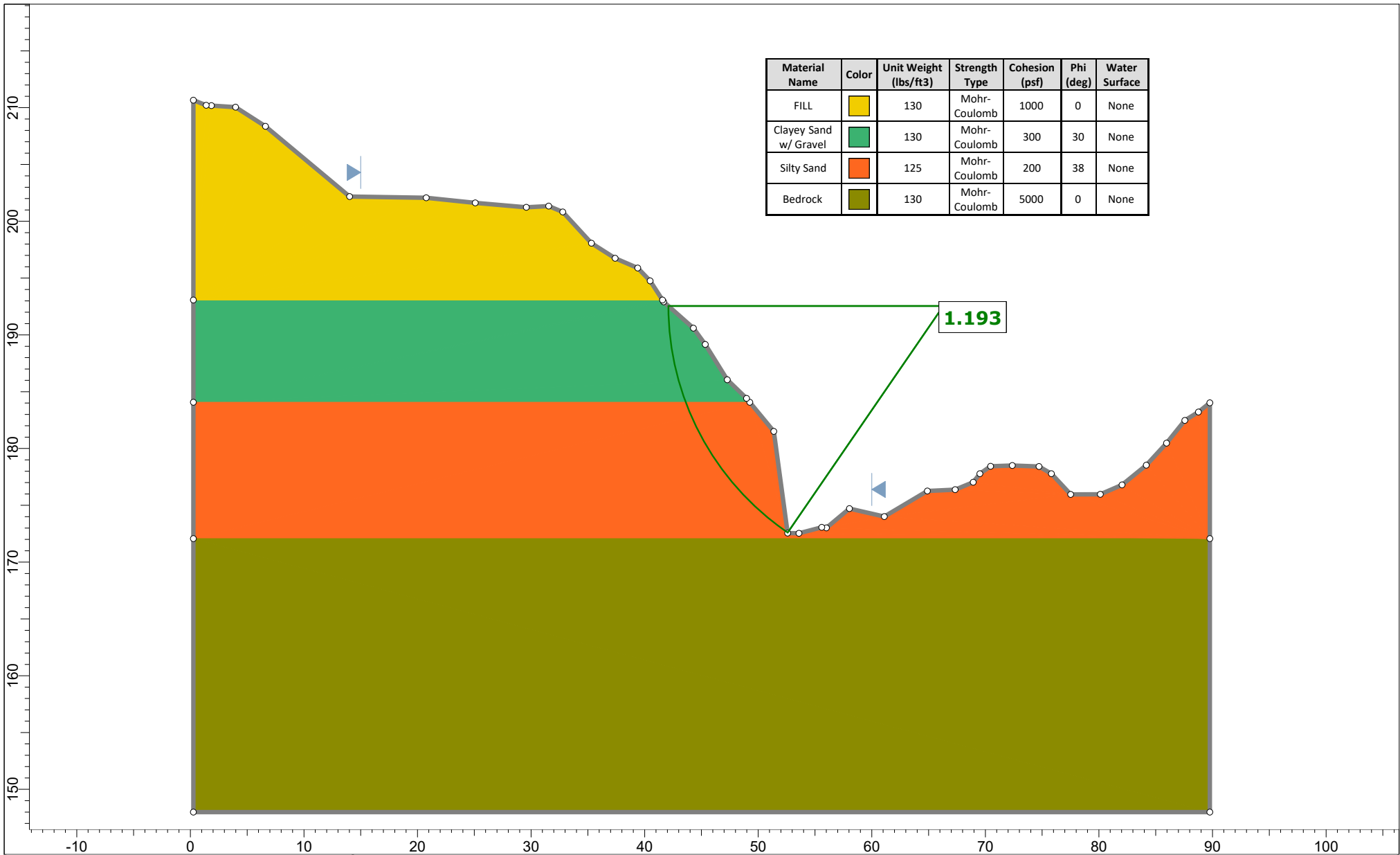
$T_{hi}$  = Horizontal load in ground anchor  $i$

$R$  = Reaction force to be resisted by subgrade (i.e., below base of excavation)

$p$  = Maximum ordinate of diagram


$$\text{TOTAL LOAD} = 0.65 K_A \gamma H^2$$

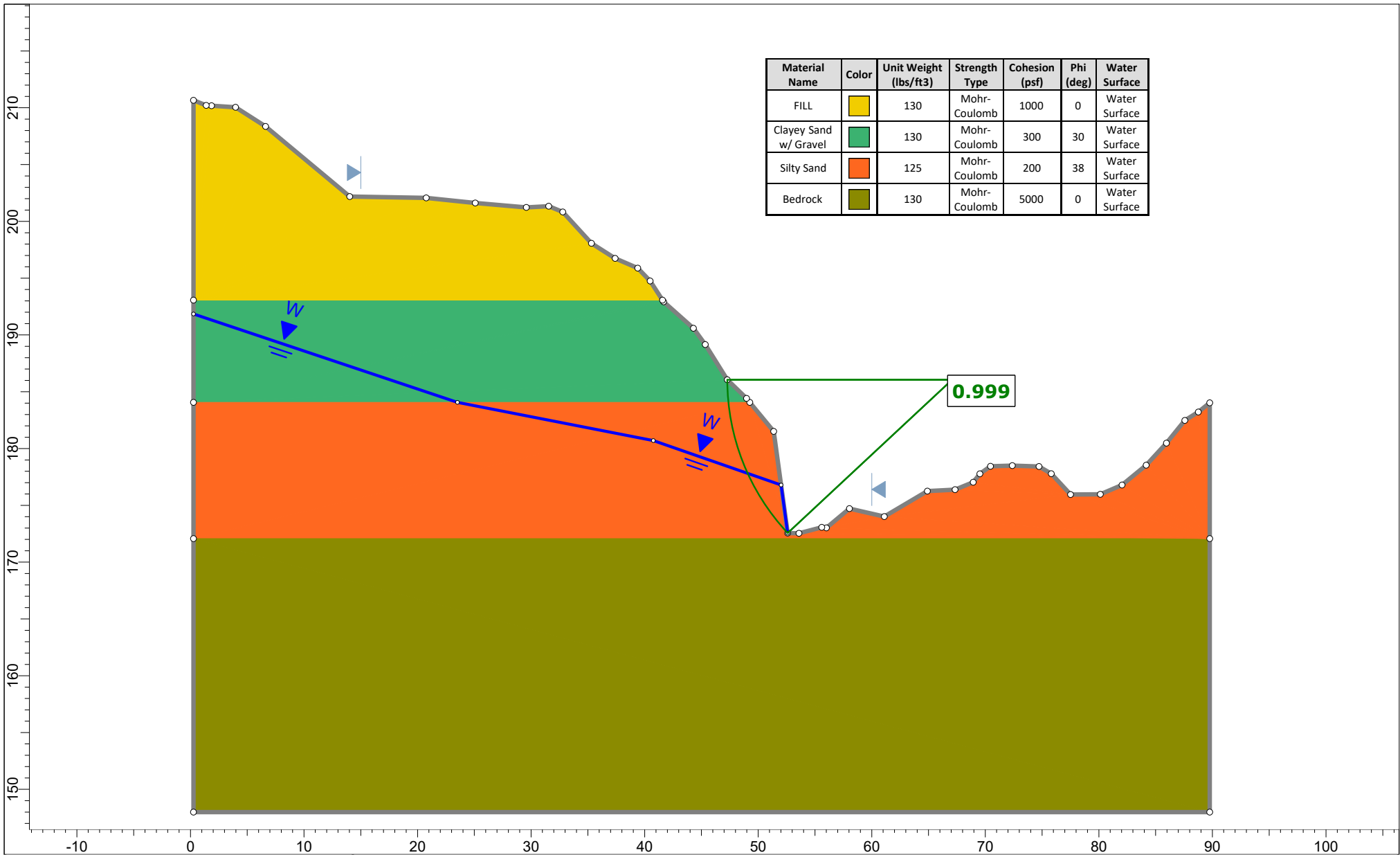
Figure 24. Recommended apparent earth pressure diagram for sands.




Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface
FILL	Yellow	130	Mohr-Coulomb	1000	0	None
Clayey Sand w/ Gravel	Green	130	Mohr-Coulomb	300	30	None
Silty Sand	Orange	125	Mohr-Coulomb	200	38	None
Bedrock	Olive Green	130	Mohr-Coulomb	5000	0	None

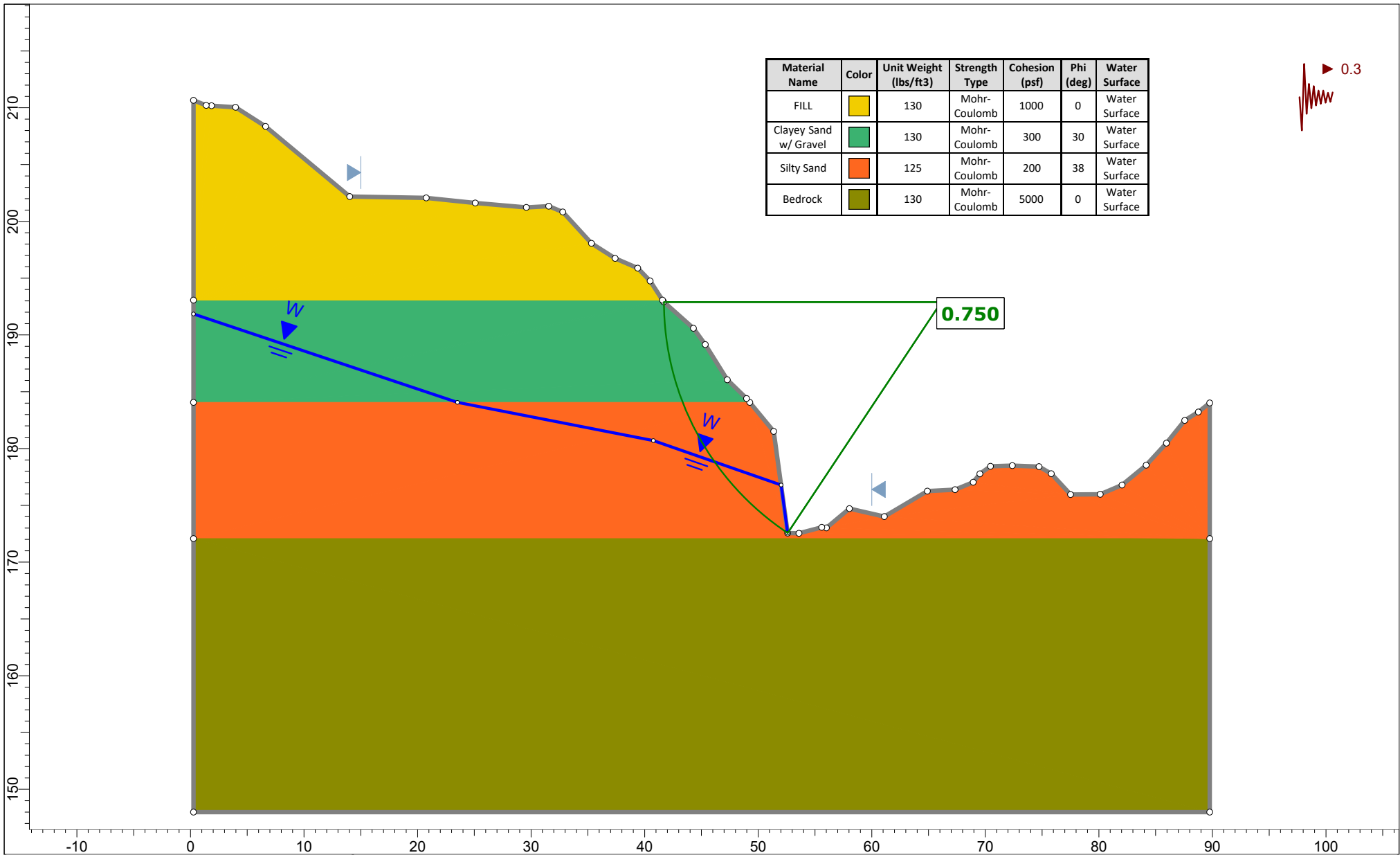
1.193

	<i>Project</i> <b>SLIDE - An Interactive Slope Stability Program</b>	
	<i>Group</i> Existing	<i>Scenario</i> Master Scenario
	<i>Drawn By</i>	<i>Company</i>
	<i>Date</i> 3/14/2023, 10:31:19 AM	<i>File Name</i> Slope Stability-20230411.slmd




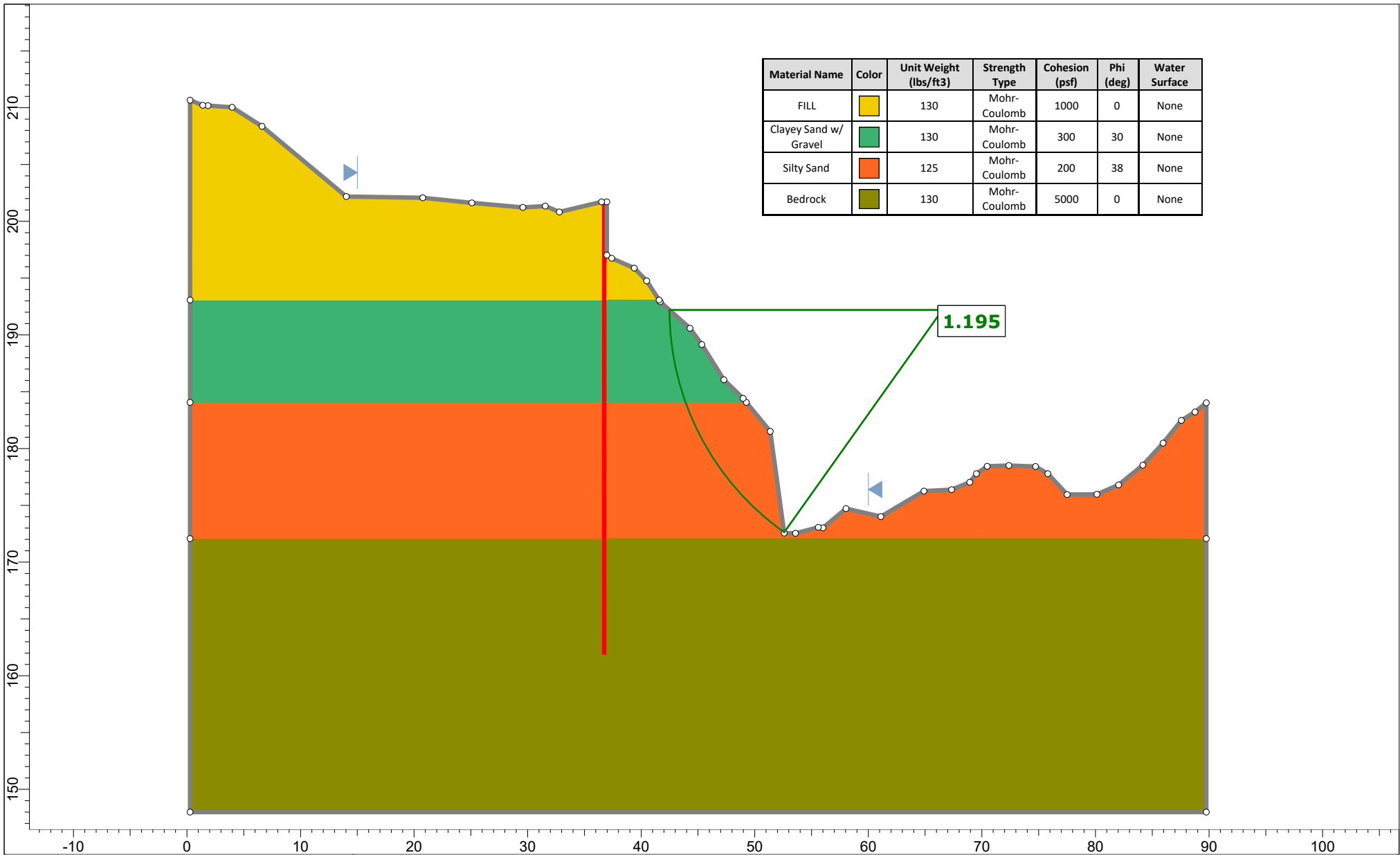
Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface
FILL	Yellow	130	Mohr-Coulomb	1000	0	Water Surface
Clayey Sand w/ Gravel	Green	130	Mohr-Coulomb	300	30	Water Surface
Silty Sand	Orange	125	Mohr-Coulomb	200	38	Water Surface
Bedrock	Olive Green	130	Mohr-Coulomb	5000	0	Water Surface

	<i>Project</i> SLIDE - An Interactive Slope Stability Program	
	<i>Group</i> Existing	<i>Scenario</i> Existing - High GW
	<i>Drawn By</i>	<i>Company</i>
	<i>Date</i> 3/14/2023, 10:31:19 AM	<i>File Name</i> Slope Stability-20230411.slmd




Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface
FILL	Yellow	130	Mohr-Coulomb	1000	0	Water Surface
Clayey Sand w/ Gravel	Green	130	Mohr-Coulomb	300	30	Water Surface
Silty Sand	Orange	125	Mohr-Coulomb	200	38	Water Surface
Bedrock	Olive Green	130	Mohr-Coulomb	5000	0	Water Surface

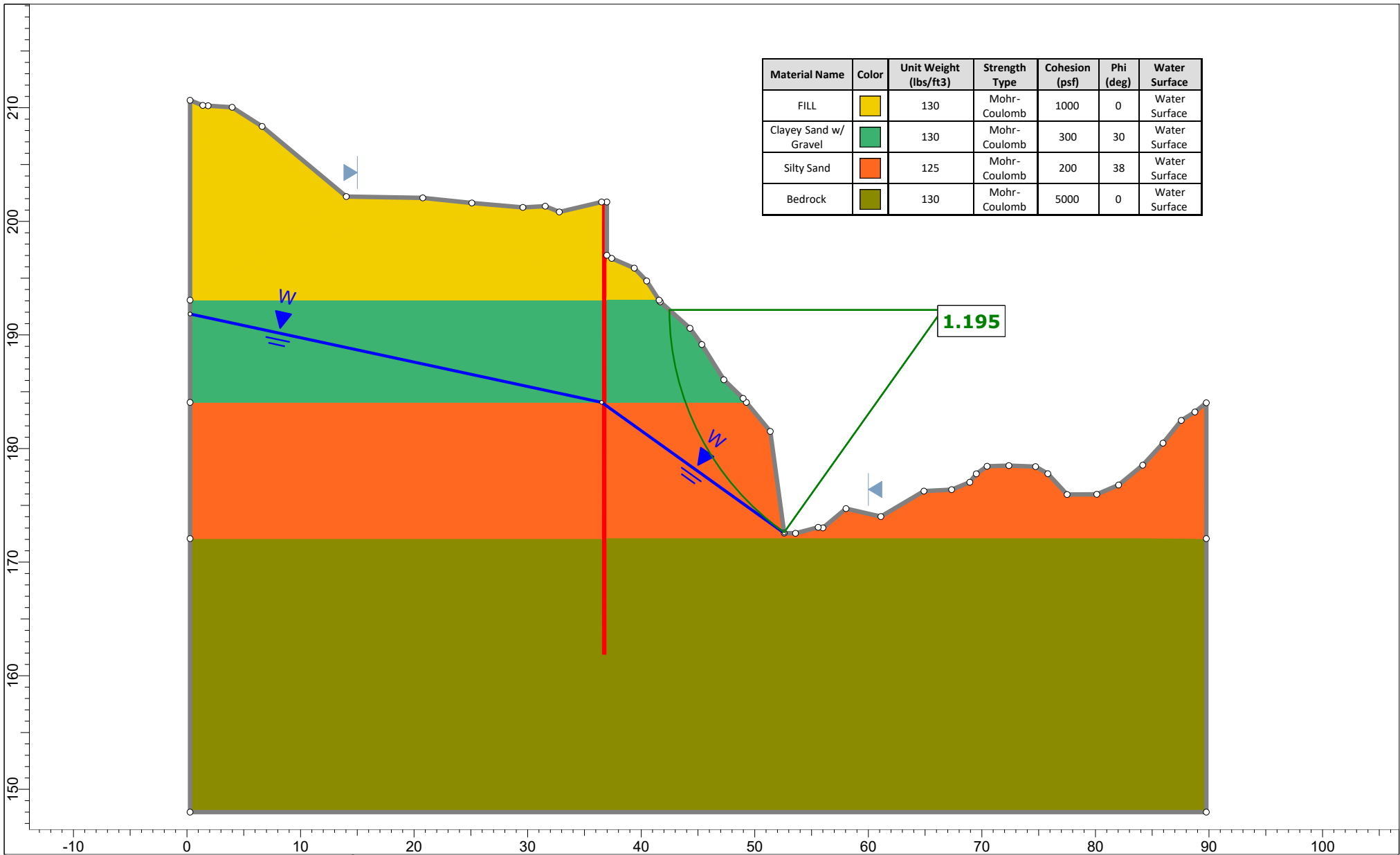
	<i>Project</i> SLIDE - An Interactive Slope Stability Program	
	<i>Group</i> Existing	<i>Scenario</i> Existing - Seismic
	<i>Drawn By</i>	<i>Company</i>
	<i>Date</i> 3/14/2023, 10:31:19 AM	<i>File Name</i> Slope Stability-20230411.slmd




Material Name	Color	Unit Weight (lbs/ft <sup>3</sup> )	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface
FILL	Yellow	130	Mohr-Coulomb	1000	0	None
Clayey Sand w/ Gravel	Green	130	Mohr-Coulomb	300	30	None
Silty Sand	Orange	125	Mohr-Coulomb	200	38	None
Bedrock	Olive Green	130	Mohr-Coulomb	5000	0	None

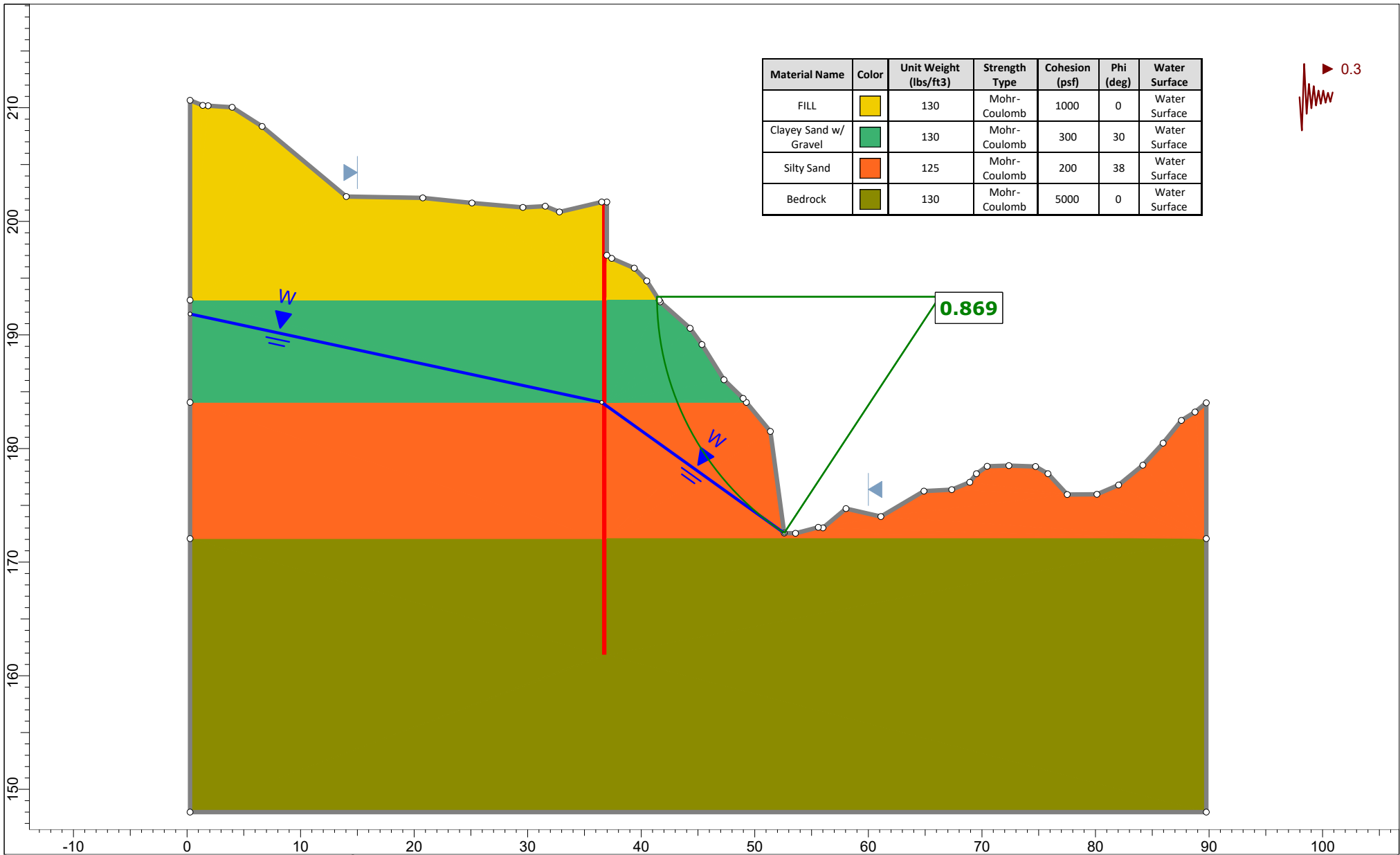
1.195


	<i>Project</i> <b>SLIDE - An Interactive Slope Stability Program</b>	
	<i>Group</i> Retaining Wall - Level Backfill	<i>Scenario</i> Master Scenario
	<i>Drawn By</i>	<i>Company</i>
	<i>Date</i> 3/14/2023, 10:31:19 AM	<i>File Name</i> Slope Stability-20230411.slmd

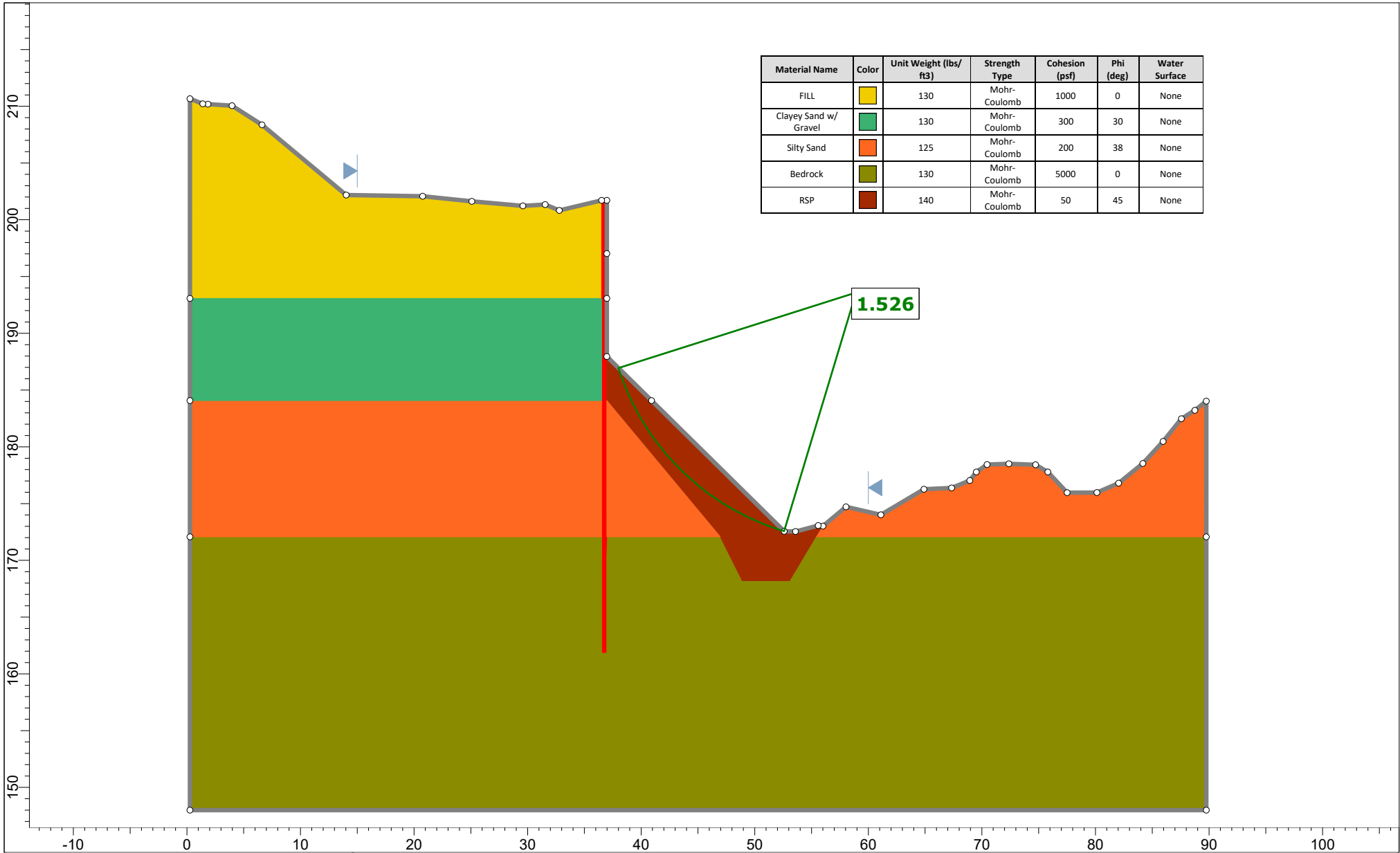


Material Name	Color	Unit Weight (lbs/ft <sup>3</sup> )	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface
FILL	Yellow	130	Mohr-Coulomb	1000	0	Water Surface
Clayey Sand w/ Gravel	Green	130	Mohr-Coulomb	300	30	Water Surface
Silty Sand	Orange	125	Mohr-Coulomb	200	38	Water Surface
Bedrock	Olive Green	130	Mohr-Coulomb	5000	0	Water Surface

	<i>Project</i> SLIDE - An Interactive Slope Stability Program	
	<i>Group</i> Retaining Wall - Level Backfill	<i>Scenario</i> Retaining Wall - High GW
	<i>Drawn By</i>	<i>Company</i>
	<i>Date</i> 3/14/2023, 10:31:19 AM	<i>File Name</i> Slope Stability-20230411.slmd
	<small>SLIDEINTERPRET 9.020</small>	



	<i>Project</i> SLIDE - An Interactive Slope Stability Program	
	<i>Group</i> Retaining Wall - Level Backfill	<i>Scenario</i> Retaining Wall - Seismic
	<i>Drawn By</i>	<i>Company</i>
	<i>Date</i> 3/14/2023, 10:31:19 AM	<i>File Name</i> Slope Stability-20230411.slmd

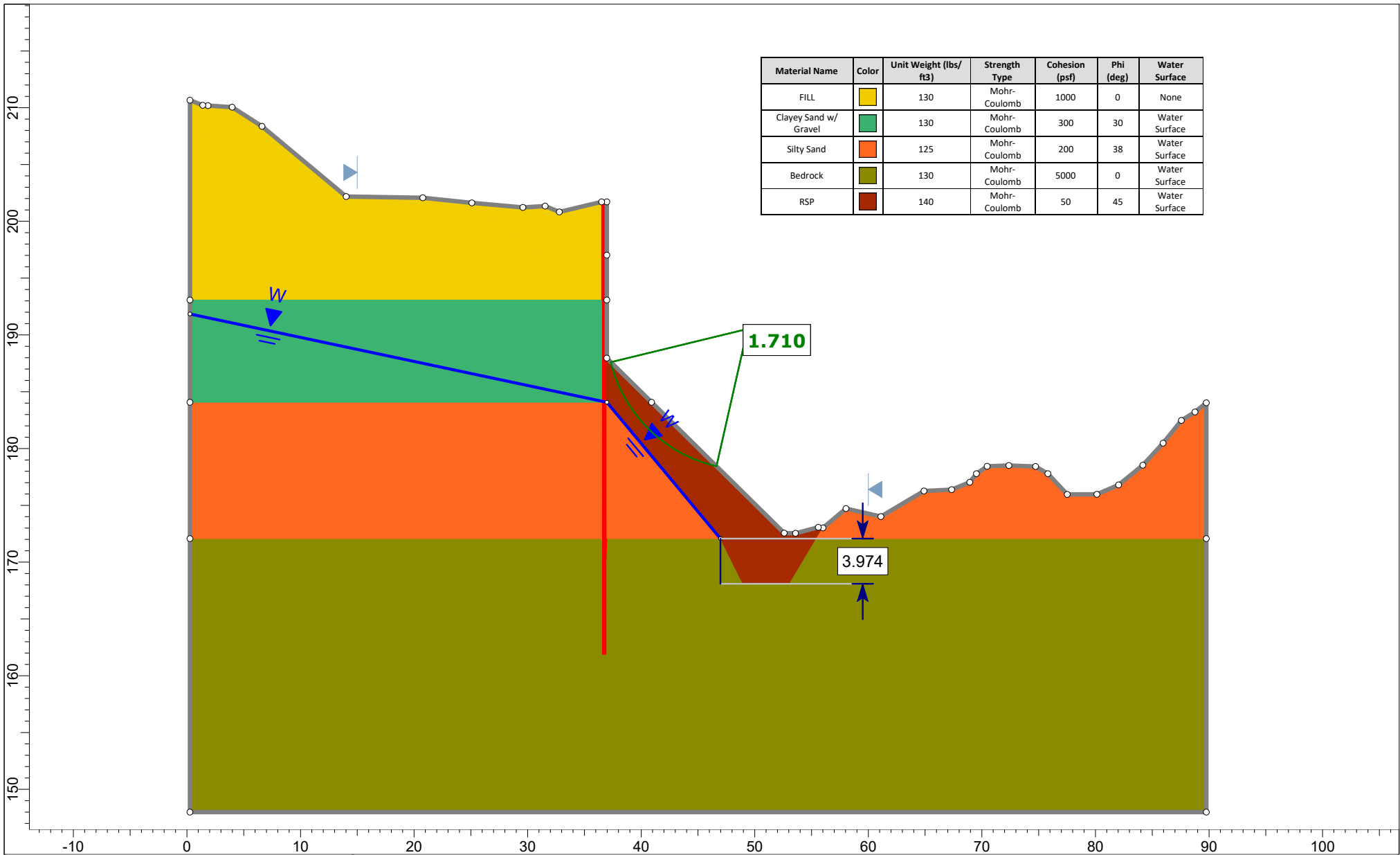


Material Name	Color	Unit Weight (lbs/ft <sup>3</sup> )	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface
FILL	Yellow	130	Mohr-Coulomb	1000	0	None
Clayey Sand w/ Gravel	Green	130	Mohr-Coulomb	300	30	None
Silty Sand	Orange	125	Mohr-Coulomb	200	38	None
Bedrock	Olive Green	130	Mohr-Coulomb	5000	0	None
RSP	Red	140	Mohr-Coulomb	50	45	None


1.526

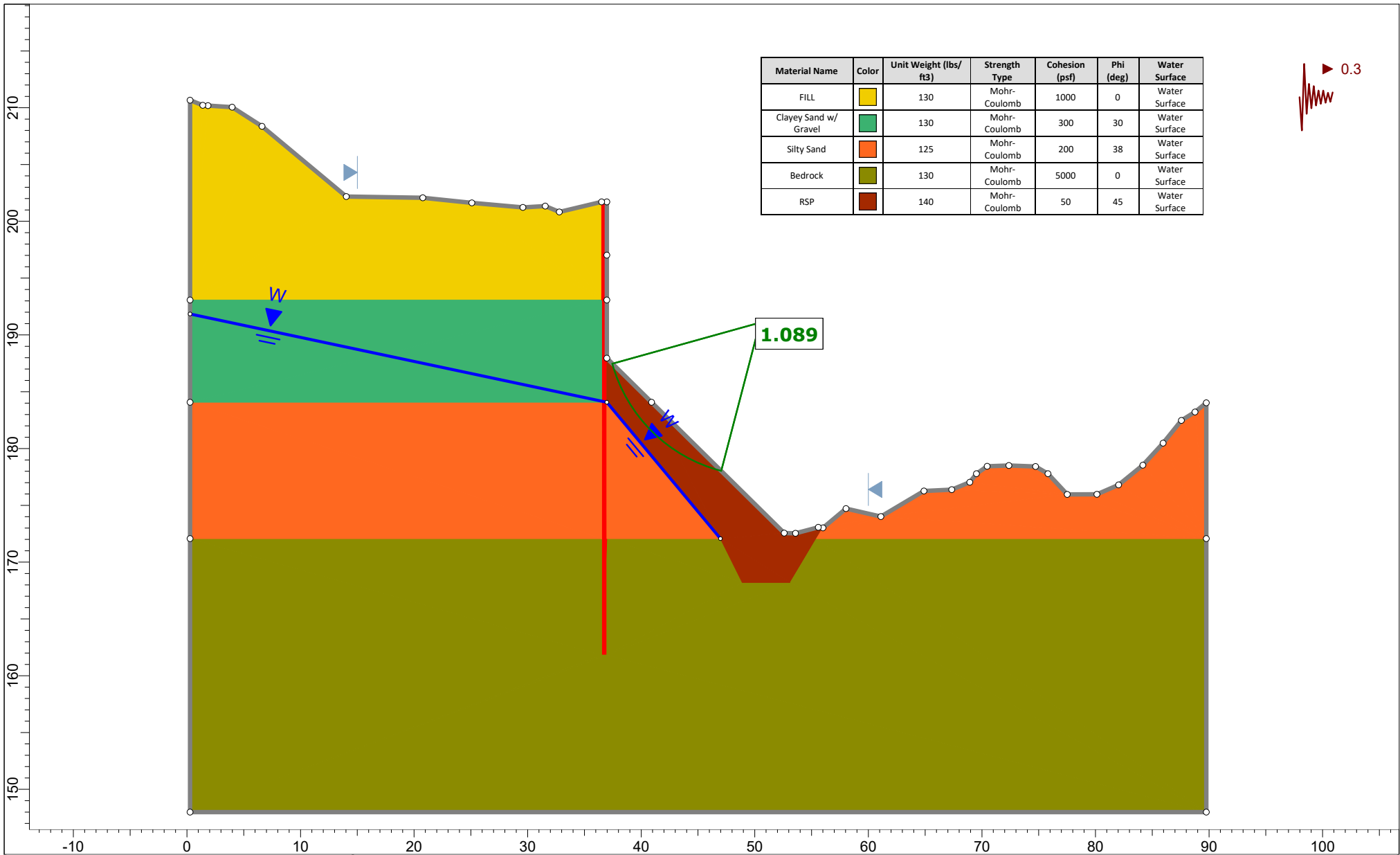


Project		SLIDE - An Interactive Slope Stability Program	
Group	Wall w/ RSP - Level Backfill	Scenario	Master Scenario
Drawn By		Company	
Date	3/14/2023, 10:31:19 AM	File Name	Slope Stability-20230411.slmd




Material Name	Color	Unit Weight (lbs/ft <sup>3</sup> )	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface
FILL	Yellow	130	Mohr-Coulomb	1000	0	None
Clayey Sand w/ Gravel	Green	130	Mohr-Coulomb	300	30	Water Surface
Silty Sand	Orange	125	Mohr-Coulomb	200	38	Water Surface
Bedrock	Olive Green	130	Mohr-Coulomb	5000	0	Water Surface
RSP	Brown	140	Mohr-Coulomb	50	45	Water Surface

	<i>Project</i> <b>SLIDE - An Interactive Slope Stability Program</b>	
	<i>Group</i> Wall w/ RSP - Level Backfill	<i>Scenario</i> Wall w/ RSP - High GW
	<i>Drawn By</i>	<i>Company</i>
	<i>Date</i> 3/14/2023, 10:31:19 AM	<i>File Name</i> Slope Stability-20230411.slmd
	<small>SLIDEINTERPRET 9.020</small>	



Material Name	Color	Unit Weight (lbs/ft <sup>3</sup> )	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface
FILL	Yellow	130	Mohr-Coulomb	1000	0	Water Surface
Clayey Sand w/ Gravel	Green	130	Mohr-Coulomb	300	30	Water Surface
Silty Sand	Orange	125	Mohr-Coulomb	200	38	Water Surface
Bedrock	Olive Green	130	Mohr-Coulomb	5000	0	Water Surface
RSP	Brown	140	Mohr-Coulomb	50	45	Water Surface

	<i>Project</i> <b>SLIDE - An Interactive Slope Stability Program</b>	
	<i>Group</i> Wall w/ RSP - Level Backfill	<i>Scenario</i> Wall w/ RSP - Seismic
	<i>Drawn By</i>	<i>Company</i>
	<i>Date</i> 3/14/2023, 10:31:19 AM	<i>File Name</i> Slope Stability-20230411.slmd
	<small>SLIDEINTERPRET 9.020</small>	