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GEOTECHNICAL DESIGN REPORT

2023 SLIP-OUT REPAIR NEAR 2180 HIGGINS CANYON ROAD

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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 2180 Higgins Canyon Road Stabilization Project in Half Moon Bay, California (Figure 1). The work has been completed to provide recommended repair alternatives to stabilize the road for the County of San Mateo (County)

1.2 PROJECT DESCRIPTION

Due to heavy rain events in December 2022 through March 2023, an approximately 130-foot-long segment of Higgins Canyon Road (near 2180 Higgins Canyon Road) has undergone vertical and horizontal displacement due to landsliding. There are at least two landslides located along the outboard edge of the road. However, tension cracking and buckling of asphalt northwest and southeast of the smaller slide scarps indicate the development of a larger main scarp (about 130 feet wide) that extends upslope of the road. These displacements have resulted in a full road closure until the road can be permanently stabilized. We understand the County has not yet identified a preferred repair option and seeks input on the most feasible and cost-effective concept based on site-specific surface and subsurface information. Photos of the project site are included in Appendix A.

During a meeting with the County, temporary and permanent repairs were discussed. The County determined that the best course of action was to develop repair plans that would provide long-term site stabilization.

In developing the scope of the design services, several slope repair designs to stabilize and restore the roadway embankment were considered. Among these preliminary design alternatives are: 1) a soldier pile wall with tiebacks and wood lagging; 2) two soldier pile walls, one on the uphill side of the road and one on the downhill side; 3) an earth repair utilizing geogrid reinforcement; and 4) a Mechanically Stabilized Earth (MSE) retaining wall incorporating geogrid reinforcement. We are just beginning the evaluation of these and other alternatives.

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 project area 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 along Higgins Canyon Road (near 2180 Higgins Canyon Road), approximately 0.3 miles northwest of its intersection with Cypress Ridge Road, in Half Moon Bay, California (Figure 1). In this area, Higgins Canyon Road is oriented northwest-southeast and runs along the base of a southwest-facing hillside. The project area is bounded by the southwest-facing hillside to the northeast, a continuation of Higgins Canyon Road to the northwest and southeast, and tree-covered slopes to the southwest that extend down to Arroyo Leon. In the immediate project area, Higgins Canyon Road is approximately 23 feet wide and is asphalt paved with two lanes. The roadway elevation gently increases from northwest to southeast, from an elevation of about 298 to 304 feet above sea level across the approximately 130-foot length of destabilized roadway. Natural slopes along the outboard edge of the roadway vary but generally slope between 27° and 30° and are densely vegetated with large eucalyptus trees. The inboard edge of the road consists of a steep (approximately 55°) road cut into the bedrock.

Detailed descriptions of the site and road distress features are further described in the site reconnaissance section of this report (Section 4.2), and 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 2.3 miles inland from the Pacific Coast.

3.2 BEDROCK GEOLOGY

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; 2000); Knudsen and others (2000); Graymer and others (2006); Witter and others (2006); and Cochrane and others (2014).

According to Brabb and others (1998), the slope below Higgins Canyon Road as well as the majority of the road itself in the project area, are mapped as being underlain by Holocene-aged alluvial deposits and older (Pleistocene) alluvial fan and stream terrace deposits, which overlay Purisima Formation bedrock (Figure 3). The mapped Holocene alluvium is described as "unconsolidated gravel, sand, silt, and clay," and the older alluvial fan and stream terrace deposits are described as "poorly consolidated gravel, sand, and silt" (Brabb and others, 1998). These deposits are mapped as overlaying Purisima Formation (Pliocene and Upper Miocene) bedrock along the inboard edge and upslope areas of Higgins Canyon Road. Purisima Formation bedrock is generally described as gray and greenish-gray to buff, fine-grained sandstone, siltstone, and mudstone, but also includes some porcelaneous shale and mudstone, chert, silty mudstone, and volcanic ash" (Brabb and others, 1998). Based on the site topography, the contact between the alluvial deposits and Purisima Formation bedrock is likely located farther downslope of Higgins Canyon Road in the immediate site vicinity. Purisima Formation sedimentary beds locally strike north-south to northwest-southeast and dip to the west-southwest at approximately 25° to 41°, which is oblique to the orientation of Higgins Canyon Road in the project area (Brabb and others, 2000)

Later mapping in the project area by Graymer and others (2006) and Cochrane and others (2014) is in general agreement with mapping by Brabb and others (1998).

3.3 SEISMICITY

The project site is located within the greater San Francisco Bay Area, recognized as one of California's more seismically active regions. 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 evaluating specific seismogenic sources. For example, with the 30 yr M≥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≥6.7 probabilities may not represent other hazard or loss measures. 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 likely to occur in the area generally depends upon the earthquake's magnitude and the distance to the epicenter.

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

Fault Name	Approx. Distance and Direction from Site to Mapped Surface Fault Traces
San Gregorio	4.5 km southwest
San Andreas	7.6 km northeast
Butano	16.7 km south
Monte-Vista Shannon	17.6 km southeast
Zayante-Vergeles	29.0 km south-southeast
Hayward	37.8 km northeast
Calaveras	45.7 km northeast
Sargent	52.4 km southeast

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. However, 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 5).

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. The materials underlying the project site are mapped as having very low to low liquefaction susceptibility (Witter and others, 2006) (Figure 5).

According to a map showing Earthquake Zones of Required Investigation for the Half Moon Bay 7.5 Minute Quadrangle, the project site is not within a liquefaction hazard zone.

According to the San Mateo County Hazard Mapping Tool, the project site has very low to low liquefaction susceptibility.

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 did 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 located within 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 along the eastern boundary of 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 does not show groundwater data for Higgins Canyon Road in the project vicinity due to its location within hilly terrain. However, it shows historically high groundwater levels less than 10 feet below the ground surface, just downslope of the road, along the Arroyo Leon creek banks.

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 INITIAL SITE RECONNAISSANCE

CE&G performed field reconnaissance of the site on January 10th, 19th, and 23rd, 2023, before 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.

During our initial site visits, an approximately 30-foot-wide landslide scarp was observed along the outboard edge of the road. Water was observed seeping in five different areas near the base of the scarp, approximately 9 feet below the road elevation. Although the slide scarp had not encroached into the adjacent asphalt pavement by our initial site visits, tensional and compressional cracking within the asphalt pavement was observed and indicated a much larger (approximately 120-foot wide) slide was likely to occur. Vertical and horizontal displacements of these cracks at the time of our first site reconnaissance were approximately 0.5 to 3 inches. By our second and third site reconnaissance on January 19th and 23rd, 2023, vertical displacements of cracks in the asphalt pavement were as high as 7 inches, and areas of compressional buckling were as high as 12 inches. During these additional site visits, a second landslide initiated by fallen trees was observed along the road shoulder approximately 33 feet wide.

4.2 LIDAR SCAN AND ADDITIONAL GEOLOGIC MAPPING

A design-level aerial light detection and ranging (LiDAR) scan of the site was performed on February 17th, 2023, to develop a topographic base map 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 base map and orthoimage were used to document surface features and geologic interpretations during our site reconnaissance on February 20th, March 6th, and March 20th, 2023. Our geologic interpretations are presented in Figure 2, and some field observations made during our final site reconnaissance are listed below.

The below notes also reference site photographs, which are included in Appendix-A.

Slope Below Higgins Road

- The slope along the outboard edge of Higgins Canyon Road is moderate to densely forested with large eucalyptus trees and a brushy understory (Photo 1).
- Natural slope inclinations downslope of the road ranges from approximately 27° to 30°.
- During our first two geologic mapping events, there were two landslide scarps directly adjacent to one another along the shoulder of the road.
- The scarp to the northwest appeared to be a combination of a rotational failure followed by an earth flow of the slide debris. The scarp was approximately 30 feet wide with a rupture surface length of about 20 to 25 feet (Photo 2). The slide debris extended from the base of the scarp down to Arroyo Leon farther downslope (Photo 3). Seepage was observed in at least 5 locations along the base of the scarp, approximately 9 feet below the adjacent road elevation (Photo 4). Overall, the slide debris likely consisted mostly of road fill and highly weathered Purisima Formation bedrock.
- The scarp to the southeast appeared to have been caused by 4 large eucalyptus trees falling during the recent storm events. The trees fell in the downslope direction, extending all the way down to Arroyo Leon and exposing their root masses near the scarp's toe. The scarp was approximately 33 feet wide and about 12 feet long.
- At the time of our first two geologic mapping events, longitudinal tension cracks were observed along the grassy shoulder between the two slide scarps and northwest of the northwestern scarp. By the time of our most recent geologic mapping event, the grassy shoulder between the two existing scarps had also failed, resulting in one large landslide along the outboard edge of Higgins Canyon Road (Photo 5). Additionally, a tension crack that previously formed along the road shoulder, northwest of the two slide scarps, had extended farther northwest, as shown in Figure 2. Vertical offsets along this crack range from approximately 0.5 to 1.5 feet and will likely increase with time.
- By the time of our last geologic mapping event, at least 7 large eucalyptus trees had fallen along the outboard edge of the road within and around the landslide areas (Photo 6).

Asphalt Pavement on Higgins Canyon Road

- The asphalt pavement in the project area is highly distressed, with visual evidence of tension, shearing, and compressional forces. The observed distress extends approximately 130 feet along Higgins Canyon Road, whereas the landslide along the outboard edge of the road only extends about 80 feet (Figure 7).

- Cracking in the road adjacent to the downslope scarps generally consists of transverse tension cracks up to about 12 inches wide, up to 30 feet long, and vertical displacements up to 14 inches (Photo 8). These cracks extend from the scarp edges about 10 to 15 feet into the road.
- The northwesternmost extent of asphalt distress is dominated by compressional buckling, resulting in buckling and thrusting within the asphalt, creating more than 12 inches of vertical displacement (Photo 9). The fold hinge lines are almost perpendicular to the road, indicating east-west compressional forces (Figure 2).
- The southeasternmost extent of observed road distress consists of a near-continuous crack that extends across the entire roadway and has a minor (approximately 3 inches) left-lateral sense of movement (Photo 10). The crack is oriented with an average azimuth of 050°. Where this crack dies out into the road cut, there are additional signs of buckling in the pavement, indicating forces in the southwest direction near the inboard edge of the road (Photo 11). The buckling is likely due to additional slide forces from upslope.
- Cracking and buckling in the roadway consistently worsened with each geologic mapping event, indicating the overall landslide continued to move from at least early January to late March 2023.

<u>Inboard Road Cut Along Higgins Canyon Road</u>

- The inboard edge of Higgins Canyon Road in the site vicinity consists of a road cut that ranges in height from about 6 to 15 feet and is inclined at about 55° (Photo 12).
- Highly fractured siltstone bedrock was observed along the upper portions of most of the road cut. The bottom half of the road cut generally consists of colluvial piles of silty soils and siltstone fragments from the bedrock above.
- Minor seepage was observed along the slope in some areas.
- Multiple shallow landslides were observed along the road cut and are depicted in Figure 2.
- Surface flows along the inboard edge of the road have formed a swale about 6 to 10 inches deep in the soil at the base of the cut slope. When not flowing, water ponds in the swale.
- Between the time of our first and last geologic mapping events, at least three eucalyptus trees along the upper portions of the inboard slope had fallen (Figure 2). This may result in additional instability of colluvial soils on the slope.

<u>Private Property Upslope of Higgins Canyon Road (2175 Higgins Canyon Road)</u>

- The private property of 2175 Higgins Canyon Road is located approximately 25 feet uphill of the road. The property consists of a relatively flat bench that extends 30 to 55 feet to the north-northeast before the land slopes again at an angle of approximately 25° to 45°. A portion of the slope is retained by a wooden retaining wall, which stands about 4 feet tall and is leaning in some areas (Photo 13).
- Existing structures on the benched property consist of two chicken coops and a wooden barn on a foundation supported by 3-foot-long concrete piers (Figure 2).
- Multiple areas along the flat-lying portion of the property (bench) and the adjacent slope to the north showed significant signs of distress. They included tension cracks and scarps with horizontal and vertical displacements (Photo 14).
- Tension cracks were observed in the grass-covered bench to the west (Photo 13) and south of the barn and extend beneath the barn's western half. The tension cracks in the bench generally trend to the northwest, with the land dropping to the southwest in some occurrences. During our site visit, the tension crack apertures ranged from approximately 0.5 to 1.2 feet. The depths of the tension cracks were also measured but were only about 1.5 feet deep due to soil in-filling the cracks. Therefore, the cracks were likely deeper when initially formed. Tension cracks that extend beneath the barn resulted in horizontal and vertical displacements in the barn's foundation (Photos 15). Most of the tension cracks were mapped and depicted in Figure 2.
- A significant landslide scarp with horizontal displacements up to 1.2 feet and vertical displacements of 1 to 5 feet was observed and mapped across the property (Figure 2). The easternmost portion of the scarp is first exposed south of the barn and has an azimuth in this area of about 145° (Photo 16). The scarp then continues beneath the western half of the barn and cuts into the slope to the north before turning west, where it peaks at an elevation of about 347 feet (Photos 13 and 17) before it dives to the southwest and continues back down the slope and onto the bench with an azimuth of about 235° (Photo 18). The scarp then dies out along the southwestern end of the bench, approximately 20 feet northeast of Higgins Canyon Road (Figure 2). This scarp appears to be the main scarp of a large landslide that likely extends beneath Higgins Canyon Road to the southwest. Additionally, the western and eastern extents of the scarp generally align with the deformation observed in the Higgins Canyon Road asphalt pavement.

Our interpretation of the geologic mapping results is further summarized in Section 5.

4.3 SUBSURFACE EXPLORATION

The subsurface investigation consisted of drilling three geotechnical borings (B-1, B-2, and B-3), as shown on Figure 2. The borings were drilled by Cenozoic Exploration on February 9th, 2023, using a track-mounted Mobile Geoprobe 7822DT drill rig equipped with 6-inch diameter solid-flight augers.

Borings B-1, B-2, and B-3 were drilled and sampled to a total depth of 26, 46.5, and 30.5 feet below ground surface (bgs), respectively. Upon completion, the borings were backfilled with neat cement grout in accordance with San Mateo County permitting requirements. 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 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 and rock 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 auto-trip 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 B. The blow counts on the boring logs represent the field values and are uncorrected.

Soil and rock 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 storage.

4.3.2 Soil and Bedrock Conditions Encountered

Subsurface conditions in the borings are comprised of asphalt and fill soils, overlaying Purisima Formation bedrock. The encountered subsurface units are described below.

Pavement

Approximately 2 inches of asphaltic concrete was encountered over fill material consisting of local weathered bedrock and low plasticity clay in borings B-1, B-2, and B-3.

Landslide Debris

Landslide debris was encountered to depths between 9 and 17 feet bgs. The landslide debris generally consists of very soft to soft lean and fat clay with angular fragments of severely weathered Purisima Formation Bedrock. The angular rock fragments consisted of friable siltstone and were present in most samples.

Purisima Formation Bedrock

Purisima Formation Bedrock was encountered beneath the landslide debris to the depths explored. The bedrock is very soft to moderately hard, fresh to slightly weathered, dark greenish-gray claystone.

A detailed description of the encountered materials is included in the boring logs in Appendix B. Our interpretation of the encountered subsurface materials, along with information gathered from our site reconnaissance, is shown in a geologic cross-section in Figure 6.

4.3.3 Groundwater Conditions Encountered

Groundwater was encountered in all three borings from about 6 to 10.5 feet below the ground surface. Groundwater levels were either based on the retrieval of wet samples at depth or from high moisture content data from tested samples.

During our site reconnaissance, seepage was also observed in the landslide scarp, approximately 9 feet below the Higgins Canyon Road elevation. Minor seepage was also observed in the road cut along the inboard edge.

4.3.4 Geotechnical Laboratory Testing

Testing was performed to obtain information concerning the samples' qualitative and quantitative physical properties 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 the detailed laboratory testing program is presented in Appendix C.

5.0 CONCLUSIONS AND DISCUSSION

5.1 GENERAL SUMMARY

Based on the results of our investigation, it is our opinion that Higgins Canyon Road in the project area is geologically unstable and will require permanent stabilization measures. Design alternatives being considered to stabilize the roadway include the following:

1. Soldier pile wall, possibly with tiebacks and wood or concrete lagging;
2. Two soldier pile walls, one on the uphill side of the road and one on the downhill side;
3. An earth repair utilizing geogrid reinforcement; and
4. A Mechanically Stabilized Earth (MSE) retaining wall incorporating geogrid reinforcement.

Geotechnical recommendations for the design and construction of the proposed structures being considered for the repair are presented in Section 6.0 of this report.

Geotechnical considerations to note during project design and construction are:

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

Detailed recommendations for these and other geotechnical aspects of the proposed improvements are presented in the following sections of this report. Our evaluations and recommendations are based on the information obtained during this investigation.

5.2 LANDSLIDE CHARACTERIZATION

Our subsurface exploration, site reconnaissance, and geologic mapping results indicate that the deformation along Higgins Canyon Road in the project area has been affected by an active landslide comprised of fill soil, colluvial soils, and moderately to highly weathered Purisima Formation siltstone and claystone. Based on our subsurface borings, the landslide surface beneath Higgins Canyon Road is characterized by a wide zone up to 5 feet thick of soft, wet, lean to fat clay, with angular fragments of siltstone. The base of the slide material was encountered between 9 and 17 feet below the road in the locations explored. This landslide extends farther upslope, where it daylights at the main scarp, located approximately 85 to 110 feet upslope and northeast of the inboard edge of Higgins Canyon Road (Figure 2). Due to the lack of subsurface information upslope of the road, the interpretation of landslide plane geometry was largely based on scarp locations, its encountered depth beneath Higgins Canyon Road, and other mapped surface geomorphic features. Our interpretation of the landslide plane is shown in Figure 6.

The toe of the surface rupture, downslope of Higgins Canyon Road, is not well defined due to minimal downslope movement of the overall slide mass and dense vegetation cover on the lower slope. However, due to the presence of groundwater seepage within the smaller landslide scarps along the outboard edge of the road, as well as up to seven fallen eucalyptus trees previously rooted at similar elevations as the seepage, we interpreted the slide plane to toe out at a similar elevation below the road.

Based on the landslide features described above, the mapped landslide is likely situated on a planar rupture surface approximately 130 to 150 feet wide and 160 to 175 feet long. We estimate that the landslide mass itself is about 15 feet thick beneath the roadway and may range from approximately 6 to 20 feet thick upslope of Higgins Canyon Road (Figure 6).

In our judgment, high groundwater levels are likely the main contributing factor to the driving force of the landslide. This judgment is based on observed seepage within the slide scarp downslope of the road and in some areas along the road cut, upslope of the road, as well as saturated, very soft soils encountered along the slide plane on our subsurface borings.

5.3 DRILLABILITY AND EXCAVATABILITY

Subsurface exploration was completed using hollow stem augers and encountered bedrock at depths of approximately 9 to 17 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 all 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 medium to hard sandstone and shale. Drilling and excavating through the hard to very hard siltstone and claystone encountered at 30 feet below the ground surface in boring B-2 will likely require additional effort, including the possible need for using jackhammers or hoe rams if encountered. We recommend that the contractor observe bedrock outcrop exposures along the road cut before construction for proper equipment selection.

5.4 SEISMIC HAZARDS

Large-magnitude earthquakes and strong ground shaking will likely affect the project area within the design lifetime of the proposed improvements. Peak ground shaking parameters are presented in Section 6.3 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.

- We judge the potential for fault ground rupture or coseismic faulting to significantly affect the proposed improvements is low.

- We judge the potential for ridgetop fissuring, ridgetop shattering, ridgetop spreading, or other seismically induced ground deformation to significantly affect the proposed improvements is low.

- We judge the potential for soil liquefaction to significantly affect the proposed project is very low due to the high clay content in shallow soils and bedrock.

5.5 SLOPE STABILITY ANALYSIS

Analysis has been performed to evaluate the existing slope and recommended repair alternatives. 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 several slope repair configurations. 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 analysis results 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 temporary shoring retaining wall upslope along the inboard shoulder was completed to analyze the slope during construction. Table 5-2 shows the results of the slope stability analysis with the temporary retaining wall during construction.

Table 5-2. Slope Stability Analysis of Temporary Retaining Wall During Construction

Analysis Condition	Factor of Safety
Temporary Wall – High GW	1.08
Temporary Wall – Low GW	1.33

A slope stability analysis featuring several generalized repair design configurations was completed to analyze the slope after it has been repaired, evaluating the high and low groundwater (GW) conditions of the slope. Table 5-3 shows the results of the slope stability analysis with a generalized repair design configuration.

Table 5-3. Slope Stability Analysis of Generalized Repair Design Configuration

Analysis Condition	Factor of Safety
Buttress – High GW	1.16
Buttress – Low GW	1.40
Retaining Wall – High GW	1.07
Retaining Wall – Low GW	1.31

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

1. A temporary cut for construction for a buttress alternative is only feasible if a shoring retaining wall is initially placed along the inboard road shoulder.

2. Control of subsurface drainage is essential for the long-term stability of any repair.

3. A slope repair increases the factor of safety of the slope during high groundwater conditions, particularly with subsurface drainage incorporated into the repair.

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

Section 5.8 below provides discussions of the slope repair alternatives.

5.6 CORROSION

Corrosion testing was performed on one soil sample at this location in general accordance with Caltrans methods. Testing results are presented below:

Table 5-4: Corrosion Testing Results

Boring ID (sample depth in feet)	Resistivity (Ohm-cm)	Chloride (mg/kg)	Sulfate (mg/kg)	pH
B-1 (0 to 5)	775	85	390	6.5

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 or water less than 1000 ohm-cm indicates the presence of high quantities of soluble salts and a higher susceptibility to being corrosive. Based on the minimum resistivity test results of the laboratory testing performed, the soil samples are required to be tested for chloride and sulfate concentration.

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.

5.7 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. It should be noted that the portion of the road requiring repair has expanded since the original roadway distress was recognized in January 2023;
- Hydraulic design conditions;
- Temporary shoring of the upslope side of the 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. The following sections are general descriptions of repair alternatives.

5.7.1 Temporary Repairs Considerations

Temporary repair alternatives were considered part of the investigation, including a temporary railroad car bridge, a bailey bridge, sheet piles, rip rap, etc. However, our slope stability analysis indicated a low safety factor with the high groundwater conditions currently present at the site. With the continuous movement of the slide mass and the potential for additional road embankment failures, the County and CE&G agreed that a more permanent solution would be safer for the public and more time and cost-effective. Additionally, temporary repair alternatives would require similar efforts to a permanent repair. Temporary repairs involving riprap in the failed areas would not support the overall slide mass, and sheet piles would result in a buildup of hydrostatic pressures beneath the road and inboard slope. A railroad car bridge installation would require bridge abutment construction, and the hazardous area's length would be too long to install a bailey bridge.

5.7.2 Geogrid Reinforced Earth Repair

A geogrid reinforced earth repair, consisting of removing and replacing the landslide material with geogrid-reinforced engineered fill as a buttress, will require significant excavation to support and stabilize the slope and the road. Excavation for the entire length of the roadway slip out down to the bedrock will be required for the buttress to be properly supported by the bedrock, which is anticipated to be between 16 and 20 feet below the ground surface. This repair option will allow for the placement of drainage, erosion protection measures, and soil reinforcement. Appropriate surface and subsurface drainage measures can also be placed on the repaired roadway. These measures will allow for a well-drained slope repair that can mitigate further slope movement. It should be noted that this repair alternative may only be accomplished in conjunction with a soldier pile wall first installed along the inboard side of the road.

5.7.3 Mechanically Stabilized Earth (MSE) Wall Repair

An MSE wall is considered a feasible alternative for the roadway repair, consisting of removing and replacing the landslide material with geogrid-reinforced engineered fill incorporating segmental block wall units. It will require significant excavation to support and stabilize the slope and the road. Excavation for the entire length of the roadway slip out down to the bedrock will be required for the MSE wall to be adequately supported by the bedrock, which is anticipated to be between 16 and 20 feet below the ground surface. This repair option will allow for the placement of drainage, erosion protection measures, and construction of the MSE wall. Appropriate surface and subsurface drainage measures can also be placed on the repaired roadway. These measures will allow for a well-drained slope repair to mitigate further slope movement. It should be noted that, like the geogrid reinforced earth repair option, this repair alternative may only be accomplished in conjunction with a soldier pile wall first installed along the inboard side of the road.

5.7.4 Soldier Pile and Lagging Walls

A soldier pile and lagging retaining wall would require less excavation than an earthwork repair. For a wall on the outboard edge of the roadway, excavation is anticipated to be as deep as the bottom of the wall face, which is anticipated to be between 16 and 20 feet deep below the ground surface. For a wall on the inboard edge of the roadway, excavation is anticipated to be approximately 12 feet deep below the ground surface. This wall type will allow for drainage panels or drain rock to be placed behind the wall to reduce hydrostatic pressure buildup behind the wall. Appropriate surface and subsurface drainage systems can tie into the slope and down the nearby creek to reduce erosion potential. To limit the deflection of the wall, tiebacks may be required for walls with a retained height of over 12 feet. However, they are anticipated to encroach onto adjacent private properties.

5.8 RECOMMENDED ALTERNATIVE

Based on our preliminary engineering evaluation, the alternatives discussed above will be developed further to be able to select the best repair option for permanent repair of the roadway. Following are some factors that we will consider in the development of the final repair concept:

1. Retaining walls require significantly less excavation, export of material, and import of engineered fill,

2. Reinforced geogrid can be placed to strengthen the engineered fill,

3. Surface and subsurface drainage will be required to significantly reduce the potential for future failures,

The final repair approach and schedule will influence the construction duration and phasing.

5.9 GEOTECHNICAL CONSIDERATIONS

Some geotechnical issues that will affect the design and construction of the roadway embankment 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 roadway embankment repair.

- **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.

- **Temporary Retaining Wall –** Based on the site geometry and the anticipated construction operation steps, a temporary retaining wall will be required on the in-board shoulder to allow for safe construction of either repair alternative. The earthwork repair alternative will require deep excavations that must be safely shored for the duration of construction. Retaining wall recommendations have been provided in Section 6.0.

- **Retaining Wall –**The anticipated wall heights are achievable based on the site geometry, depth to competent materials, and proposed retaining wall layout. 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 6.3 for the retaining wall. In addition, recommendations for a tieback retaining wall are also included.

- **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 collect and adequately accommodate the runoff volume. Subsurface drainage should reduce hydrostatic pressure buildup behind retaining walls or buttress loads and should be discharged down the slope away from the proposed repair.

6.0 RECOMMENDATIONS

6.1 GENERAL

Detailed recommendations for the geotechnical aspects of the proposed improvements are presented in the subsequent sections of this report. Our evaluations and recommendations are based on the previously discussed information provided to us and that we collected from the site. We may need to modify the recommendations presented herein if there are any changes in the proposed improvements, their layout or location, or the proposed grading.

We recommend the current slope instability be repaired using one or a combination of the following alternatives:

1. Reinforced Earth Slope Repair

2. Mechanically Stabilized Earth (MSE) Retaining Wall

3. Soldier Pile Retaining Wall

Recommendations for these options are provided below.

6.2 REINFORCED EARTH SLOPE REPAIR

6.2.1 General

The current slope instability may be repaired with a geogrid-reinforced fill embankment constructed in the same location as the current slope instability. The base or lowest portion of the repair should be located below the lowest disturbed earth, and the upper portion will extend into the existing road. We currently anticipate the base of the repair to be at an elevation of approximately 280 feet.

The geogrid-reinforced slope should be constructed using on-site materials consisting largely of sandy silt and silty sand derived from the underlying highly weathered siltstone and shale bedrock. The material removed during the excavation of the repair area is anticipated to consist of these soils. Imported soil for use in the construction of the reinforced slope should be primarily granular and meet the requirements outlined in Section 6.5.2, Material for Engineered Fill.

On-site soils with an organic content of less than 3 percent by weight, free of any hazardous or deleterious materials, and meeting the gradation requirements below may be used as general engineered fill to achieve project grades.

General engineered fill material should not contain rocks or lumps larger than 4 inches in greatest dimension, should not contain more than 15 percent of the material larger than 2½ inches, and at least 20 percent passing the No. 200 sieve.

Before delivery to the site, all import fills should be approved by the project geotechnical engineer. At least five (5) working days before importing to the site, a representative sample of the proposed import fill should be delivered to our laboratory for evaluation.

6.2.2 Geogrid

The slope should be reinforced using high-density polyethylene (HDPE) uniaxial geogrid. A high-density polyethylene (HDPE) uniaxial geogrid should be used to reinforce the slope. The geogrid should consist of Tensar UX1600 or equivalent geogrid. Primary geogrid layers should be placed between lifts, not exceeding 2 feet between layers of geotextile, and extend from the face of the temporary slope cut to the face of the permanent repair slope. We anticipate primary grid lengths to range from 15 to 20 feet in the central portion of the repair. Due to the geometry of the repair, the geogrid lengths can decrease toward the margins of the repair. We recommend a minimum primary grid length of 10 feet be used at the margins. We anticipate that to provide enough space for the minimum grid length at the margins, additional excavation will be required. Secondary geogrid layers should be placed in between primary layers, 1 foot above and below primary geogrid reinforcement. Secondary layers should extend a minimum of 6 feet from the face of the permanent slope. Geogrid placement should be performed in accordance with the manufacturer's recommendations. Geogrid overlap and lap lengths should be consistent with the manufacturer's recommendations, typically ranging from 1 to 3 feet.

The geogrid must be placed such that the strength direction of the geogrid, along the roll length in the case of Tensar UX1600, is oriented into the embankment. The geotechnical engineer's representative should be present on-site during the placement of the initial geogrid layers to observe and confirm the proper placement of the geogrid.

Care should be taken not to damage the geogrid material during placement or leave it exposed to the sun for extended periods of time.

6.2.3 Subdrains

A subdrain system should be installed at the back and base of the reinforced soil zone.

In general, a minimum 2-foot thickness of Class 2 permeable material (Caltrans Section 68) shall be placed at the back of the landslide excavation at the back of keyways and benches. The permeable material should be placed in a maximum of 8-inch lifts and compacted by a vibratory compactor. The lift thickness and the number of passes of compaction equipment required for the compaction of the permeable material shall be determined by the design engineer in the field at the time of construction. Depending on the field moisture conditions, placing greater depths of permeable material may be advisable.

A 6-inch diameter perforated Schedule 40 PVC pipe shall be placed within 4 inches of the base of the permeable material. Perforations should be no larger than ¼ inch, and the pipe should be placed at a minimum 2 percent slope to drain. To prevent fines migration, the perforated PVC pipe shall be wrapped with filter fabric. At 15 feet from the face of the slope, the pipe shall be converted to a 6-inch diameter solid Schedule 40 PVC pipe, and the Class 2 permeable material shall be terminated. Discharge locations shall be at least 10 feet below the lowest portion of the reinforced slope. Cleanouts shall be installed at pipe ends and all major bends.

We request the opportunity to review the field conditions during construction to estimate where subdrain installation will likely be required more closely.

6.3 MECHANICALLY STABILIZED EARTH (MSE) WALL

A Mechanically Stabilized Earth (MSE) wall may be considered to be incorporated into the roadway repair. MSE Walls, also known as "Segmental Block Walls," that will be constructed should be designed following the soil parameters below. These soil parameters may need to be revised once design details are available, backfill materials are chosen, or for other engineering or judgment reasons. Segmented walls will require geogrid reinforcing elements. We also recommend the use of wall systems with pinned connections between blocks, such as Keystone or Versalok wall systems, not just angled blocks that rely on gravity. All walls should be designed to include permeable granular fill behind the walls with an appropriate outlet.

MSE walls should also be drained, as described above in Section 6.2.3, and in accordance with the manufacturer's recommendations . The manufacturer's recommendations should be followed regarding the design and construction of the segmented wall.

The base of MSE walls should be setback from descending slopes. To provide adequate setback from the descending slope, the construction of a bench at the base of the wall may be required, with the bench extending 10 feet out from the base of the wall.

Design of MSE walls requires specific information, including allowable soil bearing capacities, soil strength parameters (phi angle and cohesion) for the backfill in the reinforced zone as well as the retained soils beyond the reinforced soil zone, soil unit weights for specific soils in the reinforced zone and retained zone. The values presented in Table 6-1 are intended to be used in the final design. The soil friction values are intended to be maximum values unless specific soils are specified for the reinforced soil zone and retained soil zone immediately behind the reinforced soil zone. Additional information required for design is the intended soil reinforcement, wall height, backfill condition (level or sloping), and preload loads. The design must consider sliding, overturning, and internal and global stability. A safety factor of no less than 1.5 should be considered for global stability.

Table 6-1 – Recommended Soil Parameters - MSE Walls

MSE Retaining Wall Design Considerations	
Imported Backfill Soil Soil Friction Angle (soil classified as SM, SC, SP, GM, GC) Unit Weight (Wet) (soil classified as SM, SC, SP) Maximum fines content (passing #200 sieve)	 30° 125 pcf 40%
Retained Soil Soil Friction Angle (soil classified as CL, CH, SC) Soil Cohesion (soil classified as CL, CH, SC) Unit Weight (Wet) (soil classified as CL, CH, SC)	 22° 800 psf 125 pcf
Allowable Base Friction Coefficient	0.30
Allowable Bearing Capacity at the Base of Repair (may be increased by one-third for seismic loads).	3,000 psf

6.4 SOLDIER PILE RETAINING WALLS

We recommend constructing a soldier pile retaining wall with timber, concrete, or composite lagging to stabilize the slope below the roadway. Based on the results of our subsurface exploration, the depth to the top of bedrock along a retaining wall alignment on the outboard side of the road extends up to approximately 18 feet below the anticipated top of the wall. For a soldier pile wall constructed on the inboard side of the road, we estimate the depth to the top of bedrock to be approximately 12 feet below the existing ground surface. Additionally, the existing slope below the roadway has a gradient of approximately 1H:1V immediately below the roadway edge for a horizontal distance of approximately 24 feet, beyond which the slope flattens to a gradient of 2H:1V. Retaining walls with heights of up to 18 feet and with the geometry along the proposed alignment typically require deep piers with larger structural elements for a cantilever wall or tiebacks 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.

6.4.1 Lateral Earth Pressures

Static lateral earth pressure will be imposed on all excavations. Table 6-2 summarizes the lateral earth pressures recommended for use in the design of retaining walls. 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

Pressure Type	Above Competent Material		Competent Material	
	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	45 pcf	80 pcf	35 pcf	80 pcf
At-Rest	60 pcf	90 pcf	55 pcf	90 pcf
Passive	300 pcf [1]	190 pcf [1]	440 pcf	275 pcf

[1] where slope (1.5H:1V or flatter) exists in front of a retaining 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

Deflections will likely control the design of an unbraced retaining wall. Lateral deflection at the top of the wall shall not be greater than ½-inch.

If the retaining wall will be braced, rectangular, or trapezoidal loading diagrams such as those recommended by Terzaghi & Peck, Tschebotarioff, 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 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. The retaining wall design should consider surcharge loading from traffic on the adjacent areas and construction equipment adjacent to excavations.

6.4.2 Tieback Retaining Wall

If the retaining wall requires the use of tiebacks, the 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 up to 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 34 pcf to a depth of 10 feet, 42 pcf to a depth of 18 feet, and a passive equivalent fluid pressure of 440 pcf. 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 440 psf/ft starting at the bedrock contact and acting over two pier diameters;
- A seismic equivalent fluid pressure of 30 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 is based upon the apparent earth pressure diagram on page 51 of the FHWA manual (Figure 24 – included in Appendix E). The FHWA diagram results in the distribution of the load to the anchors based on the anchor locations.
- Determination of the tieback force and soldier pile maximum moment should be based upon the comparison of 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;
- 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 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 6.5.2.

6.4.3 Seismic Design Parameters

As a result of earthquake shaking, the soil behind the retaining walls will exert an additional horizontal force on the walls. We recommend using the following additional seismic equivalent fluid pressures (EFP) to model the earthquake-induced force on the walls. The seismic equivalent fluid pressures were selected based on the design response spectrum peak ground acceleration (PGA), 2/3 of the Maximum Considered Earthquake (MCE) PGA. The PGA was 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 *Seismic Earth Pressures on Retaining Structures in Cohesive Soils,* a seismic equivalent fluid pressure equal to the following was used for cantilever retaining walls.

Cantilever (level): EFP = 0.42γ(PGA-0.15) = 30 PCF

Cantilever (2:1 slope): EFP = 0.70γ(PGA) = 61 PCF

Where γ is the total unit weight of the retaining wall backfill soil. This is based on load factors of 1.2 used for earth pressures and 1.0 for the seismic EFP was reduced by 1.2.

The following seismic design parameters in Table 6-3 below are from Chapter 16 of the 2019 California Building Code for Site Class D type soils (California Building Code, 2019) and ASCE 7-16, and latitude 37.440413, longitude -121.399563. The design parameters yielded an MCE PGA of 0.871g.

Table 6-3. Seismic Design Parameters

Item	Factor or Coefficient	Value
Site Class Definition	Site Class	D
0.2 Second Spectral Response Acceleration	S_S	1.896
1.0 Second Spectral Response Acceleration	S_1	0.714
Values of Site Coefficient	F_a	1.0
Value of Site Coefficient	F_v	--
Designed Spectral Response Acceleration for Short Periods	S_{DS}	1.264
Designed Spectral Response Acceleration for 1-Second Periods	S_{D1}	--

6.4.4 Retaining Wall Drainage

The equivalent fluid pressures assume fully drained conditions behind the soldier pile walls (temporary and permanent). 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. As an alternative to the Class 2 Permeable drainage material, a clean coarse gravel or drain rock may be used. 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 drain rock mentioned above, 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 manufacturers' recommendations on the back of the tieback wall at least 1 foot below the ground surface. It should be wrapped around a drainage pipe at the base of the wall.

6.4.5 Construction Considerations

The bottoms of soldier piles should be dry and free of loose cuttings and debris prior to the installation of the steel beams and concrete. This shall be done to the satisfaction of 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 to the depths of 21 feet during the exploratory drilling, and seepage was observed on the hillside at approximately 9 feet below the road surface. 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 a greater depth to the satisfaction of the engineer or geologist from our office observing the drilling operation.

6.5 EARTHWORK

Earthwork required for the project will include excavation to develop temporary site access and create a bench for drilling and construction of any earthwork repair. 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 and grubbed of existing vegetation. Care should be taken not to damage any utilities present. This should be done in coordination with utility providers. All existing structures and debris should be removed from the site, including but not limited to existing pavement, foundation systems, buried pipes, large fallen trees, and other related improvements. 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 and demolition of structures should be excavated to firm soil or bedrock before placement of fill.

6.5.1 Excavations

Excavations for the project are anticipated to be up to approximately 20 feet deep to construct the repair alternatives. Due to the existing road upslope surcharge, the slope must be shored with a retaining wall before any excavation. We recommend that all temporary shoring is designed in conformance with the State of California Department of Transportation, Trenching and Shoring Manual. A qualified California-registered civil engineer shall prepare the shoring design.

Temporary excavation slopes should not be steeper than 1.5H:1V and should conform to the Occupational Safety and Health Administration (OSHA) requirements for earthen slopes. CE&G should be retained to observe subsurface conditions during excavation to confirm assumed conditions and provide revised recommendations.

6.5.2 Material for Engineered Fill

On-site soils with an organic content of less than 3 percent by weight, free of any hazardous or deleterious materials, and meeting the gradation requirements below may be used as general engineered fill to achieve project grades.

General engineered fill material should not contain rocks or lumps larger than 4 inches in greatest dimension, should not contain more than 15 percent of the material larger than 2½ inches, and should contain at least 20 percent passing the No. 200 sieve.

Before delivery to the site, all import fills should be approved by the project geotechnical engineer. At least five (5) working days before importing to the site, a representative sample of the proposed import fill should be delivered to our laboratory for evaluation.

6.5.3 Engineered Fill Placement and Compaction

Before the commencement of the earthwork operation, the site should be cleared and grubbed of existing vegetation. Care should be taken not to damage existing utilities, including the underground sewer pipeline that runs beneath the road. It may be necessary to pothole and locate certain utilities if present. This should be done in coordination with utility providers. All existing structures and debris should be removed from the site. Before placement of fill materials, loose soil and vegetation should be removed from the areas to receive fill. All depressions created by tree and stump removal and demolition of structures should be excavated to firm soil before placement of fill.

Fill materials shall be spread evenly and compacted in uniform lifts not exceeding 8 inches in compacted thickness to facilitate geogrid placement at the design intervals. Materials shall be compacted to a relative compaction of 95 percent of ASTM D1557 unless specified otherwise. Fill materials that do not meet the specified relative compaction shall be ripped, moisture conditioned, and reworked until the required relative compaction and moisture content are attained. Moisture conditioning of soils should consist of adding water to the soil if it is too dry and allowing the soil to dry if it is too wet.

6.5.4 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 imported 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.

6.5.5 Erosion Control

Disturbance to the areas around the project site should be minimized as much as possible. Areas disturbed by construction activities should be protected from erosion by installing erosion control mats. As an alternative, hydroseeding may be considered and should consider a seed mix containing seeds for local native drought-resistant vegetation.

The tops of fill or cut slopes should be graded in such a way as to prevent water from ponding on the road or flowing freely across the face of the slopes. A positive gradient of 5% away from the face of the slope should be provided to direct surface water runoff away from the slopes to appropriate drainage points. Completed slopes should be provided with erosion control measures prior to the winter season following grading.

6.5.6 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 causing flooding of 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 to protect and minimize damage to their work by rainstorms.

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. A pipe should convey collected surface water from the roadway to a discharge point below active sliding or gullying. 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 bulk sample between 0 to 5 feet deep from boring B-1 was chosen to have resistivity and chloride and sulfate tests at Cooper Testing Laboratory. Based on test results, the sample was classified as one with low corrosion potential. However, based on the site's proximity to the Pacific Ocean, for design purposes, the County should use a coating for all steel beams and Class 1 corrosion protection for tiebacks. If the County has previous experience with the corrosivity of the on-site soils and import material 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 interpolating 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. They 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 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 project's construction phases. 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. The project designers should carefully review any modifications included in these addenda or memoranda 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 only presents the results of a geotechnical and geologic investigation 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 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.

8.0 REFERENCES

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Figures

BASEMAP REFERENCE

1. STREET CENTERLINES FROM CALTRANS CALIFORNIA ROAD SYSTEM, DOWNLOADED ON 18 FEB 2020.
2. ORTHOIMAGERY FROM ESRI (MAXAR), 2019.

0 0.25 0.5 1
MILES

N

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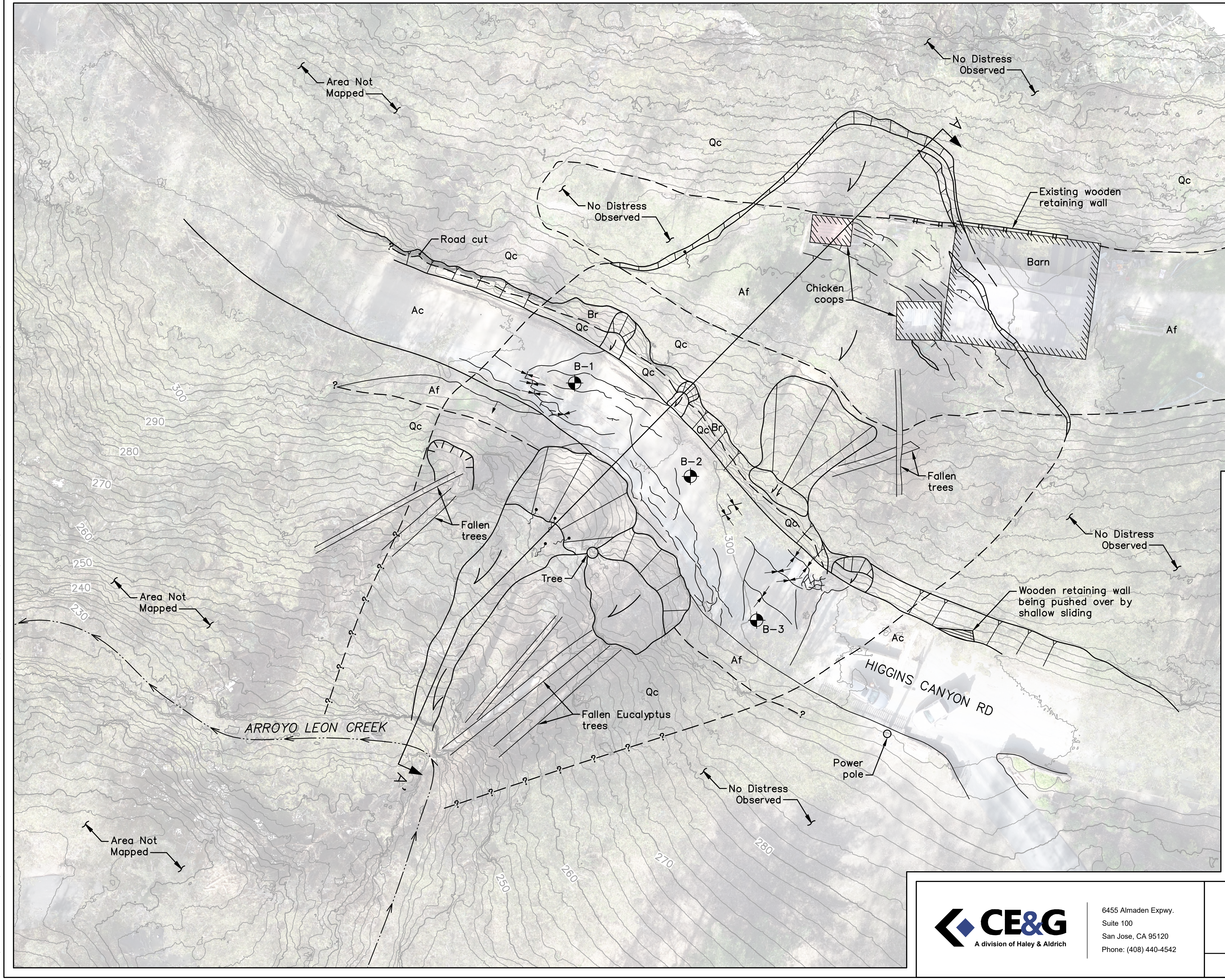
SLOPE STABILIZATION PROJECT
2180 HIGGINS CANYON ROAD
HALF MOON BAY, CALIFORNIA

SITE LOCATION MAP

0207686	APRIL2023	FIGURE 1



MAP UNIT DESCRIPTION

Qc	–	QUATERNARY COLLUVIUM
Br	–	PURISSMA FORMATION BEDROCK
Af	–	ARTIFICIAL FILL / ALTERED GROUND
Ac	–	ASPHALT CONCRETE

GEOLOGIC CONTACT

TENSION CRACK

COMPRESSIONAL BUCKLING WITH ARROWS INDICATING FORCE DIRECTION

B-3 BORINGS BY CE&G, DRILLED ON 2/9/2023

SCARP

LANDSLIDE MASS

SEEPAGE

ROAD CUT

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.

15 0 15 30
SCALE FEET

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ROAD STABILIZATION PROJECT
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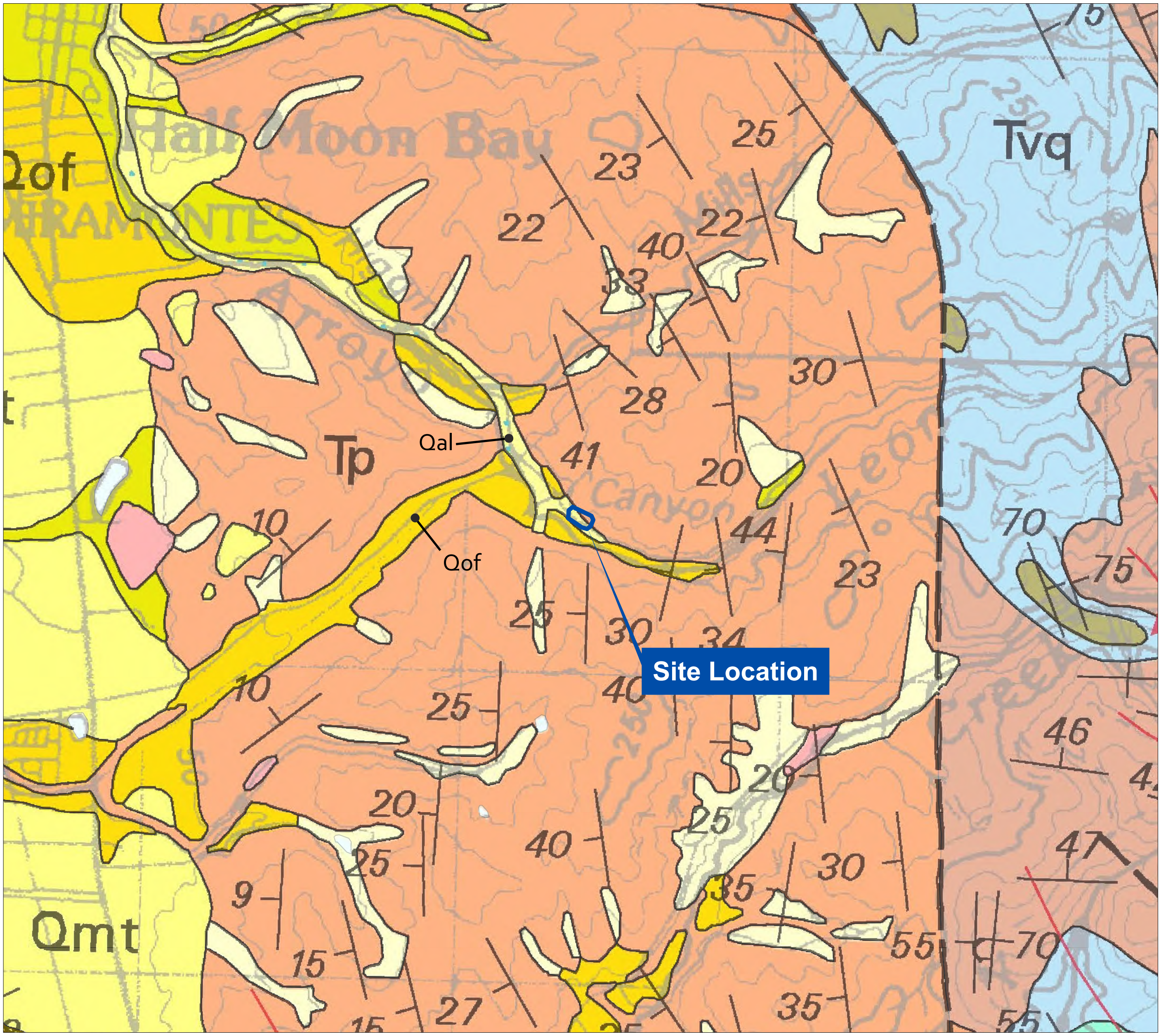
HALF MOON BAY, CALIFORNIA

SITE PLAN

0207686	APRIL 2023	FIGURE 2



BASEMAP REFERENCE

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

0 0.25 0.5 1
MILES

N

MAP UNIT DESCRIPTION

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

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SLOPE STABILIZATION PROJECT
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HALF MOON BAY, CALIFORNIA

REGIONAL GEOLOGY MAP

0207686	APRIL 2023	FIGURE 3

BASEMAP REFERENCE

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

0 1.5 3 6
MILES

N

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
- Middle and late Quaternary (<750,000 years), well constrained location
- Middle and late quaternary (<750,000 years), moderately constrained location
- Middle and late Quaternary (<750,000 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
- Class B (various age), well constrained location
- Class B (various age), moderately constrained location
- Class B (various age), inferred location

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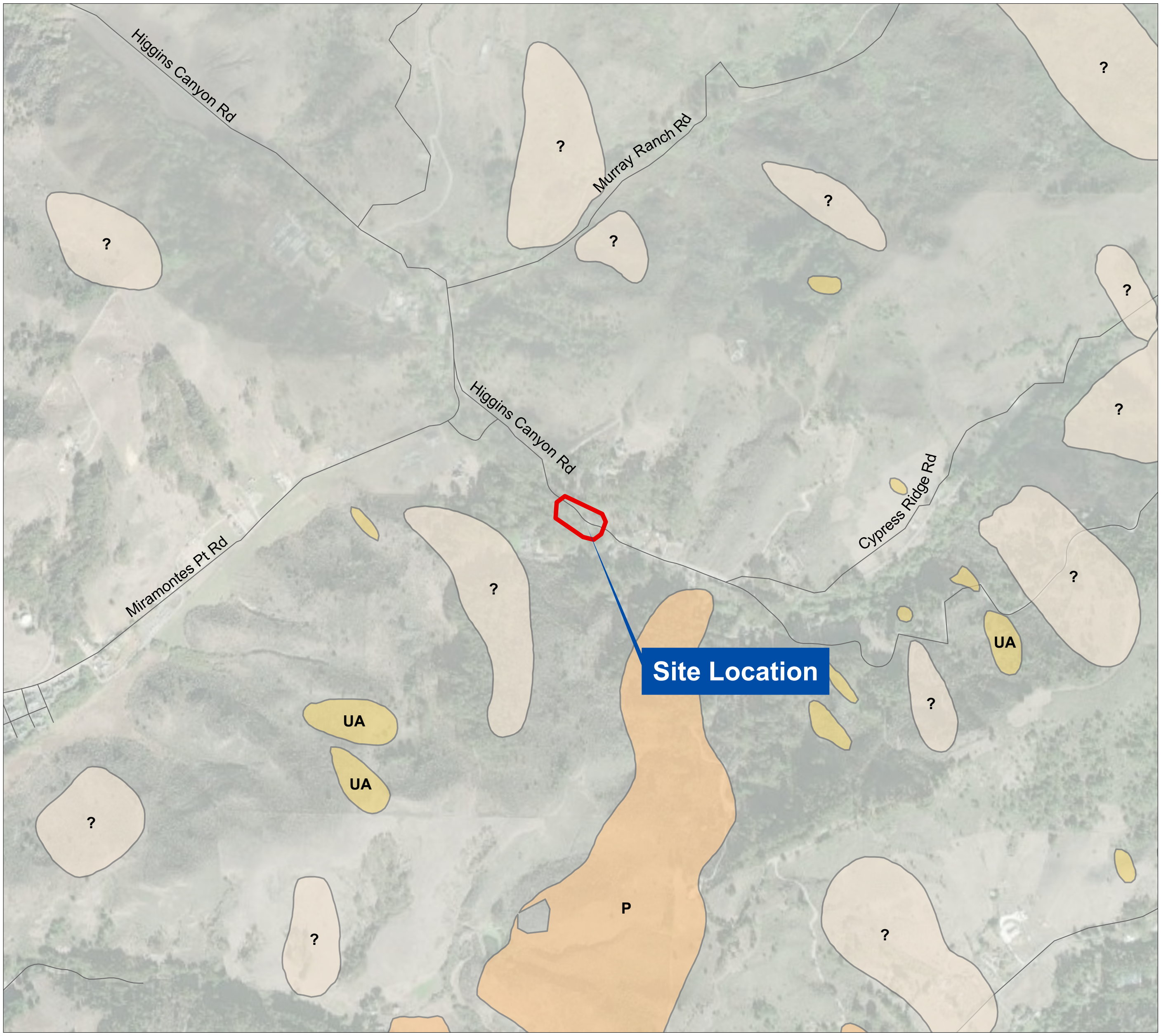
2180 HIGGINS CANYON ROAD
HALF MOON BAY, CALIFORNIA

FAULT ACTIVITY MAP

0207686	APRIL 2023	FIGURE 4



BASEMAP REFERENCE

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

0 0.13 0.25 0.5 MILES

N

MAP UNIT DESCRIPTION

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

\\haleyaldrich.com\share\granite\2023\0207686-SanMateoCo-SlipoutRepair-2180HigginsCnynRd\GIS\ArcGIS\0207686-2180HigginsCnynRd.aprx; 3/24/2023; kdrozynska

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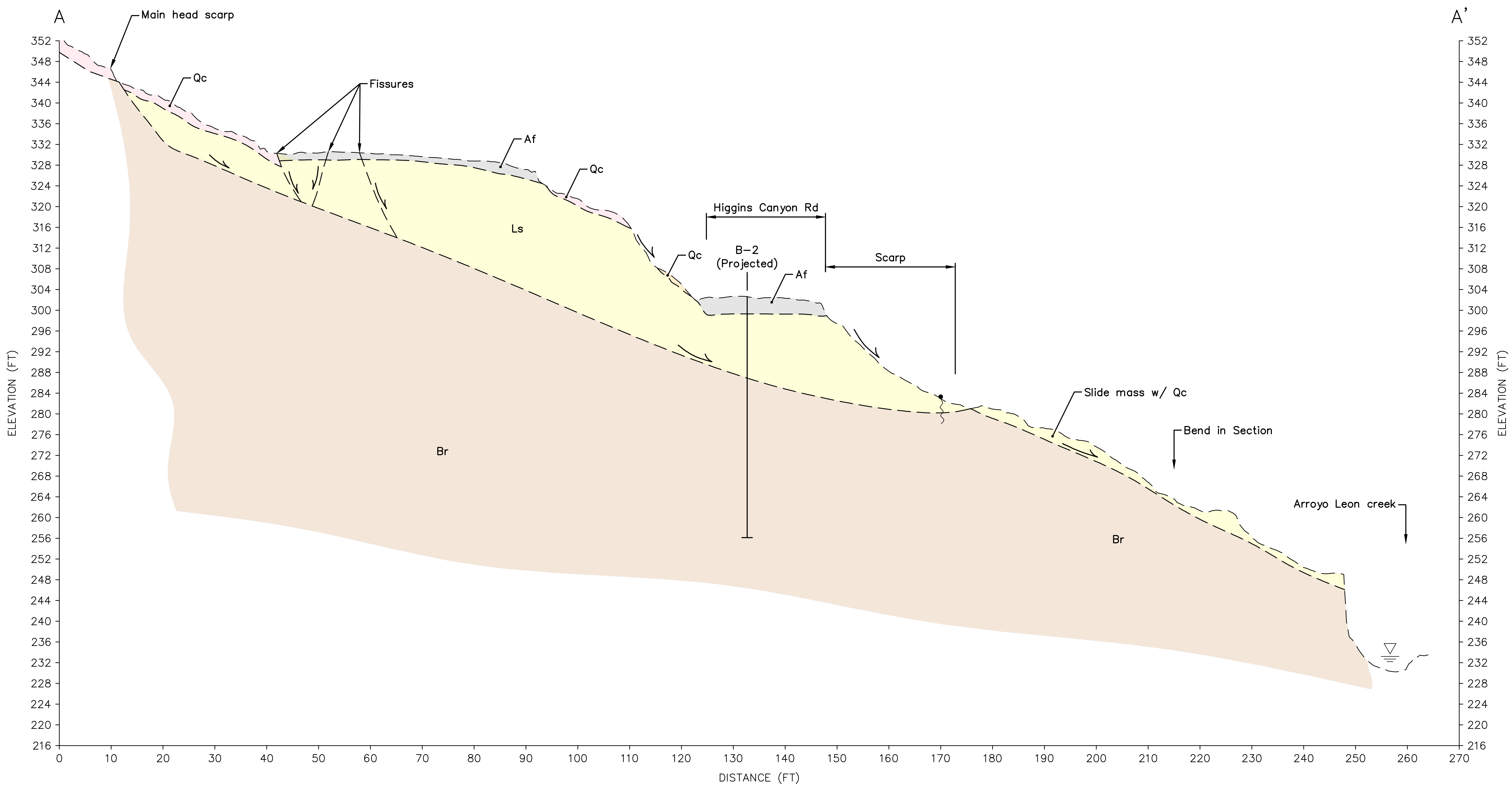
SLOPE STABILIZATION PROJECT
2180 HIGGINS CANYON ROAD
HALF MOON BAY, CALIFORNIA

LANDSLIDE INVENTORY MAP

0207686	APRIL 2023	FIGURE 5



REFERENCES

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

MAP UNIT DESCRIPTION

Qc – QUATERNARY COLLUVIUM
Br – PURISSMA FORMATION BEDROCK
Af – ARTIFICIAL FILL
Ac – ASPHALT CONCRETE
SEEPAGE

10 0 10 20
SCALE FEET

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ROAD STABILIZATION PROJECT
NEAR 2180 HIGGINS CANYON ROAD
HALF MOON BAY, CALIFORNIA

CROSS SECTION A-A'

0207686	APRIL 2023	FIGURE 6

Appendix A. Site Photographs

Photo 1: View of first slip-out (facing southeast) during our initial site visit on January 10th, 2023.

Photo 2: Northwest-facing view of the original scarp during our second site visit on February 20th, 2023

Photo 3: West-facing view of the downslope extent of the original slip-out along the outboard edge of Higgins Canyon Road. Photo was taken during our initial site visit on January 10th, 2023.

Photo 4: Southwest-facing view of the slip-out downslope of Higgins Canyon Road. Several seepage areas were observed along the base of the scarp (see black arrow). Photo was taken during our initial site visit on January 10th, 2023.

Photo 5: General southeast-facing view of slide area along the outboard edge of Higgins Canyon Road. Photo was taken during our last site visit on March 20th, 2023.

Photo 6: General southwest-facing view of slide area and fallen trees along the outboard edge of Higgins Canyon Road. Photo was taken during our last site visit on March 20th, 2023.

Photo 7: General southeast-facing view of the outboard edge of Higgins Canyon Road. Photo was taken during our last site visit on March 20th, 2023.

Photo 8: Approximately 13 inches of vertical displacement in Higgins Canyon Road pavement. Photo was taken during our last site visit on March 20th, 2023.

Photo 9: Vertical displacement in Higgins Canyon Road pavement due to buckling and thrusting. Photo was taken during our last site visit on March 20th, 2023.

Photo 10: Southeastern extent of road deformation expressed as a crack with left lateral movement during our initial site visit on January 10th, 2023.

Photo 11: Buckling and thrusting of Higgins Canyon Road pavement near the inboard edge of the road and at the eastern extent of the road deformation. Photo was taken during our last site visit on March 20th, 2023.

Photo 12: Northwestern view of the road cut along the inboard edge of Higgins Canyon Road. Photo was taken during our last site visit on March 20th, 2023.

Photo 13: Northeast-facing view of the retaining wall on the upslope private property at (2175 Higgins Canyon Road). Note the main landslide scarp exposed upslope of the retaining wall. Also note tension cracks in the grass in front of the retaining wall. Photo was taken during our last site visit on March 20th, 2023.

Photo 14: Southeast-facing view of tension cracks on the upslope private property at (2175 Higgins Canyon Road). Photo was taken during our last site visit on March 20th, 2023.

Photo 15: Northeast-facing view of the main scarp exposed beneath the barn on the upslope private property at (2175 Higgins Canyon Road). Note the main landslide scarp continues upslope after cutting beneath the barn. Also, note exposed barn foundation piers. Photo was taken during our last site visit on March 20th, 2023.

Photo 16: East-facing view of the northeastern extent of the slide deformation on the upslope private property at (2175 Higgins Canyon Road). Photo was taken during our last site visit on March 20th, 2023.

Photo 17: Northwest-facing view of the exposed main landslide scarp, upslope of the private property at 2175 Higgins Canyon Road. Photo was taken during our last site visit on March 20th, 2023.

Photo 18: North-facing view of the exposed northwestern extent of the main landslide scarp on the private property at 2175 Higgins Canyon Road. Photo was taken during our last site visit on March 20th, 2023.

Appendix B. Boring Logs

UNIFIED SOIL CLASSIFICATION SYSTEM (ASTM D-2487)

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

CLASSIFICATION OF GRAVELS & SANDS WITH 5% TO 12% FINES REQUIRES DUAL SYMBOLS

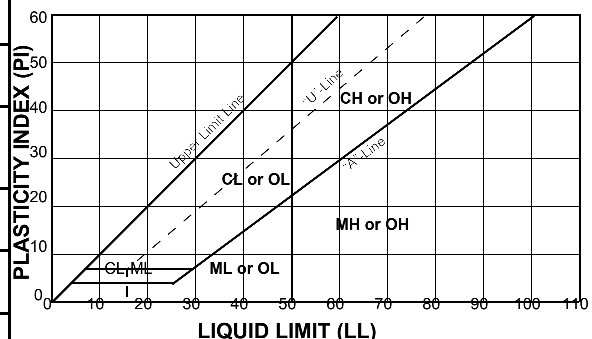
GW/GM or GP/GM: Gravel/Silty Gravel
GW/GC or GP/GC: Gravel/Clayey Gravel
SW/SM or SP/SM: Sand/Silty Sand
SW/SC or SP/SC: Sand/Clayey Sand

Field Identification		Group Symbols	Typical Names
Fine-Grained Soils More than 50% of material passes the No. 200 sieve.	**Identification Procedures on Percentage Passing the No. 40 Sieve**		
	Silts & Clays Liquid Limit less than 50%	**ML**	Inorganic silts, very fine sands, rock flour, silty or clayey fine sands with slight plasticity
		CL	Inorganic clays of low to medium plasticity, gravelly, sandy, and/or silty clays, lean clays
		OL	Organic silts, organic silty clays of low plasticity
	Silts & Clays Liquid Limit greater than 50%	**MH**	Inorganic silts, micaceous or diatomaceous fine sandy/-silty soil, elastic silts
		CH	Inorganic clays of high plasticity, fat clays
		OH	Organic clays of medium to high plasticity
HIGHLY ORGANIC SOILS		**PT**	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 to 25.5, then PI = 0.73 × (LL − 20)
Equation of "U"-Line: LL = 16 @ PI = 0 to 7, then PI = 0.9 × (LL − 8)



KEY TO SAMPLER TYPES AND OTHER LOG SYMBOLS

CS	California Standard Sampler	▽	Depth at which Groundwater was Encountered During Drilling
CM	California Modified Sampler	▼	Depth at which Groundwater was Measured After Drilling
SPT	Standard Penetration Test Sampler	**PP**	Pocket Penetrometer Test
SHL	Shelby Tube Sampler	**PTV**	Pocket Torvane Test
BU	Bulk Sample	**-#200**	% of Material Passing the No. 200 Sieve Test (ASTM D-1140)
LL	Liquid Limit of Sample (ASTM D-4318)	**PSA**	Particle-Size Analysis (ASTM D-422 & D-1140)
PI	Plasticity Index of Sample (ASTM D-4318)	**C**	Consolidation Test (ASTM D-2435)
Q_U	Unconfined Compression Test (ASTM D-2166)	**TXUU**	Unconsolidated Undrained Compression Test (ASTM D-2850)

KEY TO SAMPLE INTERVALS

CS	Length of Sampler Interval with a CS Sampler	BU	Bulk Sample Recovered for Interval Shown (i.e., cuttings)
⊠	Length of Sampler Interval with a CM Sampler	⌶	Length of Coring Run with Core Barrel Type Sampler
	Length of Sampler Interval with a SPT Sampler	NR	No Sample Recovered for Interval Shown
SHL	Length of Sampler Interval with a SHL Sampler		

Rock Hardness Descriptions

Very Hard	Cannot be scratched with knife or sharp pick. Breaking of hand specimen requires several hard blows of geologist's pick.
Hard	Can be scratched with knife or pick only with difficulty. Hard blow of hammer required to detach hand specimen.
Moderately Hard	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.
Medium	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.
Soft	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.
Very Soft	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

Fresh	Rock fresh, crystals bright, few joints may show slight staining. Rock rings under hammer if crystalline.
Very Slight	Rock generally fresh, joints may show thin clay coatings, crystals in broken face show bright. Rock rings under hammer if crystalline.
Slight	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.
Moderate	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.
Moderately Severe	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.
Severe	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.
Very Severe	All rock except quartz discolored or stained. Rock "fabric" discernible. But mass effectively reduced to "soil" with only fragments of strong rock remaining.
Complete	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, "Subsrface Investigation For Design And Construction Of Foundations Of Buildings," 1976.

KEY TO SYMBOLS

CE&G
A division of Haley & Aldrich

CLIENT San Mateo County
PROJECT NAME 2180 Higgins Canyon Road Slope Stabilization
PROJECT NUMBER 0207686
PROJECT LOCATION Half Moon Bay

LITHOLOGIC SYMBOLS
(Unified Soil Classification System)

ASPHALT: Asphalt

BEDROCK: Bedrock

CL: USCS Low Plasticity Clay

SAMPLER SYMBOLS

California Modified Sampler

Standard Penetration Test

WELL CONSTRUCTION SYMBOLS

ABBREVIATIONS

LL - LIQUID LIMIT (%)
PI - PLASTIC INDEX (%)
W - MOISTURE CONTENT (%)
DD - DRY DENSITY (PCF)
NP - NON PLASTIC
-200 - PERCENT PASSING NO. 200 SIEVE
PP - POCKET PENETROMETER (TSF)

TV - TORVANE
PID - PHOTOIONIZATION DETECTOR
UC - UNCONFINED COMPRESSION
ppm - PARTS PER MILLION
▽ Water Level at Time Drilling, or as Shown
▼ Water Level at End of Drilling, or as Shown
▼ Water Level After 24 Hours, or as Shown

BORING NUMBER B-1

CE&G — A division of Haley & Aldrich

CLIENT San Mateo County

PROJECT NAME 2180 Higgins Canyon Road Slope Stabilization

PROJECT NUMBER 0207686

PROJECT LOCATION Half Moon Bay

DATE STARTED 2/9/2023 **COMPLETED** 2/9/2023

GROUND ELEVATION 303 ft **DATUM** NAVD88 **HOLE SIZE** 6 in.

DRILLING CONTRACTOR Cenozoic Drilling

COORDINATES: LATITUDE 37.440324 **LONGITUDE** -122.399391

DRILLING RIG/METHOD 6-in. Solid Flight Auger/Geoprobe 7822DT

▽ **GROUNDWATER AT TIME OF DRILLING** 9.5 ft / Elev 293.5 ft

LOGGED BY C. Rodil **CHECKED BY** K. Loeb

GROUNDWATER AT END OF DRILLING --- Not Measured

HAMMER TYPE 140 lb hammer with 30 in. autotrip

GROUNDWATER AFTER DRILLING --- Not Measured

DEPTH (ft)	MATERIAL DESCRIPTION	SAMPLE TYPE	BLOW COUNTS (FIELD VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT (%)	ATTERBERG LIMITS: PLASTIC LIMIT (%)	ATTERBERG LIMITS: PLASTICITY INDEX (%)	FINES CONTENT (%)
0	Approximately 2" Aspahltic Concrete									
	Lean CLAY w/ Sand (CL): olive yellow (2.5Y 6/6). moist, very soft, heavy iron staining, some angular gravel from local bedrock (crumbles w/ finger pressure) [FILL/Landslide Debris] Corrosion test & R-Value @ 1 - 5 ft	CM	3-2-3		85	25				
		SPT	0-0-1							87
5		CM	3-3-3							
	-clayey lense, red	SPT	1-1-1			28				
10	▽ Fat CLAY w/ Sand (CL): light yellowish brown (2.5Y 6/4), wet, high plasticity fines, fine to coarse sand, friable gravel (crushed bedrock fragments) up to 1.25" [Landslide Debris] TXUU test @ 11.0 ft	CM	3-2-3		82	38	61	28	33	98
15	-Approxiamte Base of Landslide									
	Sandy CLAYSTONE (BEDROCK): light gray (2.5Y 7/2), dry, medium to moderately hard, moderately severe to severe weathering [Purisima Formation Bedrock]	CM	10-26-50/4"		94	28				
20	-becomes light olive brown (2.5Y 5/3), decreases in sand, heavy iron staining	CM	15-36-50		82	30				
25	Sandy CLAYSTONE (BEDROCK): dark greenish gray (GLEY 1 4/1), dry, moderately hard -becomes moist, increase in weathering	CM	28-50/4"		96	23				

Bottom of borehole at 25.8 ft. Borehole backfilled with neat cement grout.

BORING NUMBER B-2

CE&G
A division of Haley & Aldrich

CLIENT San Mateo County
PROJECT NAME 2180 Higgins Canyon Road Slope Stabilization
PROJECT NUMBER 0207686
PROJECT LOCATION Half Moon Bay
DATE STARTED 2/9/2023 **COMPLETED** 2/9/2023
GROUND ELEVATION 301 ft **DATUM** NAVD88 **HOLE SIZE** 6 in.
DRILLING CONTRACTOR Cenozoic Drilling
COORDINATES: LATITUDE 37.440246 **LONGITUDE** -122.399265
DRILLING RIG/METHOD 6-in. Solid Flight Auger/Geoprobe 7822DT
GROUNDWATER AT TIME OF DRILLING 10.5 ft / Elev 290.5 ft
LOGGED BY C. Rodil **CHECKED BY** K. Loeb
GROUNDWATER AT END OF DRILLING --- Not Measured
HAMMER TYPE 140 lb hammer with 30 in. autotrip
GROUNDWATER AFTER DRILLING 21.0 ft / Elev 280.0 ft

DEPTH (ft)	MATERIAL DESCRIPTION	SAMPLE TYPE	BLOW COUNTS (FIELD VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT (%)	ATTERBERG LIMITS: PLASTIC LIMIT (%)	ATTERBERG LIMITS: PLASTICITY INDEX (%)	FINES CONTENT (%)
0	Approximately 2" Aspahltic Concrete									
	Lean CLAY (CL): pale yellow (2.5Y 7/4), dry, stiff, low plasticity, friable gravel from local bedrock [FILL]	CM	9-9-11		86	29				88
	SILTSTONE (BEDROCK): pale yellow (2.5Y 7/4), dry, soft rock, severe weathering, iron staining, rock fragments friable to silt, some clayey pockets [Landslide Debris/Purisima Formation Bedrock]	SPT	4-4-5							
5	-becomes brownish yellow (10YR 6/6), dry to moist, moderately severe to moderate weathering, iron staining	CM	5-5-6		88	27				
		SPT	2-4-3							
10	-perched water in sample	CM	5-6-8		91	26				81
	Lean CLAY (CL): light olive brown (2.5Y 5/6), moist to wet, medium stiff, low plasticity, some siltstone fragments from local bedrock (friable to silt), iron staining [Landslide Debris]	SPT	2-2-3							
		CM	4-6-7		81	35				98
15		SPT	1-2-2							
	-becomes wet, soft									
	-Approximate Base of Landslide									
	SILTSTONE (BEDROCK): light brownish gray (10YR 6/2), soft to medium rock, moderately severe to severe weathering, iron staining [Purisima Formation Bedrock]	CM	10-22-38		92	30	62	29	33	98
20	TXUU test @ 18.0 ft	SPT	11-16-18							
	Sandy CLAYSTONE (BEDROCK): light yellowish brown (2.5Y 6/4), dry, soft to medium rock, moderate weathering									
	-becomes modertaely hard to hard rock	CM	16-37-50		88	28				
	-3" clayey lense, red	SPT	11-19-28							
25	-heavy iron staining									
	CLAYSTONE (BEDROCK): dark greenish gray (GLEY 1 4/1), moist, very soft rock, fresh to very slight weathering	CM	13-16-21		88	33				
30		CM	9-16-24		93	28				
35										

(Continued Next Page)

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CLIENT San Mateo County **PROJECT NAME** 2180 Higgins Canyon Road Slope Stabilization

PROJECT NUMBER 0207686 **PROJECT LOCATION** Half Moon Bay

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		CLAYSTONE (BEDROCK): dark greenish gray (GLEY 1 4/1), moist, very soft rock, fresh to very slight weathering *(continued)*	CM	9-15-26							
		-rig chatter in very soft rock									
40			CM	9-23-30		92	28				
45			CM	8-20-35							

Bottom of borehole at 46.5 ft. Borehole backfilled with neat cement grout.

BORING NUMBER B-3

CE&G
A division of Haley & Aldrich

CLIENT San Mateo County
PROJECT NAME 2180 Higgins Canyon Road Slope Stabilization
PROJECT NUMBER 0207686
PROJECT LOCATION Half Moon Bay
DATE STARTED 2/9/2023 **COMPLETED** 2/9/2023
GROUND ELEVATION 299 ft **DATUM** NAVD88 **HOLE SIZE** 6 in.
DRILLING CONTRACTOR Cenozoic Drilling
COORDINATES: LATITUDE 37.440124 **LONGITUDE** -122.39919
DRILLING RIG/METHOD 6-in. Solid Flight Auger/Geoprobe 7822DT
▽ **GROUNDWATER AT TIME OF DRILLING** 6.3 ft / Elev 292.8 ft
LOGGED BY C. Rodil **CHECKED BY** K. Loeb
GROUNDWATER AT END OF DRILLING --- Not Measured
HAMMER TYPE 140 lb hammer with 30 in. autotrip
GROUNDWATER AFTER DRILLING --- Not Measured

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE	BLOW COUNTS (FIELD VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT (%)	PLASTIC LIMIT (%)	PLASTICITY INDEX (%)	FINES CONTENT (%)
0		Approximately 2" Asphaltic Concrete									
		SILTSTONE (BEDROCK): brownish yellow (10YR 6/8), moist, soft rock, severely weathered, friable to silt [Purisima Formation/Landslide Debris]	CM	6-11-9							
			SPT	3-3-2			25				
5			CM	4-6-6	2.75						
		▽ Lean CLAY (CL): moist, soft, low plasticity, trace fine to coarse sand, fine gravel friable to silt	SPT	2-2-2			39				97
		-Approximate Base of Landslide									
10		CLAYSTONE (BEDROCK): olive brown (2.5YR 4/4), moist, low plasticity fines, very stiff to hard soil characteristics, very soft rock, severely weathered, fine to coarse sand [Purisima Formation Bedrock]	CM	7-14-22	4.5	84	34				99
			SPT	5-9-11							
15		Clayey SILTSTONE (BEDROCK): light yellowish brown (2.5YR 6/4), moist, soft rock, severely weathered, heavy iron staining, low plasticity fines	CM	9-15-20		83	33				95
			SPT	4-5-6							
20		-becomes medium to hard rock, moderate weathering	CM	12-22-38		95	26				
25		-becomes hard rock, moderate to slight weathering	CM	50/4"							
30		-medium to moderately hard rock, slight to moderate weathering	CM	50							

Bottom of borehole at 30.5 ft. Borehole backfilled with neat cement grout.

Appendix C. Laboratory Testing

SUMMARY OF LABORATORY RESULTS

CE&G
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CLIENT San Mateo County
PROJECT NAME 2180 Higgins Canyon Road Slope Stabilization
PROJECT NUMBER 0207686
PROJECT LOCATION Half Moon Bay

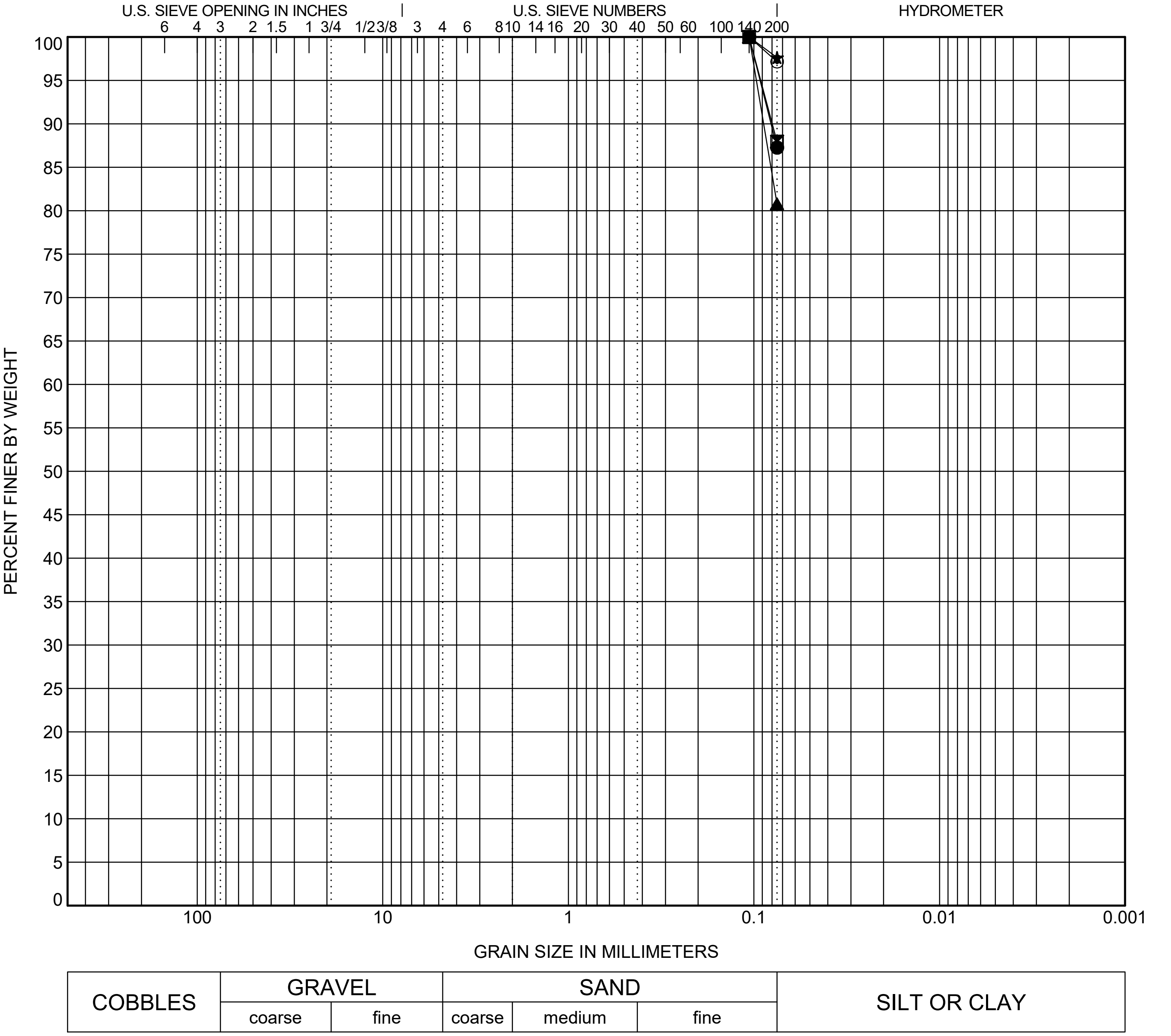
Borehole	Depth	Date Tested	Liquid Limit	Plastic Limit	Plasticity Index	Maximum Screen Size (mm)	%<#200 Sieve	Class-ification	Water Content (%)	Dry Density (pcf)	Satur-ation (%)	Void Ratio
B-1	2.0	2/15/2023							24.9	85.0		
B-1	2.5	2/15/2023				0.106	87					
B-1	6.5	2/15/2023							27.7			
B-1	11.0	2/27/2023	61	28	33	4.75	98	CH	37.5	81.7		
B-1	16.0	2/15/2023							28.3	93.9		
B-1	20.5	2/15/2023							30.0	82.0		
B-1	25.5	2/15/2023							23.0	96.1		
B-2	2.0	2/15/2023				0.106	88		29.1	86.4		
B-2	6.0	2/15/2023							27.4	87.7		
B-2	10.0	2/15/2023				0.106	81		26.5	91.2		
B-2	14.0	2/15/2023				0.106	98		35.2	81.2		
B-2	18.0	2/27/2023	62	29	33	2	98	CH	29.7	91.7		
B-2	22.5	2/15/2023							27.5	88.4		
B-2	28.5	2/15/2023							33.4	87.7		
B-2	31.0	2/15/2023							28.4	93.3		
B-2	41.0	2/15/2023							27.7	92.3		
B-3	2.5	2/15/2023							24.8			
B-3	6.5	2/15/2023				0.106	97		38.5			
B-3	11.0	2/15/2023				0.106	99		33.8	84.0		
B-3	16.0	2/15/2023				0.106	95		33.3	82.8		
B-3	21.0	2/15/2023							25.9	95.2		

GRAIN SIZE DISTRIBUTION

CE&G
CAL ENGINEERING & GEOLOGY

CLIENT San Mateo County **PROJECT NAME** 2180 Higgins Canyon Road Slip-Out

PROJECT NUMBER 0207686 **PROJECT LOCATION** Half Moon Bay



	BOREHOLE	DEPTH	DATE TESTED	Classification	LL	PL	PI	Cc	Cu
●	B-1	2.5	2/15/2023						
⧗	B-2	2.0	2/15/2023						
▲	B-2	10.0	2/15/2023						
★	B-2	14.0	2/15/2023						
⊙	B-3	6.5	2/15/2023						

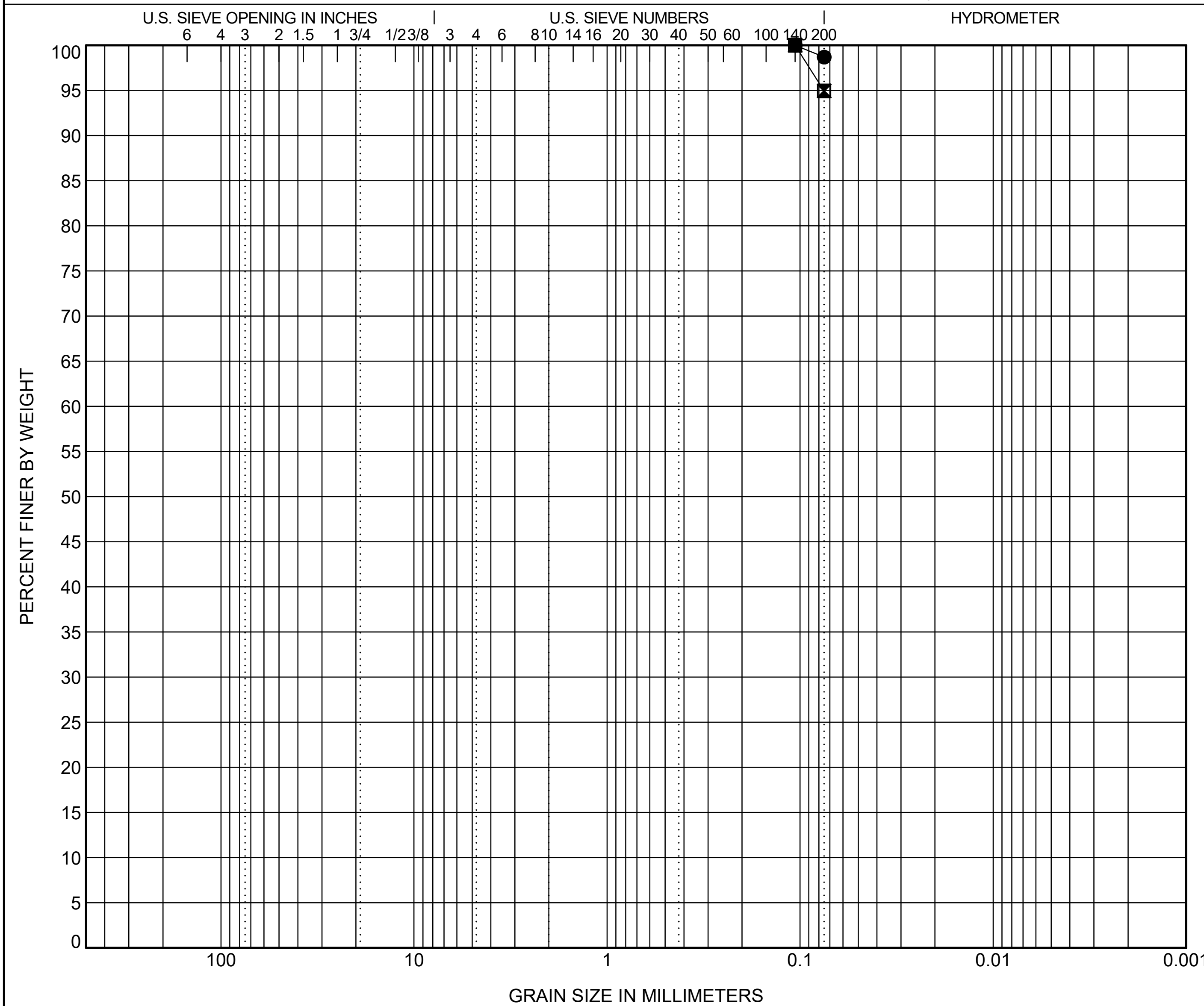
	BOREHOLE	DEPTH	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	B-1	2.5	0.106				0.0	12.7	87.3	
⧗	B-2	2.0	0.106				0.0	12.1	87.9	
▲	B-2	10.0	0.106				0.0	19.2	80.8	
★	B-2	14.0	0.106				0.0	2.4	97.6	
⊙	B-3	6.5	0.106				0.0	2.8	97.2	

CE&G
CAL ENGINEERING & GEOLOGY

GRAIN SIZE DISTRIBUTION

CLIENT San Mateo County **PROJECT NAME** 2180 Higgins Canyon Road Slip-Out

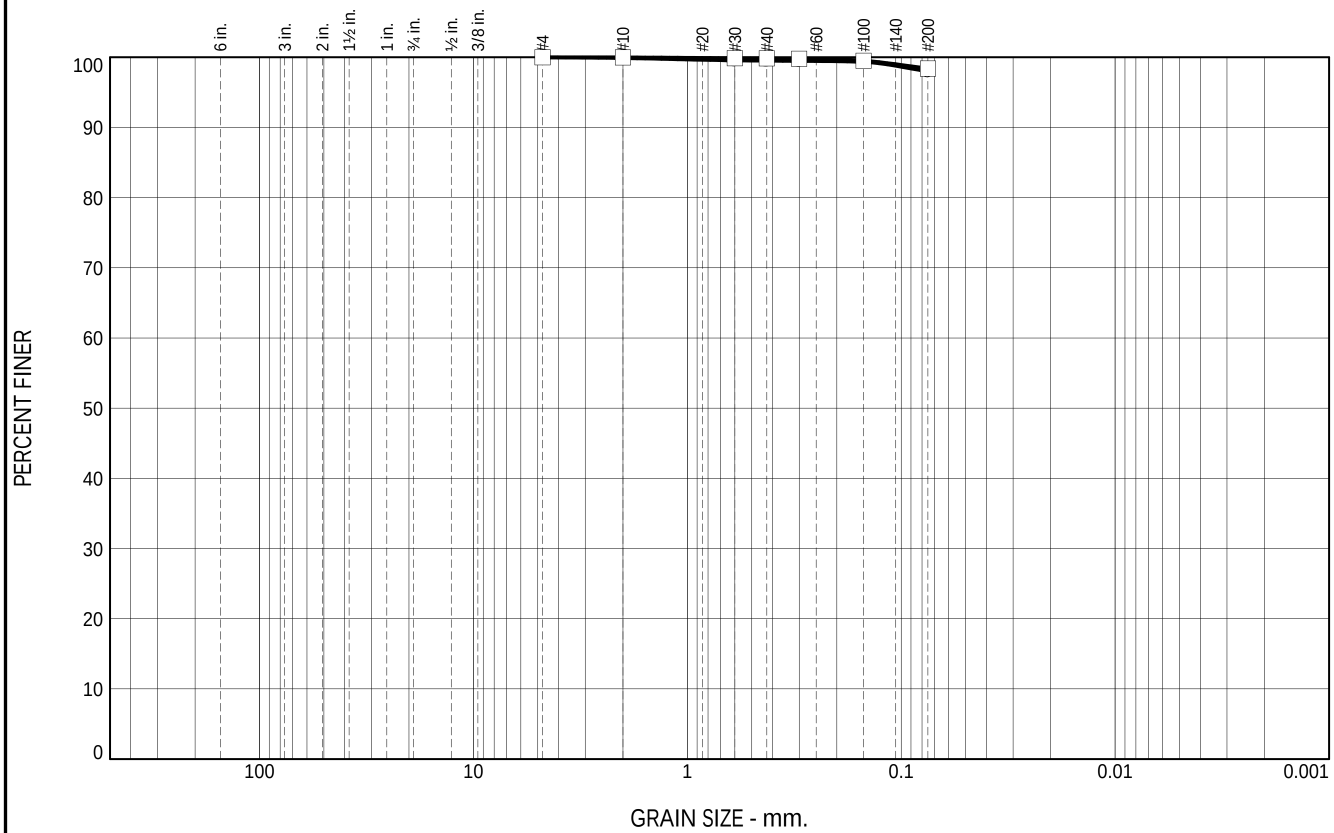
PROJECT NUMBER 0207686 **PROJECT LOCATION** Half Moon Bay



	BOREHOLE	DEPTH	DATE TESTED	Classification	LL	PL	PI	Cc	Cu
●	B-3	11.0	2/15/2023						
⌧	B-3	16.0	2/15/2023						

	BOREHOLE	DEPTH	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	B-3	11.0	0.106				0.0	1.3	98.7	
⌧	B-3	16.0	0.106				0.0	5.1	94.9	

Particle Size Distribution Report



	+3"	% GRAVEL	% SAND	% SILT	% CLAY	USCS	LL	PL	PI
○	0.0	0.0	2.0	98.0		CH	61	28	33
□	0.0	0.0	1.6	98.4		CH	62	29	33

SIEVE inches size	PERCENT FINER ○	□	
	GRAIN SIZE		
D_{60}			
D_{30}			
D_{10}			
	COEFFICIENTS		
C_c			
C_u			

SIEVE number size	PERCENT FINER ○	□	
#4	100.0	100.0	
#10	99.9	100.0	
#30	99.6	99.9	
#40	99.5	99.8	
#50	99.5	99.8	
#100	99.3	99.5	
#200	98.0	98.4	

Material Description

○ Olive Brown Fat CLAY

□ Olive Brown Fat CLAY (Claystone)

REMARKS:

○

□

○ Source of Sample: B-1 Depth: 11.0' Sample Number: 1-7

□ Source of Sample: B-2 Depth: 18.0' Sample Number: 2-4

COOPER TESTING LABORATORY

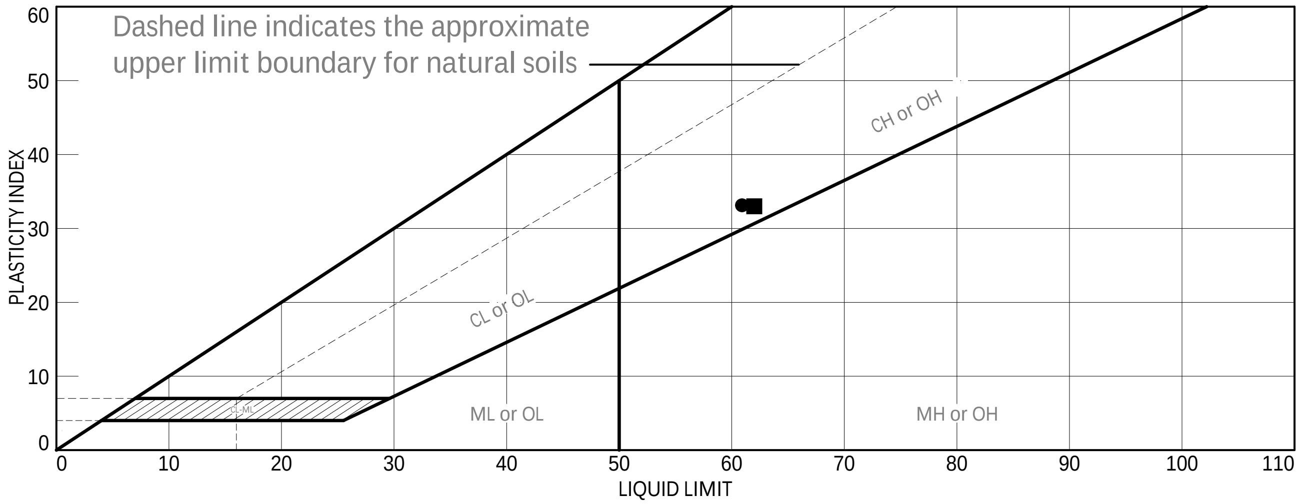
Client: Cal Engineering & Geology

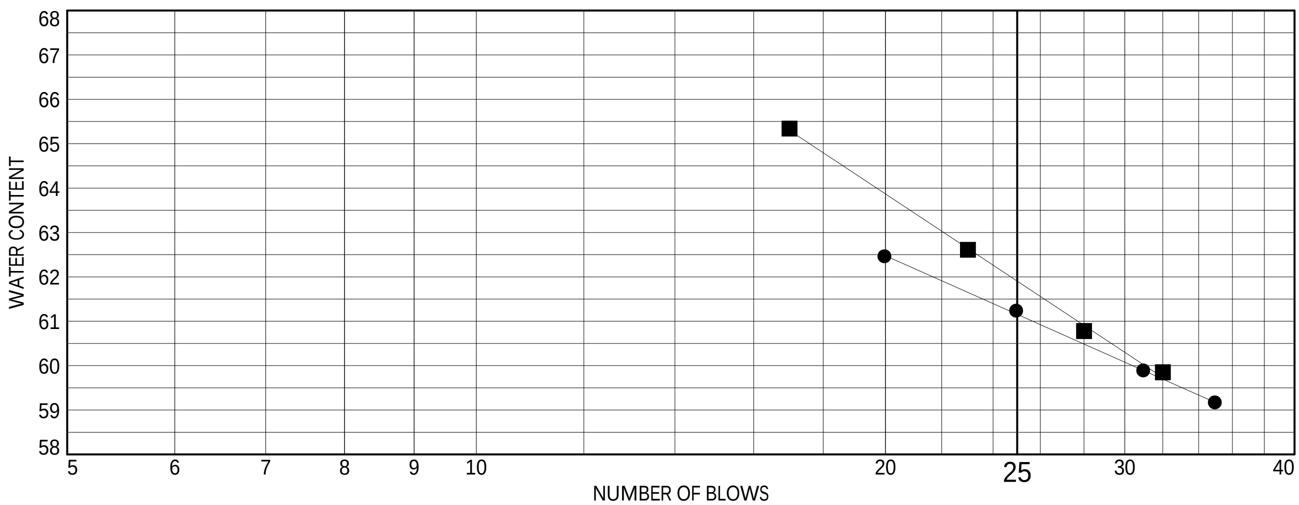
Project: 2180 Higgins Canyon Road - 00207686

Project No.: 471-405

Figure

LIQUID AND PLASTIC LIMITS TEST REPORT (ASTM D4318)





	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	Olive Brown Fat CLAY	61	28	33	99.5	98.0	CH
■	Olive Brown Fat CLAY (Claystone)	62	29	33	99.8	98.4	CH

Project No. 471-405 Client: Cal Engineering & Geology

Project: 2180 Higgins Canyon Road - 00207686

● Source of Sample: B-1 Depth: 11.0' Sample Number: 1-7

■ Source of Sample: B-2 Depth: 18.0' Sample Number: 2-4

Remarks:

COOPER TESTING LABORATORY

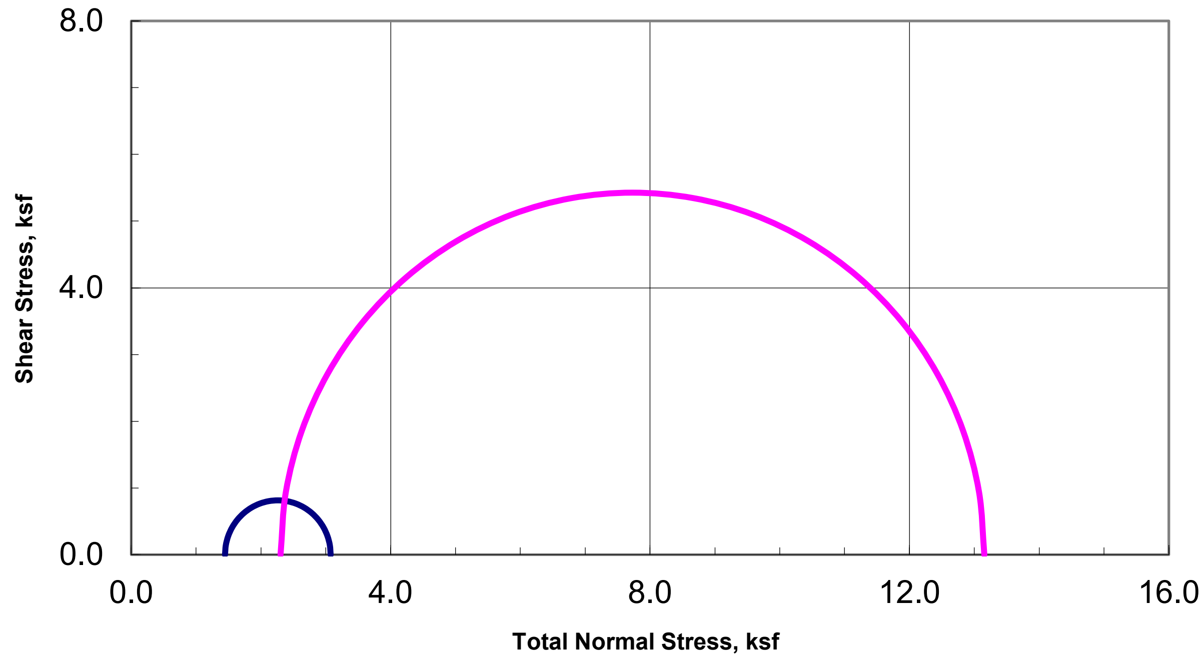
Figure

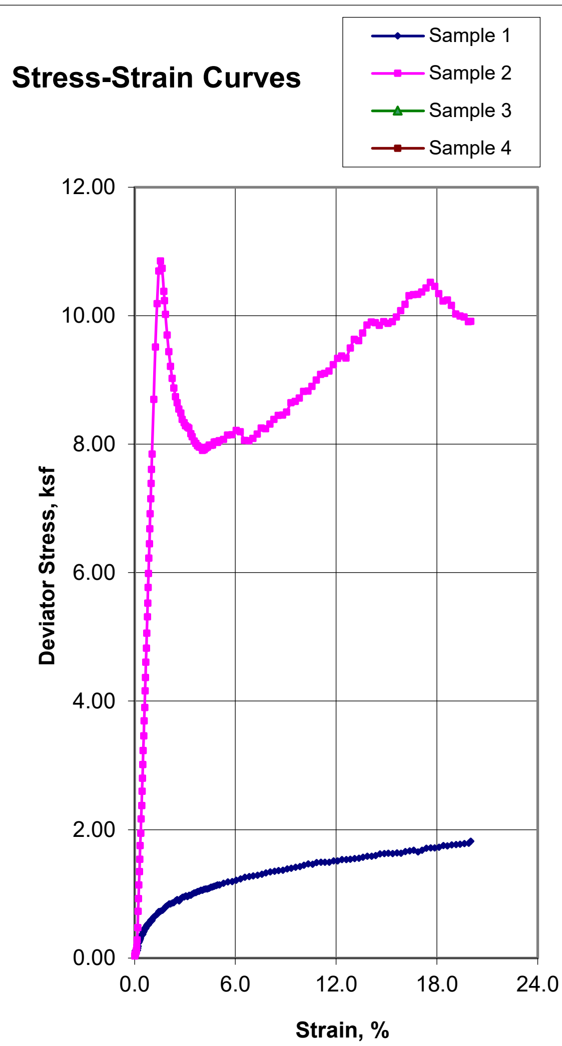
Cooper Testing Labs, Inc.
937 Commercial Street
Palo Alto, CA 94303

COOPER TESTING LABORATORY

Unconsolidated-Undrained Triaxial Test

ASTM D2850





Sample Data				
	1	2	3	4
Moisture %	37.5	29.7		
Dry Den,pcf	81.7	91.7		
Void Ratio	1.062	0.839		
Saturation %	95.2	95.7		
Height in	5.02	5.03		
Diameter in	2.39	2.42		
Cell psi	10.0	16.0		
Strain %	15.00	1.54		
Deviator, ksf	1.629	10.853		
Rate %/min	0.99	1.00		
in/min	0.050	0.050		
Job No.:	471-405			
Client:	Cal Engineering & Geology			
Project:	0207686			
Boring:	B-1	B-2		
Sample:	1-7	2-14		
Depth ft:	11.0	18.0		

Visual Soil Description	
Sample #	
1	Olive Brown Fat CLAY
2	Olive Brown Fat CLAY (Claystone)
3	
4	

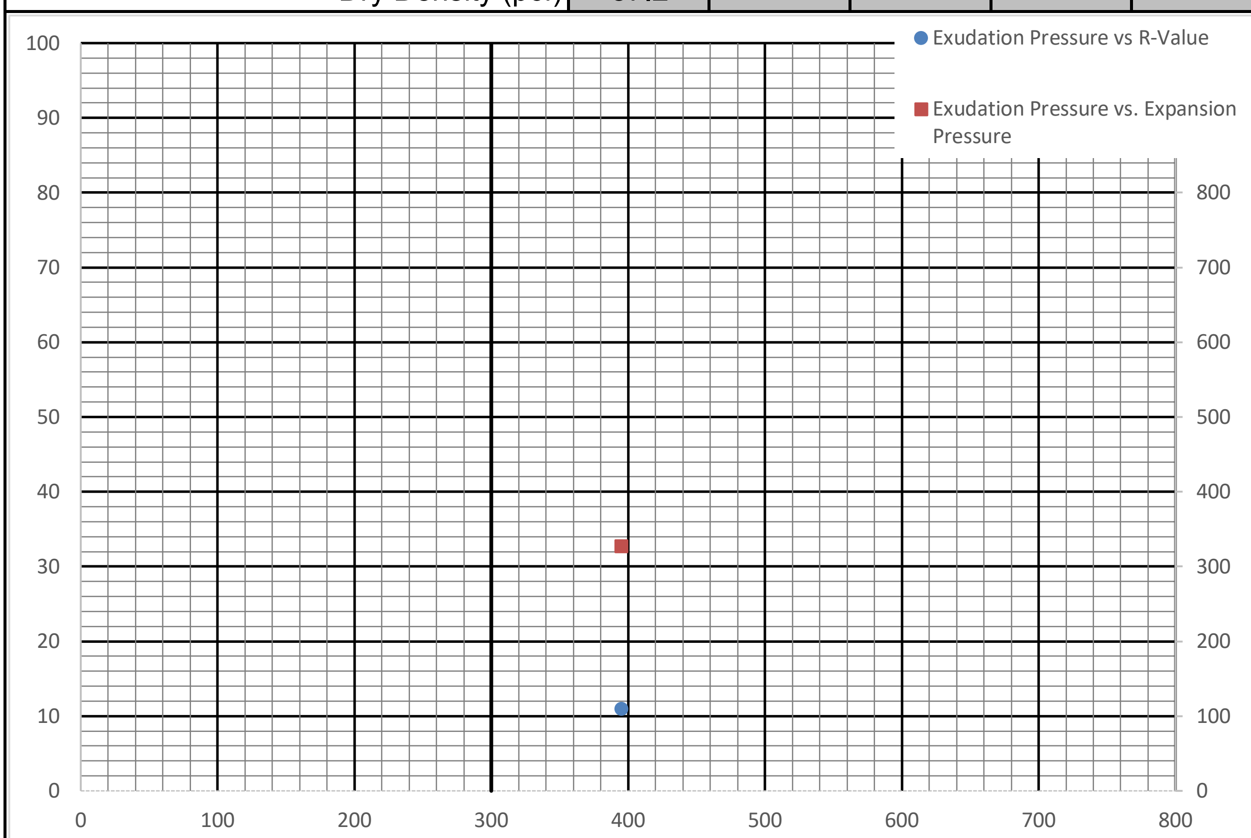
Remarks:

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.:	471-405	Boring:	B-1	Reduced By:	RU
Client:	Cal Engineering & Geology	Sample:	Bulk-1	Checked By:	PJ
Project Number:	0207686	Depth:	0-5	Date:	2/22/2023
Project Name:	2180 Higgins			**R-Value**	**<5**
Soil Description:	Yellowish Brown Sandy CLAY				
Remarks:	Soil extruded from the mold giving a false exudation pressure. Per Caltrans, the R-Value test was terminated and an R-Value of less than 5 was reported.			**Expansion Pressure**	

Specimen Designation	A	B	C	D	E
Compactor Foot Pressure (psi)	50				
Exudation Pressure (psi)	395				
Exudation Load (lbf)	4964				
Height After Compaction (in)	2.47				
Expansion Pressure (psf)	327				
Stabilometer @ 2000	132				
Turns Displacement	4.09				
R-value	11				
Corrected R-Value	11				
Moisture Content (%)	33.1				
Wet Density (pcf)	116.1				
Dry Density (pcf)	87.2				



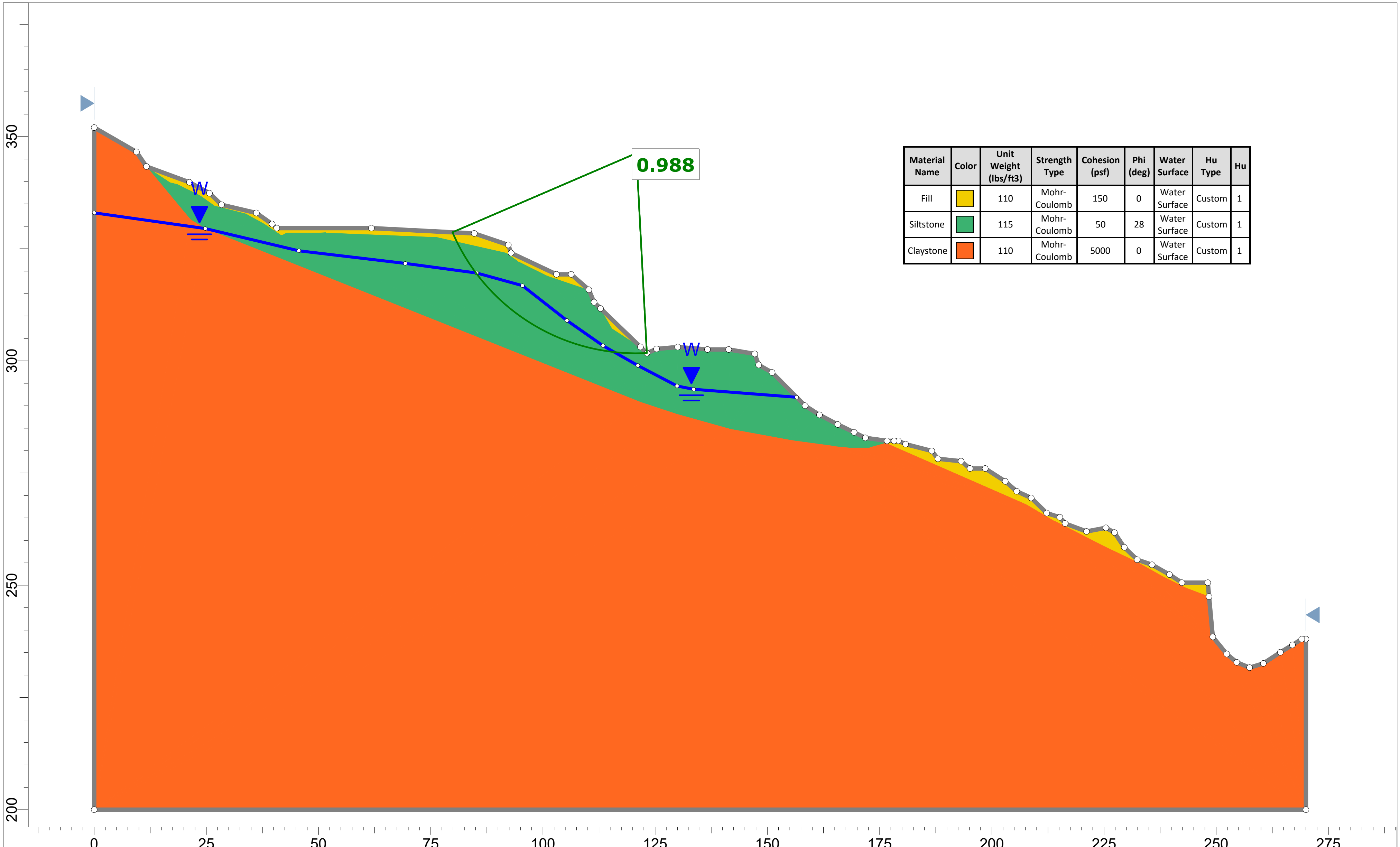
COOPER TESTING LABORATORY

Corrosivity Test Summary

CTL #	471-405	Date:	2/23/2023	Tested By:	PJ	Checked:	PJ
Client:	Cal Engineering & Geology	Project:	2180 Higgins Canyon Road			Proj. No:	0207686
Remarks:							

Sample Location or ID			Resistivity @ 15.5 °C (Ohm-cm)			Chloride	Sulfate		pH	ORP	Moisture	Soil Visual Description
Boring	Sample, No.	Depth, ft.	As Rec.	Minimum	Saturated	mg/kg	mg/kg	%		(Redox)	At Test	
						Dry Wt.	Dry Wt.	Dry Wt.		mv	%	
			ASTM G57	Cal 643	ASTM G57	Cal 422-mod.	Cal 417-mod.	Cal 417-mod.	Cal 643	SM 2580B	ASTM D2216	
B-1	Bulk-1	0-5	-	775	-	85	390	0.0390	6.5	-	5.0	Yellowish Brown Sandy CLAY

Appendix D. Slope Stability Analysis

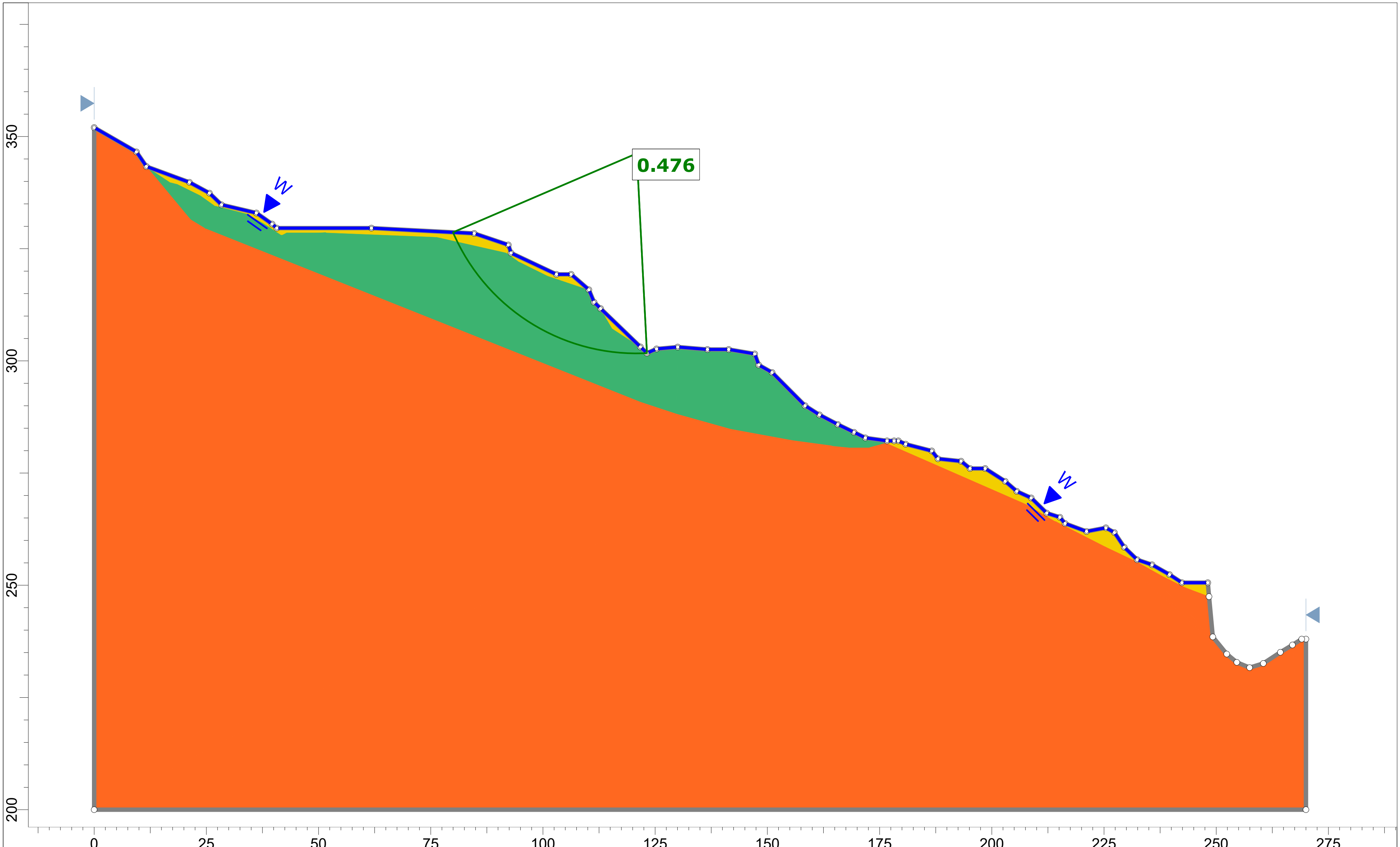


Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface	Hu Type	Hu
Fill		110	Mohr-Coulomb	150	0	Water Surface	Custom	1
Siltstone		115	Mohr-Coulomb	50	28	Water Surface	Custom	1
Claystone		110	Mohr-Coulomb	5000	0	Water Surface	Custom	1

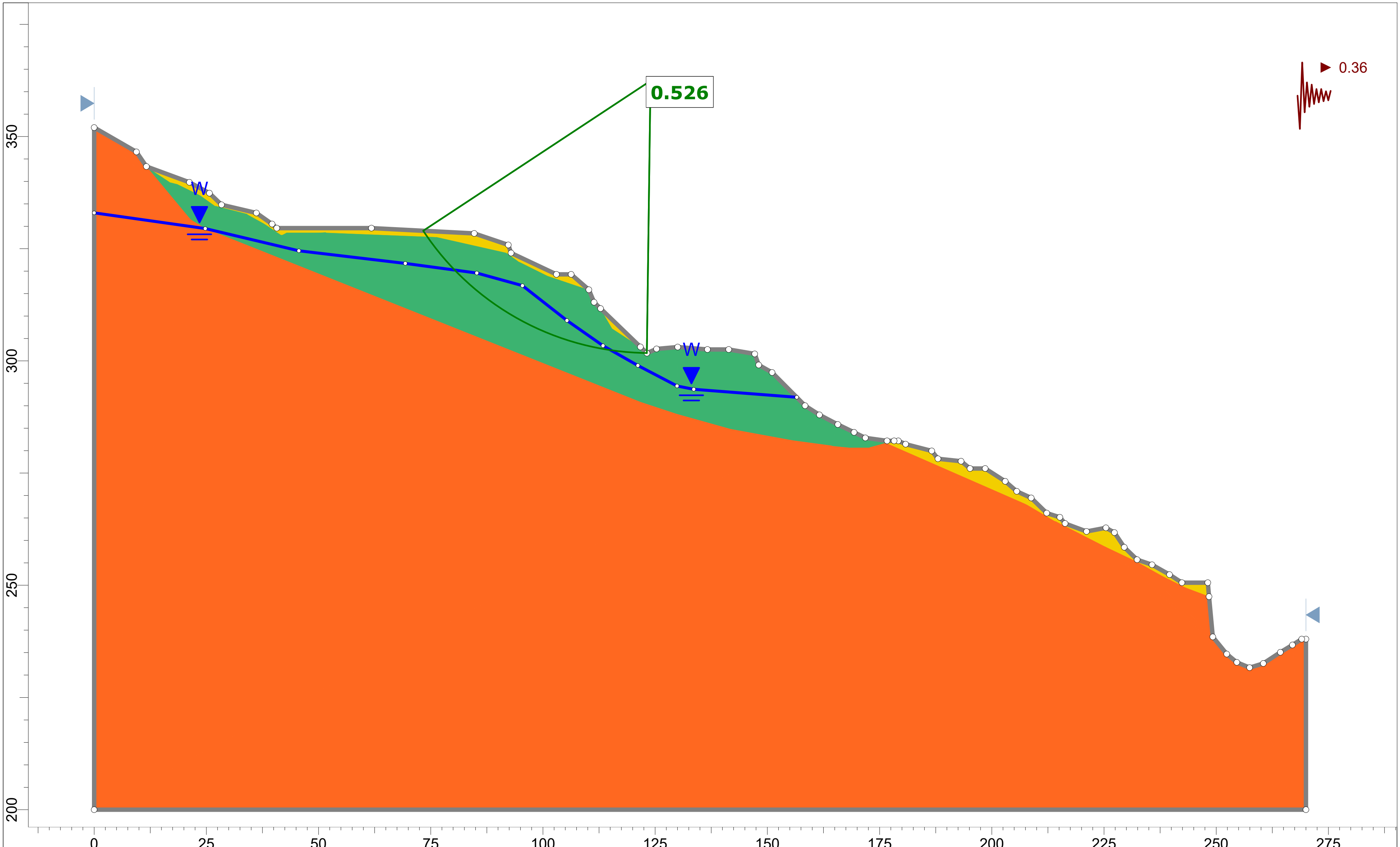
rocscience

SLIDEINTERPRET 9.020

Project	SLIDE - An Interactive Slope Stability Program		
Group	Existing	*Scenario*	Master Scenario
Drawn By		*Company*	
Date	3/17/2023, 4:23:27 PM	*File Name*	A-A'-InfStr-RetainingWall.slmd



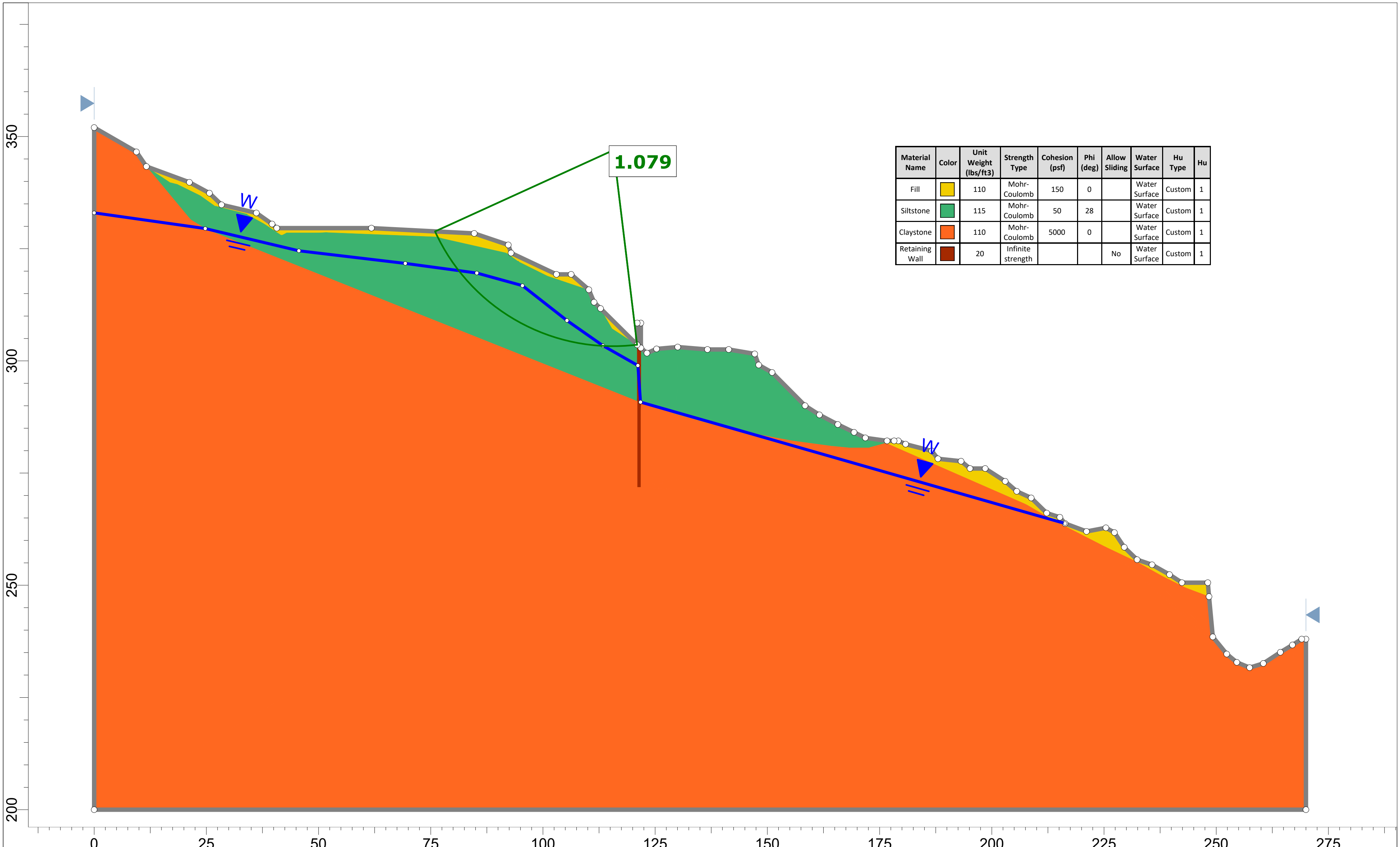
rocscience	*Project* SLIDE - An Interactive Slope Stability Program		
	Group Existing	*Scenario* Existing - Fully Saturated	
	Drawn By	*Company*	
SLIDEINTERPRET 9.020	*Date* 3/17/2023, 4:23:27 PM	*File Name* A-A'-InfStr-RetainingWall.slmd	



rocscience

Project	SLIDE - An Interactive Slope Stability Program		
Group	Existing	*Scenario*	Existing - Seismic
Drawn By		*Company*	
Date	3/17/2023, 4:23:27 PM	*File Name*	A-A'-InfStr-RetainingWall.slmd

SLIDEINTERPRET 9.020

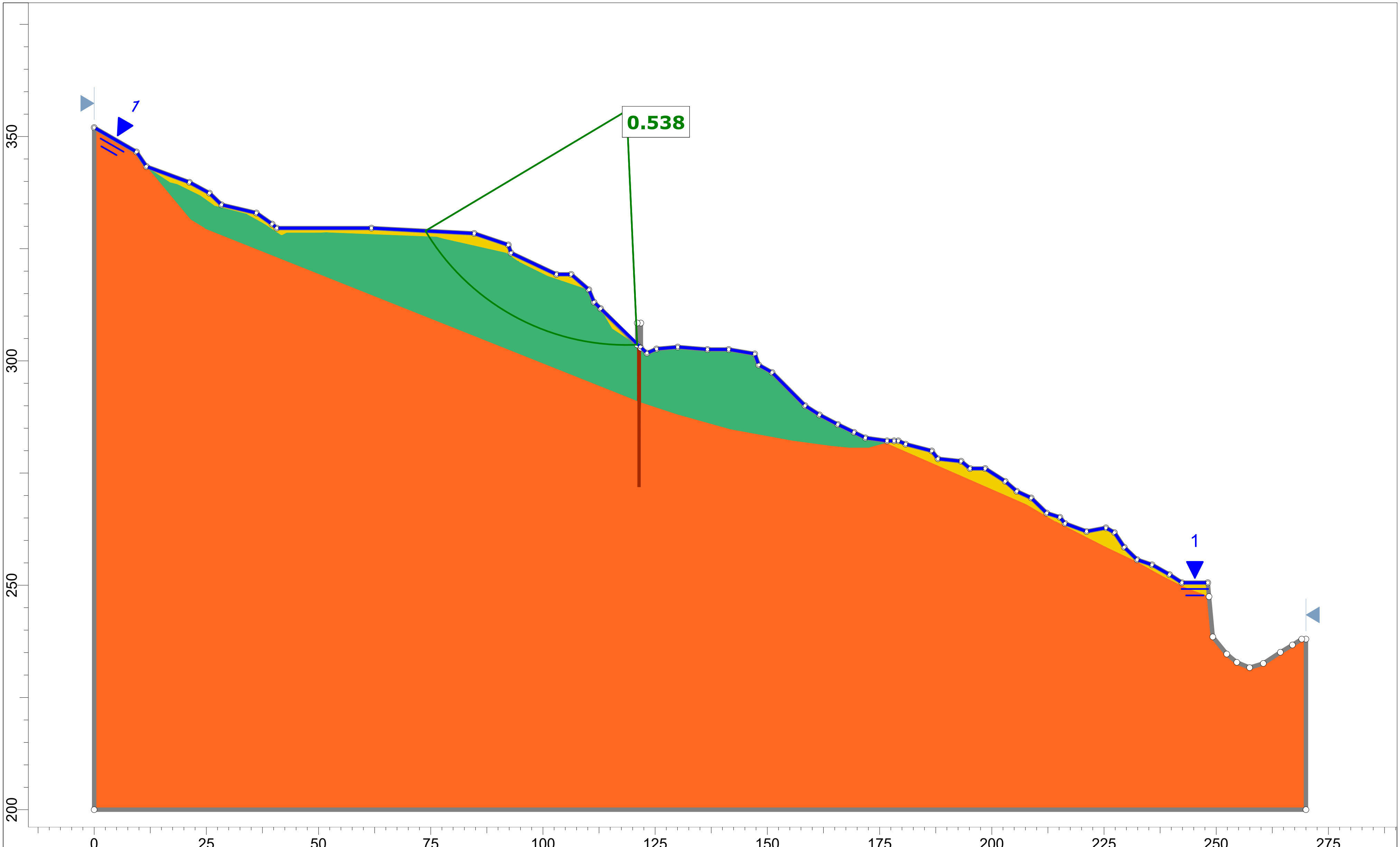


Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Allow Sliding	Water Surface	Hu Type	Hu
Fill		110	Mohr-Coulomb	150	0		Water Surface	Custom	1
Siltstone		115	Mohr-Coulomb	50	28		Water Surface	Custom	1
Claystone		110	Mohr-Coulomb	5000	0		Water Surface	Custom	1
Retaining Wall		20	Infinite strength			No	Water Surface	Custom	1

rocscience

SLIDEINTERPRET 9.020

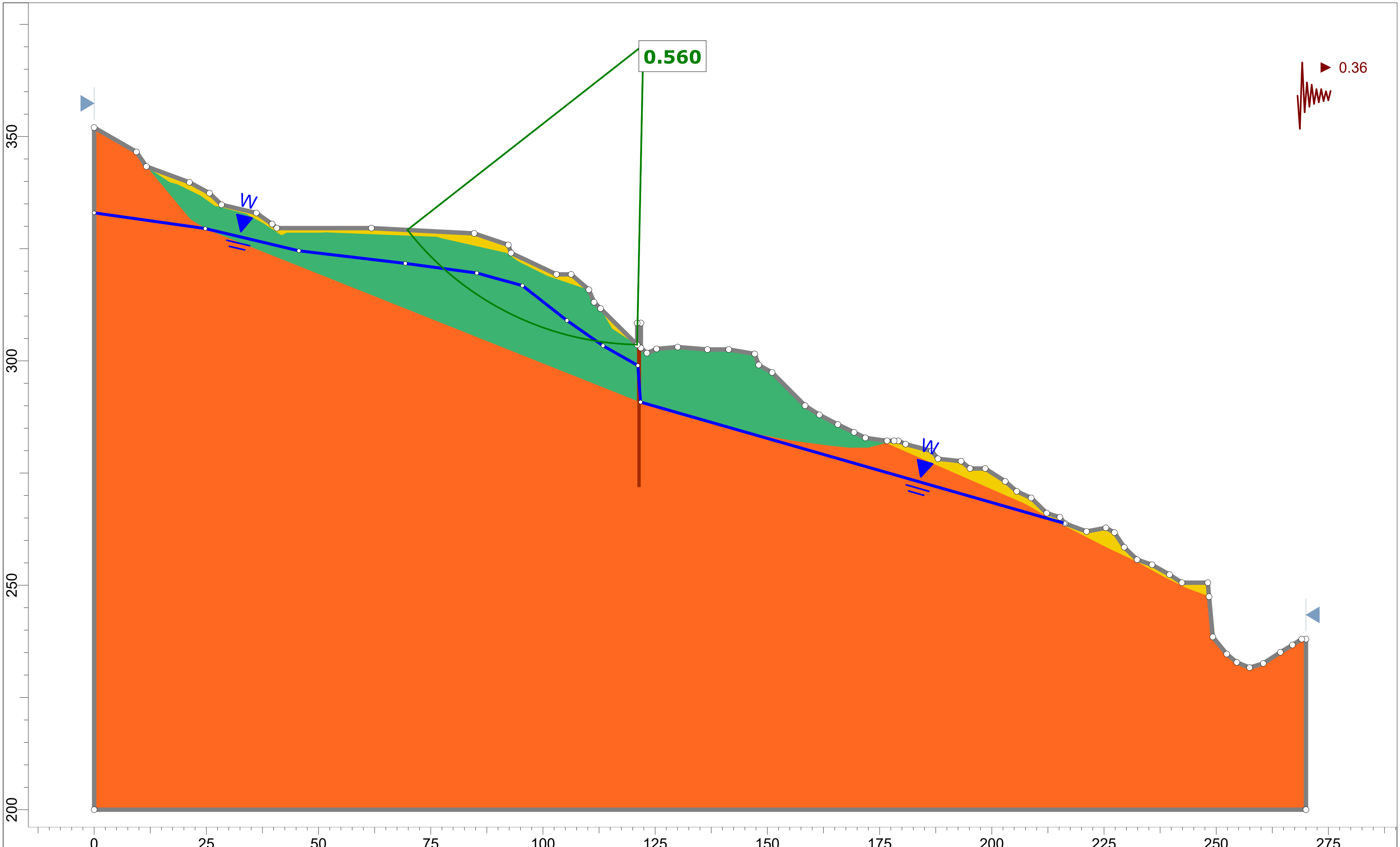
Project	SLIDE - An Interactive Slope Stability Program
Group	Temp Wall
Scenario	Master Scenario
Drawn By	
Company	
Date	3/17/2023, 4:23:27 PM
File Name	A-A'-InfStr-RetainingWall.slmd



rocscience

SLIDEINTERPRET 9.020

Project	SLIDE - An Interactive Slope Stability Program
Group: Temp Wall	Scenario: Temp Wall - Fully Saturated
Drawn By	Company
Date: 3/17/2023, 4:23:27 PM	File Name: A-A'-InfStr-RetainingWall.slmd



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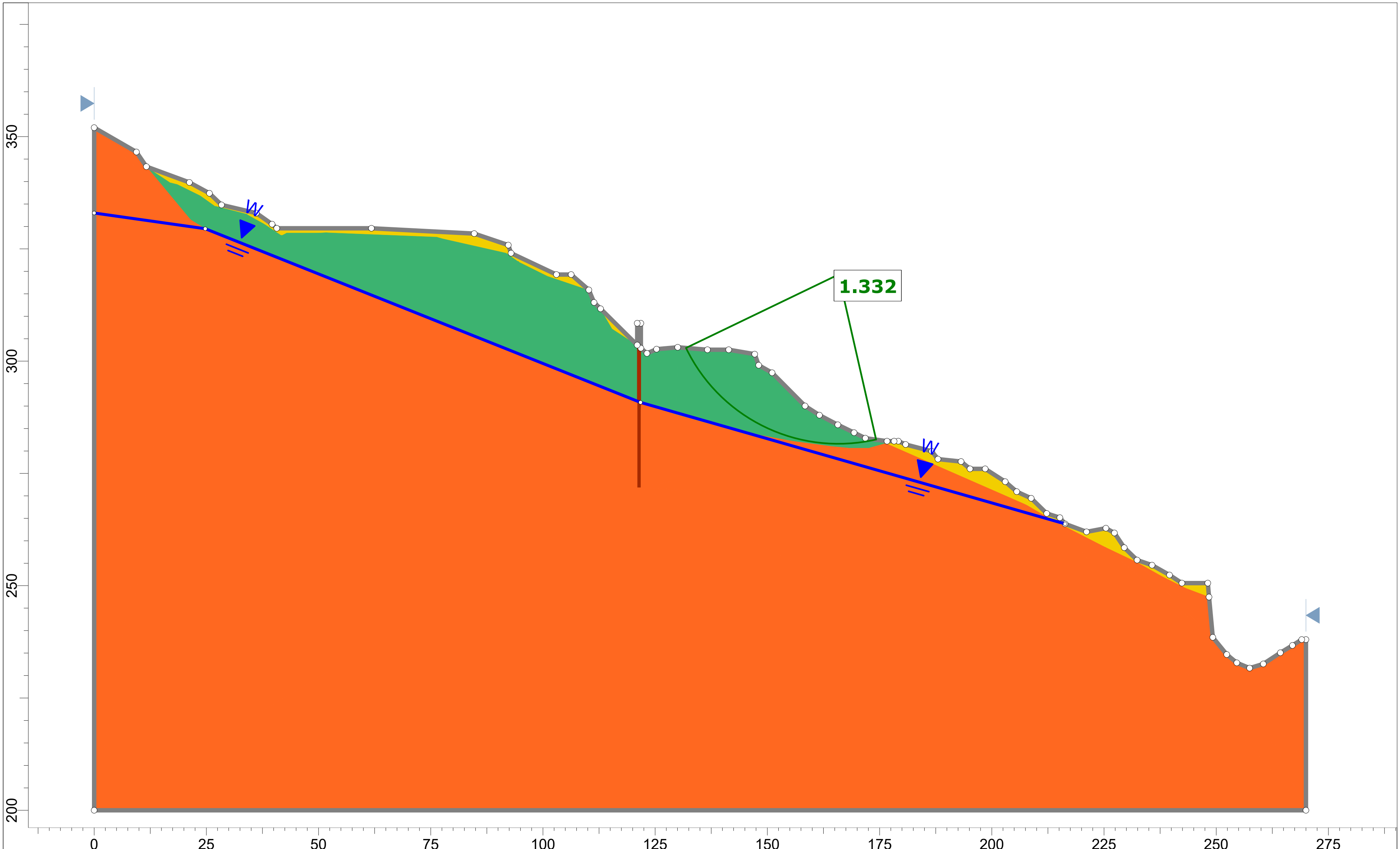
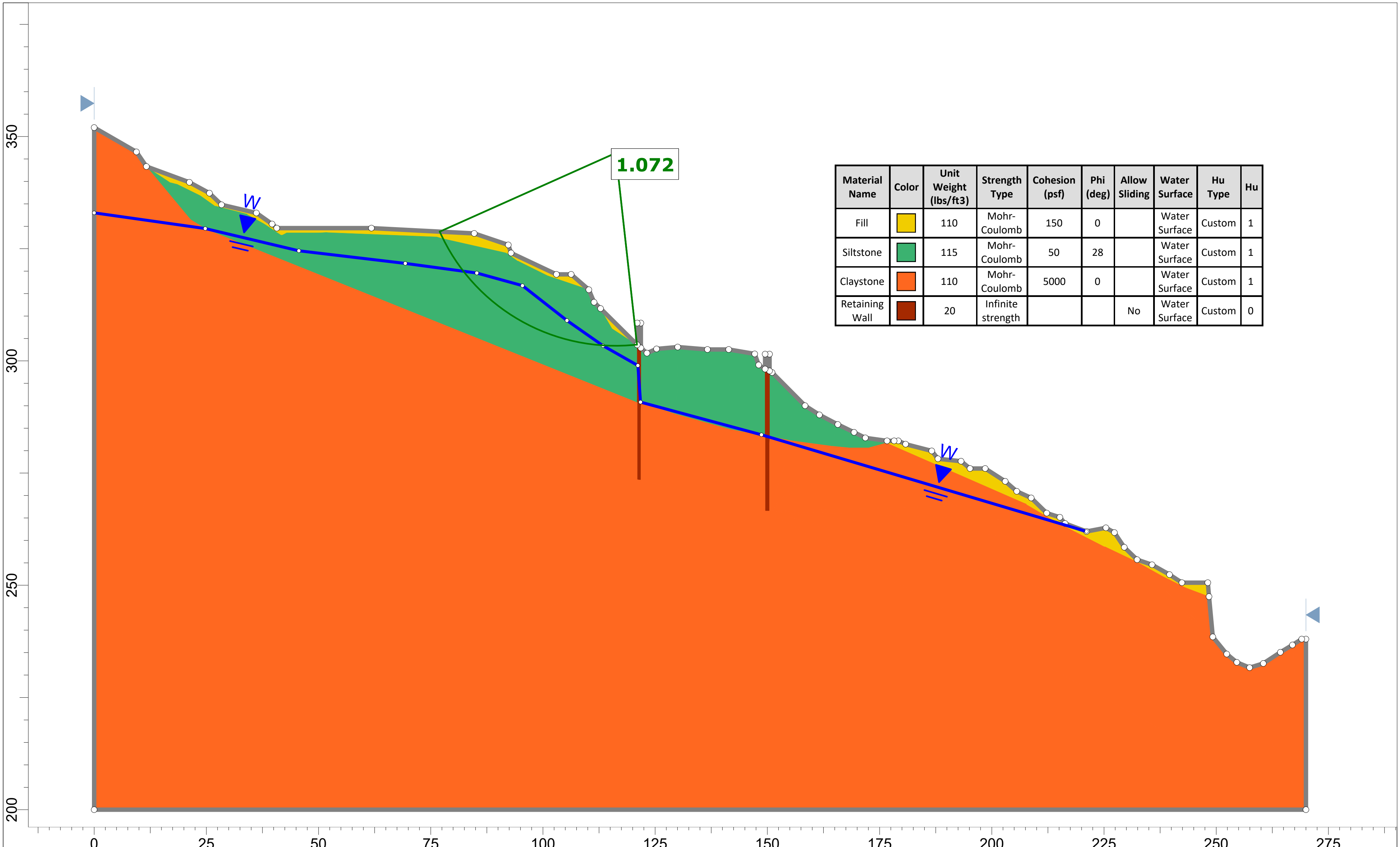
SLIDEINTERPRET 9.020

Project	SLIDE - An Interactive Slope Stability Program		
Group	Temp Wall	*Scenario*	Temp Wall - Seismic
Drawn By		*Company*	
Date	3/17/2023, 4:23:27 PM	*File Name*	A-A'-InfStr-RetainingWall.slmd

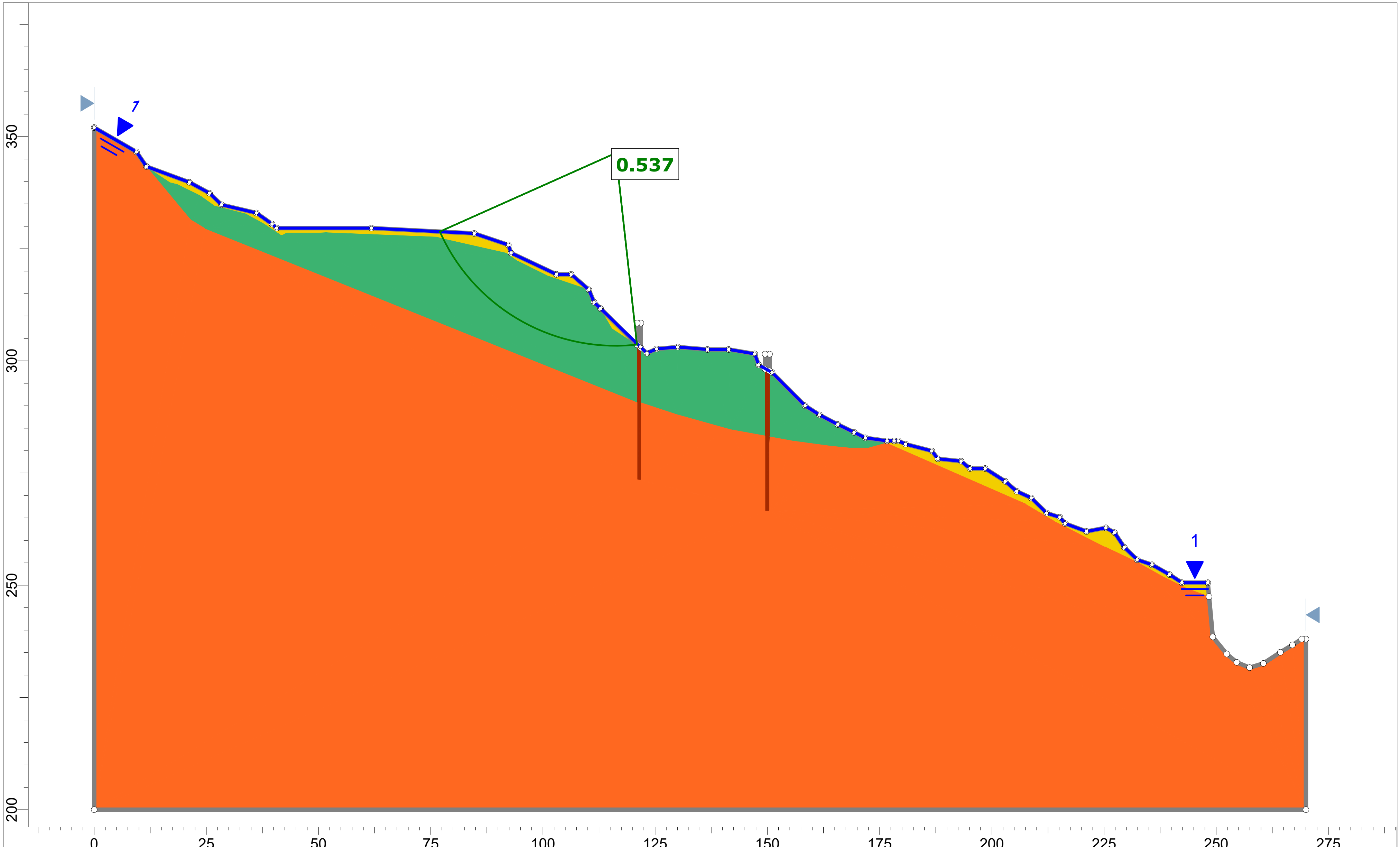


	Project	SLIDE - An Interactive Slope Stability Program	
rocscience	Group: Temp Wall	Scenario: Temp Wall - Low GW	
	Drawn By:	Company:	
SLIDEINTERPRET 9.020	Date: 3/17/2023, 4:23:27 PM	File Name: A-A'-InfStr-RetainingWall.slmd	

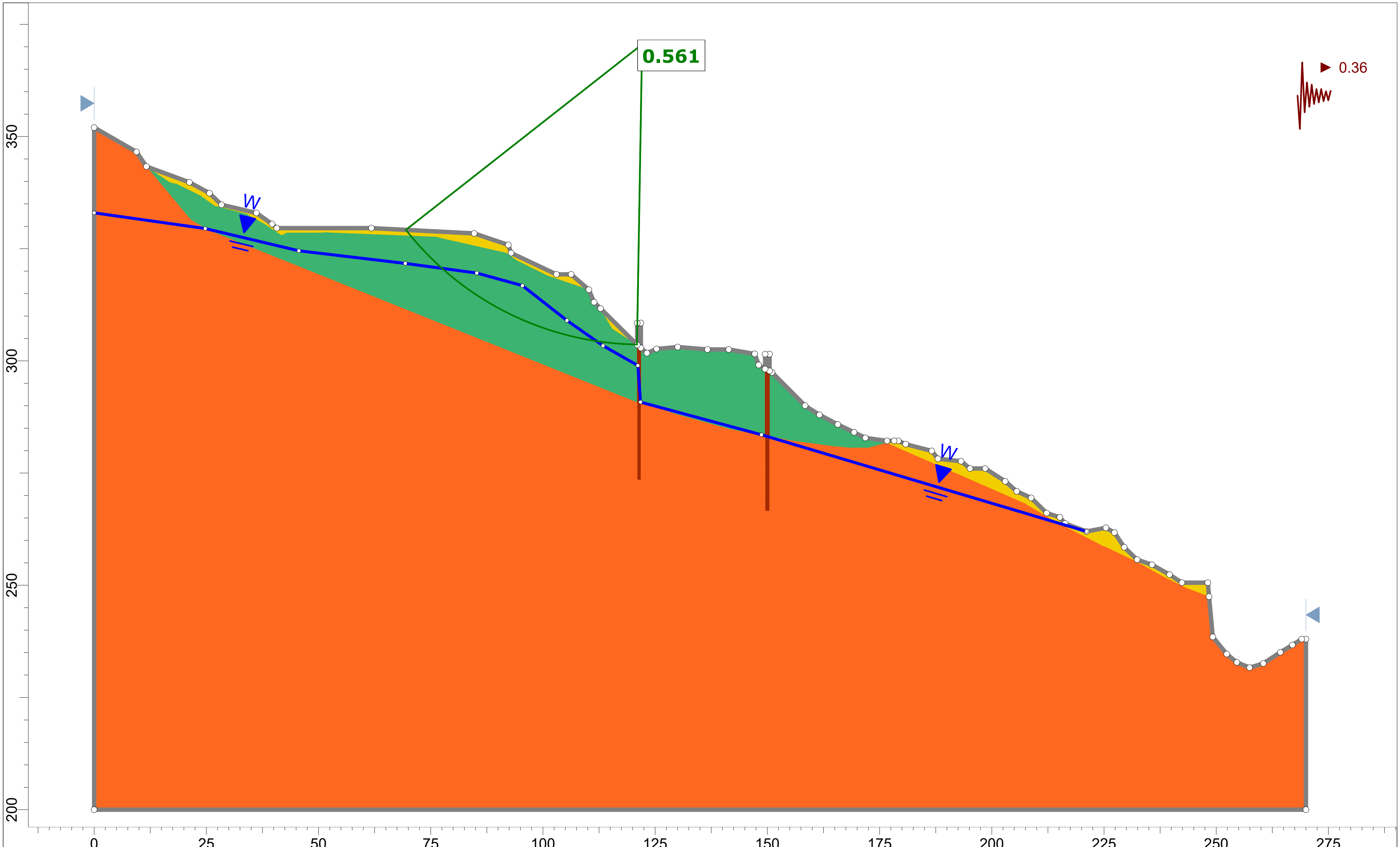


Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Allow Sliding	Water Surface	Hu Type	Hu
Fill		110	Mohr-Coulomb	150	0		Water Surface	Custom	1
Siltstone		115	Mohr-Coulomb	50	28		Water Surface	Custom	1
Claystone		110	Mohr-Coulomb	5000	0		Water Surface	Custom	1
Retaining Wall		20	Infinite strength			No	Water Surface	Custom	0

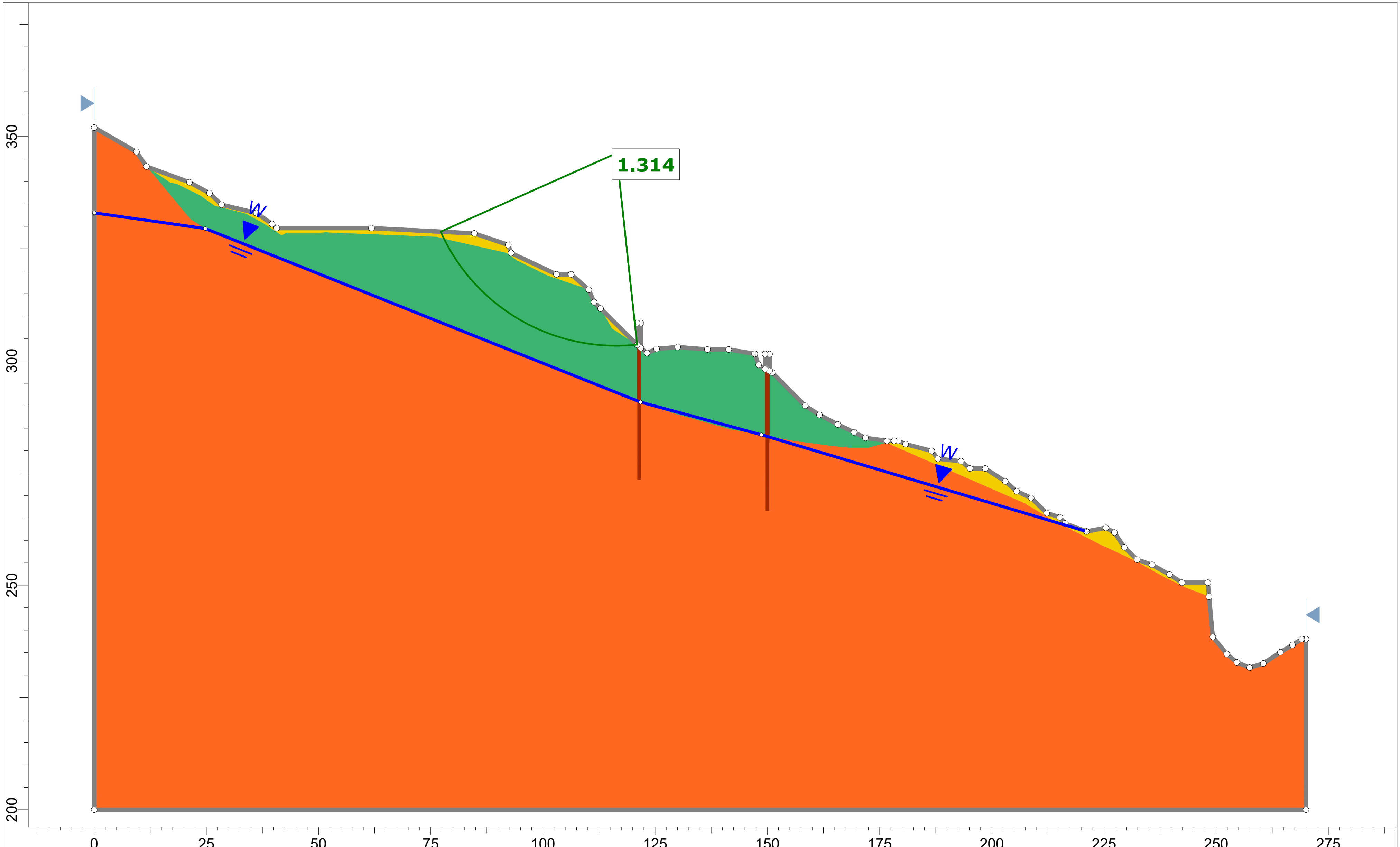
rocscience	*Project* SLIDE - An Interactive Slope Stability Program		
	Group Two Walls	*Scenario* Master Scenario	
	Drawn By	*Company*	
SLIDEINTERPRET 9.020	*Date* 3/17/2023, 4:23:27 PM	*File Name* A-A'-InfStr-RetainingWall.slmd	



rocscience	*Project* SLIDE - An Interactive Slope Stability Program		
	Group Two Walls	*Scenario* Two Walls - Fully Saturated	
	Drawn By	*Company*	
SLIDEINTERPRET 9.020	*Date* 3/17/2023, 4:23:27 PM	*File Name* A-A'-InfStr-RetainingWall.slmd	



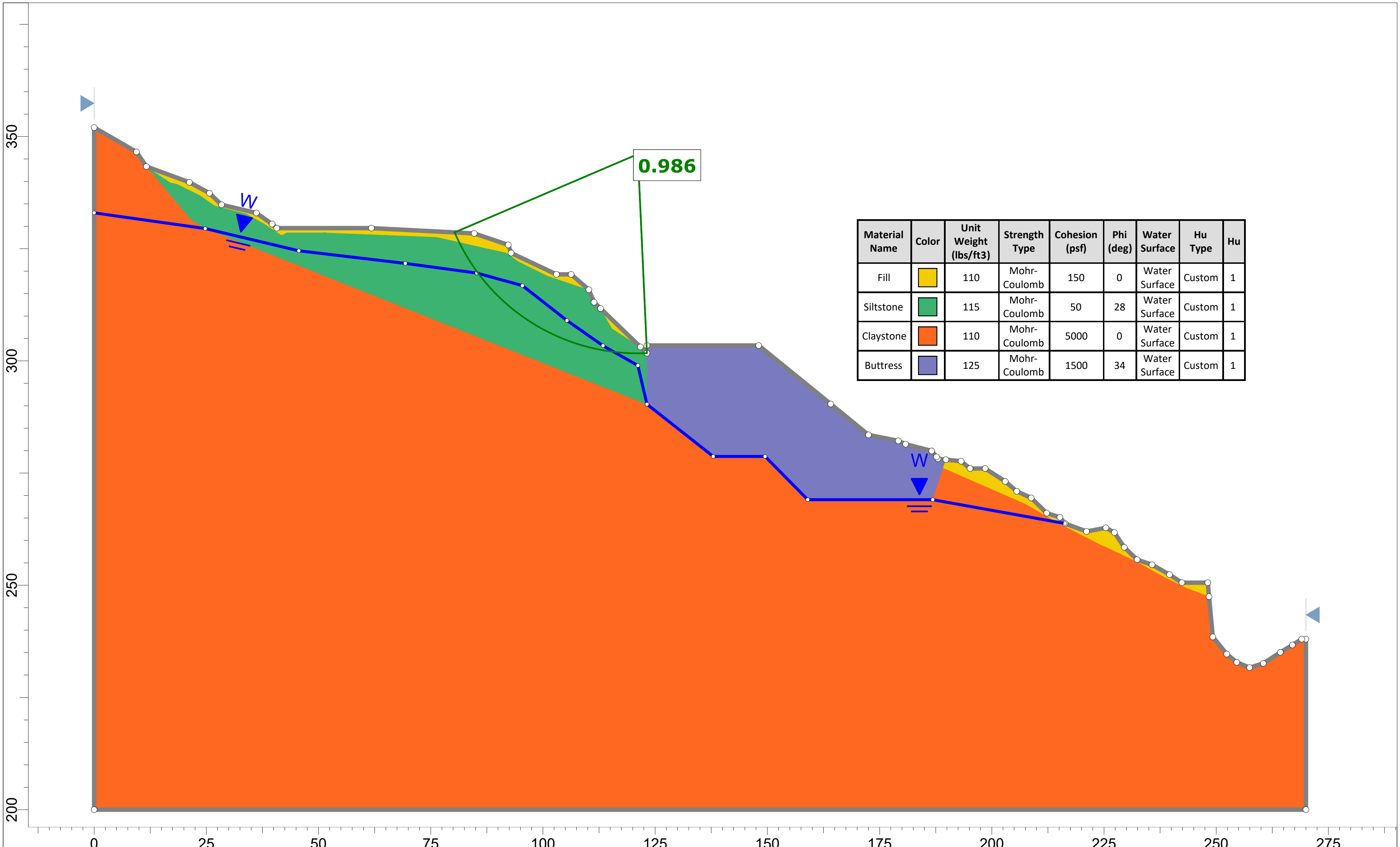
rocscience	*Project* SLIDE - An Interactive Slope Stability Program		
	Group Two Walls	*Scenario* Two Walls - Seismic	
	Drawn By	*Company*	
SLIDEINTERPRET 9.020	*Date* 3/17/2023, 4:23:27 PM	*File Name* A-A'-InfStr-RetainingWall.slmd	



rocscience

SLIDEINTERPRET 9.020

Project	SLIDE - An Interactive Slope Stability Program		
Group	Two Walls	*Scenario*	Two Walls - Low GW
Drawn By		*Company*	
Date	3/17/2023, 4:23:27 PM	*File Name*	A-A'-InfStr-RetainingWall.slmd

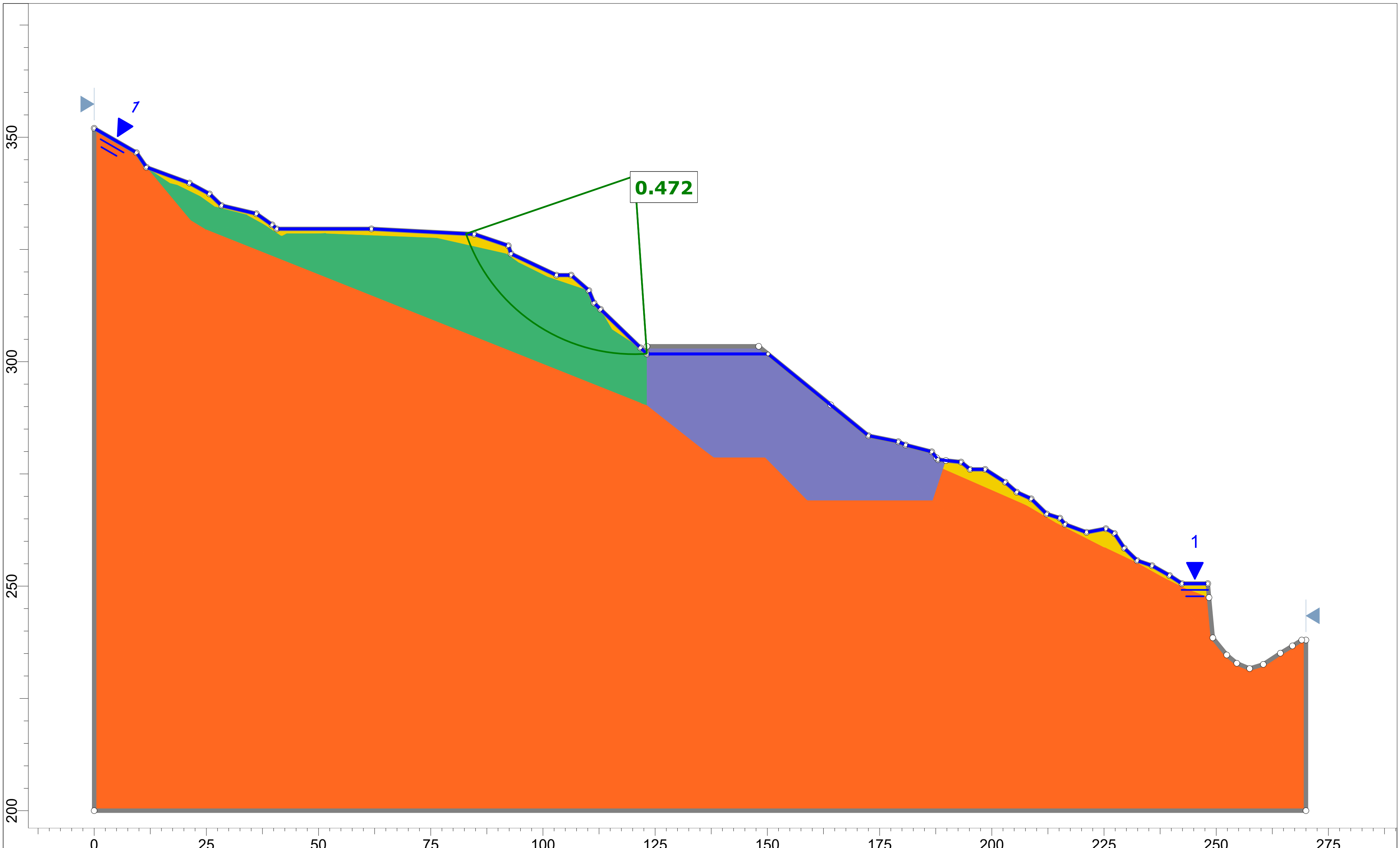


Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface	Hu Type	Hu
Fill		110	Mohr-Coulomb	150	0	Water Surface	Custom	1
Siltstone		115	Mohr-Coulomb	50	28	Water Surface	Custom	1
Claystone		110	Mohr-Coulomb	5000	0	Water Surface	Custom	1
Buttress		125	Mohr-Coulomb	1500	34	Water Surface	Custom	1

rocscience

SLIDEINTERPRET 9.020

Project	SLIDE - An Interactive Slope Stability Program		
Group	Buttress Deep	*Scenario*	Master Scenario
Drawn By		*Company*	
Date	3/17/2023, 4:23:27 PM	*File Name*	A-A'-InfStr-RetainingWall.slmd



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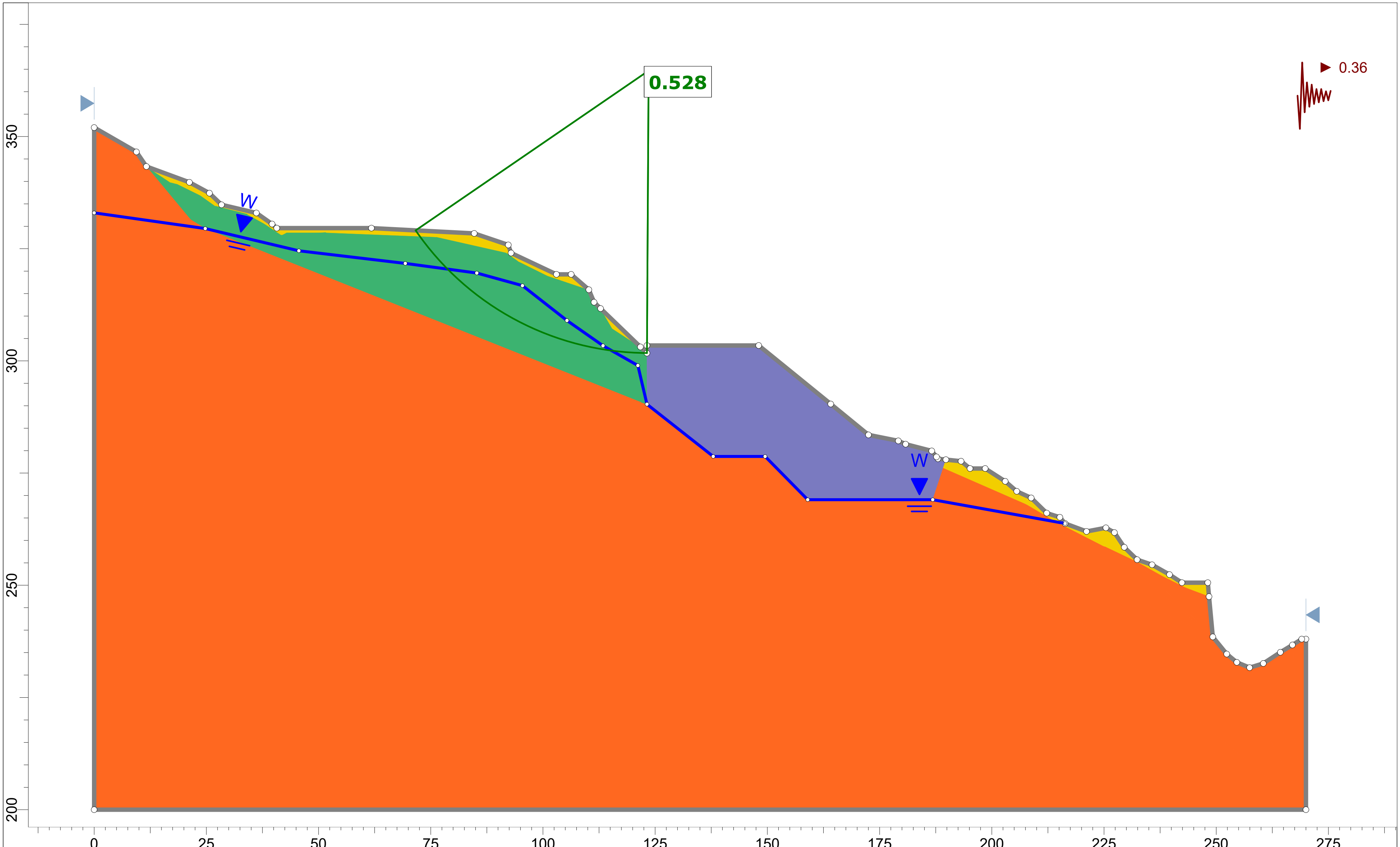
SLIDEINTERPRET 9.020

Project	SLIDE - An Interactive Slope Stability Program		
Group	Buttress Deep	*Scenario*	Buttress Deep - Fully Saturated
Drawn By		*Company*	
Date	3/17/2023, 4:23:27 PM	*File Name*	A-A'-InfStr-RetainingWall.slmd



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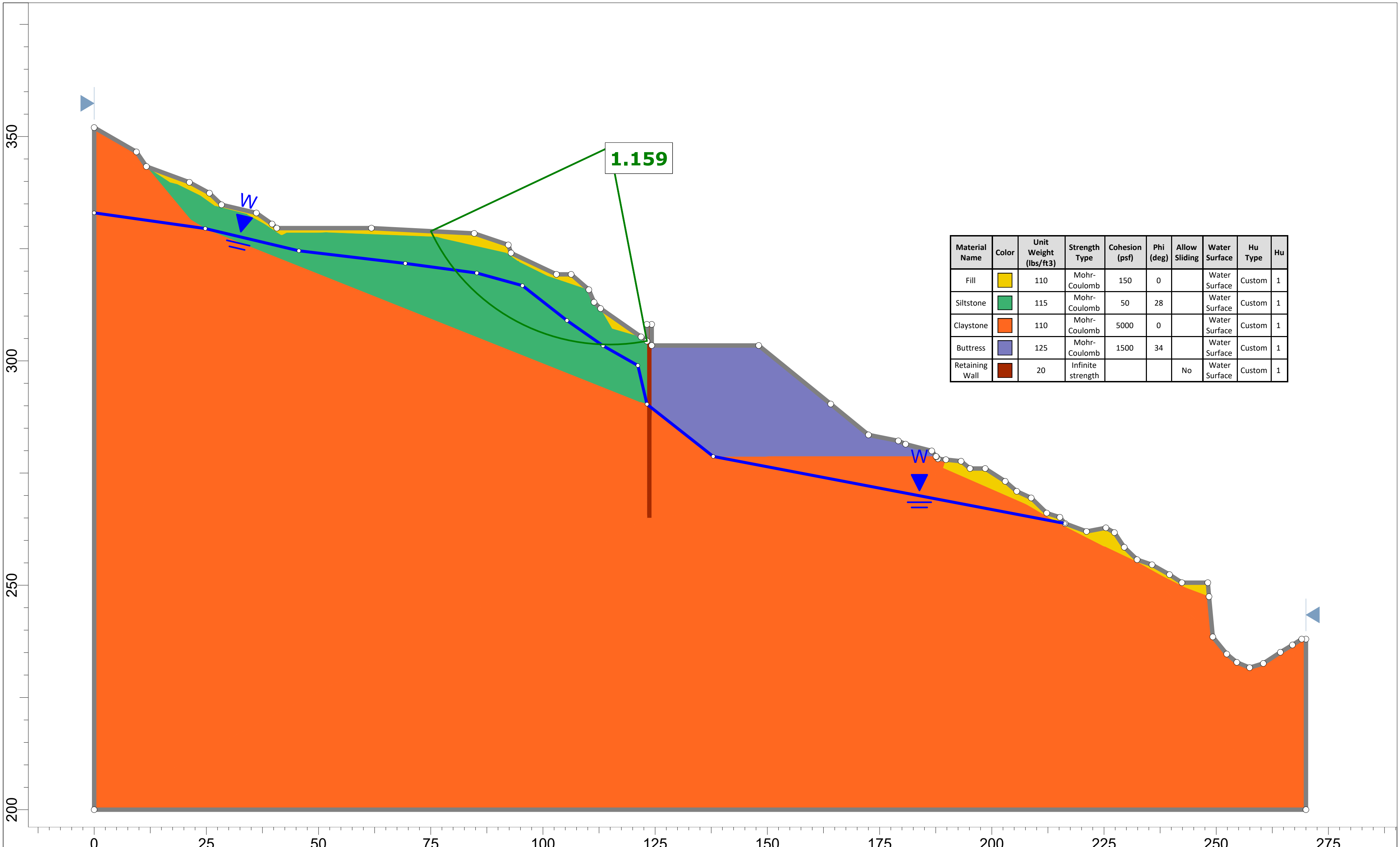
SLIDEINTERPRET 9.020

Project	SLIDE - An Interactive Slope Stability Program		
Group	Buttress Deep	Scenario	Buttress Deep - Seismic
Drawn By		Company	
Date	3/17/2023, 4:23:27 PM	File Name	A-A'-InfStr-RetainingWall.slmd



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Siltstone		115	Mohr-Coulomb	50	28		Water Surface	Custom	1
Claystone		110	Mohr-Coulomb	5000	0		Water Surface	Custom	1
Buttress		125	Mohr-Coulomb	1500	34		Water Surface	Custom	1
Retaining Wall		20	Infinite strength			No	Water Surface	Custom	1

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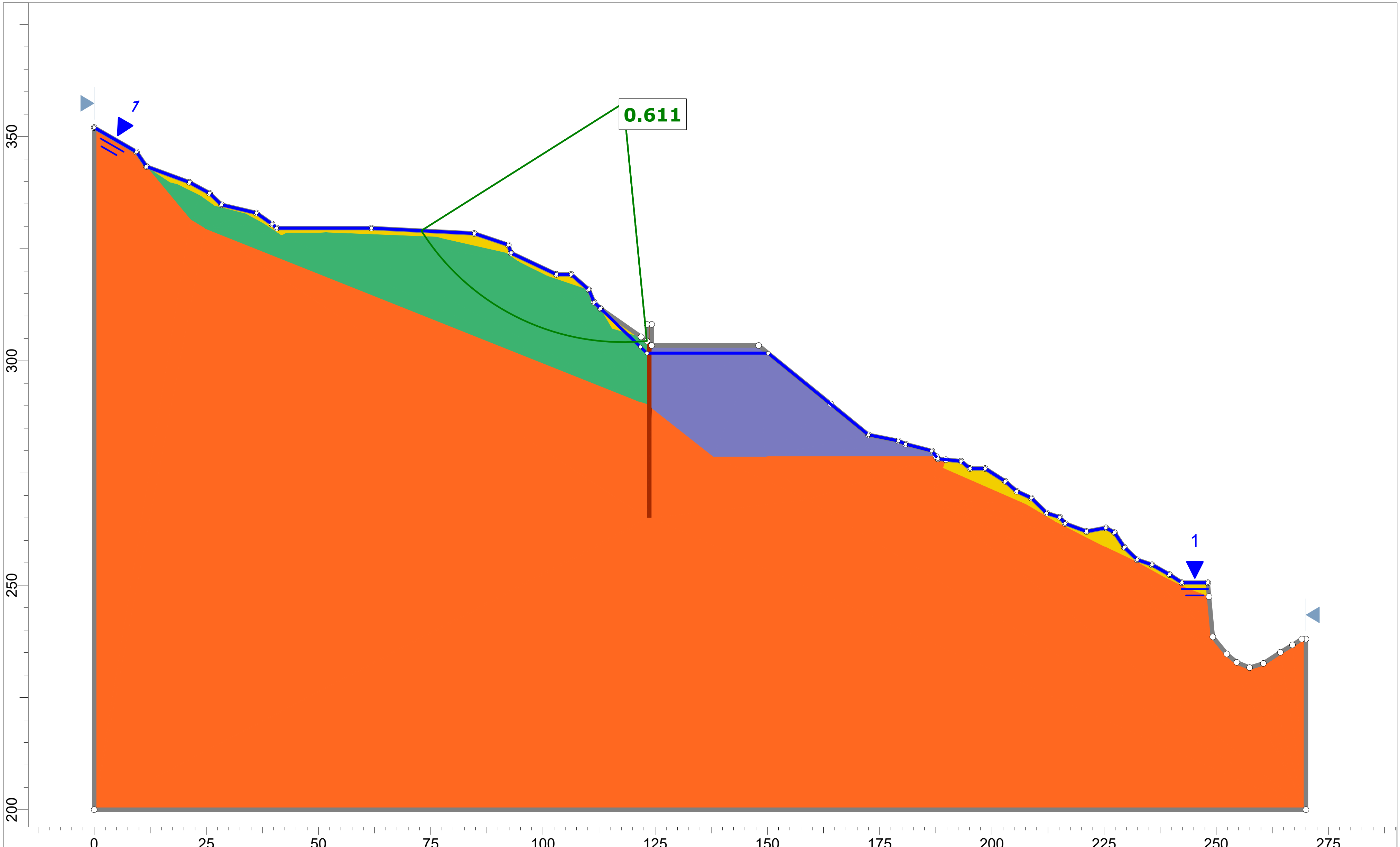
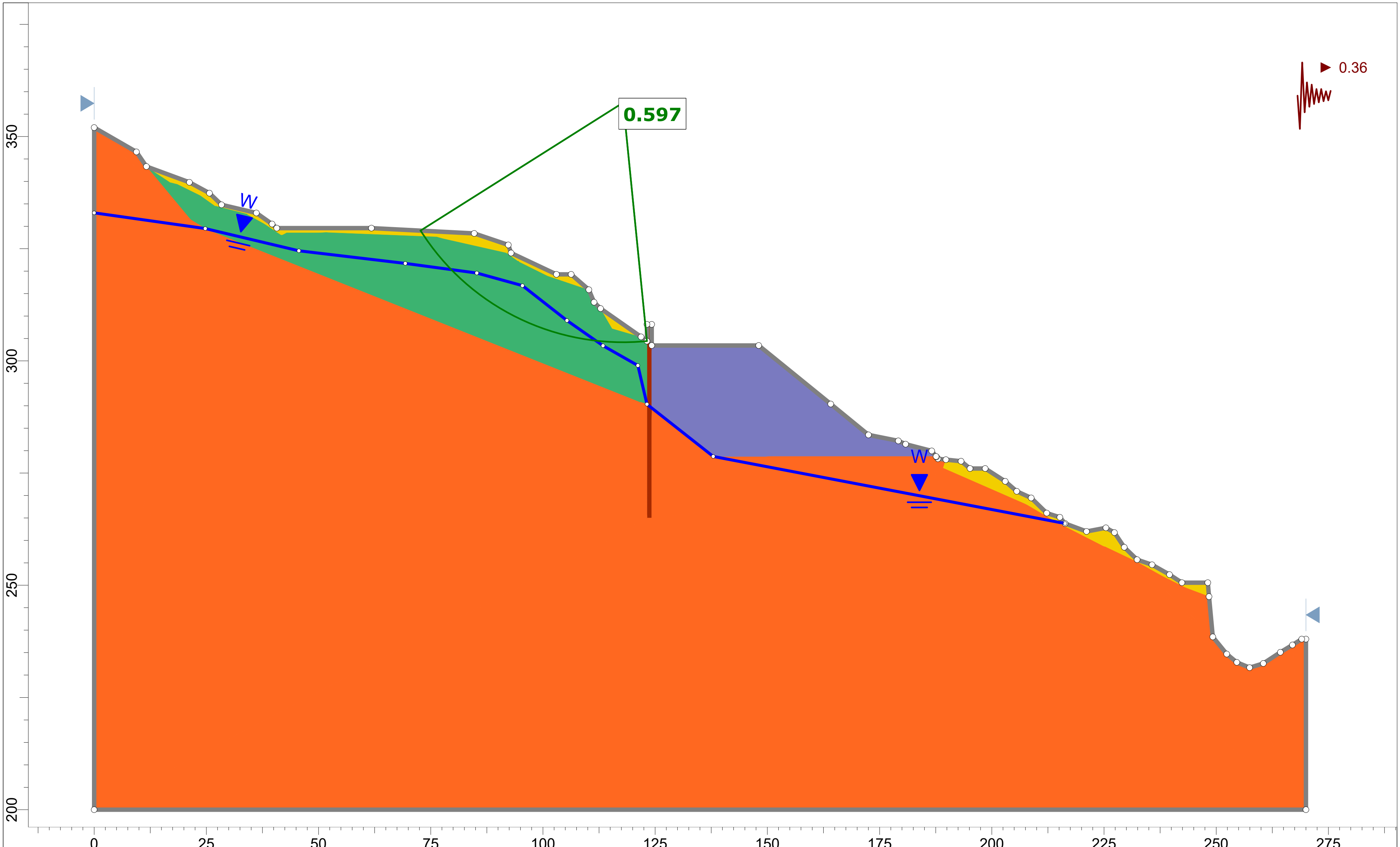
SLIDEINTERPRET 9.020

Project	SLIDE - An Interactive Slope Stability Program		
Group	Buttress Shallow	*Scenario*	Master Scenario
Drawn By		*Company*	
Date	3/17/2023, 4:23:27 PM	*File Name*	A-A'-InfStr-RetainingWall.slmd



Project	SLIDE - An Interactive Slope Stability Program		
Group	Buttress Shallow	Scenario	Buttress Shallow - Fully Saturated
Drawn By		Company	
Date	3/17/2023, 4:23:27 PM	File Name	A-A'-InfStr-RetainingWall.slmd

SLIDEINTERPRET 9.020



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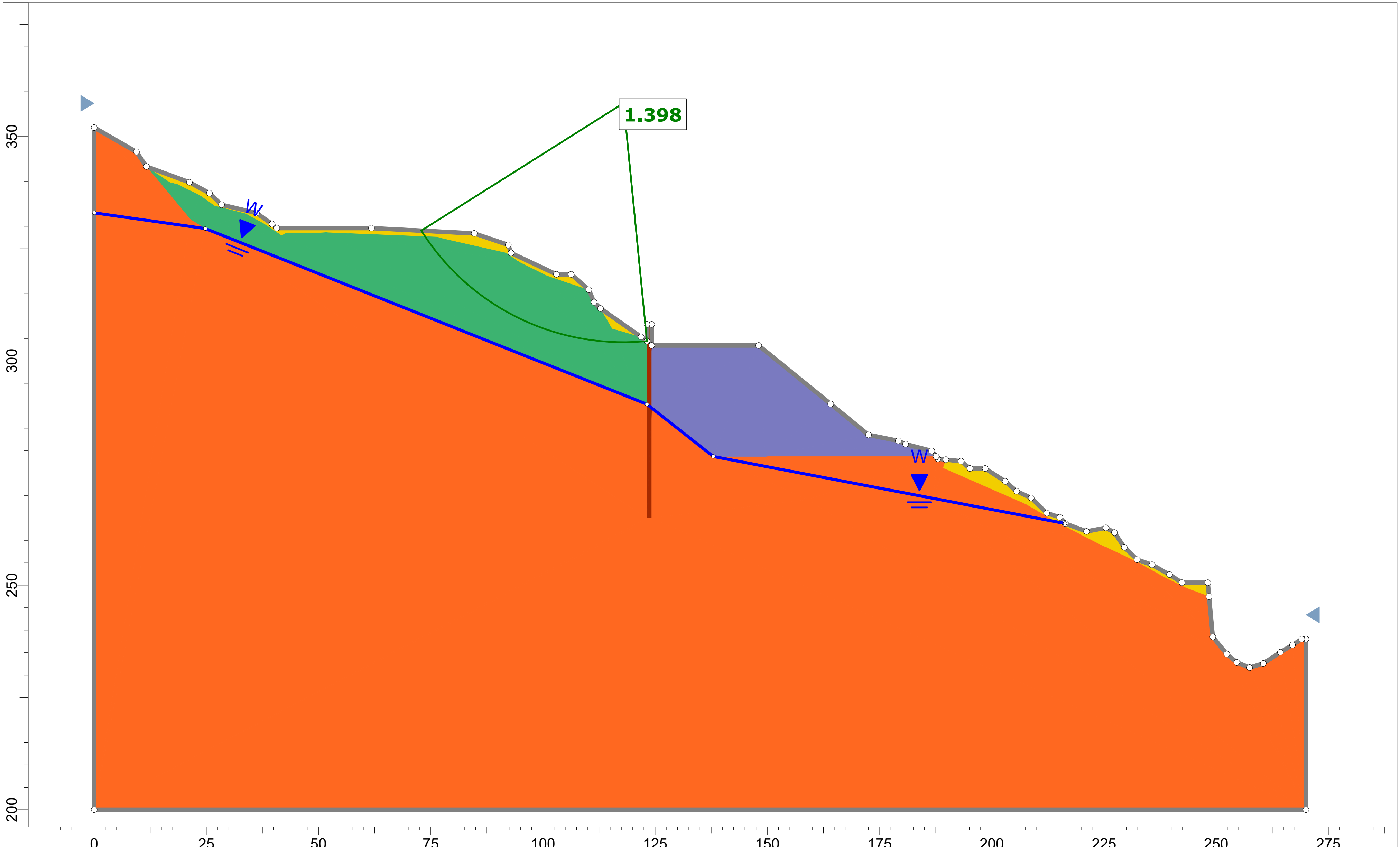
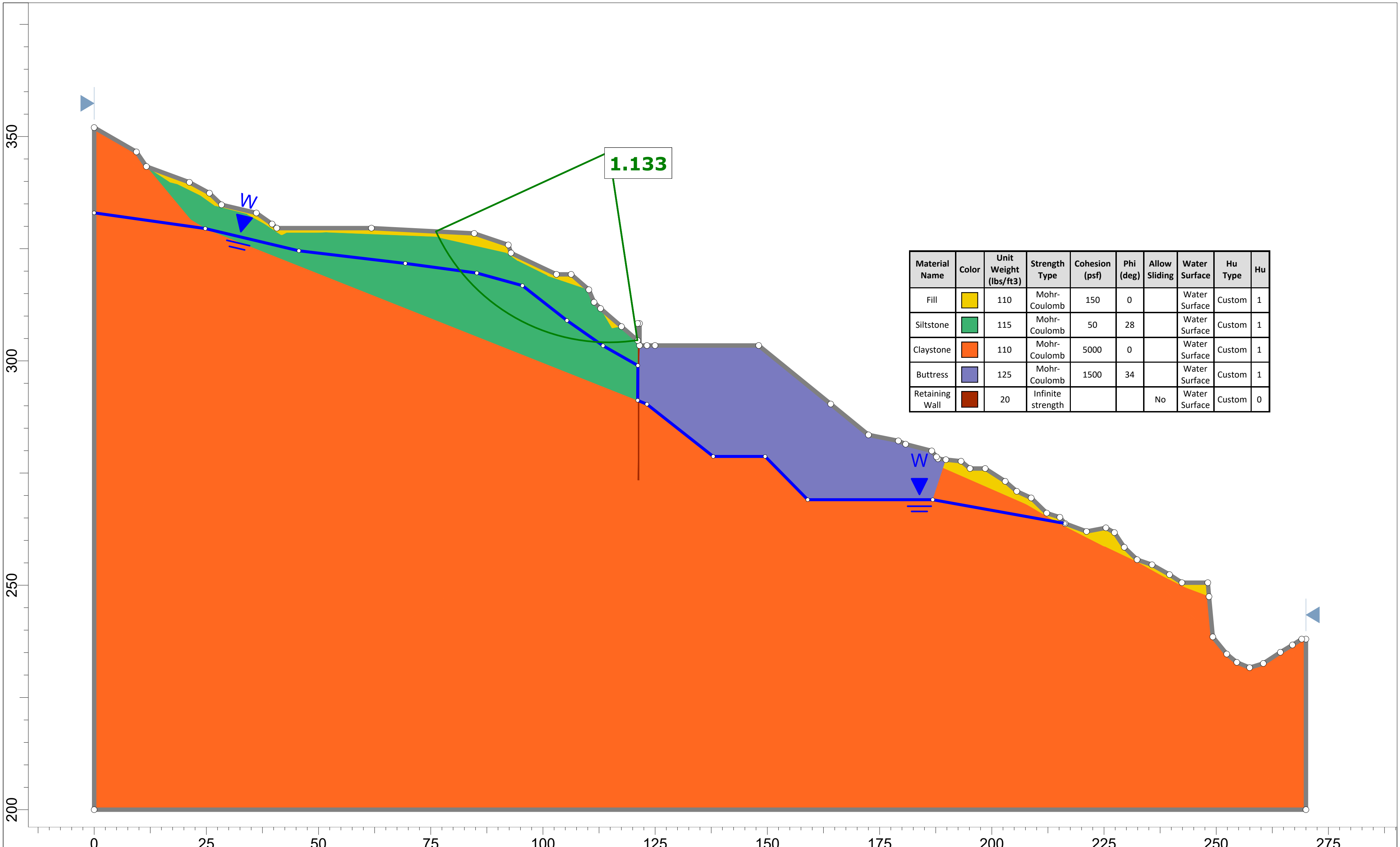
Project	SLIDE - An Interactive Slope Stability Program		
Group	Buttress Shallow	Scenario	Buttress Shallow - Seismic
Drawn By		Company	
Date	3/17/2023, 4:23:27 PM	File Name	A-A'-InfStr-RetainingWall.slmd

SLIDEINTERPRET 9.020



rocscience	Project	SLIDE - An Interactive Slope Stability Program	
	Group: Buttress Shallow	Scenario: Buttress Shallow - Low GW	
	Drawn By	Company	
SLIDEINTERPRET 9.020	Date: 3/17/2023, 4:23:27 PM	File Name: A-A'-InfStr-RetainingWall.slmd	



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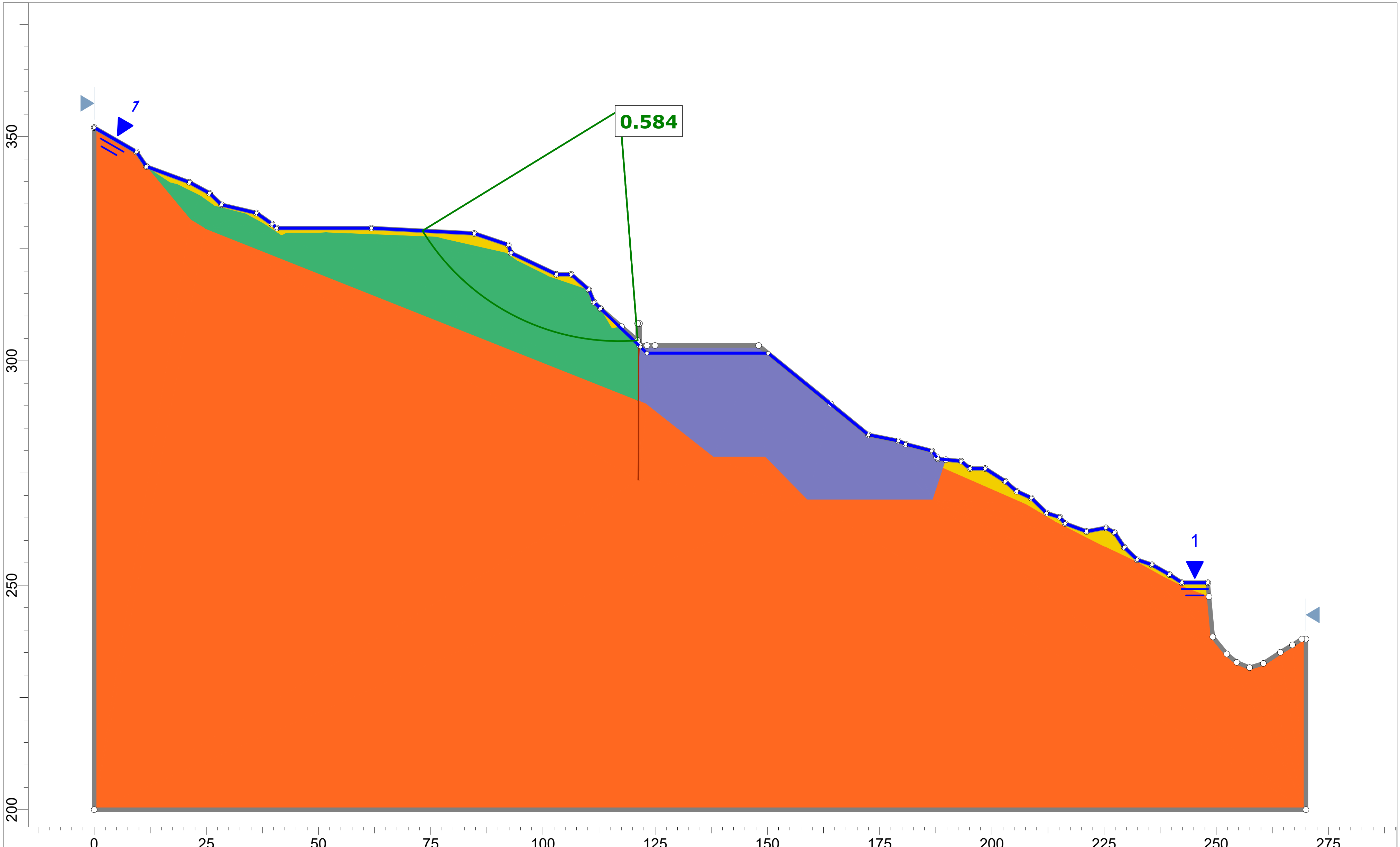
Siltstone		115	Mohr-Coulomb	50	28		Water Surface	Custom	1
Claystone		110	Mohr-Coulomb	5000	0		Water Surface	Custom	1
Buttress		125	Mohr-Coulomb	1500	34		Water Surface	Custom	1
Retaining Wall		20	Infinite strength			No	Water Surface	Custom	0

Project	SLIDE - An Interactive Slope Stability Program		
Group	Buttress w/ Wall	*Scenario*	Master Scenario
Drawn By		*Company*	
Date	3/17/2023, 4:23:27 PM	*File Name*	A-A'-InfStr-RetainingWall.slmd

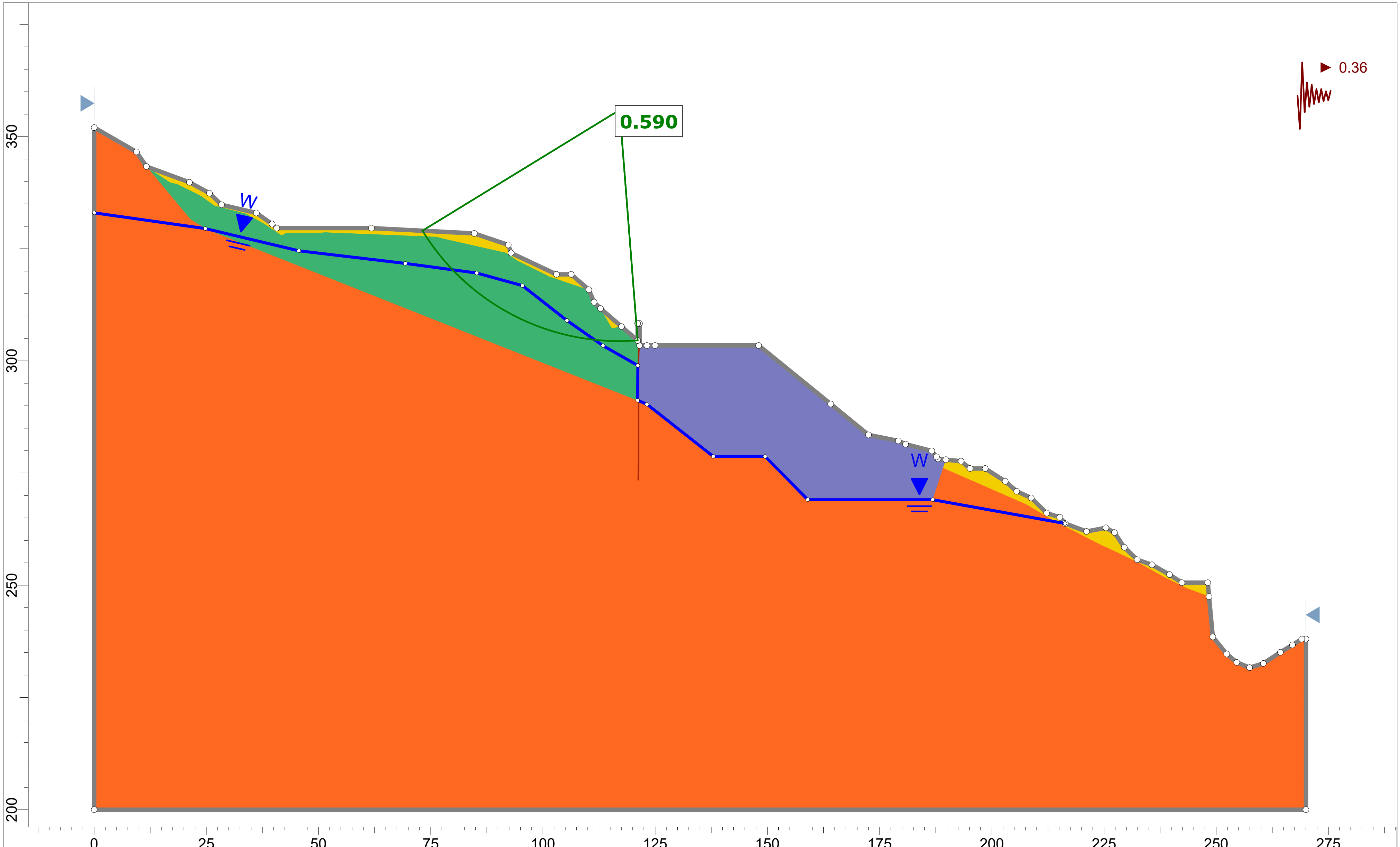
SLIDEINTERPRET 9.020



rocscience

SLIDEINTERPRET 9.020

Project	SLIDE - An Interactive Slope Stability Program		
Group	Buttress w/ Wall	*Scenario*	Buttress w/ Wall - Fully Saturated
Drawn By		*Company*	
Date	3/17/2023, 4:23:27 PM	*File Name*	A-A'-InfStr-RetainingWall.slmd



rocscience	Project: SLIDE - An Interactive Slope Stability Program		
	Group: Buttress w/ Wall	Scenario: Buttress w/ Wall - Seismic	
	Drawn By:	Company:	
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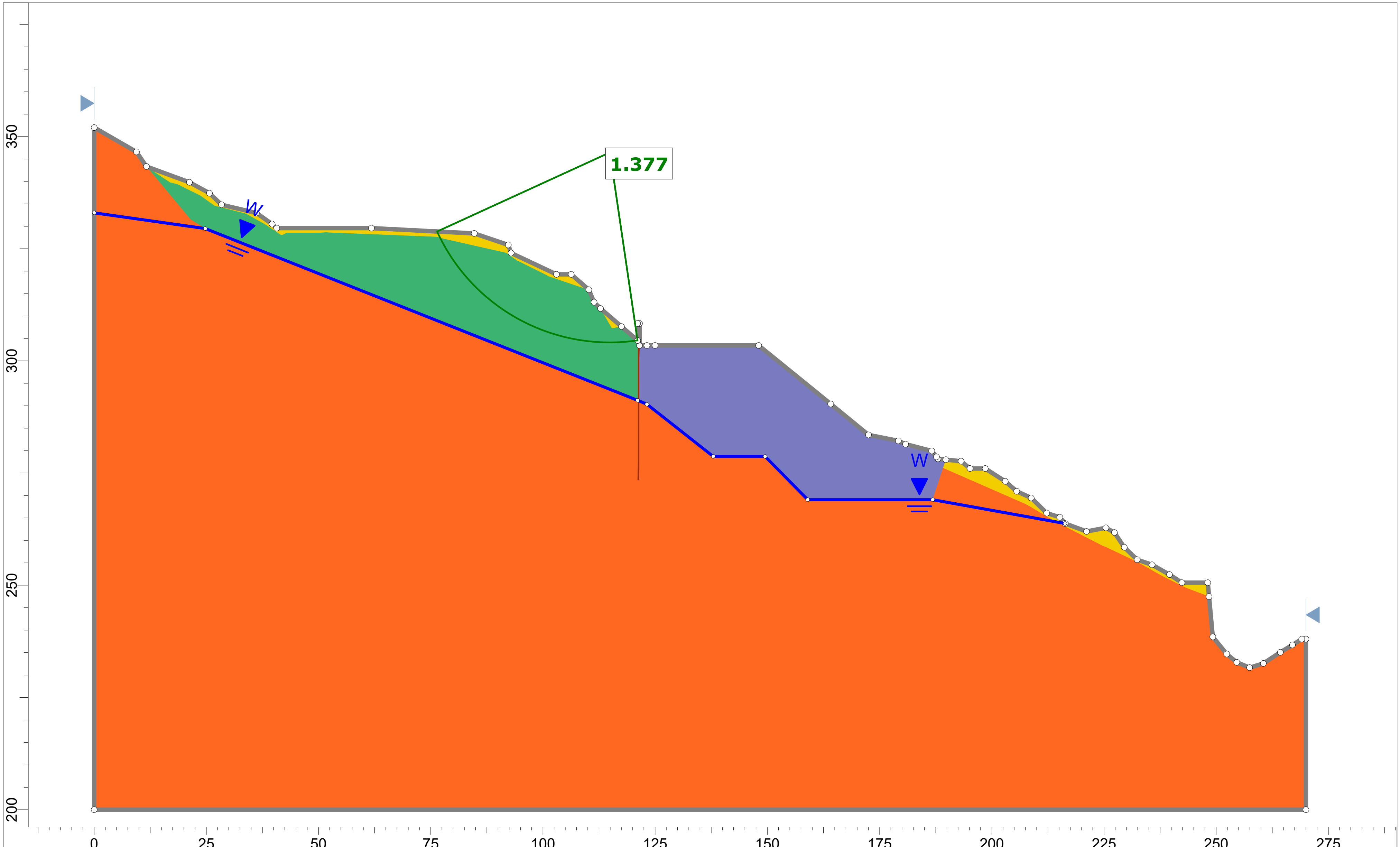

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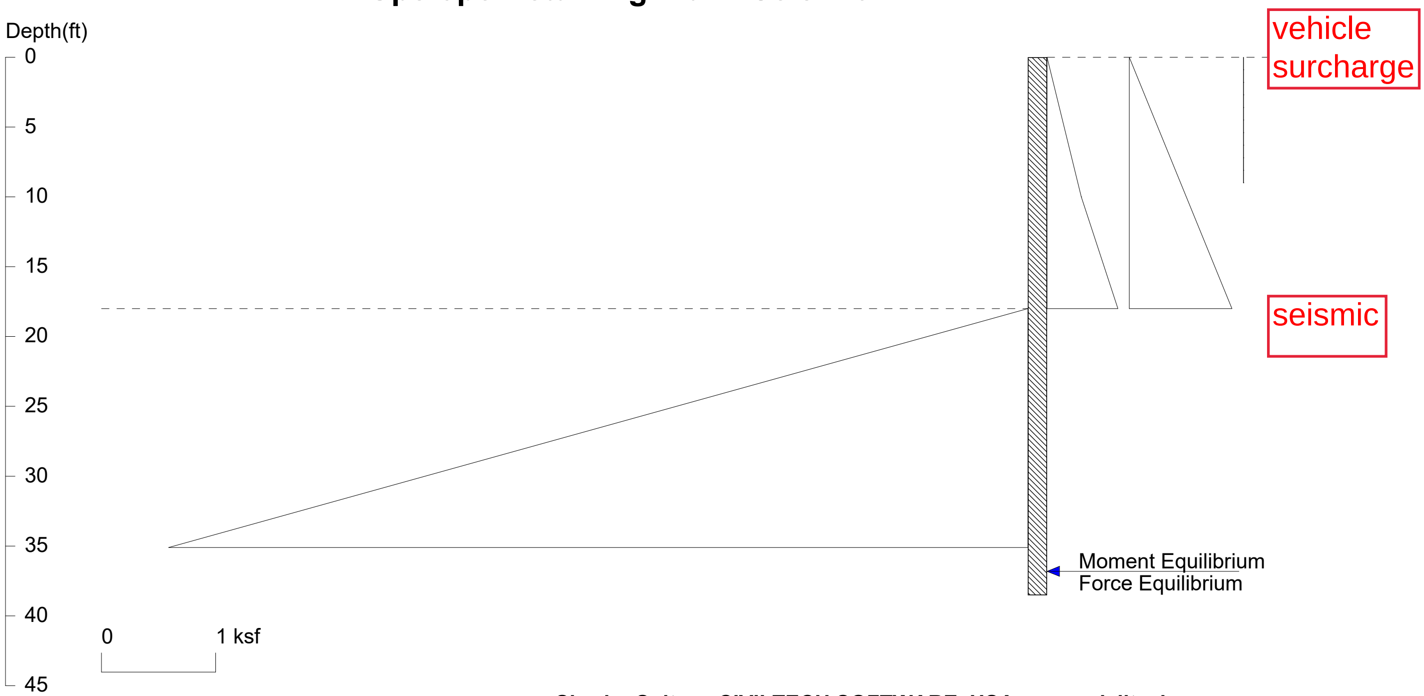
SLIDEINTERPRET 9.020

Project	SLIDE - An Interactive Slope Stability Program		
Group	Buttress w/ Wall	*Scenario*	Buttress w/ Wall - Low GW
Drawn By		*Company*	
Date	3/17/2023, 4:23:27 PM	*File Name*	A-A'-InfStr-RetainingWall.slmd

2180 Higgins Canyon Slope Stabilization
Upslope Retaining Wall - Seismic



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File: Y:\2023\0207686-SanMateoCo-SlipoutRepair-2180HigginsCnynRd\Calculations\CTShoring\Upslope Retaining Wall -

Wall Height=18.0 Pile Diameter=2.0 Pile Spacing=6.0 Wall Type: 2. Soldier Pile, Drilled

PILE LENGTH: Min. Embedment=20.52 Min. Pile Length=38.52
MOMENT IN PILE: Max. Moment=923.79 per Pile Spacing=6.0 at Depth=26.52

PILE SELECTION:
Request Min. Section Modulus = 335.9 in3/pile=5504.78 cm3/pile, Fy= 50 ksi = 345 MPa, Fb/Fy=0.66
-> Piles meet Min. Section Requirements: Top Deflection is shown in (in)
W12X252J (2.73) W14X211J (2.79) W18X175J (2.15) HP18X204 (2.15) W18X192J (1.92)
W21X166 (1.73) W24X146 (1.62) W27X129 (1.56) W30X124 (1.38) W33X118 (1.26)
W36X135 (0.95) W40X149 (0.76) W44X230 (0.36)

FS 1.2 included

DRIVING PRESSURES (ACTIVE, WATER, & SURCHARGE):

Z1	P1	Z2	P2	Slope
0	0	10	0.300	.03
10	0.300	18	0.530	.04
0	0	18	0.900	.05
*	Sur-	charg		
0.000	0.000	0.900	0.000	0.000357
0.900	0.000	1.800	0.000	0.000094
1.800	0.000	2.700	0.000	-0.00007
2.700	0.000	3.600	0.000	-0.00010
3.600	0.000	4.500	0.000	-0.00008
4.500	0.000	5.400	0.000	-0.00005
5.400	0.000	6.300	0.000	-0.00003
6.300	0.000	7.200	0.000	-0.00002
7.200	0.000	8.100	0.000	-0.00001
8.100	0.000	9.000	0.000	-0.00001

PASSIVE PRESSURES: Pressures below will be divided by a Factor of Safety =1.2

Z1	P1	Z2	P2	Slope
18	0	800	344.0	.440

ACTIVE SPACING:

No.	Z depth	Spacing
1	0.00	6.00
2	18.00	2.00

PASSIVE SPACING:

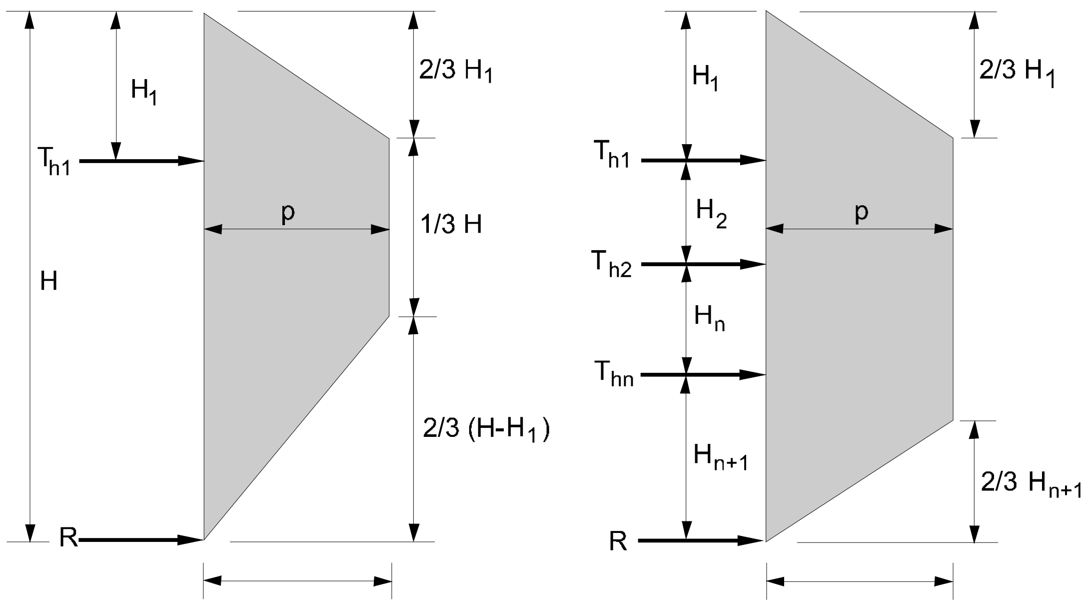
No.	Z depth	Spacing
1	18.00	6.00

UNITS: Width,Spacing,Diameter,Length,and Depth - ft; Force - kip; Moment - kip-ft
Friction,Bearing,and Pressure - ksf; Pres. Slope - kip/ft3; Deflection - in

Appendix E. FHWA Loading Diagram

$$p = 0.65 K_A \gamma H \qquad \text{(Equation 10b)}$$

where ϕ' 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.



$$p = \frac{\text{TOTAL LOAD}}{2/3\ H} \approx K_A \gamma H$$

(a) Walls with one level of ground anchors

$$p = \frac{\text{TOTAL LOAD}}{H - 1/3\ H_1 - 1/3\ H_{n+1}}$$

(b) Walls with multiple levels of ground anchors

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

TOTAL LOAD = $0.65\ K_A \gamma H^2$

Figure 24. Recommended apparent earth pressure diagram for sands.