# ***NORFLEET CONSULTANTS***

Engineering
Geology
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Mr. J. Voss
Stevens Creek Quarry, Inc.
12100 Stevens Canyon Road
Cupertino, CA 95014

December 7, 202
NC Proj. No. 201881.61

RE: Stevens Creek Quarry Use Permit and
Reclamation Plan Application
12100 Stevens Canyon Road
Cupertino, CA 95014

Dear Mr. Voss,

I have reviewed the proposed working and final slope angles outlined in the 2020 Stevens Creek Quarry Use Permit and Reclamation Plan Application. The proposed working (1.5 to 1) temporary cut slopes and final fill slope angles of 2 to 1 and 3 to 1 are consistent with the slope recommendations discussed in my previous reports, Norfleet Consultants (2008, 2020a, and 2020b). The final 3 to 1 fill slope angle is also consistent with the recommendations in the Bagg (2020) report.

**LIMITATIONS OF THIS REPORT**

This report was prepared at the request of, and for the exclusive use of the addressee. Release to any other company, concern, or individual is solely the responsibility of the addressee. Norfleet Consultants is an independent consultant who was retained to provide a preliminary evaluation of slope instability causes. Any other use of this report is strictly forbidden by Norfleet Consultants.

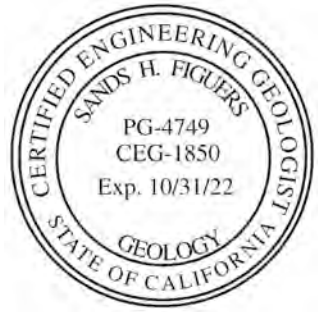
We have employed generally accepted civil engineering and engineering geology procedures. Our observations, professional opinions and conclusions were made using that degree of care and skill ordinarily exercised, under similar conditions, by civil engineers engineering geologists, geophysicists practicing in this area at this time. Norfleet Consultants expressly denies any third party liability arising from the unauthorized use of this report.

If you have any questions, please contact us at 925-606-8595.

Yours truly,
Norfleet Consultants

S. Figuers

S. Figuers

References:

Bagg, 2020, BAGG Engineers, 2019, In-depth Engineering Geologic Investigation and Slope Stability Analysis Western Rim Slope Stevens Creek Canyon Road, Cupertino, California 94117, dated January 2, 2019

Norfleet Consultants, 2008, Geologic and Slope Stability Analysis, Reclamation Plan Amendment, Stevens Creek Quarry, California Mine ID 91-43-007, San Jose, California., Dated January 22, 2008

Norfleet Consultants, 2020a,Preliminary Analysis of Slope Failure at the Northeast Corner of the Stevens Creek Quarry 12100 Stevens Canyon Road Cupertino, CA 94501, dated May 18, 2020

Norfleet Consultants, 2020b, Stability Assessment of the Proposed Cut Slope of the Northwest Corner of the Stevens Creek Quarry. 12100 Stevens Canyon Road Cupertino, CA 95014, dated September 19, 2020

# *NORFLEET CONSULTANTS*

Engineering
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6430 Preston Ave.
Suite A
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Stevens Creek Quarry
Box 26430
San Jose CA 95159

January 22, 2008

Attention: Mr. R. Voss

Re: Geologic and Slope Stability Analysis,
Reclamation Plan Amendment
Stevens Creek Quarry
California Mine ID 91-43-007
San Jose, California

Dear Mr. Voss:

At your request, we have completed our geologic and slope stability evaluation relating to the Reclamation Plan Amendment for the Stevens Creek Quarry. The Reclamation Plan Amendment is an update of the approved 1983 Reclamation Plan for the site.

Our scope of work included:

- A site meeting and overall site reconnaissance with quarry personnel and several data collection site visits to the quarry.

- Compilation, review and summary of available pertinent geologic and geotechnical documents, including a review of recent aerial photographs of the site, to support slope design analysis and recommendations for the Reclamation Plan Amendment.

- Numerical evaluation of cross-sections for slope stability in static and pseudo-static loading conditions of the proposed reclamation slope geometry.

- Discussions with quarry personnel about the implications of the findings of this study.

- Preparation of this report.

The intent and purpose of this of this report is to provide a summary of the geologic and geotechnical issues as they pertain to long-term, global slope stability of the final slope geometries as defined in the Reclamation Plan Amendment. Working and interim slope stability were not evaluated. This study evaluated the pit west of the Berrocal fault, referred to by the operator as Parcel B.

## GEOLOGIC SETTING AND SITE GEOLOGY

The quarry is located on unincorporated land just west of the city of Cupertino in the western foothills of Santa Clara County, east of the San Andreas Fault (Figure 1 and Photo 1). Currently, Franciscan-aged greenstone rocks are mined in the western pit.

The area was regionally mapped in 1909 by Branner et al., and again by the California Geological Survey in 1961. The area was mapped in greater detail by Dibblee (1966), who mapped the general rock types and faults in the study area (Figure 2). The first detailed mapping of the site was performed by Rogers and Armstrong (1973, their Plate 1) at a scale of 1 inch to 1000 feet. That study identified rock types, landslides, faults, and a shear zone in the quarry area (Figure 3). The faults in the area were mapped in detail by Sorg and McLaughlin (1975; Figure 5) at a scale of 1 inch to 2000 feet (their field map had a scale of 1 inch to 100 feet).

Franciscan-aged greenstone (metabasalt) is the primary rock type mined in the pit (Figure 6). A small volume of Franciscan-aged limestone and graywacke (Calera Limestone – Sliter and McGann, 1992; Walker, 1950) have been mined in the northeast corner of the pit. Field observations indicate that the majority of the rocks in the pit are sheared metamorphosed mafic volcanics, with occasional metamorphosed pillow basalts found along the upper part of the west side of the pit. Bailey and Everhart (1964) and McLaughlin and Clark (1997) contain excellent descriptions of the rock types in the quarry area. The north and west sides of the pit are separated by a NW-SE trending shear zone that is 50 to 100 feet wide (Rogers and Armstrong, 1973, and Sorg and McLaughlin, 1975).

All rocks in the pit are fractured/jointed/sheared to varying levels. The rocks underwent multiple stages of deformation/shearing during subduction and later tectonic events. Localized shearing also occurred during development of the Berrocal fault. Field observations indicate that rocks within the pit can be separated into three zones (Figure 6). These zones consist of two linear greenstone cores and a limestone (sedimentary Franciscan) unit. They are separated from each other by high dip shear zones. Both the shear zones and the rock cores appear to trend southeast-northwest at an oblique angle to the northerly trending Berrocal fault. These units are part of the Franciscan melange (Raymond, 1984). Even though they appear to be separate units at quarry scale, the rock cores and shear zones are not regional in scale.

Fracturing within the greenstone cores is relatively widely spaced, and the unfractured greenstone is quite hard (Photo 4). When the cores are mined, the larger greenstone blocks are broken up with a concrete breaker (these rocks were blasted in the past). Fracture spacing, block size, and global rock competence all decrease away from the core to the degree that the rock can be ripped. The shear between the two greenstone zones appears to be combination of serpentine, clay, and highly sheared greenstone (Photos 5 and 6). It can be easily broken apart with a geologist's hammer. Surface topography mimics rock competence. The high ridges overlie the competent cores, while a valley is located over the more fractured rocks in and around the shear zone. In this report, we will refer to shears/joints/fractures as joints unless otherwise indicated.

Superimposed on these relationships is the effect of weathering. The upper 2 to 20 feet consists of a reddish brown residual soil (Photos 7 and 8). This overlies moderately to highly weathered bedrock (a 50 to 90 percent rock/soil mixture) that can extend another 5 to 20 feet. Below this is slightly weathered bedrock. This has weathered brown but contains no observable soil. It is more fractured than the underlying unweathered bedrock. Overall weathering and fracturing (with respect to gross rock competence) decreases with depth. Based on color changes and failure mechanisms, the weathered zone extends 80 to 100 feet below the ground surface. We will refer to the rocks below the visibly weathered units (about 100 feet below the ground surface) as unweathered rocks even though weathering on the microscopic level likely extends hundreds of feet below the ground surface. It appears that as weathering increases, joint persistence is reduced. This change increases the surface unraveling of rocks in the weathered zone, but, at the same time, reduces the potential for large-scale wedge failure.

A small area of Franciscan limestones and sedimentary units is located at the northeast corner of the pit (Photo 9). This unit appears to be the southern continuation of a limestone trend on the Kaiser-Permanente quarry. A shear zone separates greenstone from limestone units. The shear zone is 50 to 80 feet wide. Shear indicators were not visible. The Berrocal fault marks the eastern boundary of this area. Like the greenstones, the limestones and sedimentary units are strongly fractured, and it appears that fracturing increases adjacent to the Berrocal fault. Sandstone units at the northeast corner of the quarry (adjacent to the Berrocal fault) showed indications of mineralization while adjacent clays (not the shear zone clays) were moist. No free groundwater was encountered. The moist zone was about 100 feet in diameter and confined to the clays along the eastern border of the pit.

We walked approximately ½ mile of valley (rattlesnake Canyon) just southwest of the quarry. The valley floor appeared to have been cut in hard greenstone. We did not observe obvious indications of shear zones.

*Berrocal Fault*

The Berrocal fault trends northerly-southerly a few hundred feet east of the pit (Figures 5 and 6). It appears to be high-angle reverse fault, dipping 50 to 70 degrees west. The units west of the fault (Franciscan units) were thrust east over the Santa Clara formation. It is unlikely that there is a specific fault plane. Instead, the fault appears to be a shear zone 50 to 100 feet wide. Mapping by Sorg and McLaughlin (1975) at the southeast corner of the pit suggests that deposition of the Santa Clara formation pre-dates (or occurred early in) development of the Berrocal fault. The original Berrocal fault was mapped by Bailey and Everhart ( 1964, p. 84 and 92) as a strike-slip fault southwest of the New Alamden mining district (about 20 miles southeast of Los Gatos). The name was subsequently applied by Sorg and McLaughlin (1975), and McLaughlin and Clark (1997) to the fault in the Stevens Creek area. These two faults likely have a similar genesis, but we believe that they are separate, unrelated faults. For the sake of continuity we will use the term Berrocal fault in this report, but our discussion only refers to the fault in the Stevens Creek area.

We identified the approximate trace of the Berrocal fault adjacent to the east side of the quarry. There was no obvious surface expression of the fault trace on the ground or on aerial photographs except for the juxtaposition of Franciscan with Santa Clara units. We identified three areas where the location of the Berrocal fault could be narrowed down to between 50 and 100 feet (Locations 1, 2, and 3 on Figures 5 and 12; Photos 2, 3, 25, and 26). These locations had been previously identified by Sorg and McLaughlin (1975) and Rogers and Armstrong (1973). These locations constrain the strike and dip of the fault zone.

At the southern two locations (Locations 2 and 3 -Figures 5 and 12 and Photos 3 and 26), readily identifiable Santa Clara units crop out, but the Franciscan is covered with float. The fault zone is west of these locations. These locations are at approximately the same elevation (650 to 675 feet) and provide an approximate fault trace of N $5^{o}$ to $7^{o}$ W.

The other location is adjacent to the northeast corner of the quarry (Location 1 - Figures 5 and 12 and Photo 2). Here, the fault cuts obliquely across a north-south trending dirt road. Franciscan limestones crop out in the road south of the fault zone and apparently undistrupted Santa Clara units crop out in sidecuts a hundred feet or so to the north. This outcrop is at an elevation of about 915 feet.

The Sorg and McLaughlin map (Figure 5) shows the fault with a shallower dip (<40 degrees west) cuting through Franciscan units at Location 2. This localized change in fault dip is inconsistent with thrust behavior. It is more likely that Santa Clara units extend further west at location 2 (than shown on the Sorg and McLaughlin map) and the Berrocal fault has a steeper dip. A three-point evaluation suggests that the fault zone currently dips 50 to 70 degrees to the west. The Santa Clara east of the Berrocal fault has a gentle synclinal form with an axis that trends about N45W and dips to the southeast, oblique to the trend of the Berrocal fault in this area.

It is likely that the Berrocal fault formed as part of a flower structure related to slip on the San Andreas. It is unlikely that it is currently an independent seismogenic feature. Slip may occur when the near-by section of the San Andreas fault shifts. The lack of surface displacement features along the trace of the Berrocal fault in the Stevens Creek area suggests that there is little historic (10,000 years or more) displacement on the fault. It is also possible that this section of the fault has been rotated to a steeper dip (20 to 30 degrees) by subsequent movement on deeper faults and is now no longer active. It appears that the global movement of the Franciscan units in this area are north-northeast (not east) and that this section of the Stevens Creek fault was never a true thrust, but is instead a high-angle, lateral reverse fault with oblique movement.

*Air Photo Analysis*

Table 1 contains a list of aerial photographs of the Stevens Creek Quarry area reviewed as part of this study. Landslides had been mapped by Sorg and McLaughlin (1975; Figure 5), Rogers and Armstrong (1973; Figure 4), and Pike (1997). Our air photo analysis does not support the identification of landslides in the vicinity of the quarry interpreted by previous workers. The large landslide at the northwest corner of the quarry mapped by Sorg and McLaughlin (1975) is located along a ridge crest, not on the side of a ridge. It appears to be a tectonic block bound by shear zones. The trace of the Berrocal fault could not be readily identified on the aerial photographs.

Table 1
Aerial photographs evaluated as part of this study.

| Source | Date | Line and Photo Nos. | Scale |
|---|---|---|---|
| Pacific Aerial | 7-14-04 | AV8769-2-7, 8, 9 | 1:7,200 |
| Pacific Aerial | 7-14-04 | AV8769-1-7, 8, 9 | 1:7,200 |
| Pacific Aerial | 7-28-97 | AV5472-3-9, 10, 11 | 1:24,000 |
| Pacific Aerial | 7-28-97 | AV5472-4-8, 9, 10 | 1:24,000 |
| Pacific Aerial | 10-8-96 | AV5200-17-72, 73, 74 | 1:12,000 |

*Seismicity*

The San Andreas fault is approximately 5 miles west of the quarry. This section of the San Andreas fault is classified as a Type A fault and has an estimated Mmax of 7.9 (ICBO, 1998). The site has a 10 percent chance in 50 years of experiencing 0.57g peak ground acceleration (PGA) (USGS, 2007; Earthquake Ground Motion web site).

The quarry was active during the Loma Prieta Earthquake of October 17, 1989. The estimated ground acceleration from that earthquake at the quarry was about 0.2g. Quarry personnel indicated that the quake did not cause rock falls or slope failures. Reportedly, only a single water glass fell off a counter in a nearby house during the Loma Prieta earthquake. Historic aerial photograph review indicates that the quarry was smaller in 1989. The highest slopes were 100 to 200 feet high at the time of the 1989 Loma Prieta Earthquake.

A study of aftershocks from the Loma Prieta Earthquake in the Santa Cruz Mountains (Lindley and Archuleta, 1994) found that Franciscan ridgetops had little ridgetop amplification, and the average amplification at Franciscan sites was 3 times less than amplification at Miocene and Pliocene sites.

The slopes surrounding the quarry floor were identified by the California Geological Survey (2002, Cupertino Quad) as having a potential for permanent ground displacements (earthquake-induced landslides). No liquefaction potential was identified in this area.

*Groundwater*

There is a series of houses on the hill south of the quarry (Monte Bello Ridge). The water supply to some of those houses is provided by wells. The bottom of some of the eastern wells extends below the elevation of the quarry floor while the bottom of wells higher in the hills is above the elevation of the quarry floor. The quarry is separated from these houses (and wells) by an unnamed stream in Rattlesnake Canyon (local name). The elevation of the stream (and the base of the valley) adjacent to the quarry is between 650 and 690 feet. The lowest elevation of the quarry floor is projected to be between 700 and 725 feet. When quarrying is finished, the quarry will be filled with ~150 feet of fill. Subdrain lines are and will be incorporated into the fill. The quarry is relatively dry, and there is no record of long-term, large water inflows into the quarry or historic need for drainage wells to control water inflows. There is no record of water wells within 1000 feet west, north, or east of the quarry.

We observed two seepage areas in the quarry walls (Figure 8). One is located in the west face near the south end of the quarry, and the second is located in the middle of the north face. The western seepage area (Photo 10) consists of a series of sub-horizontal seeps that extend 100 to 150 feet at about the 800 foot elevation. At the time of our site visit (in the fall, the driest time of year), only the southernmost seep was active, producing in the range of 5 to 10 gallons of water per hour. The remainder of the seeps were marked by rinds of efflorescent salts. There was no obvious alteration/weathering of the bedrock in the vicinity of the seeps. This area is at the base of weathered greenstone, and it appears that this zone is related to slope interflow through the weathered zone. It is likely that the flow increases during the winter.

The second seep area is located in hard bedrock in the middle of the north face (Photos 11 and 12) at about elevation 925 feet (the top of the face is above 1200 feet elevation). This zone consists of two seeps, spaced 20 to 30 feet apart at about the same elevation. The flow is in the range of 10 to 20 gallons per hour. There is a 2 to 3 inch wide, vertical clay zone below the eastern seep. The flow from these seeps is currently directed into the existing gravity drainage system. There is no indication that drainage wells have been used in or around the quarry. The majority of the quarry walls are covered with fill, and no obvious indications of seepage were seen in those areas. It is likely that there is some seepage in the northeast corner of the quarry. A seasonally dry valley and dry stream above this part of the quarry trend towards the northwest corner of the quarry.

Quarry personnel indicated that a few years ago, a gush of water occurred when a new cut was made at the east end of the north face in the limestone area. The flow of water was initially large. The flow slowly decreased over a few days and was negligible a week or two later. This flow appears to have occurred at the junction between the greenstone and the limestone at the northeast corner of the pit. The nature of the flow suggests that this was an isolated pocket.

This pit has been active for more than 40 years, and portions have been excavated to approximately 725 foot elevation. The quarry acts as a very large diameter drainage pit.

Currently, total flow from the quarry is in the range of 5 to 10 gallons per minute. The majority of effects on the surrounding groundwater have already occurred. It is likely that bedrock groundwater levels adjacent to the quarry will rise when the quarry is backfilled.

Rattlesnake Canyon acts as a hydrologic barrier between the quarry and the hill south of the quarry. We are unaware of any complaints or comments about groundwater elevation changes in the surrounding area that might be related to quarry operations.

**Slope Stability Considerations**

More than 60 percent of the northern and western faces was covered with side cast fill at the time of our site visits. We observed numerous landslides within the fill (at all scales) but did not observe obvious indications of failure of the underlying rock in the side cast areas.

The current western and northern quarry faces are 250 to 300 feet high and slope steeply. The western face is about 12 years old (slopes 40 to 50 degrees east; Photos 8 and 17). The upper bench was cut 6 to 7 years ago. The northern face is 2 to 3 years old (slopes 45 to 70 degrees south; (Photo 13). Variously sized wedge failures occur in the lower, unweathered material in the western face (Photo 14). These failures range in size from a few cubic yards to hundreds of cubic yards. We did not observe similar wedge failures in the northern face. It is likely that the observed failure differences between the two faces is a function of joint patterns and the trend of each face.

A northwest-southeast trending shear zone is located at the northwest corner of the quarry. Much of this area is covered with fill. The only exposure of the shear zone is a 40 foot high by ~100 foot long cut at the base of the slope (about 200 feet below the original ground surface; Photo 5). The western part of the zone was covered, but the zone is in the range of 50 to 100 feet wide. The shear zone is serpentine. Shearing is pervasive. At small scale, shears occur in almost all directions, but the shears are short (a few inches), curvilinear, and are truncated by other shears (Photo 6). Polished shear surfaces are common.

In outcrop scale, the overall shear trend in the exposed face is N25-30W with a high dip (~80 degrees east/west). The eastern end of the face contains fracture-bound angular greenstone blocks (6 inches to a few feet in size), while the western end of the face is highly sheared serpentine that contains numerous greenish pods in a black matrix. The pods are football shaped that are few inches to a few feet long. The long axes are sub-parallel to the overall shear strike direction. The pods do not appear to be significantly stronger than the surrounding matrix. We picked up a pod that was about 2 feet long by the ends. After a few minutes, the pod fell apart under its own weight along internal fractures. The pods and matrix can easily be broken apart with a rock hammer. The cut face is perpendicular to the overall shear trend and is a few months old. The face is failing by localized wedge failures and face spalling (Photos 15). There is no evidence of large-scale arcuate failures.

An inactive eastern face extends the length of the quarry (Photo 16). The northern end of the face was cut in Franciscan sedimentary units (limestones, sandstones, and clays). The rest of the face was cut in weathered greenstone. The greenstone face is 50 to 75 feet high and slopes 45 to 55 degrees west. This face is sub-parallel to the Berrocal fault which is 300 to 400 feet to the east. The southern end of the face is more than 45 years old, the middle part of the face is about 30 years old and the northern end of the face is 8 to 12 years old. The middle part of the face contains a series of landslides (both circular and planar failure surfaces). The area adjacent to the toe of this slope is used for temporary rock storage. The toe of this slope is occasionaly destabilized as the stored material is removed.

*Joints*

We mapped fracture/joint trends in the northern and western quarry faces as well as in two exposures in the middle of the quarry. There is a wide variation in joint density, orientation, and length. We did not observe quarry-wide joint patterns. The majority are short (a foot to less than 40 feet long) and truncate against other joints. Joint spacing varied from less than an inch to 5 to 10 feet. Some joints were planar, but most were curvilinear (the strike and dip could vary ±20 to 30 degrees). The joints are rarely filled, and the joints in the unweathered greenstone are tight. Scattered slickensides were observed. Occasional shear zones were observed in the western face. These appeared to be late stage for they were not cut by other joints or shears. The zones are 1 to 5 feet wide and 20 to 60 feet long. These shears have a high dip (70 to 90 degrees) and trend easterly-westerly. The rock within the shear zone is broken into smaller pieces but no gouge was visible. The shears were occasionally filled with vein material (1/4 to ½ inch wide).

Stereonet plots of all measured joints are shown in Figure 9, and stereonet plots of joints in the western face are shown in Figure 10 (combined weathered and unweathered units). There is a wide scatter, but there is a general northeast-southwest strike trend with dips steeper than 40 degrees to the east. This is based on a limited data set (73 data points). Several hundred data points would be needed to confirm these trends. Few fractures were measured in the northern face because the face was steep and the lower 10 to 20 feet of the face was dangerous to climb on. The western face data is consistent with the wedge failures on the west face. The apparent lack of persistent joints with a moderate dip to the south is consistent with the lack of large-scale wedge failures in the north face.

*Failure Types*

We did not observe large-scale failures in the quarry walls, and there has been no reported history of large-scale failures. The majority of observed rock failures were relatively small block and wedge failures related to joint orientation.

A zone of wedge failures in the unweathered greenstone occurs along the central section of the western face between elevations 750 to 1050[1] feet (Photos 14 and 18). These failures progress upwards to but do not appear to extend into the overlying weathered greenstone. This face trends ~N20E and slopes ~45 degrees east. We measured the failure planes of several of the wedge failures. The basal surface of those failures trends N10W to N20E and dips 45 to 60 degrees south. The dips of joints with a N-S trend and easterly dip in this area were mainly 45 to 60 degrees, but some dipped 25 to 35 degrees. The wedge failures appeared to be restricted to the more competent, less fractured greenstones. They did not extend south into more fractured greenstone (either weathered or unweathered).

In the west slope, engineered wedge fill placed as part of the reclamation plan will extend to between 1000 and 1050 feet elevation, and much of the current zone of wedge failures will be covered and buttressed with fill. The western face final rock slope above the fill will dip approximately 32 degrees east (1.5:1). It appears at this time that the dip of basal planes of the current wedge failures will not daylight in the final rock slope.

There are partial failures of the weathered greenstone face in the high bench in the western slope (Photo 19). The face is about 40 feet high, slopes about 75 degrees east, and trends north-south. There is a series of shears that trend east-west across that face. The shears are semi-vertical and are spaced 10 to 20 feet apart. There were several failures in the cut slope. It appears that the these failures began with spalling of fractured rocks within the shear zone itself and progressively widened laterally (Photo 20). There was no obvious classical wedge failure or global failure of the cut face. The cut face in this bench more than 17 years old. The cut face below the bench is 7 to 8 years old.

There are failures in weathered bedrock along the eastern cut slope of the quarry (Photo 16). That cut slope varies from 30 to 60 feet high and is 20 to 45 years old. The face trends ~N15W and slopes 45 to 55 degrees west. This face is quasi-parallel to and several hundred feet west of the trace of the Berrocal fault. The middle part of the face contains a series of landslides (both circular and planar failure surfaces; Photo 21). The area adjacent to the toe of this slope is used for temporary rock storage. The toe of this slope is routinely destabilized (a minor amount) by equipment as the stored material is removed.

The northern face trends ~N60E. The west end of the slope is covered with spill fill and slopes ~45 degrees south (Photo 13). The fill is marginally stable, and soil-related landslide features are common. We did not observe obvious large-scale failure of the underlying bedrock. The center part of the slope contains exposed hard greenstone that dips ~60 degrees south. The face was irregular, and smaller block failures were common. We did not observe obvious wedge failures (as observed in the western face) in this area.

[1] Note: All elevations are approximate.

## SLOPE STABILITY ANALYSIS

*Final slope configuration*

The proposed Reclamation Plan Amendment (Resource Design Technology, 2007; Figure 11) indicates that reclamation includes construction of an engineered wedge fill around the perimeter of the quarry. The base of the wedge fill will rest on the quarry floor at approximately 700 foot elevation and the upper edge of the wedge fill will extend to about 1000 foot elevation and rest against the quarry walls. The surface of the wedge fill will slope 2:1 towards the interior of the quarry. The center of the quarry will be filled with ~150 feet of fill (to ~850 foot elevation). The western and northern quarry faces will extend from tens of feet to approximately 250 feet above the top of the wedge fill. The exposed rock face (weathered and unweathered units) will have a 1.5:1 slope.

*Limit Equilibrium Method*

We used GSTABL7, a computer program, to evaluate the Factor of Safety (FS) for various slope orientations and material properties. We performed both static and pseudo-static (seismic) slope evaluations. Bishop's method of slices was used to evaluate circular failure modes. Joint mapping did not identify persistent fracture sets that would justify evaluation of the slopes with Janbu's method. Based on our seismicity analysis, we used a pseudo-static coefficient of 0.2g to evaluate the stability of each slope for pseudo-static (seismic) loading conditions.

Under the Uniform Building Code (UBC), the minimum static FS for slopes where human occupancy is planned is 1.5, and 1.1 for pseudo-static conditions. Based on the use of the site after reclamation as open space, with no engineered structures or concentrated public access, we propose that a static FS between 1.3 and 1.5 is acceptable. Table 2 lists the significance of various Factors of Safety according to Sowers (1979, p. 587).

Table 2
Significance of the Factor of Safety (Sowers, 1979, p. 587)

| Factor of Safety | Significance |
|---|---|
| Less than 1.0 | Unsafe |
| 1.0 to 1.2 | Questionable safety |
| 1.3 – 1.4 | Satisfactory for cuts and fills |
| 1.5- 1.75 | Safe for dams |

The limit equilibrium method was developed for soil slope stability analysis and assumes particle friction, a relatively homogenous material, and a smooth (arcuate) failure surface. When used for rock slopes, the phi and cohesion values are average, non-directional rock mass parameters. They can only account for fractures and other material irregularities in an indirect manner. Most of the time, rock slope stability is controlled by other factors such as particle/block interlock or failure on existing, non-circular surfaces. Equivalent phi angle and cohesion strength [phi $_{eqv}$ ($P_{eqv}$) and cohesion $_{eqv}$ ($C_{eqv}$)] are used to signify estimated rock properties in the limit equilibrium analysis.

Rock mass rating systems began to be developed in the 1960's to evaluate the stability of underground openings. Several have been expanded to evaluate rock slopes. These include: RMR, MRMR, RMS, SMR, SRMR, SSPC, CSMR, GSI, USC, Q, M-RMR, BQ, RMi, and others. All of these rating systems attempt to identify and incorporate the main features of a rock mass that define rock shear strength, and, subsequently, rock stability. The basic parameters used in these rating systems include block size and spacing (typically defined by RQD, derived from drill cores, but some allow scan mapping of a rock face), the nature of rock defects (persistence, roughness, infilling, width, weathering, spacing, orientation), and ground water. These parameters are evaluated and combined into a single value. That value is typically used with design curves to estimate overall rock stability. Some of the classification systems are based on specific rock types, conditions, and slope height and have limited applications. Ongoing debates about the nature and incorporation of the various rock parameters cause modifications of existing rating systems and creation of new ones. Palmstrom (2001) contains a good review of rock characterization for rock rating systems. Hack (2002) and Douglas (2002) reviewed many of the rock mass rating systems.

Numeric modeling methods (FEM, FDM, Distinct Element, Discontinuous Deformation Analysis) are also used to evaluate rock slopes. FEM, FDM codes require an extensive set of rock and joint properties which can be difficult to reasonably define. It is also difficult to model rock that is extensively fractured.

A Block-in-Matrix (BIM) rock analysis is useful to evaluate soil/rock mixtures (Lindquist, 1994; Medley, 1994; and Kim, Snell, and Medley, 2004). Except for the residual soils, the majority of the quarry rocks (weathered and unweathered) contain greater than 75 percent rock. This rock percentage indicates that the exposed greenstone slopes are fractured rocks instead of BIM rocks. The serpentine shear zone in the northwest corner of the quarry could be considered a BIM rock. The effect of blocks within a fine-grained matrix is to increase create a complex shaped failure surface. This is represented in a limit equilibrium model by increasing the phi angle by 10 to 20 degrees (Kim, Snell, and Medley, 2004; Medley, 1994).

A limit equilibrium method with a circular failure surface analysis is used in this evaluation for the following reasons.

> The deeper quarried slopes will be backfilled and supported by engineered fill. Rock mass rating systems are not designed to evaluate soil slopes. They could provide an estimate of the stability of the rock portion of the slopes, but that estimate would have to be transferred into a limit equilibrium analysis. Only up to about 100-150 feet of unweathered rock will be exposed. At this depth, internal rock dilation/deformation is expected to be minimal.

Parametric studies of rock properties can more easily be performed with a limit equilibrium analysis. Phi and cohesion values can be quickly varied to accommodate layer thickness variations, estimate effects of joints on global rock properties, and estimate variations in both soil and rock types on slope stability.

For the most part, weathered rock will be present on the upper cut slopes above the engineered fill. Rock weathering reduces the effect of rock structure and increases the likelihood of arcuate, soil-like failures.

It is unlikely that a rock mass rating evaluation would provide a better evaluation of the final rock slopes. The rock mass ratings are lumped parameter characterization systems, not classification systems or design methodologies (Stille and Palmstrom, 2003). They can only provide an estimate of the global stability of a slope. The proposed quarry final slope height and dip are at the lower end of most of the rock mass rating system design curves. It is also difficult to vary the parameters that make up a rock mass rating evaluation in order to perform parametric evaluations.

*Material properties*

For stability evaluation purposes, four rock types (unweathered greenstone, weathered greenstone, sheared rock, and fill) are used in the stability analyses (Table 3). Spatial changes in weathering, joint density and persistence will cause variations in rock properties, but the range of that variation can only be estimated at this time.

We performed a back-analysis on both cut and natural slopes to estimate in-place weathered and unweathered rock properties (phi $_{eqv}$ and cohesion $_{eqv}$). We then lowered those strength values to include lower strength rock/joint conditions (called lower bound values). It is likely that the actual rock strength values are closer to or higher than the back-analysis properties. We chose to vary cohesion and keep phi values fixed.

Table 3
Assumed Engineering Material Properties

| Material Type | Lower bound Cohesion (C psf) | Back-calculated Cohesion (C psf) | Friction Angle (Phi - $\phi$) (degrees) | Unit Weight (pcf) | Analysis Layer Number |
|---|---|---|---|---|---|
| Unweathered Greenstone | 2000 ($C_{eqv}$) | 5000 ($C_{eqv}$) | 32 ($P_{eqv}$) | 155 | 1 |
| Weathered Greenstone | 1000 ($C_{eqv}$) | 3000 ($C_{eqv}$) | 28 ($P_{eqv}$) | 155 | 2 |
| Compacted Fill | 150 | - | 31 | 130 | 3 |
| Sheared rock | 500 ($C_{eqv}$) | 1000 ($C_{eqv}$) | 38 ($P_{eqv}$) | 130 | 4 |

COMPACTED FILL - The CGS (2002) seismic hazard zone report listed a regional value of fill (af) as 20 degrees/ 560 to 651 pcf. The CGS values are the mean/median of 27 tests.

A value of 31 degrees/ 150 pcf were used in this report. This value come from triaxial testing of two samples from on-site imported fill (Appendix A). It is likely that this material is similar to the material that will be used to fill the quarry. Additional strength testing of the onsite fill could refine this value. The fill comes from the greater San Jose area. It is typically sandy and has been tested for contaminants. Little to no bay muds are imported. These are low end values. The material was sieved prior to testing to remove larger (> 1/2 inch) material. Having larger sized material in the fill will tend to increase the phi angle (create a BIM like material).

WEATHERED GREENSTONE - The CGS (2002) seismic hazard zone report listed greenstone (fg) strength properties as 28 degrees/ 680 to 565 pcf. The CGS values are the mean/median of 43 tests. It is likely that the majority of these values represent deeply weathered greenstone (10 to 40 feet from the ground surface) instead of mild or moderately unweathered greenstone.

The northern side of the Rattlesnake Canyon is adjacent to the southwest corner of the quarry (Figure 7; Photo 22). The natural slope adjacent to the quarry is about 500 feet high. The top of the slope (1100 to 1225 feet elevation) dips south at 2:1 (~26 degrees). The lower part of the slope (725 to 1100 feet elevation) dips south at 1.5:1 (32 degrees). The bottom of the slope is at about 700 feet elevation. The overall dip is 1.65:1 (31 degrees) south. We walked this slope to confirm the nature of the slope and the topography. The slope cover is soil with occasional rock outcrops, and the dip is relatively planar. We observed a small, shallow landslide at the base of the slope (below 825 feet elevation). This landslide appears to have occurred in soil, not bedrock. It is not visible on the aerial photographs because of tree cover. Aerial photograph evaluation indicates that this area is consistent with the overall slope in this canyon. This slope is thousands of years old and has experienced numerous large earthquakes from the nearby San Andreas fault. We did not observe obvious large landslides on the historic aerial photographs.

We back-calculated the stability of this slope using varying phi and cohesion values to estimate lower bound strength properties of weathered greenstone using static and pseudo-static slope stability analyses. The initiation points and termination limits in the limit equilibrium analysis were set to force the failure surfaces to extend over more than one-half the slope. The resulting factors of safety (FS) are shown in Table 4. These analyses suggest that the cohesion $_{eqv}$ of weathered bedrock would be in the range of 2500 psf to obtain an FS value of 1.5.

Table 4
Static FS for various equivalent phi and cohesion values for cross-section W-W'

| Phi $_{eqv}$ | Cohesion $_{eqv}$ 500 psf | 1000 psf | 1500 psf | 2000 psf | 2500 psf | 3000 psf |
|---|---|---|---|---|---|---|
| 28 | 1.03 | 1.16 | 1.24 | 1.32 | - | 1.47 |
| 30 | 1.10 | 1.24 | 1.33 | 1.41 | 1.48 | - |
| 32 | 1.18 | 1.32 | 1.42 | 1.49 | 1.57 | - |

To evaluate a lower bound FS for this slope, we completed a pseudo-static back analysis using the static failure surfaces that developed from apparent phi and cohesion values of 28 degrees / 3000 psf and 32 degrees / 2000 psf. Using a horizontal acceleration of 0.2 g, the cohesion $_{eqv}$ was increased until an FS of 1.15 was obtained. For a 28 degree phi $_{eqv}$, the back-calculated cohesion $_{eqv}$ was 4000 psf. For a 32 degree phi $_{eqv}$, the back-calculated cohesion $_{eqv}$ was 3000 psf.

The weathered greenstone layer in cross-sections A-A' and B-B' was evaluated with a phi $_{eqv}$ of 28 degrees and a cohesion $_{eqv}$ of 3000 psf (high strength value) as well as with a phi $_{eqv}$ of 28 degrees and cohesion $_{eqv}$ of 1000 psf (low strength value).

UNWEATHERED GREENSTONE - The CGS (2002) seismic hazard zone report listed greenstone (fg) strength properties as 28 degrees/ 680 to 565 pcf. It is likely that these values represent weathered greenstone, not unweathered values (100 feet or more below the ground surface).

Cross-section Z-Z' is a current cut slope located at the north end of the quarry where the more competent greenstone is located. This area is less fractured than other parts of the quarry. Table 5 shows the results of the static back analyses for this cross-section.

Table 5
Static FS for various equivalent phi and cohesion values for cross-section Z-Z'

| Phi $_{eqv}$ | Cohesion $_{eqv}$ 2000 psf | 3000 psf | 4000 psf | 5000 psf | 6000 psf |
|---|---|---|---|---|---|
| 32 | 0.98 | 1.21 | 1.27 | 1.41 | 1.53 |
| 35 | 1.05 | 1.21 | 1.35 | 1.49 | 1.62 |

The static analysis assumes minimal jointing and a high cohesion value can be assumed. An overlying weathered rock layer was included in the cross-section. The strength of the weathered rock had little effect on the overall FS of the slope. If the weathered zone was set to unweathered rock properties (at phi $_{eqv}$ of 32 degrees and a cohesion $_{eqv}$ of 5000), the FS increased from 1.41 to 1.46.

The unweathered greenstone layer in cross-sections A-A' and B-B' was evaluated with a phi $_{eqv}$ of 32 degrees and a cohesion $_{eqv}$ of 5000 psf (high strength value) and with a phi $_{eqv}$ of 32 degrees and a cohesion $_{eqv}$ of 2000 psf (low strength value).

## RESULTS

The results of our slope stability analyses are listed below. The GSTABL7 computer outputs are included in Appendix B. The slope configurations, material properties, and critical failure surfaces determined for these cross-section locations are shown on the computer output figures. We assumed total stress conditions and that groundwater levels were below the base of potential failure surfaces. Quarry fill consists of two layers. A wedge buttress fill placed against the cut slopes and a flat fill (up to 150 feet thick) placed in the middle of the quarry. The two fills will be referred to as the wedge fill and the flat fill, respectively.

*Cross-section A-A', west*

Cross-section A-A' west trends east-west across the southern end of the west face of the quarry (Figure 7; Photo 22). A series of slope stability analyses was done with varying rock and weathered rock cohesion values to evaluate overall slope stability (Table 3). The results are summarized in Table 6. On this section, the wedge fill extends to 1050 feet elevation, and the fill layer is 150 feet thick.

Table 6
FS values for cross-section A-A' west. Cohesion $_{eqv}$ in psf.

| Analysis Type | Section Evaluated | Unweathered rock | | Weathered Rock | | FS | Newmark Displacment |
|---|---|---|---|---|---|---|---|
| | | $P_{eqv}$ | $C_{eqv}$ | $P_{eqv}$ | $C_{eqv}$ | | |
| Static | Full Slope | 32 | 5000 | 28 | 3000 | 1.76 | - |
| Static | Full Slope | 32 | 2000 | 28 | 1000 | 1.42 | - |
| Static | Rock only | 32 | 5000 | 28 | 3000 | 2.55 | - |
| Static | Rock only | 32 | 2000 | 28 | 1000 | 1.73 | - |
| Static | Fill only | 31 (fill) | 150/250 | - | - | 1.39 / 1.47 | - |
| Pseudo-Static | Full Slope | 32 | 5000 | 28 | 3000 | 1.22 | 0.17 ft |
| Pseudo-Static | Full Slope | 32 | 3000 | 28 | 2000 | 1.11 | - |
| Pseudo-Static | Full Slope | 32 | 2000 | 28 | 1000 | 0.97 | 0.54 ft |

Full Slope Analysis – The initiation points and termination limits in the limit equilibrium analysis were set to force the failure surfaces to start at the upper part of the slope and daylight in the vicinity of the toe of the fill slope. Both high and low rock properties were evaluated.

Rock Only Analysis - The initiation points and termination limits in the limit equilibrium analysis were set to force the failure surfaces to start at the upper part of the slope and daylight in the vicinity of the top of the fill slope. The failure surface would be in rock only. Both high and low rock properties were evaluated.

Fill Only Analysis - The initiation points and termination limits in the limit equilibrium analysis were set to force the failure surfaces to start at the top of the fill and daylight in the vicinity of the toe of the fill slope.

The variation in cohesion $_{eqv}$ values suggests that the slope will be stable (above 1.3 FS) for a wide range of rock cohesion $_{eqv}$. Rock cohesion $_{eqv}$ has to be at least 3000 psf for the pseudo-static FS to exceed 1.1

A zero fill cohesion value create a shallow, surface failure within the wedge fill.

*Cross Section B-B'*

Cross-section B-B' trends north-south across the middle of the quarry (Figure 7; Photo 23). The slope in Section B is not as high as the slope in Section A-west. It has a similar layer geometry as section A-A', west, but the ground surface north of the quarry property line drops in elevation. On this section, the wedge fill extends to 950 feet elevation, and the fill layer is 150 feet thick.

Fill stability was not evaluated because fill has the same geometry and material properties as in Section A-A', west and the FS for fill in Section B-B' will be similar to that in Section A-A', west. The results are summarized in Table 7.

Table 7
FS values for cross-section B-B'. Cohesion $_{eqv}$ in psf
Phi $_{eqv}$ held constant. Fill properties held constant.

| Analysis Type | | Unweathered rock | | Weathered Rock | | FS | Newmark displacement |
|---|---|---|---|---|---|---|---|
| | | $P_{eqv}$ | $C_{eqv}$ | $P_{eqv}$ | $C_{eqv}$ | | |
| Static | Full Slope | 32 | 5000 | 28 | 3000 | 2.00 | - |
| Static | Full Slope | 32 | 2000 | 28 | 1000 | 1.52 | - |
| Static | Rock only | 32 | 5000 | 28 | 3000 | 2.62 | - |
| Static | Rock only | 32 | 2000 | 28 | 1000 | 1.72 | - |
| Pseudo-Static | Full Slope | 32 | 5000 | 28 | 3000 | 1.40 | 0.09 ft |
| Pseudo-Static | Full Slope | 32 | 2000 | 28 | 1000 | 1.05 | 0.35 ft |

The factors of safety for section B-B' are higher than in section A-A' because of the change in the surface geometry along the top of the slope.

*Cross Section A-A', east*

Cross-section A-A', east trends east-west across the east side of the quarry at the south end of the eastern face (Figure 7; Photo 24). The flat fill will extend above unweathered rock and only weathered rock will be exposed. The nature of the weathered rock in this area indicated that lower cohesion values should be used in the evaluation. The results are summarized in Table 7.

Table 7
FS values for cross-section A-A', east. Cohesion $_{eqv}$ in psf
Phi $_{eqv}$ held constant. Fill properties held constant.

| Conditions | FS |
|---|---|
| Static; moderate failure, Rock $C_{eqv}$ =1500, Weath. Rock $C_{eqv}$ =450 | 1..6 to 1.9 |
| Static; shallow failure, Rock $C_{eqv}$ =1500, Weath. Rock $C_{eqv}$ = 450 | 1.51 |

Very low unweathered rock values were assumed in this analysis, but the failure surfaces did not pass through unweathered rock. The current east face is over steepened (50 to 60 degrees dip) and is marginally stable (FS<1.2). That slope will be reconfigured to a 2:1 slope.

*Cross Section E-E'*

Cross-section E-E' trends southeast-northwest across the west side of the quarry just south of section A-A', west (Figure 7; Photo 22). The wedge will extends to 1050 feet and no flat fill is planned to be placed against the wedge fill in this area. The fill properties and rock/weathered rock phi values were fixed. The results are summarized in Table 8.

Table 8
FS values for cross-section E-E' west. Cohesion $_{eqv}$ in psf.

| Analysis Type | | Unweathered rock | | Weathered Rock | | FS | Newmark Disp. |
|---|---|---|---|---|---|---|---|
| | | $P_{eqv}$ | $C_{eqv}$ | $P_{eqv}$ | $C_{eqv}$ | | |
| Static | Full Slope | 32 | 5000 | 28 | 3000 | 1.61 | - |
| Static | Full Slope | 32 | 2000 | 28 | 1000 | 1.37 | - |
| Static | Rock only | 32 | 5000 | 28 | 3000 | 2.33 | - |
| Static | Rock only | 32 | 2000 | 28 | 1000 | 1.65 | - |
| Static | Wedge Fill only | 31 | 0<br>150<br>300 | - | - | 1.29<br>1.33<br>1.40 | - |
| Pseudo-Static | Full Slope | 32 | 5000 | 28 | 3000 | 1.07 | 0.33 ft |
| Pseudo-Static | Full Slope | 32 | 1500 | 28 | 1000 | 0.93 | 0.81 ft |
| Pseudo-Static | Wedge Fill only | 31 | 150<br>300 | - | - | 0.87<br>0.92 | 1.1 ft<br>0.74 ft |

## CONCLUSIONS

Based on the results of our limited field investigation and mapping, review of the reclamation plan, and static and pseudo-static slope stability analyses, it is our opinion that the planned reclamation configuration will result in permanent slopes which will have acceptable stability for their intended use. The slopes stability analyses indicate that using reasonable lower bound strength values for the various rock and soil types, the static factors of safety exceed 1.3 and some are greater than 1.5. Since the strength values used in the analyses are considered to be representative of lower bound strengths, we believe that the demonstrated level of long-term stability is acceptable. If the long-term intended use of the reclaimed site changes from open space use, it may be warranted to perform additional studies relating to in-situ rock and soil strengths to better define as-constructed factors-of-safety.

Until vegetation is established, it is likely that there will be localized surface unraveling of bare rock slopes. The final rock slopes will be shaped several years prior to placement of the wedge fills. This will provide time for vegetation to become established on the rock slopes.

Operational constraints may be needed to reduce wedge failures along the western slope until fill can be placed.

When fill has been placed, bedrock water levels will rise. We have assumed that groundwater levels will remain below potential failure surfaces. This is based upon the current elevation of seeps in the west and northern walls and the fill being drained.

We do not know the width and extent of the shear zone in the northwest corner of the quarry. The western end is covered and we assumed that it is relatively narrow (less than 150 to 200 feet wide). This should be confirmed during mining. If the shear zone is significantly wider, an additional wedge fill cover (~25 feet thick) may have to be placed on top of this zone. A sample of this material was tested (Appendix A).

The perimeter road along the west and north sides of the quarry will be located on in-place residual soils. We recommend that these soils be removed to a depth of 10 to 20 feet and replaced/recompacted to form an engineered fill/embankment. The actual depth of excavation should be determined in the field. The perimeter road should be located on top of this engineered fill.

## LIMITATIONS

These conclusion assume that the material properties of the imported fill that was tested are representative of the fill material that will be placed and that the nature of weathered and unweathered bedrock and the observed orientations of joints and shears on the existing quarry slopes are representative of the actual field conditions on the proposed final cut slopes.

The Public Resources Code (PRC), Title14, Article 9, Section 3704, states that lead regulatory agencies shall require formal slope stability investigations whenever design-slopes approach or exceed *critical gradient.* Critical gradient is defined as the maximum unsupported slope which can be maintained under the most adverse conditions. The term "most adverse conditions" is not a engineering term and it is not defined in the regulations. Our calculations were performed using conservative, reasonable assumptions about adverse natural conditions. The final design slopes are considered not to approach or exceed the critical gradient.

The express purpose of this slope stability investigation is to provide for public safety. The regulations do not require that the final design slopes be brought into compliance with Uniform Building Code (UBC) requirements for engineered slopes.

The analysis, conclusions, and Factors of Safety are not valid for evaluation of working slopes or the final slopes prior to placement of backfill.

The analysis, conclusions, and Factors of Safety determined in this report are based on the final slope geometries with the backfill in place as shown in Sheet 3 of the Resource Design Technology report (2007). If changes are made to the final slope geometry or backfill depths as described in that report, then the conclusions and recommendations presented in this report should be considered invalid by all parties. We should be allowed to review and prepare written responses to comments to this report or to changes in the final slope geometry. If possible, we will prepare modified recommendations after a review of the proposed changes. Additional field and laboratory testing work may be required for us to develop any modifications to our recommendations.

The opinions and/or recommendations presented in this report could be subject to revision should additional information become available. The timing and location of events reported to us by the owners or their representatives were not independently confirmed.

We have employed generally accepted civil engineering and engineering geology procedures. Our observations, professional opinions and conclusions were made using that degree of care and skill ordinarily exercised, under similar conditions, by civil engineers and engineering geologists practicing in this area at this time. Norfleet consultants expressly denies any third party liability arising from the unauthorized use of this report.

Yours Truly,

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Photo 1: Stevens Creek Quarry, looking southeast. Note: all photographs taken by S. Figures in the fall of 2007.

Photo 2: A dirt road northeast of the quarry, looking north. The red line indicates the approximate location of the Berrocal fault northeast of the Quarry. See Figure 6 for the location. Santa Clara units crop out on the far side of the red line. Franciscan limestones crop out in the road on the near side of the red line. Also see photograph 16.


Photo 3: Looking north along a dirt road at the southeast corner of the quarry. See Figure 6 for the location. Santa Clara units (labeled sc, dipping more than 40 degrees to the east) are on the right side of the road. The Berrocal fault trends semi-parallel to and left (west) of the road. Franciscan float (Fg) is on the left side of the road. See Photograph 25 for location.

Photo 4: Hard (less jointed) greenstone in the core of the north face.

Photo 5: The serpentine shear zone at the northwest corner of the quarry, looking north. Note the wedge-like failure. Picture 4 is at the far right side of this photograph. The cliff at the wedge-like failure is about 40 feet high.

Photo 6: Shearing within the serpentine wedge zone seen in the previous photograph.

Photo 7: In-place residual soils at the top of the north face. In this area, the residual soils are more that 40 feet thick.


Photo 8: The west face. Note the change in color marking the irregular boundary between weathered and unweathered greenstone. The slope on the right is side cast fill. The elevations are approximate.

Photo 9: Franciscan limestones at the northeast corner of the quarry. This outcrop has been mined.

Photo 10: The lower circled area marks a sub-horizontal zone of seeps at the south end of the west face during the summer. The upper circled area marks seeps active during the winter. There were no seep to the right of these zones.

Photo 11: Seeps in the core of the north face (arrows).

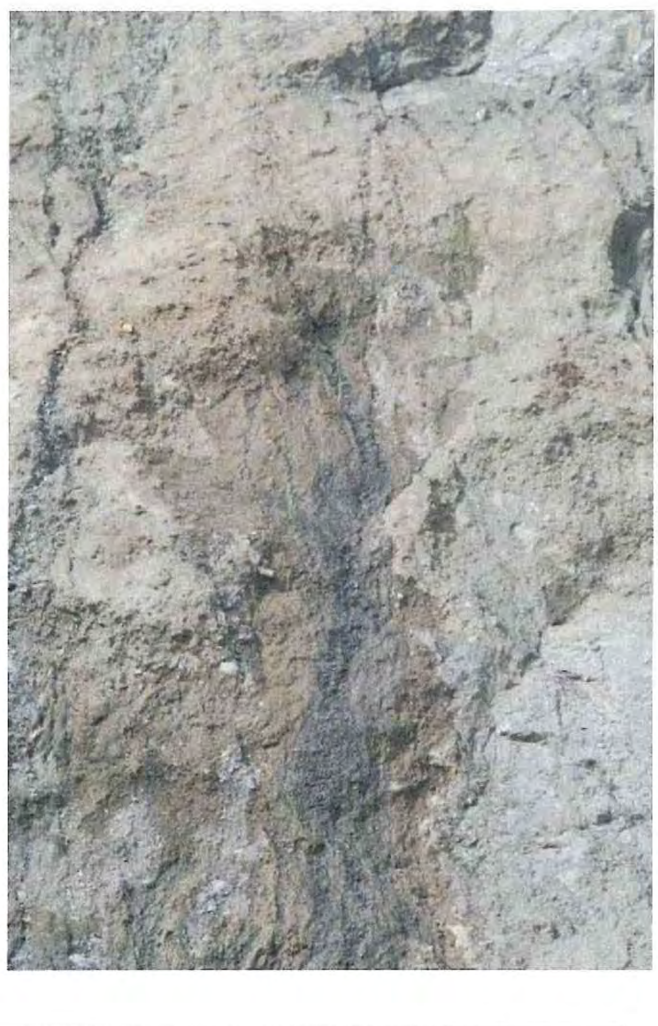


Photo 12: A close-up of the right seep shown in the previous photograph. Note the gray clay seam below the water entry point


Photo 13: The north face. The reddish-brown slopes on either side of the center of the face are side-cast fills.


Photo 14: Wedge failures in the lower part of the western face.

Photo 15: An apparent wedge failure in the shear zone in the northwest corner of the quarry. This is, instead, a spalling failure. A narrow zone along a vertical shear plane began to fail. There was then progressive spalling laterally away from the shear. The cliff at the wedge failure is about 40 feet high.

Photo 16: The east face (weathered greenstone). The arrows indicate landslides where machinery has removed the toe of the slope and caused landslides (see photograph 21). The Berrocal fault is semi-parallel to and just on the other (east) side of the power lines.

Photo 17: The west slope looking south. The arrow indicates the approximate location of the western property line.

Photo 18: Wedge failure in the lower part of the western face.

Photo 19: The upper bench in the west face. Note the localized failures. This face is more than 17 years old.

Photo 20: The failure in the middle of the upper bench. This is a progressive spalling failure rather than a wedge failure. Note the small size of the debris. This face is about 40 feet high.

Photo 21: A landslide in the middle of the eastern face. The left arrow in photograph 16 shows its location. The lower arrows in this photograph mark the slide plane of the landslide that has been exposed by machinery as spoil piles were removed.


Photo 22: The approximate locations of cross-sections W-W' and A-A'-west (red lines).

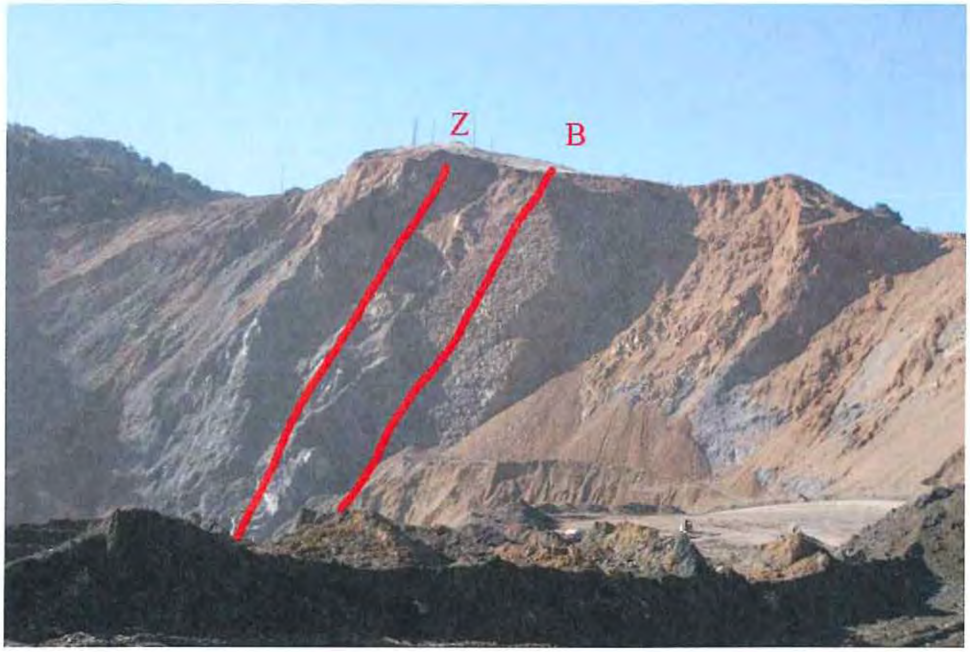
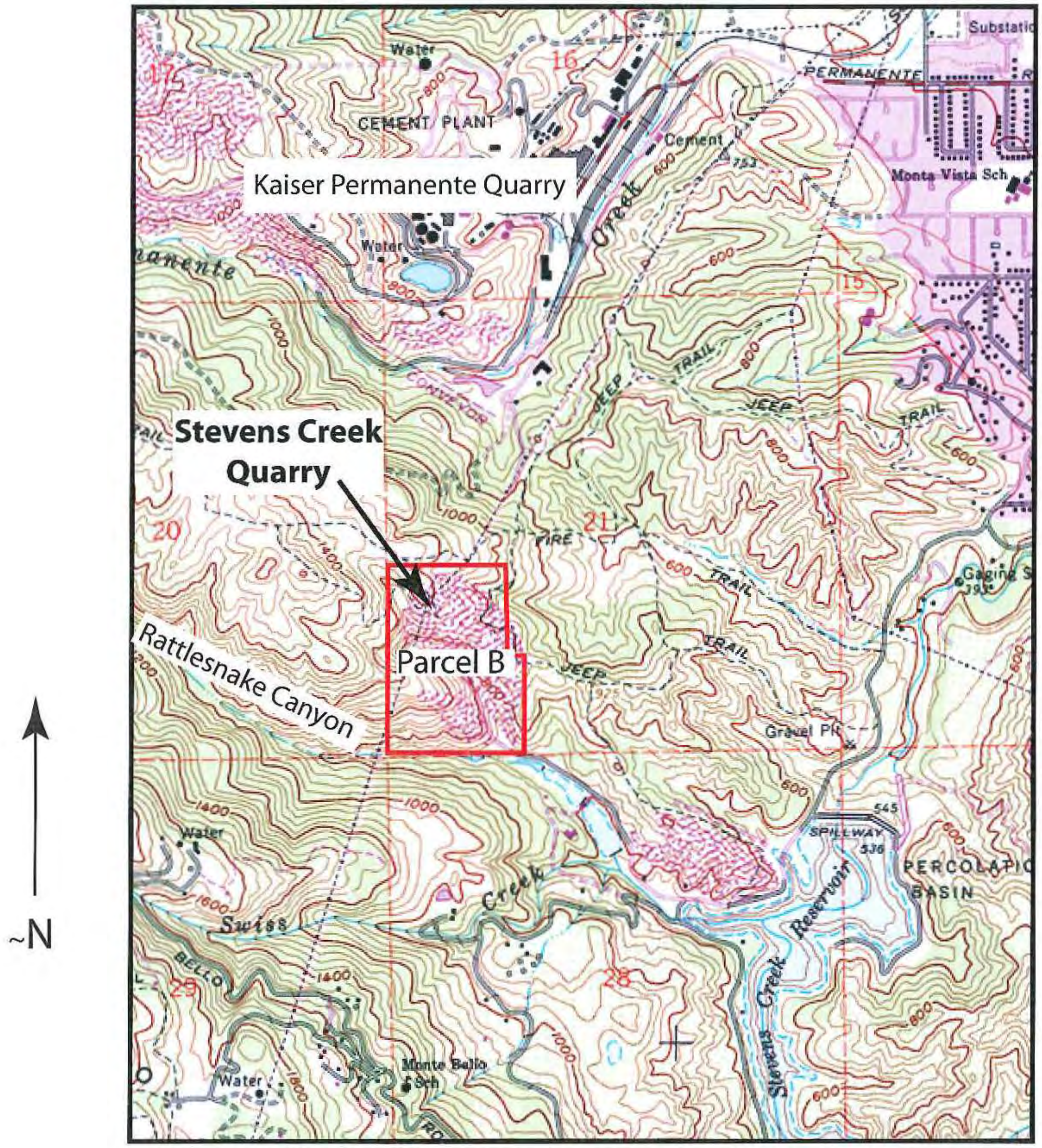



Photo 23: The approximate location of cross-sections Z-Z' and B-B' (red lines).

Photo 24: The approximate location of cross-section A-A'- east (red line).


Photo 25: The approximate location of the Berrocal fault (red line). Looking north across the access road to Parcel B. The arrow indicates the location of Photograph 3 The southeast corner of the quarry is at the upper left side of the photograph.


Photo 26: The red line indicates the approximate location of the Berrocal fault. View is to west on the south side of Rattlesnake canyon. The arrow marks the location of the basal Santa Clara fossil bed mapped by Sorg and McLaughlin (1974).



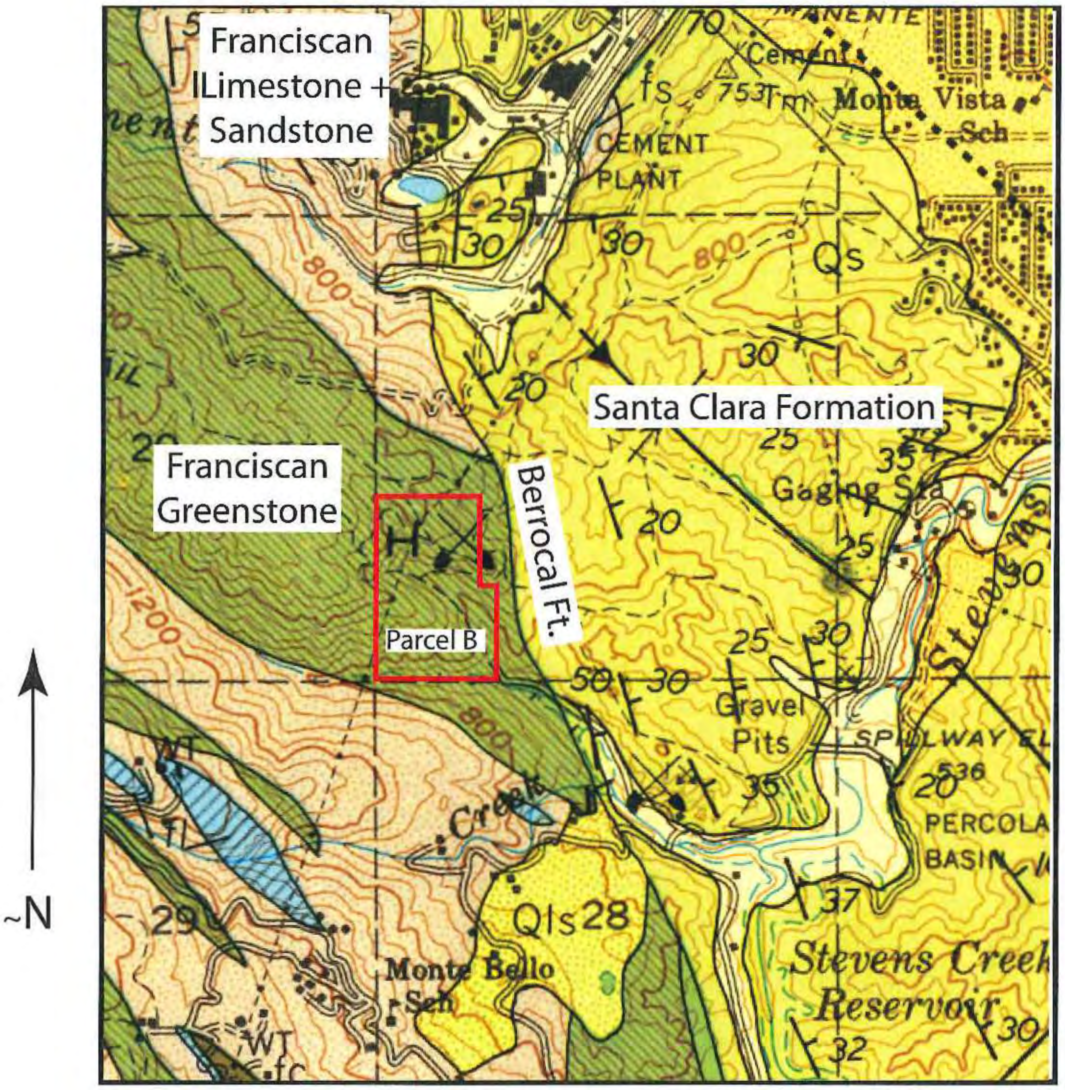

Base is USGS Topographic Map, Cupertino 7-1/2 min. Quad.

The quarry boundary is approximate

The approximate quarry coordinates are 37.2931° and 122.0125°

Red grid on map is 1 mile long.

| Norfleet Consultants | Stevens Creek Quarry, Parcel B | | |
|---|---|---|---|
| | Location Map | | |
| | PROJ NO: 071781 | DATE: Dec. 15, 2007 | FIGURE: 1 |



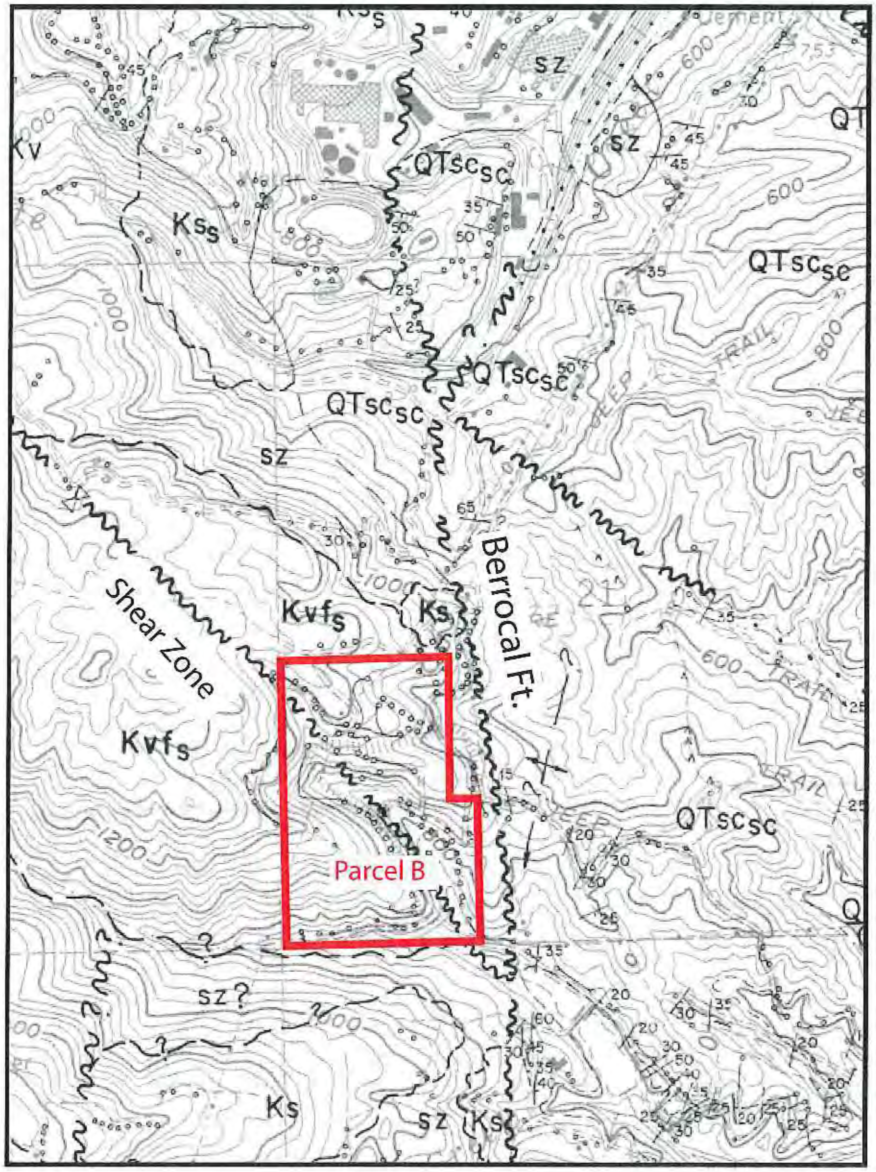

A portion of Dibblee Jr., T.W.; 1966; Geology of the Palo Alto Quadrangle, Santa Clara and San Mateo Counties, California; California Division of Mines and Geology, Map Sheet 8

The quarry boundary is approximate

Black dashed grid on map is 1 mile long.

| | | | |
|---|---|---|---|
| **Norfleet Consultants** | Stevens Creek Quarry, Parcel B | | |
| | Dibblee Map | | |
| | PROJ NO: 071781 | DATE: Dec. 15, 2007 | FIGURE: 2 |

$QTsc_{sc}$ Santa Clara Formation, Stevens Creek Member
$Ks_{s}$ Franciscan Formation, sheared sandstone
Ks Franciscan Formation, sandstone
$Kvf_{s}$ Franciscan Formation, sheared fragmental volcanic rocks
sz Shear zone (melange)




~N

A portion of Plate 1 from Rogers, T. and Armstrong, C.; 1973; Environmental Geologic Analysis of the Monte Bello Ridge Mountain Study Area Santa Clara County, California; California Division of Mines and Geology, Preliminary Report 17

The quarry boundary is approximate
Black dashed grid on map is 1 mile long.

| | | | |
|---|---|---|---|
| **Norfleet Consultants** | Stevens Creek Quarry, Parcel B | | |
| | Rogers and Armstrong Geologic Map | | |
| | PROJ NO: 071781 | DATE: Dec. 15, 2007 | FIGURE: 3 |

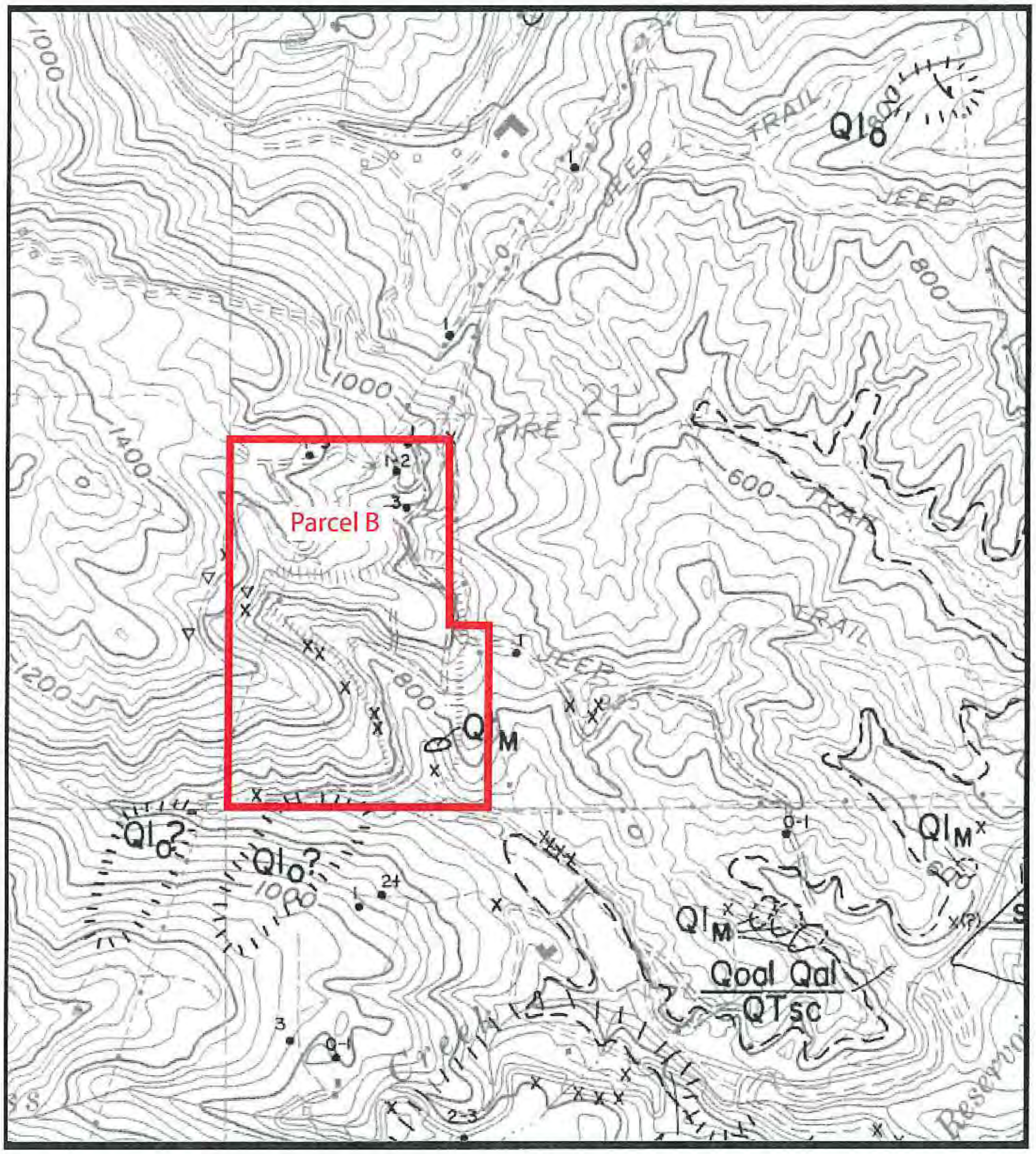
$Ql_o$ Old Landslide

$Ql_m$ Modern Landslide

x Modern Landslide, max. dimension < 100 ft

▽ Colluvium-filled ancient stream channel

●$_{10}$ Thickness of colluvium




~N

A portion of Plate 2 from Rogers, T. and Armstrong, C.; 1973; Environmental Geologic Analysis of the Monte Bello Ridge Mountain Study Area Santa Clara County, California; California Division of Mines and Geology, Preliminary Report 17

The quarry boundary is approximate.

The western quarry property line is about 2700 feet long.

| Norfleet Consultants | Stevens Creek Quarry, Parcel B | | |
|---|---|---|---|
| | Rogers and Armstrong Landslide Map | | |
| | PROJ NO: 071781 | DATE: Dec. 15, 2007 | FIGURE: 4 |

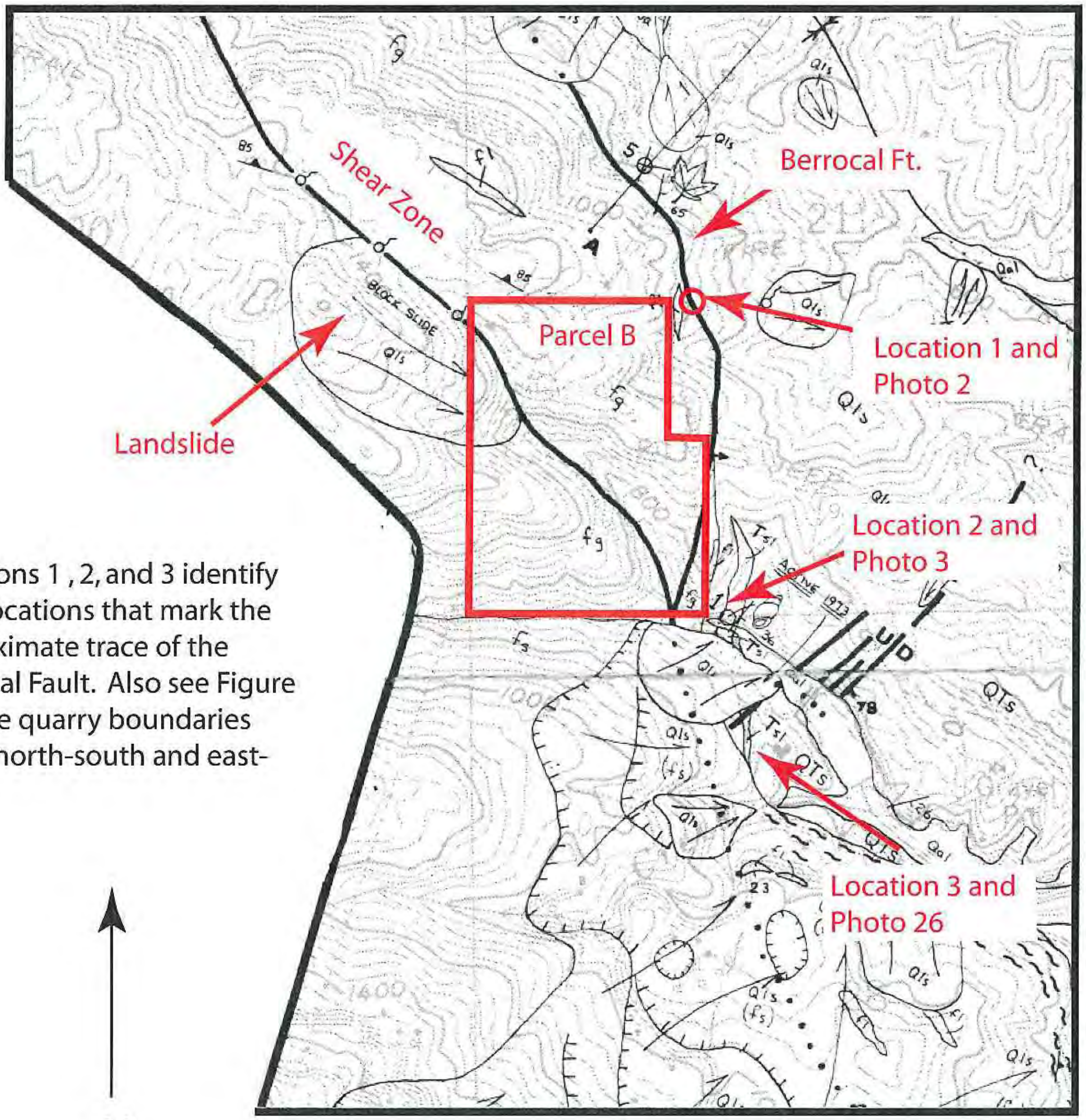
$Ql_s$ Landslide

$QT_s$ Santa Clara Formation

$T_{sl}$ Santa Clara Formation - Lake beds

$f_g$ Franciscan Assemblage - greenstone member

$f_l$ Franciscan Assemblage - Calera limestone member

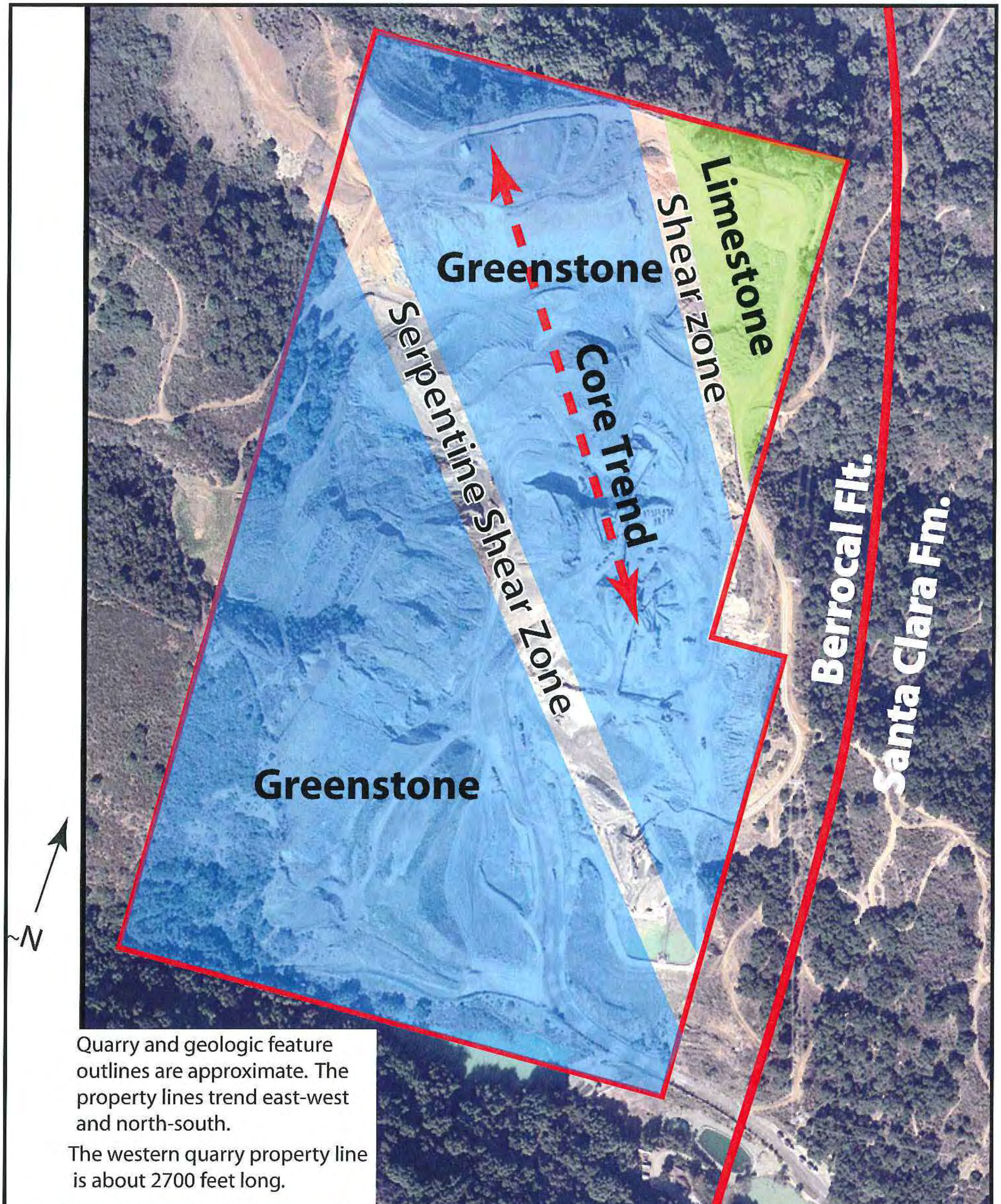



Locations 1 , 2, and 3 identify field locations that mark the approximate trace of the Berrocal Fault. Also see Figure 12. The quarry boundaries trend north-south and east-west.

A portion of Sorg, D. and McLaughlin, R.; 1975; Geologic map of the Sargent-Berrocal Fault Zone between Los Gatos and Los Altos Hills, Santa Clara County, California; USGS MF-643

The western quarry property line is about 2700 feet long. The quarry boundary is approximate.

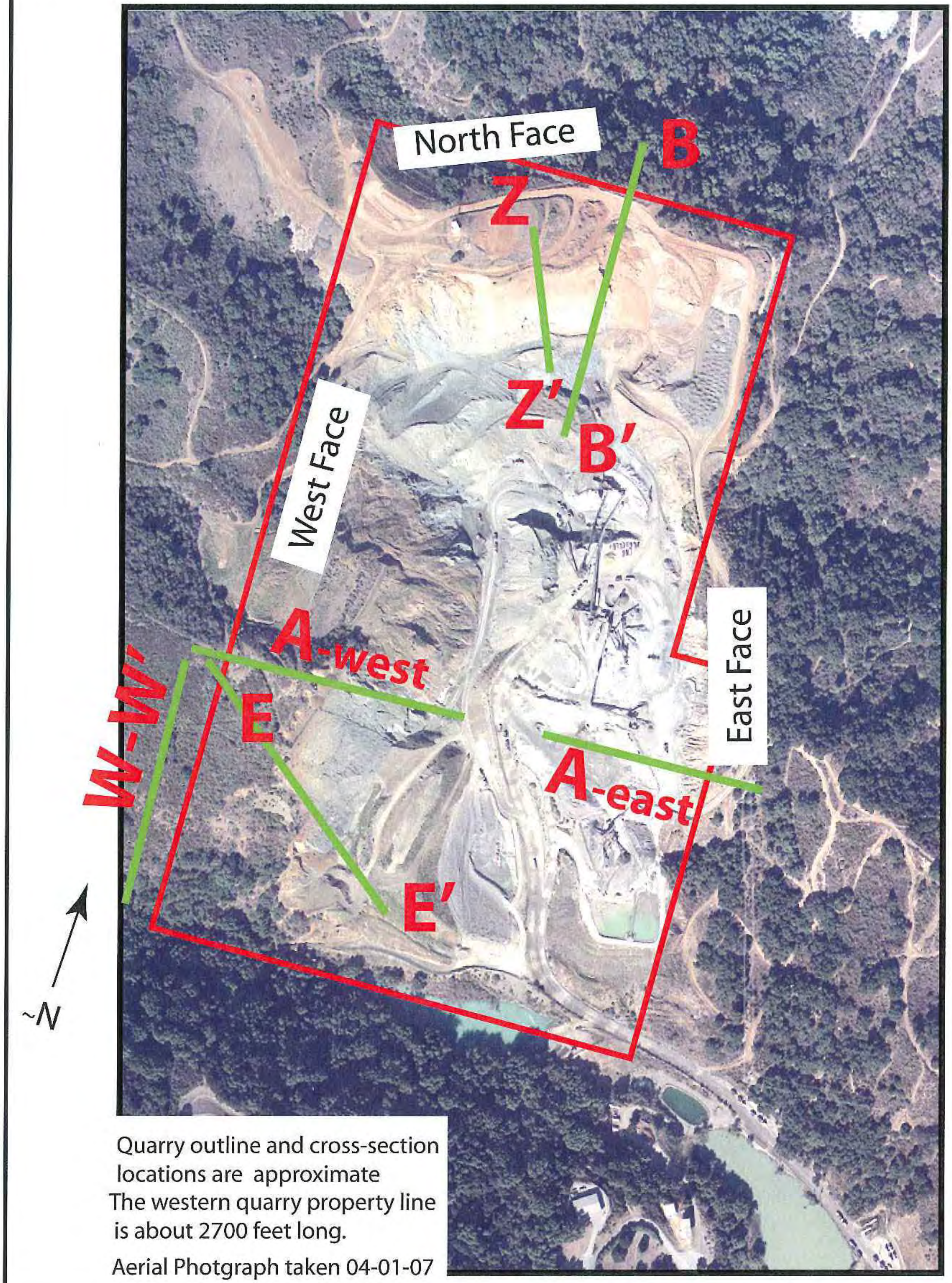
| Norfleet Consultants | Stevens Creek Quarry, Parcel B | | |
|---|---|---|---|
| | Sorg and McLaughlin Geologic Map | | |
| | PROJ NO: 071781 | DATE: Dec. 15, 2007 | FIGURE: 5 |




Quarry and geologic feature outlines are approximate. The property lines trend east-west and north-south.

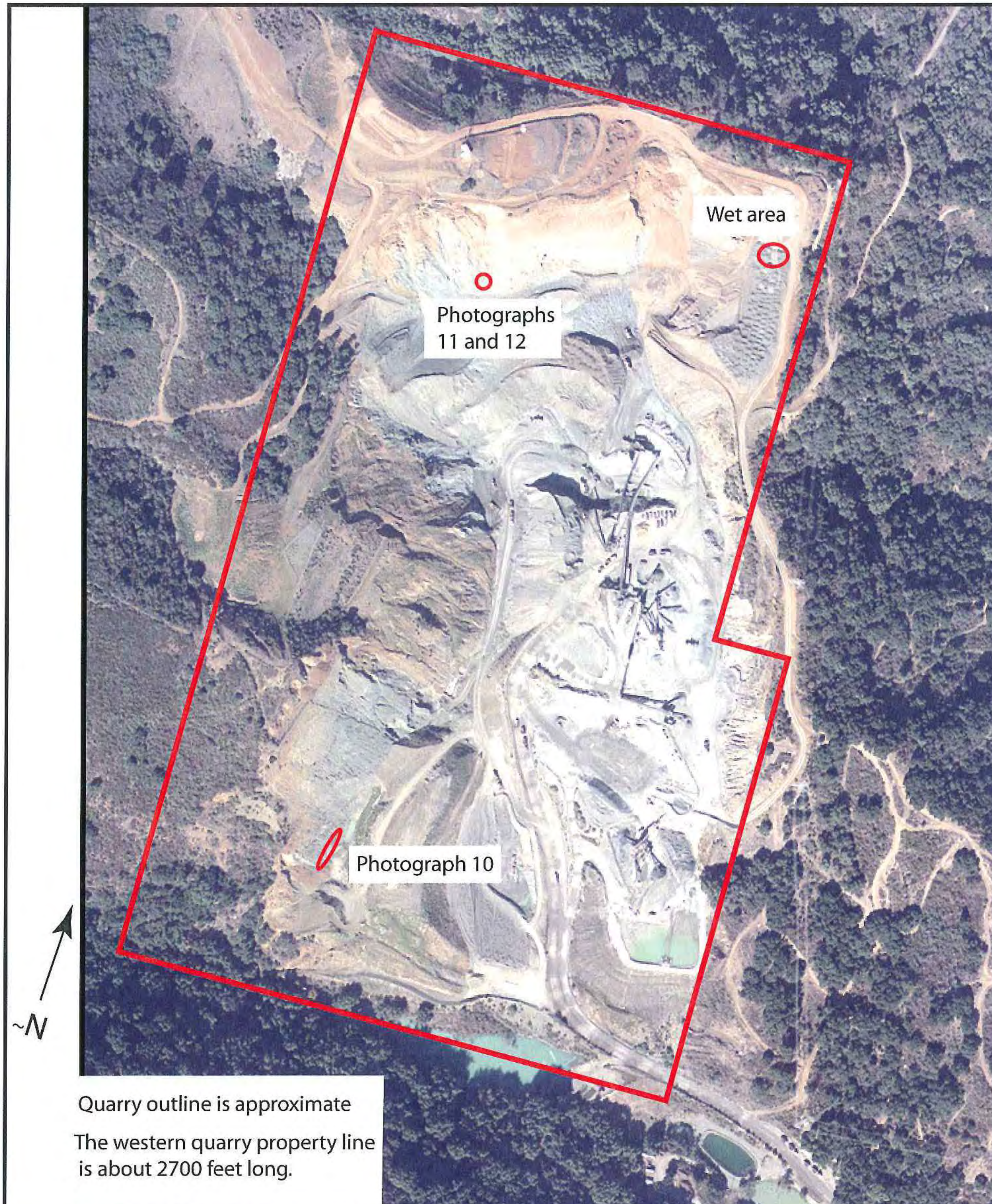
The western quarry property line is about 2700 feet long.

| | | | |
|---|---|---|---|
| **Norfleet Consultants** | Stevens Creek Quarry, Parcel B | | |
| | General Geology | | |
| | PROJ NO: 071781 | DATE: Dec. 15, 2007 | FIGURE: 6 |

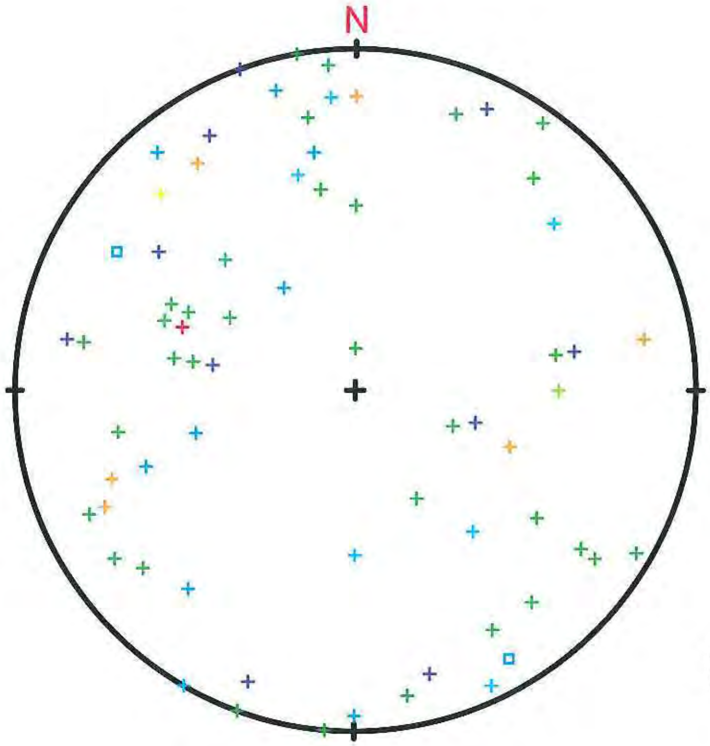



Quarry outline and cross-section locations are approximate
The western quarry property line is about 2700 feet long.
Aerial Photgraph taken 04-01-07

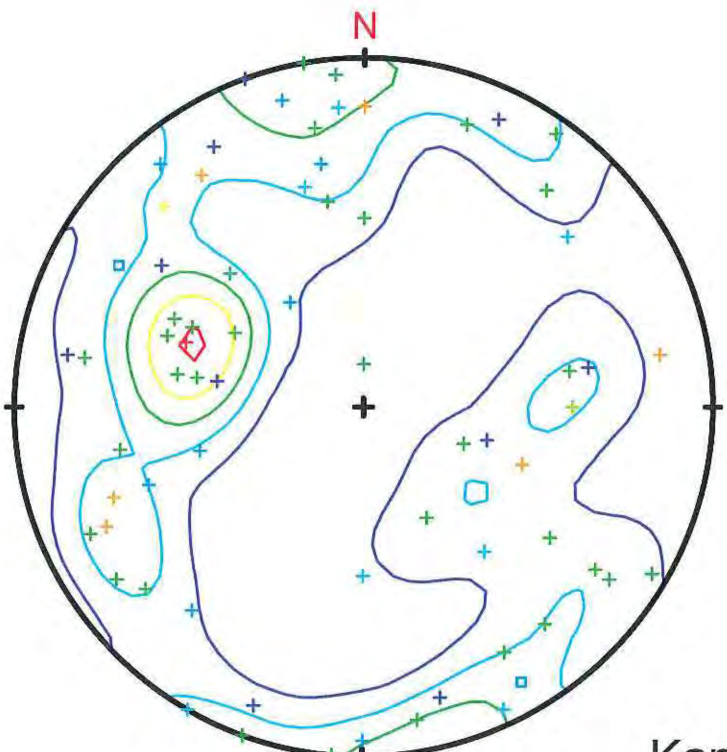
| | |
|---|---|
| **Norfleet Consultants** | Stevens Creek Quarry, Parcel B |
| | Quarry Details |
| | PROJ NO: 071781   DATE: Dec. 15, 2007   FIGURE: 7 |




| | | | |
|---|---|---|---|
| **Norfleet Consultants** | Stevens Creek Quarry, Parcel B | | |
| | Seepage Areas | | |
| | PROJ NO:071781 | DATE: Dec. 15, 2007 | FIGURE: 8 |




Pole Plot

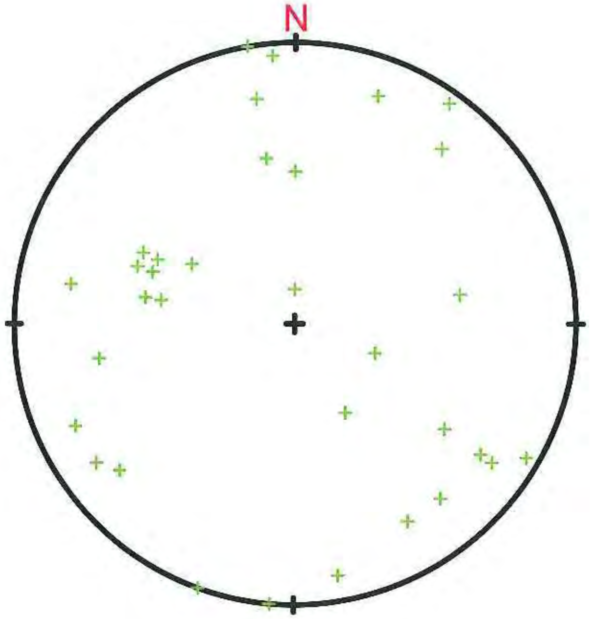



Kamb Contour Plot

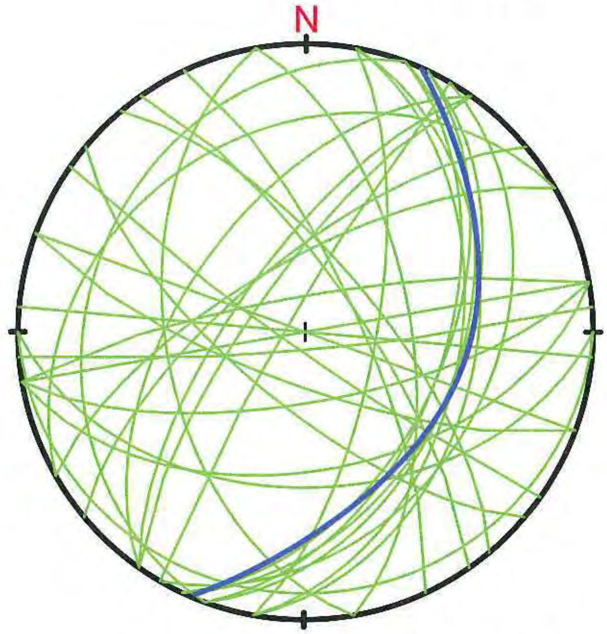
73 Data Points
Equal Area, Lower Hemisphere Plot

| | | | | | |
|---|---|---|---|---|---|
| **Norfleet Consultants** | Stevens Creek Quarry | | | | |
| | Stereonet Plot of all Joint/Shear Data | | | | |
| | PROJ NO: 071781 | DATE: Dec. 15, 2007 | FIGURE: 9 | | |

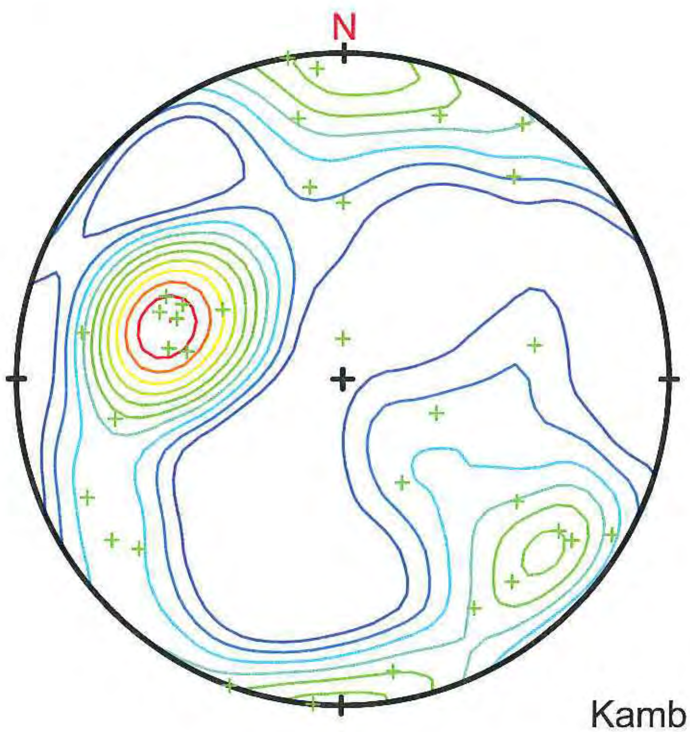
The blue line is the trend of the west face




Pole Plot




Plane Plot




Kamb Contour Plot

35 Data Points
Equal Area, Lower Hemisphere Plot

| | | | | | |
|---|---|---|---|---|---|
| **Norfleet Consultants** | Stevens Creek Quarry | | | | |
| | Stereonet Plot of Joint/Shear Data in West Face | | | | |
| | PROJ NO: | 071781 | DATE: Dec. 15, 2007 | FIGURE: | 10 |

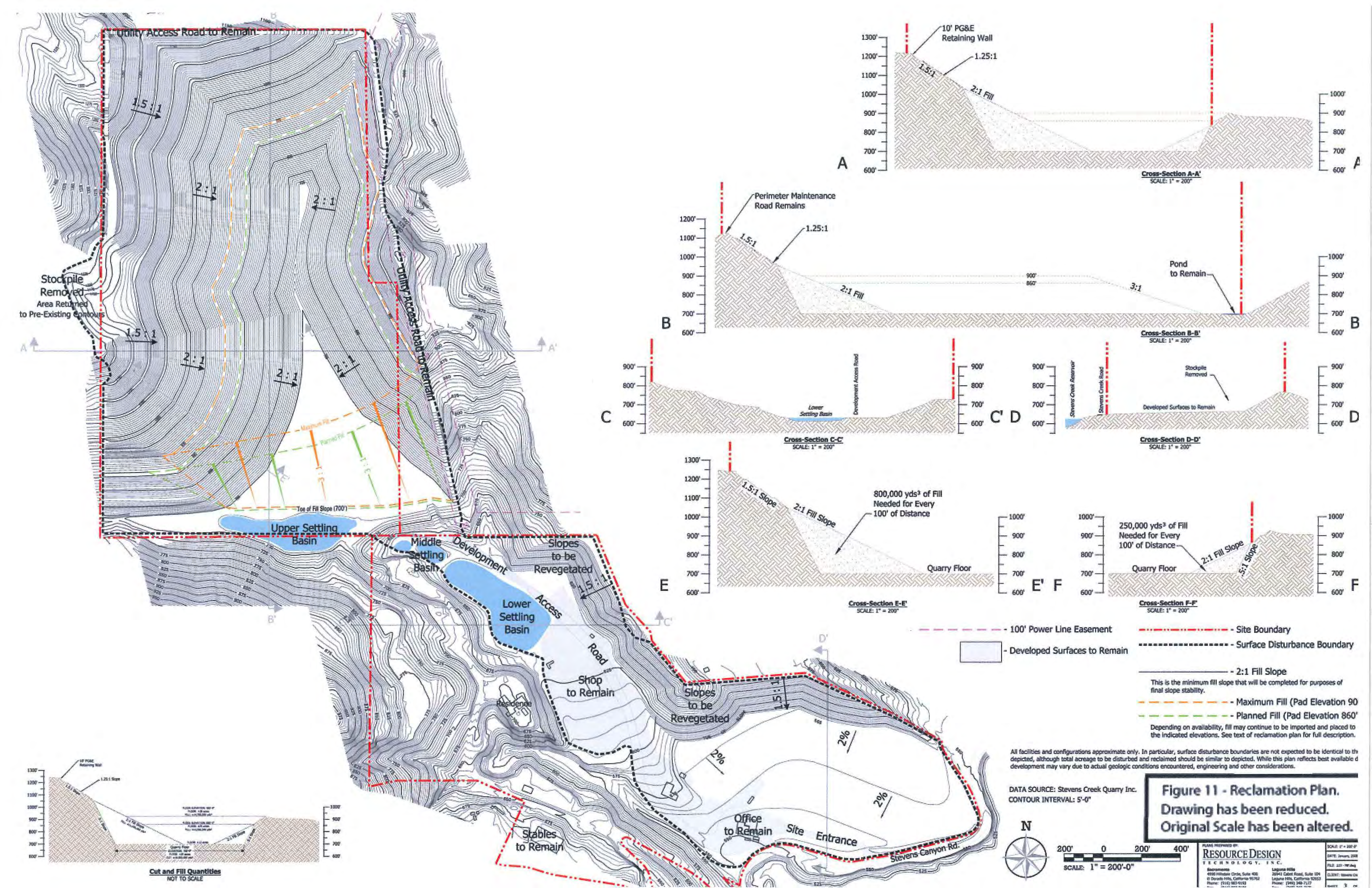
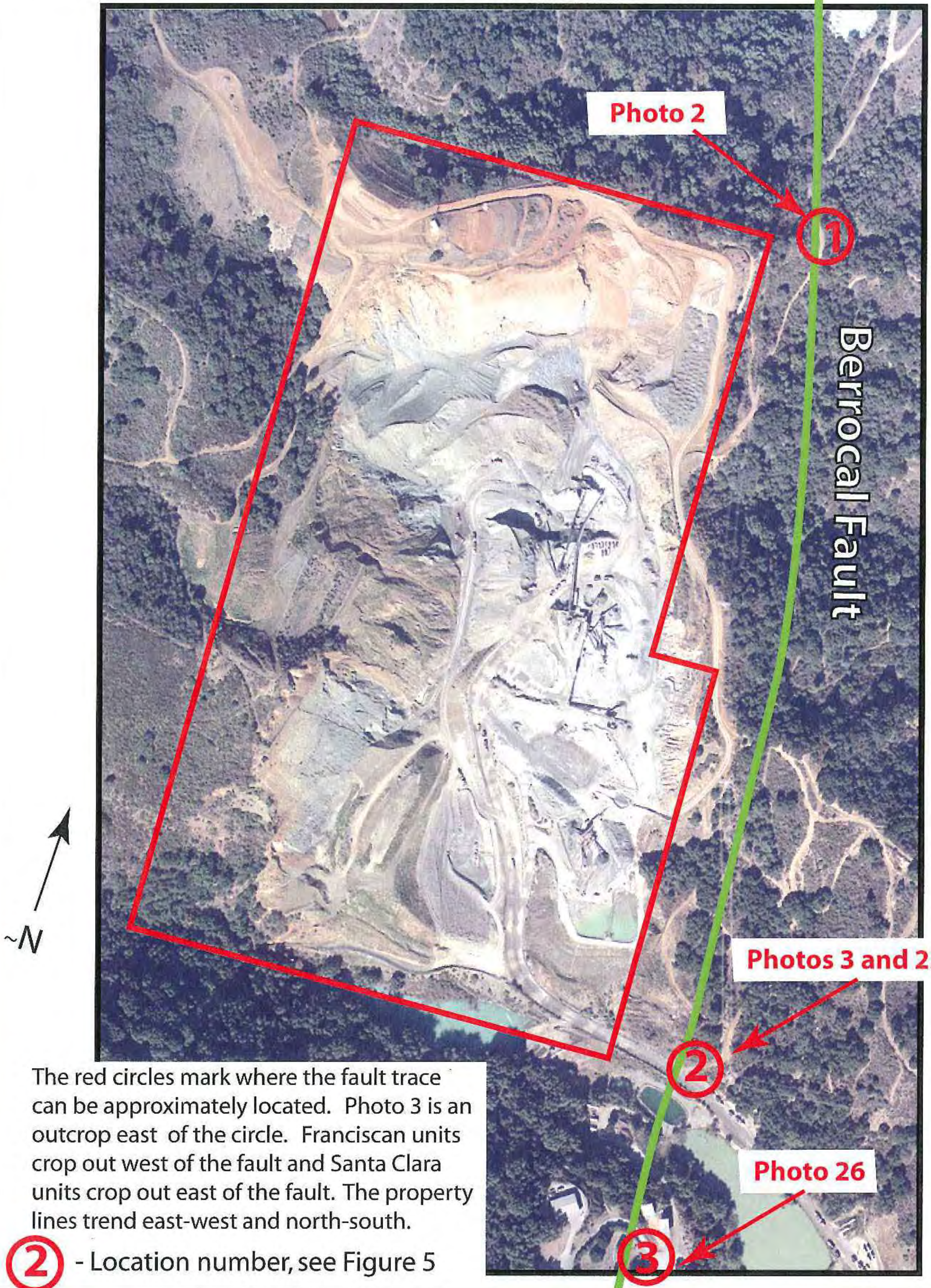



Figure 11 - Reclamation Plan. Drawing has been reduced. Original Scale has been altered.




The red circles mark where the fault trace can be approximately located. Photo 3 is an outcrop east of the circle. Franciscan units crop out west of the fault and Santa Clara units crop out east of the fault. The property lines trend east-west and north-south.

(2) - Location number, see Figure 5

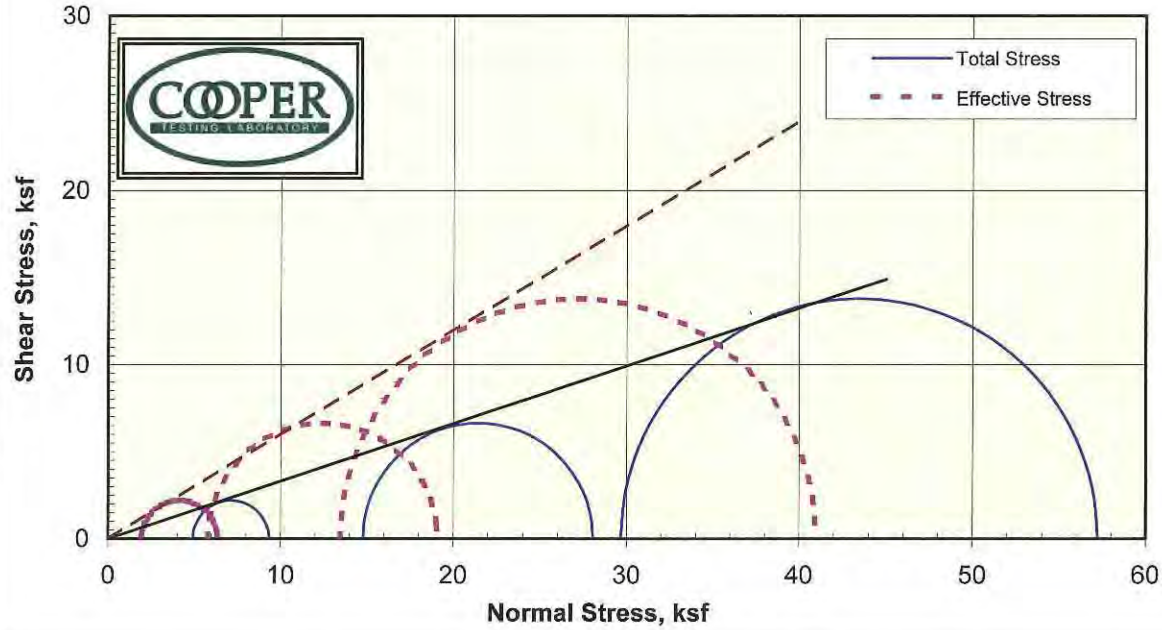
| Norfleet Consultants | Stevens Creek Quarry, Parcel B | | |
|---|---|---|---|
| | Berrocal Fault | | |
| | PROJ NO: 071781 | DATE: Dec. 15, 2007 | FIGURE: 12 |

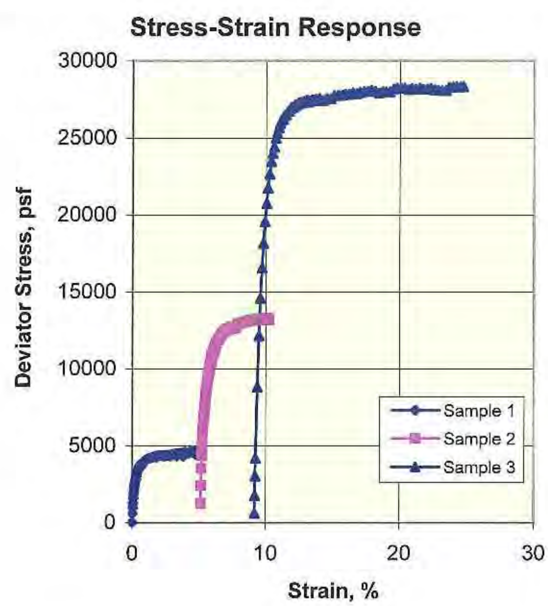
# APPENDIX A

## Soil tests.

Two samples of imported material (Import #1 [Figure 13] and Import #2 [Figure 14]), and one sample of the shear zone material (Shear [Figure 15]) from the northwest corner of the quarry were triaxially tested. All the tests were triaxial consolidated undrained with pore pressure (ASTM D-4767). The tests were modified in that they were staged tests. Import 2 was tested with a 3 inch ring (the sample contained ½ inch sized material). The Import 1 and Shear samples were tested with a 2.5 inch diameter ring (minus ½ inch sized material). The removal of plus ½ inch material means that the material properties are likely lower than the actual values.

The Imported samples were taken from a stock pile just north of the quarry offices. The Shear sample was taken from a fresh rock face at the northwest corner of the quarry (Photo 15). That location was at about elevation 850 feet, about 200 feet below the original ground surface.




| Stage | 1 | 2 | 3 |
|---|---|---|---|
| MC, % | 12.5 | | |
| Dry Dens., pcf. | 115.0 | | |
| Sat. % | 72.5 | | |
| Void Ratio | 0.466 | | |
| Diameter in | 2.38 | | |
| Height, in | 5.00 | | |
| | | Final | |
| MC, % | 15.7 | 13.8 | 12.4 |
| Dry Dens., pcf. | 118.0 | 122.5 | 125.8 |
| Sat. % | 100.0 | 100.0 | 100.0 |
| Void Ratio | 0.425 | 0.372 | 0.336 |
| Diameter, in | 2.36 | 2.36 | 2.37 |
| Height, in | 4.94 | 4.76 | 4.58 |
| Cell, psi | 94.7 | 164.2 | 268.3 |
| BP, psi | 60.6 | 61.6 | 62.2 |
| | | Effective Stresses At: | |
| Strain, % | 5.0 | 5.0 | 5.0 |
| Deviator ksf | 4.434 | 13.292 | 27.575 |
| Excess PP | 2.993 | 8.978 | 16.238 |
| Sigma 1 | 6.355 | 19.086 | 41.013 |
| Sigma 3 | 1.921 | 5.794 | 13.438 |
| P, ksf | 4.138 | 12.440 | 27.225 |
| Q, ksf | 2.217 | 6.646 | 13.787 |
| Stress Ratio | 3.308 | 3.294 | 3.052 |
| Rate in/min | 0.001 | 0.001 | 0.001 |

| | | | |
|---|---|---|---|
| Total C | 0 | Effective C | 0.0 |
| Total Phi | 18.3 | Effective Phi | 31.0 |

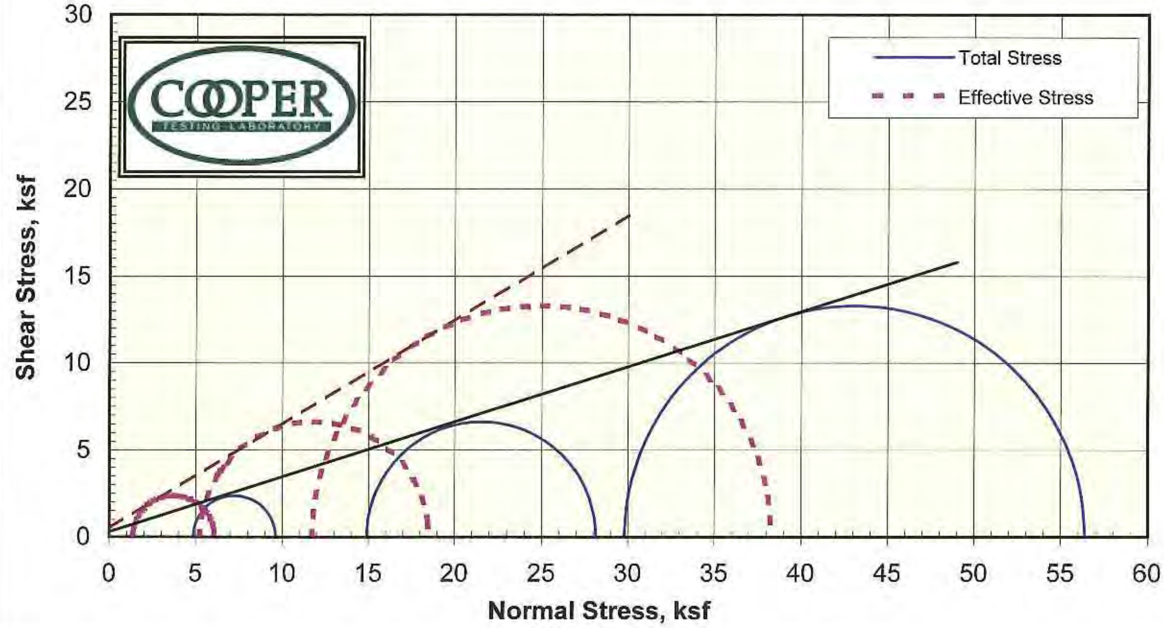
**Job No.:** 471-021 **Date:** 1/10/2008

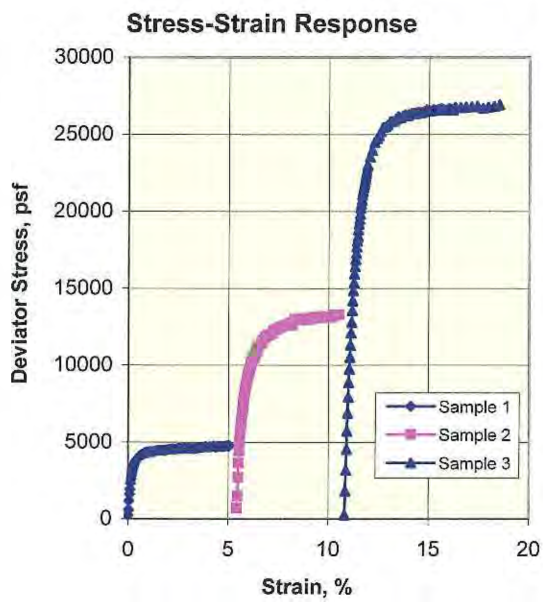
**Client:** Cal Engineering & Geology BY:DC

**Project:** Stevens Creek Quarry - 071210

**Sample:** Import #1 Brown Sandy CLAY w/ Gravel

**Remarks:** ** Staged Test ** Strengths at 5% strain.
Remolding Target: 90% of 128.1 @ 12.5 (OP +2). This sample was taken from the processed imported material stock pile just north of the Qarry office (Parcel A).




| Stage | 1 | 2 | 3 |
|---|---|---|---|
| MC, % | 12.3 | | |
| Dry Dens., pcf. | 119.0 | | |
| Sat. % | 79.9 | | |
| Void Ratio | 0.416 | | |
| Diameter in | 2.88 | | |
| Height, in | 6.00 | | |
| | Final | | |
| MC, % | 14.8 | 13.1 | 11.8 |
| Dry Dens., pcf. | 123.1 | 127.2 | 130.8 |
| Sat. % | 100.0 | 100.0 | 100.0 |
| Void Ratio | 0.399 | 0.354 | 0.318 |
| Diameter, in | 2.85 | 2.86 | 2.89 |
| Height, in | 5.91 | 5.69 | 5.41 |
| Cell, psi | 94.7 | 164.2 | 268.3 |
| BP, psi | 60.9 | 60.7 | 61.3 |
| | Effective Stresses At: | | |
| Strain, % | 5.0 | 5.0 | 5.0 |
| Deviator ksf | 4.755 | 13.255 | 26.585 |
| Excess PP | 3.519 | 9.664 | 18.081 |
| Sigma 1 | 6.109 | 18.496 | 38.310 |
| Sigma 3 | 1.354 | 5.241 | 11.725 |
| P, ksf | 3.731 | 11.868 | 25.017 |
| Q, ksf | 2.378 | 6.627 | 13.292 |
| Stress Ratio | 4.513 | 3.529 | 3.267 |
| Rate in/min | 0.001 | 0.001 | 0.001 |

| | | | |
|---|---|---|---|
| Total C | 0.3 | Effective C | 0.5 |
| Total Phi | 17.6 | Effective Phi | 31.0 |

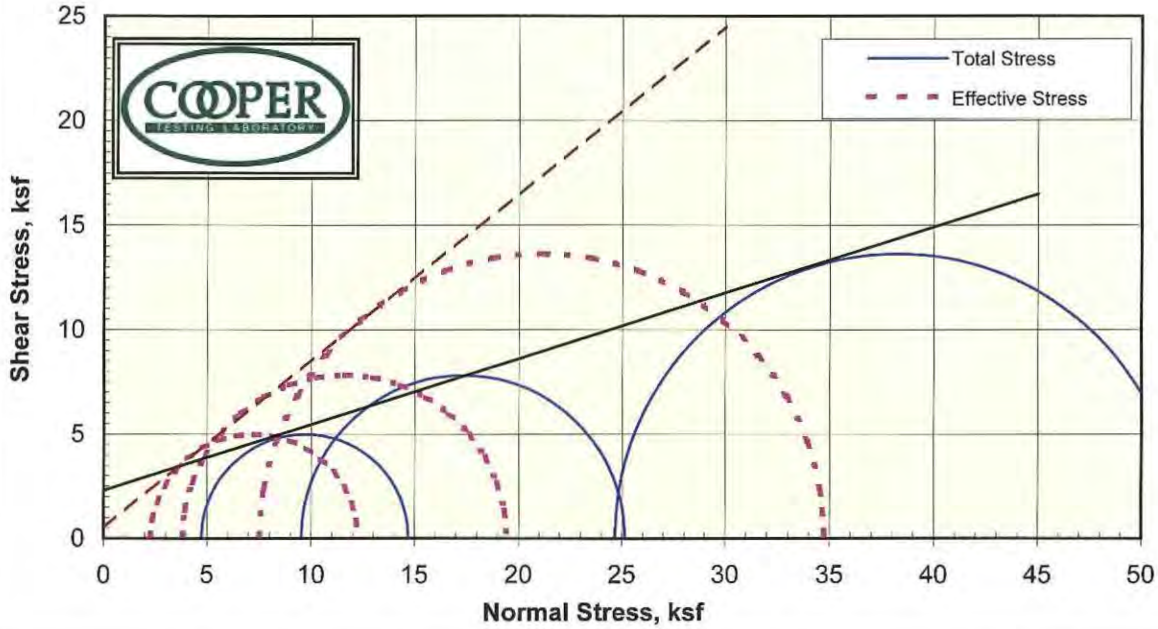
**Job No.:** 471-021 **Date:** 1/10/2008

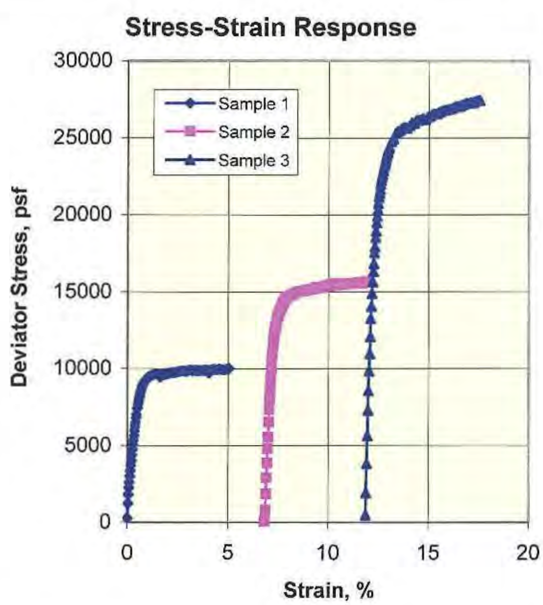
**Client:** Cal Engineering & Geology BY:DC

**Project:** Stevens Creek Quarry - 071210

**Sample:** Import #2 Brown Clayey SAND w/ Gravel

**Remarks:** ** Staged Test ** Strengths at 5% strain.
Remolding Target: 90% of 132 @ 12.3 (OPT +2). This sample was taken from the processed imported material stock pile just north of the Qarry office (Parcel A).




| Stage | 1 | 2 | 3 |
|---|---|---|---|
| MC, % | 10.3 | | |
| Dry Dens., pcf. | 125.1 | | |
| Sat. % | 80.1 | | |
| Void Ratio | 0.347 | | |
| Diameter in | 2.38 | | |
| Height, in | 5.00 | | |
| | Final | | |
| MC, % | 13.6 | 13.3 | 12.0 |
| Dry Dens., pcf. | 126.1 | 127.1 | 130.3 |
| Sat. % | 100.0 | 100.0 | 100.0 |
| Void Ratio | 0.368 | 0.358 | 0.325 |
| Diameter, in | 2.38 | 2.44 | 2.46 |
| Height, in | 4.96 | 4.68 | 4.47 |
| Cell, psi | 114.7 | 149.4 | 253.6 |
| BP, psi | 81.8 | 83.2 | 82.4 |
| | Effective Stresses At: | | |
| Strain, % | 5.0 | 5.0 | 5.0 |
| Deviator ksf | 9.977 | 15.619 | 27.258 |
| Excess PP | 2.463 | 5.719 | 17.152 |
| Sigma 1 | 12.246 | 19.436 | 34.754 |
| Sigma 3 | 2.270 | 3.817 | 7.496 |
| P, ksf | 7.258 | 11.627 | 21.125 |
| Q, ksf | 4.988 | 7.810 | 13.629 |
| Stress Ratio | 5.396 | 5.092 | 4.636 |
| Rate in/min | 0.001 | 0.001 | 0.001 |

| | | | |
|---|---|---|---|
| Total C | 2.3 | Effective C | 0.5 |
| Total Phi | 17.5 | Effective Phi | 38.7 |

**Job No.:** 471-021 **Date:** 1/10/2008

**Client:** Cal Engineering & Geology BY:DC

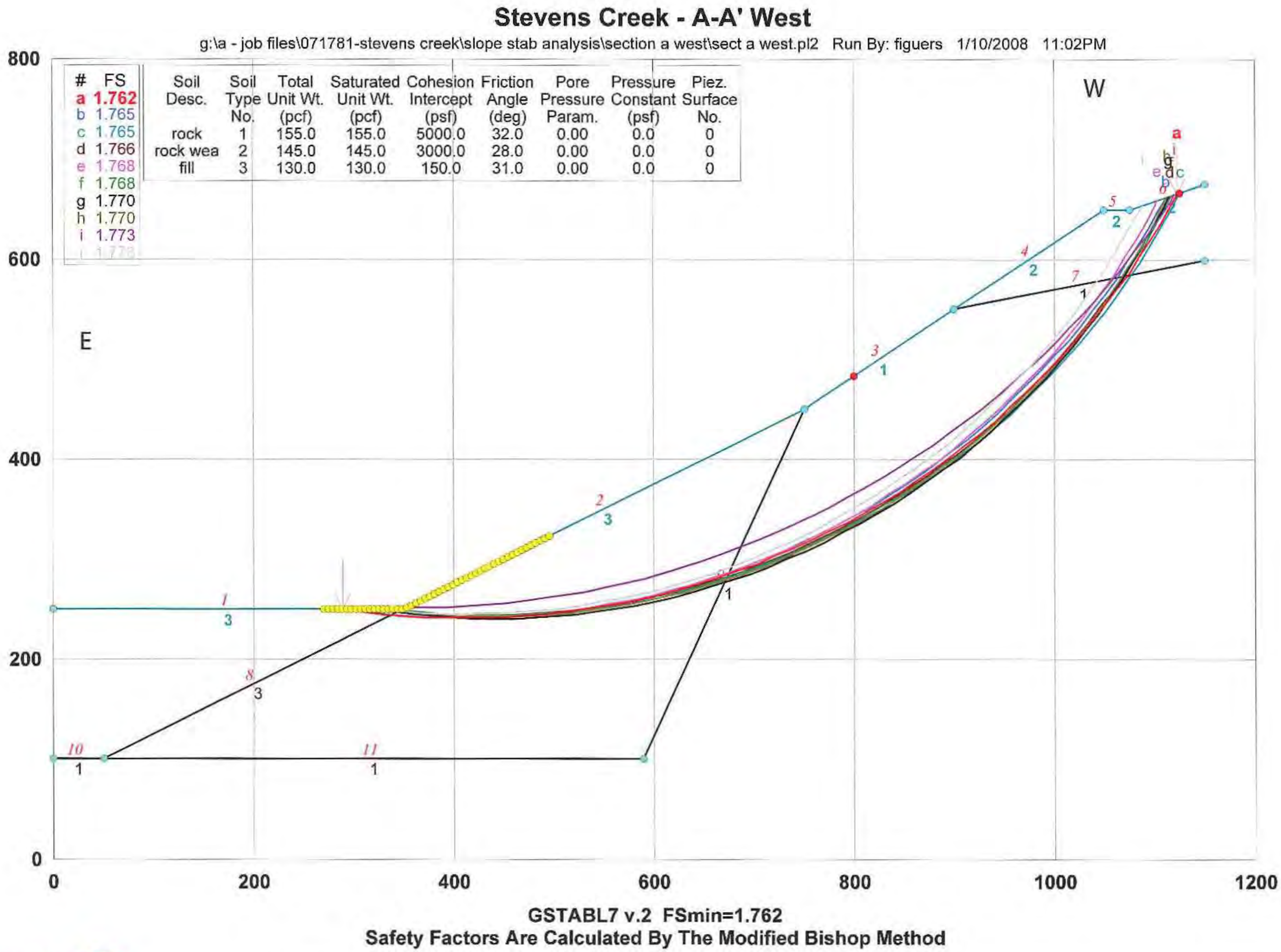
**Project:** Stevens Creek Quarry - 071210

**Sample:** Shear Gray GRAVEL w/ Clay & Sand

**Remarks:** ** Staged Test ** Strengths at 5% strain.
Remolding Target: 90% of 139.3 pcf @ 10.3 (OPT+2). This sample was taken from the shear zone at the NW corner of the Quarry (at elev ~850 ft).

# APPENDIX B

Representative copies of slope stability diagrams are included in this appendix.




| Soil Desc. | Soil Type No. | Total Unit Wt. (pcf) | Saturated Unit Wt. (pcf) | Cohesion Intercept (psf) | Friction Angle (deg) | Pore Pressure Param. | Pressure Constant (psf) | Piez. Surface No. |
|---|---|---|---|---|---|---|---|---|
| rock | 1 | 155.0 | 155.0 | 5000.0 | 32.0 | 0.00 | 0.0 | 0 |
| rock wea | 2 | 145.0 | 145.0 | 3000.0 | 28.0 | 0.00 | 0.0 | 0 |
| fill | 3 | 130.0 | 130.0 | 150.0 | 31.0 | 0.00 | 0.0 | 0 |

GSTABL7

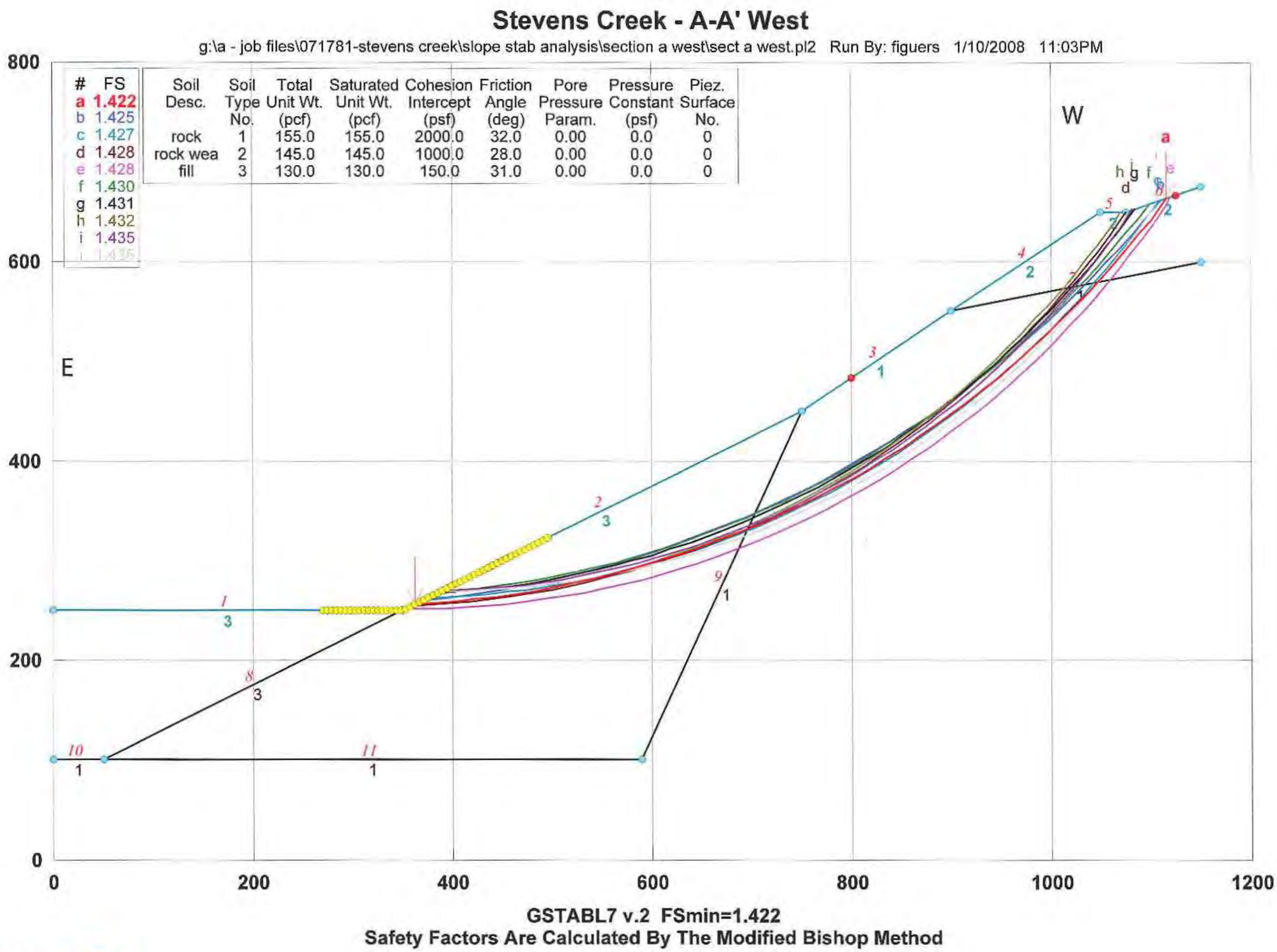
Add 600 ft to vertical scale for actual elevations.

**Norfleet Consultants**

Limit Equlibrum Analysis - Stevens Creek Quarry

Static analysis, High rk strength, Full failure only, Cross-sect A-A -west'

| PROJ NO: | 071781 | DATE: | Jan 10, 2008 | FIGURE: | 16 |
|---|---|---|---|---|---|



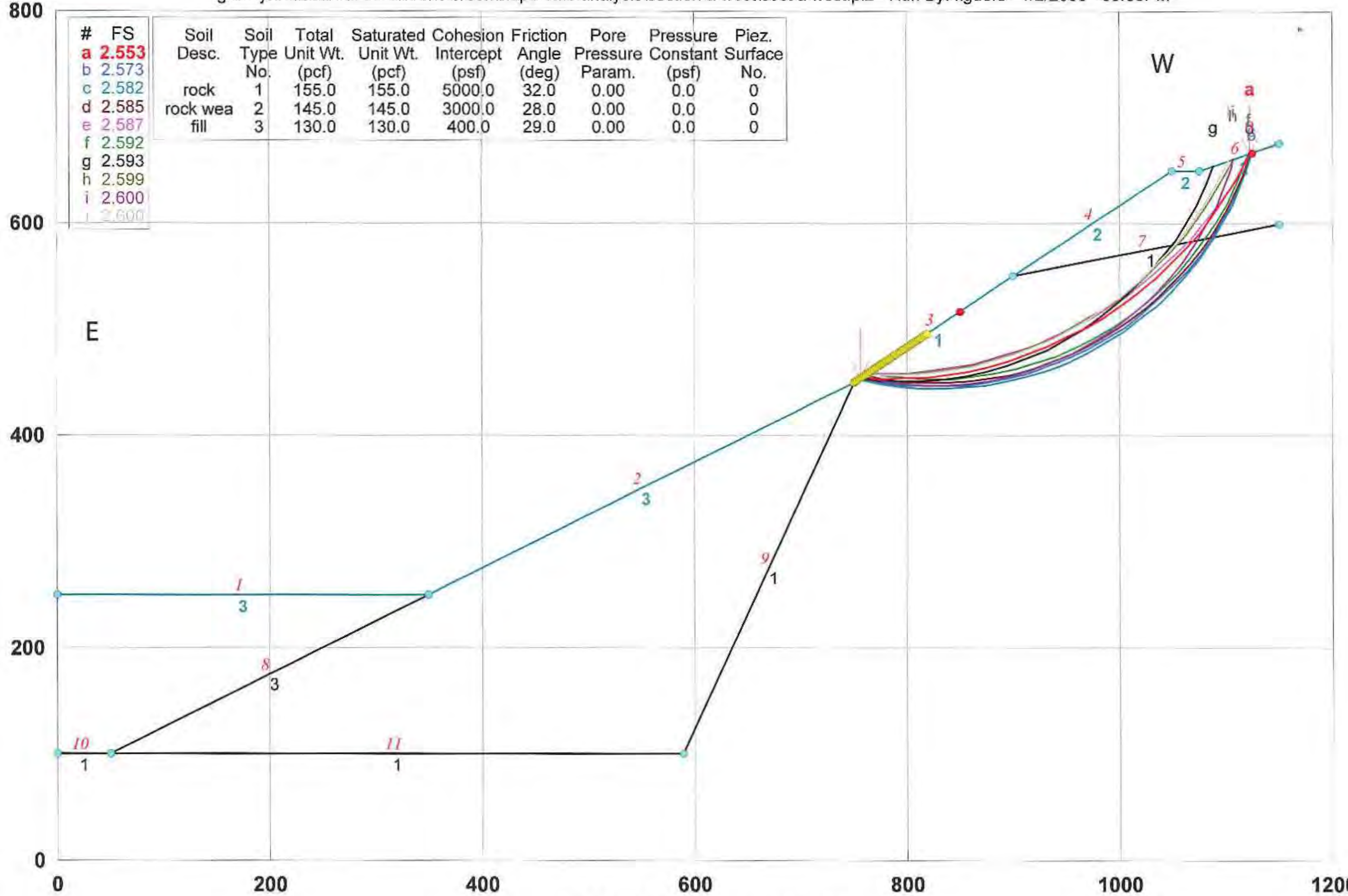

GSTABL7

Add 600 ft to vertical scale for actual elevations.

| | | | | | |
|---|---|---|---|---|---|
| **Norfleet Consultants** | Limit Equlibrum Analysis - Stevens Creek Quarry | | | | |
| | Static analysis, Low rk strength, Full failure only, Cross-sect A-A -west' | | | | |
| | PROJ NO: 071781 | DATE: | Jan 10, 2008 | FIGURE: | 17 |

# Stevens Creek sect A -rock slope

g:\a - job files\071781-stevens creek\slope stab analysis\section a west\sect a west.pl2 Run By: figuers 1/2/2008 09:05PM




| Soil Desc. | Soil Type No. | Total Unit Wt. (pcf) | Saturated Unit Wt. (pcf) | Cohesion Intercept (psf) | Friction Angle (deg) | Pore Pressure Param. | Pressure Constant (psf) | Piez. Surface No. |
|---|---|---|---|---|---|---|---|---|
| rock | 1 | 155.0 | 155.0 | 5000.0 | 32.0 | 0.00 | 0.0 | 0 |
| rock wea | 2 | 145.0 | 145.0 | 3000.0 | 28.0 | 0.00 | 0.0 | 0 |
| fill | 3 | 130.0 | 130.0 | 400.0 | 29.0 | 0.00 | 0.0 | 0 |

**GSTABL7 v.2 FSmin=2.553**
**Safety Factors Are Calculated By The Modified Bishop Method**

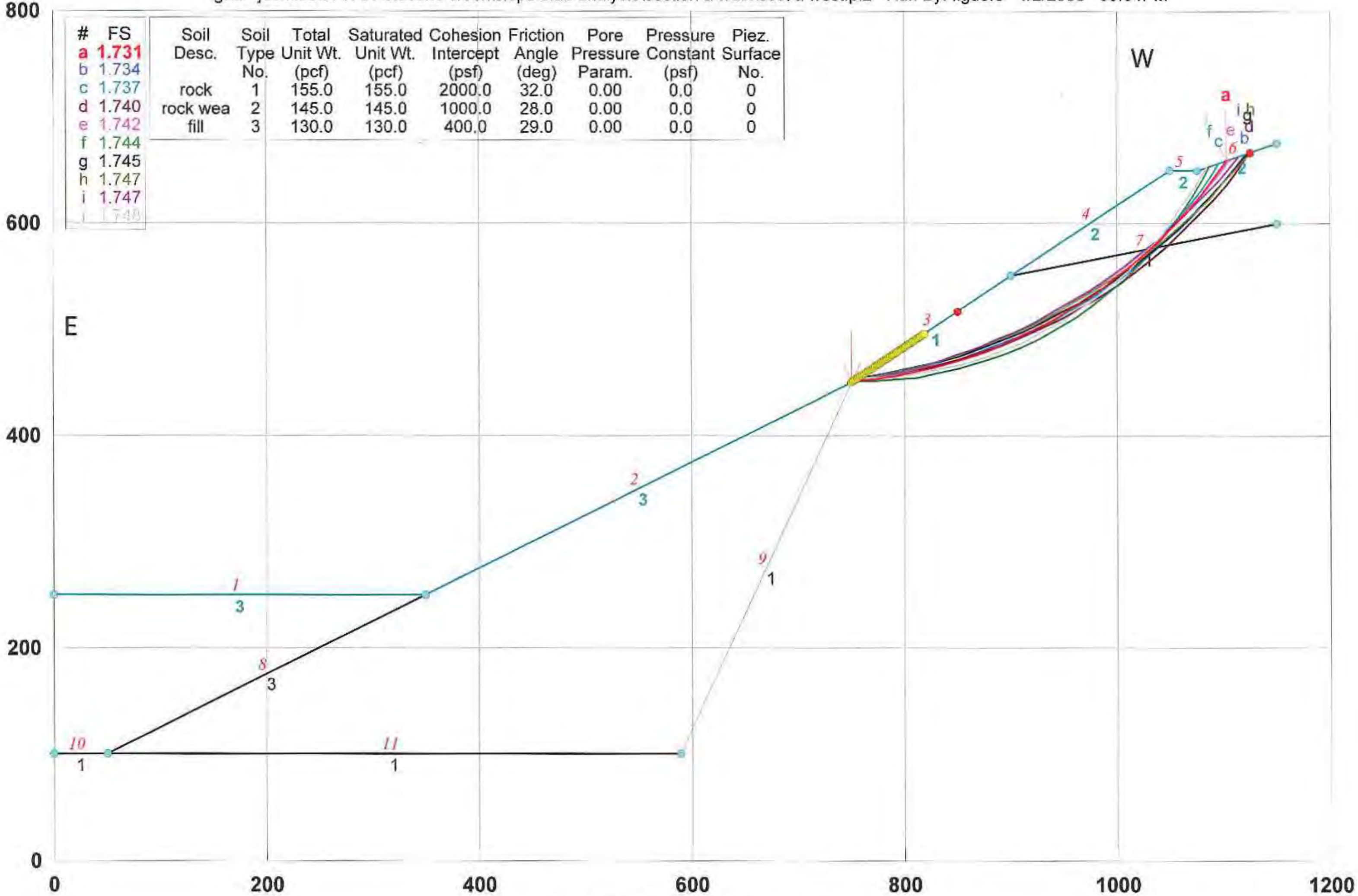
GSTABL7

Add 600 ft to vertical scale for actual elevations.

| | | | | | |
|---|---|---|---|---|---|
| **Norfleet Consultants** | Limit Equlibrum Analysis - Stevens Creek Quarry | | | | |
| | Static analysis, High rk strength, Rock failure only, Cross-sect A-A -west' | | | | |
| | PROJ NO: 071781 | DATE: | Jan 10, 2008 | FIGURE: | 18 |

## Stevens Creek sect A -rock slope

g:\a - job files\071781-stevens creek\slope stab analysis\section a west\sect a west.pl2 Run By: figuers 1/2/2008 09:04PM




| Soil Desc. | Soil Type No. | Total Unit Wt. (pcf) | Saturated Unit Wt. (pcf) | Cohesion Intercept (psf) | Friction Angle (deg) | Pore Pressure Param. | Pressure Constant (psf) | Piez. Surface No. |
|---|---|---|---|---|---|---|---|---|
| rock | 1 | 155.0 | 155.0 | 2000.0 | 32.0 | 0.00 | 0.0 | 0 |
| rock wea | 2 | 145.0 | 145.0 | 1000.0 | 28.0 | 0.00 | 0.0 | 0 |
| fill | 3 | 130.0 | 130.0 | 400.0 | 29.0 | 0.00 | 0.0 | 0 |

**GSTABL7 v.2 FSmin=1.731**
**Safety Factors Are Calculated By The Modified Bishop Method**

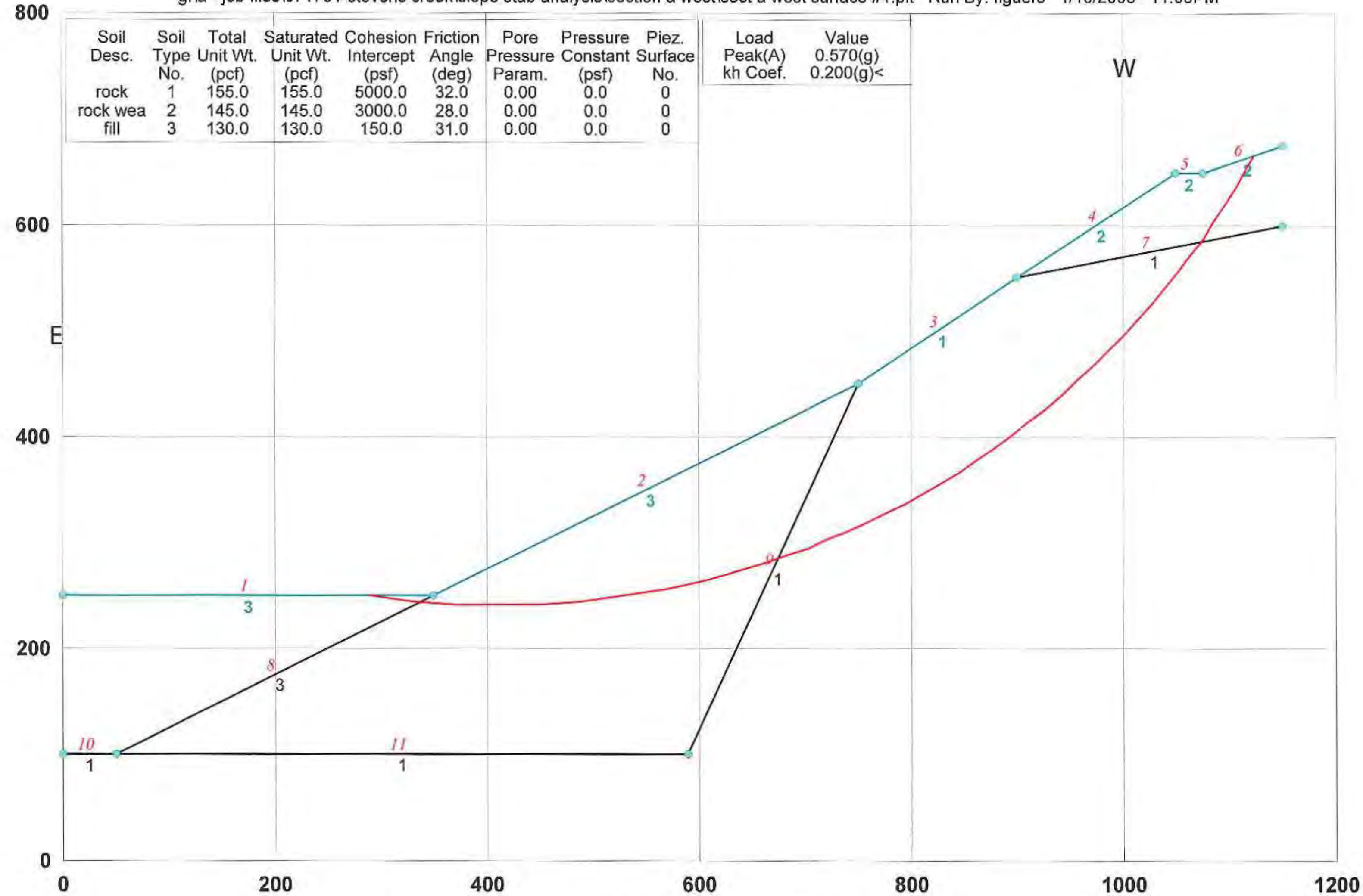
GSTABL7

Add 600 ft to vertical scale for actual elevations.

| | | | | | |
|---|---|---|---|---|---|
| **Norfleet Consultants** | Limit Equlibrum Analysis - Stevens Creek Quarry | | | | |
| | Static analysis, Low rk strength, Rock failure only, Cross-sect A-A -west' | | | | |
| | PROJ NO: 071781 | DATE: | Jan 10, 2008 | FIGURE: | 19 |

**Stevens Creek - A-A' West**

g:\a - job files\071781-stevens creek\slope stab analysis\section a west\sect a west surface #1.plt Run By: figuers 1/10/2008 11:06PM

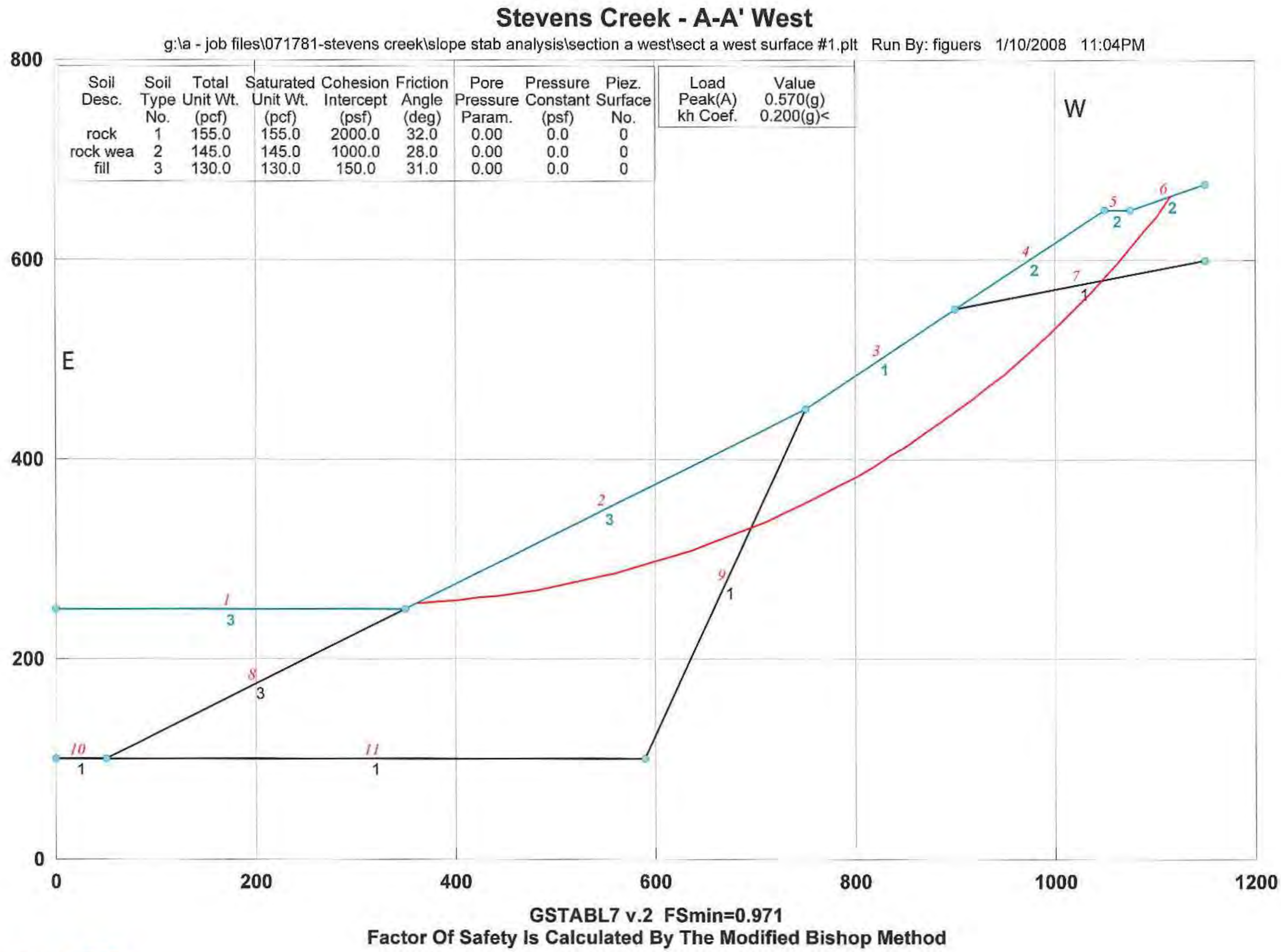



**GSTABL7 v.2 FSmin=1.219**

**Factor Of Safety Is Calculated By The Modified Bishop Method**

Add 600 ft to vertical scale for actual elevations.

| | | | | | |
|---|---|---|---|---|---|
| **Norfleet Consultants** | Limit Equlibrum Analysis - Stevens Creek Quarry | | | | |
| | Pseudo-static analysis, High strength, Full failure, Cross-sect A-A -west' | | | | |
| | PROJ NO: 071781 | DATE: | Jan 10, 2008 | FIGURE: | 20 |



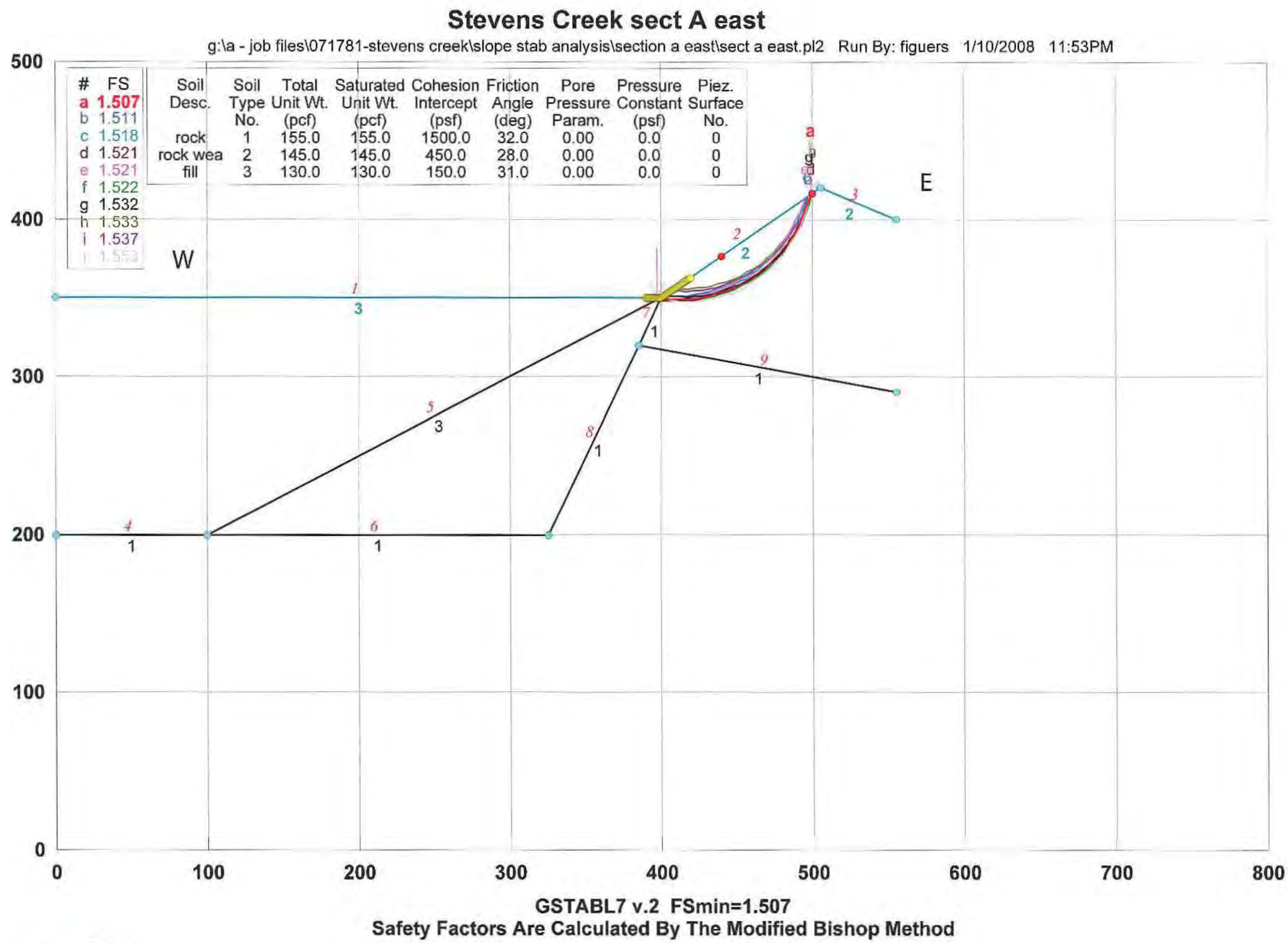

| Soil Desc. | Soil Type No. | Total Unit Wt. (pcf) | Saturated Unit Wt. (pcf) | Cohesion Intercept (psf) | Friction Angle (deg) | Pore Pressure Param. | Pressure Constant (psf) | Piez. Surface No. |
|---|---|---|---|---|---|---|---|---|
| rock | 1 | 155.0 | 155.0 | 2000.0 | 32.0 | 0.00 | 0.0 | 0 |
| rock wea | 2 | 145.0 | 145.0 | 1000.0 | 28.0 | 0.00 | 0.0 | 0 |
| fill | 3 | 130.0 | 130.0 | 150.0 | 31.0 | 0.00 | 0.0 | 0 |

| Load | Value |
|---|---|
| Peak(A) | 0.570(g) |
| kh Coef. | 0.200(g)< |

GSTABL7

Add 600 ft to vertical scale for actual elevations.

| Norfleet Consultants | Limit Equlibrum Analysis - Stevens Creek Quarry | | | | |
|---|---|---|---|---|---|
| | Pseudo-static analysis, Low strength, Full failure, Cross-sect A-A -west' | | | | |
| | PROJ NO: 071781 | DATE: Jan 10, 2008 | | FIGURE: | 21 |




GSTABL7

Add 500 ft to vertical scale for actual elevations.

**Norfleet Consultants**

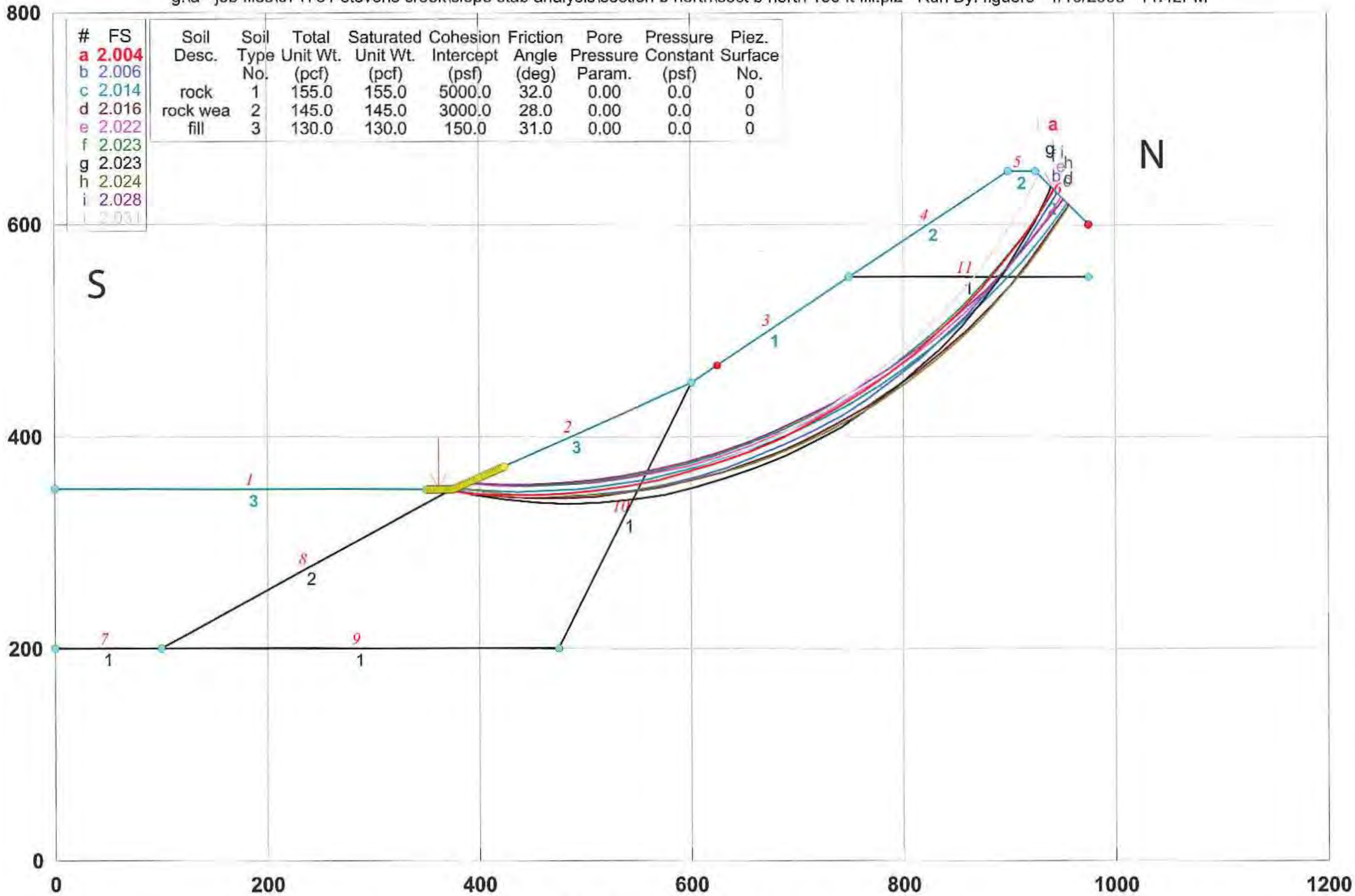
Limit Equlibrum Analysis - Stevens Creek Quarry

Static analysis, , Full failure, Cross-sect A-A' -east

| PROJ NO: | 071781 | DATE: | Jan 10, 2008 | FIGURE: | 22 |
|---|---|---|---|---|---|

## Stevens Creek sect B

g:\a - job files\071781-stevens creek\slope stab analysis\section b north\sect b north 150 ft fill.pl2 Run By: figuers 1/10/2008 11:42PM



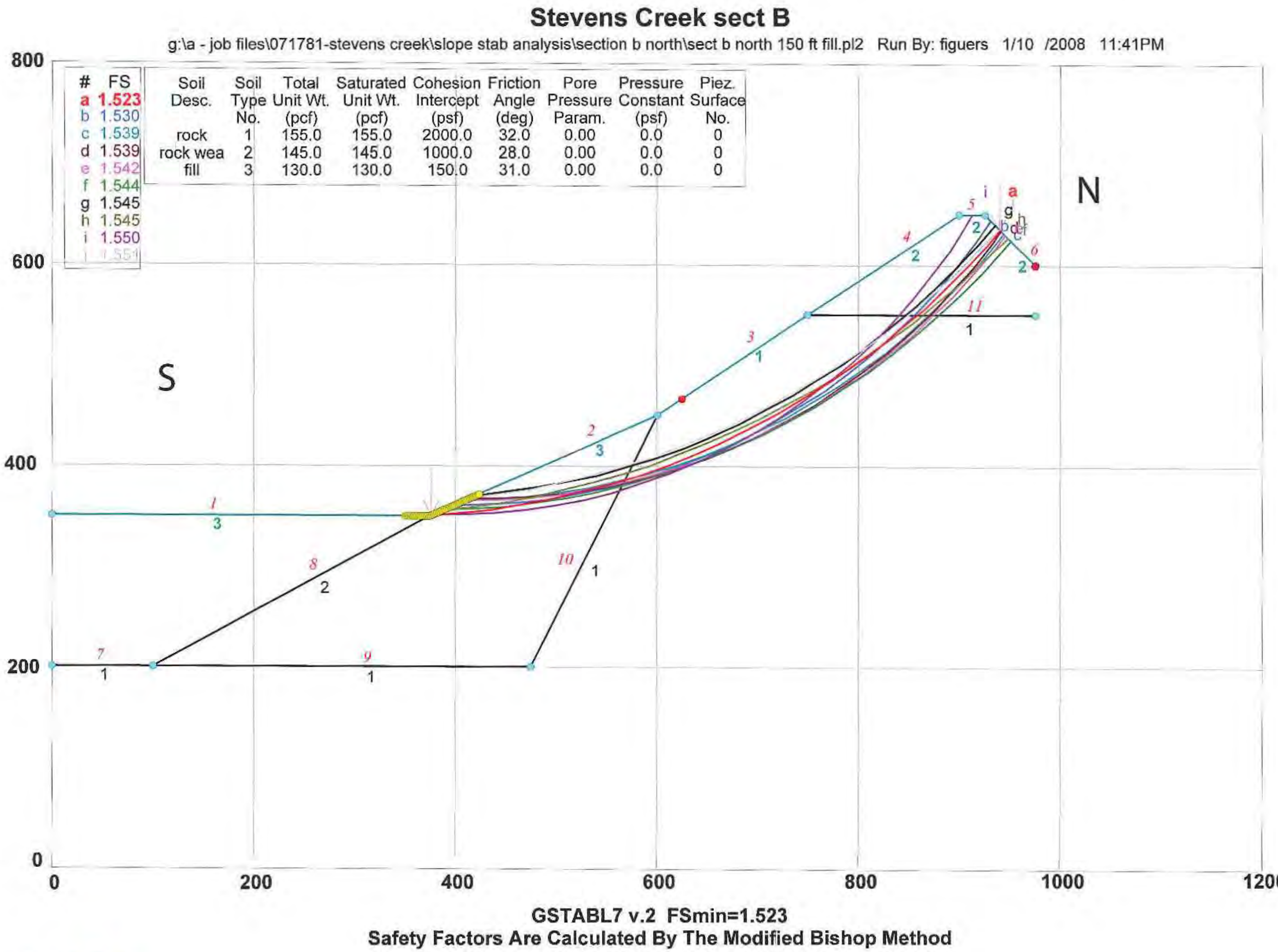

| Soil Desc. | Soil Type No. | Total Unit Wt. (pcf) | Saturated Unit Wt. (pcf) | Cohesion Intercept (psf) | Friction Angle (deg) | Pore Pressure Param. | Pressure Constant (psf) | Piez. Surface No. |
|---|---|---|---|---|---|---|---|---|
| rock | 1 | 155.0 | 155.0 | 5000.0 | 32.0 | 0.00 | 0.0 | 0 |
| rock wea | 2 | 145.0 | 145.0 | 3000.0 | 28.0 | 0.00 | 0.0 | 0 |
| fill | 3 | 130.0 | 130.0 | 150.0 | 31.0 | 0.00 | 0.0 | 0 |

**GSTABL7 v.2 FSmin=2.004**
**Safety Factors Are Calculated By The Modified Bishop Method**

GSTABL7

Add 500 ft to vertical scale for actual elevations.

| Norfleet Consultants | Limit Equlibrum Analysis - Stevens Creek Quarry | | | | |
|---|---|---|---|---|---|
| | Static analysis, High Rk Strenght, Full failure, Cross-sect B-B’ | | | | |
| | PROJ NO: 071781 | DATE: Jan 10, 2008 | FIGURE: 23 | | |

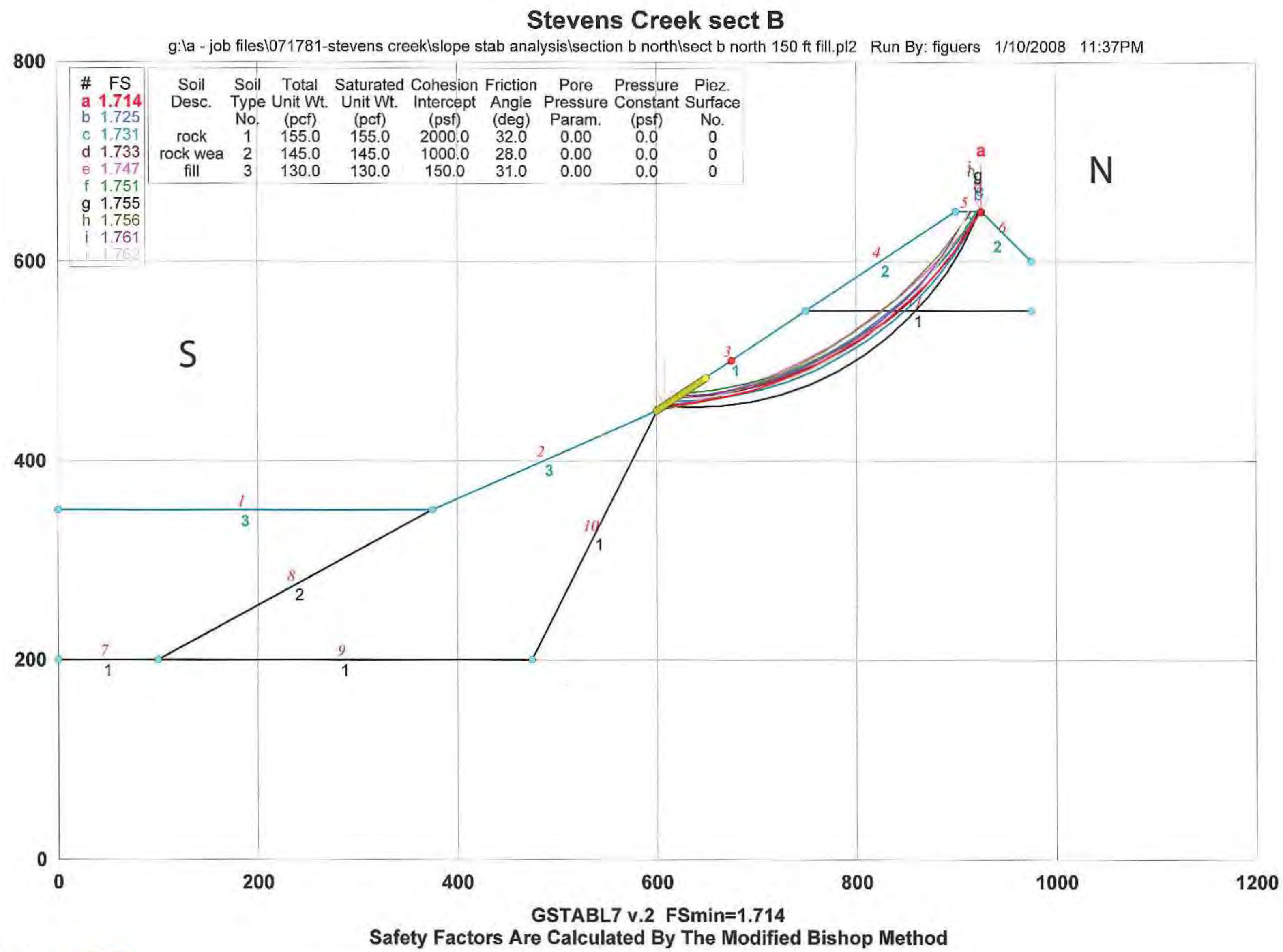



GSTABL7

Add 500 ft to vertical scale for actual elevations.

**Norfleet Consultants**

| Limit Equlibrum Analysis - Stevens Creek Quarry | | |
|---|---|---|
| Static analysis, Low Rk Strenght, Full failure, Cross-sect B-B' | | |
| PROJ NO: 071781 | DATE: Jan 10, 2008 | FIGURE: 24 |



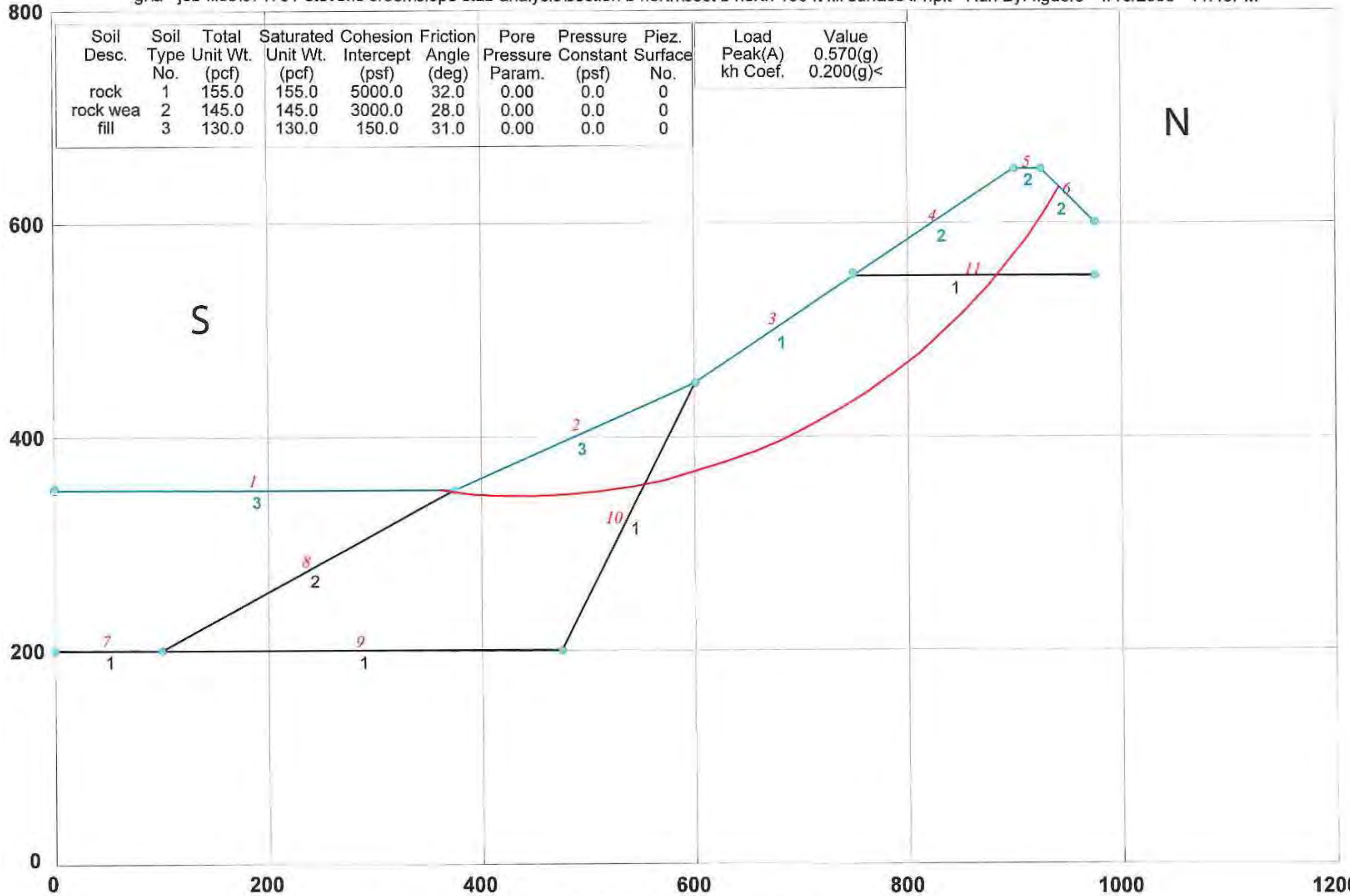

Add 500 ft to vertical scale for actual elevations.

**Norfleet Consultants**

| Limit Equlibrum Analysis - Stevens Creek Quarry | | | | | |
|---|---|---|---|---|---|
| Static analysis, Low Rk Strenght, Rk failure, Cross-sect B-B' | | | | | |
| PROJ NO: | 071781 | DATE: | Jan 10, 2008 | FIGURE: | 25 |

## Stevens Creek sect B

g:\a - job files\071781-stevens creek\slope stab analysis\section b north\sect b north 150 ft fill surface #1.plt Run By: figuers 1/10/2008 11:43PM




| Soil Desc. | Soil Type No. | Total Unit Wt. (pcf) | Saturated Unit Wt. (pcf) | Cohesion Intercept (psf) | Friction Angle (deg) | Pore Pressure Param. | Pressure Constant (psf) | Piez. Surface No. |
|---|---|---|---|---|---|---|---|---|
| rock | 1 | 155.0 | 155.0 | 5000.0 | 32.0 | 0.00 | 0.0 | 0 |
| rock wea | 2 | 145.0 | 145.0 | 3000.0 | 28.0 | 0.00 | 0.0 | 0 |
| fill | 3 | 130.0 | 130.0 | 150.0 | 31.0 | 0.00 | 0.0 | 0 |

| Load | Value |
|---|---|
| Peak(A) | 0.570(g) |
| kh Coef. | 0.200(g)< |

**GSTABL7 v.2 FSmin=1.405**
**Factor Of Safety Is Calculated By The Modified Bishop Method**

GSTABL7

Add 500 ft to vertical scale for actual elevations.

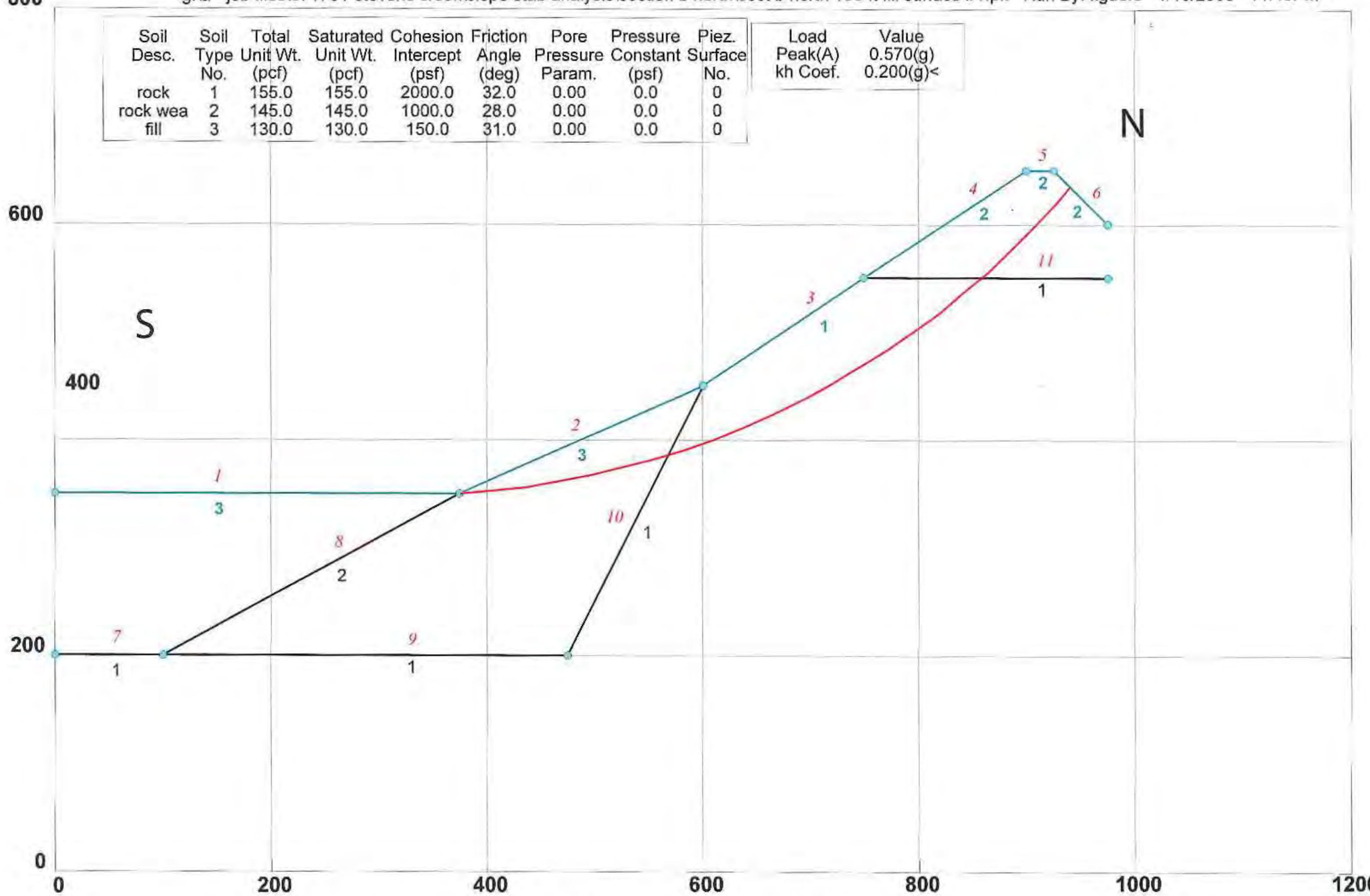
| Norfleet Consultants | Limit Equlibrum Analysis - Stevens Creek Quarry |
|---|---|
| | Pseudo-static analysis, High Rk Strenght, Cross-sect B-B' |
| | PROJ NO: 071781 / DATE: Jan 10, 2008 / FIGURE: 26 |

## Stevens Creek sect B

g:\a - job files\071781-stevens creek\slope stab analysis\section b north\sect b north 150 ft fill surface #1.plt Run By: figuers 1/10/2008 11:45PM

| Soil Desc. | Soil Type No. | Total Unit Wt. (pcf) | Saturated Unit Wt. (pcf) | Cohesion Intercept (psf) | Friction Angle (deg) | Pore Pressure Param. | Pressure Constant (psf) | Piez. Surface No. |
|---|---|---|---|---|---|---|---|---|
| rock | 1 | 155.0 | 155.0 | 2000.0 | 32.0 | 0.00 | 0.0 | 0 |
| rock wea | 2 | 145.0 | 145.0 | 1000.0 | 28.0 | 0.00 | 0.0 | 0 |
| fill | 3 | 130.0 | 130.0 | 150.0 | 31.0 | 0.00 | 0.0 | 0 |

| Load | Value |
|---|---|
| Peak(A) | 0.570(g) |
| kh Coef. | 0.200(g)< |

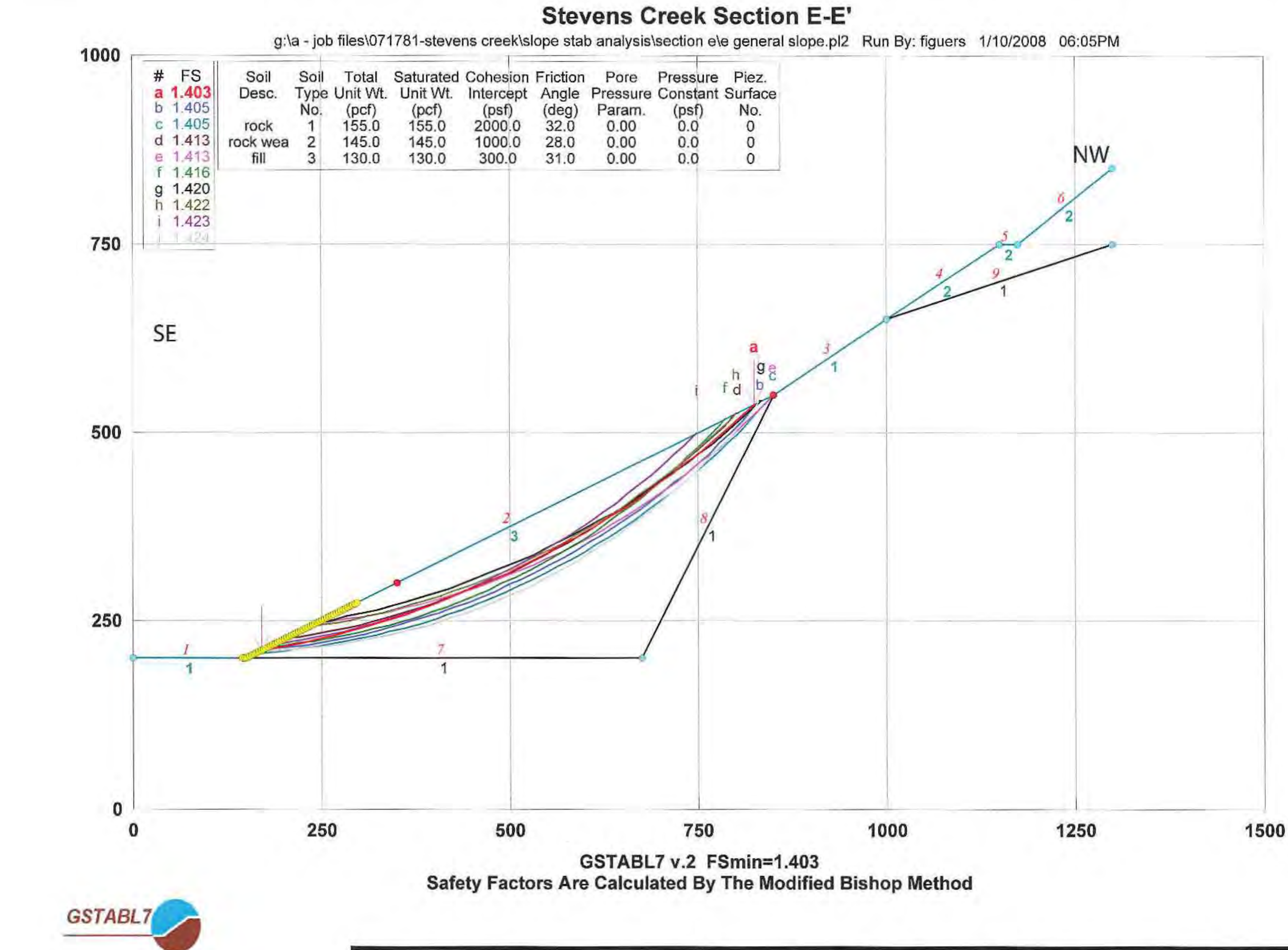



**GSTABL7 v.2 FSmin=1.049**
**Factor Of Safety Is Calculated By The Modified Bishop Method**

GSTABL7

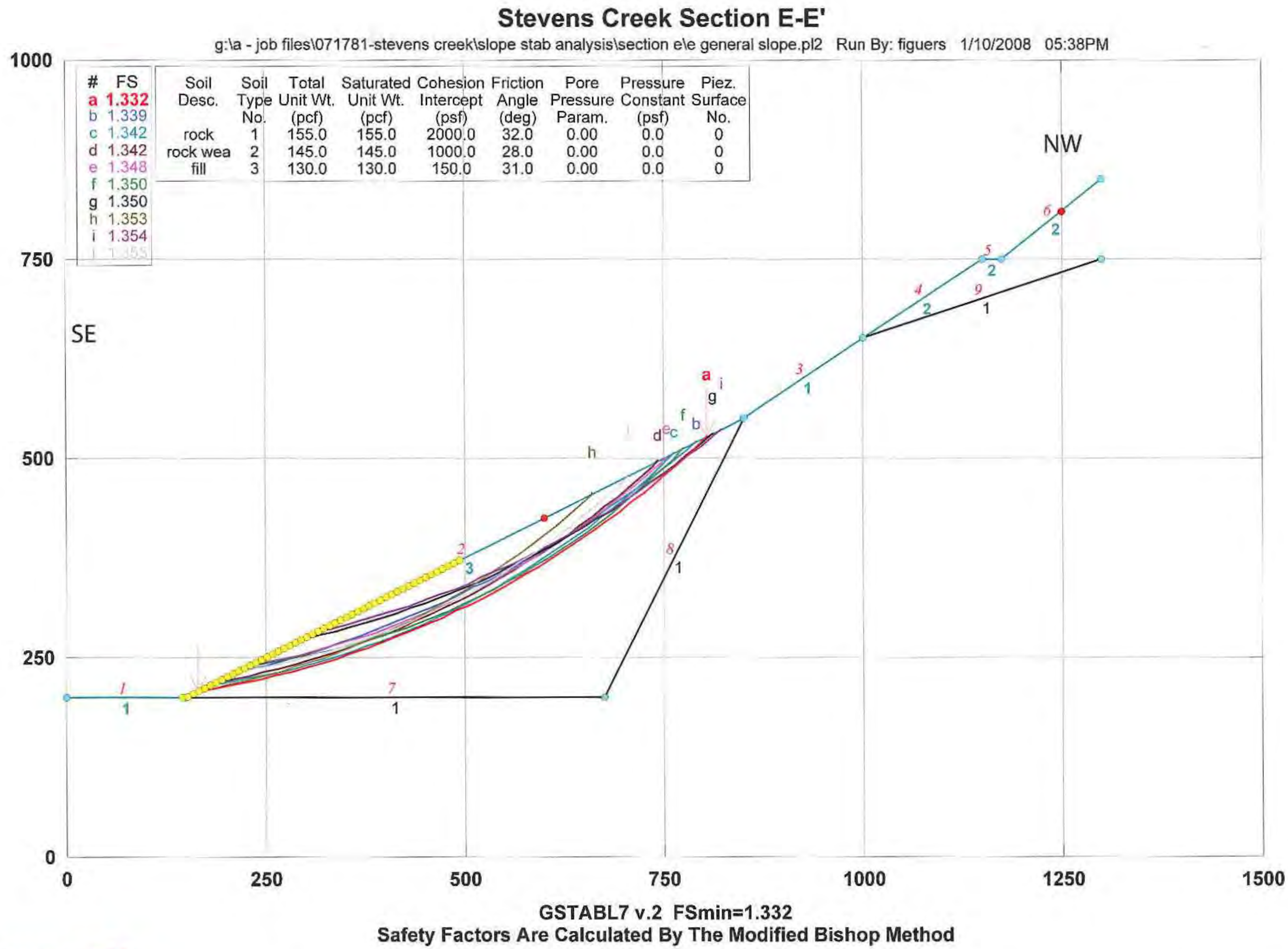
Add 500 ft to vertical scale for actual elevations.

| Norfleet Consultants | Limit Equlibrum Analysis - Stevens Creek Quarry | | |
|---|---|---|---|
| | Pseudo-static analysis, Low Rk Strenght, Cross-sect B-B' | | |
| | PROJ NO: 071781 | DATE: Jan 10, 2008 | FIGURE: 27 |




Add 500 ft to vertical scale for actual elevations.

| | |
|---|---|
| **Norfleet Consultants** | Limit Equlibrum Analysis - Stevens Creek Quarry |
| | Static analysis, 300 psf Coh. wedge fill only, cross-section E-E' |
| | PROJ NO: 071781 — DATE: Jan 10, 2008 — FIGURE: 28 |




| # | FS |
|---|---|
| a | 1.332 |
| b | 1.339 |
| c | 1.342 |
| d | 1.342 |
| e | 1.348 |
| f | 1.350 |
| g | 1.350 |
| h | 1.353 |
| i | 1.354 |
| j | 1.355 |

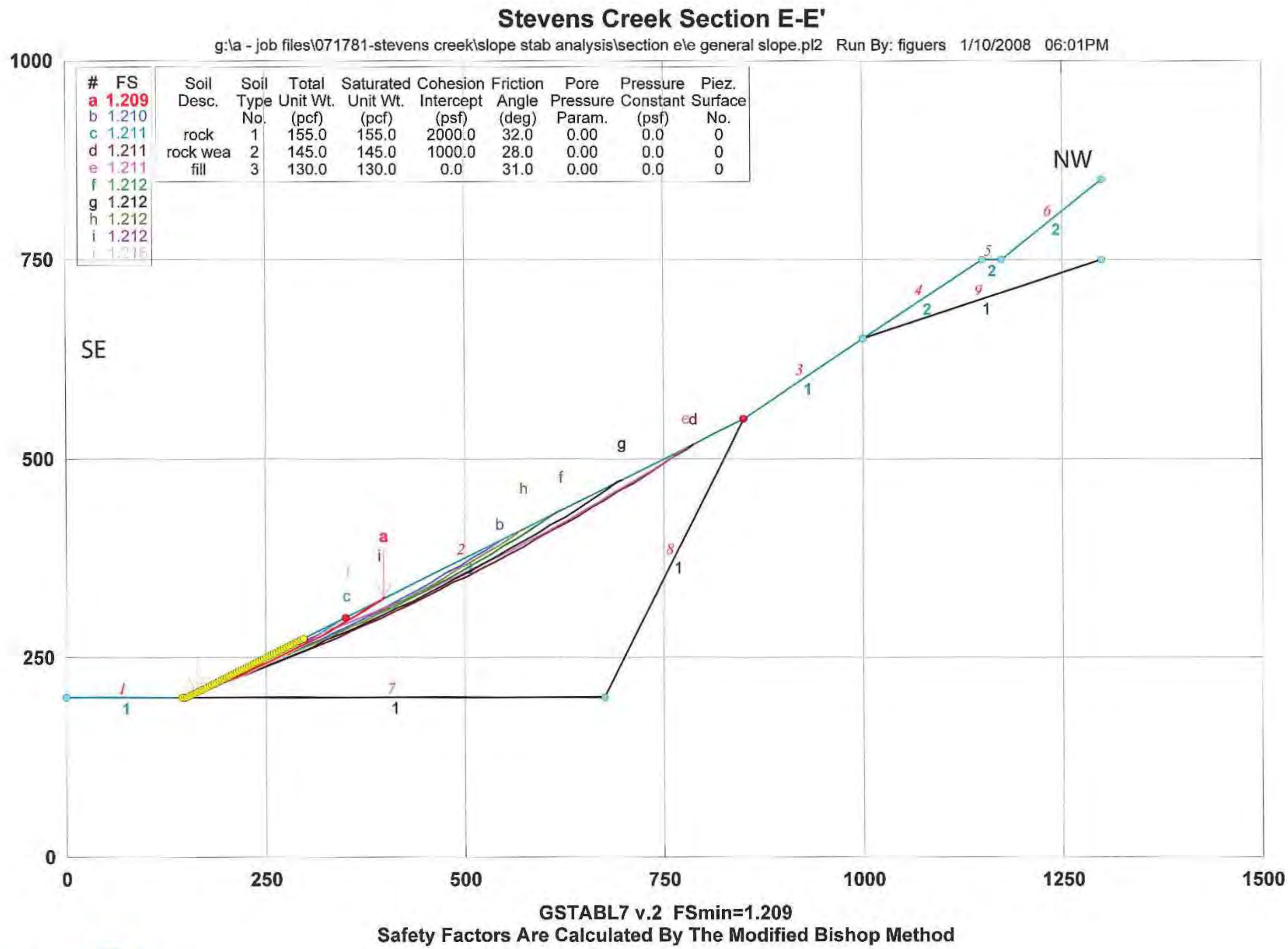
| Soil Desc. | Soil Type No. | Total Unit Wt. (pcf) | Saturated Unit Wt. (pcf) | Cohesion Intercept (psf) | Friction Angle (deg) | Pore Pressure Param. | Pressure Constant (psf) | Piez. Surface No. |
|---|---|---|---|---|---|---|---|---|
| rock | 1 | 155.0 | 155.0 | 2000.0 | 32.0 | 0.00 | 0.0 | 0 |
| rock wea | 2 | 145.0 | 145.0 | 1000.0 | 28.0 | 0.00 | 0.0 | 0 |
| fill | 3 | 130.0 | 130.0 | 150.0 | 31.0 | 0.00 | 0.0 | 0 |

GSTABL7

Add 500 ft to vertical scale for actual elevations.

**Norfleet Consultants**

| Limit Equlibrum Analysis - Stevens Creek Quarry | | | | | |
|---|---|---|---|---|---|
| Static analysis, 150 psf Coh. wedge fill only, cross-section E-E' | | | | | |
| PROJ NO: | 071781 | DATE: | Jan 10, 2008 | FIGURE: | 29 |



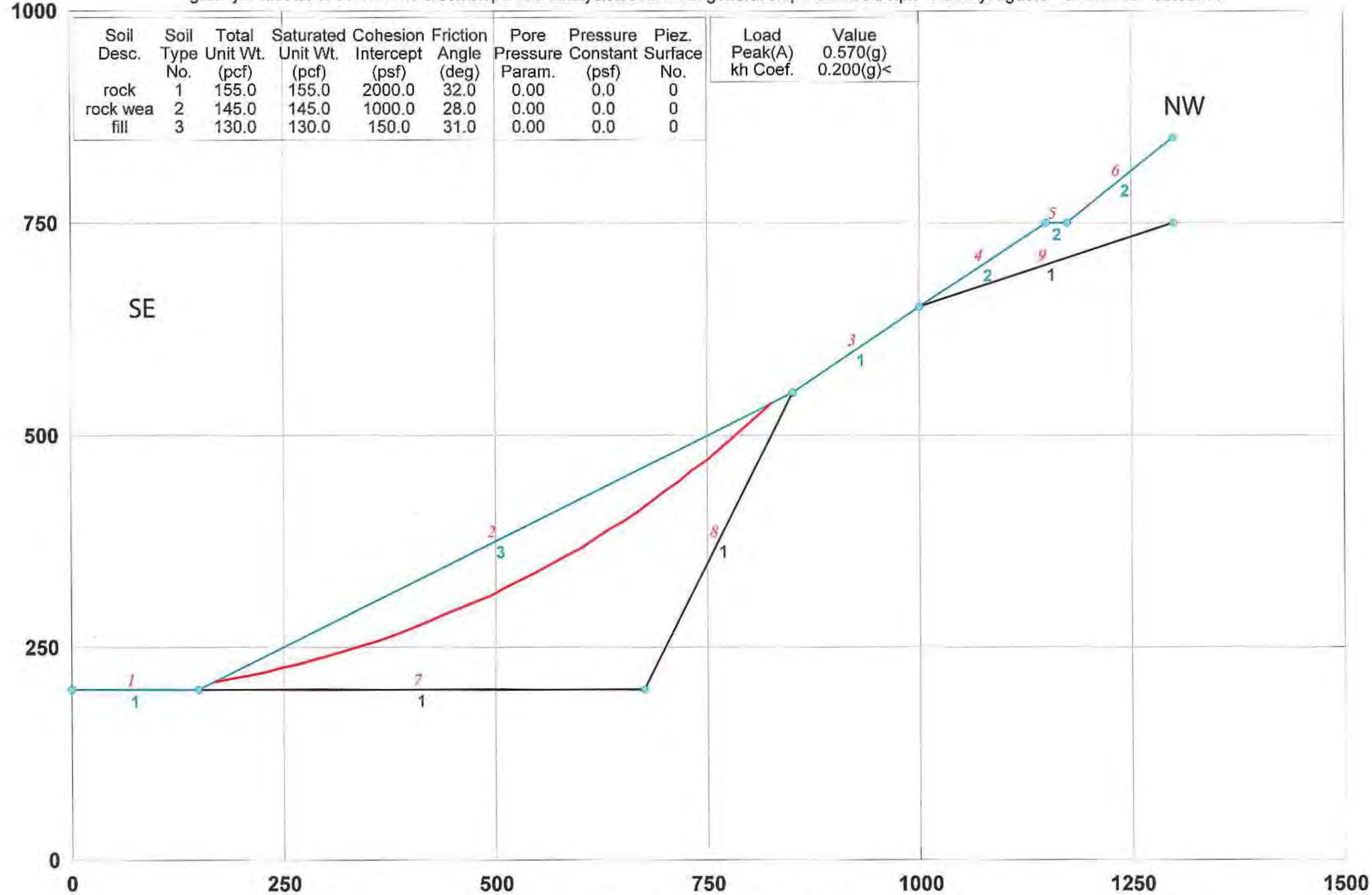

Add 500 ft to vertical scale for actual elevations.

**Norfleet Consultants**

| Limit Equlibrum Analysis - Stevens Creek Quarry | | | | | |
|---|---|---|---|---|---|
| Static analysis, 0 Coh. wedge fill only, cross-section E-E' | | | | | |
| PROJ NO: | 071781 | DATE: | Jan 10, 2008 | FIGURE: | 30 |

## Stevens Creek Section E-E'

g:\a - job files\071781-stevens creek\slope stab analysis\section e\e general slope surface #1.plt Run By: figuers 1/10/2008 06:00PM




| Soil Desc. | Soil Type No. | Total Unit Wt. (pcf) | Saturated Unit Wt. (pcf) | Cohesion Intercept (psf) | Friction Angle (deg) | Pore Pressure Param. | Pressure Constant (psf) | Piez. Surface No. |
|---|---|---|---|---|---|---|---|---|
| rock | 1 | 155.0 | 155.0 | 2000.0 | 32.0 | 0.00 | 0.0 | 0 |
| rock wea | 2 | 145.0 | 145.0 | 1000.0 | 28.0 | 0.00 | 0.0 | 0 |
| fill | 3 | 130.0 | 130.0 | 150.0 | 31.0 | 0.00 | 0.0 | 0 |

| Load | Value |
|---|---|
| Peak(A) | 0.570(g) |
| kh Coef. | 0.200(g)< |

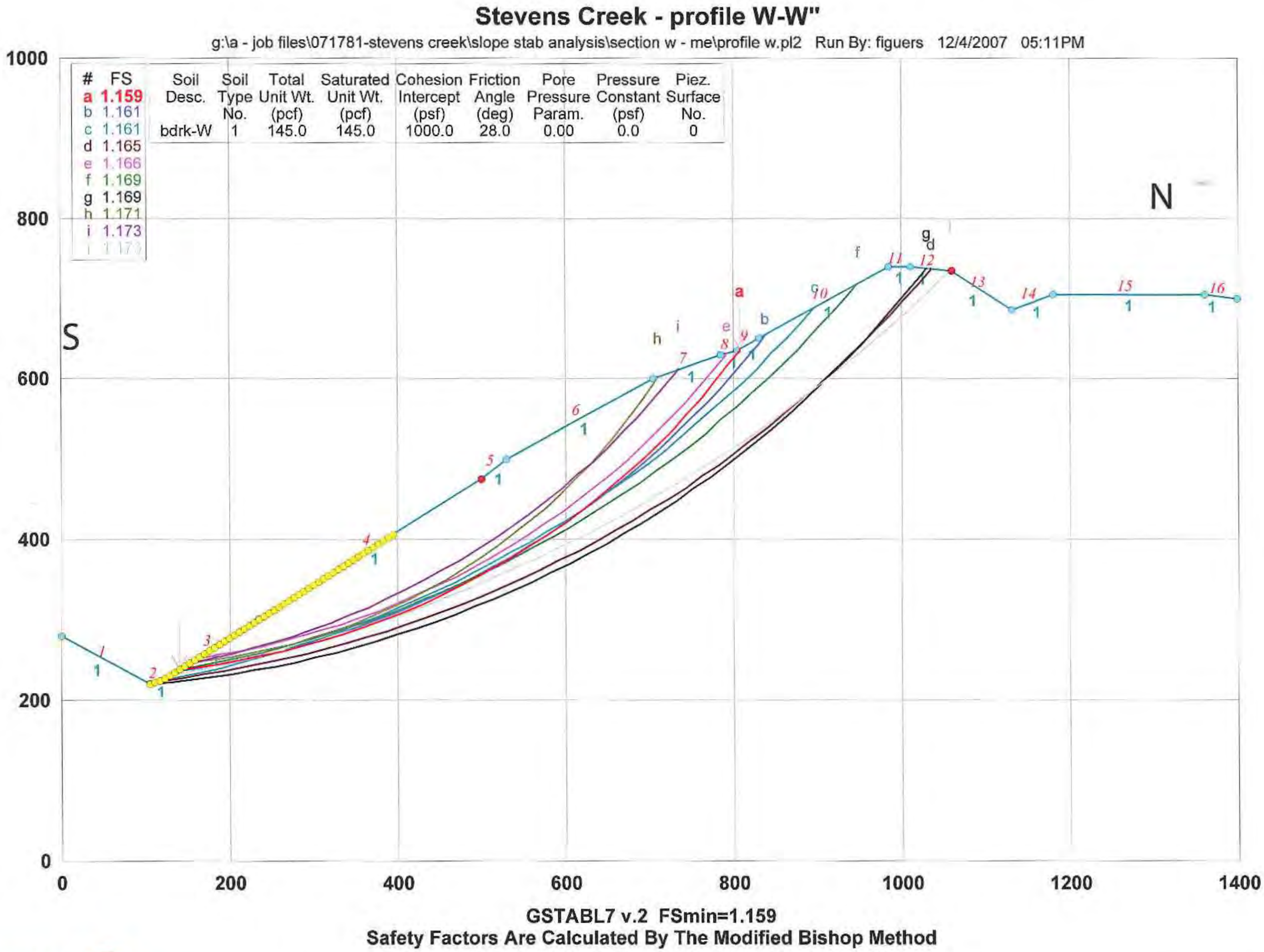
GSTABL7 v.2 FSmin=0.867

Factor Of Safety Is Calculated By The Modified Bishop Method

GSTABL7

Add 500 ft to vertical scale for actual elevations.

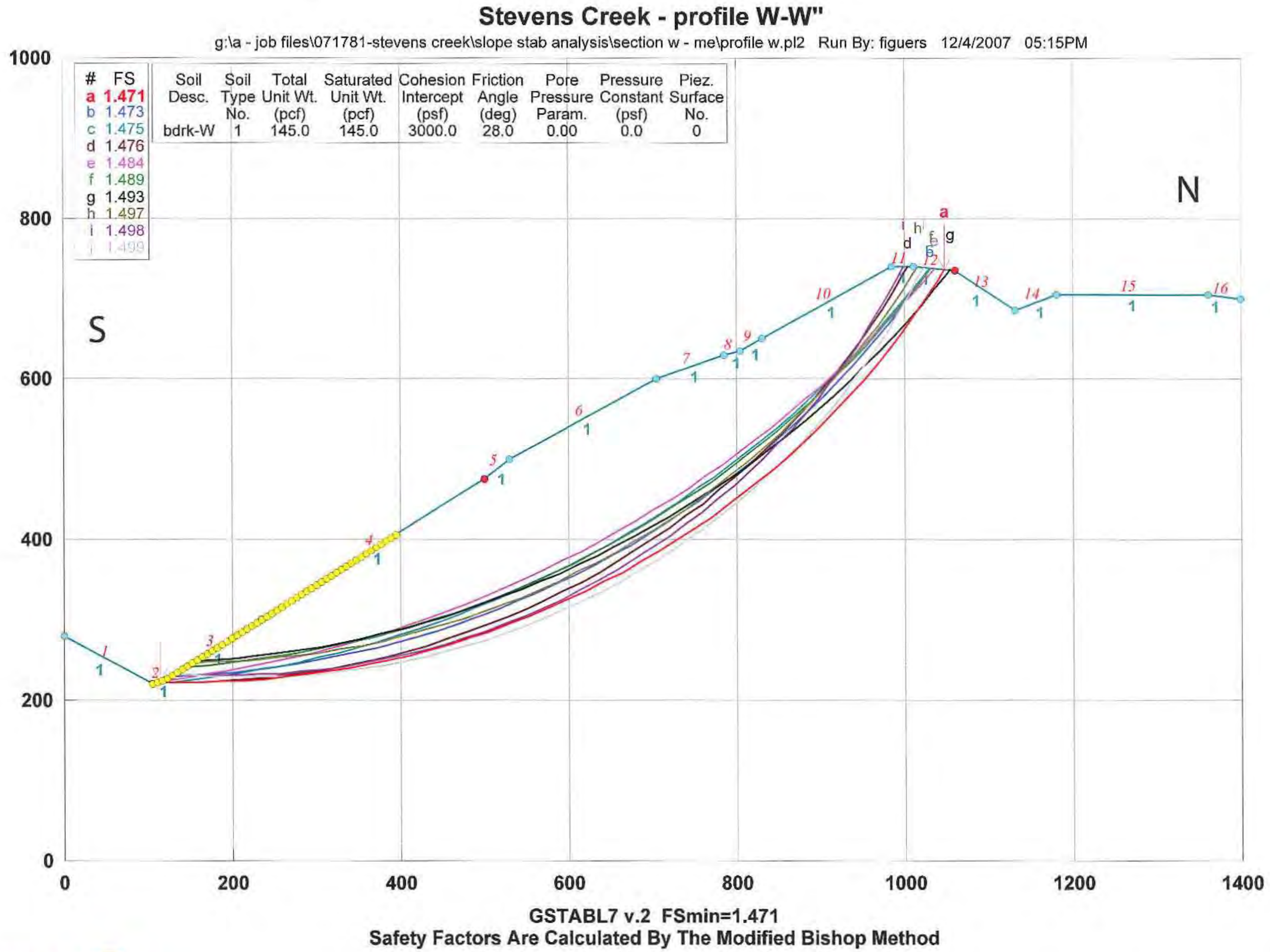
| | | |
|---|---|---|
| **Norfleet Consultants** | Limit Equlibrum Analysis - Stevens Creek Quarry | |
| | Pseudo-static analysis, 150 coh., wedge fill only, cross-section E-E' | |
| | PROJ NO: 071781 | DATE: Jan 10, 2008 | FIGURE: 31 |




GSTABL7

Add 500 ft to vertical scale for actual elevations.

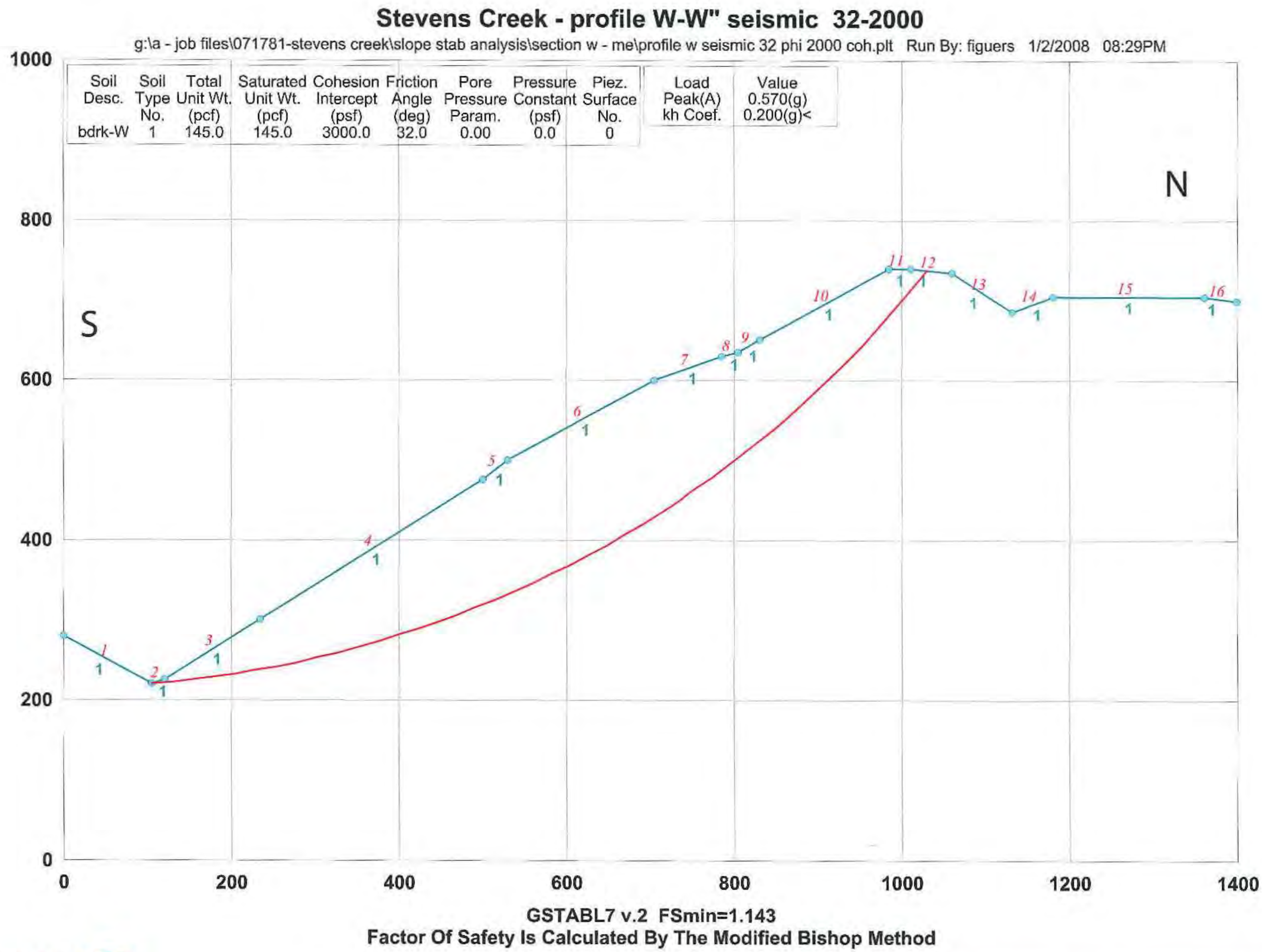
| | | | |
|---|---|---|---|
| **Norfleet Consultants** | Limit Equlibrum Analysis - Stevens Creek Quarry | | |
| | Static analysis, 1000 psf Coh., weathered Greenstone, cross-section W | | |
| | PROJ NO: 071781 | DATE: Jan 10, 2008 | FIGURE: 32 |




GSTABL7

Add 500 ft to vertical scale for actual elevations.

| | | | | | |
|---|---|---|---|---|---|
| **Norfleet Consultants** | Limit Equlibrum Analysis - Stevens Creek Quarry | | | | |
| | Static analysis, 3000 psf Coh., weathered Greenstone, cross-section W | | | | |
| | PROJ NO: | 071781 | DATE: Jan 10, 2008 | FIGURE: | 33 |



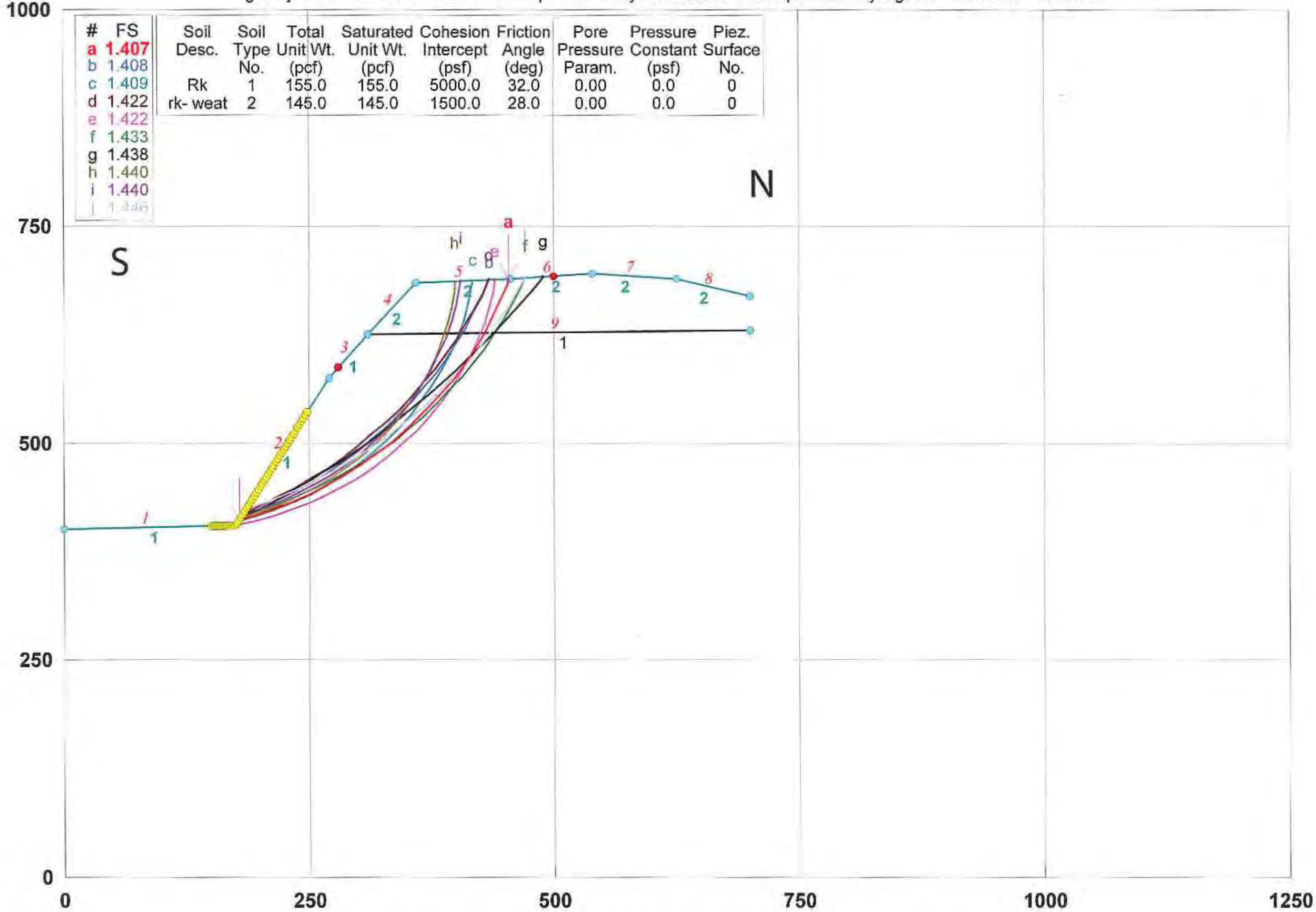

GSTABL7

Add 500 ft to vertical scale for actual elevations.

| Norfleet Consultants | Limit Equlibrum Analysis - Stevens Creek Quarry | | |
|---|---|---|---|
| | Pseudo-static analysis, weathered Greenstone, cross-section W | | |
| | PROJ NO: 071781 | DATE: Jan 10, 2008 | FIGURE: 34 |

## Stevens Creek - Section Z

g:\a - job files\071781-stevens creek\slope stab analysis\section z\z base.pl2 Run By: figuers 12/4/2007 07:24PM




| Soil Desc. | Soil Type No. | Total Unit Wt. (pcf) | Saturated Unit Wt. (pcf) | Cohesion Intercept (psf) | Friction Angle (deg) | Pore Pressure Param. | Pressure Constant (psf) | Piez. Surface No. |
|---|---|---|---|---|---|---|---|---|
| Rk | 1 | 155.0 | 155.0 | 5000.0 | 32.0 | 0.00 | 0.0 | 0 |
| rk- weat | 2 | 145.0 | 145.0 | 1500.0 | 28.0 | 0.00 | 0.0 | 0 |

**GSTABL7 v.2 FSmin=1.407**
**Safety Factors Are Calculated By The Modified Bishop Method**

GSTABL7

Add 500 ft to vertical scale for actual elevations.

| Norfleet Consultants | Limit Equlibrum Analysis - Stevens Creek Quarry | | |
|---|---|---|---|
| | Static analysis, 5000 psf Coh., unweath. Greenstone, cross-section Z | | |
| | PROJ NO: 071781 | DATE: Jan 10, 2008 | FIGURE: 35 |

January 9, 2020
BAGG Job No: STEVE-18-02
California Mine ID 91-43-007

Mr. Jason Voss
jvoss@scqinc.com
Stevens Creek Quarry, Inc. (SCQ)
12100 Stevens Canyon Road
Cupertino, California 95014

**REVISED REPORT**
In-Depth Engineering Geologic Investigation
and Slope Stability Analysis
Western Rim Slope
Stevens Creek Quarry
12100 Stevens Canyon Road
Cupertino, California 95014

Dear Mr. Voss:

This revised letter report presents the results of our engineering geologic evaluation and slope stability analysis performed for the approximately 2,000-foot long Western Rim Slope at the SCQ in Cupertino, California. BAGG Engineers has issued this report initially on January 3, 2019 and it is being revised herein to check if the modified Reclamation Plan, developed after the issuance of our January 2019 report with input from our technical experts for the Western Rim Slope, has satisfactory long-term factors of safety.

It is important to note that this letter report pertains exclusively to the Western Rim Slope portion of the quarry. This report is intended to assess the stability of the temporary (short-term) and permanent (long-term) cut and fill slopes proposed currently along the Western Rim Slope as part of the quarry's revised/modified Reclamation Plan and it also presents grading recommendations for the planned fill placement.

## SITE DESCRIPTION AND PLANNED RECLAMATION

The attached Plate 1, Vicinity Map, delineates the general location of the overall quarry while Plates 2 and 3, which were provided to us by Benchmark Resources, show the magnitude and configuration of the proposed Cut and Fill Phases along the subject slope with six scaled cross section lines and cross sections (labeled as Cross Sections 1-1' through 6-6' on both plates). Plates 4 and 5, show the Cut and Fill Phases planned at the Western Rim Slope and the cross section lines presented at a scale (one inch equals 200 feet) that matches our structural geologic cross sections, which we will discuss further in upcoming sections of this report.

The northern portion of the east-facing Western Rim Slope (subject slope) abuts the mining pit along its western side while the central and southern parts of the subject slope are situated along the west side of the main staging/processing and jaw crusher areas, respectively. Mining cuts along the subject slope were generally initiated along the quarry's western property line and then extended downslope eastward. Based on preliminary cross sections we performed utilizing a topographic base map, prepared by Muir Consulting, Inc. and flown December 2018, the mined slope gradient appeared to vary between about 1.4H:1V and 1.6H:1V (Horizontal to Vertical) overall, although some localized areas appeared to have steeper gradients.

The northern portion of the Western Rim Slope opposite the mining pit has experienced surficial slumping and failure of the Franciscan Complex greenstone bedrock nearly along the entire height of the mined slope as a result of the noted mining cuts. The central and southern sections of the Western Rim Slope have been covered with fill stockpiles that obscured the greenstone bedrock and which appeared to have experienced surficial slumping and slope movement also, except at the far southern end of the subject slope where slumping greenstone bedrock was exposed at the ground surface. A prominent landslide, which we will discuss further in upcoming sections of this report, has occurred along the northernmost portion of the subject slope (northwestern corner of the quarry) near its connection with the Northern Rim Slope. The landslide's headscarp is evident in the field manifesting itself as open arcuate soil cracks coupled with down-dropped zones that extended upslope beyond the property line and encroached onto the adjacent property to the west.

Furthermore, there are three Pacific and Gas Electric Company (PG&E) wooden pole installations present near the top of the northern, central, and southern parts of the Western Rim Slope generally along the quarry's western property line. The northernmost PG&E Installation #1 consists of wooden poles that are located near the radio station and storage containers present along the top of the slope where the above-noted landslide's arcuate extensional soil cracks have developed upslope and around the noted power poles as a result of the mentioned slope failure and mass wasting downslope along the cut slope face. Plate 6 is an aerial site plan that approximately delineates the location of the three PG&E wooden pole installations along the quarry's western property line in addition to our surficial geologic mapping performed as part of our scope along the Western Rim Slope.

The new Reclamation Plan configuration prepared by Benchmark Resources (2019) along the subject slope was developed with input from our Certified Engineering Geologist (CEG) and it generally consisted of Cut and Fill Phases. The Cut Phase (see Plates 2 and 4) will consist of initiating 1.5H:1V cuts at the western property line and then extending the cut downslope to a set elevation of 1,050 feet above mean sea level (MSL) where a 100-foot wide mid-slope bench will be constructed. Downslope of the noted mid-slope bench, a temporary mining slope will be cut at a steeper 1H:1V gradient down to and terminating at elevation 700 feet MSL. Upslope of the quarry's western property line, the 1.5H:1V cut will be extended until daylighting higher up the slope. Based on six preliminary cross sections (1-1' through 6-6'), it appears that the top of the cut would daylight between about 80 and 440 feet beyond the quarry's western property line depending on the localized topographic conditions. Once the Cut Phase is completed, the Fill Phase will be undertaken and it will generally consist of depositing engineered fills starting at elevation 700 feet MSL and terminating at elevation 900 feet MSL with a 3H:1V fill slope extending from the 900-foot elevation to the downslope edge of the planned mid-slope bench constructed at elevation 1,050 feet MSL. No fill will be deposited higher than the noted mid-slope bench elevation of 1,050 feet MSL.

## PURPOSE AND SCOPE OF SERVICES

The purpose of our services was to observe the existing field conditions, provide input to the quarry manager and Benchmark Resources, analyze the stability of the proposed Cut and Fill Phases of the revised/modified Reclamation Plan, and provide grading recommendations for the construction of the planned fill and cut slopes. It is important to note that Benchmark Resources has provided us with topographic base maps depicting the existing ground surface during December 2018. the planned Cut and Fill Phases (Plates 2 through 5) and six cross sections (1-1' through 6-6' shown on Plates 2 and 3) the locations of which were selected by us. We utilized Cross Sections 1-1' through 6-6' prepared by Benchmark Resources to depict our geologic models and to perform the stability analysis, which will be discussed in upcoming sections of this letter report.

Specifically, our scope of work included the following elements:

- Review the pertinent parts of the Reclamation Plan amendment dated 2008 and a site-specific geologic and stability analysis report prepared by Norfleet Consultants in 2008 (Appendix D of the noted amendment).
- Perform slope reconnaissance and mapping visits to the Western Rim Slope by our CEG and geotechnical engineer. Our slope reconnaissance included observing the Western Rim Slope including the locations of the three PG&E power pole installations present along the top of the Western Rim Slope roughly located along the quarry's western property line.
- Collect clayey gouge samples from a rock slide basal rupture surface along the central part of the Western Rim Slope and from prominent shear/slip surfaces near the northern end of the subject slope that have developed within the Franciscan Complex greenstone exposed along the slope face.
- Atterberg Limits and torsional ring shear testing were conducted on the two clayey samples noted above by an independent testing laboratory (Cooper Testing Laboratory).
- Evaluate the collected data and perform slope stability analyses under static and pseudo-static (seismic) loading conditions.

- Meeting attendance and consultation with the quarry manager and other design team members.
- Prepare this letter report summarizing our findings, conclusions, and recommendations based on our analysis of six geologic Cross Sections 1-1’ through 6-6’ that were extended across portions of the Western Rim Slope where approximately shown on Plates 2 through 5. This report includes a vicinity map, site plans, area geologic map, geologic cross sections, laboratory testing results, stability analysis plots, and our conclusions and recommendations as they pertain to the stability of the planned Cut and Fill Phases. The stability plots were based on geologic cross sections and models, which we developed as part of our scope and which we discuss further in subsequent sections of this letter report.

## PROJECT BACKGROUND

Norfleet Consultants (Norfleet) issued a report titled *Geologic and Stability Analysis, Reclamation Plan Amendment, Stevens Creek Quarry, California Mine ID 91-43-007, San Jose, California* dated January 22, 2008. The Norfleet study included site reconnaissance, review of available documentation, laboratory testing of fill import, and numerical evaluation of cross sections for slope stability of the proposed reclamation slope geometry.

The Norfleet study was intended to provide a summary of the geologic and geotechnical issues as they pertain to long-term, global stability of the final slope geometries as defined in the Reclamation Plan amendment revised January 2008 for the “pit west of the Berrocal fault” referred to as Parcel B by the quarry operator.

## GEOLOGY AND SEISMICITY

### Area and Site Geology

The geology and seismicity of the site area have been described in detail by Norfleet in 2008 and the intent of this section is to provide an overall summary of the site geology and seismicity. The site area has been mapped by several mappers including Dibblee (1966), Rogers (1972), Rogers and Armstrong (1973), Rogers and Williams (1974), Sorg and McLaughlin (1975), Brabb et al. (1998), Brabb et al. (2000), and Dibblee and Minch (2007). The site area is underlain by Cretaceous/Jurassic age Franciscan Complex greenstone bedrock that is closely and highly fractured, sheared, and foliated. Our mapping of the surficial geology observed along the subject slope is depicted on Plate 6, Aerial Site Plan and Geology while the portion of Brabb et al. (1998) that covers the site area is included as Plate 7, Area Geology Map.

The upper approximately 40 to 60 feet of the greenstone bedrock appeared weathered and colored yellowish brown due to oxidation while the greenstone bedrock exposed on the mined slope generally appeared greenish gray due to reduction below the upper oxidized zone.

A prominent shear zone was observed along the north end of the Western Rim Slope (near the northwestern corner of the quarry pit), which consisted of several steep shear planes some of which were lined with plastic greenish clayey gouge. The shears extended the entire height of the cut slope and several of the shear planes appeared to strike east/west and dip steeply to the south with one prominent shear plane trending northwestward and dipping steeply to the southwest. The noted shear zone can be seen in Figure 8, Site Photo, which was taken prior to the slope failures that occurred in that area subsequently. The shear zone was mapped by Sorg and McLaughlin (1975) to extend diagonally across Parcel B of the quarry and connect with the main trace of the Berrocal fault to the southeast, which was mapped by them extending along the east side of Parcel B. Norfleet (2008) shows the shear zone as a band of serpentine that extended through the greenstone bedrock. Although we observed the shear zone on the initial cut near the northern end of the Western Rim Slope, our CEG did not observe the serpentine inclusions delineated by Norfleet in 2008 as the area was underlain by greenstone entirely.

Initially, the Western Rim Slope was mined from the top and up to 8 drainage terraces (benches) were constructed across the slope face. However, due to the height and steep gradient of the mined cut (0.5H to 0.75H:1V) and the foliated, sheared, and fractured nature of the greenstone, slope failures developed along the cut face, which resulted in shearing and damaging the noted intermediate benches and further steepening the cut slope locally. The slope failures appeared to have mobilized in a translational fashion due to removal of lateral support nearly along the entire Western Rim Slope except along the central portion where fill stockpiles covered the slope face and provided some lateral support. Extensional cracking was observed during our reconnaissance of the slope along the top of the cut and even upslope of the noted fill stockpiles that are present along the central and southern portions of the Western Rim Slope. Arcuate soil cracking developed subsequently beyond the top of the mined slope along the northern end of the subject slope extending upslope of and around the wooden power poles at the PG&E Installation #1 (see Plates 6 and 8). The soil cracking marked the upper reaches of the prominent landslide that has developed near the northernmost part of the subject slope, which extended beyond the quarry's western property line. Numerous concrete stitch piers and a soldier pile and lagging retaining wall have been installed immediately downslope of the wooden power pole to help protect it and container-like structures present along the top of the slope in that area. As the ground surface down-dropped along the top of the slope where PG&E Installation #1 is situated, the tops of the concrete stitch piers noted above were observed sticking up higher than the ground surface. PG&E Installation #2 did not appear to have been impacted by the surficial slumping occurring along the slope face. At PG&E Installation #3 situated along the southern part of the subject slope, another soldier pile and lagging retaining wall has been constructed to support the power installation, which did not appear to have been impacted by the surficial slope failures that have occurred along the cut slope face downslope of that installation.

## Landslides

None of the referenced mappers delineated landslide deposits along the Western Rim Slope except Sorg and McLaughlin (1975) who mapped a large-scale block slide upslope of the shear zone mapped near the northernmost portion of the subject slope and discussed above. Brabb et al. (1998) mapped a large-scale landslide deposit on the south side of the drainage course bordering the Western Rim Slope to the south. The noted landslide is shown to have encroached on and shifted the drainage channel located to the south of the Western Rim Slope, curving it northeastward slightly (See Plate 8). Our CEG did not observe evidence of the large-scale block slide mapped by Sorg and McLaughlin (1975) on the mined cut slope face where the side margins and basal rupture surface should have been present if the slide had existed. Furthermore, as part of our scope, we reviewed historical Google Earth Pro aerial photographs that spanned the period 1948 (before the slope was mined) through 2018 but we did not observe geomorphic evidence of the noted block slide.

The Western Rim Slope is shown by the California Geological Survey (CGS) on their regulatory Seismic Hazard Zone maps (2002) to be within a Seismic Hazard Zone associated with earthquake-induced landslides. The subject slope was not shown to be within a Seismic Hazard Zone associated with soil liquefaction, however.

**Faulting and Seismicity**

The main trace of the Berrocal fault has been mapped by Sorg and McLaughlin (1975), Brabb et al. (1998), and Norfleet (2008) in addition to several other mappers to extend roughly north/south along the east side of the quarry's Parcel B where the active mining pit is situated. The Barrocal fault is a high-angle reverse fault dipping between 50 to 70 degrees to the west. The older Franciscan units to the west of the fault appear to have been thrusted over the younger terrestrial Santa Clara Formation sedimentary units. Norfleet (2008) indicated that it is unlikely a specific fault plane is present and that the fault is represented by a shear zone measuring between 50 to 100 feet in width and which extended along the east side of the main mining pit.

The Berrocal fault was not zoned as active by the Division of Mines and Geology (DMG, 1974) and the CGS (2000) because it did not meet their zonation criteria. However, while the fault is within a Santa Clara County (County) Fault Rupture Hazard Zone (SCC, 2012), the fault trace and the hazard zone delineated by the County do not encroach onto the subject Western Rim Slope.

The San Andreas fault is mapped about 2 miles to the southwest and the Monte Vista-Shannon fault is mapped about 1.25 miles to the northeast of the site area. Norfleet (2008) indicated that while the quarry was active during the Loma Prieta Earthquake of October 17, 1989, the quarry personnel reported that the quake did not cause rockfalls or slope failures and only a single water glass fell off a counter in a nearby house. Furthermore, Norfleet (2008) indicated that a study of aftershocks from the 1989 earthquake in the Santa Cruz Mountains performed by Lindley and Archuleta (1994) found that Franciscan ridgetops had little ridgetop amplification and shatter and that the average amplification at Franciscan Complex sites was 3 times less than amplification at Miocene and Pliocene sites.

**Groundwater**

Based on input from the quarry operator, groundwater has not been encountered at the site area for along as the quarry has functioned. In addition, the quarry operator reported that a well drilled at a residence within the immediate area of the quarry did not encounter a groundwater phreatic level. Isolated seepages were observed along the subject slope face and free water was present within the main mining pit and also within the Upper, Middle, and Lower Settling Basins within the overall quarry, the noted water is detained storm water runoff and not groundwater. It is important to note that groundwater levels can vary seasonally due to inclement weather and irrigation.

## Site Reconnaissance and Observations

As part of our current scope, our CEG performed a reconnaissance of the Western Rim Slope and his summarized observations are presented below:

- The overall slope, which measured about 2,000 feet in length, has been mined starting near the property line at an approximate gradient of about 1.4-1.6H:1V. According to the Reclamation Plan amendment (2008), the mining excavation will bottom out at about 700 feet MSL and the height of the mined Western Rim Slope will vary between about 600 and 700 feet in height.
- The noted mining cuts generally exposed foliated and highly/closely fractured and sheared Franciscan Complex greenstone bedrock except for the central portion of the subject slope, which was obscured with fill stockpiles. The upper approximately 40 to 60 feet of greenstone bedrock appeared yellowish brown due to weathering and oxidation while the lower remaining exposed portion of the slope appeared gray to greenish gray due to reduction.
- Isolated water seepages coupled with white evaporate mineral staining were observed near the northern end of the subject slope.
- A prominent and well-developed shear plane trending northwestward and dipping steeply to the southwest was observed extending the entire height of the mined slope generally near the northern end of the subject slope. The shear plane surface was lined with greenish gray clayey gouge that appeared wet and moderately to highly plastic. Sorg and McLaughlin (1975) and Norfleet (2008) mapped a shear zone, which is associated with the Berrocal fault, in the general area of the noted shear plane/zone. They extended the shear zone southeastward where it is shown to cross the entire Parcel B of the quarry in a diagonal fashion and connecting with the main trace of the Berrocal fault near the southeast corner of Parcel B. Norfleet (2008) mapped the feature as a serpentine shear zone and although the clays lining the shear plane we observed appeared greenish/bluish gray, our CEG did not observe any serpentine in the immediate vicinity of the shear plane/zone, as we noted above.
- Due to the fractured and weak nature of the greenstone bedrock and the relatively high mining cuts made along the subject Western Rim Slope, the outer approximately 30 to 40 feet of the exposed greenstone underlying the slope face appeared to have experienced failure in a chiefly translational mode leading to the occurrence of rockfalls, rock slides, and block gliding coupled with minor toppling nearly along the entire length and height of the slope except where fill is stockpiled blanketing the greenstone along the central and southern sections of the subject slope. Along the uppermost part of the placed fill stockpiles against the central and southern portions of the subject slope, our CEG observed arcuate, open extensional soil cracking marking the upper boundaries of the fill.

- Near the northern end of the subject slope and as noted above, our CEG observed an active landslide that had occurred along the mining cut slope face near the northern end of the Western Rim Slope. The landslide extended the whole height of the mined slope and its headscarp formed arcuate open soil cracks that appeared to extend westward beyond the quarry's western property line. A level pad was created along the top of the slope in that area where three container-like structures and wooden power poles were observed. In response to the noted slope movement and cracking in that area, several concrete "stitch" piers have been installed downslope of the noted structures and a soldier pile and lagging retaining wall was installed along the edge of the cut. Slope movement appeared to have continued since the installation of the concrete piers and the retaining wall since the noted cracking extended under a portion of the retaining wall and voids were observed around the tops of the visible concrete piers as the ground level appears to have settled and dropped in that area. The open soil cracks extended around and upslope of the wooden power pole (PG&E Installation #1) present in that area and they displayed lateral separation that is coupled with vertical displacement.
- The central part of the Western Rim Slope, generally upslope of the conveyor belt that spans the haul road and connects with the Jaw Crusher, was observed covered with fill stockpiles that have relatively high side slopes and near-level tops.
- PG&E Installation #2 situated near the central top of the subject slope, consists of a single wooden power pole that is located about 650 feet to the southwest of PG&E Installation #1 and about 100 feet upslope and to the west beyond the top of the mining cut. No separation/gap between the base of the power pole and the surrounding soil was observed and no soil cracks were observed in the immediate vicinity of the power pole at this location.
- Farther to the south and approximately 700 feet to the south of PG&E Installation #2, we observed another PG&E support structure (PG&E Installation #3) consisting of three wooden poles that are supported by a soldier pile and lagging wall along their eastern side, which separates the power poles from the edge of the mining cut. No open soil cracks or other distress features were observed in the vicinity of PG&E Installation #3 and the edge of the cut appeared to be about 50-75 feet downslope of the power poles.
- A fill buttress has been initiated near the toe of the Western Rim Slope northern end and its construction is ongoing.
- During our reconnaissance, our CEG collected two disturbed clayey gouge samples for Atterberg Limits and torsional shear testing purposes by Cooper Testing laboratory. Sample A was collected from clays lining the prominent shear plane present near the northern end of the subject slope and discussed above while Sample B was obtained from a basal rock glide rupture surface exposed near the top of the central mined slope.

## LABORATORY TESTING

Atterberg Limits and torsional ring shear testing were performed by Cooper Testing Laboratory on the two samples collected from the shear surfaces described above to evaluate their plasticity index and to generate peak and residual internal angles of friction values for landslide rupture surfaces. The laboratory test plots are included in Appendix A along with remolded shear testing plots developed by Norfleet Consultants for the import fill material.

## SLOPE STABILITY ANALYSIS

### Geologic Model

As discussed above, the mining cuts along the Western Rim Slope were initiated near the quarry's western property line and extended downslope. Numerous intermittent drainage/access benches were initially constructed as part of the mining operation (See Plate 8, Site Photo). The noted mining cuts resulted in relatively high and steep slope gradients that were made in sheared/foliated/fractured and weak greenstone bedrock. As a result of the noted cuts, surficial translational slope failures occurred along the entire height of the mined slope displacing and damaging the mentioned drainage/access benches.

According to the latest Reclamation Plan (Benchmark Resources, 2019) and as discussed above, the Cut Phase will consist of permanent 1.5H:1V mining cuts that will be initiated at the quarry's western property line and carried downslope to an elevation of 1,050 feet MSL where a 100-foot wide mid-slope bench will be constructed. Downslope of the noted bench, the slope will be cut at a steeper temporary 1H:1V configuration down to elevation 700 feet MSL. Upslope of the quarry's western property line, the 1.5H:1V cuts will be extended higher towards the west until they daylight out of the slope and depending on the topographical constraints, the cuts are expected to daylight between about 70 and 400 feet beyond the quarry's western property line. The Fill Phase, according to the latest Reclamation Plan, will consist of depositing engineered fills starting at elevation 700 feet MSL and terminating at elevation 900 feet MSL. A permanent 3H:1V reclamation fill slope will extend from elevation 900 feet MSL to the downslope edge of the 100-foot wide mid-slope bench planned at elevation 1,050 feet MSL. It is important to note that Cross Sections 4-4' and 5-5' (Benchmark Resources, 2019), show the permanent slope cuts above the mid-slope bench planned at elevation 1,050 feet MSL to have a shallower 1.9H:1V gradient, which is most likely dictated by the topographic conditions in those areas.

As part of developing our geologic model, we have assumed that the greenstone bedrock will be exposed along the permanent cut slope planned upslope of the mid-slope bench at 1,050 feet MSL once the undocumented fill stockpiles are over-excavated and removed along the central and southern sections of the Western Rim Slope. Furthermore, we anticipate that the planned 1.5H:1V permanent cut planned upslope of the mid-slope bench will remove the landslide debris at Cross Section 1-1'. Plate 9 depicts the planned reclamation grading along Cross Section 1-1' and based on the knowledge that the prominent landslide mapped near the northern end of the Western Rim Slope is anticipated to measure between 40 and 60 feet in depth, it appears that the planned cuts will result in removing the majority of the landslide debris. Our CEG should be presented the opportunity to observe the noted cut slopes at this area and the undocumented fill stockpile areas to verify that the existing fill and landslide debris have been over-excavated fully. If the planned cuts do not result in the complete removal of the undocumented fill and landslide debris, additional remedial grading recommendations will be developed and implemented, depending on the lateral extent and depth of the remaining undocumented fill and/or landslide debris.

To develop our geologic models, we delineated the geologic units on Cross Sections 1-1’ through 6-6’ needed for our slope stability analysis. Accordingly, our geologic models accounted for the planned engineered fill (AF) to be constructed as part of the Reclamation Plan downslope of the 100-foot wide mid-slope bench, the upper weathered and oxidized greenstone (wgs), and the gray fresh un-weathered greenstone (gs). Plates 10 through 15 present Cross Sections 1-1’ through 6-6’ with the geologic conditions depicted on them and which we utilized in our slope stability analysis.

**Slope Modeling and Analysis Method**

The stability of the cut and fill slopes was evaluated with the conventional method of limit equilibrium stability analysis on two dimensional slope cross section with the aid of the computer program GeoStudio 2019 (Slope/W). Our analysis used the Morgenstern-Price Method, which considers both interslice shear and normal forces of the individual slices, into which the soil mass above the failure surface is divided, and includes both moment and force equilibrium. Various trial failure surfaces are analyzed in this manner until a minimum factor of safety is obtained.

**Soil Strength Parameters**

For stability analysis purposes and as noted above, three (3) earth material types were established, which include engineered fill soil (AF), weathered yellowish brown oxidized greenstone bedrock (wgs), and fresh (un-weathered) gray greenstone (gs). Norfleet (2008) performed back-calculations on both cut and natural greenstone slopes to estimate in-place weathered and un-weathered greenstone rock phi angles and cohesion properties. They then lowered the cohesion values to include lower strength rock/joint conditions (lower bound values) while keeping the phi values fixed. Norfleet concluded that it is likely the actual rock strength values are closer to or higher than the back-analysis properties.

For the weathered greenstone (wgs), Norfleet obtained a back-calculated cohesion value of 3,000 psf and assigned a 1,000 psf for the lower bound cohesion value coupled with a phi angle of 28 degrees. For our analysis, we selected to use the conservative lower bound cohesion value of 1,000 psf and the 28-degree phi angle for the weathered greenstone.

For the fresh (un-weathered) greenstone bedrock (gs), Norfleet obtained a back-calculated cohesion value of 5,000 psf and assigned a 2,000 psf for the lower bound cohesion value coupled with a phi angle of 32 degrees. For our analysis, we utilized the lower bound cohesion value of 2,000 psf for all our cross sections except at Cross Section 2-2’ where we utilized a reasonable cohesion value of 3,000 psf instead while maintaining the 32-degree phi angle assigned by Norfleet in 2008.

Furthermore, as part of the Norfleet 2008 study, Cooper Testing Laboratory performed shear testing on two import remolded fill samples that Norfleet submitted to establish their phi and cohesion values. For our analysis, we elected to use a lower bound cohesion value of 150 psf and a phi value of 31 degrees, which were assigned by Norfleet for the engineered fill (AF).

Additionally, as noted in our previous report, groundwater has not been encountered anywhere in the quarry and a well drilled within the limits of the quarry indicated that groundwater levels are relatively deep. Hence, groundwater was not included in our stability analysis. The strength parameters for the various earth materials mentioned above and which we incorporated into our analysis are presented in the following table.

**Soil Strength Parameters**

| Material Type | Lower Bound Cohesion C (psf) | Friction Angle Phi-ϕ (degrees) | Unit Weight (pcf) |
|---|---|---|---|
| Artificial Fill (AF) | 150 | 31 | 130 |
| Weathered Greenstone (wgs) | 1,000 | 28 | 155 |
| Fresh Greenstone (gs) All sections except 2-2' | 2,000 | 32 | 155 |
| Fresh Greenstone (gs) Section 2-2' | 3,000 | 32 | 155 |

## Cut Phase Slope Stability Analysis

As noted previously, the Cut Phase will consist of cutting the slope portion above elevation 1,050 feet MSL (bench) at a 1.5H:1V gradient except at Cross Sections 4 and 5 where the cut will be made at a 1.9H:1V gradient. In addition, the slope portion below elevation 1,050 feet MSL will be cut at a 1H:1V gradient along the entire length of the slope and at all six cross sections. Based on the encountered geologic conditions, the selected strength parameters and the geometry of Cross Sections 1-1' through 6-6', the results of our slope stability analysis yielded static safety factors ranging from 1.15 to 1.32 for the Cut Phase condition as is shown on Plates 16 through 21.

It is important to note that the 1H:1V cut downslope of the proposed mid-slope bench at elevation 1,050 feet MSL is a temporary (short-term) configuration since engineered fill will be placed to buttress the 1H:1V cut as part of the Fill Phase. A factor of safety of 1.0 is deemed acceptable under static conditions and no seismic loading was modeled since the noted 1H:1V configuration is temporary as noted prior. In addition, the stability of the 1.5H-1.9H:1V permanent reclamation cut slopes above the mid-slope bench were analyzed as part of the Fill Phase, which we discuss below.

## Fill Phase Slope Stability Analysis

### Static Slope Stability Analysis

Engineered fill will be deposited starting at elevation 700 feet MSL and terminating at elevation 900 feet MSL with 3H:1V fill slopes extending from elevation 900 feet MSL to the downslope edge of the mid-slope future bench to be constructed at elevation 1,050 feet MSL. Based on the results of our analysis, the Fill

Phase static factors of safety at Cross Sections 1-1’ through 6-6’ ranged from about 1.57 to 2.04 with a factor of safety of 1.5 as the minimum required.

### Seismic Slope Stability Analysis

The seismic stability of the permanent long term condition was analyzed using a pseudo-static approach per the general guidelines included in CGS Special Publication 117A (2008) and the Southern California Earthquake Center (2002). The Earthquake Engineering Research Institute has published a screening analysis procedure for seismic slope stability (Stewart et al., 2003), which takes into account local variations in the seismicity as presented by the earthquake magnitude, as well as the distance from the fault that most significantly contributes to the ground motion hazard at the site. The screening procedure is based on a statistical relationship previously developed by Bray et al. (1998) between seismic slope displacement (u), peak amplitude of shaking in the underlying bedrock (kmax), significant duration of shaking (D5-95), and the ratio of slope resistance to peak demand (ky/kmax), where ky is the yield acceleration, or the horizontal acceleration required to reduce the safety factor to unity. A tolerable seismic slope displacement (u) for residential range from 5 cm to 15 cm. A safety factor of 1 is the minimum required for passing the screening procedure.

Using the slope screening procedure, a pseudo-static coefficient of 0.29g was estimated for the analyses based on respective deformation of 15 cm. A minimum seismic factor of safety of 1.0 was obtained for all six cross sections analyzed as is shown on the stability analysis plots presented on Plates 28 through 33. In addition, the results of our static and seismic slope stability analyses are summarized in the table below. The results of the analyses indicate that the planned temporary cut slopes (downslope of the bench) and permanent cut and fill reclamation slopes (above and below the bench, respectively) are generally considered stable based on the geometry of the cross sections provided by Benchmark Resources utilizing the noted strength parameters coupled with the assumption that the reclamation grading and earthwork operation will remove the landslide debris and fill stockpiles and expose relatively firm Greenstone (gs) along the slope surface.

**Summary of Slope Stability Analyses Results**

| Section | Cut Phase | Fill Phase | |
|---|---|---|---|
| | Static Factor of Safety* | Static Factor of Safety* | Seismic Factor of Safety* (0.29g) |
| 1-1' | 1.15 | 1.58 | 1.00 |
| 2-2' | 1.32 | 1.61 | 1.01 |
| 3-3' | 1.16 | 1.61 | 1.00 |
| 4-4' | 1.15 | 2.04 | 1.01 |
| 5-5' | 1.15 | 2.01 | 1.01 |
| 6-6' | 1.15 | 1.57 | 1.00 |

* Rounded to two decimal points

## CONCLUSIONS AND RECOMMENDATIONS

### General

1. Our slope stability analysis results indicate that all the planned temporary (short-term) cut slopes and permanent (long-term) cut and fill slopes have acceptable factors of safety under static conditions and seismic loading.

2. Cross Sections 4-4’ and 5-5’ indicate that the permanent reclamation cut slopes above the mid-slope bench planned at elevation 1,050 feet MSL will have 1.9H:1V gradient while the remaining sections (1-1’ through 3-3’ and 6-6’) indicate that the noted permanent reclamation cut slopes above the mid-slope bench will have a 1.5H:1V gradient. We recommend that the gradient variation should be made gradual laterally and should not be made abruptly from 1.5H:1V to 1.9H:1V and vice versa. Such an approach will allow for a seamless lateral transition between the sections.

3. Our CEG should be provided the opportunity to observe the condition of the temporary and permanent cut slopes prior to the placement of the planned fill to verify that the landslide debris at Cross Section 1-1’ and the existing fill stockpiles have been over-excavated completely and to develop remedial grading recommendations to address the presence of undocumented fills or landslide debris on the cut slopes, if deemed necessary during construction.

4. The planned 100-foot wide mid-slope bench should be constructed to slightly dip into the hillside so that storm water runoff is directed towards the inboard side of the bench. A cobble-lined earthen ditch or a concrete V-ditch should be constructed along the inboard edge (heel) of the bench to collect runoff water and direct it to flow laterally into approved drainage inlets.

5. The proposed reclamation fill should be moisture conditioned, and deposited horizontally in 8-inch (loose thickness) lifts, compacted to a minimum of 90 percent relative compaction of the maximum dry density at near the optimum moisture content in accordance with ASTM method D1557.

6. The fill should be benched and keyed into the backcut slope as the fill placement progresses upslope. The fill slope face should be overbuilt and then trimmed back so that a uniform and compacted slope face is exposed. This recommendation is made because it is difficult to compact soil along the outer edge of the fill, which is needed to help prevent the occurrence of subsequent shallow slope failures and localized slumps.

7. Thick-walled subdrain lines consisting of perforated 4- to 6-inch diameter PVC pipes (Type SDR-23.5 or equivalent) that are encased in a 2-foot wide by 3-foot high envelope of Caltrans Class 2 permeable material should be placed beneath and behind the proposed fill section, in a manner which would provide positive gravity flow, to help keep subsurface water from encroaching on and adversely impacting the sides and underside of the fill. The subdrain lines/pipes should be

installed at no greater than 30-foot vertical intervals and be equipped with cleanouts at all ends and bends to permit future access for cleaning and video viewing.

8. The bottom of all excavations and the placement of subdrain lines should be observed by the project CEG prior to fill placement.

9. The fill placement and compaction should be performed under the direct observation of the project Geotechnical Engineer and/or his field representatives. Field observation and compaction testing should be performed periodically so that the process of fill placement, moisture conditioning, and compaction effort is consistent. Our field representative should be provided the opportunity to perform compaction testing on every foot of fill placed and compacted prior to the placement of the next fill lift. The frequency of compaction testing will depend on the contractor's compaction production rates and the lateral extent of area to be filled.

### Plan Review

It is recommended that the Geotechnical Engineer (BAGG Engineers) be retained to review the final grading plans. This review is to assess general suitability of the earthwork and drainage recommendations contained in this report and to verify the appropriate implementation of our recommendations into the project plans and specifications.

### Observation and Testing

It is recommended that BAGG Engineers be retained to provide observation and testing services during site grading, excavation, and backfilling phases of work. This is intended to verify that the work in the field is completed per our recommendations and in accordance with the approved plans and specifications, and more importantly verify that subsurface conditions encountered during construction are similar to those anticipated during the design phase. Unanticipated soil conditions may warrant revised recommendations. Therefore, BAGG cannot accept responsibility for the recommendations contained in this report if we are not retained to provide observation and testing services during construction.

## CLOSURE

This report has been prepared in accordance with generally accepted engineering geology and geotechnical engineering practices for the strict use of Stevens Creek Quarry in Cupertino, and other professionals associated with the specific project described in this report. The recommendations presented in this report are based on our understanding of the proposed project as depicted and described on the Reclamation Plans prepared and provided to us by Benchmark Resources (2019). Furthermore, the conclusions and recommendations contained in this report are based on the observations of our CEG, review of available published geologic literature developed by the U.S. Geological Survey and CGS and site-specific studies prepared by other consultants, limited laboratory testing, stability analysis results, and our experience with similar projects in Northern California. No subsurface exploration was performed as part of our current scope of work. It is not uncommon for unanticipated conditions to be encountered during site grading and it is not possible for all such variations to be detected by our limited program for this type of project. The recommendations contained in this report are therefore contingent upon the review of the final grading and drainage plans by this office, and upon geotechnical observation and testing by BAGG Engineers of all pertinent aspects of site grading, including placement of subdrains, fills and backfills.

Subsurface conditions and standards of practice change with time. Therefore, we should be consulted to update this report, if grading and construction does not commence within five years from the date of this report provided the site conditions, the building code or standard of practice in this area do not change significantly. Additionally, the recommendations of this report are only valid for the proposed project as described herein. If the proposed project is modified, our recommendations should be reviewed and approved or modified by this office in writing.

We trust this letter report provides you with the information required at this time. If you have any questions, please feel free to contact us.

Very truly years,
**BAGG Engineers**

Sadek M. Derrega, PG, CEG #2175
Principal Engineering Geologist

Mike Matusich, PE, GE #3013
Senior Geotechnical Engineer

SMD/MM

Attachments:

**Plates**

| | |
|---|---|
| Plate 1 | Vicinity Map |
| Plate 2 | Site Plan-Cut Phase, Cross Sections & Section Lines |
| Plate 3 | Site Plan-Fill Phase, Cross Sections & Section Lines |
| Plate 4 | Site Plan-Cut Phase, Location of Section Lines |
| Plate 5 | Site Plan-Fill Phase, Location of Section Lines |
| Plate 6 | Aerial Site Plan and Geology |
| Plate 7 | Area Geology Map |
| Plate 8 | Site Photo |
| Plate 9 | Geologic Cross Section 1-1’ Showing Prominent Landslide |
| Plate 10 | Analyzed Geologic Cross Section 1-1’ |
| Plate 11 | Analyzed Geologic Cross Section 2-2’ |
| Plate 12 | Analyzed Geologic Cross Section 3-3’ |
| Plate 13 | Analyzed Geologic Cross Section 4-4’ |
| Plate 14 | Analyzed Geologic Cross Section 5-5’ |

| | |
|---|---|
| Plate 15 | Analyzed Geologic Cross Section 6-6' |
| Plates 16 through 33 | Slope Stability Analysis Plots |

**Appendix A**

Atterberg Limits Test Results
Drained Residual and peak Torsional Shear Strengths (clayey gouge)
Remolded Shear Strength Test Data (Norfleet Consultants, 2008)

ASFE document titled "Important Information About Your Geotechnical Engineering Report"

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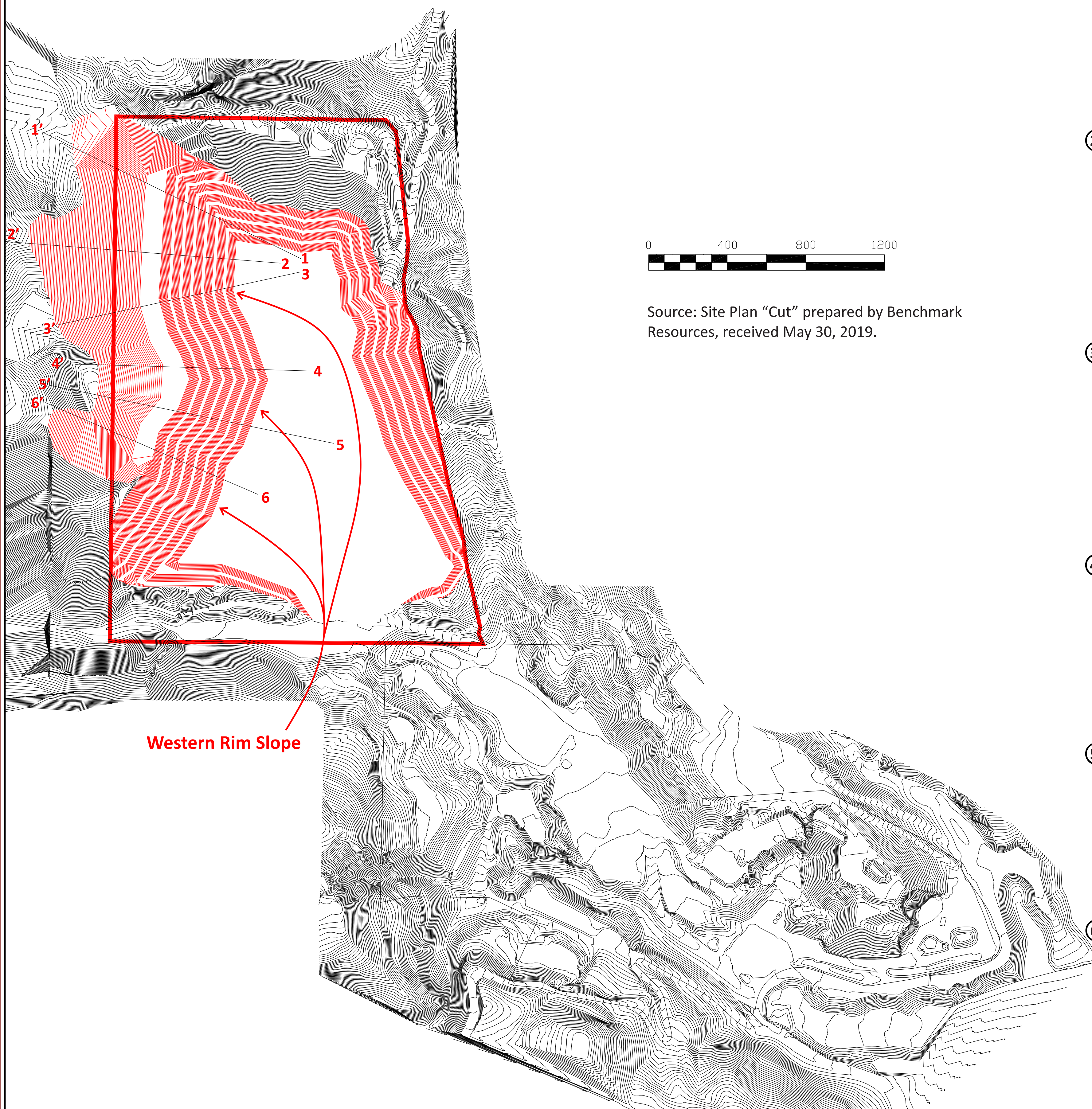
*Base Map: Google Earth*

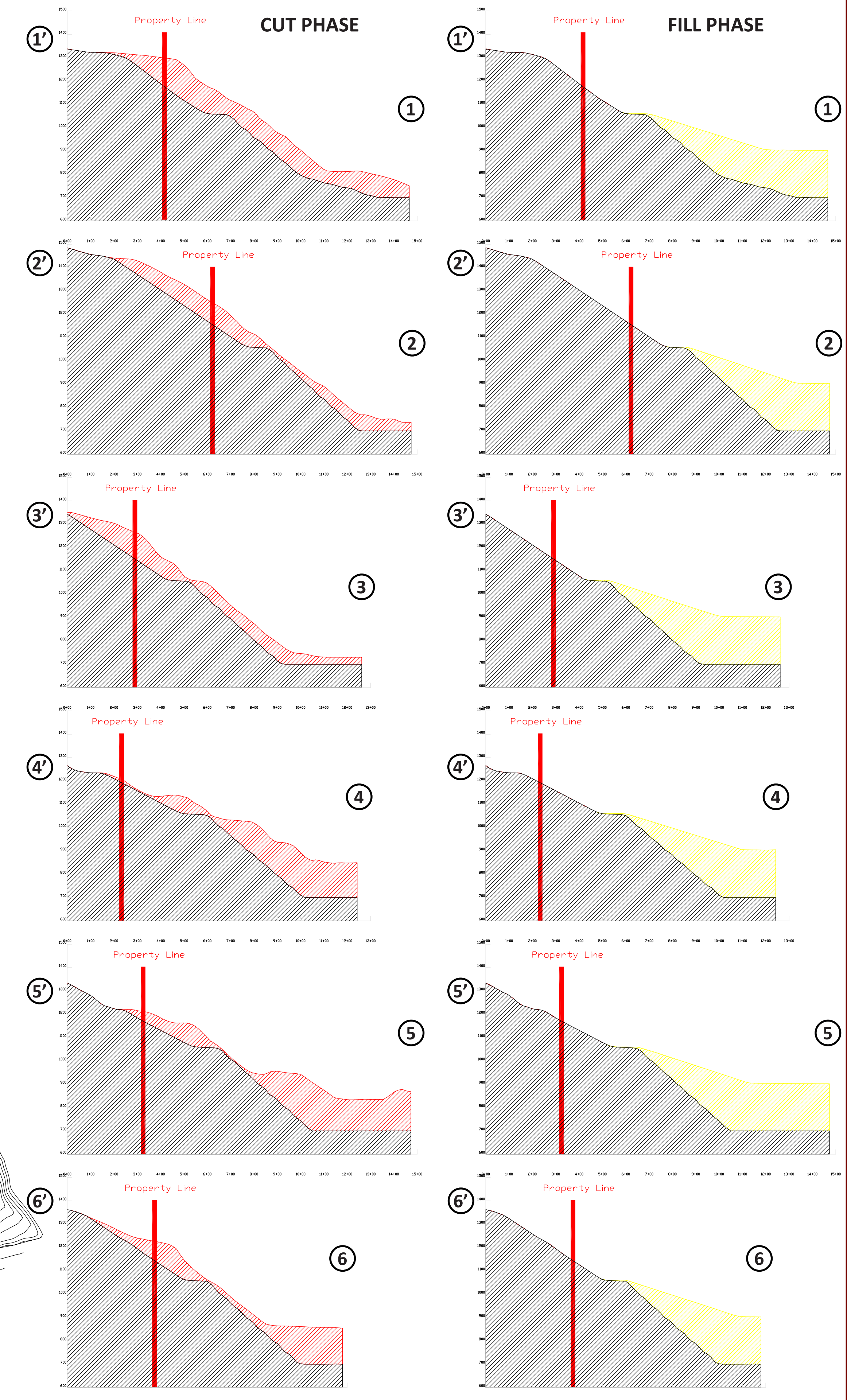
**ENGINEERING GEOLOGIC SLOPE CHARACTERIZATION, STABILITY ANALYSIS AND GEOTECHNICAL CONSULTATION**
**WESTERN RIM SLOPE**
**STEVENS CREEK QUARRY**
**CUPERTINO, CALIFORNIA 95014**

# VICINITY MAP

| DATE: | JOB NUMBER: | PLATE |
|---|---|---|
| December 2019 | STEVE-18-02 | 1 |




Source: Site Plan “Cut” prepared by Benchmark Resources, received May 30, 2019.




# SITE PLAN- CUT PHASE
# CROSS SECTIONS & SECTION LINES

PLATE 2

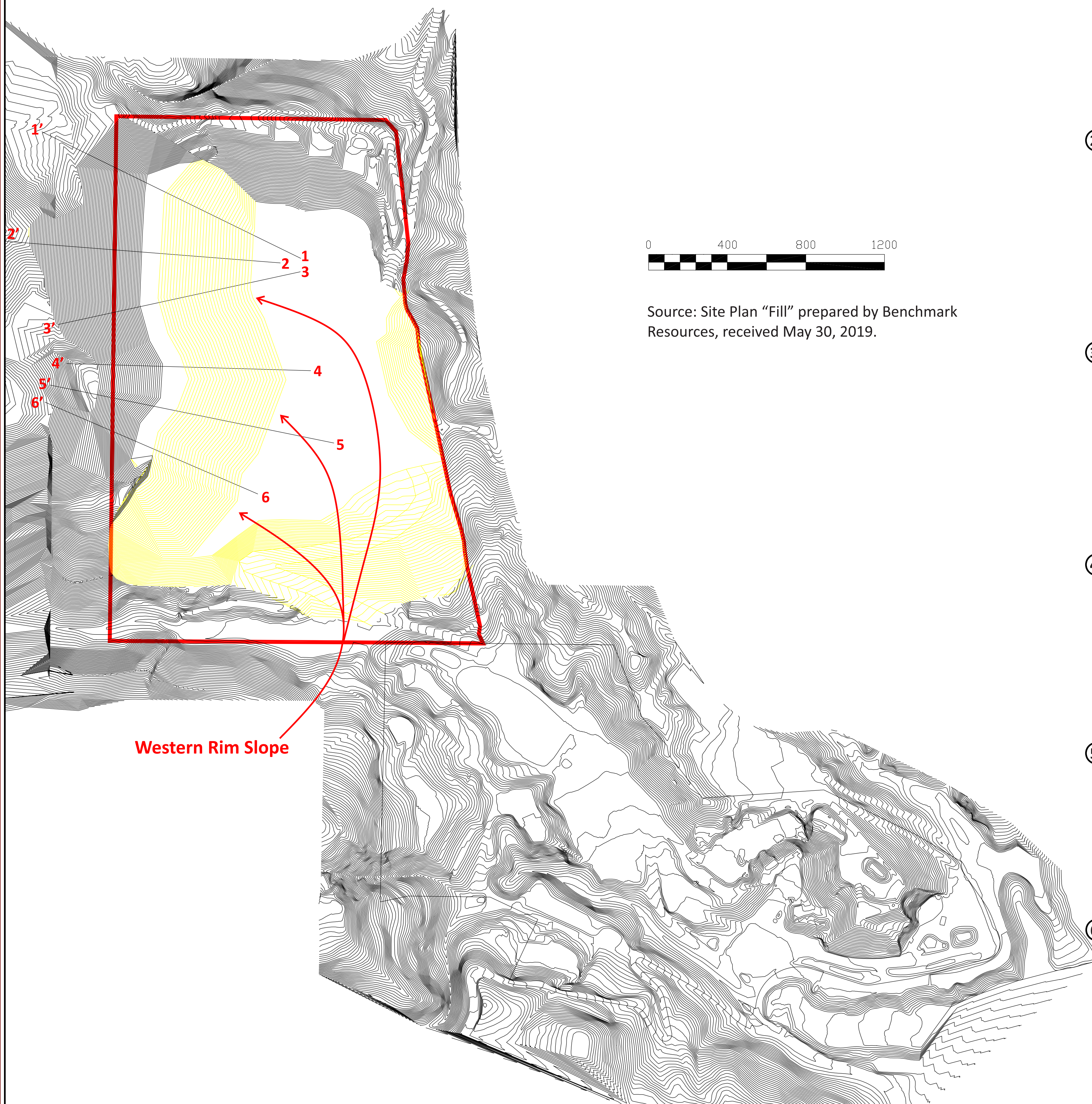
DATE December 2019

SCALE: 1” ≈ 600’

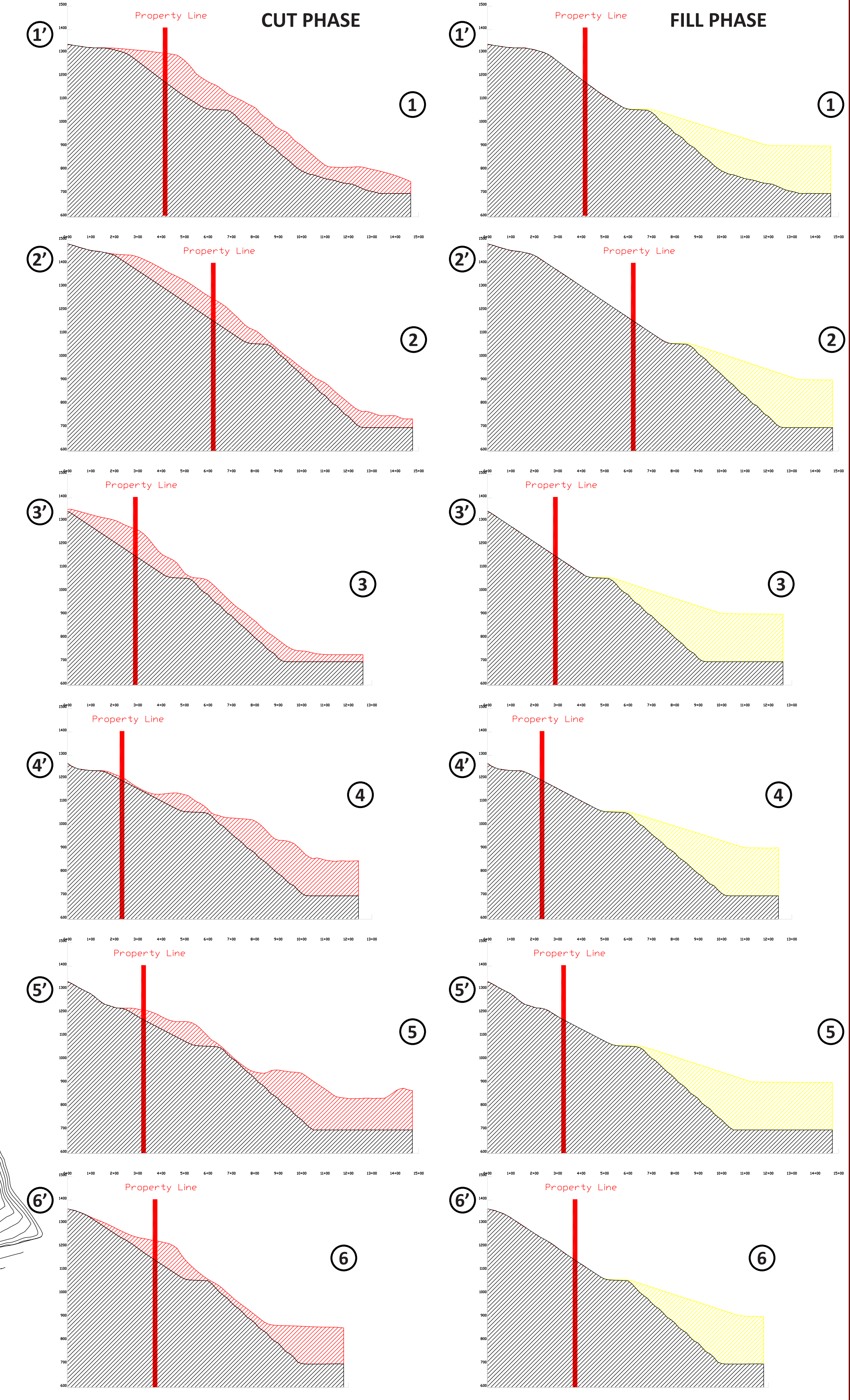
JOB NO.: STEVE-18-02

BAGG ENGINEERS

ENGINEERING GEOLOGIC SLOPE CHARACTERIZATION,
STABILITY ANALYSIS AND GEOTECHNICAL CONSULTATION
WESTERN RIM SLOPE
STEVENS CREEK QUARRY
CUPERTINO, CALIFORNIA 95014




Source: Site Plan "Fill" prepared by Benchmark Resources, received May 30, 2019.




# SITE PLAN- FILL PHASE CROSS SECTIONS & SECTION LINES

PLATE 3

DATE December 2019

SCALE: 1" ≈ 600'

JOB NO.: STEVE-18-02

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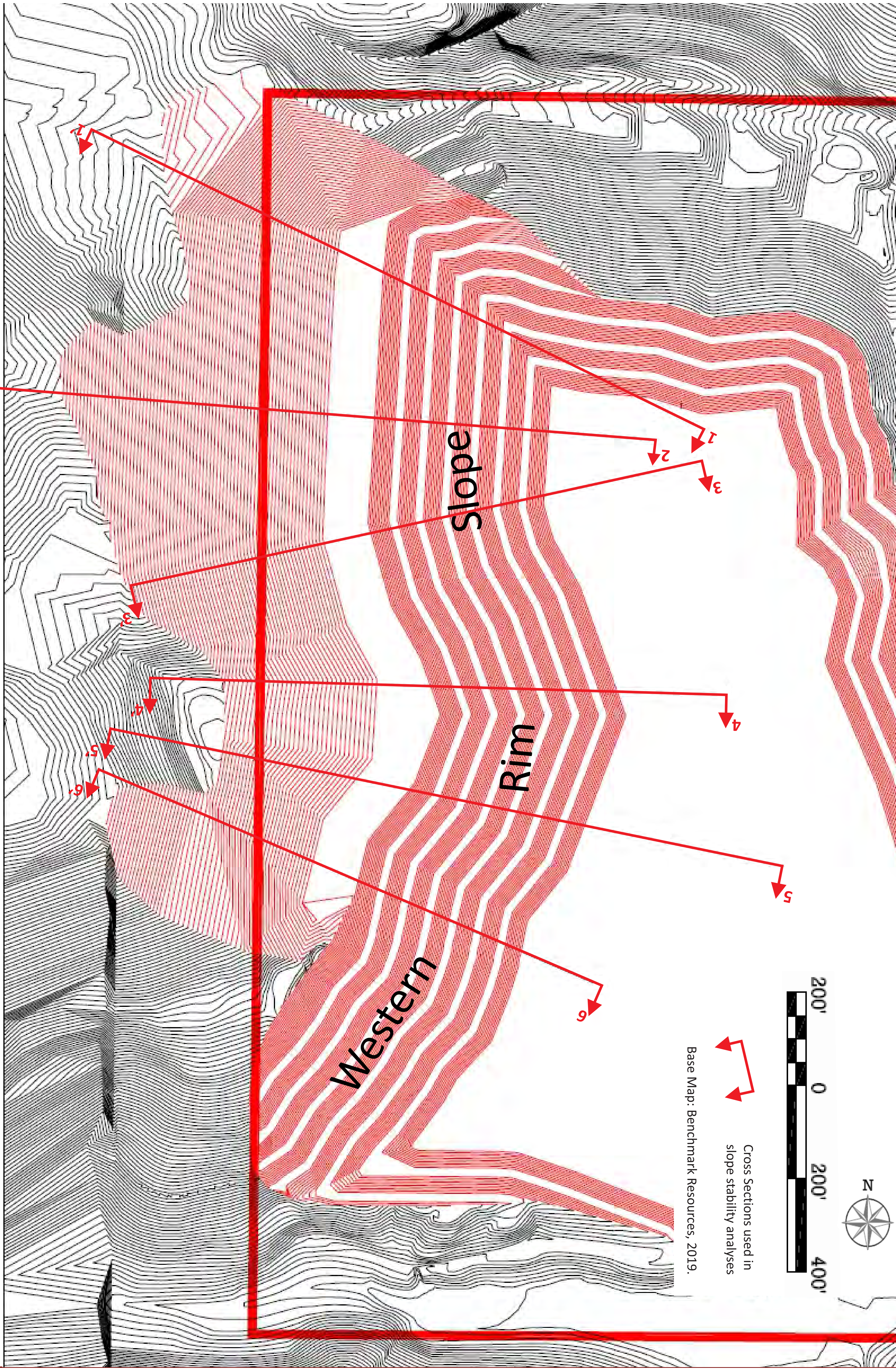
WESTERN RIM SLOPE
STEVENS CREEK QUARRY
CUPERTINO, CALIFORNIA 95014

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CUPERTINO, CALIFORNIA 95014

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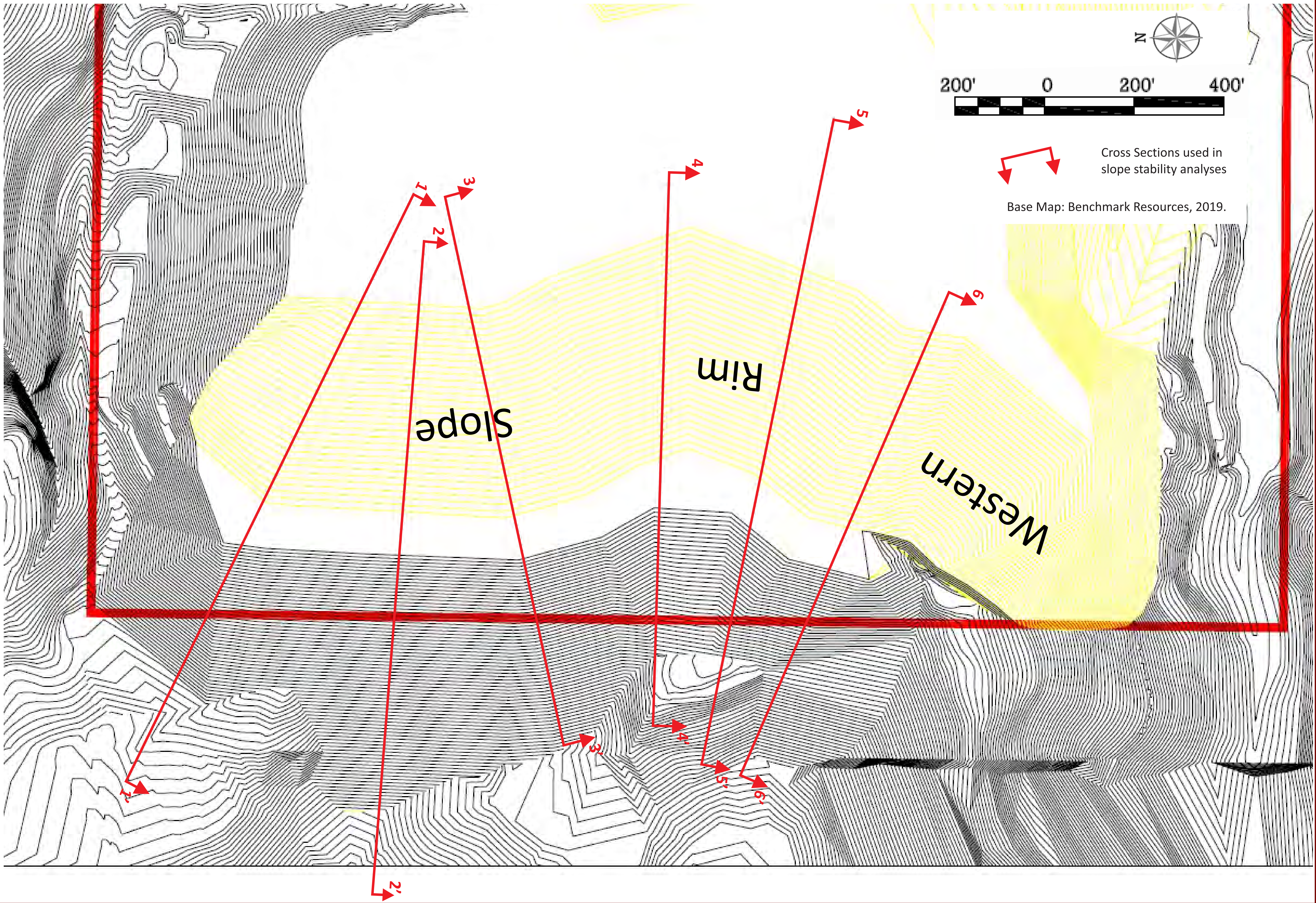
**SITE PLAN- CUT PHASE**
**LOCATION OF SECTION LINES**

| JOB NO.: | SCALE: | DATE | PLATE |
|---|---|---|---|
| STEVE-18-02 | 1" = 200' | December 2019 | 4 |




## SITE PLAN- FILL PHASE
## LOCATION OF SECTION LINES

| | |
|---|---|
| PLATE | 5 |
| DATE | December 2019 |
| SCALE: | 1" = 200' |
| JOB NO.: | STEVE-18-02 |

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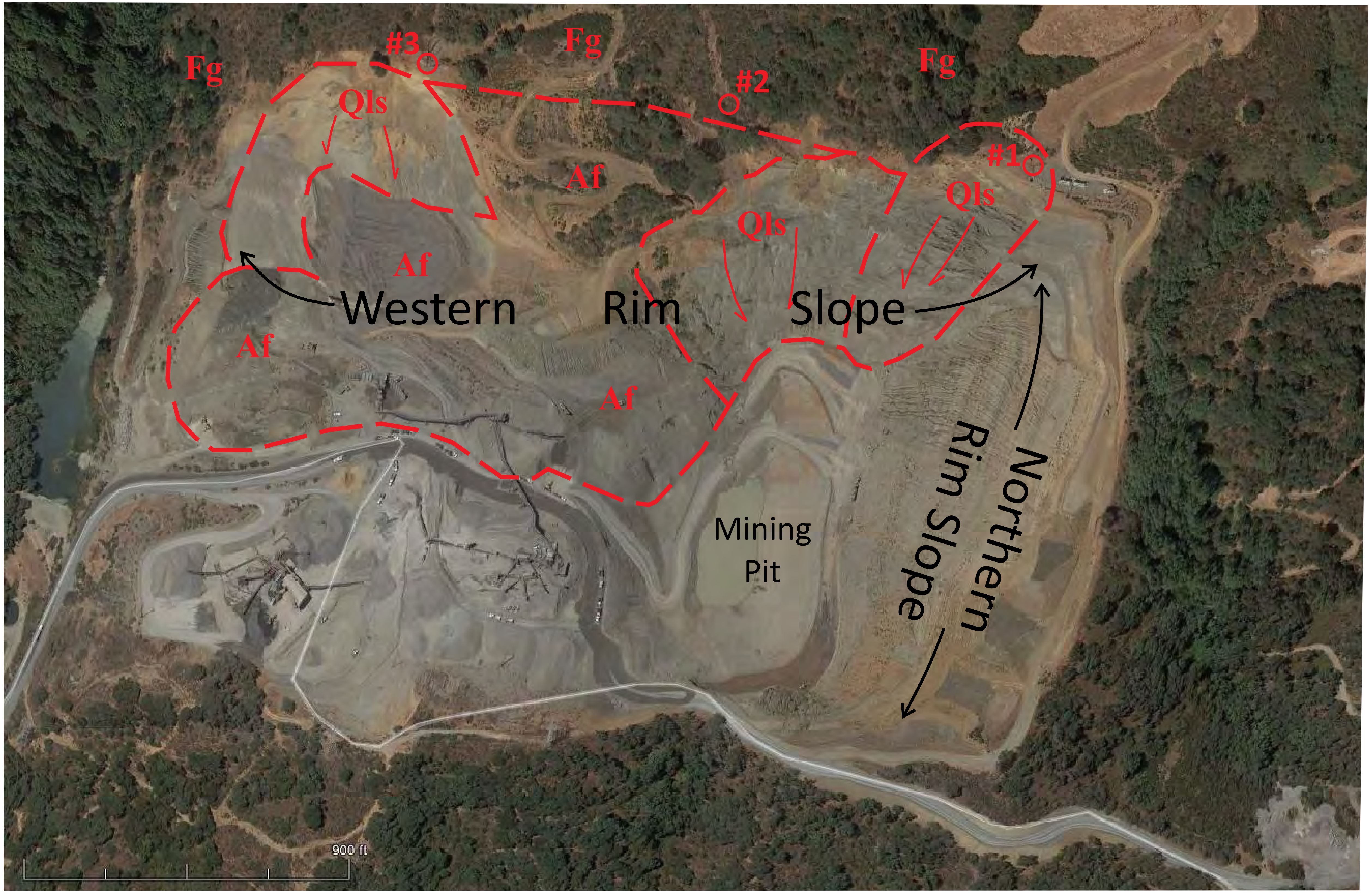


**Af** Artificial Fill

**Qls** Landslide

**Fg** Franciscan Complex Greenstone

○ **#1** PG&E Installation

Approximate Geologic Contact

Base Map:
*Google Earth Image*

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WESTERN RIM SLOPE
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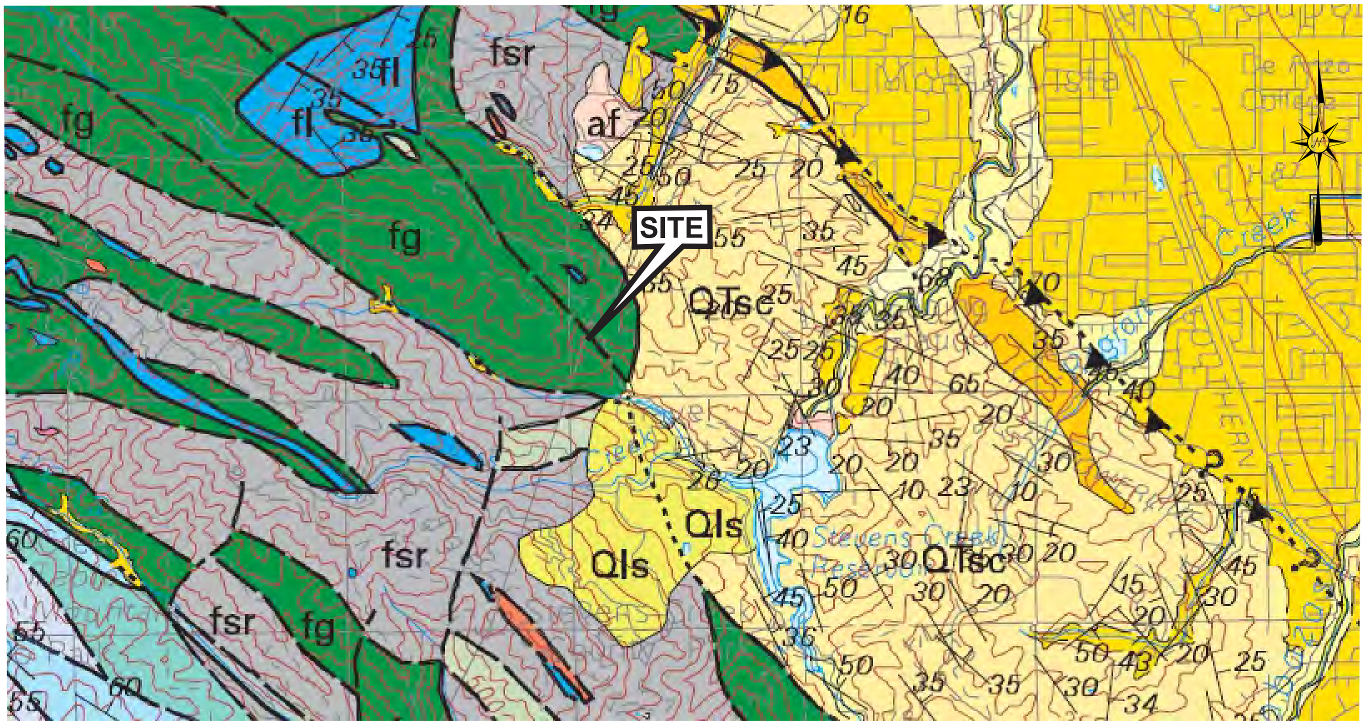
**AERIAL SITE PLAN AND GEOLOGY**

| JOB NUMBER: | SCALE: | DATE: | PLATE: |
|---|---|---|---|
| STEVE-18-02 | as indicated | December 2019 | 6 |




## LEGEND

**af** **Artificial Fill (Historic) --** Loose to very well consolidated gravel, silt, sand, clay, rock fragments, organic matter, and man-made debris in various combinations. Thickness is variable and may exceed 30 meters in places. Some is compacted and quite firm, but fill made before 1965 is nearly everywhere not compacted and consists simply of dumped materials.

**Qls** **Landslide Deposits (Pleistocene and/or Holocene) --** Poorly sorted clay, silt, sand and gravel. Only a few very large landslides have been mapped. For a more complete map of landslide deposits, see Nilsen and other (1979).

**QTsc** **Santa Clara Formation (lower Pleistocene and upper Pliocene) --** Gray to red brown poorly indurated conglomerate, sandstone, and mudstone in irregular and lenticular beds. Conglomerate consists mainly of subangular to subrounded cobbles in a sandy matrix but locally includes pebbles and boulders. On Coal Mine Ridge, south of Portola Valley, conglomerate contains boulders of an older conglomerate as long as one meter. Gray to buff claystone and siltstone beds on Coal Mine Ridge, contain carbonized wood fragments as large as 60 cm in diameter. Included in Santa Clara Formation are similar coarse-grained clastic deposits near Burlingame. Sarna-Wojcicki (1976) found a tuff bed in Santa Clara Formation near Woodside, and correlated it with a similar tuff in the Merced Formation. Later work indicated that the tuff correlates with the 435 ka Rockland ash (Sarna-Wojcicki, oral comm., 1997). Thickness is variable but reaches a maximum of about 500 meters along Coal Mine Ridge.

**fg** **Greenstone of Franciscan Complex (Cretaceous and Jurrasic) --** Dark green to red altered basaltic rocks, including flows, pillow lavas, breccias, tuff breccias, tuffs, and minor related intrusive rocks, in unknownj proportions. Unit includes some Franciscan chert and limestone bodies that are too small to show on map. Greenstone crops out in lenticular bodies varying in thickness from a few meters to many hundreds of meters.

**fs** **Greenstone of Franciscan Complex (Cretaceous and Jurrasic) --** Dark green to red altered basaltic rocks, including flows, pillow lavas, breccias, tuff breccias, tuffs, and minor related intrusive rocks, in unknownj proportions. Unit includes some Franciscan chert and limestone bodies that are too small to show on map. Greenstone crops out in lenticular bodies varying in thickness from a few meters to many hundreds of meters.

**fl** **Limestone of Franciscan Complex (Cretaceous and Jurrasic) --** Light gray, finely to coarsely crystalline limestone. In places limestone is unbedded, in other places it is distinctly bedded between beds of black chert. Limestone crops out in lenticular bodies up to 120 meters thick, in most places surrounded by Franciscan greenstone.

**fsr** **Shearerd Rock (melange) of Franciscan Complex (Cretaceous and Jurrasic) --** Predominantly graywacke, siltstone, and shale, substantial portions of which have been sheared, but includes hard blocks of all other Franciscan rock types. Total thickness of unit is unknown, but is probably at least several tens of meters.

***Reference:*** Geology of Palot Alto 30x60 Minute Quadrangle, California: A Digital Database by E.E. Brabb, R.W. Graymer, and D.L. Jones, Pamphlet Dervied From Digital Open-File Report 98-348

| ENGINEERING GEOLOGIC SLOPE CHARACTERIZATION, STABILITY ANALYSIS AND GEOTECHNICAL CONSULTATION<br>WESTERN RIM SLOPE<br>STEVENS CREEK QUARRY<br>CUPERTINO, CALIFORNIA 95014 | AREA GEOLOGY MAP | | |
|---|---|---|---|
| | DATE:<br>December 2019 | JOB NUMBER:<br>STEVE-18-02 | PLATE<br>7 |

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Base Map: Google Earth

| ENGINEERING GEOLOGIC SLOPE CHARACTERIZATION, STABILITY ANALYSIS AND GEOTECHNICAL CONSULTATION<br>WESTERN RIM SLOPE<br>STEVENS CREEK QUARRY<br>CUPERTINO, CALIFORNIA 95014 | SITE PHOTO | | |
|---|---|---|---|
| | DATE:<br>December 2019 | JOB NUMBER:<br>STEVE-18-02 | PLATE<br>8 |

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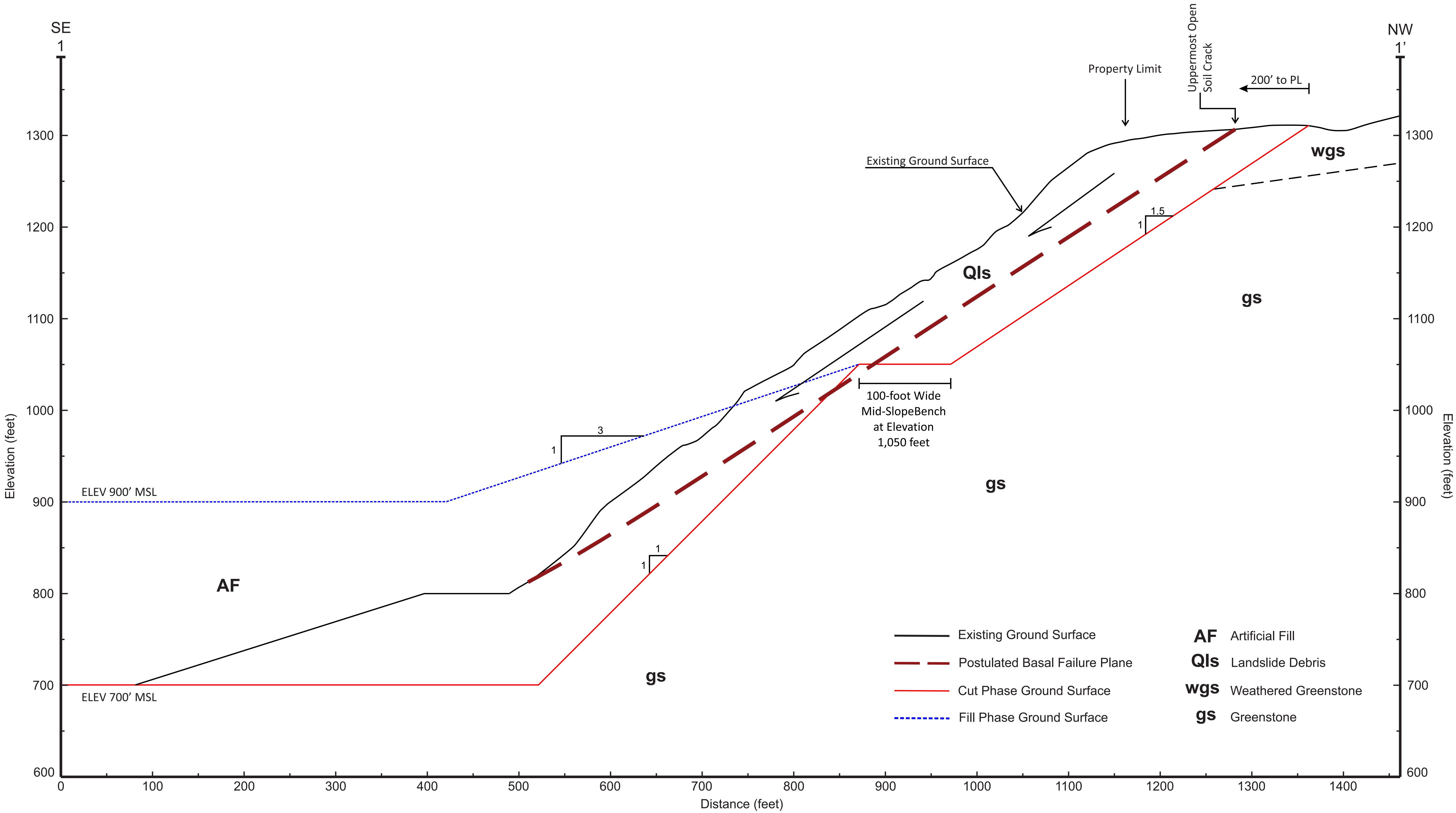


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**WESTERN RIM SLOPE**
**STEVENS CREEK QUARRY,**
**CUPERTINO, CALIFORNIA 95014**

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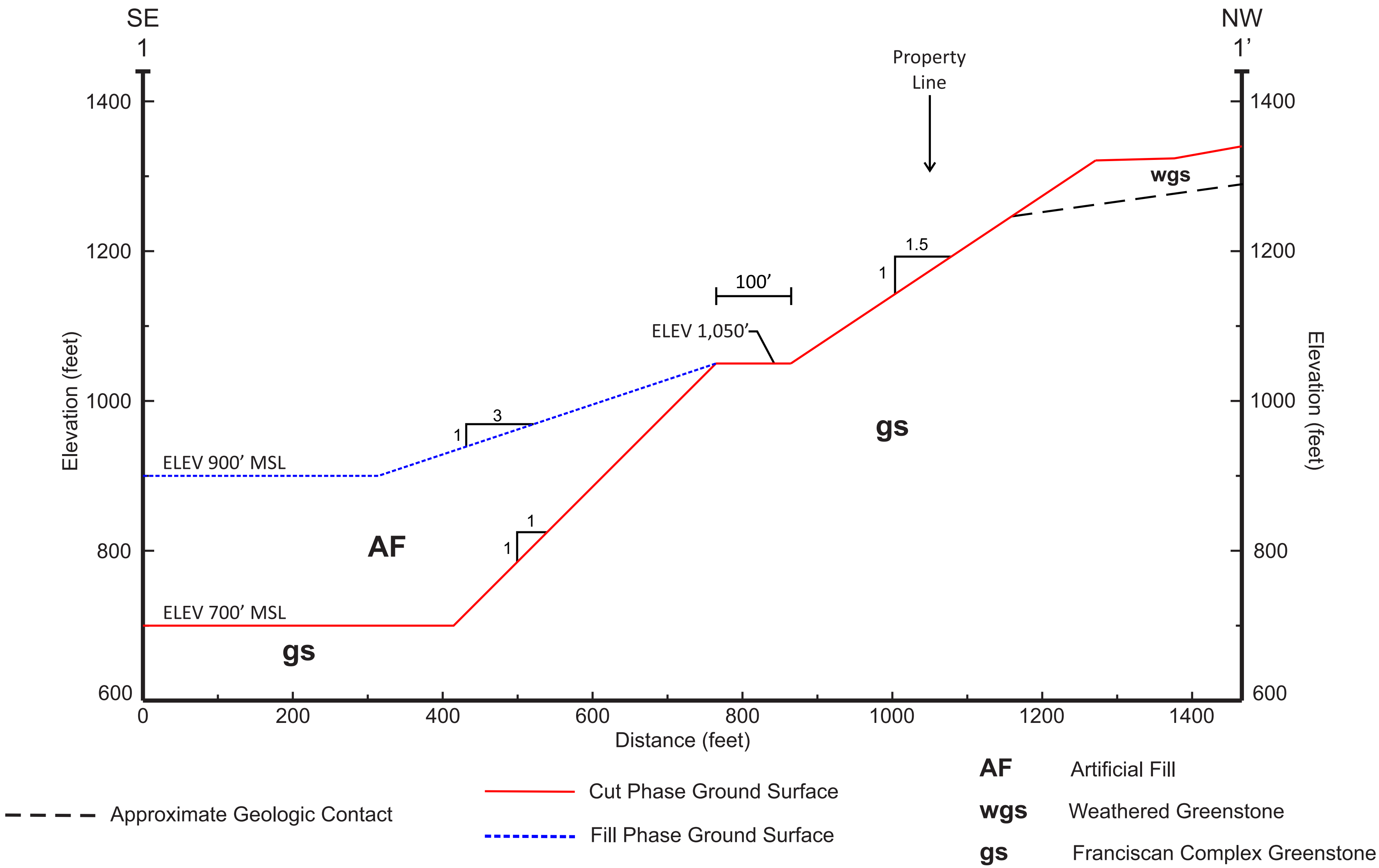
**GEOLOGIC CROSS SECTION 1-1'**
**SHOWING PROMINENT LANDSLIDE**

| JOB NUMBER: | SCALE: | DATE: | PLATE: |
|---|---|---|---|
| STEVE-18-02 | 1" = 100' | December 2019 | 9 |

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**STEVENS CREEK QUARRY,**
**CUPERTINO, CALIFORNIA 95014**

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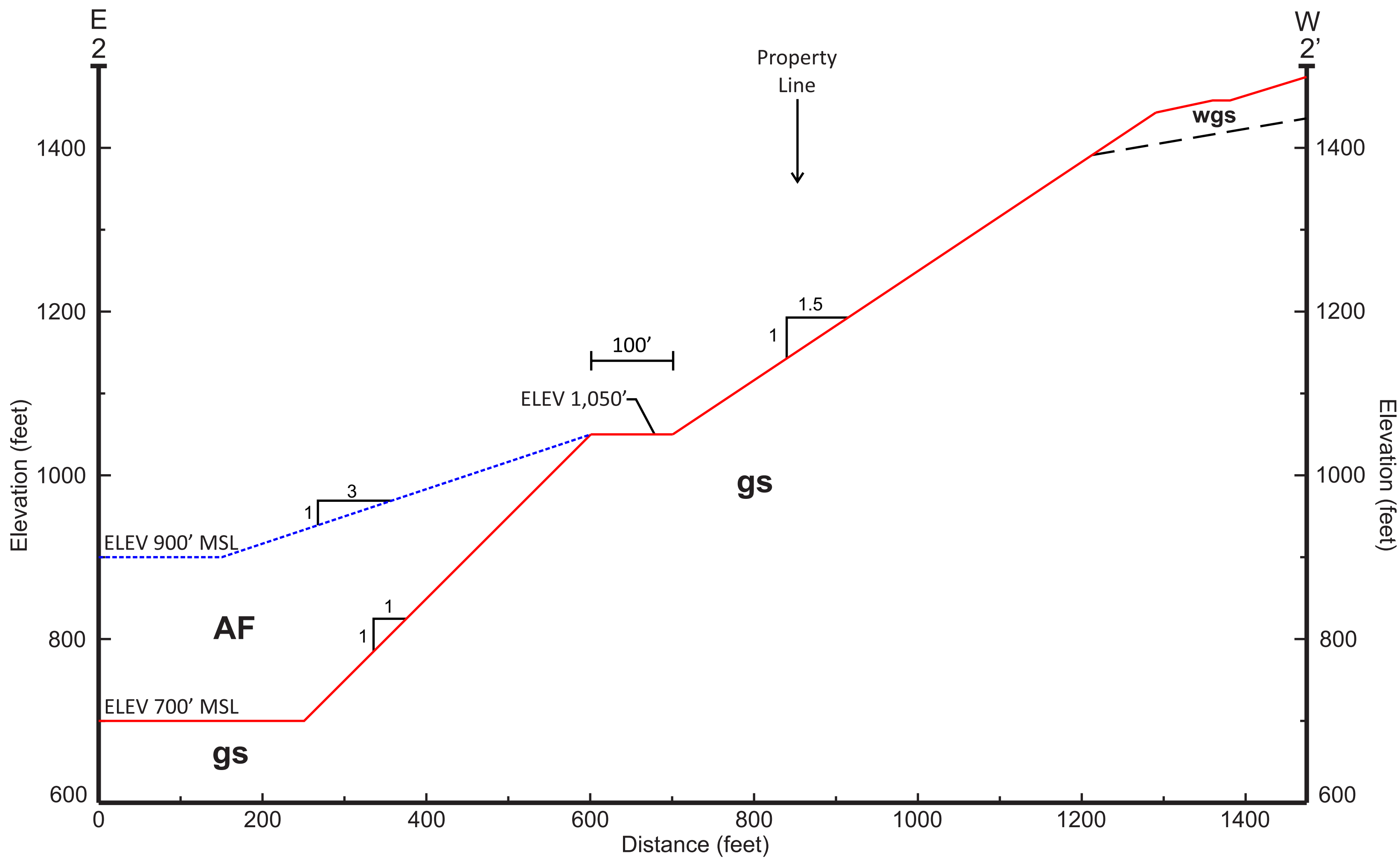
**ANALYZED GEOLOGIC CROSS SECTION 1-1'**

| DATE: | JOB NUMBER: | PLATE |
|---|---|---|
| December 2019 | STEVE-18-02 | 10 |




Approximate Geologic Contact

Cut Phase Ground Surface

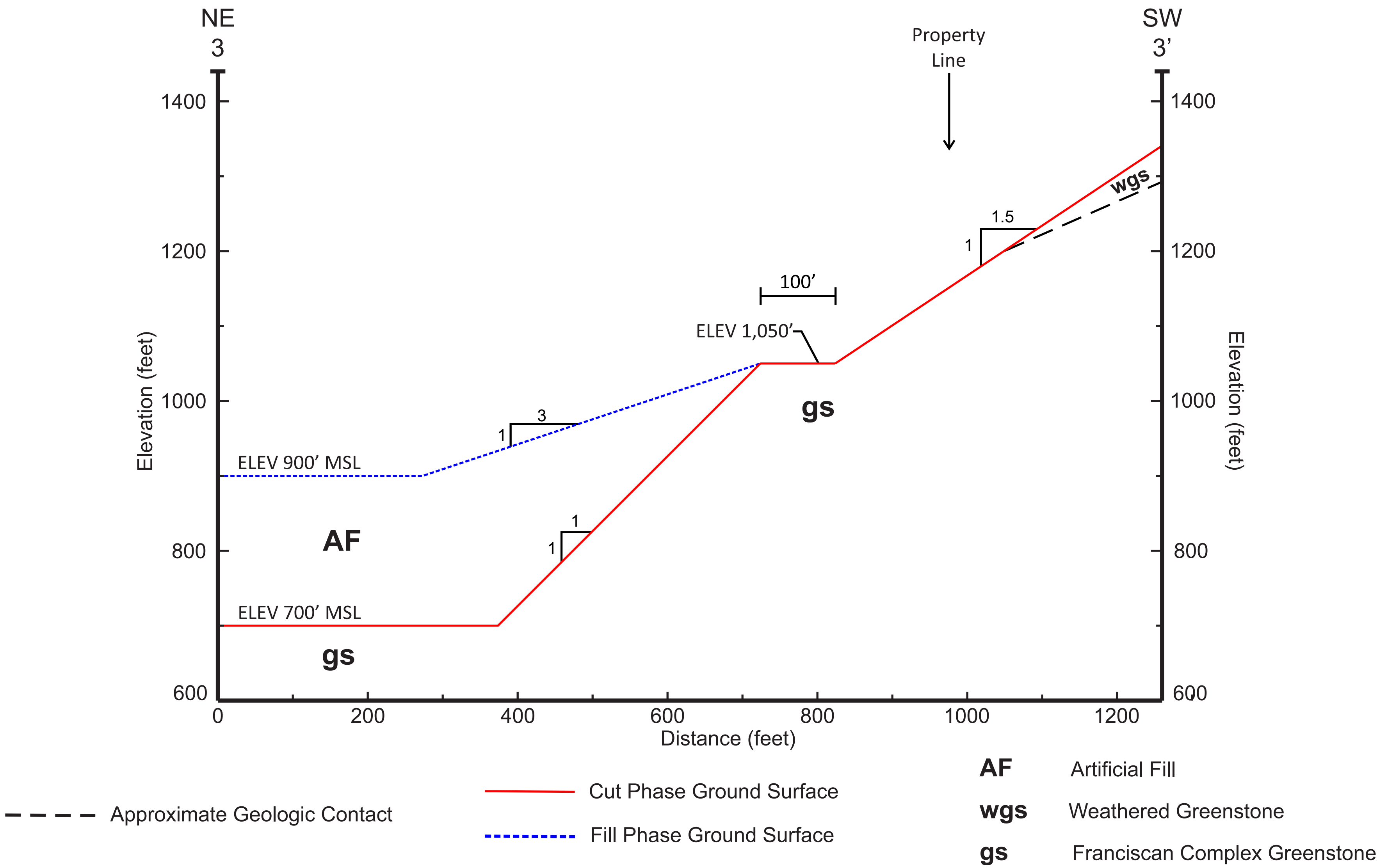
Fill Phase Ground Surface

**AF** Artificial Fill

**wgs** Weathered Greenstone

**gs** Franciscan Complex Greenstone

| | | | | |
|---|---|---|---|---|
| **ENGINEERING GEOLOGIC SLOPE CHARACTERIZATION, STABILITY ANALYSIS AND GEOTECHNICAL CONSULTATION WESTERN RIM SLOPE STEVENS CREEK QUARRY, CUPERTINO, CALIFORNIA 95014** | BAGG ENGINEERS | **ANALYZED GEOLOGIC CROSS SECTION 2-2'** | | |
| | | DATE: December 2019 | JOB NUMBER: STEVE-18-02 | PLATE 11 |

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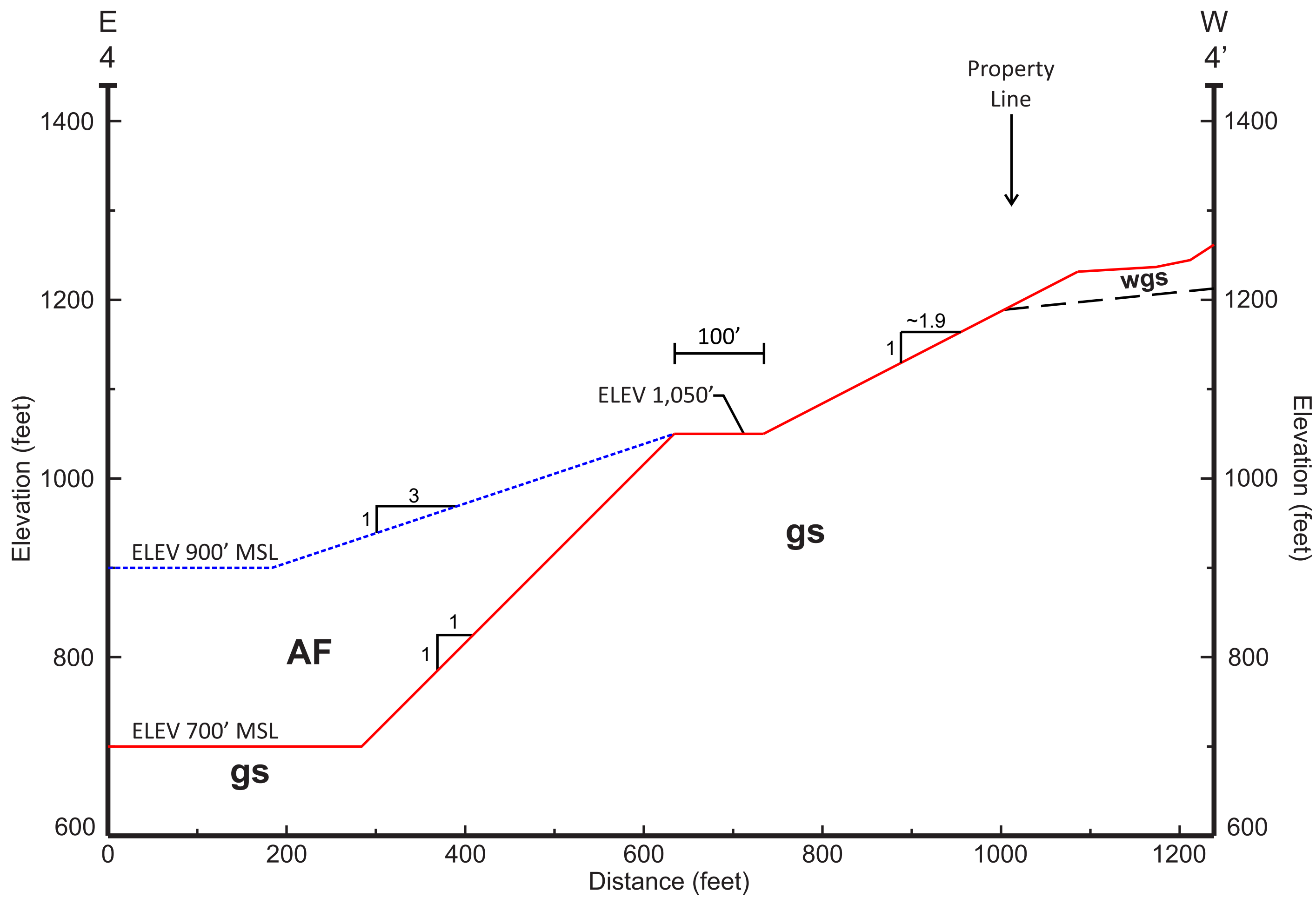
**ANALYZED GEOLOGIC CROSS SECTION 3-3'**

| DATE: | JOB NUMBER: | PLATE |
|---|---|---|
| December 2019 | STEVE-18-02 | 12 |




Approximate Geologic Contact

Cut Phase Ground Surface

Fill Phase Ground Surface

**AF** Artificial Fill

**wgs** Weathered Greenstone

**gs** Franciscan Complex Greenstone

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WESTERN RIM SLOPE
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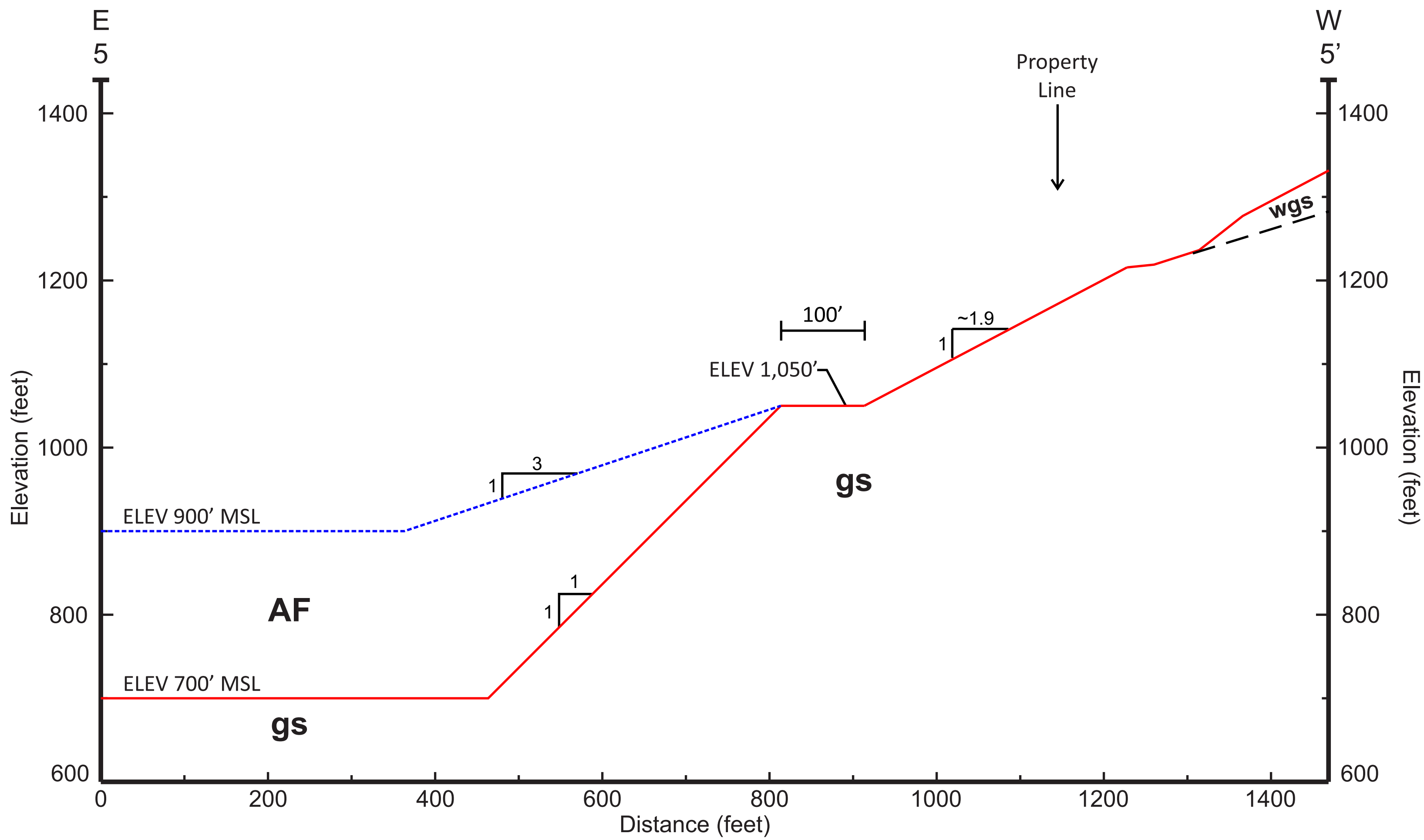
**ANALYZED GEOLOGIC CROSS SECTION 4-4’**

| DATE: | JOB NUMBER: | PLATE |
|---|---|---|
| December 2019 | STEVE-18-02 | 13 |




Approximate Geologic Contact

Cut Phase Ground Surface

Fill Phase Ground Surface

| | |
|---|---|
| **AF** | Artificial Fill |
| **wgs** | Weathered Greenstone |
| **gs** | Franciscan Complex Greenstone |

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**CUPERTINO, CALIFORNIA 95014**

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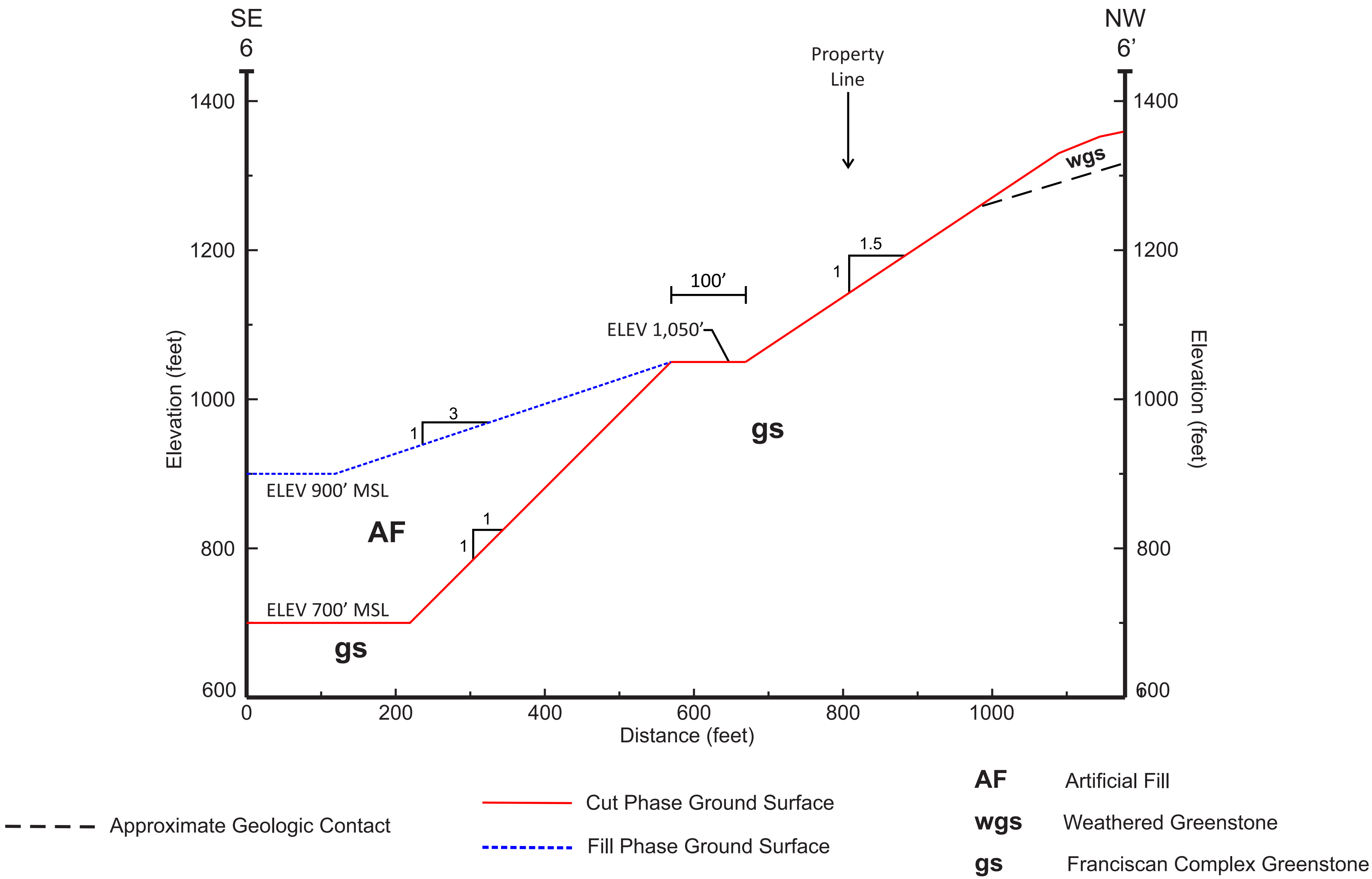
**ANALYZED GEOLOGIC CROSS SECTION 5-5'**

| DATE: | JOB NUMBER: | PLATE |
|---|---|---|
| December 2019 | STEVE-18-02 | 14 |

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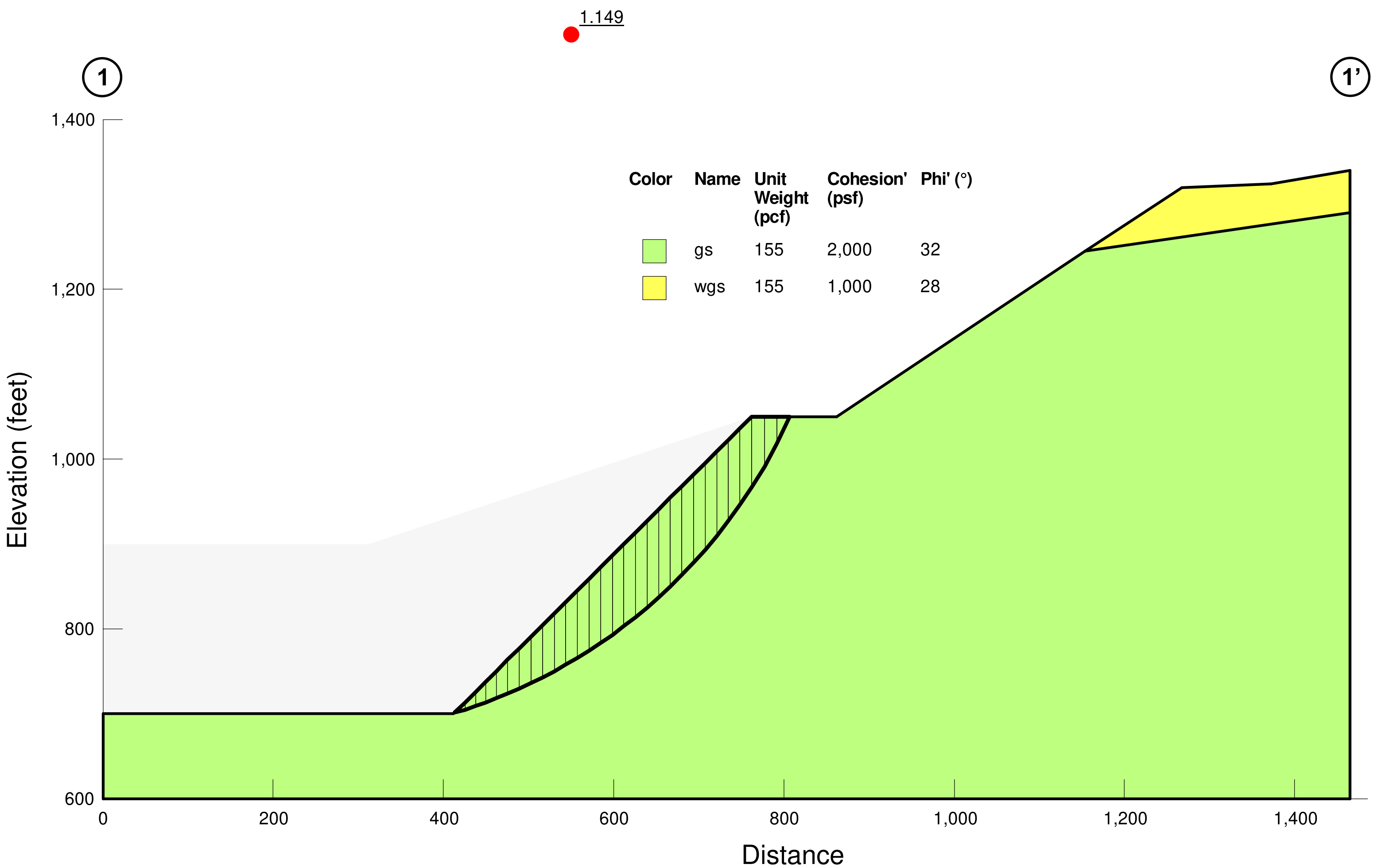
**ANALYZED GEOLOGIC CROSS SECTION 6-6'**

| DATE: | JOB NUMBER: | PLATE |
|---|---|---|
| December 2019 | STEVE-18-02 | 15 |

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**CUPERTINO, CALIFORNIA 95014**

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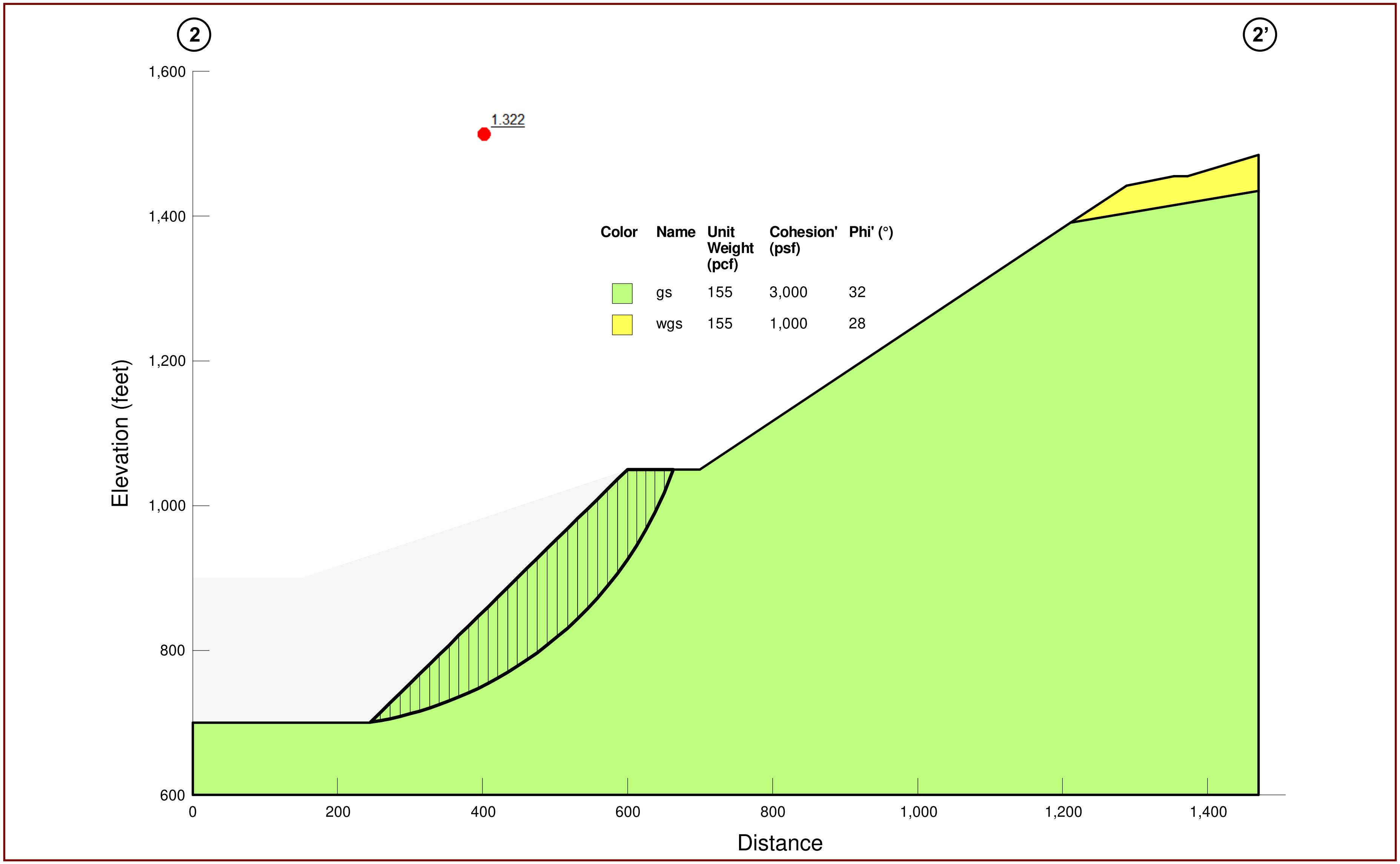
**SLOPE STABILITY ANALYSIS - CUT PHASE**
**CROSS SECTION 1-1', STATIC**

| DATE: | JOB NUMBER: | PLATE |
|---|---|---|
| December 2019 | STEVE-18-02 | 16 |

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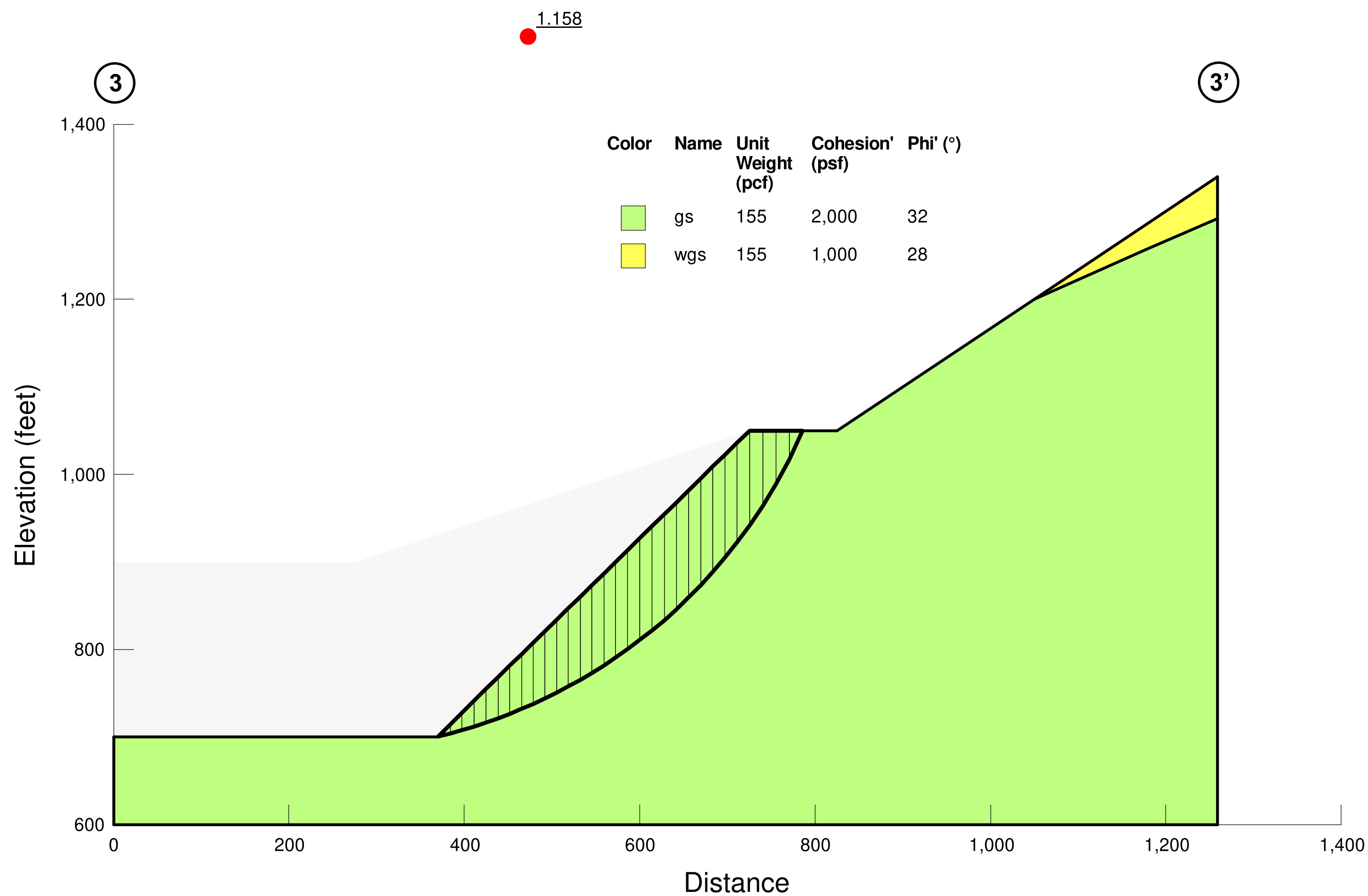
**SLOPE STABILITY ANALYSIS - CUT PHASE**
**CROSS SECTION 2-2', STATIC**

| DATE: | JOB NUMBER: | PLATE |
|---|---|---|
| December 2019 | STEVE-18-02 | 17 |




| Color | Name | Unit Weight (pcf) | Cohesion' (psf) | Phi' (°) |
|---|---|---|---|---|
| | gs | 155 | 2,000 | 32 |
| | wgs | 155 | 1,000 | 28 |

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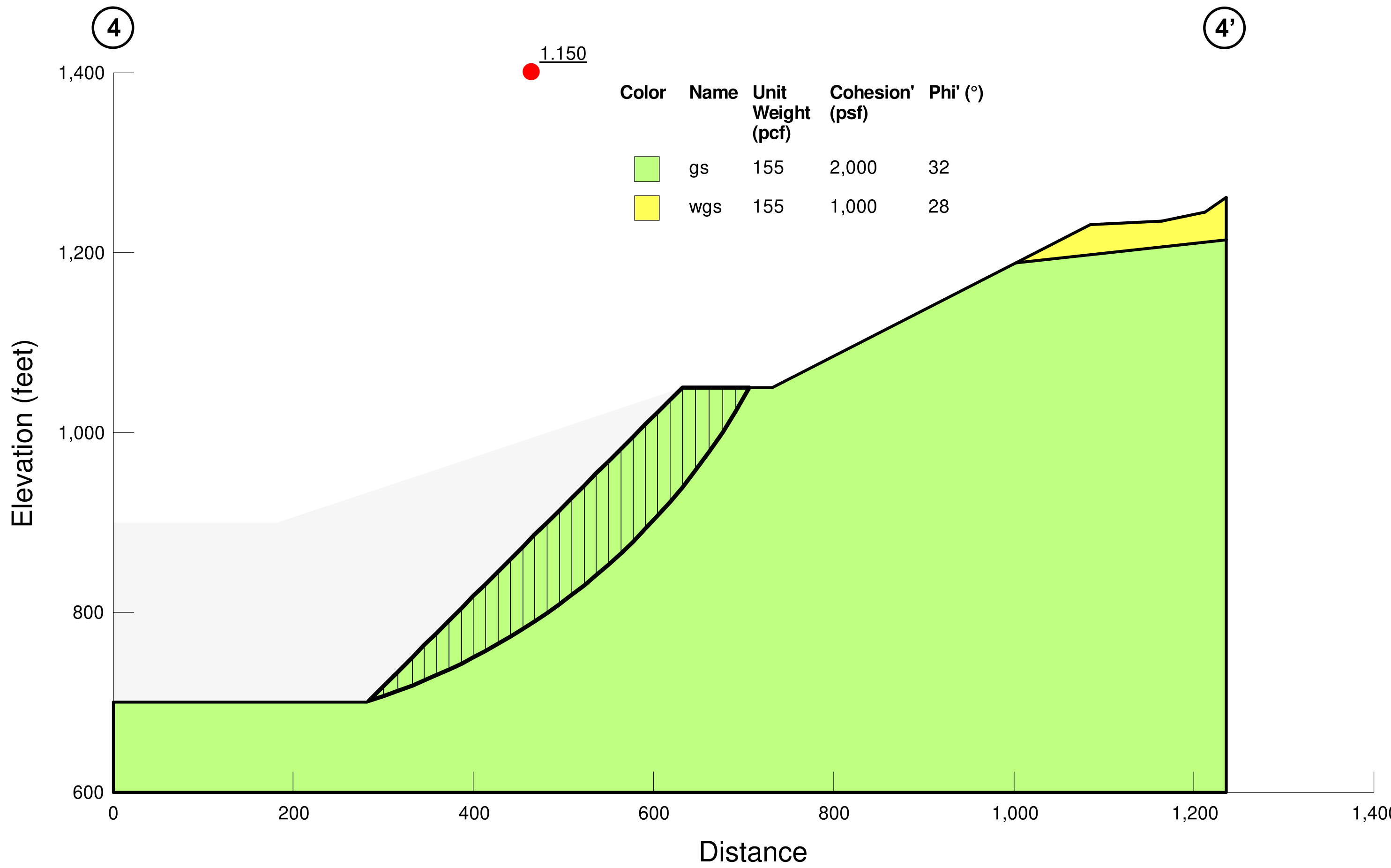
**SLOPE STABILITY ANALYSIS - CUT PHASE**
**CROSS SECTION 3-3', STATIC**

| DATE: | JOB NUMBER: | PLATE |
|---|---|---|
| December 2019 | STEVE-18-02 | 18 |

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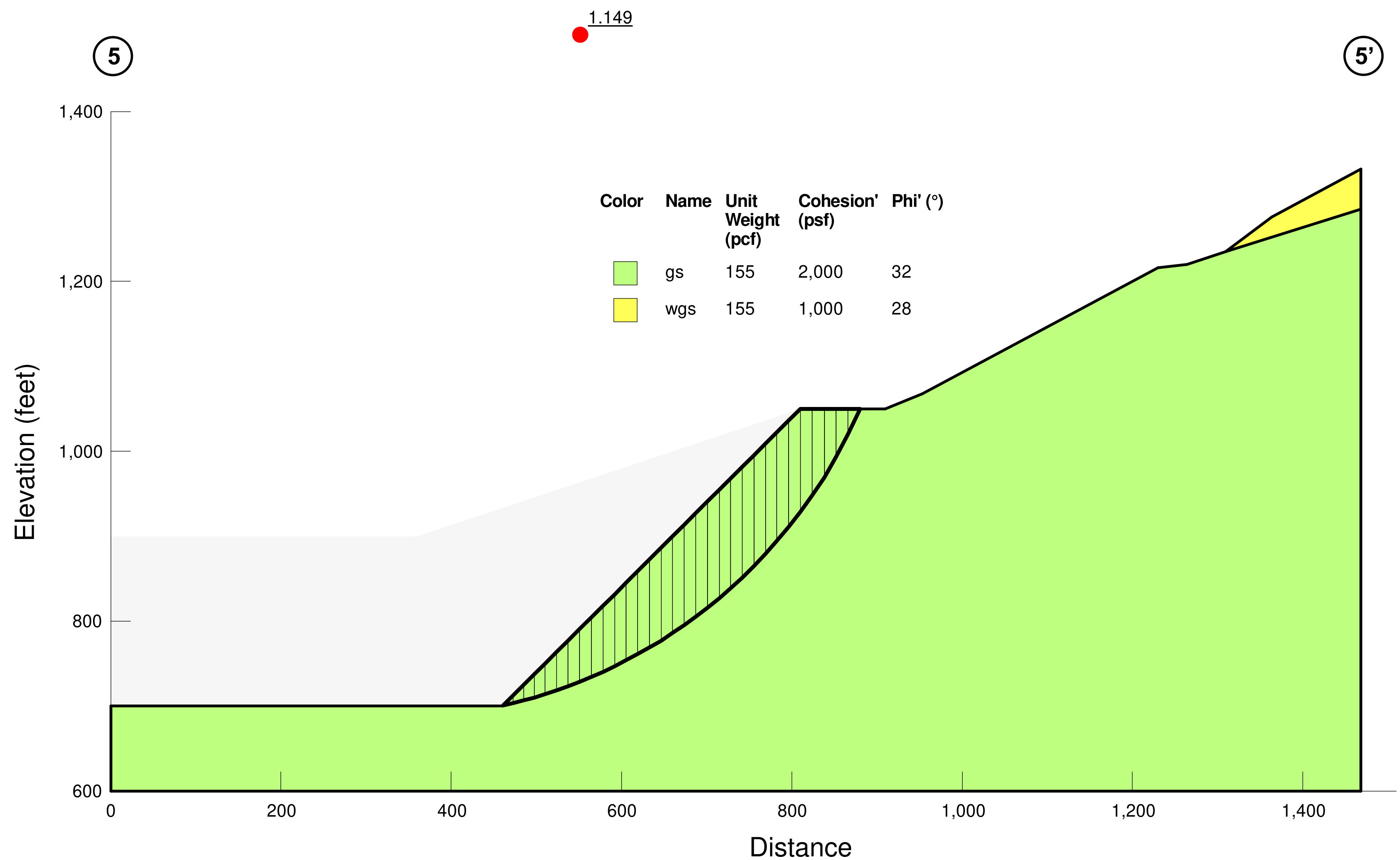
**SLOPE STABILITY ANALYSIS - CUT PHASE**
**CROSS SECTION 4-4', STATIC**

| DATE: | JOB NUMBER: | PLATE |
|---|---|---|
| December 2019 | STEVE-18-02 | 19 |

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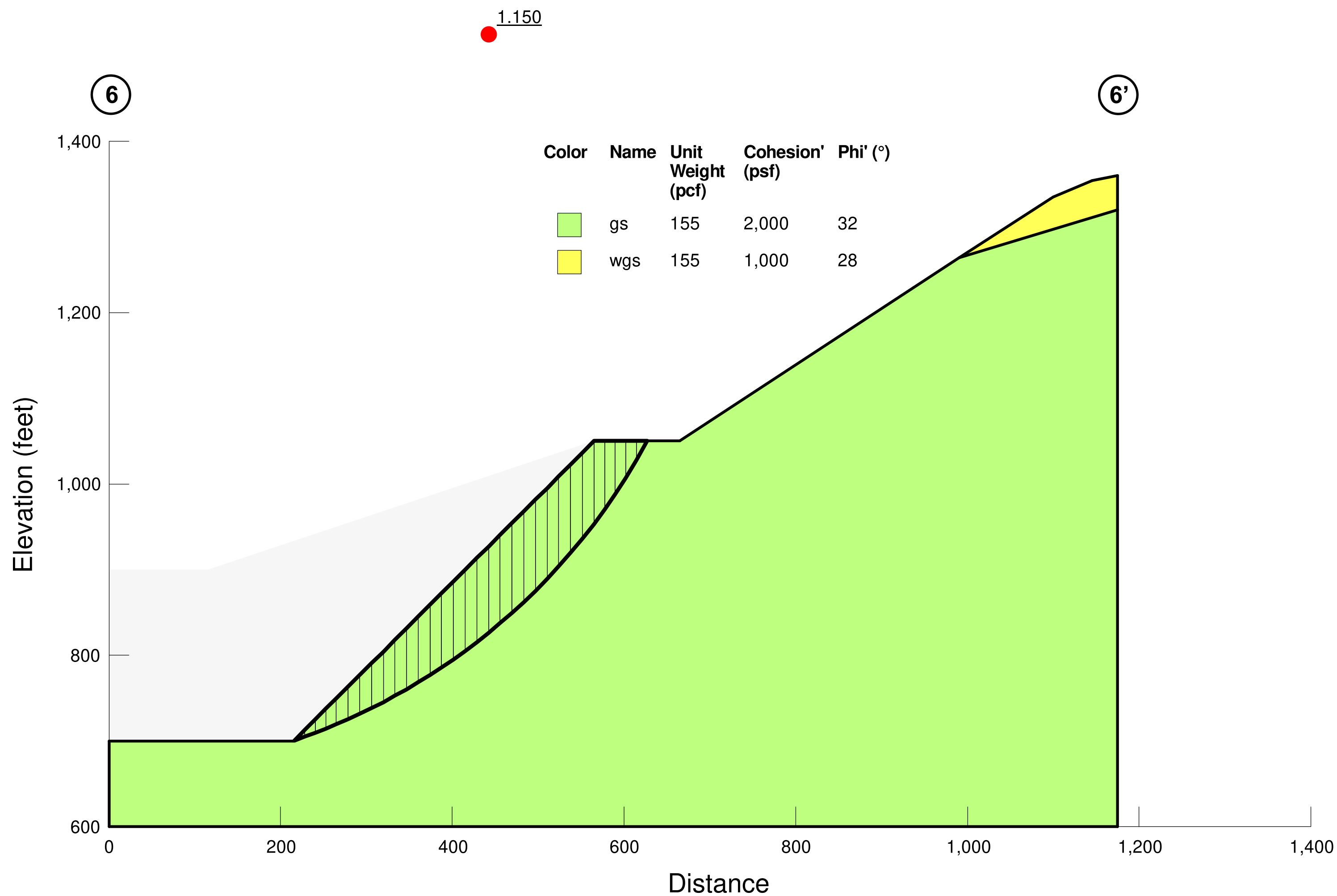
**SLOPE STABILITY ANALYSIS - CUT PHASE**
**CROSS SECTION 5-5', STATIC**

| DATE: | JOB NUMBER: | PLATE |
|---|---|---|
| December 2019 | STEVE-18-02 | 20 |

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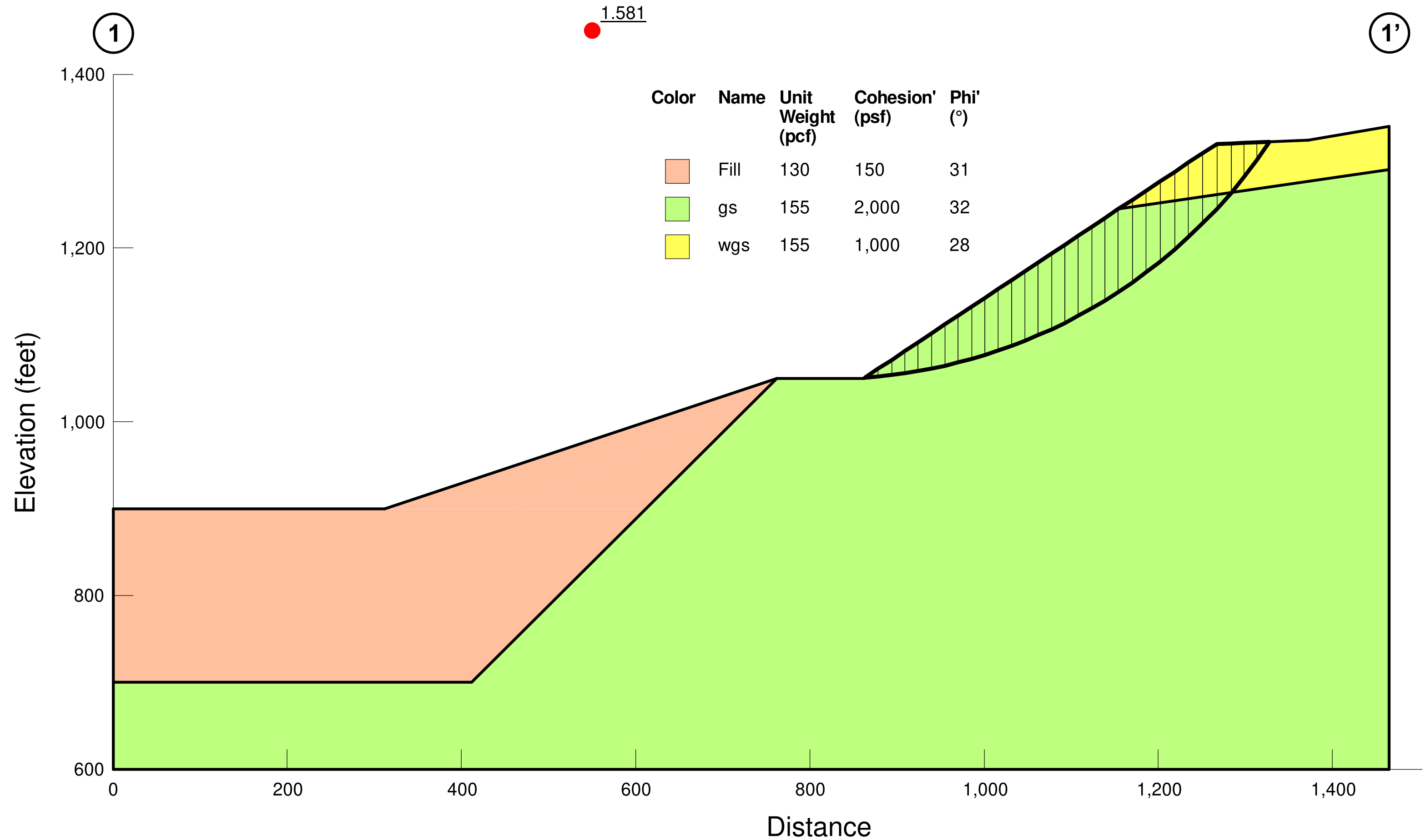
SLOPE STABILITY ANALYSIS - CUT PHASE
CROSS SECTION 6-6', STATIC

| DATE: | JOB NUMBER: | PLATE |
|---|---|---|
| December 2019 | STEVE-18-02 | 21 |

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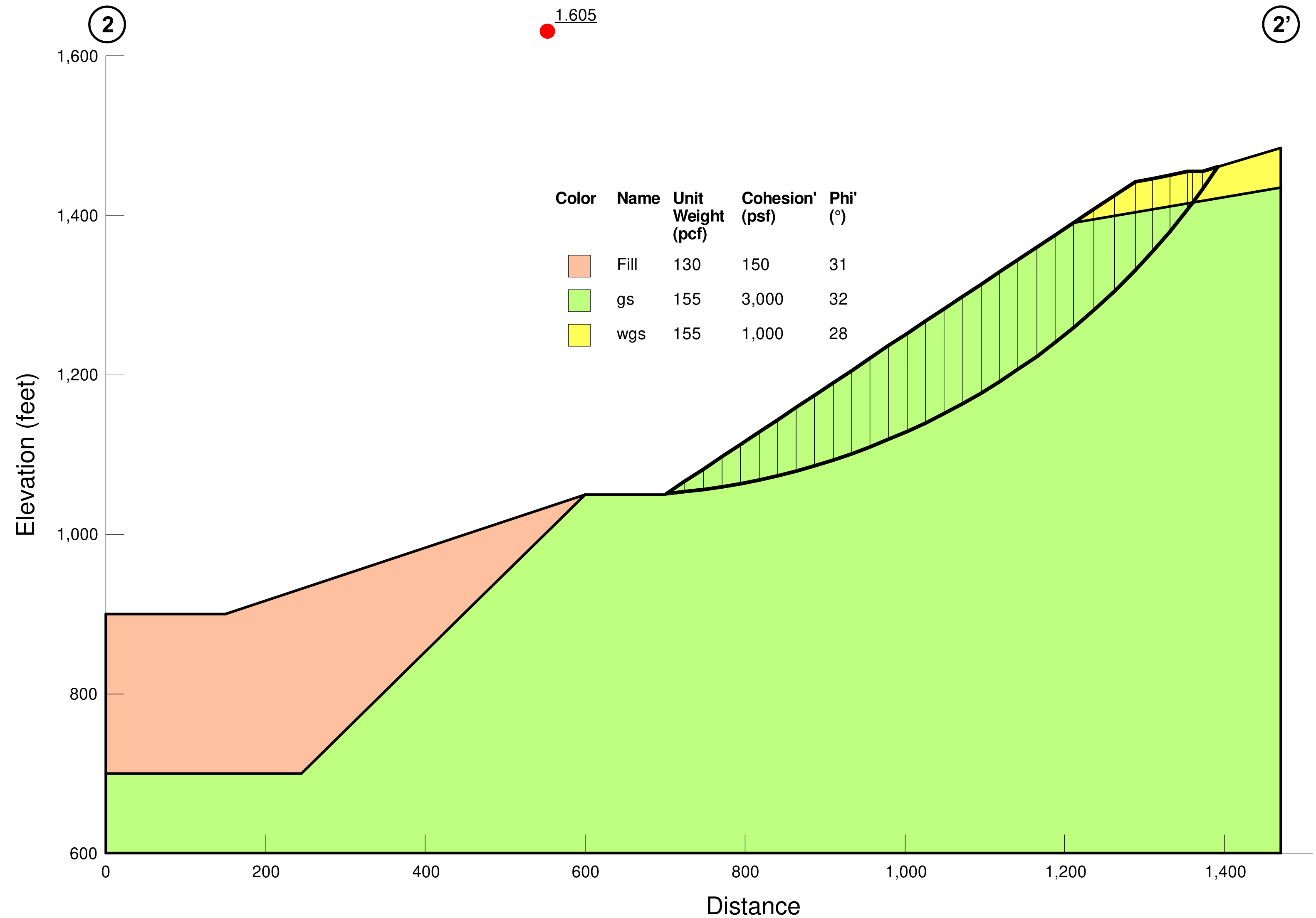
**SLOPE STABILITY ANALYSIS - FILL PHASE**
**CROSS SECTION 1-1', STATIC**

| DATE: | JOB NUMBER: | PLATE |
|---|---|---|
| December 2019 | STEVE-18-02 | 22 |

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CUPERTINO, CALIFORNIA 95014

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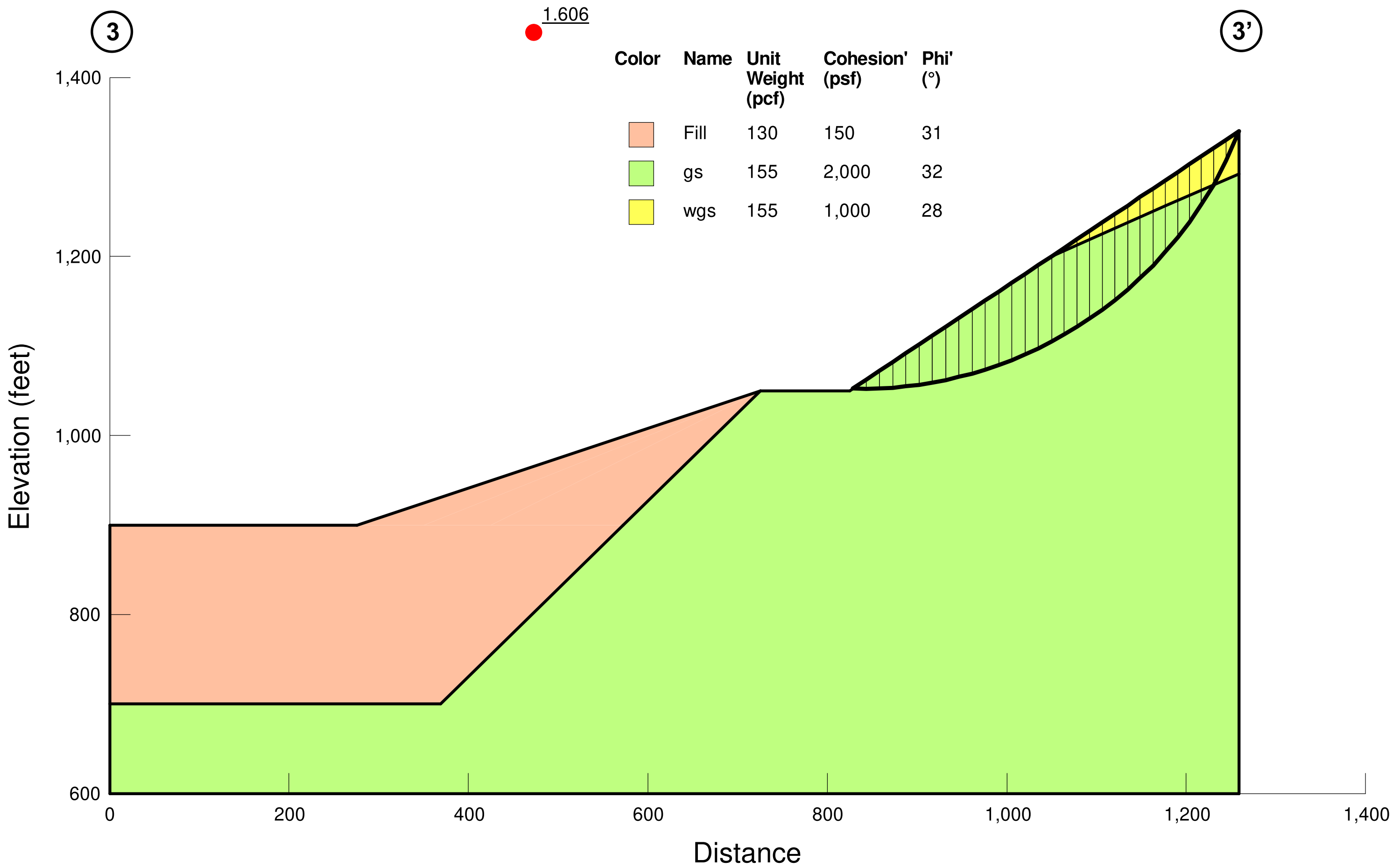
**SLOPE STABILITY ANALYSIS - FILL PHASE**
**CROSS SECTION 2-2', STATIC**

| DATE: | JOB NUMBER: | PLATE |
|---|---|---|
| December 2019 | STEVE-18-02 | 23 |

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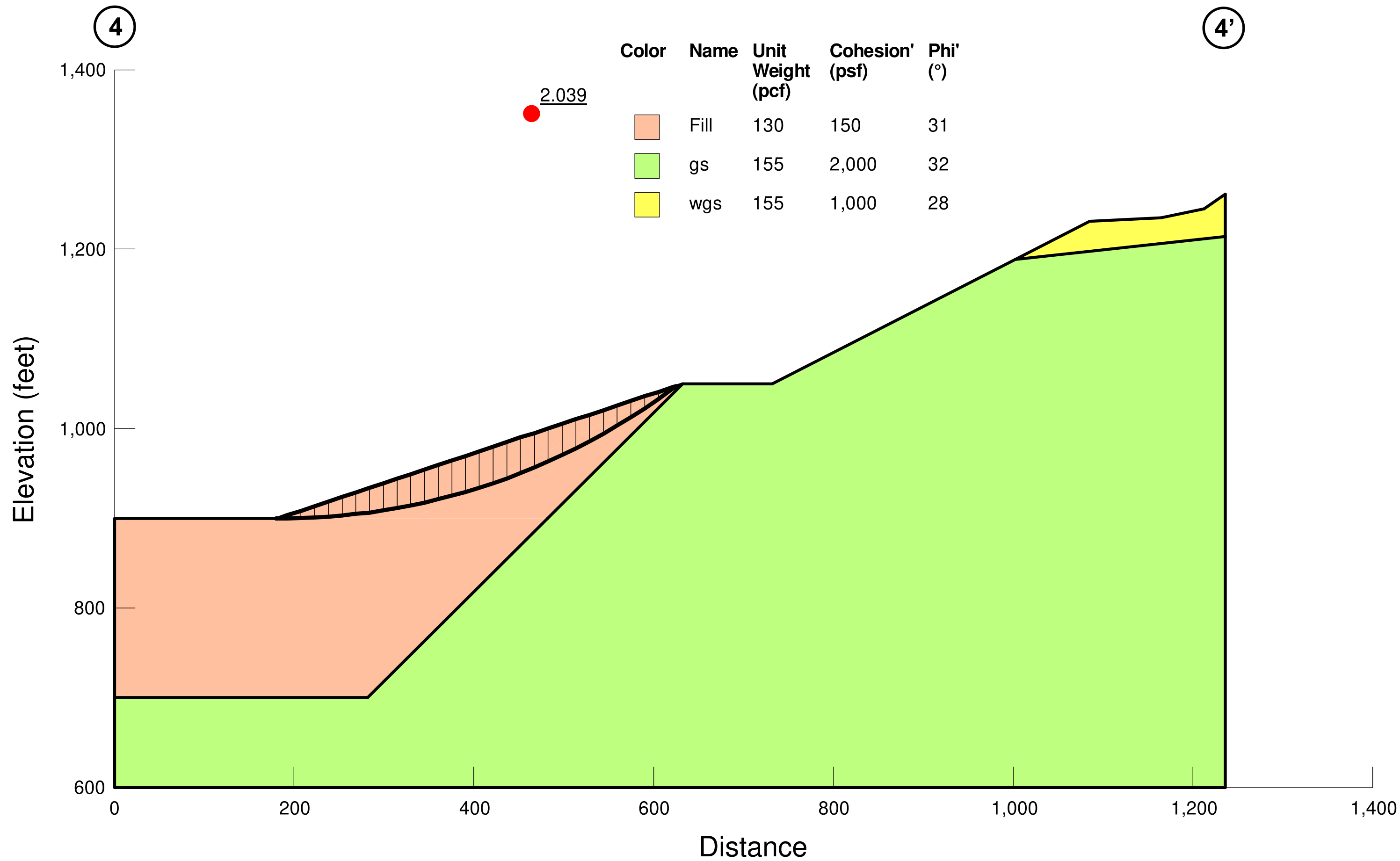
**SLOPE STABILITY ANALYSIS - FILL PHASE**
**CROSS SECTION 3-3', STATIC**

| DATE: | JOB NUMBER: | PLATE |
|---|---|---|
| December 2019 | STEVE-18-02 | 24 |

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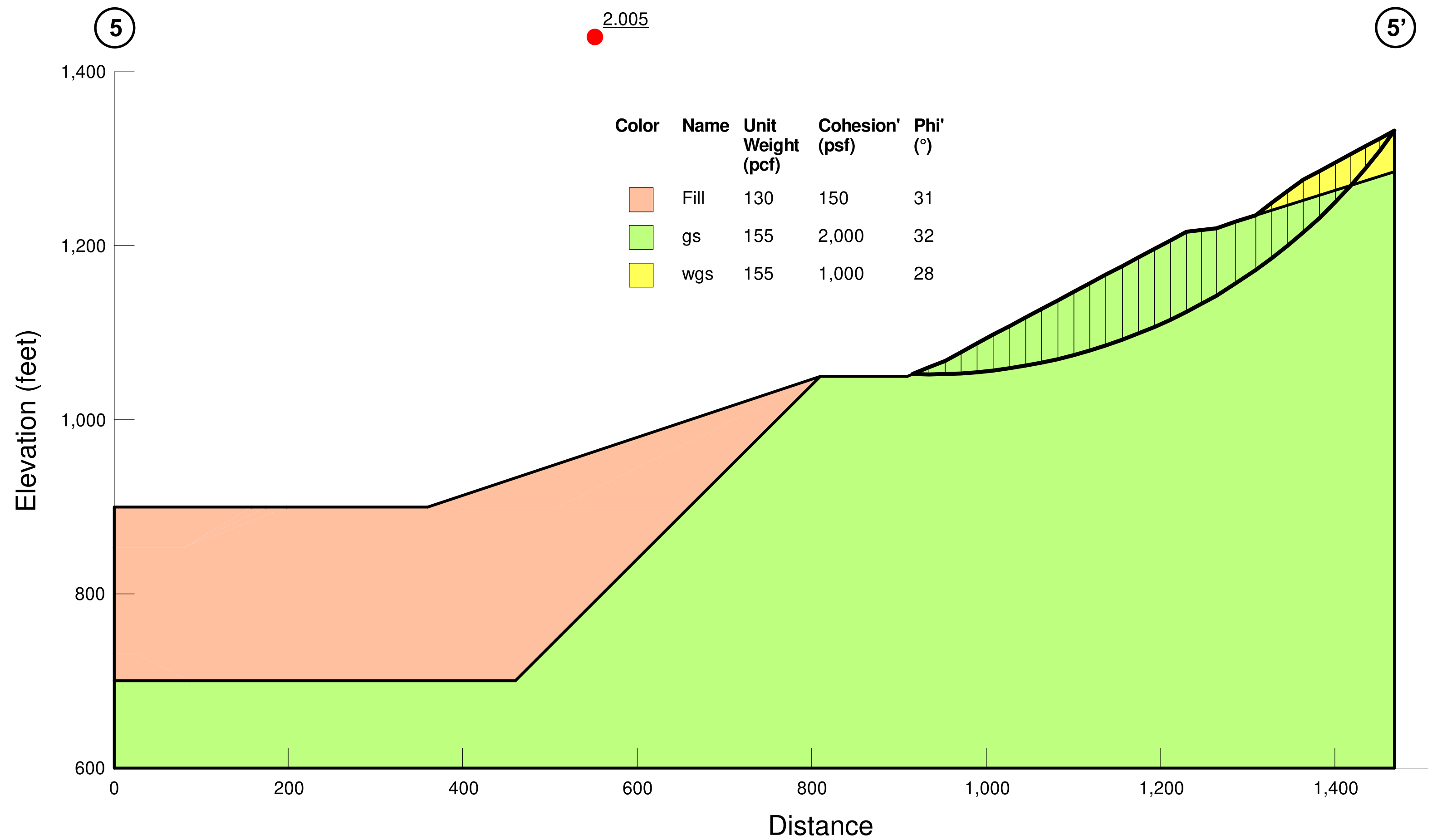
**SLOPE STABILITY ANALYSIS - FILL PHASE**
**CROSS SECTION 4-4', STATIC**

| DATE: | JOB NUMBER: | PLATE |
|---|---|---|
| December 2019 | STEVE-18-02 | 25 |

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CUPERTINO, CALIFORNIA 95014

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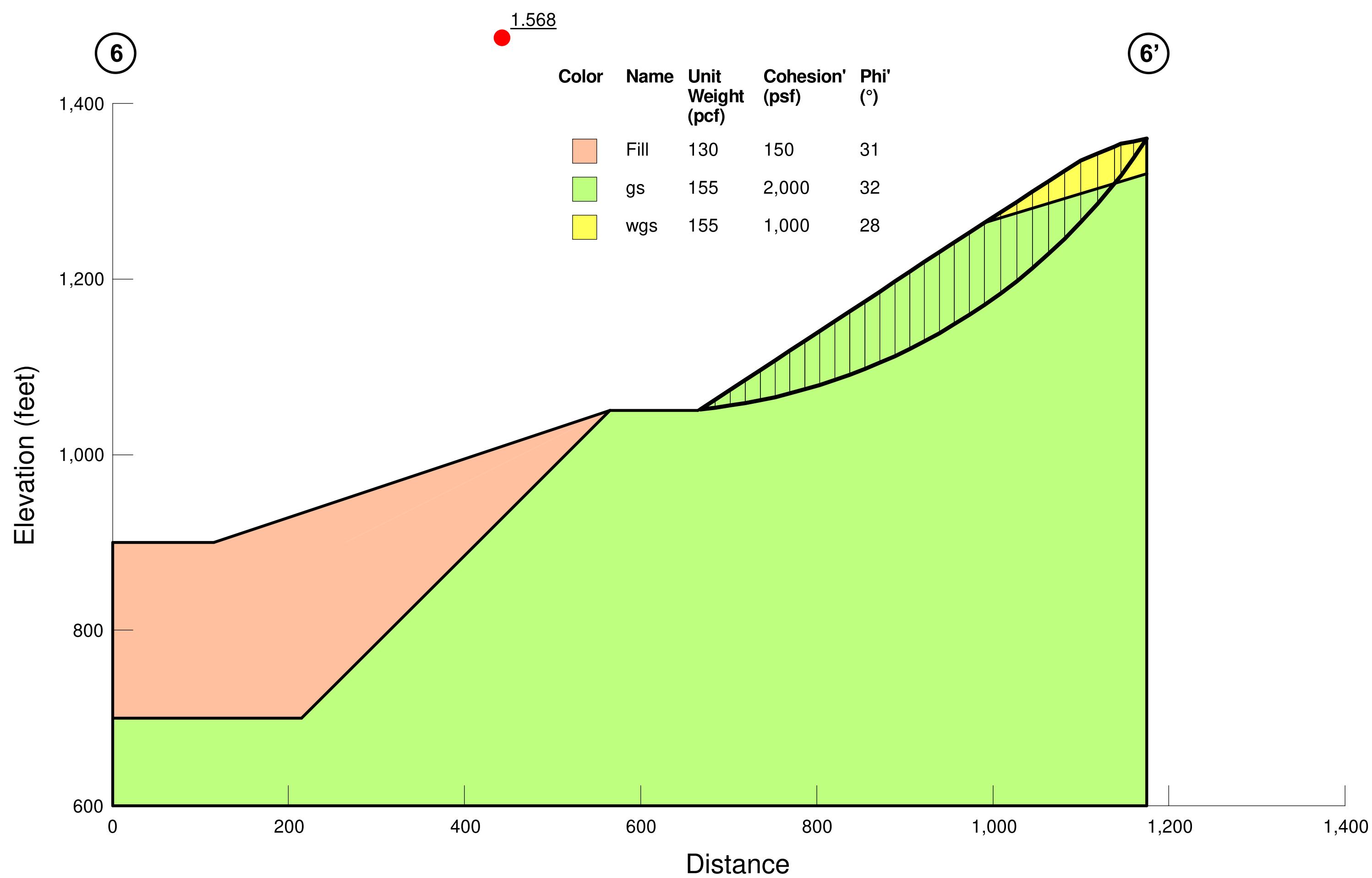
**SLOPE STABILITY ANALYSIS - FILL PHASE**
**CROSS SECTION 5-5', STATIC**

| DATE: | JOB NUMBER: | PLATE |
|---|---|---|
| December 2019 | STEVE-18-02 | 26 |

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WESTERN RIM SLOPE
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CUPERTINO, CALIFORNIA 95014

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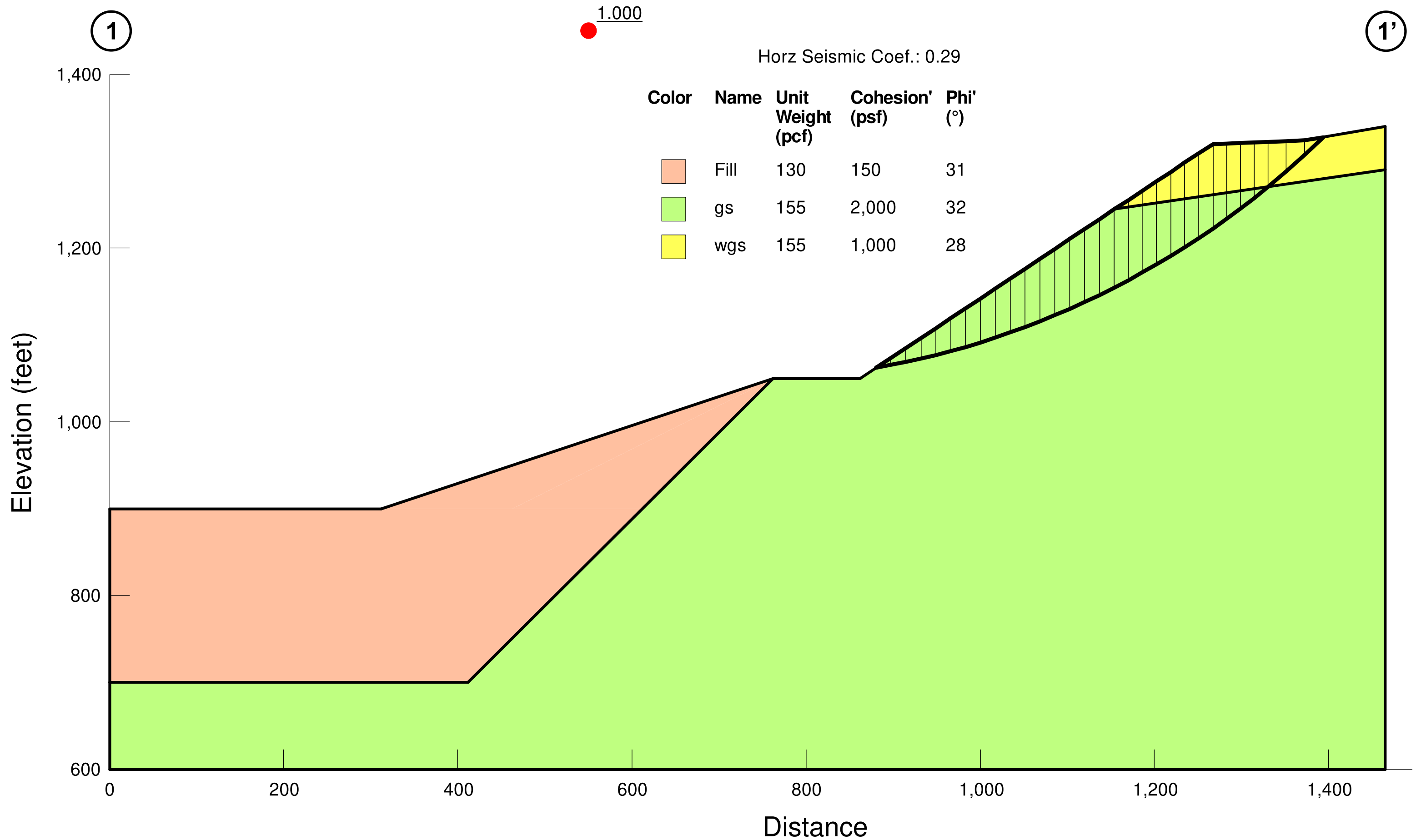
**SLOPE STABILITY ANALYSIS - FILL PHASE**
**CROSS SECTION 6-6', STATIC**

| DATE: | JOB NUMBER: | PLATE |
|---|---|---|
| December 2019 | STEVE-18-02 | 27 |

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STEVENS CREEK QUARRY
CUPERTINO, CALIFORNIA 95014

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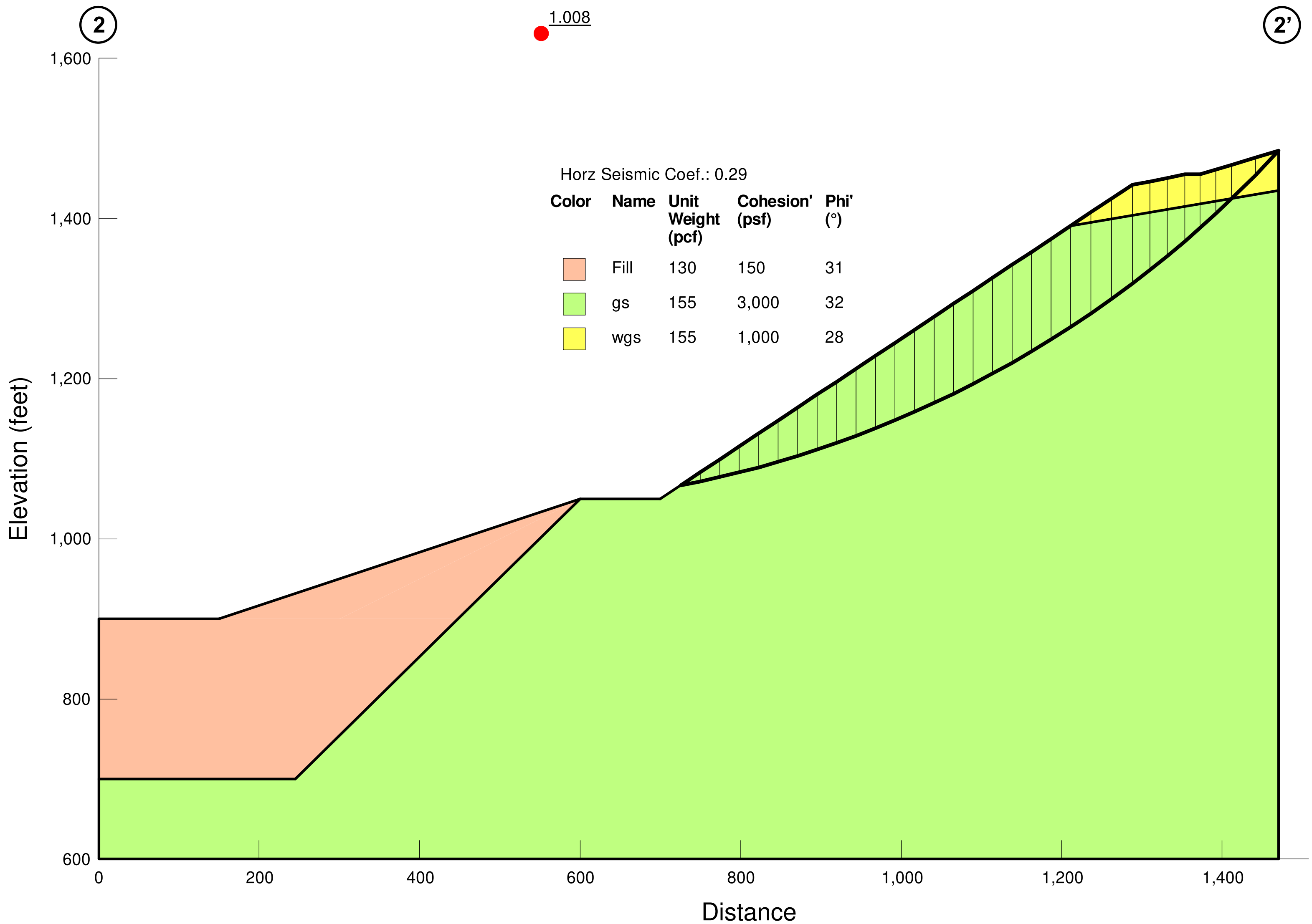
SLOPE STABILITY ANALYSIS - FILL PHASE
CROSS SECTION 1-1', PSEUDO-STATIC

| DATE: | JOB NUMBER: | PLATE |
|---|---|---|
| December 2019 | STEVE-18-02 | 28 |

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STABILITY ANALYSIS AND GEOTECHNICAL CONSULTATION
WESTERN RIM SLOPE
STEVENS CREEK QUARRY
CUPERTINO, CALIFORNIA 95014

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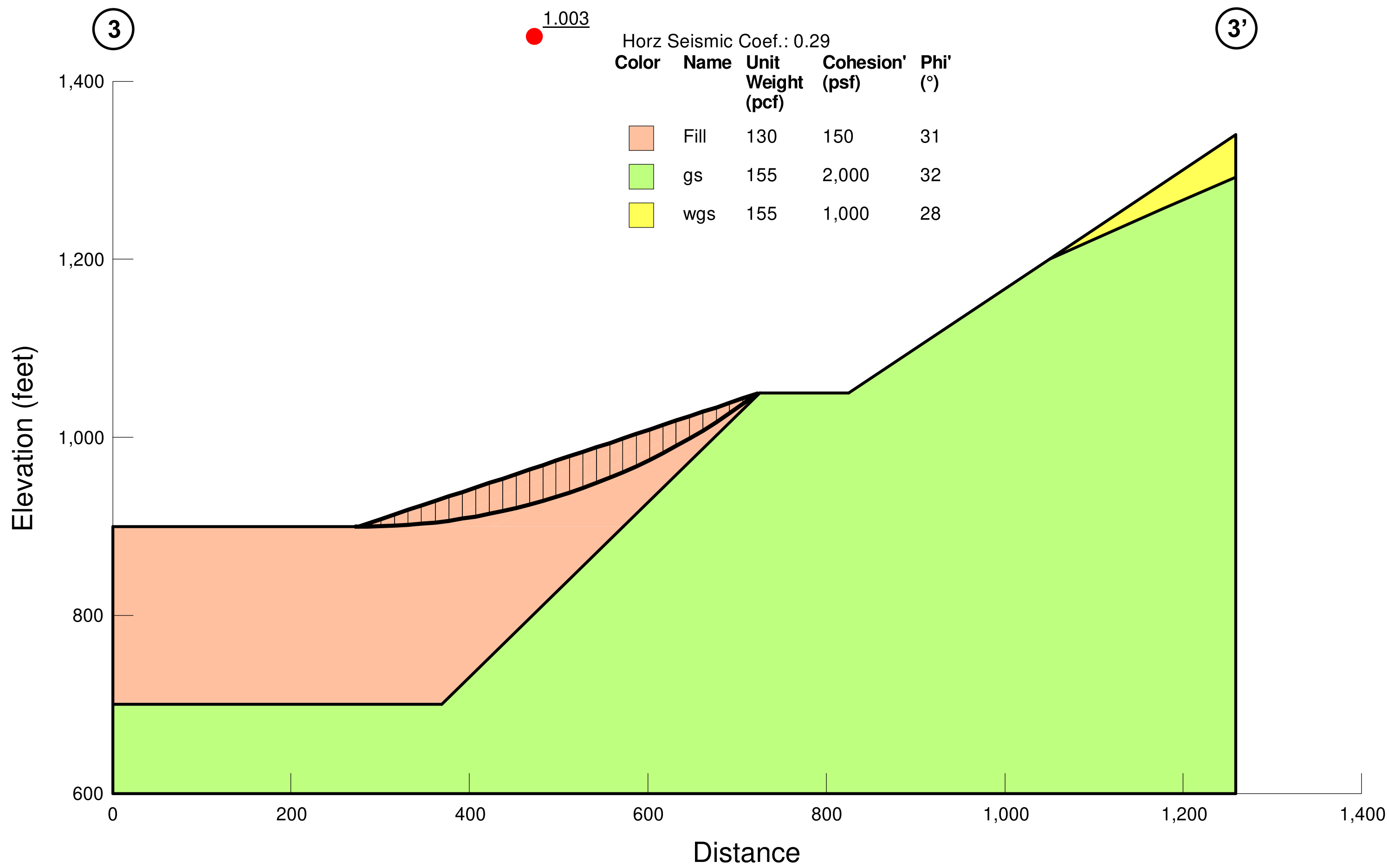
**SLOPE STABILITY ANALYSIS - FILL PHASE**
**CROSS SECTION 2-2', PSEUDO-STATIC**

| DATE: | JOB NUMBER: | PLATE |
|---|---|---|
| December 2019 | STEVE-18-02 | 29 |

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WESTERN RIM SLOPE
STEVENS CREEK QUARRY
CUPERTINO, CALIFORNIA 95014

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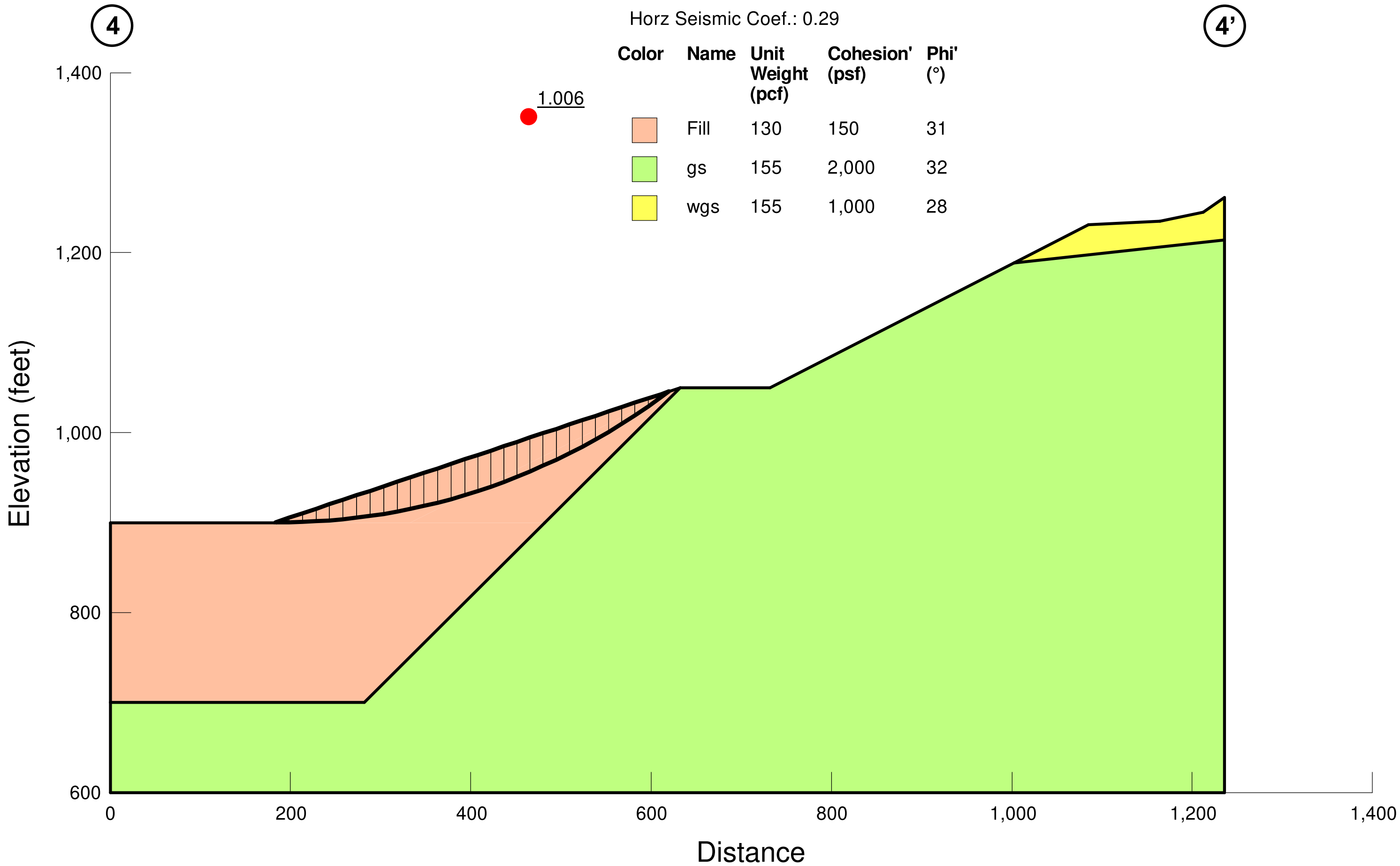
**SLOPE STABILITY ANALYSIS - FILL PHASE**
**CROSS SECTION 3-3', PSEUDO-STATIC**

| DATE: | JOB NUMBER: | PLATE |
|---|---|---|
| December 2019 | STEVE-18-02 | 30 |



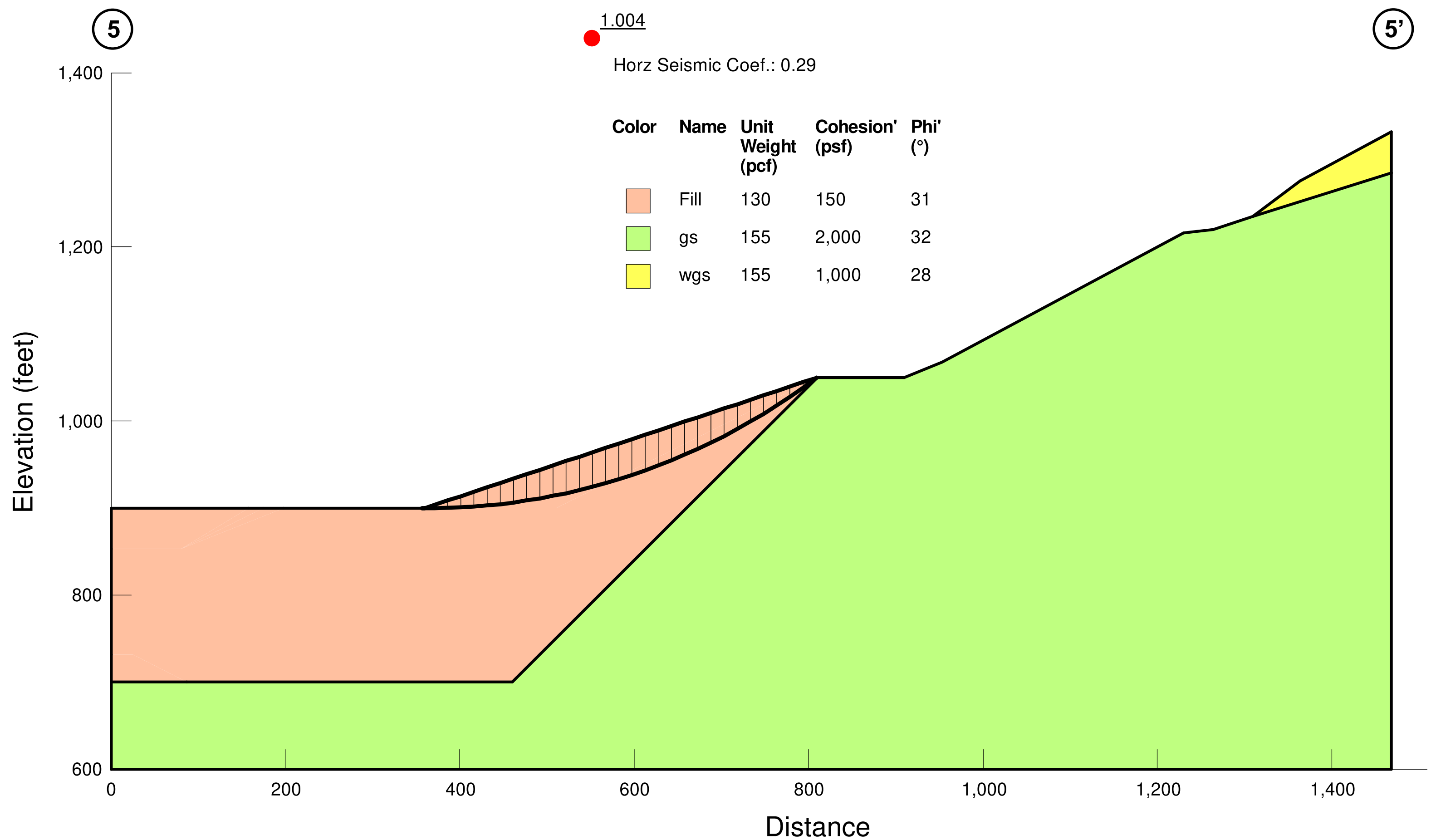

| Color | Name | Unit Weight (pcf) | Cohesion' (psf) | Phi' (°) |
|---|---|---|---|---|
| | Fill | 130 | 150 | 31 |
| | gs | 155 | 2,000 | 32 |
| | wgs | 155 | 1,000 | 28 |

**ENGINEERING GEOLOGIC SLOPE CHARACTERIZATION, STABILITY ANALYSIS AND GEOTECHNICAL CONSULTATION**
**WESTERN RIM SLOPE**
**STEVENS CREEK QUARRY**
**CUPERTINO, CALIFORNIA 95014**

BAGG ENGINEERS

**SLOPE STABILITY ANALYSIS - FILL PHASE**
**CROSS SECTION 4-4', PSEUDO-STATIC**

| DATE: | JOB NUMBER: | PLATE |
|---|---|---|
| December 2019 | STEVE-18-02 | 31 |

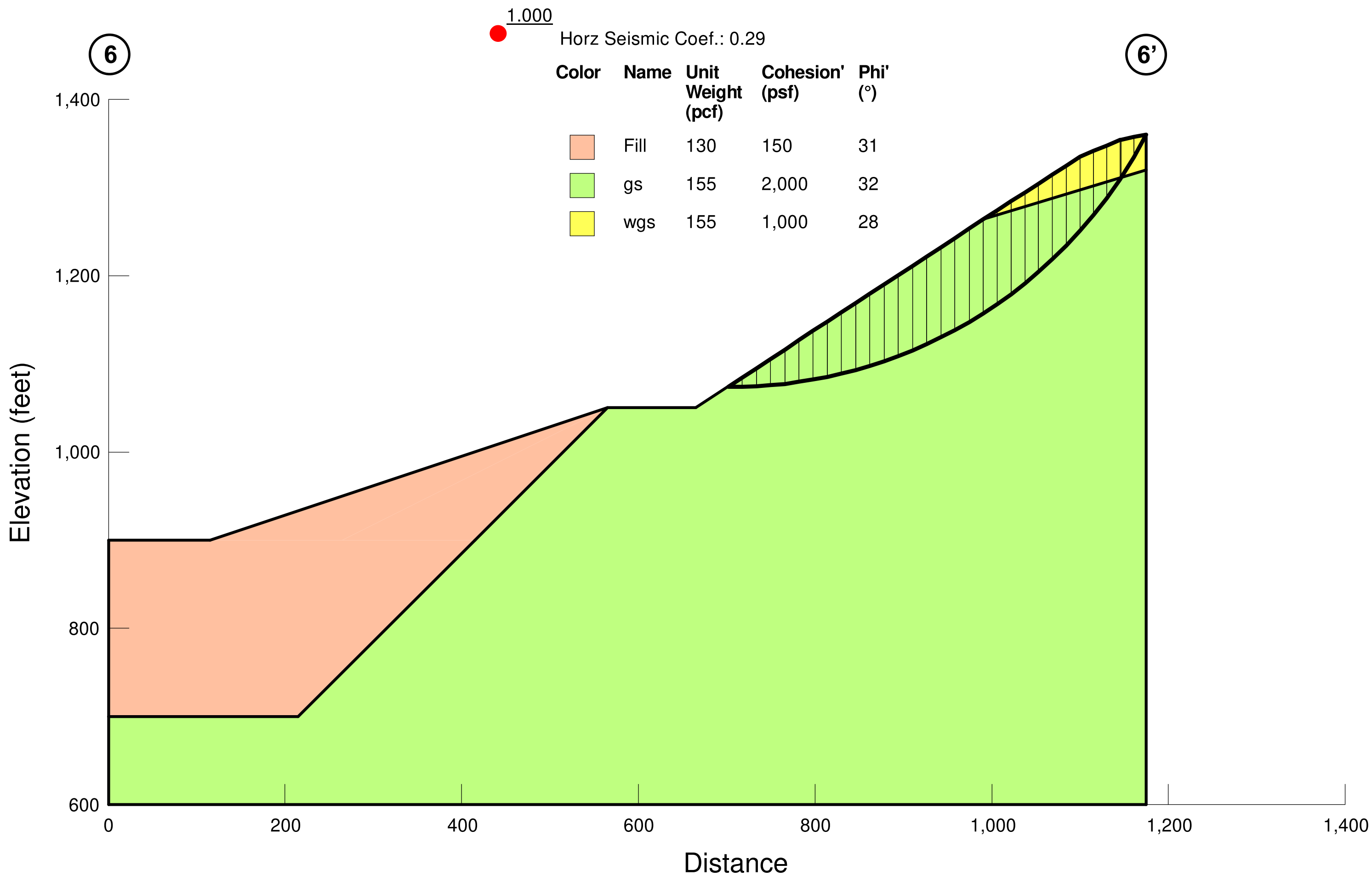



ENGINEERING GEOLOGIC SLOPE CHARACTERIZATION, STABILITY ANALYSIS AND GEOTECHNICAL CONSULTATION
WESTERN RIM SLOPE
STEVENS CREEK QUARRY
CUPERTINO, CALIFORNIA 95014

BAGG ENGINEERS

**SLOPE STABILITY ANALYSIS - FILL PHASE**
**CROSS SECTION 5-5’, PSEUDO-STATIC**

| DATE: | JOB NUMBER: | PLATE |
|---|---|---|
| December 2019 | STEVE-18-02 | 32 |




ENGINEERING GEOLOGIC SLOPE CHARACTERIZATION,
STABILITY ANALYSIS AND GEOTECHNICAL CONSULTATION
WESTERN RIM SLOPE
STEVENS CREEK QUARRY
CUPERTINO, CALIFORNIA 95014

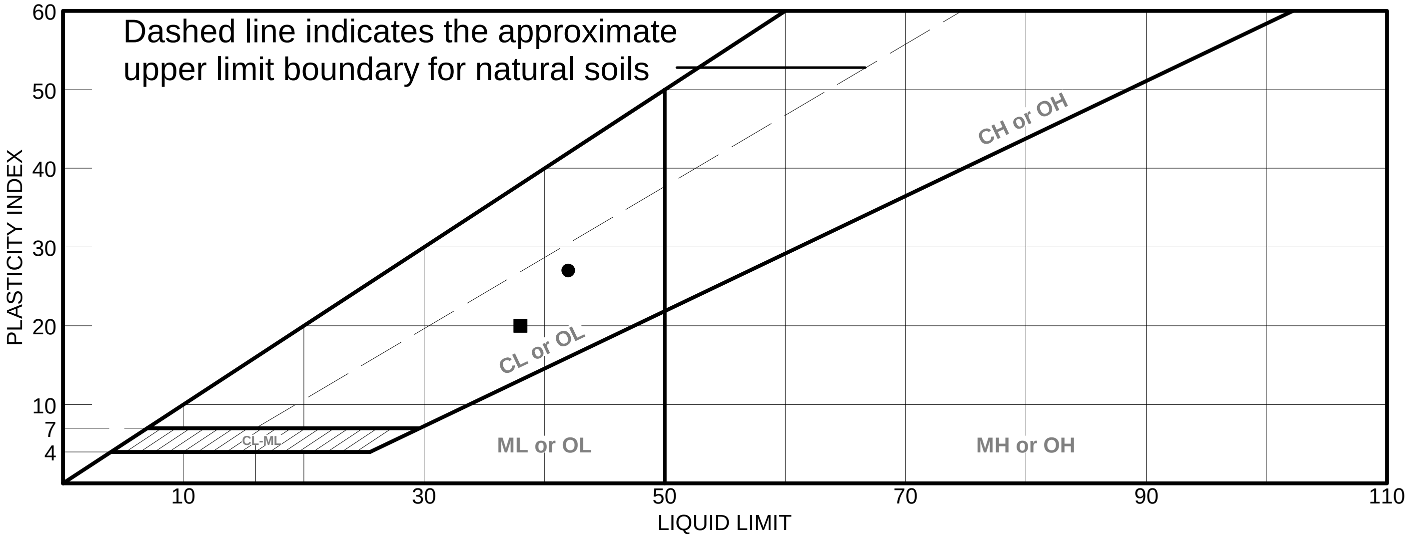
BAGG ENGINEERS

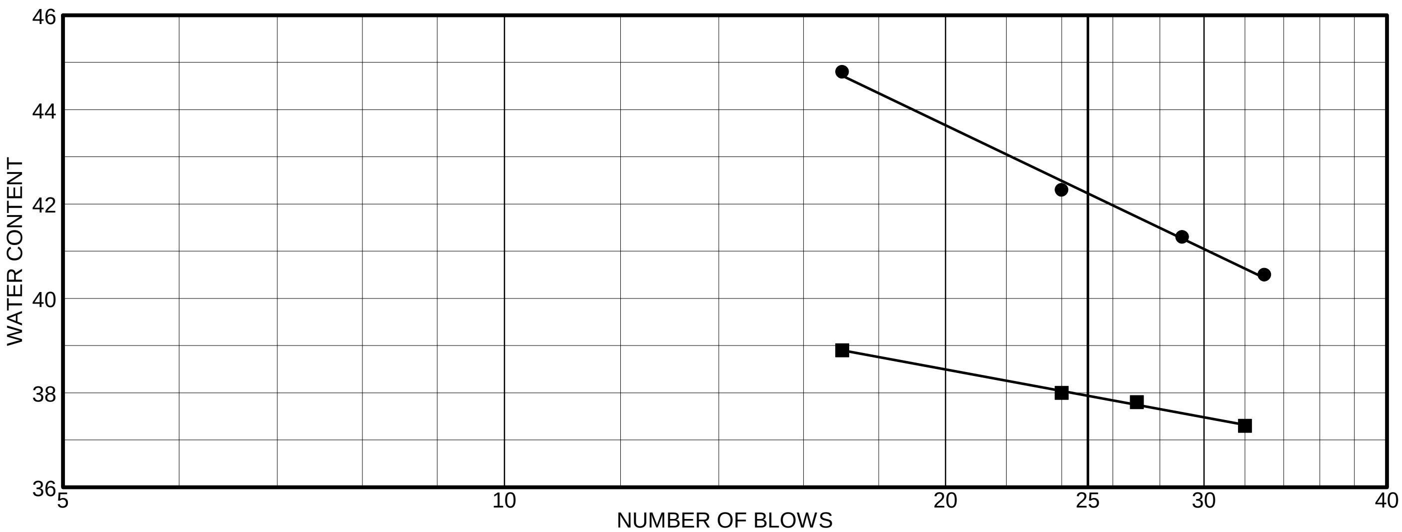
**SLOPE STABILITY ANALYSIS - FILL PHASE**
**CROSS SECTION 6-6', PSEUDO-STATIC**

| DATE: | JOB NUMBER: | PLATE |
|---|---|---|
| December 2019 | STEVE-18-02 | 33 |

# Appendix A

# LIQUID AND PLASTIC LIMITS TEST REPORT




| | MATERIAL DESCRIPTION | LL | PL | PI | %<#40 | %<#200 | USCS |
|---|---|---|---|---|---|---|---|
| ● | Greenish Gray Lean CLAY w/ Sand | 42 | 15 | 27 | | | |
| ■ | Greenish Gray Lean CLAY w/ Sand | 38 | 18 | 20 | | | |
| | | | | | | | |
| | | | | | | | |
| | | | | | | | |

**Project No.** 011-828 **Client:** BAGG

**Project:** Stevens Creek Quarry, Western Slope - STEVE-18-02

● **Source:** A-SCQ **Elev./Depth:** 50-75'

■ **Source:** B-SCQ

**Remarks:**

●

■

LIQUID AND PLASTIC LIMITS TEST REPORT

**COOPER TESTING LABORATORY**

**Figure**

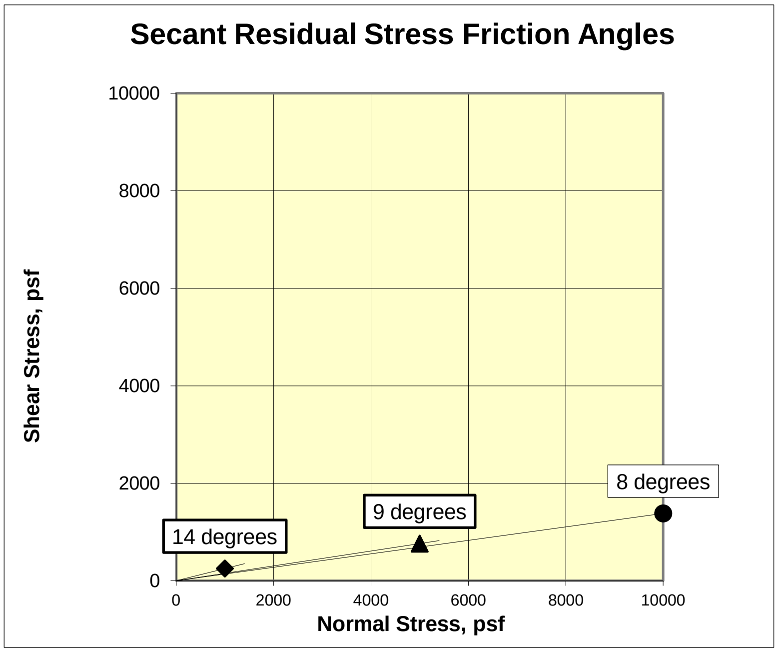
# Drained Residual Torsional Shear Strength
# (ASTM D6467)

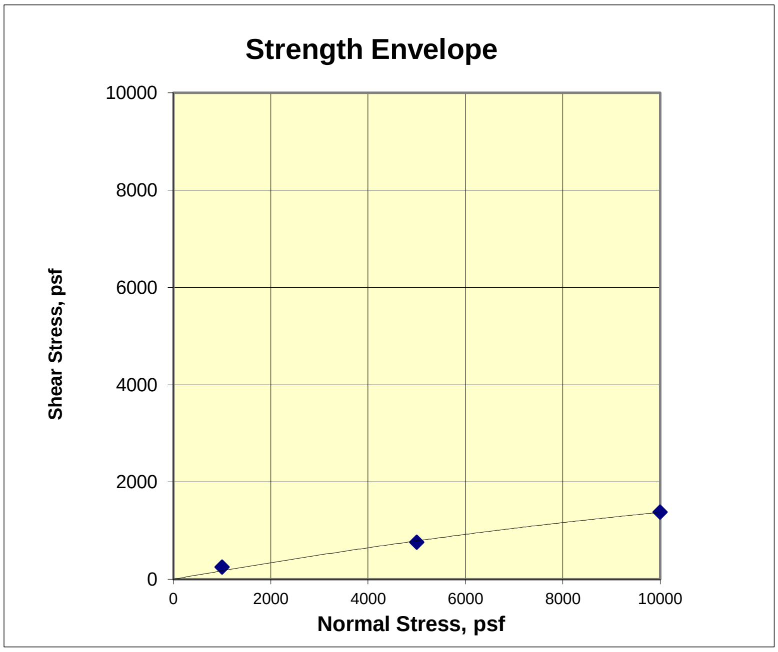
| | | | | | | | |
|---|---|---|---|---|---|---|---|
| **CTL Job No.:** | 011-828 | **Boring:** | | **Date:** | 11/1/2018 | **Clay, %:** | |
| **Client:** | BAGG | **Sample:** | A-SCQ | **By:** | PJ | **LL:** | 42 |
| **Project Name:** | Stevens Creek Quarry, Western Slope | **Depth (ft):** | 50-75 | **Checked:** | DC | **PL:** | 15 |
| **Project Number:** | STEVE-18-02 | **Test Type:** | Fully Softened Residual | | | | |

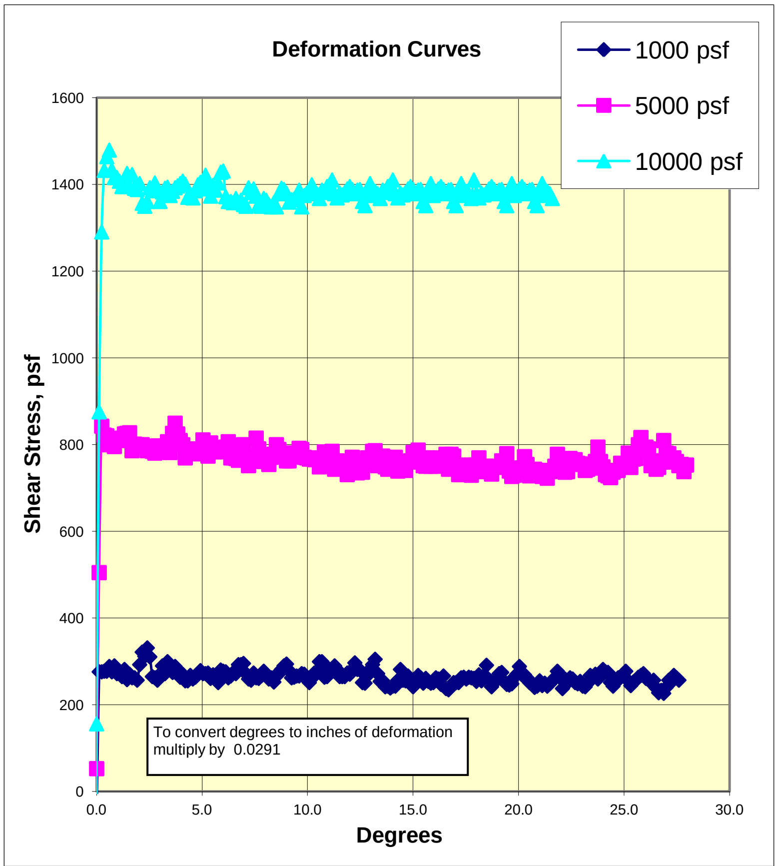
**Soil Type:** Greenish Gray Lean CLAY w/ Sand

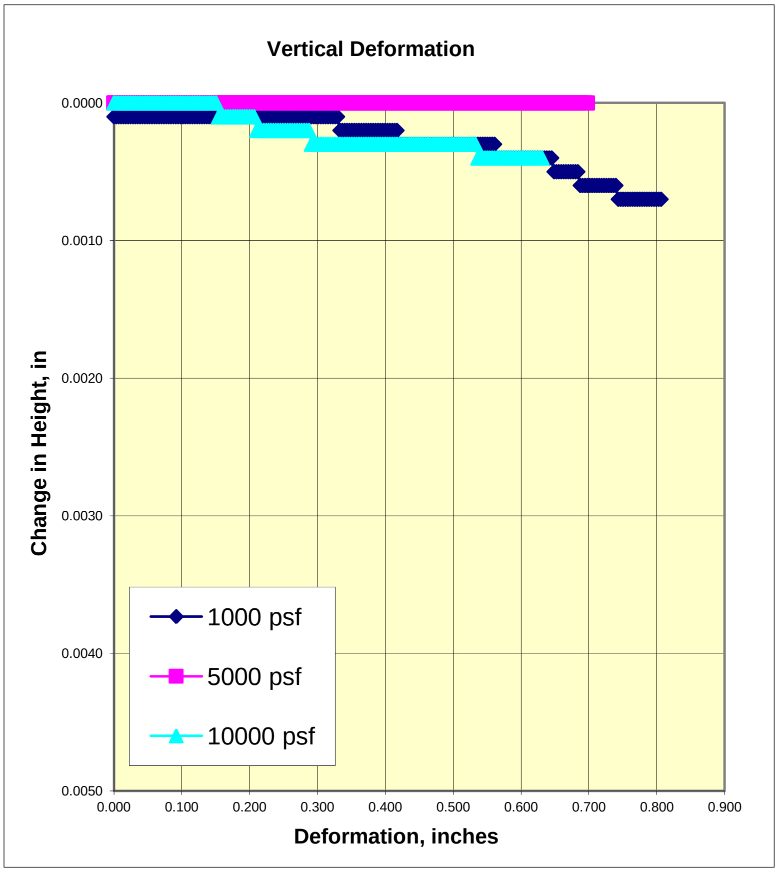
| **Normal Stress, psf:** | 1000 | 5000 | 10000 |
|---|---|---|---|
| **Secant Phi, deg.:** | 14 | 9 | 8 |

Remarks: A small friction correction was applied to each point.



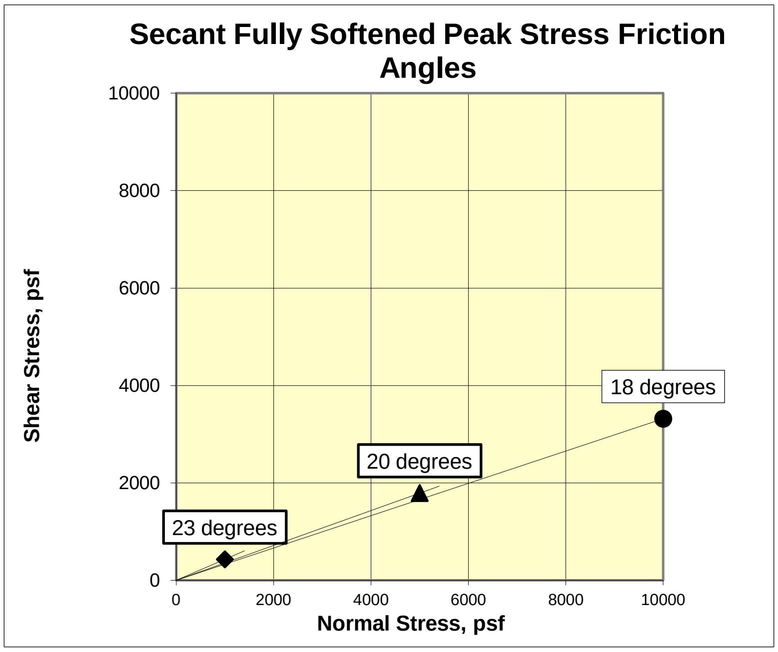
# Drained Fully Softened Peak Torsional Shear Strength (ASTM D7608)

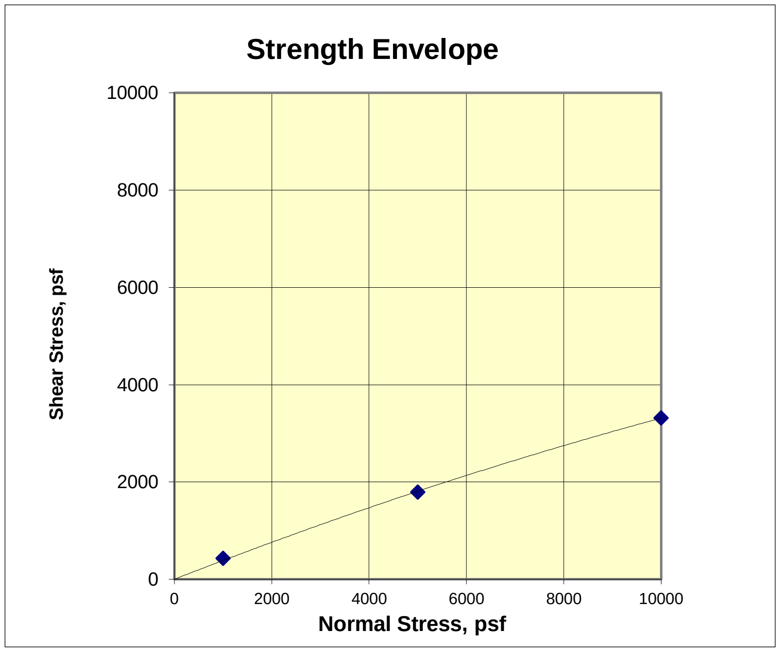
| | | | | | | | |
|---|---|---|---|---|---|---|---|
| **CTL Job No.:** | 011-828 | **Boring:** | | **Date:** | 11/1/2018 | **Clay, %:** | |
| **Client:** | BAGG | **Sample:** | A-SCQ | **By:** | PJ | **LL:** | 42 |
| **Project Name:** | Stevens Creek Quarry, Western Slope | **Depth (ft):** | 50-75 | **Checked:** | DC | **PL:** | 15 |
| **Project Number:** | STEVE-18-02 | **Test Type:** | Fully Softened Peak | | | | |

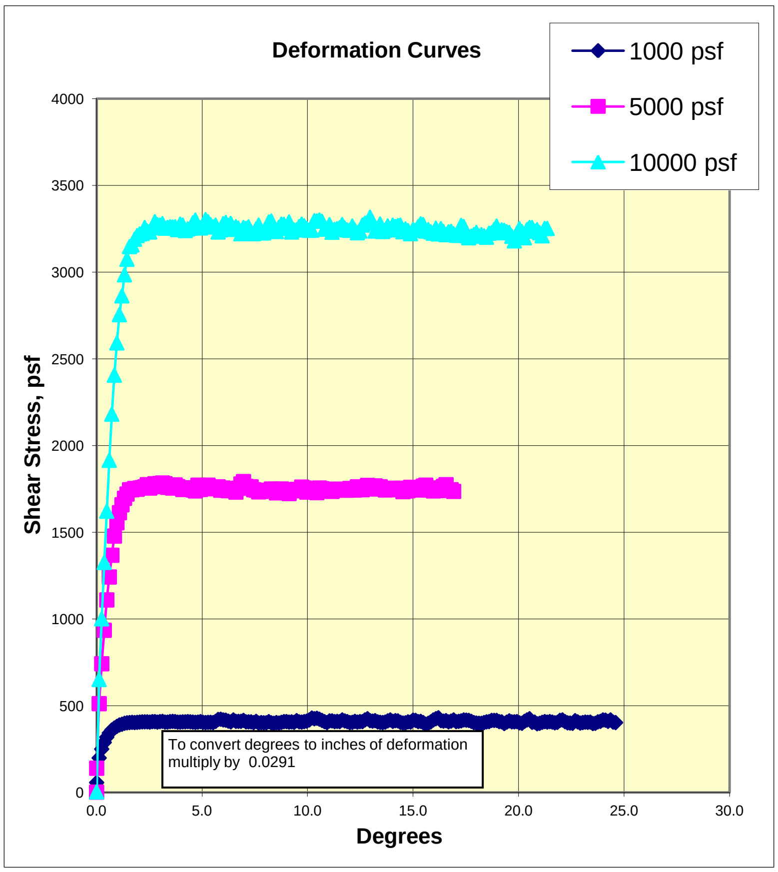
**Soil Type:** Greenish Gray Lean CLAY w/ Sand

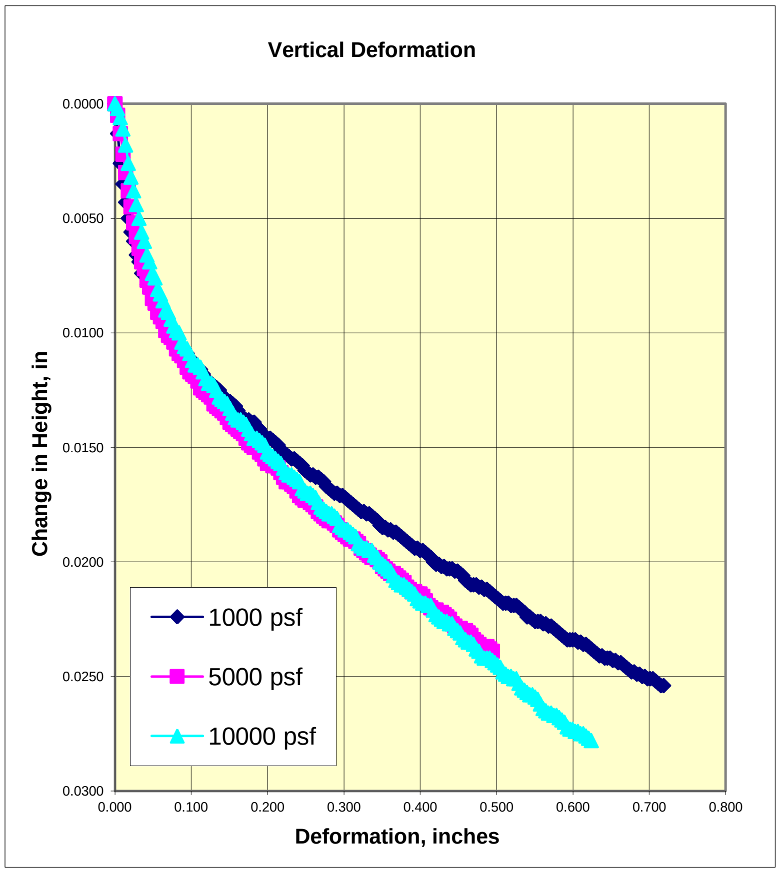
Remarks:

| **Normal Stress, psf:** | 1000 | 5000 | 10000 |
|---|---|---|---|
| **Secant Phi, deg.:** | 23 | 20 | 18 |



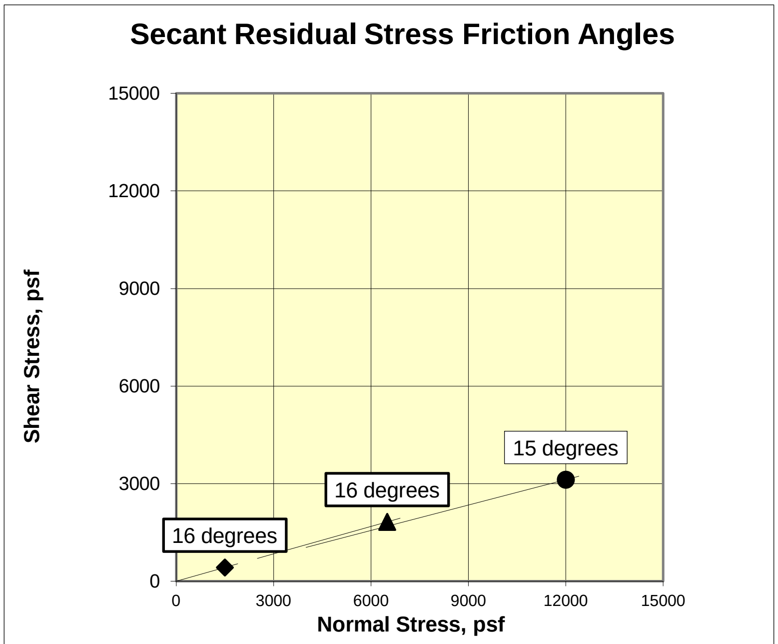
# Drained Residual Torsional Shear Strength (ASTM D6467)

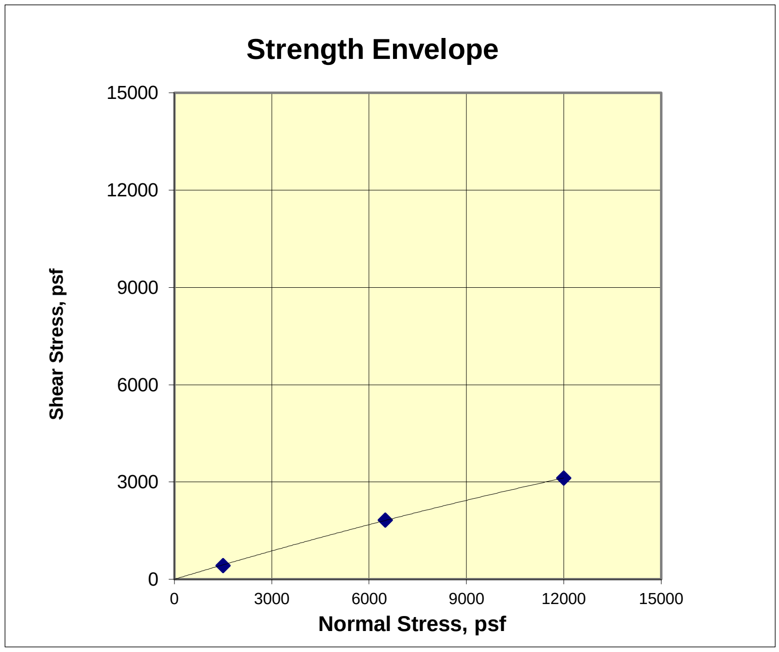
| | | | | | | | |
|---|---|---|---|---|---|---|---|
| **CTL Job No.:** | 011-828 | **Boring:** | | **Date:** | 11/28/2018 | **Clay, %:** | |
| **Client:** | BAGG | **Sample:** | B-SCQ | **By:** | PJ | **LL:** | 38 |
| **Project Name:** | Stevens Creek Quarry, Western Slope | **Depth (ft):** | | **Checked:** | DC | **PL:** | 18 |
| **Project Number:** | STEVE-18-02 | **Test Type:** | Fully Softened Residual | | | | |

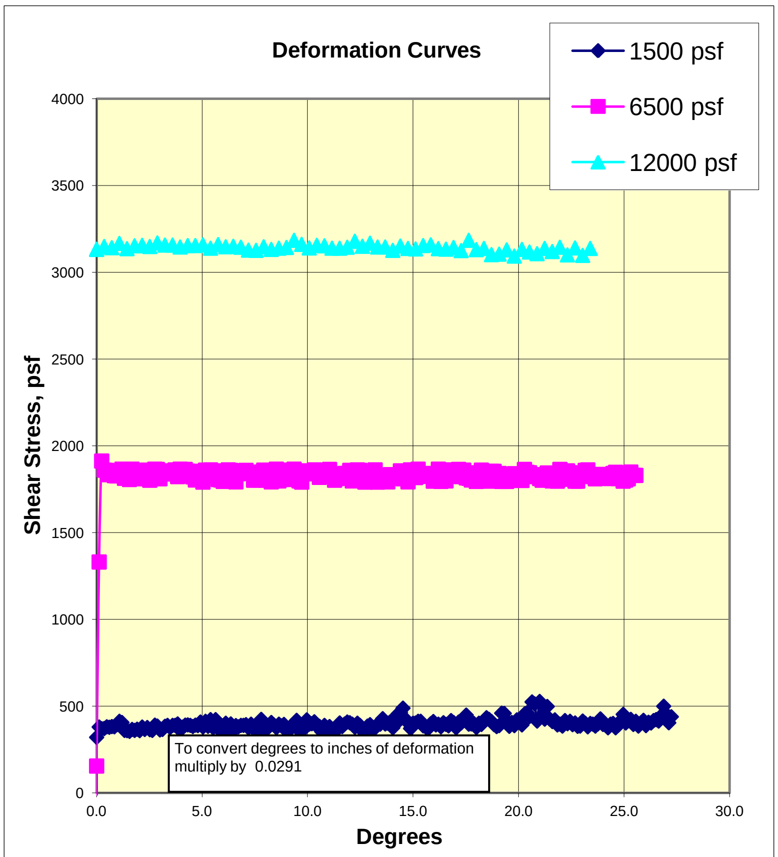
**Soil Type:** Greenish Gray Lean CLAY w/ Sand

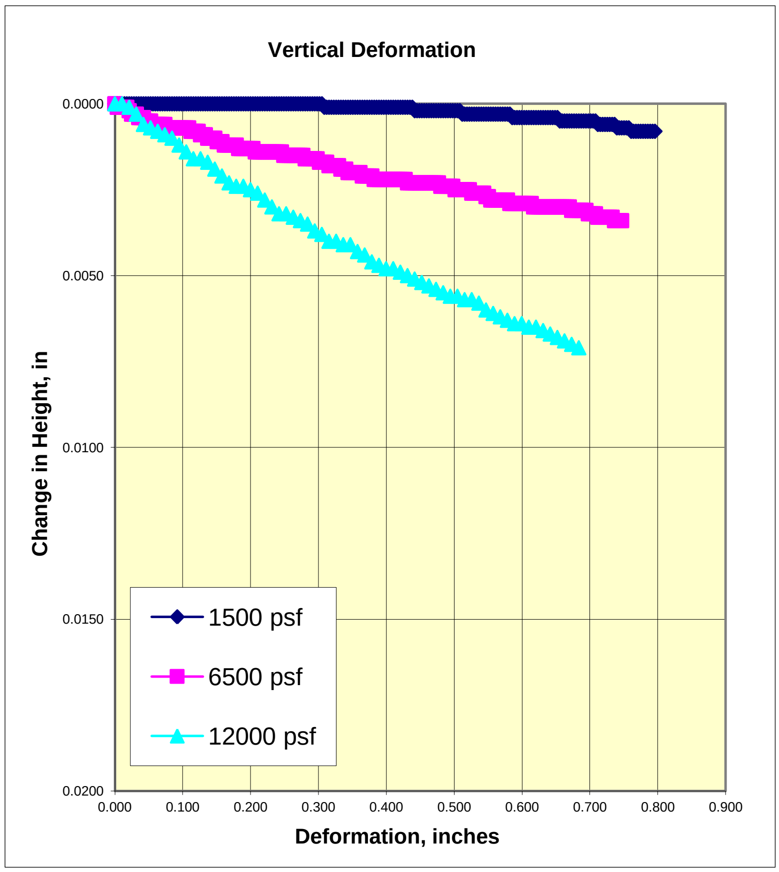
| | | | |
|---|---|---|---|
| **Normal Stress, psf:** | 1500 | 6500 | 12000 |
| **Secant Phi, deg.:** | 16 | 16 | 15 |

Remarks: A small friction correction was applied to each point.



# Drained Fully Softened Peak Torsional Shear Strength (ASTM D7608)

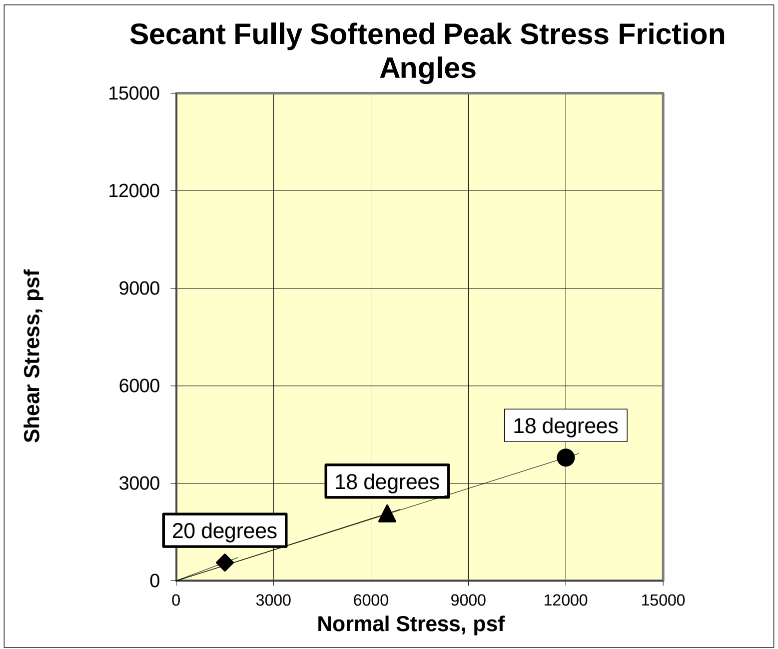
COOPER TESTING LABORATORY

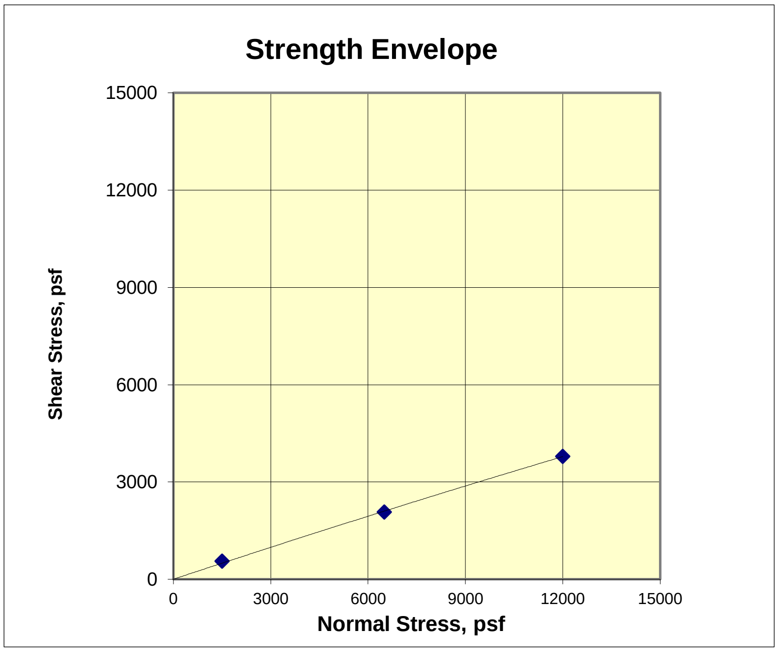
| | | | | | | | |
|---|---|---|---|---|---|---|---|
| **CTL Job No.:** | 011-828 | **Boring:** | | **Date:** | 11/28/2018 | **Clay, %:** | |
| **Client:** | BAGG | **Sample:** | B-SCQ | **By:** | PJ | **LL:** | 38 |
| **Project Name:** | Stevens Creek Quarry, Western Slope | **Depth (ft):** | | **Checked:** | DC | **PL:** | 18 |
| **Project Number:** | STEVE-18-02 | **Test Type:** | Fully Softened Peak | | | | |

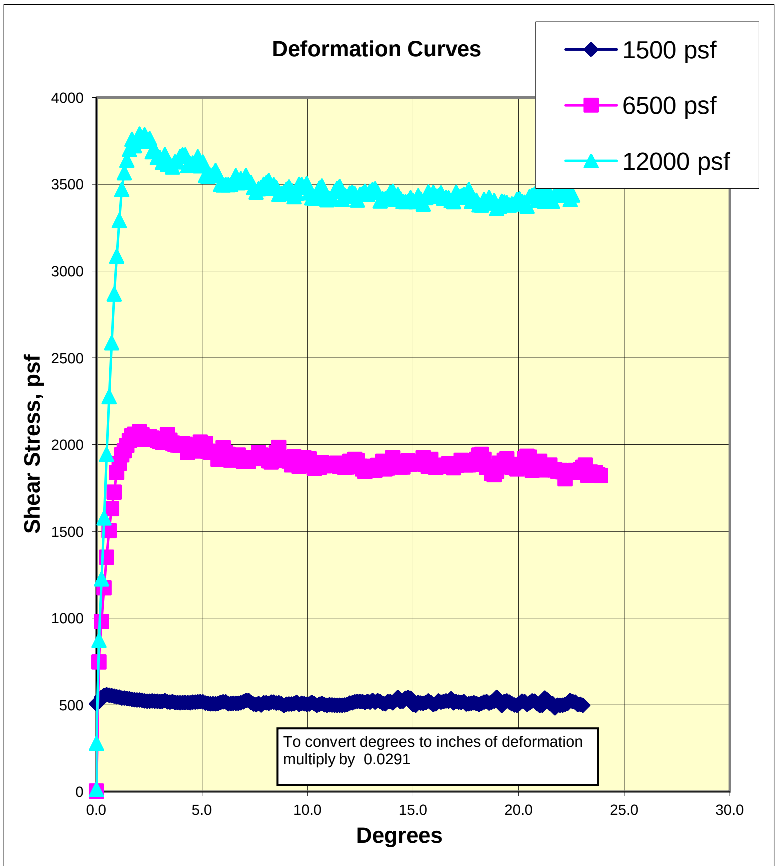
**Soil Type:** Greenish Gray Lean CLAY w/ Sand

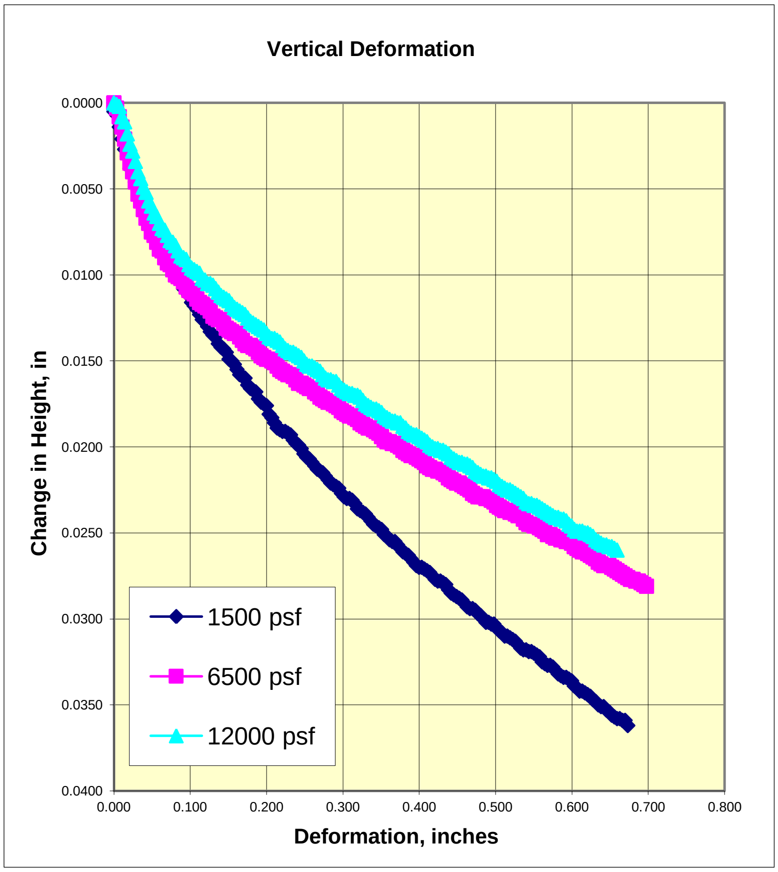
**Remarks:**

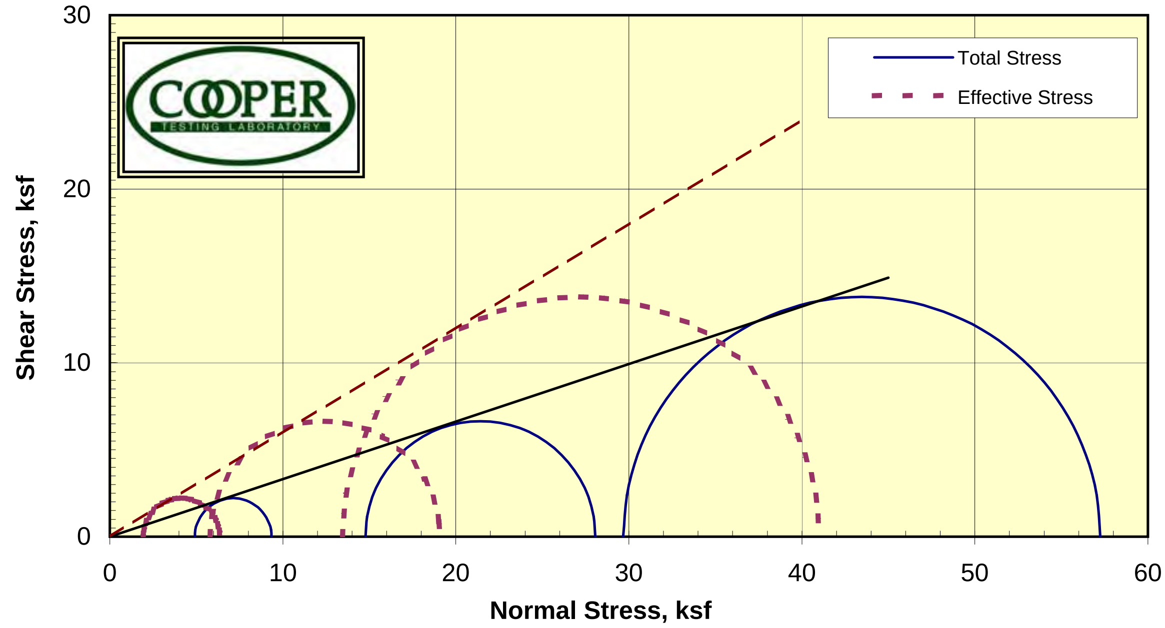
| | | | |
|---|---|---|---|
| **Normal Stress, psf:** | 1500 | 6500 | 12000 |
| **Secant Phi, deg.:** | 20 | 18 | 18 |

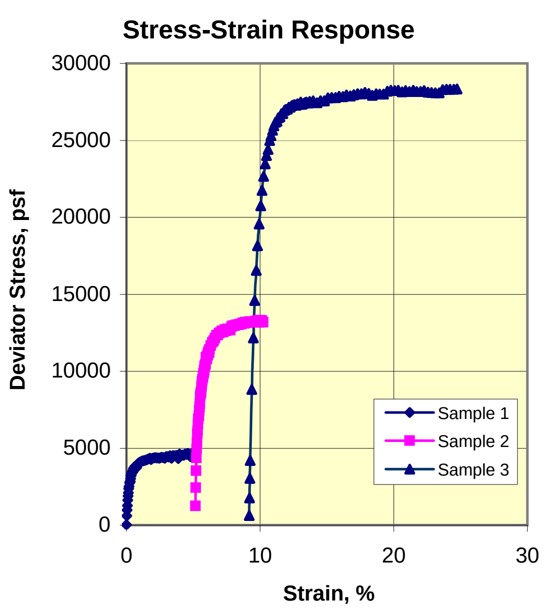





**Job No.:** 471-021 **Date:** 1/10/2008

**Client:** Cal Engineering & Geology BY:DC

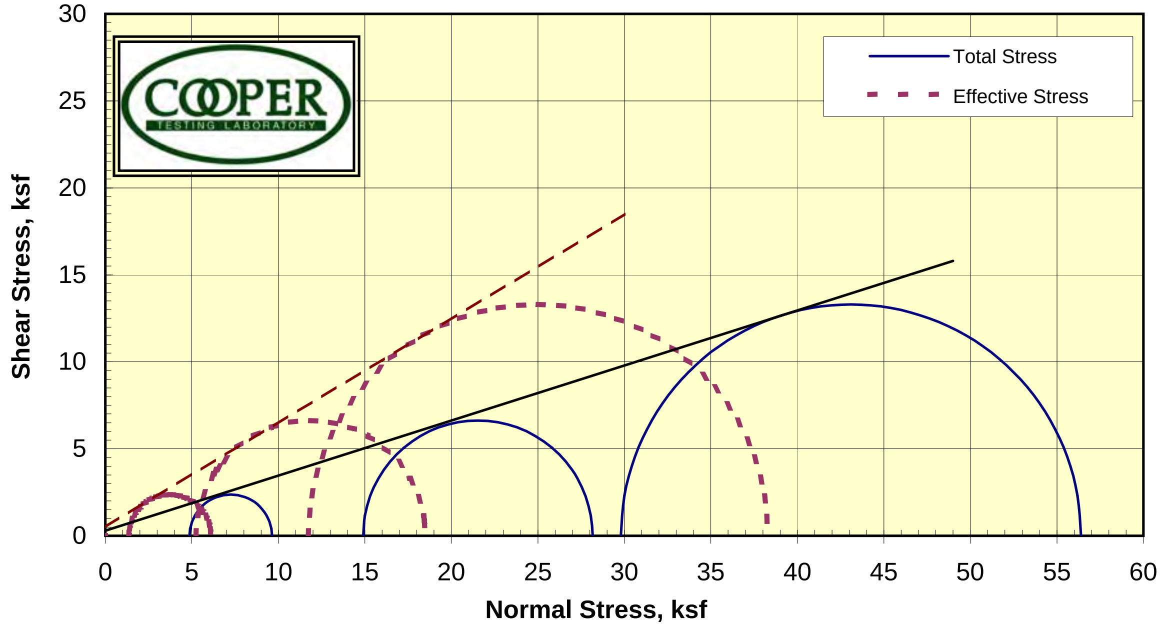
**Project:** Stevens Creek Quarry - 071210

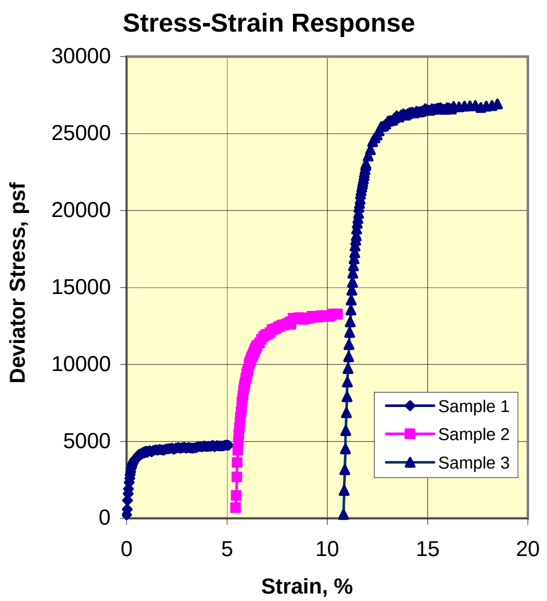
**Sample:** Import #1 Brown Sandy CLAY w/ Gravel

**Remarks:** ** Staged Test ** Strengths at 5% strain. Remolding Target: 90% of 128.1 @ 12.5 (OP +2). This sample was taken from the processed imported material stock pile just north of the Qarry office (Parcel A).

| **Stage** | **1** | **2** | **3** |
|---|---|---|---|
| **MC, %** | 12.5 | | |
| **Dry Dens., pcf.** | 115.0 | | |
| **Sat. %** | 72.5 | | |
| **Void Ratio** | 0.466 | | |
| **Diameter in** | 2.38 | | |
| **Height, in** | 5.00 | | |
| | | **Final** | |
| **MC, %** | 15.7 | 13.8 | 12.4 |
| **Dry Dens., pcf.** | 118.0 | 122.5 | 125.8 |
| **Sat. %** | 100.0 | 100.0 | 100.0 |
| **Void Ratio** | 0.425 | 0.372 | 0.336 |
| **Diameter, in** | 2.36 | 2.36 | 2.37 |
| **Height, in** | 4.94 | 4.76 | 4.58 |
| **Cell, psi** | 94.7 | 164.2 | 268.3 |
| **BP, psi** | 60.6 | 61.6 | 62.2 |
| | | **Effective Stresses At:** | |
| **Strain, %** | 5.0 | 5.0 | 5.0 |
| **Deviator ksf** | 4.434 | 13.292 | 27.575 |
| **Excess PP** | 2.993 | 8.978 | 16.238 |
| **Sigma 1** | 6.355 | 19.086 | 41.013 |
| **Sigma 3** | 1.921 | 5.794 | 13.438 |
| **P, ksf** | 4.138 | 12.440 | 27.225 |
| **Q, ksf** | 2.217 | 6.646 | 13.787 |
| **Stress Ratio** | 3.308 | 3.294 | 3.052 |
| **Rate in/min** | 0.001 | 0.001 | 0.001 |

| | | | |
|---|---|---|---|
| Total C | 0 | Effective C | 0.0 |
| Total Phi | 18.3 | Effective Phi | 31.0 |




| Stage | 1 | 2 | 3 |
|---|---|---|---|
| MC, % | 12.3 | | |
| Dry Dens., pcf. | 119.0 | | |
| Sat. % | 79.9 | | |
| Void Ratio | 0.416 | | |
| Diameter in | 2.88 | | |
| Height, in | 6.00 | | |
| | | Final | |
| MC, % | 14.8 | 13.1 | 11.8 |
| Dry Dens., pcf. | 123.1 | 127.2 | 130.8 |
| Sat. % | 100.0 | 100.0 | 100.0 |
| Void Ratio | 0.399 | 0.354 | 0.318 |
| Diameter, in | 2.85 | 2.86 | 2.89 |
| Height, in | 5.91 | 5.69 | 5.41 |
| Cell, psi | 94.7 | 164.2 | 268.3 |
| BP, psi | 60.9 | 60.7 | 61.3 |
| | | Effective Stresses At: | |
| Strain, % | 5.0 | 5.0 | 5.0 |
| Deviator ksf | 4.755 | 13.255 | 26.585 |
| Excess PP | 3.519 | 9.664 | 18.081 |
| Sigma 1 | 6.109 | 18.496 | 38.310 |
| Sigma 3 | 1.354 | 5.241 | 11.725 |
| P, ksf | 3.731 | 11.868 | 25.017 |
| Q, ksf | 2.378 | 6.627 | 13.292 |
| Stress Ratio | 4.513 | 3.529 | 3.267 |
| Rate in/min | 0.001 | 0.001 | 0.001 |

| | | | |
|---|---|---|---|
| Total C | 0.3 | Effective C | 0.5 |
| Total Phi | 17.6 | Effective Phi | 31.0 |

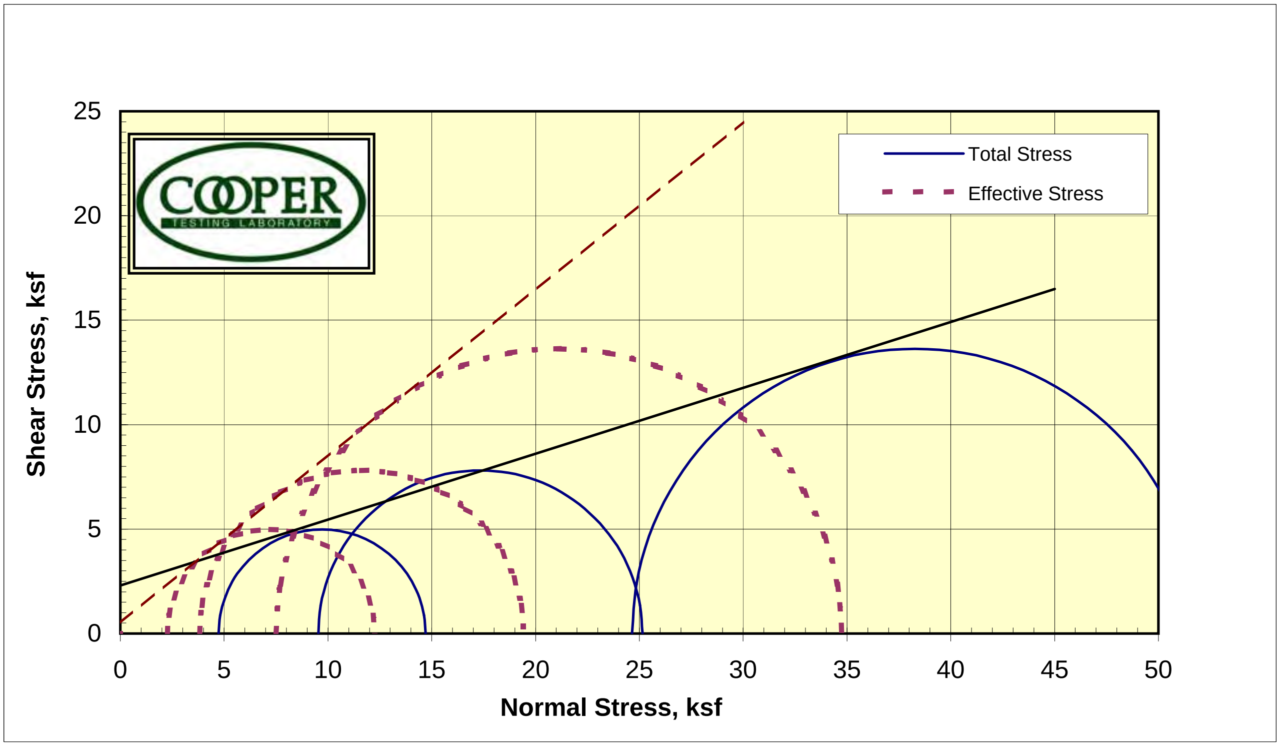
**Job No.:** 471-021 **Date:** 1/10/2008

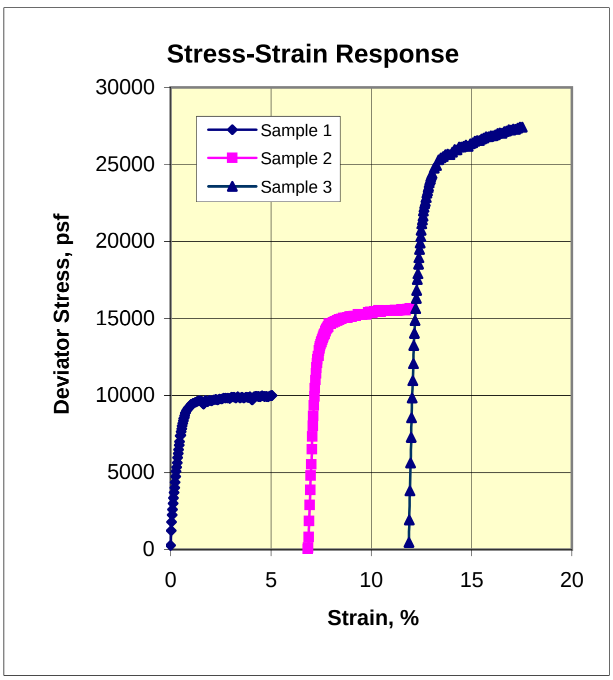
**Client:** Cal Engineering & Geolog) BY:DC

**Project:** Stevens Creek Quarry - 071210

**Sample:** Import #2 Brown Clayey SAND w/ Gravel

**Remarks:** ** Staged Test ** Strengths at 5% strain.
Remolding Target: 90% of 132 @ 12.3 (OPT +2). This sample was taken from the processed imported material stock pile just north of the Qarry office (Parcel A).




| | | | |
|---|---|---|---|
| **Job No.:** | 471-021 | **Date:** | 1/10/2008 |
| **Client:** | Cal Engineering & Geology | | BY:DC |
| **Project:** | Stevens Creek Quarry - 071210 | | |
| **Sample:** | Shear | Gray GRAVEL w/ Clay & Sand | |

**Remarks:** ** Staged Test ** Strengths at 5% strain. Remolding Target: 90% of 139.3 pcf @ 10.3 (OPT+2). This sample was taken from the shear zone at the NW corner of the Quarry (at elev ~850 ft).

| **Stage** | 1 | 2 | 3 |
|---|---|---|---|
| **MC, %** | 10.3 | | |
| **Dry Dens., pcf.** | 125.1 | | |
| **Sat. %** | 80.1 | | |
| **Void Ratio** | 0.347 | | |
| **Diameter in** | 2.38 | | |
| **Height, in** | 5.00 | | |
| | | **Final** | |
| **MC, %** | 13.6 | 13.3 | 12.0 |
| **Dry Dens., pcf.** | 126.1 | 127.1 | 130.3 |
| **Sat. %** | 100.0 | 100.0 | 100.0 |
| **Void Ratio** | 0.368 | 0.358 | 0.325 |
| **Diameter, in** | 2.38 | 2.44 | 2.46 |
| **Height, in** | 4.96 | 4.68 | 4.47 |
| **Cell, psi** | 114.7 | 149.4 | 253.6 |
| **BP, psi** | 81.8 | 83.2 | 82.4 |
| | | **Effective Stresses At:** | |
| **Strain, %** | 5.0 | 5.0 | 5.0 |
| **Deviator ksf** | 9.977 | 15.619 | 27.258 |
| **Excess PP** | 2.463 | 5.719 | 17.152 |
| **Sigma 1** | 12.246 | 19.436 | 34.754 |
| **Sigma 3** | 2.270 | 3.817 | 7.496 |
| **P, ksf** | 7.258 | 11.627 | 21.125 |
| **Q, ksf** | 4.988 | 7.810 | 13.629 |
| **Stress Ratio** | 5.396 | 5.092 | 4.636 |
| **Rate in/min** | 0.001 | 0.001 | 0.001 |

| | | | |
|---|---|---|---|
| Total C | 2.3 | Effective C | 0.5 |
| Total Phi | 17.5 | Effective Phi | 38.7 |

# Important Information about This

# Geotechnical-Engineering Report

**Subsurface problems are a principal cause of construction delays, cost overruns, claims, and disputes.**

**While you cannot eliminate all such risks, you can manage them. The following information is provided to help.**

**The Geoprofessional Business Association (GBA) has prepared this advisory to help you – assumedly a client representative – interpret and apply this geotechnical-engineering report as effectively as possible. In that way, clients can benefit from a lowered exposure to the subsurface problems that, for decades, have been a principal cause of construction delays, cost overruns, claims, and disputes. If you have questions or want more information about any of the issues discussed below, contact your GBA-member geotechnical engineer. Active involvement in the Geoprofessional Business Association exposes geotechnical engineers to a wide array of risk-confrontation techniques that can be of genuine benefit for everyone involved with a construction project.**

## Geotechnical-Engineering Services Are Performed for Specific Purposes, Persons, and Projects

Geotechnical engineers structure their services to meet the specific needs of their clients. A geotechnical-engineering study conducted for a given civil engineer will not likely meet the needs of a civil-works constructor or even a different civil engineer. Because each geotechnical-engineering study is unique, each geotechnical-engineering report is unique, prepared *solely* for the client. *Those who rely on a geotechnical-engineering report prepared for a different client can be seriously misled*. No one except authorized client representatives should rely on this geotechnical-engineering report without first conferring with the geotechnical engineer who prepared it. *And no one – not even you – should apply this report for any purpose or project except the one originally contemplated.*

## Read this Report in Full

Costly problems have occurred because those relying on a geotechnical-engineering report did not read it *in its entirety*. Do not rely on an executive summary. Do not read selected elements only. *Read this report in full.*

## You Need to Inform Your Geotechnical Engineer about Change

Your geotechnical engineer considered unique, project-specific factors when designing the study behind this report and developing the confirmation-dependent recommendations the report conveys. A few typical factors include:

- the client's goals, objectives, budget, schedule, and risk-management preferences;
- the general nature of the structure involved, its size, configuration, and performance criteria;
- the structure's location and orientation on the site; and
- other planned or existing site improvements, such as retaining walls, access roads, parking lots, and underground utilities.

Typical changes that could erode the reliability of this report include those that affect:

- the site's size or shape;
- the function of the proposed structure, as when it's changed from a parking garage to an office building, or from a light-industrial plant to a refrigerated warehouse;
- the elevation, configuration, location, orientation, or weight of the proposed structure;
- the composition of the design team; or
- project ownership.

As a general rule, *always* inform your geotechnical engineer of project changes – even minor ones – and request an assessment of their impact. *The geotechnical engineer who prepared this report cannot accept responsibility or liability for problems that arise because the geotechnical engineer was not informed about developments the engineer otherwise would have considered.*

## This Report May Not Be Reliable

*Do not rely on this report* if your geotechnical engineer prepared it:

- for a different client;
- for a different project;
- for a different site (that may or may not include all or a portion of the original site); or
- before important events occurred at the site or adjacent to it; e.g., man-made events like construction or environmental remediation, or natural events like floods, droughts, earthquakes, or groundwater fluctuations.

Note, too, that it could be unwise to rely on a geotechnical-engineering report whose reliability may have been affected by the passage of time, because of factors like changed subsurface conditions; new or modified codes, standards, or regulations; or new techniques or tools. *If your geotechnical engineer has not indicated an "apply-by" date on the report, ask what it should be*, and, in general, *if you are the least bit uncertain* about the continued reliability of this report, contact your geotechnical engineer before applying it. A minor amount of additional testing or analysis – if any is required at all – could prevent major problems.

## Most of the "Findings" Related in This Report Are Professional Opinions

Before construction begins, geotechnical engineers explore a site's subsurface through various sampling and testing procedures. *Geotechnical engineers can observe actual subsurface conditions only at those specific locations where sampling and testing were performed*. The data derived from that sampling and testing were reviewed by your geotechnical engineer, who then applied professional judgment to form opinions about subsurface conditions throughout the site. Actual sitewide-subsurface conditions may differ – maybe significantly – from those indicated in this report. Confront that risk by retaining your geotechnical engineer to serve on the design team from project start to project finish, so the individual can provide informed guidance quickly, whenever needed.

## This Report's Recommendations Are Confirmation-Dependent

The recommendations included in this report – including any options or alternatives – are confirmation-dependent. In other words, *they are not final*, because the geotechnical engineer who developed them relied heavily on judgment and opinion to do so. Your geotechnical engineer can finalize the recommendations *only after observing actual subsurface conditions* revealed during construction. If through observation your geotechnical engineer confirms that the conditions assumed to exist actually do exist, the recommendations can be relied upon, assuming no other changes have occurred. *The geotechnical engineer who prepared this report cannot assume responsibility or liability for confirmation-dependent recommendations if you fail to retain that engineer to perform construction observation.*

## This Report Could Be Misinterpreted

Other design professionals' misinterpretation of geotechnical-engineering reports has resulted in costly problems. Confront that risk by having your geotechnical engineer serve as a full-time member of the design team, to:

- confer with other design-team members,
- help develop specifications,
- review pertinent elements of other design professionals' plans and specifications, and
- be on hand quickly whenever geotechnical-engineering guidance is needed.

You should also confront the risk of constructors misinterpreting this report. Do so by retaining your geotechnical engineer to participate in prebid and preconstruction conferences and to perform construction observation.

## Give Constructors a Complete Report and Guidance

Some owners and design professionals mistakenly believe they can shift unanticipated-subsurface-conditions liability to constructors by limiting the information they provide for bid preparation. To help prevent the costly, contentious problems this practice has caused, include the complete geotechnical-engineering report, along with any attachments or appendices, with your contract documents, *but be certain to note conspicuously that you've included the material for informational purposes only*. To avoid misunderstanding, you may also want to note that "informational purposes" means constructors have no right to rely on the interpretations, opinions, conclusions, or recommendations in the report, but they may rely on the factual data relative to the specific times, locations, and depths/elevations referenced. Be certain that constructors know they may learn about specific project requirements, including options selected from the report, *only* from the design drawings and specifications. Remind constructors that they may perform their own studies if they want to, and *be sure to allow enough time* to permit them to do so. Only then might you be in a position to give constructors the information available to you, while requiring them to at least share some of the financial responsibilities stemming from unanticipated conditions. Conducting prebid and preconstruction conferences can also be valuable in this respect.

## Read Responsibility Provisions Closely

Some client representatives, design professionals, and constructors do not realize that geotechnical engineering is far less exact than other engineering disciplines. That lack of understanding has nurtured unrealistic expectations that have resulted in disappointments, delays, cost overruns, claims, and disputes. To confront that risk, geotechnical engineers commonly include explanatory provisions in their reports. Sometimes labeled "limitations," many of these provisions indicate where geotechnical engineers' responsibilities begin and end, to help others recognize their own responsibilities and risks. *Read these provisions closely*. Ask questions. Your geotechnical engineer should respond fully and frankly.

## Geoenvironmental Concerns Are Not Covered

The personnel, equipment, and techniques used to perform an environmental study – e.g., a "phase-one" or "phase-two" environmental site assessment – differ significantly from those used to perform a geotechnical-engineering study. For that reason, a geotechnical-engineering report does not usually relate any environmental findings, conclusions, or recommendations; e.g., about the likelihood of encountering underground storage tanks or regulated contaminants. *Unanticipated subsurface environmental problems have led to project failures*. If you have not yet obtained your own environmental information, ask your geotechnical consultant for risk-management guidance. As a general rule, *do not rely on an environmental report prepared for a different client, site, or project, or that is more than six months old.*

## Obtain Professional Assistance to Deal with Moisture Infiltration and Mold

While your geotechnical engineer may have addressed groundwater, water infiltration, or similar issues in this report, none of the engineer's services were designed, conducted, or intended to prevent uncontrolled migration of moisture – including water vapor – from the soil through building slabs and walls and into the building interior, where it can cause mold growth and material-performance deficiencies. Accordingly, ***proper implementation of the geotechnical engineer's recommendations will not of itself be sufficient to prevent moisture infiltration.*** Confront the risk of moisture infiltration by including building-envelope or mold specialists on the design team. ***Geotechnical engineers are not building-envelope or mold specialists.***

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May 18, 2020
NC Proj. No. 201881.5

Mr. J. Voss
Stevens Creek Quarry, Inc.
12100 Stevens Canyon Road
Cupertino, CA 95014

RE: Preliminary Analysis of Slope Failure at the Northeast Corner of the Stevens Creek Quarry.
12100 Stevens Canyon Road
Cupertino, CA 95014

Dear Mr. Voss,

The County of Santa Clara sent a letter, dated October 23, 2019, to Stevens Creek Quarry concerning the Quarry's Reclamation Plan Amendment. In Section II, Paragraph 10 (p.5) in that letter, the county stated:

> The County has observed significant ground-cracks at the top of the slope of the north and east wall of the Quarry pit. The RPA will need to include a geotechnical report that evaluates ground cracks and the stability buttress fill that is already in place.

At your request, I performed an initial evaluation of the cause of that cracking and landsliding in the Northeast (NE) corner of the quarry.

**Field Observations**

I visited the site on May 8, 2019. Figure 1 is a June, 2019 Google aerial photograph of the NE corner of the quarry. It shows the location of the features discussed in this report and will be used as an index map. Photos 1 and 2 show oblique views of the NE corner of the quarry, the location of Pad 1, and the recent landslide.

The settling pond (Photos 4 and 5) is located at the north end of Pad 1. Runoff from the uphill area is directed into the pond. Silt settles into the pond, and the runoff flows out though the pipe at the east end of the pond. That water flows to the west side of the haul road (Figure 1) and then flows down an unlined v-ditch at the side of the haul road to the base of the pit. Note that the pipes are not placed to completely drain the pond. The water remaining in the pond either evaporates or seeps into the ground. I did not perform hydraulic calculations on the pond.

Photos 6 and 7 show Pads 1 and 2 (looking south). They are flat and have berms along their outside edges. Photos 7 and 8 show the ground cracks in the south end of Pad 2. The cracks exhibit both horizontal and vertical offset (south side down). The orientation of the northern crack in Pad 2 indicates that the crack dip is fairly steep. The cracks extend west into Pad 1, where they extend one-half to three-quarters of the way across Pad 1. The cracks narrow as they go into Pad 1. Cracking did not appear to extend to the west side of Pad 1.

I was unable to trace the cracks southeast of Pad 2.

**Google Aerial Photograph Evaluation**

I downloaded a series of aerial photographs from Google Earth, from March, 2003 to June, 2019 (the most recent photograph available). Evaluation of these photographs was done to evaluate the timing, location, and nature of fill placement in this part of the quarry. Not all photographs within this time frame were downloaded. Some of the photographs were cloud covered or had insufficient resolution.

The discussion for each photograph is on the Figures (not in the report text). Figure 1 is an index to the features discussed in the following Figures. Figures 2 through 10 illustrate the mining history of the NE corner (2014 through 2019). Figure 11 shows the conditions that existed just prior to development of the large cracks (fractures) (October, 2017).

**Analysis and Conclusions**

The fill material in this area is similar to fill material in other parts of the quarry. The fill soil properties in this area are consistent with other fill slopes in the quarry.

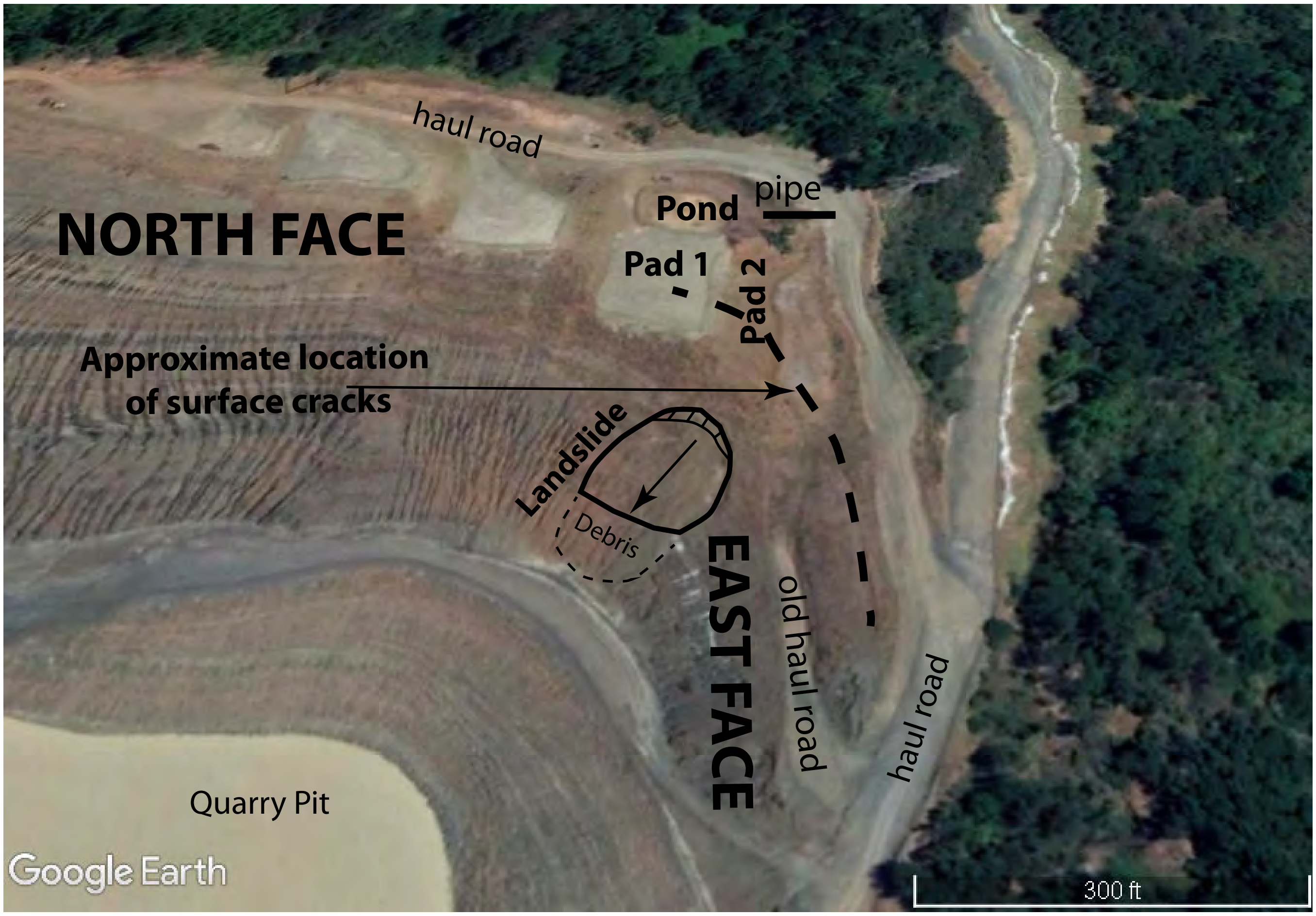
The recent surface landslide (Photos 2 and 3) was caused by the rains during the 2019-20 winter (likely in December, 2019). It is a surface feature and does not present a global slope stability problem. The white, horizontal bands visible on the slope face to the right of the lower part of the landside on Photo 3 are suggestive of groundwater evaporation from that zone. Water evaporates, leaving calcium minerals on the ground surface. On a smaller scale, the process is called efflorescent deposition. This is most likely unsaturated water flow. Because of the excavated geometry of this area (a curved valley), it appears that subsurface water concentrates in this area. There are no seeps/vegetation growth that would indicate saturated flow (current or historic) flowing out of the fill.

The fractures developed during the winter of 2017-2018. There were heavy rainfalls throughout that winter. Rainfall totals reached 100% of historic averages by early February and the rains continued into March. Figure 11 shows the relationships of surface and subsurface slope features in October, 2018, prior to development of the fractures. The location of the fractures and the buried gully (Figure 2) are approximately shown. The fill face in this area appears to have had its historically steepest slope angle. Note the curved road cut into the base of the slope at this location. These geometrical relationships coupled with a very rainy winter caused incipient slope failure, creating the cracks at the top of the slope. Subsequently, additional fill was placed on the toe, helping to stabilize it (reducing the slope angle).

The minor extension of the fractures into Pad 2 in 2018 (Figure 10) suggest that this slope is still marginally stable. Placing additional fill at the bottom of this slope would increase the Factor of Safety by further reducing the slope angle.

The settling pond does allow water to seep in the ground episodically during the winter (depending on rain events). I doubt it was a direct contributory factor to the 2017 failure, but it likely had some effect. The surficial slope surface failure during the 2018-19 winter suggests that this area has a higher fill soil moisture content during the winter than surrounding areas because of the geometry of this area.

Google photograph date: June, 2019



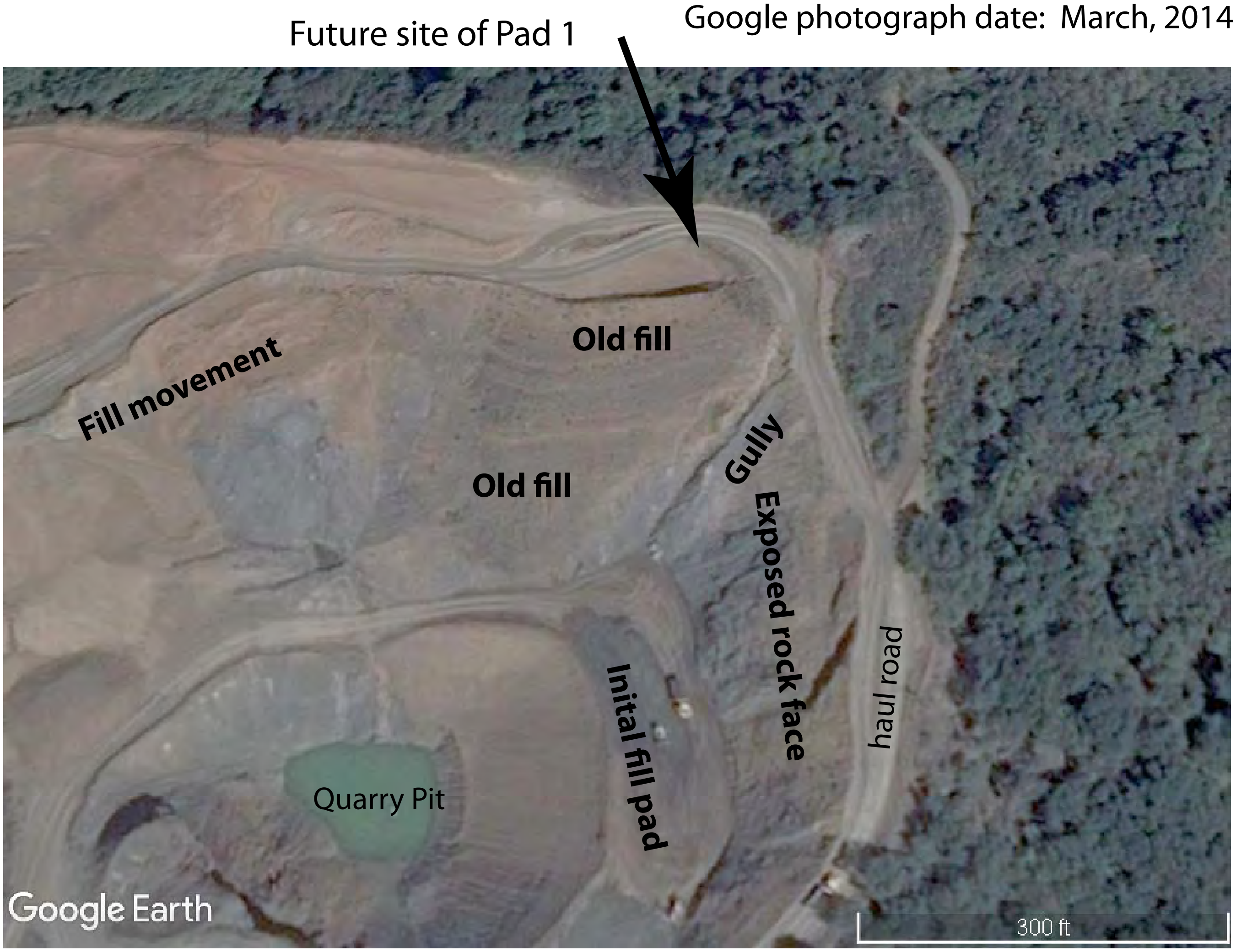

~N

## Index Map

This photo shows the approximate location of features that are the subject of this report. This is the most current photo of this area on Google Earth. The location of the landslide is approximate. That landslide likely occurred October-November, 2019, after this photograph was taken.

| Norfleet Consultants | North East Corner of Stevens Creek Quarry Pit | | |
|---|---|---|---|
| | Index Map - June, 2019 | | |
| | PROJ NO: 201881.5 | DATE: 2-25-2020 | FIGURE: 1 |

Google photograph date:  March, 2014




This photo shows the quarry soon after mining of the eastern face stopped (labeled - exposed rock).  New fill is beginning to be placed along the base of the east face (initial fill pad).  Note the gully that marks the junction between exposed rock and the old fill that had been placed on the north face.  As fill was placed in this corner of the quarry, the gully formed a buried valley and a focus for groundwater.

Old fill is several years old and can be identifed by bushes growing on  it.  At this time, new fill is also being placed/stored on the west side of the north face (fill movement).  Note the shifting of pond at the bottom of the pit (quarry pit) as mining progresses.

| | | | |
|---|---|---|---|
| **Norfleet Consultants** | North East Corner of Stevens Creek Quarry Pit | | |
| | March, 2014 | | |
| | PROJ NO: 201881.5 | DATE: 2-25-2020 | FIGURE: 2 |

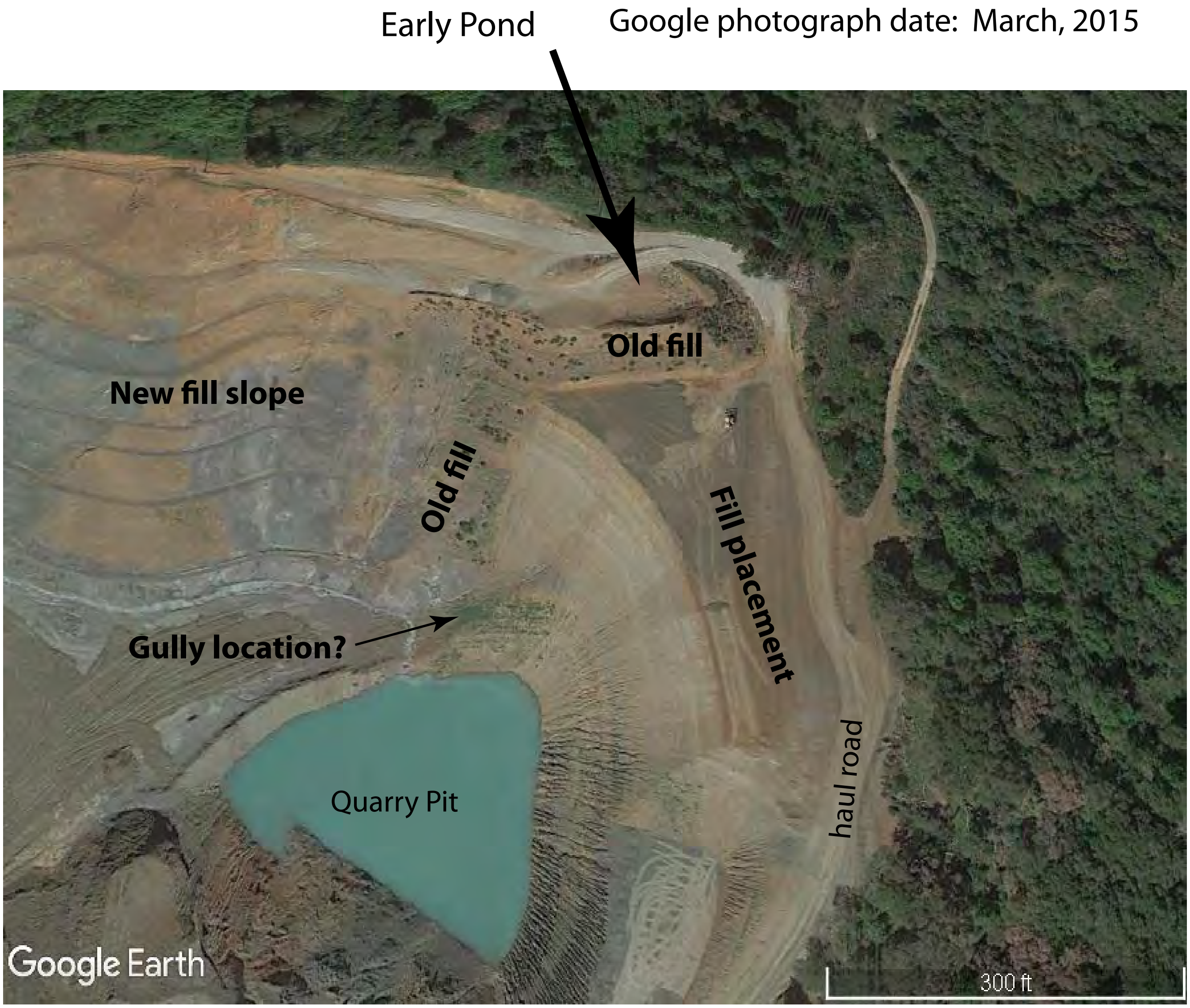
Google photograph date: November 2014

Future site of Pad 1


~N

Fill continues to be placed on the east quarry face. New fill has also been placed on the north face. Just a small area of old fill remains unburied. The arrow indicates the approximate future location of pads 1 and 2 and the pond. Only the upper portion of the gully is visible.

| | | | |
|---|---|---|---|
| **Norfleet Consultants** | North East Corner of Stevens Creek Quarry Pit | | |
| | November, 2014 | | |
| | PROJ NO: 201881.5 | DATE: 2-25-2020 | FIGURE: 3 |

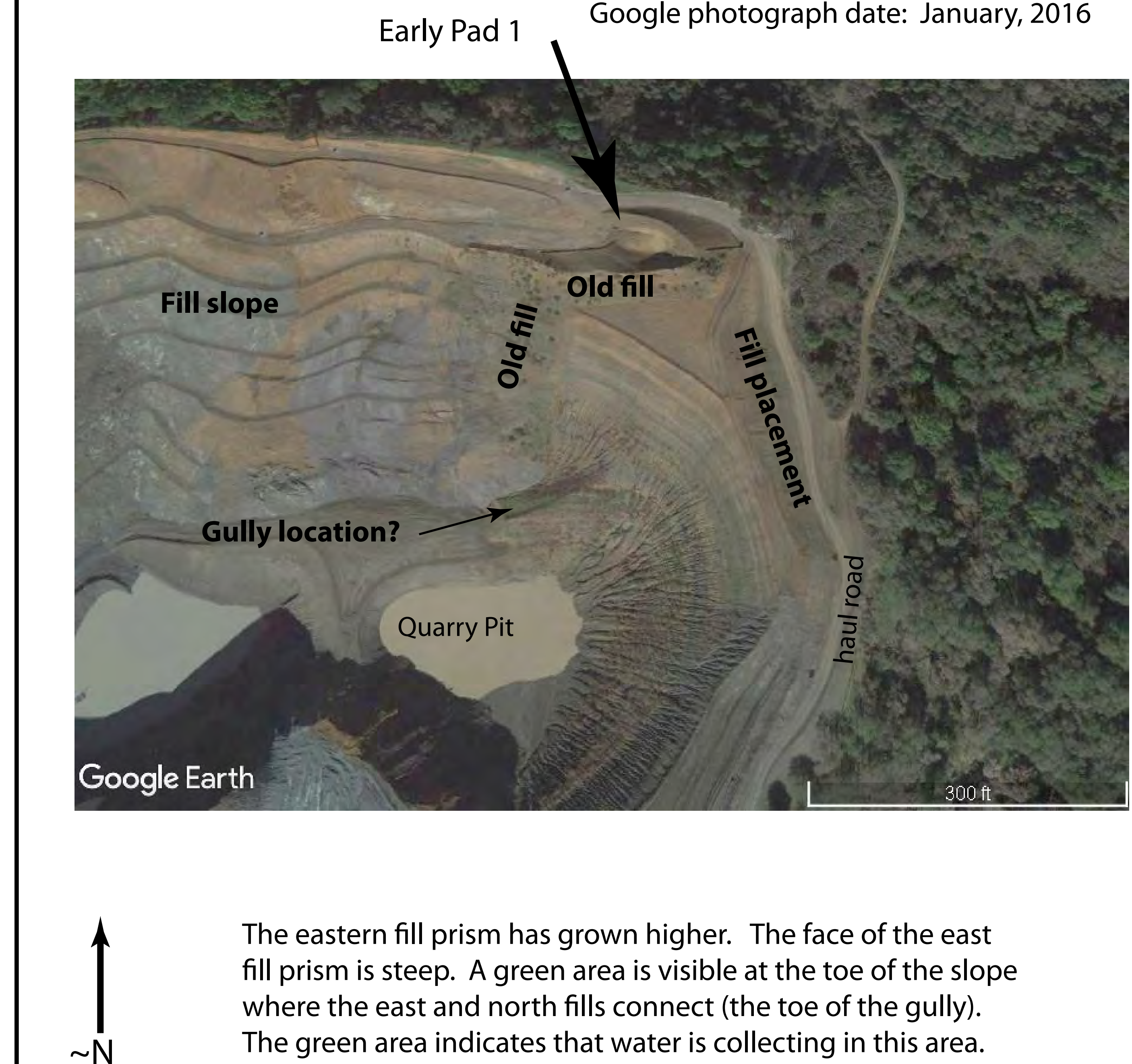




The fill prism on the east face has continued to grow. Only small areas of old fill in the NE corner remain exposed. The gully is completely covered. It appears that a pond or pad had been constructed at the top of the NE quarry corner.

| Norfleet Consultants | North East Corner of Stevens Creek Quarry Pit | | |
|---|---|---|---|
| | March, 2015 | | |
| | PROJ NO: 201881.5 | DATE: 2-25-2020 | FIGURE: 4 |

Google photograph date:  January, 2016




The eastern fill prism has grown higher.  The face of the east fill prism is steep.  A green area is visible at the toe of the slope where the east and north fills connect (the toe of the gully). The green area indicates that water is collecting in this area. The photo was taken in January and it is likely that surface water is concentrating in this area.

| | | | |
|---|---|---|---|
| **Norfleet Consultants** | North East Corner of Stevens Creek Quarry Pit | | |
| | January, 2016 | | |
| | PROJ NO: 201881.5 | DATE: 2-25-2020 | FIGURE: 5 |

Google photograph date: November, 2016

Creation of Pond ?


~N

Fill continues to be placed on the north face and in the north-east corner. Fill is also being placed at the location of the current pond. Only a small sliver of old fill is visible. Note the shifting of the quarry pit.

| | | | | | |
|---|---|---|---|---|---|
| **Norfleet Consultants** | North East Corner of Stevens Creek Quarry Pit | | | | |
| | November, 2016 | | | | |
| | PROJ NO: 201881.5 | | DATE: 2-25-2020 | | FIGURE: 6 |

Settling pond

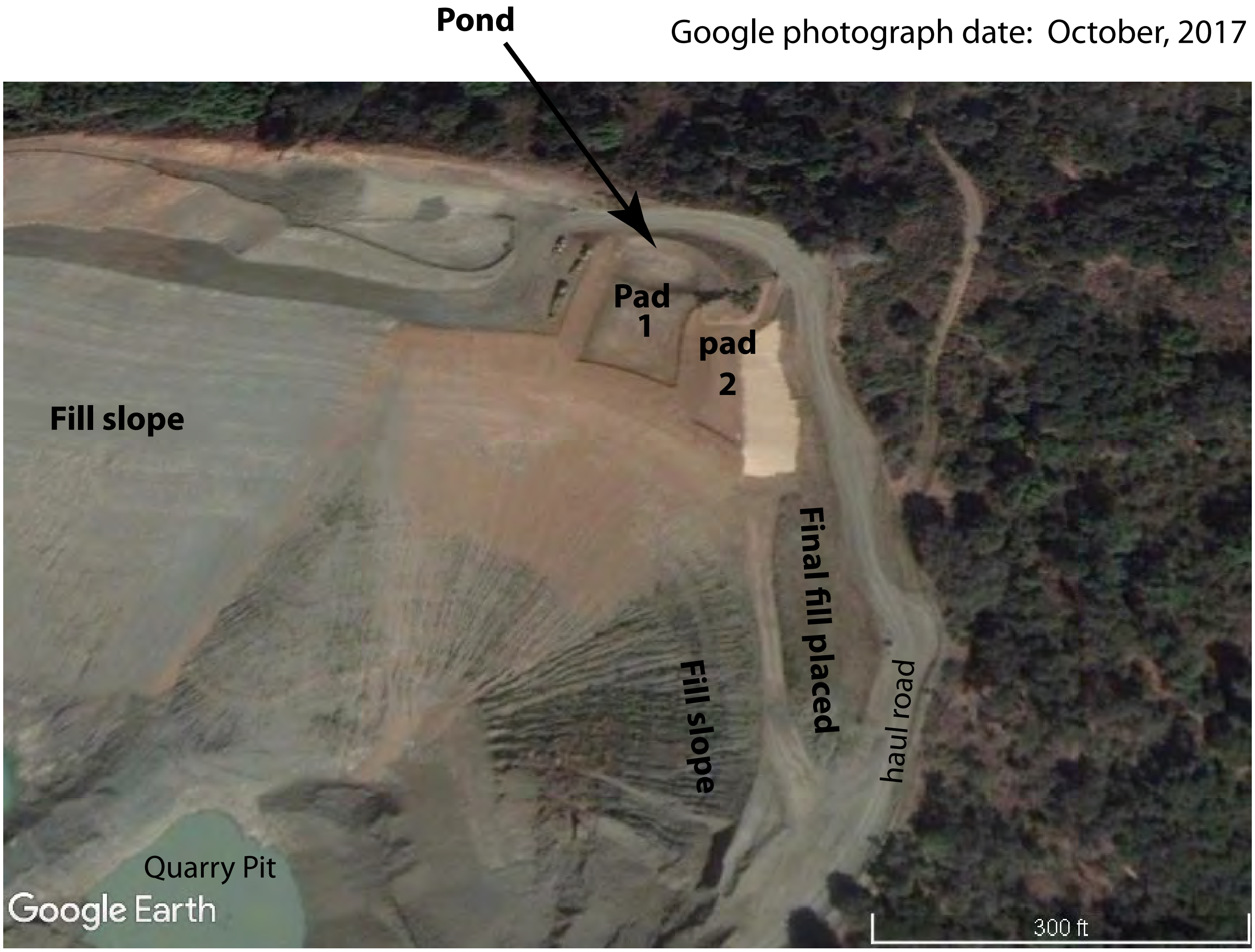
Google photograph date: September, 2017

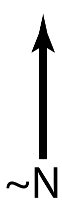

~N

Fill continued to be placed on the north slope. Fill was removed from the base of the east slope (note the new quarry pit), and the top of the east face fill prism has narrowed slightly (see fill configuration on Figure 6.

| | | | |
|---|---|---|---|
| **Norfleet Consultants** | North East Corner of Stevens Creek Quarry Pit | | |
| | September, 2017 | | |
| | PROJ NO: 201881.5 | DATE: 2-25-2020 | FIGURE: 7 |

Google photograph date: October, 2017

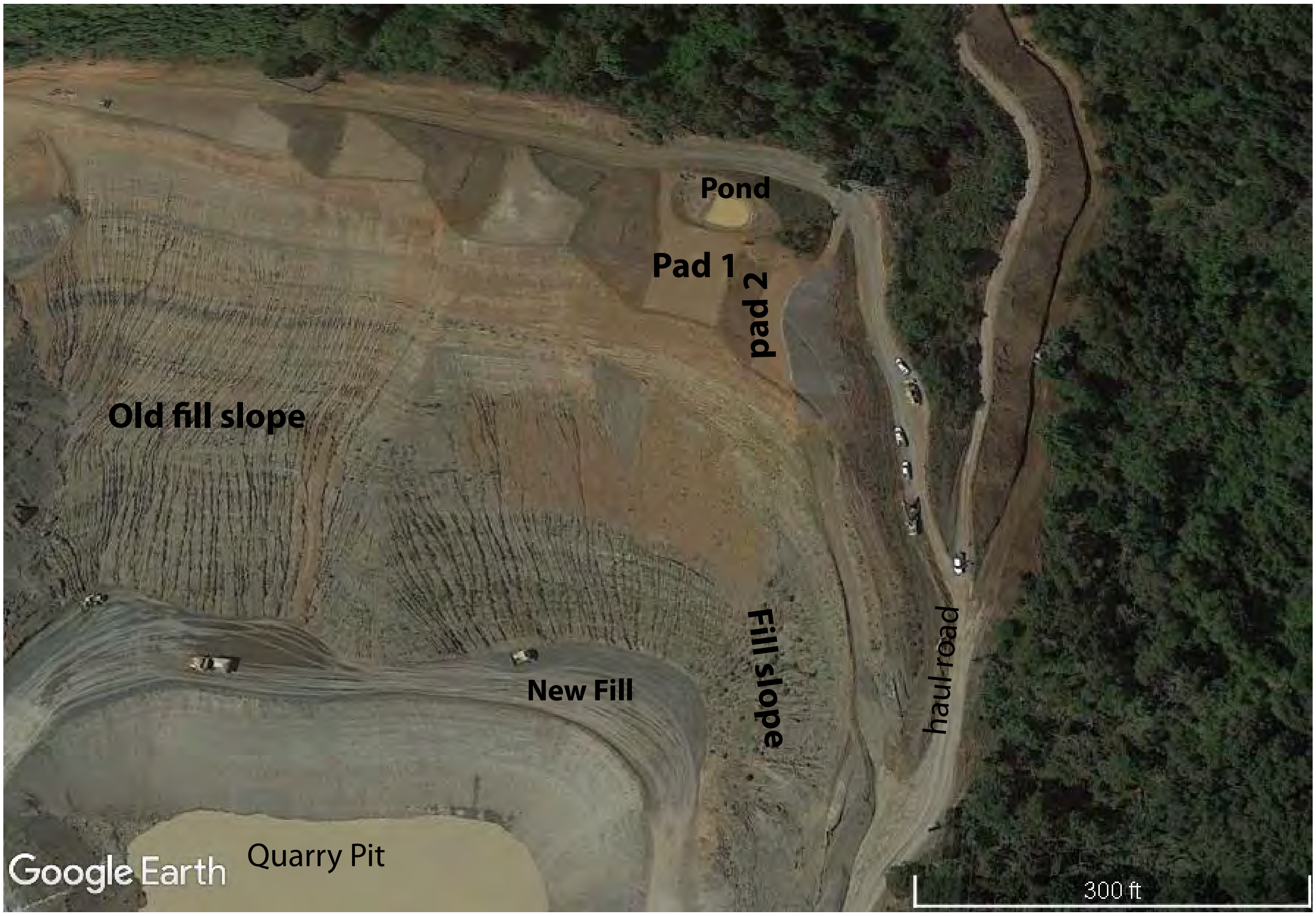



The current pads and settling pond are in place. No old fill is visible. Additional fill was placed at the top of the east face fill prism.

| | |
|---|---|
| **Norfleet Consultants** | Northeast Corner of Stevens Creek Quarry Pit |
| | October, 2017 |
| | PROJ NO: 201881.5 / DATE: 2-25-2020 / FIGURE: 8 |

Google photograph date:  May, 2018




~N

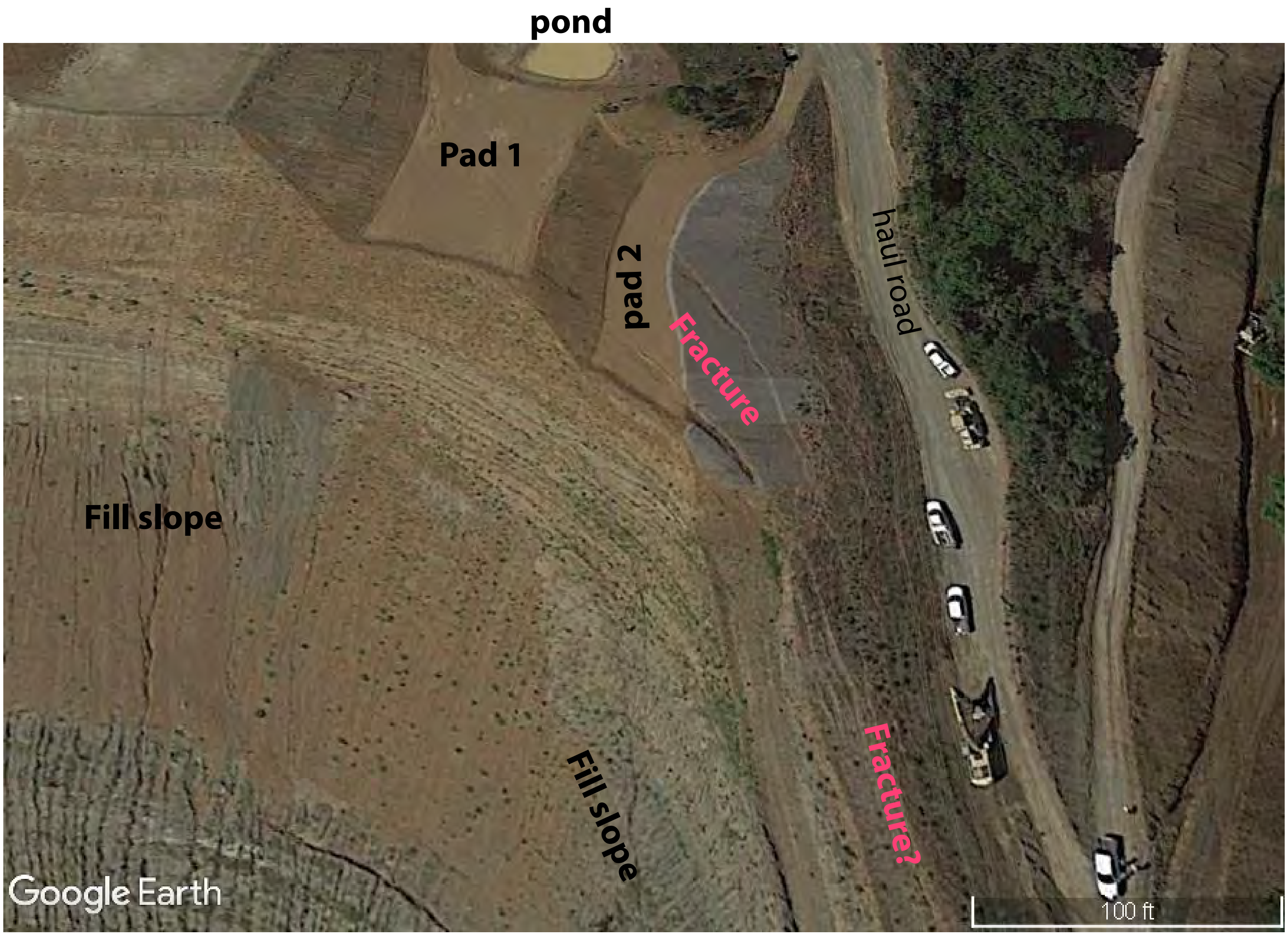
The quarry has deepened.  Fractures have developed along the south edges of pads 1 and 2 (see next figures).

The winter of 2017-2018 was quite rainy.  One hundred percent of historic rainfall had fallen by early February and rain continued into March.  Note the gullying in the fill faces.

No landsliding is visible on the quarry faces.

| | | | |
|---|---|---|---|
| **Norfleet Consultants** | North East Corner of Stevens Creek Quarry Pit | | |
| | May, 2018 | | |
| | PROJ NO: 201881.5 | DATE: 2-25-2020 | FIGURE: 9 |

Google photograph date: May, 2018

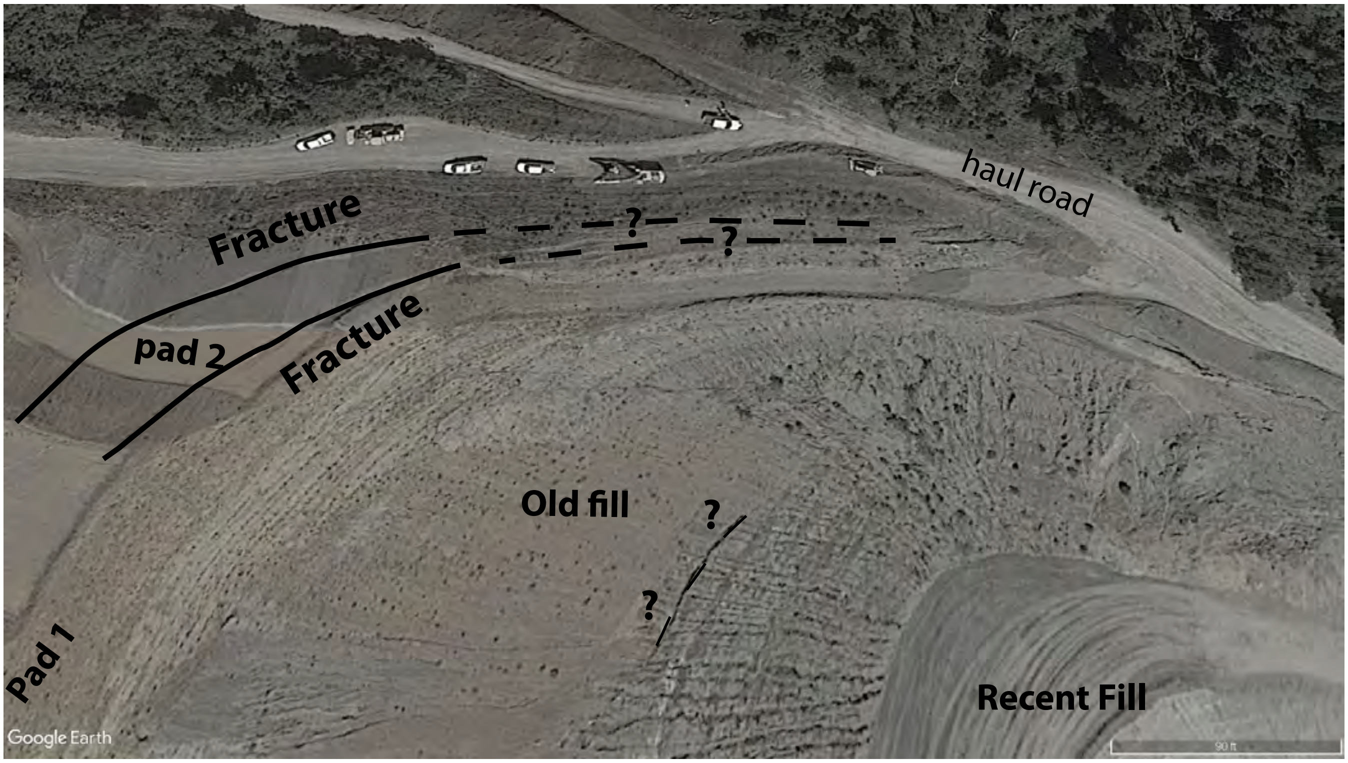



~N

This is an enlargement of the photograph in figure 9. The fractures extend from the east side of Pad 1, across Pad 2 and curve south along the top of the east fill prism. The northern end of of the two fractures is easy to see, but their southern extent is unclear (hence the ?).

| Norfleet Consultants | North East Corner of Stevens Creek Quarry Pit | | |
|---|---|---|---|
| | May, 2018 | | |
| | PROJ NO: 201881.5 | DATE: 2-25-2020 | FIGURE: 9a |

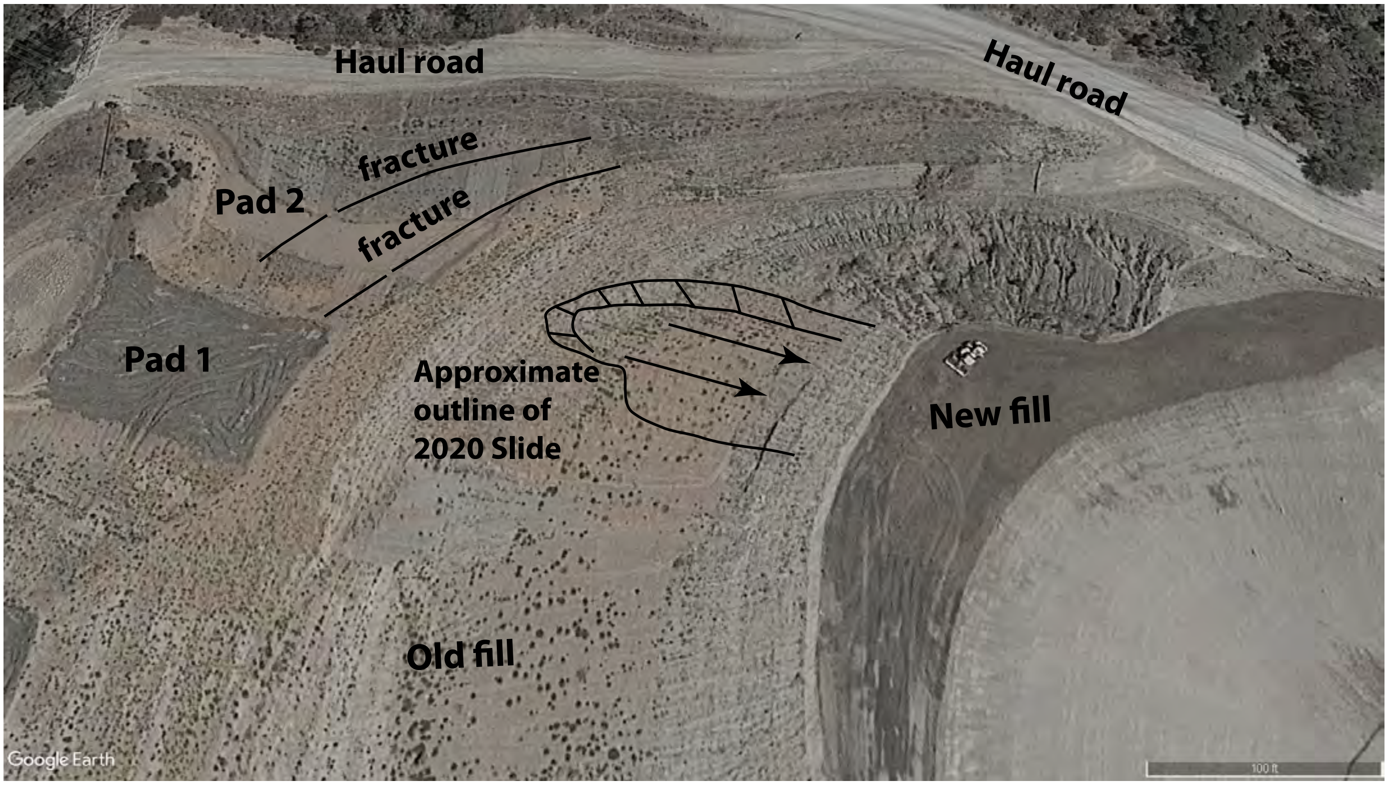
Google photograph date: May, 2018




This is an oblique view of the Figure 9a photograph. The fracture traces are shown (dashed where uncertain). There maybe a fracture in the fill face at the approximate location of the future fill face landslide.


| Norfleet Consultants | North East Corner of Stevens Creek Quarry Pit, Oblique View | | |
|---|---|---|---|
| | May, 2018 | | |
| | PROJ NO: 201881.5 | DATE: 2-25-2020 | FIGURE: 9b |

Google photograph date: August, 2018




~N

This is the most recent photograph on Google Earth. The approximate location and size of the currently visible landslide on the fill face is shown. I observed that landslide in May, 2019. See Figures 2 and 3.

It appears that the surface of Pad 1 was smoothed in early 2018. The current extension of the fractures into Pad 1 indicates that there has been minor movement on the fractures.

| | | | |
|---|---|---|---|
| **Norfleet Consultants** | North East Corner of Stevens Creek Quarry Pit | | |
| | August, 2018 | | |
| | PROJ NO: 201881.5 | DATE: 2-25-2020 | FIGURE: 10 |

Google photograph date: October, 2017


~N

These are the physical relationships just prior to when the fractures formed (winter of 2017-18). The fractures occurred at the crest of the slope and define the zone of movement. The fill above the buried gully is at its the thickest when compared to either side (see Figure 2). This section of the face also has its steepest historical slope because fill was removed from the base of this slope (compare with Figure 6).

| | | | |
|---|---|---|---|
| **Norfleet Consultants** | North East Corner of Stevens Creek Quarry Pit | | |
| | October, 2017 | | |
| | PROJ NO: 201881.5 | DATE: 2-25-2020 | FIGURE: 11 |

**LIMITATIONS OF THIS REPORT**

This report was prepared at the request of, and for the exclusive use of the addressee. Release to any other company, concern, or individual is solely the responsibility of the addressee. Norfleet Consultants is an independent consultant who was retained to provide a preliminary evaluation of slope instability causes. Any other use of this report is strictly forbidden by Norfleet Consultants.

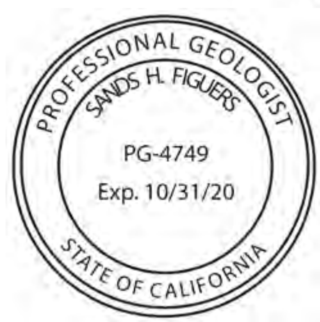
We have employed generally accepted civil engineering, engineering geology, and geophysical procedures. Our observations, professional opinions and conclusions were made using that degree of care and skill ordinarily exercised, under similar conditions, by civil engineers engineering geologists, geophysicists practicing in this area at this time. Norfleet Consultants expressly denies any third party liability arising from the unauthorized use of this report.

If you have any questions, please contact us at 925-606-8595.

Yours truly,
Norfleet Consultants



S. Figuers



Photograph 1. An oblique view, looking east towards the NE corner of the quarry (from the NW corner). The horizontal arrow marks the 2019 landslide. The vertical arrow marks pad 1.

Photograph 2. The east end of the north face, looking north. The horizontal arrow marks the 2019 landslide. The vertical arrow marks pad 1. The white band marks are on the east quarry face and are a possible indication of water seepage.

Photograph 3. An enlarged area from Photograph 2. The landslide is visible and Pad 1 is above the landslide. The landslide toe does not extend to the bottom of the slope.

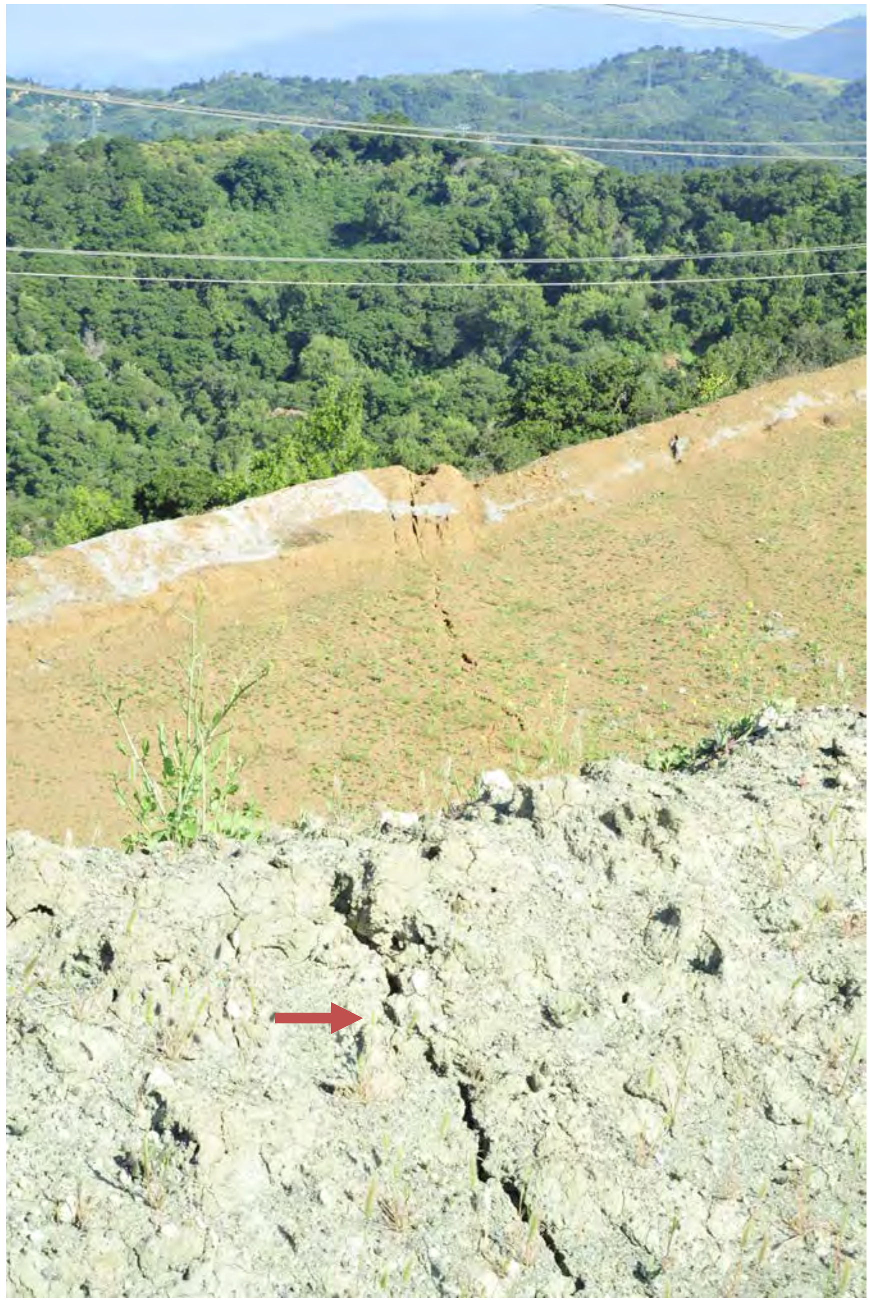
Photograph 4. The settling pond at the north end of Pad 1, looking east. Water flows into the pond through the pipe in the foreground. It leaves the pond through the pipe opening (black dot) in the far bank. Pad 1 is on the right. Note the bathtub rings that mark water levels in the pond.

Photograph 5. The settling pond, looking NW. Water enters the pond through the pipe visible below the truck.

Photograph 6. Pad 1, looking south towards the pit face.

Photograph 7. Pad 2 as viewed from Pad 1. Ground cracks are visible at the south end of Pad 2.

Photograph 8. Ground cracks at the south end of Pad 2 (visible in Photo 7). Looking from Pad 1 (foreground). The cracks extend into Pad 1 (arrow).



Photograph 9.

Extension of a crack in Pad 2 into Pad 1 (arrow). The crack extends about 3/4 of the width of Pad 1 (to the south). The orientation of the crack suggests that the failure plane dip is fairly steep.

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Mr. J. Voss
Stevens Creek Quarry, Inc.
12100 Stevens Canyon Road
Cupertino, CA 95014

September 19, 2020
NC Proj. No. 201881.5

RE: Stability Assessment of the Proposed Cut Slope of the Northwest Corner of the Stevens Creek Quarry.
12100 Stevens Canyon Road
Cupertino, CA 95014

Dear Mr. Voss,

At your request, I performed a slope stability assessment of the proposed final 2:1 cut slope at the northwest (NW) corner of the quarry (with and without fill). Rock and soil properties in this area were evaluated by Norfleet (2008), BaGG (2019), and Stantec (2019).

**Slope Stability Analysis**

The location of the evaluated slope profile was provided by Benchmark Resources. The cross-section is the longest part of the proposed cut. The cut slope will have a 2:1, east dipping slope (~1550 feet long with a ~750 vertical elevation change). The cut is primarily in weathered greenstone bedrock, but a thin layer (40-60 feet) of deeply weathered bedrock will crop out around the western rim of the cut. This material has been observed by BAGG, Norfleet, and Cotton and Shires. A large, flat topped fill pad will be constructed along the eastern side of the cut slope. It will have a 200 foot maximum thickness.

Slope stability analyses were performed to evaluate the stability of the cut slope. The first analysis (Figures 2 and 3) evaluated the overall stability of the cut slope (without fill pad). The second (Figures 4 and 5) evaluated the stability of the near surface weathered greenstone that would remain at the top of the west end of the cut slope . The third set (Figures 5 and 6) evaluated the overall stability of the cut slope with the 200 foot thick fill pad.

To be conservative, the material properties used (Table 1) are at the lower end of the range of property values discussed in Norfleet (2008), BaGG (2019), and Stantec (2019). The results of the stability analyses are listed in Tables 2 and 3. The static evaluation was performed using the modified Bishop Method. Seismic displacement was determined by a Newmark analysis.

Table 1. Rock Material Properties.

| Rock type | Graph No | Total Unit Wt. (pcf.) | Cohesion (pcf.) | Phi Angle (deg.) |
|---|---|---|---|---|
| Sheared Greenstone | 1 | 130 | 500 | 31 |
| Weathered Sheared Greenstone | 2 | 120 | 300 | 26 |
| Fill | 3 | 150 | 150 | 31 |

Table 2 Results of stability analysis (without fill pad). The PGA was 0.6g

| Rock Type Analyzed | Static FS | Seismic FS | Seismic Displacement |
|---|---|---|---|
| Sheared Greenstone | 1.39 | 0.82 | 4.6 cm |
| Weathered Sheared Greenstone | 1.56 | 0.94 | 1.2 cm |

Table 3 Results of stability analysis (with fill pad). The PGA was 0.6g

| Rock Type Analyzed | Static FS | Seismic FS | Seismic Displacement |
|---|---|---|---|
| Sheared Greenstone | 1.55 | 0.9 | 2.2 cm |
| Weathered Sheared Greenstone | 1.56 | 0.94 | 1.2 cm |

**Cotton and Shires Proposed Landslide**

Cotton and Shires (2020 a and b) prepared a PowerPoint presentation for the City of Cupertino based on Google Earth historic aerial photographs of the Permanete Quarry property and the Stevens Creek Quarry areas. On Google photographs of the NW corner of the Stevens Creek Quarry area (their Figures 37 and 38) they noted a feature that appeared to be the headscarp of a large, deep-seated landslide that they called the NW Wall Landslide (Figure 7). This proposed landslide was more than 1000 feet long, hundreds of feet wide and deep, with the toe extending into the floor of the quarry.

This feature first appeared in a November, 2014, Google photograph. The presence of what they interpreted as a recent head scarp (Cotton and Shire's Figure 38 and my Figure 8) suggested to them that this landslide is recent. There is no field evidence to support this conjecture. What is visible on the Google photographs is a shadow. Cotton and Shire interpreted this shadow as a ditch. It could just as likely be a small earthen berm created by grading. The top of this hill, and many flat surfaces in the surrounding area show evidence of the quasi-annual grass/brush cutting fire prevention program. Grading is a common part of this activity.

Cotton and Shire (2020b, p 4) further noted: "It appears that attempts were made to grade some drainage ditches in the headscarp area to divert water away from the slide". The drainage ditch appears to have been cut directly within and along the headscarp (as proposed by Cotton and Shires). Both Stevens Creek and Permanente Quarry operators have years of experience dealing with landslides. Excavating a ditch within a headscarp "crack" would only concentrate and direct water into the headscarp, having the opposite effect of what was intended. This would be contradictory to actions taken by companies with decades of experience dealing with landslides.

I observed and photographed the northwest corner of the quarry since that slope was finished prior to visible failure (ca. 2012), as well as the beginnings of slope failure and its development. The initial failure was a localized small feature high on the slope. Over the next five years additional localized failures developed that then coalesced into mass failure of the face. I did not observe failures in the lower third of the slope and there was and still is no visible single basal failure surface. The observed failure style of the slope is inconsistent with the type of failure proposed by Cotton and Shires (2020).

I visited the area (including the hill top) in early September, 2020. The graded area was still visible, but no headscarp was visible. There is a landslide in this area, but it is much smaller and shallower than suggested by Cotton and Shires. This landslide is restricted to the deeply weathered surface bedrock (upper 40 to 50 feet).

**The NW Landslide**

Based on my observations, the failures in the NW face are shallow, not deep-seated. Figure 9 is a Google photograph of the NW face in May, 2018. Landsliding began as several scattered small features in the vicinity of A. As those features enlarged, they destabilized the slope above them (B) and removed lateral support from either side (D) causeing those areas to destabilize. Debris from the upper slides slid down and covered the lower part of the slope. I did not observe landslide deformation in the base of the slope. The failures visible in Figure 8 took 4 to 5 years of slow, progressive movement to develop.

Figure 10 is an oblique view to the east of the top of the slope. The quarry is in the background. The three major features are (from right to left): a slope (labeled ancient landslide), a dissected alluvial fan and associated streams, and a low bedrock hill. The ancient landslide is no longer active. However, it directs surface and ground water west into the alluvial fan. The alluvial fan was formed by soil from the landslide mass. When the soil supply reduced, the alluvial fan began to erode. The streams flow to the east and drain onto the top of the cut face. More importantly, ground water also flows to the east.

Figure 11 illustrates the relationship between the top of the west face and the top of the ridge. Ridge top surface water funnels through a narrow gully on to the top of the west face. The water flows onto the slope above the location where the first landslide developed.

During the winter, groundwater is recharged. As it percolates it will eventually encounter less permeable material and begin to flow quasi-horizontally to the east. The initiation location of landsliding on the NW quarry face (Figure 7, location C) likely marks the area where groundwater flows from the cut face surface. This area is about 1/3 the way down from the top of the slope. Groundwater currently seeps from the face, with the seepage points in the middle third of the slope. There is no visible seepage from either the lower or upper parts of the slope.

**Conclusions**

The static FS of the cut slope (without fill pad) is 1.39. The seismic displacement is 4.6 cm. When the fill pad is constructed, the static FS increase to 1.55 and the seismic displacement decreases to 2.2 cm. These calculation are based on lower bound material strength values.

Surface and groundwater movement around the western rim of the proposed cut slope within the weathered zone should be assumed. Appropriate surface and subsurface preventive measures should be taken to gather/control this water. Currently, the weathered rock zone gathers and directs groundwater east towards the quarry. It is one of the drivers of the current landslide movement on the NW face. Bedrock in this area is sheared. It is likely that the operator will encounter variable strength rock in this area (no wide-spread rock strength consistency). The weathered/less weathered rock interface is gradational and will vary in depth and thickness. I found no evidence for the large-scale, deep-seated landslide proposed by Cotton and Shires.

**LIMITATIONS OF THIS REPORT**

This report was prepared at the request of, and for the exclusive use of the addressee. Release to any other company, concern, or individual is solely the responsibility of the addressee. Norfleet Consultants is an independent consultant who was retained to provide a preliminary evaluation of slope instability causes. Any other use of this report is strictly forbidden by Norfleet Consultants.

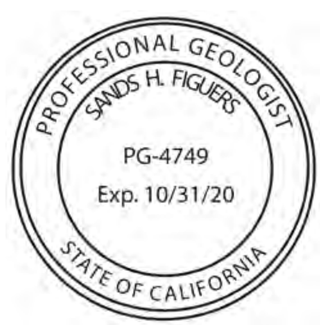
We have employed generally accepted civil engineering and engineering geology procedures. Our observations, professional opinions and conclusions were made using that degree of care and skill ordinarily exercised, under similar conditions, by civil engineers engineering geologists, geophysicists practicing in this area at this time. Norfleet Consultants expressly denies any third party liability arising from the unauthorized use of this report.

If you have any questions, please contact us at 925-606-8595.

Yours truly,
Norfleet Consultants

S.Figuers

S. Figuers

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Cotton and Shires, 2020a, Outline of Cupertino Quarry Issues Corresponding to Presentation Slides, dated May, 2020, reference G6020, 5pp

Cotton and Shires, 2020b, Geologic and Geotechnical Review of Permanente Quarry and Stevens Creek Quarry Reclamation Plan Amendments, Cupertino, California, dated April, 2020, copy of presentation slides, 57 pp

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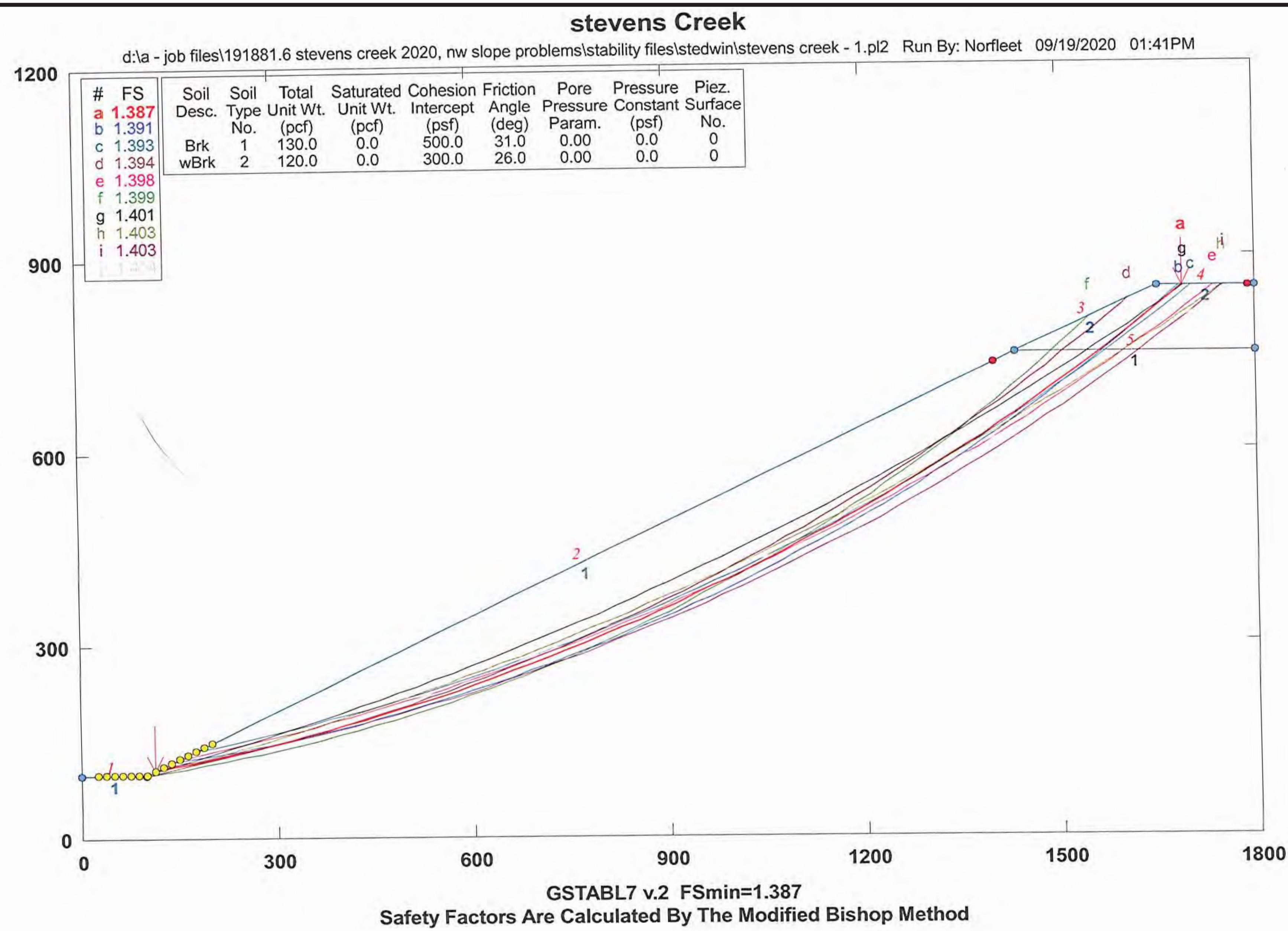
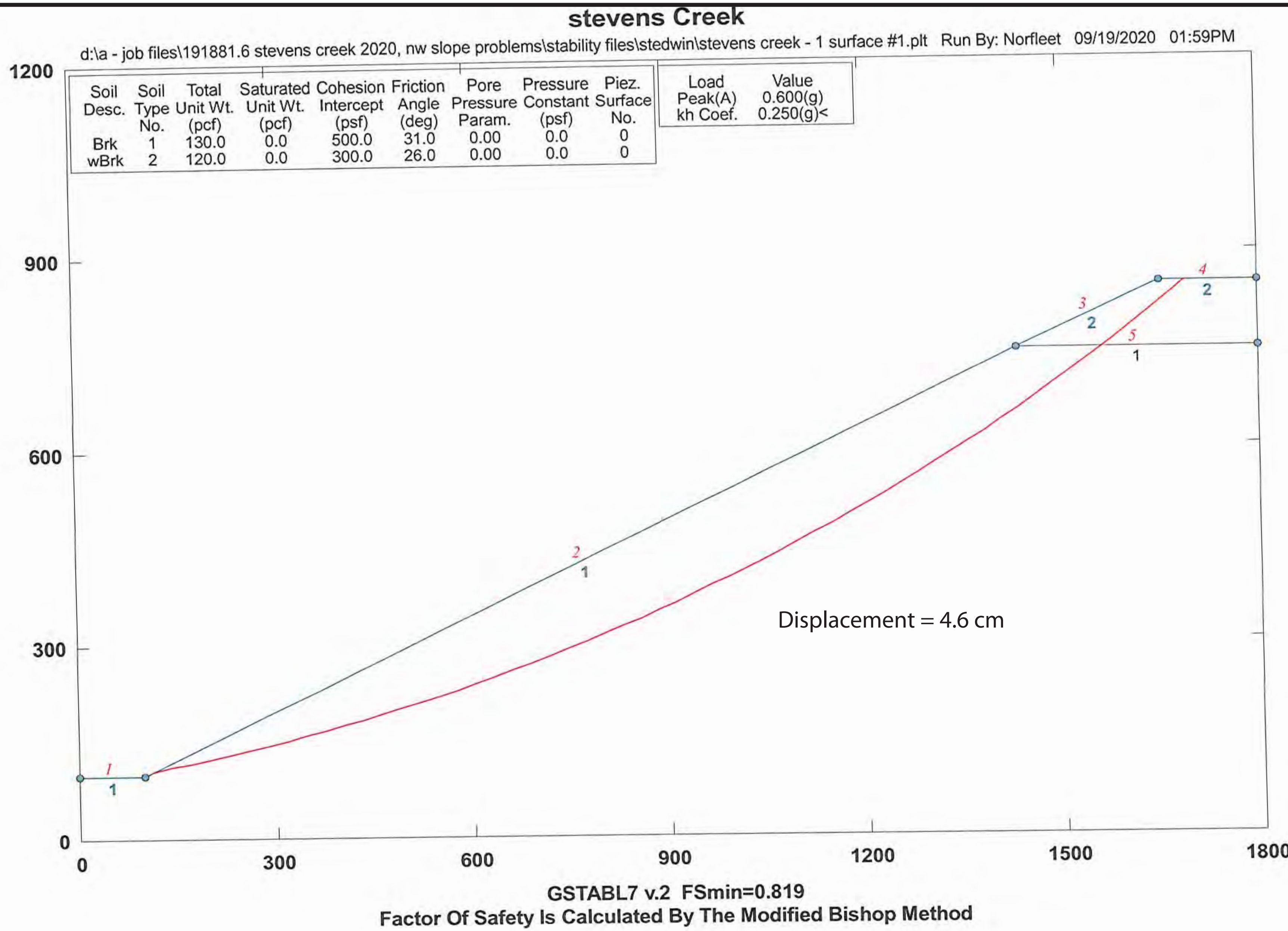



| Norfleet Consultants | Stevens Creek Quarry | | |
|---|---|---|---|
| | Stability Analysis of Full Slope | | |
| | PROJ NO: 191881.6 | DATE: 9-17-2020 | FIGURE: 1 |




| Soil Desc. | Soil Type No. | Total Unit Wt. (pcf) | Saturated Unit Wt. (pcf) | Cohesion Intercept (psf) | Friction Angle (deg) | Pore Pressure Param. | Pressure Constant (psf) | Piez. Surface No. |
|---|---|---|---|---|---|---|---|---|
| Brk | 1 | 130.0 | 0.0 | 500.0 | 31.0 | 0.00 | 0.0 | 0 |
| wBrk | 2 | 120.0 | 0.0 | 300.0 | 26.0 | 0.00 | 0.0 | 0 |

| Load | Value |
|---|---|
| Peak(A) | 0.600(g) |
| kh Coef. | 0.250(g)< |

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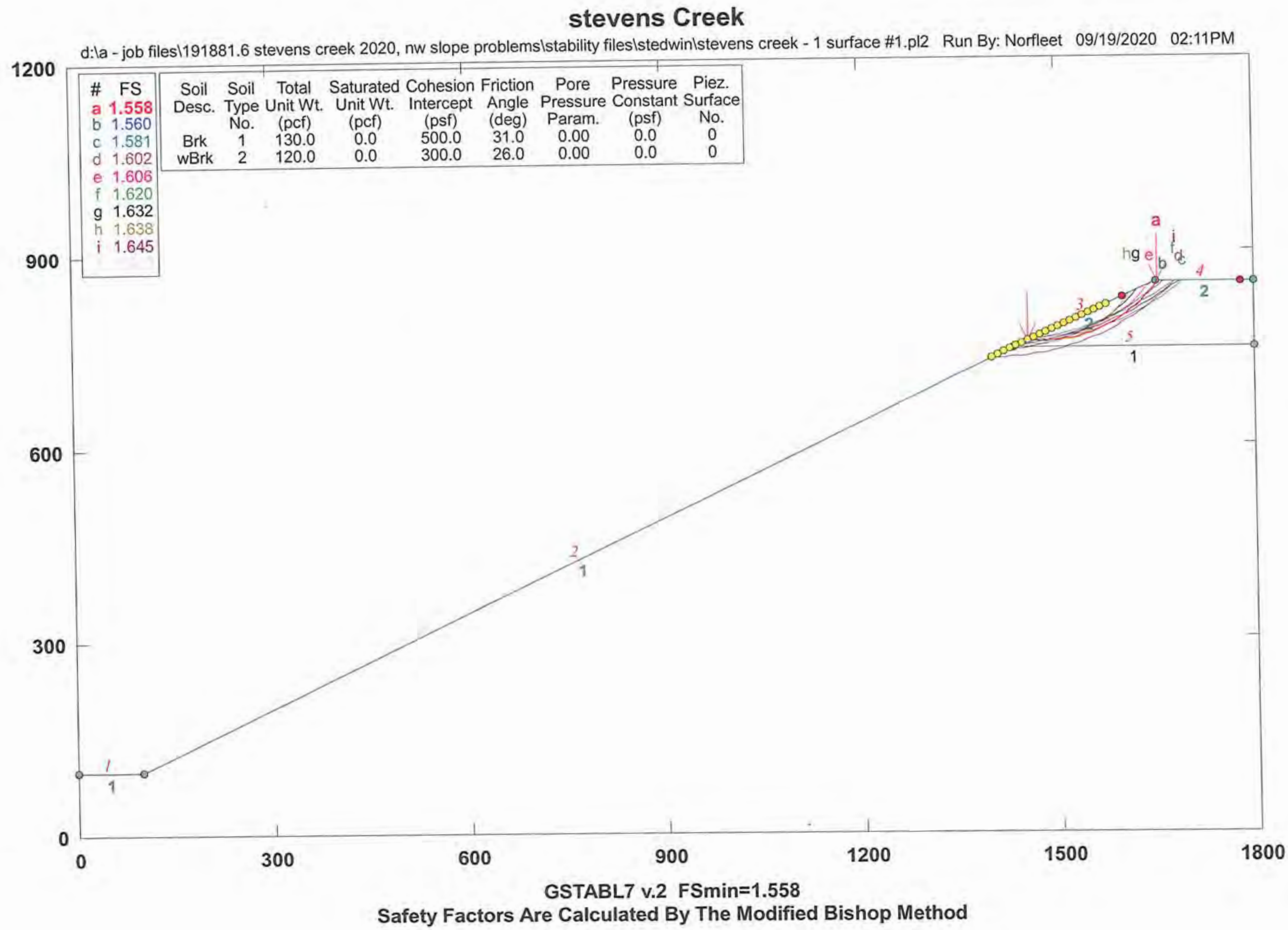
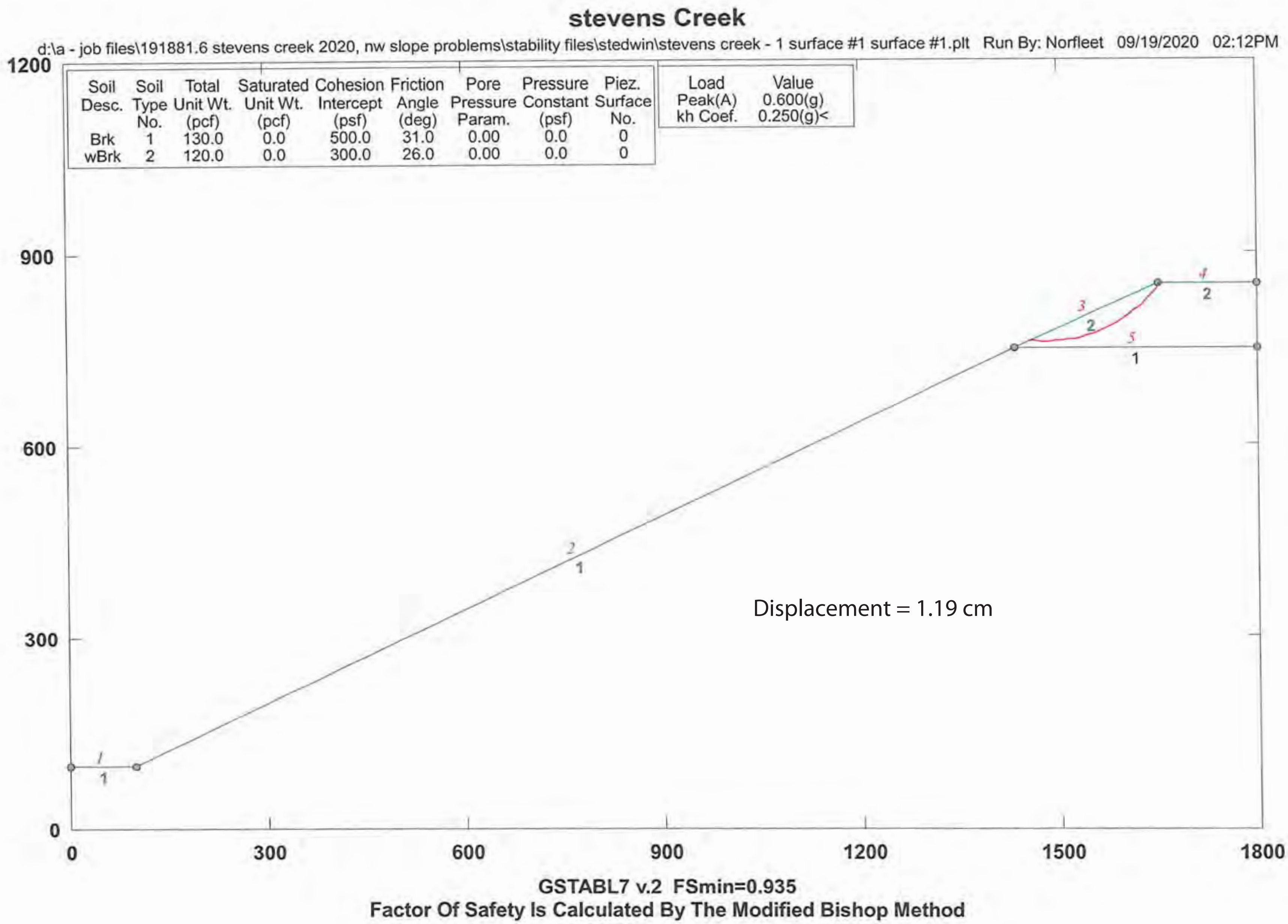
Stevens Creek Quarry

Seismic Analysis of Full Slope

PROJ NO: 191881.6 DATE: 9-17-2020 FIGURE: 2




| | | |
|---|---|---|
| **Norfleet Consultants** | Stevens Creek Quarry | |
| | Stability Analysis of Weathered Layer | |
| PROJ NO: 191881.6 | DATE: 9-17-2020 | FIGURE: 3 |




| Soil Desc. | Soil Type No. | Total Unit Wt. (pcf) | Saturated Unit Wt. (pcf) | Cohesion Intercept (psf) | Friction Angle (deg) | Pore Pressure Param. | Pressure Constant (psf) | Piez. Surface No. |
|---|---|---|---|---|---|---|---|---|
| Brk | 1 | 130.0 | 0.0 | 500.0 | 31.0 | 0.00 | 0.0 | 0 |
| wBrk | 2 | 120.0 | 0.0 | 300.0 | 26.0 | 0.00 | 0.0 | 0 |

| Load | Value |
|---|---|
| Peak(A) | 0.600(g) |
| kh Coef. | 0.250(g)< |

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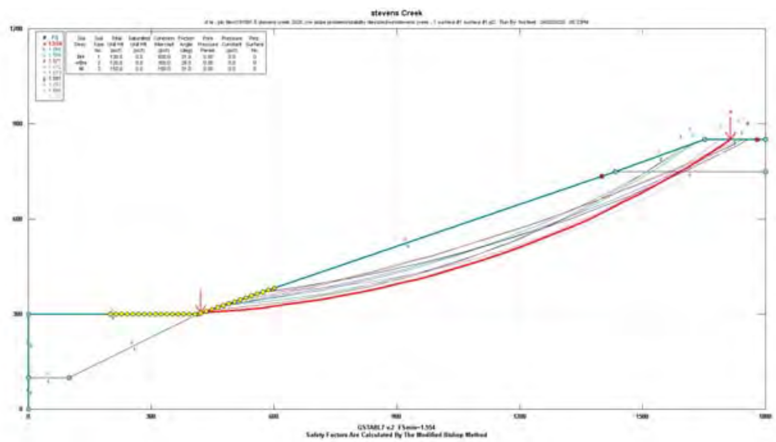
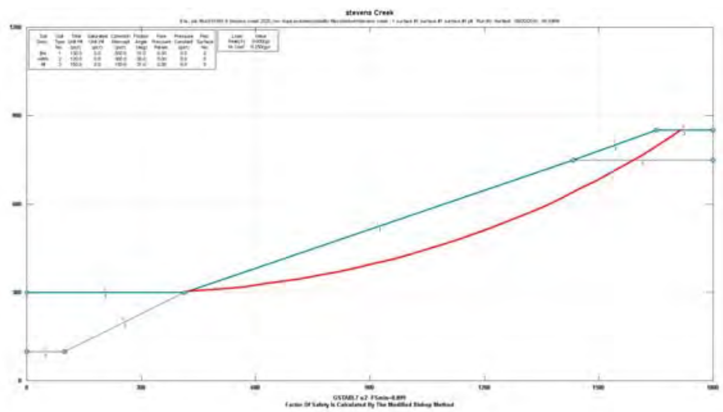
Stevens Creek Quarry

Seismic Analysis of Weathered Layer

| PROJ NO: 191881.6 | DATE: 9-17-2020 | FIGURE: 4 |
|---|---|---|



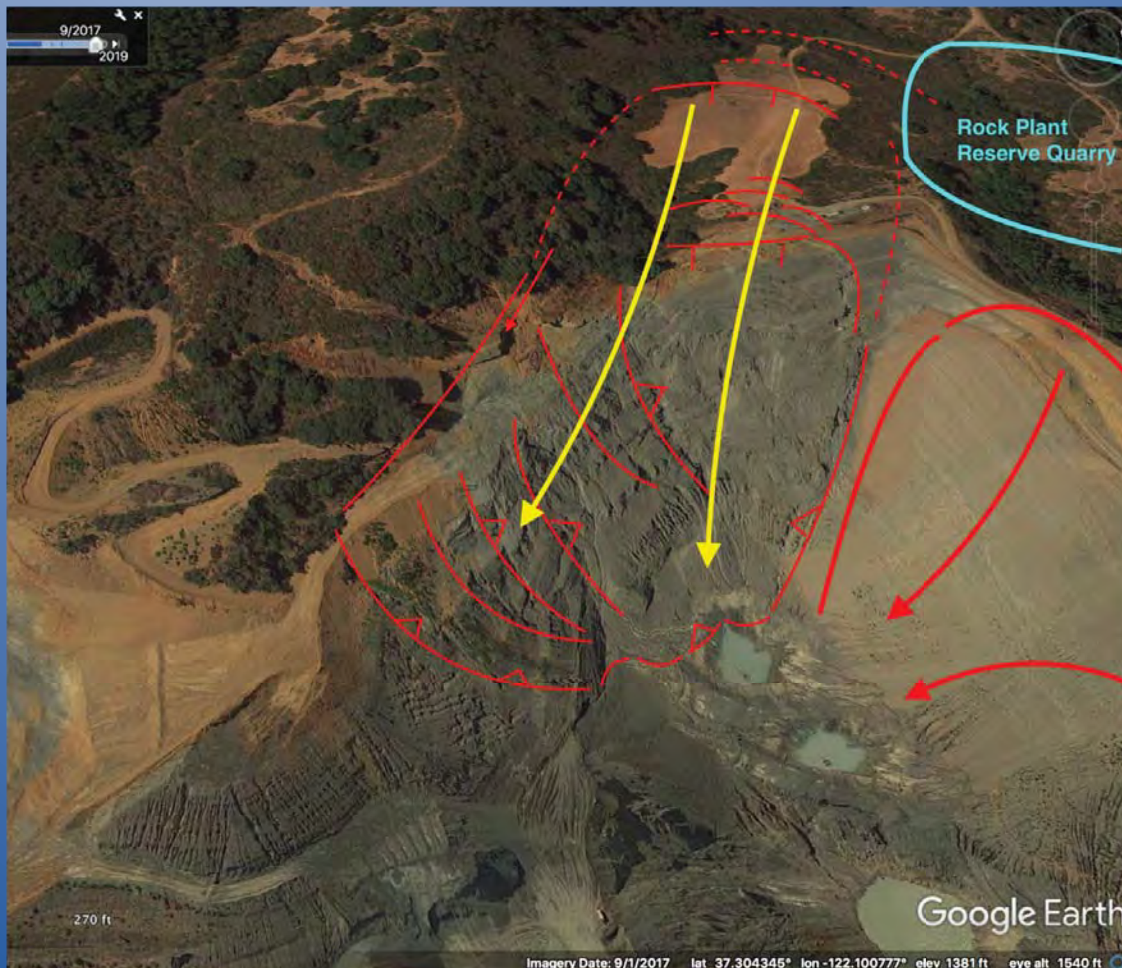

| | | | | | |
|---|---|---|---|---|---|
| **Norfleet Consultants** | Stevens Creek Quarry | | | | |
| | Stability Analysis of Full Slope with Fill Pad | | | | |
| | PROJ NO: 191881.6 | DATE: 9-17-2020 | | FIGURE: 5 | |




Displacement = 2.2 cm

| | | |
|---|---|---|
| **Norfleet Consultants** | Stevens Creek Quarry | |
| | Seismic Analysis of Full Slope with Fill Pad | |
| PROJ NO: 191881.6 | DATE: 9-17-2020 | FIGURE: 6 |

# Stevens Creek NW Wall Landslide




- Activated in 2013
- 1,300 feet long
- 550 feet wide
- 250 to 350 feet deep
- Up to 10 mil cu. Yrds
- Approx 500 to 700 feet into Lehigh Property

2017

COTTON, SHIRES & ASSOCIATES, INC.
CONSULTING ENGINEERS AND GEOLOGISTS

| Norfleet Consultants | Stevens Creek Quarry | | | | |
|---|---|---|---|---|---|
| | Cotton and Shire, Figure 37 | | | | |
| | PROJ NO: 191881.6 | DATE: | 9-17-2020 | FIGURE: | 7 |

# Massive Bedrock Landslide

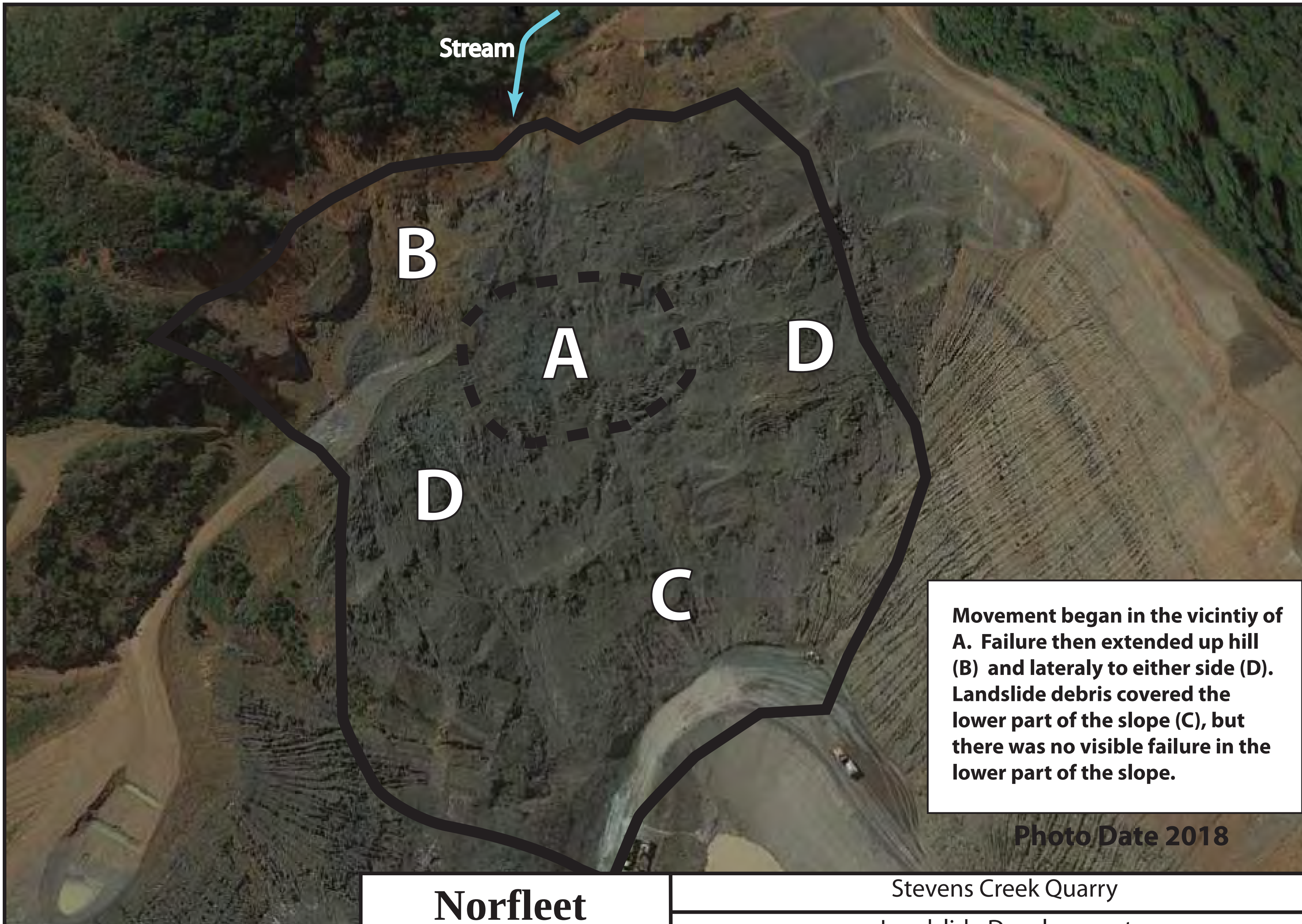

The white arrows indicate features that Cotton and Shires associate with the headscarp of a large, deep-seated landslide.

- **Ground Cracks 740 feet from property line**
- **Not just a cut slope failure**

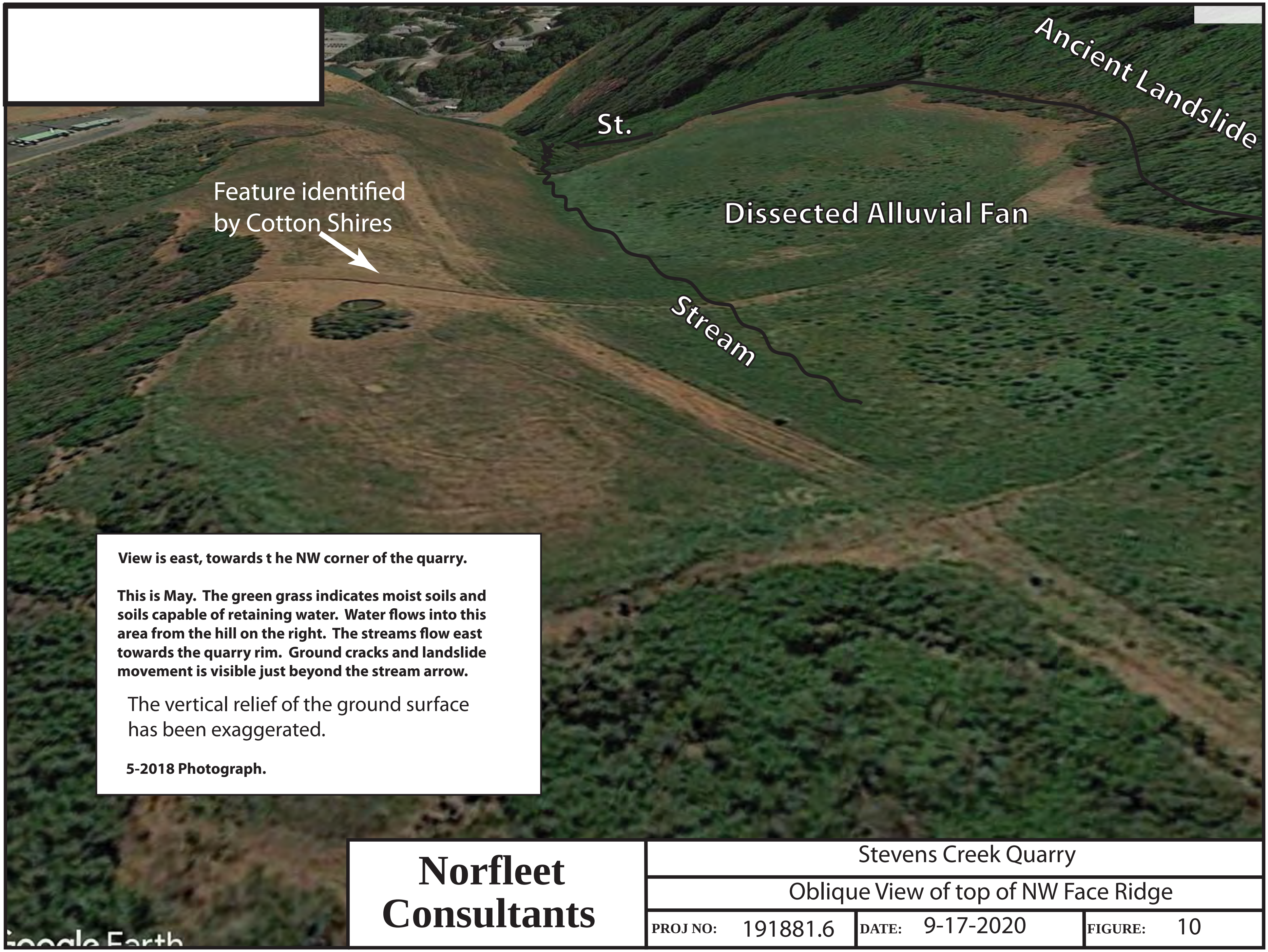
**March 2015**

COTTON, SHIRES & ASSOCIATES, INC.
CONSULTING ENGINEERS AND GEOLOGISTS

| | | | | | |
|---|---|---|---|---|---|
| **Norfleet Consultants** | Stevens Creek Quarry | | | | |
| | Cotton and Shire, Figure 37 | | | | |
| | PROJ NO: 191881.6 | | DATE: 9-17-2020 | | FIGURE: 8 |




| | | | | | |
|---|---|---|---|---|---|
| **Norfleet Consultants** | Stevens Creek Quarry | | | | |
| | Landslide Development | | | | |
| | PROJ NO: | 191881.6 | DATE: 9-17-2020 | FIGURE: | 9 |



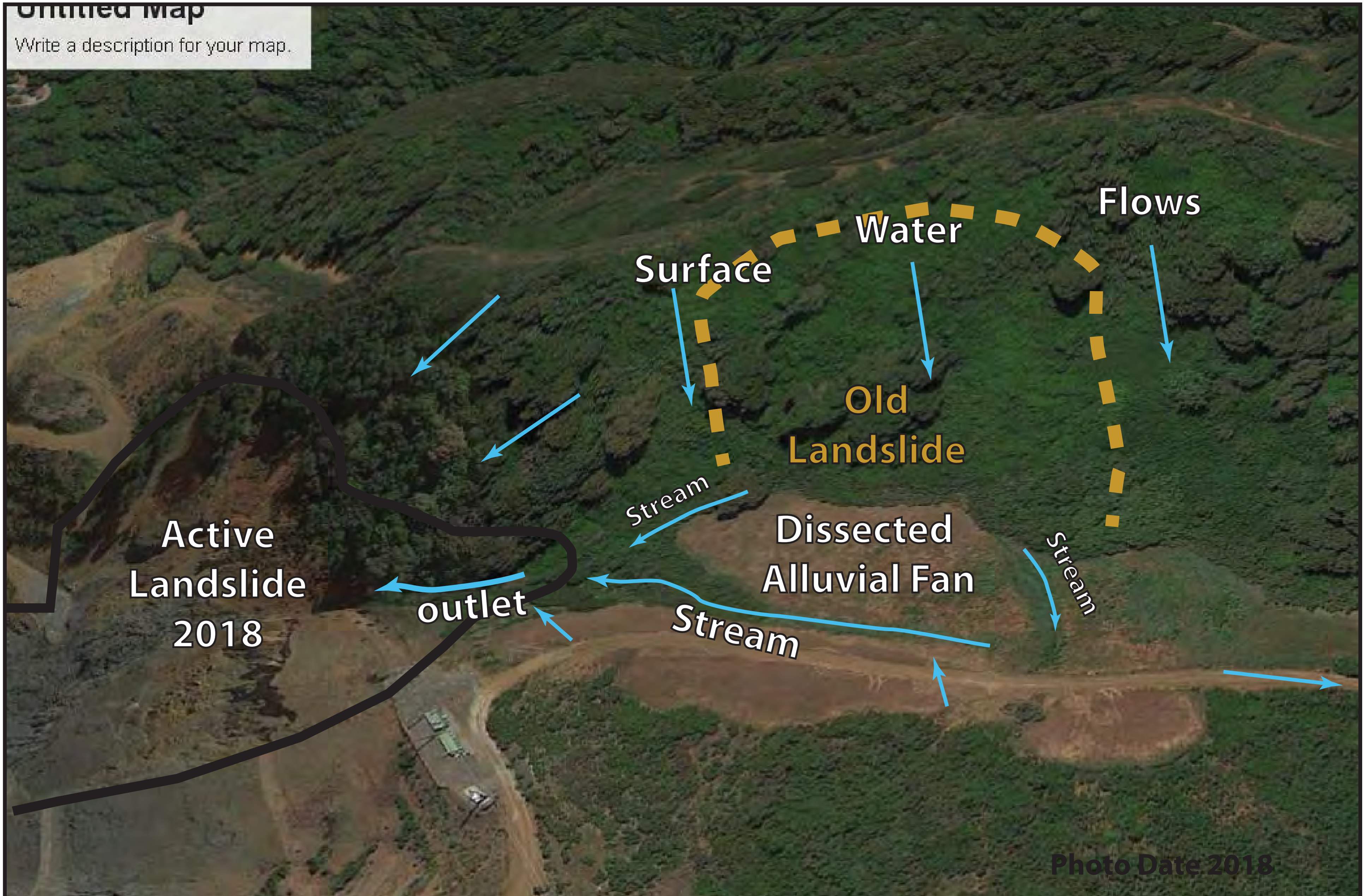

**View is east, towards t he NW corner of the quarry.**

**This is May. The green grass indicates moist soils and soils capable of retaining water. Water flows into this area from the hill on the right. The streams flow east towards the quarry rim. Ground cracks and landslide movement is visible just beyond the stream arrow.**

The vertical relief of the ground surface has been exaggerated.

**5-2018 Photograph.**

| | | | |
|---|---|---|---|
| **Norfleet Consultants** | Stevens Creek Quarry | | |
| | Oblique View of top of NW Face Ridge | | |
| | PROJ NO: 191881.6 | DATE: 9-17-2020 | FIGURE: 10 |




| Norfleet Consultants | Stevens Creek Quarry | | | | | |
|---|---|---|---|---|---|---|
| | Relationship between Active Landslide and Stream Flow | | | | | |
| | PROJ NO: | 191881.6 | DATE: | 9-17-2020 | FIGURE: | 11 |