Geotechnical Evaluation

Mills Canyon Sewer Access Road Repairs
City Project No. 85090
Burlingame, California

City of Burlingame

501 Primrose Road | Burlingame, California 94010

February 19, 2018 | Project No. 403150001





Geotechnical | Environmental | Construction Inspection & Testing | Forensic Engineering & Expert Witness

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Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

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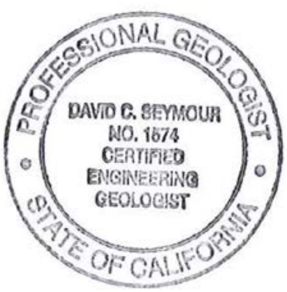

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1 INTRODUCTION

In accordance with your request and authorization, we have performed a geotechnical evaluation for repair of three locations along the Mills Canyon Sewer Access Road in the City of Burlingame, California (Figure 1). The purpose of our study was to evaluate the subsurface conditions at the three specified locations and provide geotechnical recommendations for the design of remedial options to repair the access road. This report presents our findings, conclusions, and recommendations for selected slope repair options.

2 SCOPE OF SERVICES

Our scope of services included the following:

- Project coordination and review of readily available geologic maps, published literature, aerial photographs, geotechnical reports, and previous design plans provided by the City of Burlingame.

- Site reconnaissance to evaluate and map the surficial geologic conditions at each site and to locate the proposed borings for coordination with Underground Services Alert for underground utility location.

- Surveying of the three locations by Alexander & Associates, a California licensed surveyor. They also identified the City's easement for the access road and sewer line. The easement limits are depicted on the accompanying site plans in this report.

- Subsurface exploration consisting of the excavation, sampling, and logging of six (6) small-diameter borings to depths of up to approximately 17 feet below the ground surface. The borings were drilled using a limited-access drill rig utilizing solid flight augers to evaluate the subsurface conditions and to collect samples for laboratory testing. The materials encountered in the borings were classified and logged in accordance with the Unified Soil Classification System (USCS). Relatively undisturbed and bulk samples were obtained at selected intervals from the borings. The soil samples were transported to our laboratory for testing.

- Laboratory testing of representative soil samples. Laboratory tests included evaluation of in-situ moisture and density, gradation analysis, Atterberg limits, direct shear strength, and unconfined compression.

- Data compilation and engineering analysis of the information obtained from our background review, subsurface evaluation, and laboratory testing.

- Conducted slope stability analysis to evaluate the stability of the existing site conditions and selected repair options.

- Preparation of this geotechnical report presenting our findings, conclusions, and geotechnical recommendations for the project.

3 SITE DESCRIPTION AND BACKGROUND

The Mills Canyon sewer access road is located above the southern bank of Mills Creek in the eastern portion of the City of Burlingame (Figure 1). The road is about 0.4 miles long and consists of an asphalt paved road surface that is typically 10 feet wide. The sewer line and access road were constructed in 1980. The sewer line consists of an 8-inch diameter pipe that is located beneath the inboard side of the road. Since its construction, the roadway has experienced several slope failures that required the design and construction of retaining structures along the outboard side of the road. A previous geotechnical study was conducted by Kleinfelder (1999), which was used to prepare design plans by Veizades & Associates, Inc. (1999) and Terrain, Inc. (2006).

During the winter of 2016/2017, three areas along the roadway experienced soil movement that damaged the roadway and jeopardized the stability of the sewer line. The locations of the three areas are shown on Figure 2 and are designated as Areas 1, 2 and 3, with Area 1 being the location closest to the main access point off of La Mesa Court. Summaries of the conditions observed during our evaluation for each area are provided in the following sections.

3.1 Area 1

Area 1 includes an existing retaining structure that was constructed in 2006 (Terrain, Inc., 2006). The wall is located at the first bend in the access road, where the road descends towards Mill Creek (Figures 2 and 4). The wall is about 40 feet long and consists of steel beams placed about 4 feet on center with timber lagging. The timber lagging is 2 feet in height. Based on a review of design plans, the steel beams are 12-feet long and embedded 10 feet below the ground surface.

The sewer line is located at the toe of a natural slope that descends from the wall toward the creek (Figure 5). The slope is about 30 feet in height and inclined at about 26 degrees from horizontal (slope ratio of 2:1 – horizontal to vertical). Features suggestive of recent slope movement were not observed on the slope; however, the surface of the slope is hummocky, suggesting that older landslide events may have occurred on the slope.

Surface flow from the ascending portion of the access road flows directly toward the wall. Two distinct settlement cracks with vertical displacements of up to 6 inches run parallel to the top of the wall through the asphalt pavement. A separation of about 14 inches was observed between the bottom of the timber lagging and the soil ground surface.

3.2 Area 2

Area 2 is located about ¼ mile along the roadway, adjacent to Sewer Manhole No. 13 (Figures 2 and 6). The area of damaged road extends about 60 linear feet parallel to the roadway and includes several ground cracks through the asphalt with vertical displacements of up to 6 inches. Horizontal separations across the cracks are up to 8 inches wide. One of the prominent cracks overlies the sewer line and extends approximately 30 feet. The cracks were observed in the asphalt section, but were not observed in the adjoining slope that descends to Mill Creek (Figure 7). The descending natural slope is about 30 feet high with a slope ratio of about 1.5:1 (horizontal to vertical). The slope surface was generally comprised of vegetation and surficial soil. We did not observe indications of significant soil movement on the slope.

3.3 Area 3

Area 3 is located at the eastern end of the roadway, about 0.4 miles from the La Mesa Court entrance (Figures 2 and 8). The area of damaged road at Area 3 extends about 40 linear feet parallel to the roadway and includes a few main ground cracks through the asphalt with vertical displacements of up to 8 inches. Horizontal separations across the main crack are up to 9 inches. The cracks were observed in the asphalt section but were not observed in the adjoining slope that descends to a tributary to Mill Creek (Figure 9). The descending natural slope is about 45 feet high with a slope ratio of about 1.5:1 (horizontal to vertical). The slope surface was generally comprised of vegetation and surficial soil. We did not observe indications of significant soil movement on the slope.

4 PROJECT DESCRIPTION

This phase of the project includes geotechnical evaluations of the three locations and engineering analysis to determine the stability of the sites and recommendations for stabilization. Preliminary design recommendations are provided along with approximate quantities and preliminary engineer’s cost estimates.

5 SUBSURFACE EVALUATION AND LABORATORY TESTING

Our subsurface exploration at the site was performed on November 29, 2017 and December 1, 2017 and consisted of the drilling, logging, and sampling of six (6) small-diameter borings to depths of up to approximately 17 feet below the surface. Two borings were drilled at each location using a limited-access drill rig utilizing 3½-inch diameter solid-flight augers. The borings were excavated to evaluate the subsurface conditions and to collect samples for laboratory testing, and were logged by a representative from our firm. Bulk and relatively undisturbed soil samples were obtained at selected depths for laboratory testing. The approximate locations of the borings are presented on Figure 2. The logs of the exploratory borings are presented in Appendix A.

Laboratory testing of representative soil samples included tests to evaluate in-situ moisture and density, sieve analysis, Atterberg limits, direct shear strength, and unconfined compressive strength. The results of our in-situ moisture content and dry density evaluation are presented on the boring logs in Appendix A. The remaining laboratory testing results are presented in Appendix B.

6 GEOLOGIC AND SUBSURFACE CONDITIONS

6.1 Regional Geologic Setting

According to published regional geologic maps (Brabb et al., 1998, and Pampeyan, 1994), the surrounding area is underlain by bedrock of the Jurassic/Cretaceous age Franciscan Complex (Figure 3). Along Mills Canyon the bedrock is overlain by colluvial soil deposits. The Franciscan rocks in Mills Canyon typically consist of greenstone, meta-basalt, serpentinite, blueschist, chert, sandstone, and shale. These rocks vary in hardness and vary from highly weathered to slightly weathered near the surface. Overlying the bedrock is a layer of colluvial soil that has accumulated over time on top of the bedrock. The contact between the colluvium and bedrock commonly forms a layer of weakness where shallow landslides often occur. Groundwater often migrates along this contact, especially during periods of heavy rain, and reduces the resisting forces along the contact, initiating shallow slope failures.

6.2 Site Geology

The results of our geologic mapping and subsurface exploration indicate that the site is generally underlain by artificial fill, colluvial soil, landslide debris, and bedrock. Our interpretations of the surface and subsurface conditions at each site are shown on Figures 4 through 9. Generalized descriptions of the materials encountered at each location during our subsurface exploration are presented below. More detailed descriptions of the materials encountered in our exploratory excavations are shown on the boring logs in Appendix A.

6.2.1 Area 1 Geology

Geologic units encountered at Area 1 include artificial fill, colluvial soil, landslide debris, and bedrock. The roadway pavement section encountered in Borings B-1 and B-2 consisted of about 5 inches of asphalt concrete (AC) over 6 to 12 inches of aggregate base (AB). Artificial fill, used as backfill behind the retaining wall, was encountered in Borings B-1 and B-2 and consisted of drain rock (gravel) and medium dense clayey gravel with sand.

Colluvial soil was encountered beneath the fill to depths of 6 to 8 feet below the roadway. The colluvial soil consisted of brown, moist, loose, clayey sand and clayey gravel. Bedrock was encountered below the colluvial soil to the depths explored of 16½ and 17 feet. The bedrock consisted of metabasalt and metashale. Both rock types are weathered and vary from weak to strong at depth. Landslide debris was not encountered in the borings; however, the descending slope below the retaining wall may be mantled by landslide debris.

6.2.2 Area 2 Geology

Geologic units encountered at Area 2 include artificial fill, colluvial soil, and bedrock. The roadway pavement section encountered in Borings B-3 and B-4 consisted of 3½ to 4 inches of AC over 3 to 5 inches of AB. The artificial fill encountered in Borings B-3 and B-4 consisted of dark brown, moist, very stiff lean clay and loose, clayey sand. The fill was placed during construction of the access road. Colluvial soil was encountered beneath the fill to depths of 8 to 10 feet below the roadway. The colluvial soil consisted of dark brown, moist, very stiff lean clay and loose, clayey sand. Bedrock was encountered below the colluvial soil to the depths explored of 16 and 17 feet. The bedrock consisted of weathered serpentinite, which varies from weak to strong at depth.

6.2.3 Area 3 Geology

Geologic units encountered at Area 3 include artificial fill, colluvial soil, and bedrock. The roadway pavement section encountered in Boring B-5 consisted of 3 inches of AC over 3½ inches of AB. The artificial fill encountered in Borings B-5 and B-6 consisted of dark brown, moist, very stiff lean clay and was placed during construction of the access road. Colluvial soil was encountered beneath the fill to depths of 8 to 11 feet below the roadway. The colluvial soil consisted of dark brown, moist, stiff to very stiff lean clay with variable amounts of sand and gravel. Bedrock was encountered below the colluvial soil to the depths explored of 14 and 17 feet. The bedrock consisted of weathered sandstone, which varies from weak to strong at depth.

6.3 Groundwater

Groundwater was not encountered in our exploratory excavations at the site. Fluctuations in the level of groundwater may occur due to variations in ground surface topography, subsurface stratification, rainfall, irrigation practices, groundwater pumping, and other factors which may not have been evident at the time of our field evaluation.

6.4 Landslides

According to a regional study of debris flow potential in San Mateo County by the United States Geological Survey (Mark, 1992), the slopes above and below the access road have a moderate to high probability of generating debris flows during high intensity rainstorms. Our review of historical USGS topographic maps of the Montara Mountain 7.5 minute quadrangle and historical aerial photographs indicates that the slopes along the access road have experienced landslides in the past, and will most likely experience landslides in the future.

Several landslides have occurred along the Mills Canyon sewer access road since the road's construction in 1980. A previous study by Kleinfelder (1999) evaluated several landslides that occurred during the 1997-1998 El Nino rainstorms. Subsequent repair plans were prepared by Veizades & Associates, Inc. in 1999 and by Terrain, Inc. in 2006 to repair landslide failures and distressed pavement areas identified in the Kleinfelder (1999) report. The distress observed at the three locations in this study show signs of soil movement; however, evidence of recent sliding on the outboard descending slopes was not observed at the three locations.

7 FAULTING AND SEISMICITY

The subject site is not located within a State of California Earthquake Fault Zone (formerly known as Alquist-Priolo Special Studies Zone) (California Geological Survey, 2007 and 2018). However, the site is located in a seismically active area, as is the majority of California, and the potential for strong ground motion in the project area is considered significant during the design life of the proposed improvements. Based on our review, the active San Andreas Fault Zone is located approximately 0.6 miles southwest of the site.

7.1 Ground Motion

The 2016 California Building Code (CBC) specifies that the Risk-Targeted, Maximum Considered Earthquake (MCER) ground motion response accelerations be used to evaluate seismic loads for design of buildings and other structures. The MCER ground motion response accelerations are based on the spectral response accelerations for 5 percent damping in the direction of maximum horizontal response and incorporate a target risk for structural collapse equivalent to 1 percent in 50 years with deterministic limits for near-source effects. The horizontal peak ground acceleration (PGA) that corresponds to the MCER for the site was calculated as 0.991g using the USGS (USGS, 2018) seismic design tool (web-based).

8 SLOPE STABILITY

Our evaluation focused on providing repair options for the roadway at three specific locations. Slope stability analyses were conducted for the existing site conditions and selected remedial options. The results of our slope stability analyses are presented in Appendix C.

Slope stability analyses were performed in order to evaluate the stability of the slopes in the vicinity of the access road in Area 1, Area 2, and Area 3. Our analyses were limited to areas of the slope near the outboard side of the access road, generally corresponding to areas where the road was damaged by soil movement, and did not consider potential surfaces upslope of the access road. Geologic cross-sections (Figures 5, 7, and 9) were prepared to represent the existing site and slope conditions. The locations of the cross-section lines are shown on Figures 4, 6, and 8. We developed our model for the slope stability analysis based on the information obtained from our geologic reconnaissance and subsurface data associated with our exploratory borings and review of the previous geotechnical evaluation (Kleinfelder, 1999). Shear strength parameters used in our analysis were based on the results of laboratory tests, including grain size, Atterberg limits, and direct shear tests and published correlations between index properties and shear strength. The parameters used in our analysis are summarized in Table 1.

Table 1 – Slope Stability Shear Strength Parameters

Area/Cross Section	Geologic Unit	Moist Unit Weight (pcf)	Friction Angle (degrees)	Cohesion (psf)
1/A	Artificial Fill	115	27	0
	Colluvium	115	27	0
	Bedrock	125	26	200
2/B	Artificial Fill	115	27	0
	Colluvium	115	27	0
	Bedrock	125	26	450
3/C	Artificial Fill	115	25	0
	Colluvium	115	25	0
	Bedrock	120	30	0

Our slope stability analyses were performed based on the Morgenstern and Price limit equilibrium method using a two-dimensional stability analysis program, Slope/W (Geo-Slope International Ltd., 2012) for static and pseudo-static conditions. Failure surfaces were generated using the “entry and exit” search algorithms. Iterations using these subroutines yield what we consider to be critical (lowest factor of safety) potential failure surfaces that could impact the access road. Potential shallow failure surfaces, less than about 2 feet in thickness, were not considered in the analyses. As suggested in Special Publication (SP) 117A (California Geological Survey, 2008), slopes with a factor of safety of 1.5, or more, for the static condition are considered adequately stable for static conditions. Pseudo-static slope analyses were conducted to evaluate the yield acceleration of the slope, with the retaining wall included, for use in simplified seismic displacement analyses. The yield acceleration is defined as the horizontal seismic coefficient that results in a factor of safety of about 1.0.

The results of our slope stability evaluation indicate that the existing slope and retaining wall in cross-section A-A' has an adequate (more than 1.5) factor of safety against instability under static and pseudo-static loading conditions. However, the results for our slope stability evaluation in cross-sections B-B' and C-C' indicate that the existing slopes do not have factors of safety of 1.5 or more under static condition for instability. Based on these results, mitigation measures consisting of a retaining wall element were considered for Sections B-B' and C-C'. The pile elements representing a retaining wall were included as vertical reinforcement strips, 20-feet deep with 6-foot on-center spacing. The shear force of the pile was increased until a factor of safety of 1.5, or more, was obtained. Pseudo-static slope analyses were then conducted to evaluate the yield acceleration of the slope, with the retaining wall shear force from the static analyses included. The results of our analyses are summarized below in Table 2.

Table 2 – Slope Stability Analysis Results

Area/Cross Section	Existing Factor of Safety (Static)	Factor of Safety With New Wall (Static)	Reinforcement Shear Force (kips)	Yield Coefficient, Ky
1/A	>3.0	N/A	N/A	N/A
2/B	0.95	1.54	12	0.16
3/C	0.79	1.53	23	0.12

Based on the results of our pseudo-static analysis, the estimated seismic displacement would be about 17 inches for Area 2 and 22 inches for Area 3, assuming a 50 percent probability of exceedance using the Bray and Travasarou simplified displacement method (Bray and Travasaraou, 2007). We consider the estimated displacement to be tolerable based on no structures being near the slope. The required reinforcement shear force for design can be increased, if needed, to reduce the estimated potential seismic slope displacement. If the material upslope of the retaining wall is removed and replaced with new fill, we anticipate the slope displacement will occur primarily downslope of the new wall.

9 CONCLUSIONS

Based on our review of the referenced background data, our site field reconnaissance, subsurface evaluation, laboratory testing, and engineering analyses, it is our opinion that repair of the roadway at the three locations and proposed construction is feasible from a geotechnical standpoint. Based on the results of our evaluation, the following conclusions were developed.

- Subgrade soils underlying the three evaluated areas have settled up to 12 inches and show signs of lateral movement toward the adjoining descending slopes.
- Evidence of recent soil movement was not observed on the descending slopes at the three evaluated areas; however, these slope surfaces consisted of relatively loose soil and vegetation which can obscure minor amounts of soil movement. These slopes should be considered susceptible to failure, especially during periods of heavy rainfall.
- At Area 1, based on the results of our site observations and slope stability evaluation, the soil movement and cracks in the road are caused by inadequate lagging embedment, in combination with poor control of surface drainage, which flows directly at the wall from the ascending roadway. A separation of about 14 inches was observed between the bottom of the timber lagging and the soil ground surface. Water flow has resulted in soil migration from beneath the lagging and progressive loss of material causing removal of support beneath the roadway. Other contributing factors could include inadequate embedment depth of the soldier piles for the existing retaining wall. Based on the need to significantly extend the lagging depth, a new retaining wall with longer soldier piles is recommended for this area.
- At Area 2, soil movement beneath the access road has caused an area of damaged road that is about 60 linear feet parallel to the roadway and includes several ground cracks through the asphalt with vertical displacements of up to 6 inches and horizontal separations across the cracks up to 8 inches wide. Based on the results of our site observations and slope stability evaluation, we recommend construction of a new soldier pile and wood lagging retaining wall along the outboard edge of the roadway.
- At Area 3, soil movement beneath the access road has caused an area of damaged road that is about 40 linear feet parallel to the roadway and includes ground cracks through the asphalt with vertical displacements of up to about 8 inches and horizontal separations up to about 9 inches. Based on the results of our site observations and slope stability evaluation, we recommend construction of a new soldier pile and wood lagging retaining wall, or other slope improvement system, along the outboard edge of the roadway.
- Excavations during construction should be generally feasible with earthmoving equipment in good working order. Due to the variability of weathering in the bedrock materials and potential for hard zones, hard materials should be anticipated that may result in oversize material and difficult excavation during construction of the recommended retaining wall foundations. Special excavating equipment, such as, but not limited to, rippers, pneumatic chippers, rock hammers, or rock coring equipment may be needed to excavate the foundations to their design depth.
- We anticipate that existing fill soils, colluvium and processed bedrock materials at the site should be generally suitable for use as compacted fill, placed in accordance with our recommendations.

- Although groundwater was not encountered during our subsurface exploration, the depth to groundwater varies due to seasonal precipitation, subsurface conditions, irrigation, groundwater pumping, and other factors. Seepage and fluctuations in the groundwater levels at the site should be anticipated.
- The site is not located within a State of California Earthquake Fault Zone (formerly Alquist-Priolo Special Studies Zone). Based on our review of published geologic maps and aerial photographs, no known active or potentially active faults underlie the site. The potential for surface fault rupture at the site is considered to be low.
- The design PGA was estimated to be 0.991g based on the USGS (2018) ground motion calculator (web-based).

10 RECOMMENDATIONS

The following sections present our geotechnical recommendations for the design and construction of the proposed improvements. The project improvements should be designed and constructed in accordance with these recommendations, applicable codes, and appropriate construction practices.

10.1 Earthwork

Earthwork at the site is anticipated to generally consist of cuts and fills related to foundation construction for the proposed retaining wall and preparation of ground areas to receive fill soils related to the wall backfill and roadway construction. Earthwork operations should be performed in accordance with the requirements of applicable governing agencies and the recommendations presented in the following sections of this report.

10.1.1 Pre-Construction Conference

We recommend that a pre-construction conference be held. The owner and/or their representative, the governing agencies' representatives, the civil engineer, Ninyo & Moore, and the contractor should be in attendance to discuss the work plan and project schedule and earthwork requirements.

10.1.2 Site Preparation

Site preparation should begin with the removal of vegetation, utility lines, debris and other deleterious materials from areas to be graded. Tree stumps and roots should be removed to such a depth that organic material is generally not present. Clearing and grubbing should extend to the outside of the proposed excavation and fill areas. Rubble and excavated materials that do not meet criteria for use as fill should be disposed of in an appropriate landfill. Existing utilities to be abandoned should be removed, crushed in place, or backfilled with grout.

Excavations resulting from removal of buried utilities, tree stumps, or obstructions should be backfilled with compacted fill in accordance with the recommendations in the following sections.

10.1.3 Subgrade Observations

Prior to placement of fill, erection of forms or placement of reinforcement for foundations, the client should request an evaluation of the exposed subgrade by Ninyo & Moore. Materials that are considered unsuitable shall be excavated under the observation of the geotechnical engineer in accordance with the recommendations in this section, or the field recommendations of the geotechnical engineer.

Unsuitable materials include, but may not be limited to dry, loose, soft, wet, expansive, organic, or compressible natural soil; and undocumented or otherwise deleterious fill materials. Unsuitable materials should be removed from trench bottoms and below bearing surfaces to a depth at which suitable foundation subgrade, as evaluated in the field by the geotechnical engineer, is exposed.

10.1.4 Excavation Characteristics

Based on our exploratory borings and review of geologic background materials, we anticipate that excavation within the fill and bedrock materials present on site may generally be accomplished with grading equipment in good operating condition. We anticipate that the bedrock materials will generally disaggregate and/or break down with processing to be reused as fill. However, based on our experience, the degrees of weathering, decomposition, and hardness of the bedrock may vary widely with relatively abrupt changes on a site. Bedrock with lesser degrees of weathering may involve special excavating equipment, such as rippers, pneumatic chippers, or jackhammers. Excavations in hard rock zones may generate oversize rock fragments that are not generally suitable for fill material.

10.1.5 Fill Material

In general, the on-site fill soils and excavated bedrock materials should be suitable for use as general fill, including retaining wall backfill and roadway subgrade fill, provided they are free of trash, debris, roots, vegetation, boulders, or other deleterious materials. Fill should generally be free of rocks or lumps of material in excess of 4 inches in diameter. Rocks or hard lumps larger than approximately 4 inches in diameter should be broken into smaller pieces or should be removed from the site. On-site soils used for fill may involve moisture conditioning to achieve appropriate moisture content for compaction.

Import fill should consist of clean, granular soils with an expansion index (EI) of 50 or less as evaluated by ASTM D 4829. Soil should also be tested for corrosive properties prior to importing. We recommend that imported materials satisfy the Caltrans (2012) criteria for non-corrosive soils (i.e., soils having a chloride concentration of 500 parts per million [ppm] or less, a soluble sulfate content of approximately 0.20 percent [2,000 ppm] or less, a pH value of 5.5 or higher and a minimum resistivity of 1,000 ohm-cm or higher). Materials for use as fill should be evaluated by Ninyo & Moore prior to importing. The contractor should be responsible for the uniformity of import material brought to the site.

10.1.6 Fill Placement and Compaction

Fill and backfill should be compacted in horizontal lifts. The allowable uncompacted thickness of each lift of fill depends on the type of compaction equipment utilized, but generally should not exceed 8 inches in loose thickness. Fill should be compacted in horizontal lifts to a relative compaction of 90 percent or more, as evaluated by ASTM D 1557. Aggregate base beneath the access road should be compacted to a relative compaction of 95 percent, or more, as evaluated by ASTM D 1557. Fill soils should be placed at or above the optimum moisture content as evaluated by ASTM D 1557. Placement and compaction of the fill soils should be in general accordance with local grading ordinances and good construction practice.

Compacted fill should be maintained in a moist (but not saturated) condition by the periodic sprinkling of water prior to placement of additional overlying fill. Fill that has been permitted to dry out and loosen or develop desiccation cracking should be scarified, moisture conditioned, and recompacted as per the requirements above.

10.2 Retaining Walls

The proposed soldier pile and lagging retaining walls can consist of steel beams embedded in cast-in-drilled-hole (CIDH) caisson foundations with treated wood or concrete lagging. The wall designs should be performed in conjunction with drainage improvements that include providing suitable subsurface drainage behind the wall and conveying surface overland water toward drainage collection devices.

The embedment depth, CIDH diameter, lagging design, and steel reinforcement design should be evaluated by the project design engineer based on the following recommendations.

General design and construction considerations for the soldier pile and lagging walls are presented below.

- The drilled shafts should be 24 inches or more in diameter. The final depth and spacing of the shafts and soldier piles should be evaluated by the project design engineer based on the estimated lateral loads. The shafts should be installed at a center-to-center spacing of about three diameters.
- In calculating the total lateral load acting on a given pile, the spacing between adjacent piles should be considered as the span length. For example, for a pile center-to-center spacing of 6 feet, a wall span length of 6 feet should be considered in design.
- Hydrostatic pressures may be neglected, provided that suitable drainage of the retained soil is provided. The retained soil should be drained by an 18-inch wide backdrain consisting of ¾-inch crushed rock wrapped in filter fabric (Mirafi 140N, or equivalent). The backdrain should be capped by pavement or 12 inches of native soil. The backdrain should be drained by a perforated pipe (Schedule 40 polyvinyl chloride pipe, or similar) at the bottom of the crushed rock. The pipe should be sloped at 1 percent or more to discharge at an appropriate outlet away from the wall. Alternatively, geocomposite drain panels (Miradrain 6000XL, or similar) placed against the back of the wall may be used as a backdrain.
- Drilled shaft excavations and placement of soldier piles should be observed by Ninyo & Moore to check materials encountered, pier diameters, and embedment depths. The drilled holes should be cleaned of loose soil and/or rock and construction debris prior to placing steel and pouring concrete. It is the contractor’s responsibility to take the appropriate measures to provide for the integrity of the drilled holes and to see that the holes are cleaned and straight and that sloughed loose soil/debris is removed from the bottom of the hole prior to the placement of concrete. Piers should be checked for alignment and plumbness during installation. The amount of acceptable misalignment of a pier is approximately 3 inches from the plan location. It is usually acceptable for a pier to be out of plumb by one percent of the depth of the pier.
- Drilled pier excavations may encounter groundwater and cohesionless soils which may be unstable and need to be stabilized by temporary casing or use of drilling mud. Each pier should be drilled to the specified depth, and the pier bottom should be cleaned of loose material prior to pouring concrete. Standing water should be removed from the pier excavation, or the concrete should be delivered to the bottom of the excavation, below the water surface by tremie pipe. Casing should be removed from the excavation as the concrete is placed. Concrete should be placed in the piers in a manner that reduces the potential for segregation of the components.
- The soldier pile wall details should be included in the project plans. The project plans should be signed and stamped by a professional engineer registered in the state of California. Ninyo & Moore should be given the opportunity to review the project plans to check for compliance with design and construction recommendations presented herein.

Design criteria and recommendations for each area are presented below:

10.2.1 Area 1

- Retaining walls retaining level backfill should be designed for an equivalent fluid pressure of 40 pounds per cubic feet (pcf). For pier embedment design, the retained height should be assumed to be 4 feet, or more, in height. For piers located approximately 2 feet downslope of the existing wall, the depths to colluvium and bedrock assumed for design should be 4 feet and 8 feet, respectively, from the top of the wall.
- Retaining walls with retained heights of more than 6 feet and retaining level backfill should be designed for a seismic equivalent fluid pressure of 20 pcf.
- Retaining wall designs should consider vehicle surcharges by including a uniform lateral pressure of 120 pounds per square feet.
- A passive earth pressure increasing at a rate of 275 psf for colluvium and 375 psf for bedrock per foot of depth may be used to evaluate the lateral resistance for the portion of piles embedded in colluvium and bedrock. Lateral resistance should be ignored for the portion of a pile in fill materials, to a depth of 5 feet, and where the lateral distance to the slope face is 5 feet or less.
- The lagging should extend to a depth of 4 feet, or more, below the top of the piers and the bottom lagging should be embedded 6 inches or more below the adjacent ground surface.
- The loose material behind the existing retaining wall should be removed until competent, firm material is encountered. The removed material should be replaced with engineered compacted fill.

10.2.2 Area 2

- Retaining walls retaining level backfill should be designed for an equivalent fluid pressure of 40 pounds per cubic feet (pcf). For pier embedment design, the retained height should be assumed to be 5 feet, or more, in height. For piers located along the outboard edge of the road, the depths to colluvium and bedrock assumed for design should be 5 feet and 9 feet, respectively, from the top of the wall.
- If the loose fill material in the area beneath the damaged portion of the road behind the wall is not removed and replaced with new engineered fill, the structural capacity of the pier should be designed to provide 12 kips of lateral force resistance using a p-y type lateral analysis.
- Retaining walls with retained heights of more than 6 feet and retaining level backfill should be designed for a seismic equivalent fluid pressure of 20 pcf.
- Retaining wall designs should consider vehicle surcharges by including a uniform lateral pressure of 120 pounds per square feet.
- A passive earth pressure increasing at a rate of 275 psf for colluvium and 450 psf for bedrock per foot of depth may be used to evaluate the lateral resistance for the portion of piles embedded in colluvium and bedrock. Lateral resistance should be ignored for the portion of a pile in fill materials, to a depth of 5 feet, and where the lateral distance to the slope face is 5 feet or less.

- The lagging should extend to a depth of 2 feet, or more, below the top of the piers and the bottom lagging should be embedded 6 inches or more below the adjacent ground surface.
- The loose material below the damaged portions of the road should be removed until competent, firm material is encountered. The removed material should be replaced with engineered compacted fill.

10.2.3 Area 3

- Retaining walls retaining level backfill should be designed for an equivalent fluid pressure of 40 pounds per cubic feet (pcf). For pier embedment design, the retained height should be assumed to be 6 feet, or more, in height. For piers located along the outboard edge of the road, the depths to colluvium and bedrock assumed for design should be 6 feet and 9 feet, respectively, from the top of the wall.
- If the loose fill material in the area beneath the damaged portion of the road behind the wall is not removed and replaced with new engineered fill, the structural capacity of the pier should be designed to provide 23 kips of lateral force resistance using a p-y type lateral analysis.
- Retaining walls with retained heights of more than 6 feet and retaining level backfill should be designed for a seismic equivalent fluid pressure of 20 pcf.
- Retaining wall designs should consider vehicle surcharges by including a uniform lateral pressure of 120 pounds per square feet.
- A passive earth pressure increasing at a rate of 275 psf for colluvium and 375 psf for bedrock per foot of depth may be used to evaluate the lateral resistance for the portion of piles embedded in colluvium and bedrock. Lateral resistance should be ignored for the portion of a pile in fill materials, to a depth of 5 feet, and where the lateral distance to the slope face is 5 feet or less.
- The lagging should extend to a depth of 3 feet, or more, below the top of the piers and the bottom lagging should be embedded 6 inches or more below the adjacent ground surface.
- The loose material below the damaged portions of the road should be removed until competent, firm material is encountered. The removed material should be replaced with engineered compacted fill.

10.3 Concrete Placement

Concrete in contact with soil or water that contains high concentrations of soluble sulfates can be subject to chemical and/or physical deterioration. Due to the potential variability in soil conditions across the site, we recommend that Type V cement with a water/cement ratio of 0.45 or less be considered for the project.

10.4 Drainage

Proper surface drainage is imperative for satisfactory site performance. Positive drainage should be provided and maintained to direct surface water away from the top of the descending slopes. Positive drainage is defined as a slope of 2 percent or more over a distance of 5 feet away from tops of slopes. Runoff should then be directed by the use of swales or pipes into a collective drainage system. Surface waters should not be allowed to pond adjacent to the top of walls.

11 DESIGN ALTERNATIVES AND ESTIMATED COSTS

Based on our evaluation we have evaluated design alternatives to stabilize the three subject areas. Design alternatives and estimated costs are provided in the following sections for each location.

11.1 Area 1

The proposed repair includes approximately 40 linear feet of wall along the outboard side of the existing retaining structure. For planning and cost estimating purposes, we considered a wall that would consist of steel beam reinforced concrete piers, with each pier 24 inches in diameter and spaced at approximately 6 feet on-center. Each pier was assumed to be about 20 feet in length with about 4 feet of steel beam exposed and supporting wood lagging.

11.2 Area 2

The proposed repair includes approximately 80 linear feet of wall along the outboard side of the existing access road. For planning and cost estimating purposes, we considered a wall that would consist of steel beam reinforced concrete piers, with each pier 24 inches in diameter and spaced at approximately 6 feet on-center. Each pier was assumed to be about 20 feet in length with about 2 feet of steel beam exposed and supporting wood lagging.

11.3 Area 3

The proposed repair includes approximately 60 linear feet of wall or similar slope reinforcement along the outboard side of the existing access road.

11.3.1 Alternative 1 – steel beam and wood lagging wall

For planning and cost estimating purposes, we considered a wall that would consist of steel beam reinforced concrete piers, with each pier 24 inches in diameter and spaced at approximately 6 feet on-center. Each pier was assumed to be about 20 feet in length with about 3 feet of steel beam exposed and supporting wood lagging.

11.3.2 Alternative 2 – micropile reinforced slope

As an alternative to the steel beam and wood lagging wall, other types of walls were considered. Based on site and access constraints, delivery of concrete to the Area 3 site could be problematic and costly. Alternative slope improvement systems were considered that would minimize quantities of concrete needed. Earth anchors, such as tiebacks, manta-rays, or helical anchors, were not considered feasible based on the location of the existing sewer pipe and potential for damage to the sewer pipe during earth anchor installation. A slope reinforcement system comprised of micropiles and geogrid reinforced fill is considered a potential alternative to mitigate the existing road damage.

A micropile is generally defined as a small-diameter drilled and grouted pile that is reinforced with steel casing and core steel, and is typically installed using a relatively small track-mounted drill rig and a small on-site grout mixing plant. Removing the existing fill beneath the damaged portion of the road and replacing with geogrid reinforced fill would reduce the amount of driving force and therefore reduce the resting force design requirements for the micropiles. Micropiles should be designed in accordance with the Federal Highway Administration (FHWA) Micropile Design and Construction Manual (2005). Geogrid reinforced fill should be placed and compacted in accordance with recommendations provided in Section 10.1.6 of this report and include geogrid, such as Miragrid® 2XT or equivalent, placed at approximately 1 foot vertical spacings.

For planning and cost estimating purposes, we considered the slope improvement would consist of about 30 micropiles, with each micropile assumed to be about 25 feet in length. We assumed the removal of existing fill and replacement with geogrid reinforced fill would be approximately 50 cubic yards.

11.4 Estimated Construction Costs

Cost estimates for construction of the proposed options are provided below in Table 3. It should be noted these are approximate estimates to be used for preliminary planning purposes and are not considered detailed estimates based on a final design. Estimates of costs are based on preliminary design quantities and previous experience with similar types of construction projects. Actual values could vary, potentially substantially, depending on the final design. These estimates do not include engineering design, construction management, or construction observation services.

Table 3 – Engineer's Cost Estimates		
Area	**Description**	**Construction Cost Estimate**
1	40 lineal feet of wall consisting of 20-foot long steel beams in concrete spaced at 6-foot on-centers with wood lagging and a backdrain	$70,000
2	80 lineal feet of wall consisting of 20-foot long steel beams in concrete spaced at 6-foot on-centers with wood lagging and a backdrain	$125,000
3a	60 lineal feet of wall consisting of 20-foot long steel beams in concrete spaced at 6-foot on-centers with wood lagging and a backdrain	$100,000
3b	60 lineal feet of slope improvement consisting of 25-foot long micropiles and geogrid reinforced fill	$100,000

12 CONSTRUCTION OBSERVATION

The recommendations provided in this report are based on our understanding of the proposed project and on our evaluation of the data collected based on subsurface conditions disclosed by widely spaced exploratory borings. It is imperative that the interpolated subsurface conditions be checked by a qualified person during construction. Observation of foundation excavations and observation and testing of compacted fill and backfill should be performed by a qualified person during construction. In addition, the project plans and specifications should be reviewed to check for conformance with the recommendations of this report prior to construction. It should be noted that, upon review of these documents, some recommendations presented in this report might be revised or modified.

During construction we recommend that the duties of the geotechnical consultant include, but not be limited to:

- Observing excavation bottoms and the placement and compaction of fill, including retaining wall backfill.
- Evaluating imported materials prior to their use as fill, if used.
- Performing field tests to evaluate fill compaction.
- Observing foundation excavations for bearing materials and cleaning prior to placement of reinforcing steel or concrete.

13 LIMITATIONS

The field evaluation, laboratory testing, and geotechnical analyses presented in this geotechnical report have been conducted in general accordance with current practice and the standard of care exercised by geotechnical consultants performing similar tasks in the project area. No warranty, expressed or implied, is made regarding the conclusions, recommendations, and opinions presented in this report. There is no evaluation detailed enough to reveal every subsurface condition. Variations may exist, and conditions not observed or described in this report may be encountered during construction. Uncertainties relative to subsurface conditions can be reduced through additional subsurface exploration. Additional subsurface evaluation will be performed upon request.

This document is intended to be used only in its entirety. No portion of the document, by itself, is designed to completely represent any aspect of the project described herein. Ninyo & Moore should be contacted if the reader requires additional information or has questions regarding the content, interpretations presented, or completeness of this document.

This report is intended for design purposes only. It does not provide sufficient data to prepare an accurate bid by contractors. It is suggested that the bidders and their geotechnical consultant perform an independent evaluation of the subsurface conditions in the project areas. The independent evaluations may include, but not be limited to, review of other geotechnical reports prepared for the adjacent areas, site reconnaissance, and additional exploration and laboratory testing.

Our conclusions, recommendations, and opinions are based on an analysis of the observed site conditions. If geotechnical conditions different from those described in this report are encountered, our office should be notified, and additional recommendations, if warranted, will be provided upon request. It should be understood that the conditions of a site could change with time as a result of natural processes or the activities of man at the subject site or nearby sites. In addition, changes to the applicable laws, regulations, codes, and standards of practice may occur due to government action or the broadening of knowledge. The findings of this report may, therefore, be invalidated over time, in part or in whole, by changes over which Ninyo & Moore has no control.

This report is intended exclusively for use by the client. Any use or reuse of the findings, conclusions, and/or recommendations of this report by parties other than the client is undertaken at said parties' sole risk.

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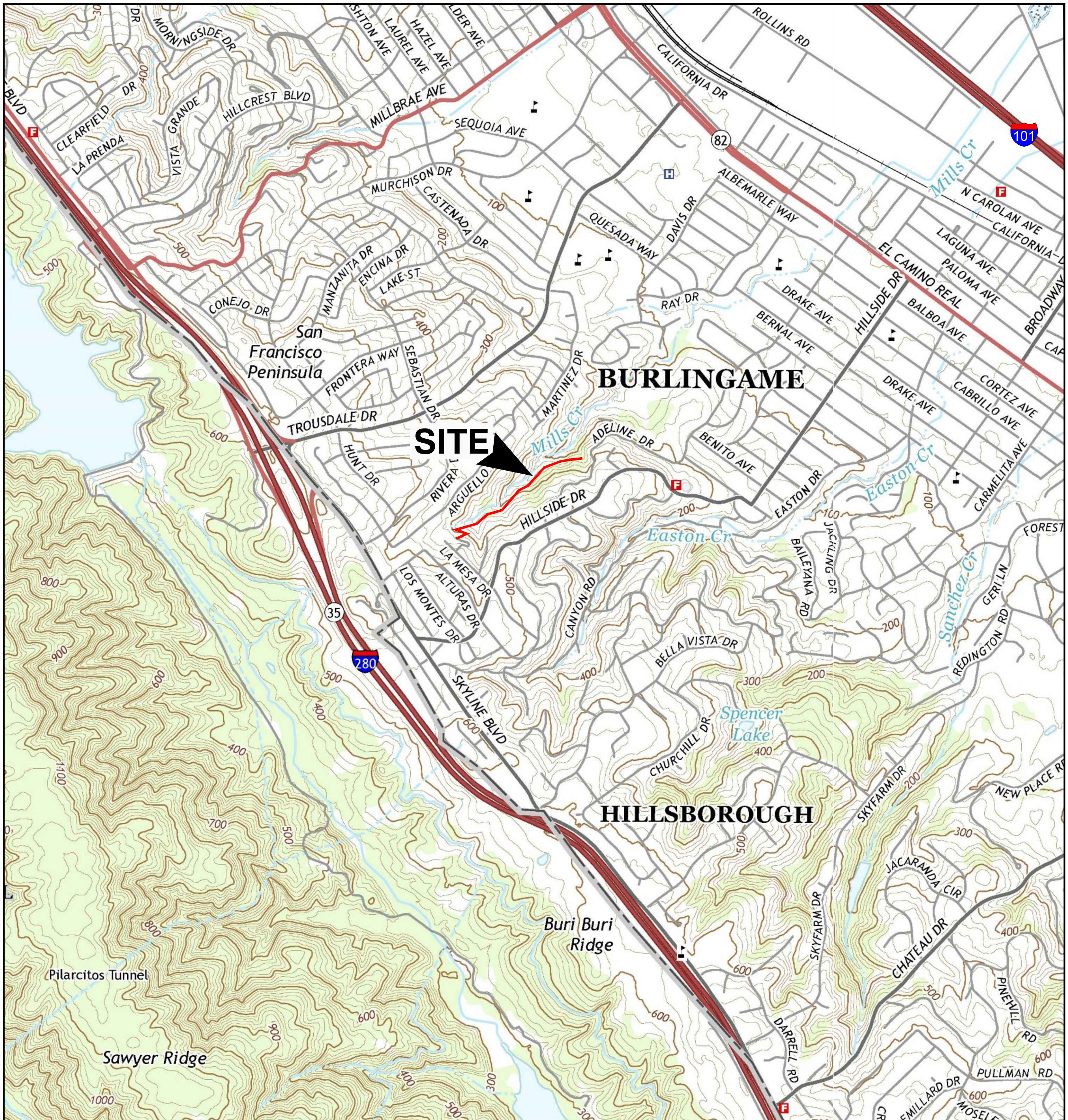
FIGURES



NOTE: DIMENSIONS, DIRECTIONS AND LOCATIONS ARE APPROXIMATE | REFERENCE: USGS, 2015

N

FEET

0 2,000 4,000

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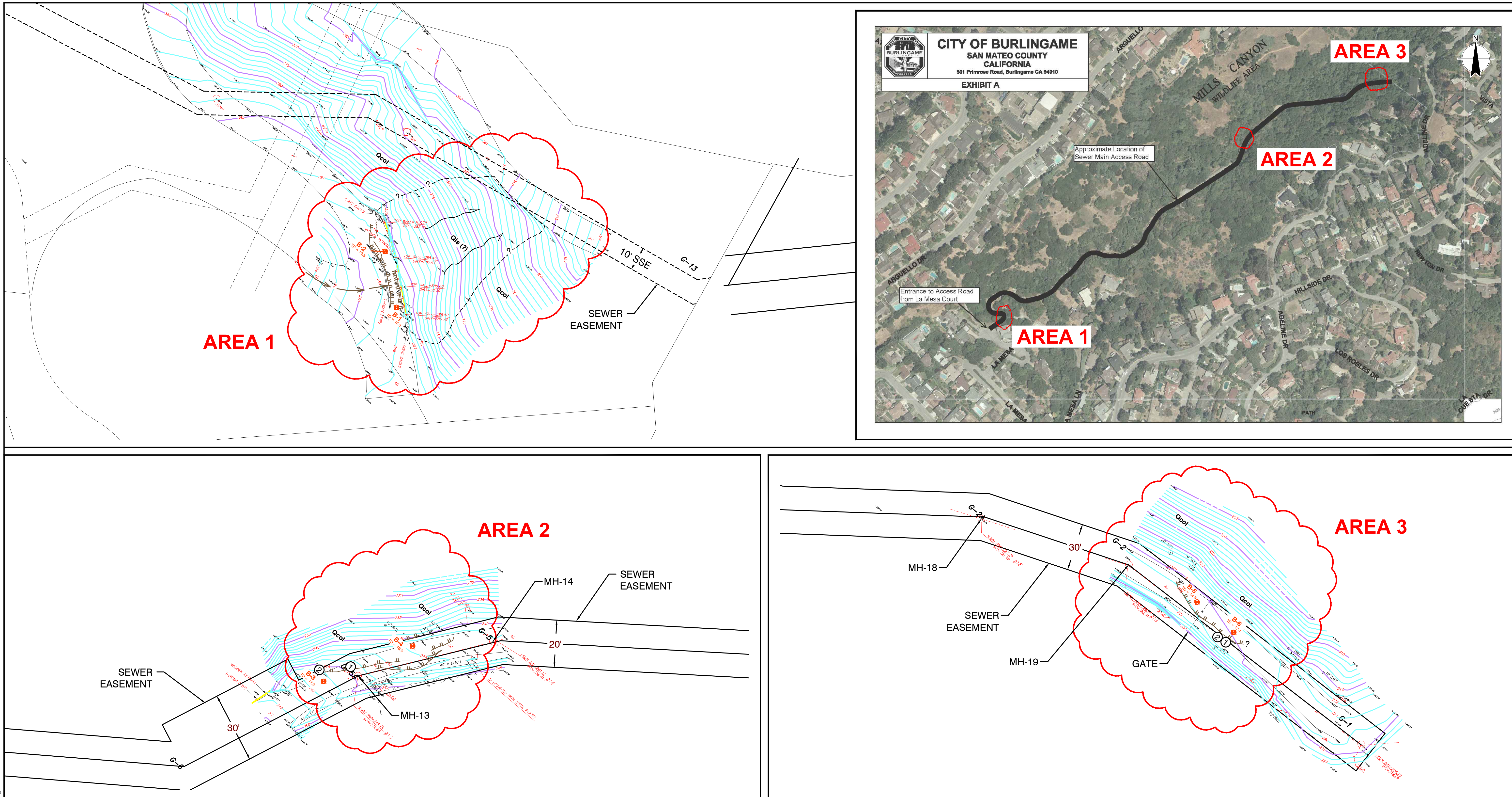
FIGURE 1

Ninyo & Moore

Geotechnical & Environmental Sciences Consultants

SITE LOCATION

MILLS CANYON SEWER ACCESS ROAD REPAIRS
CITY PROJECT NO. 85090
BURLINGAME, CALIFORNIA
403150001 | 02/18



NOTE: DIMENSIONS, DIRECTIONS AND LOCATIONS ARE APPROXIMATE | REFERENCE: ALEXANDER & ASSOCIATES, INC., 2018

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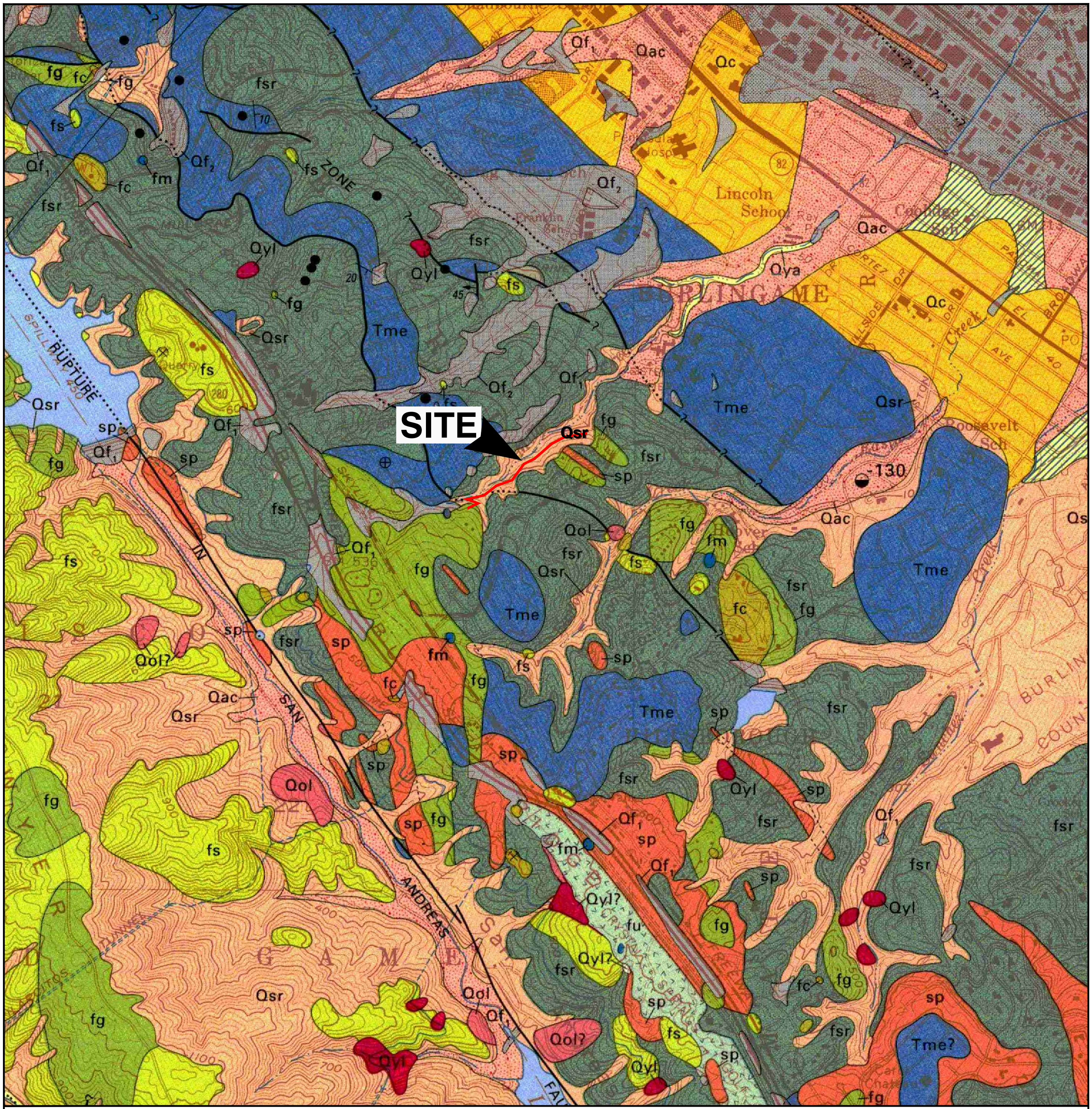
FIGURE 2

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

AREA LOCATIONS
MILLS CANYON SEWER ACCESS ROAD REPAIRS
CITY PROJECT NO. 85090
BURLINGAME, CALIFORNIA
403150001 I 02/18



LEGEND

Qsr — Slope wash and colluvium (Holocene)

Tme — Merced Formation (Pliocene)

Franciscan Complex (Cretaceous/Jurassic)

fg — Greenstone

fsr — Sheared rock

sp — Serpentinite

N

FEET

0 2,000 4,000

NOTE: DIMENSIONS, DIRECTIONS AND LOCATIONS ARE APPROXIMATE | REFERENCE: PAMPEYAN 1994 USGS MIS I-2390

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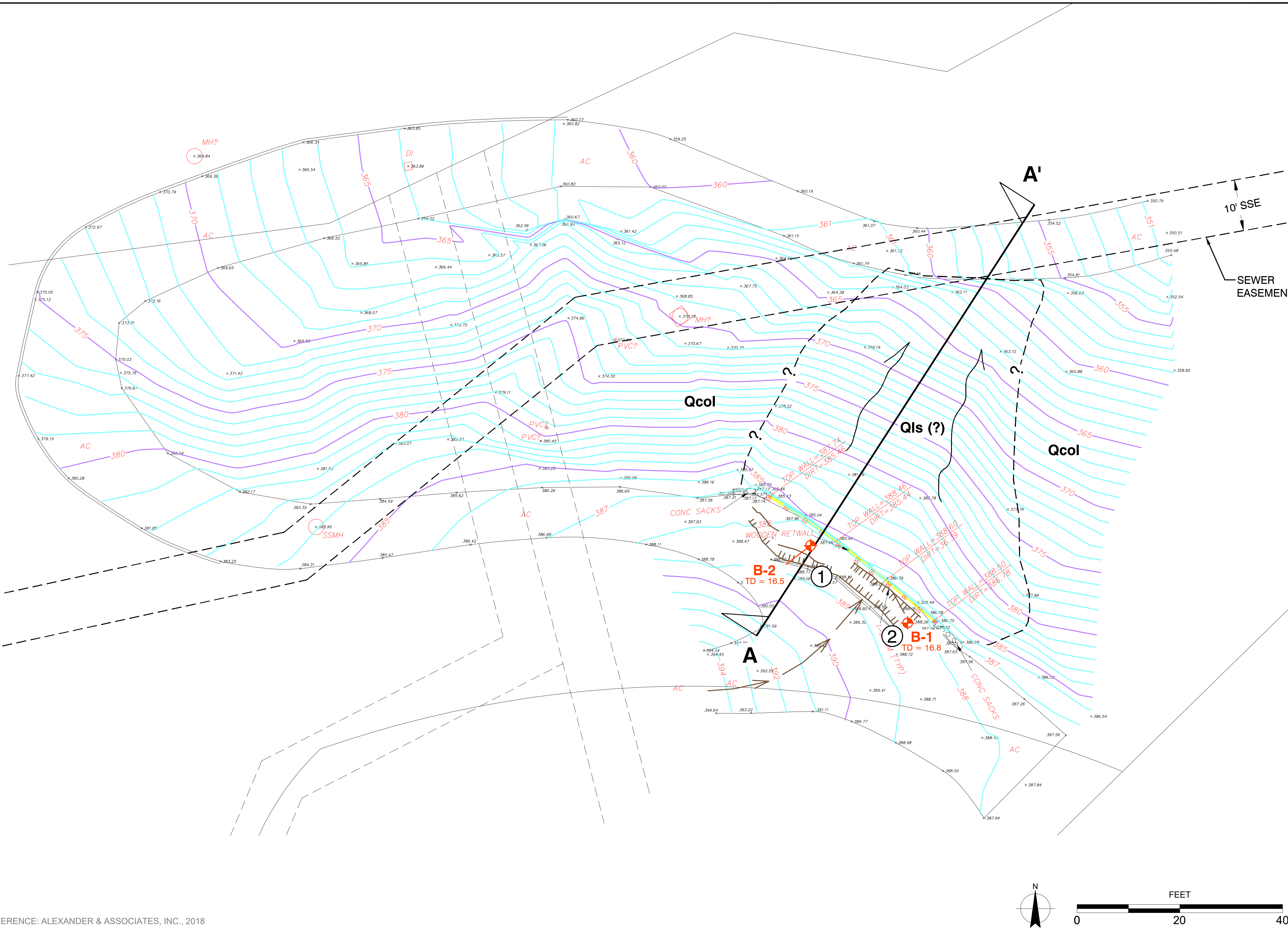
FIGURE 3

Ninyo & Moore

Geotechnical & Environmental Sciences Consultants

REGIONAL GEOLOGY

MILLS CANYON SEWER ACCESS ROAD REPAIRS
CITY PROJECT NO. 85090
BURLINGAME, CALIFORNIA
403150001 | 02/18



LEGEND

Qcol COLLUVIUM

Qls LANDSLIDE DEBRIS

LANDSLIDE (QUERIED WHERE UNCERTAIN)

SURFACE FLOW

GROUND CRACK

B-1 TD=16.8 BORING LOCATION TD=TOTAL DEPTH IN FEET

A A' CROSS SECTION

NOTES

1. 6" OF VERTICAL DISPLACEMENT (11/29/2017)
2. 1/2" OF VERTICAL DISPLACEMENT (11/29/2017)

NOTE: DIMENSIONS, DIRECTIONS AND LOCATIONS ARE APPROXIMATE | REFERENCE: ALEXANDER & ASSOCIATES, INC., 2018

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FIGURE 4

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

PRELIMINARY GEOLOGIC MAP - AREA 1

MILLS CANYON SEWER ACCESS ROAD REPAIRS
CITY PROJECT NO. 85090
BURLINGAME, CALIFORNIA

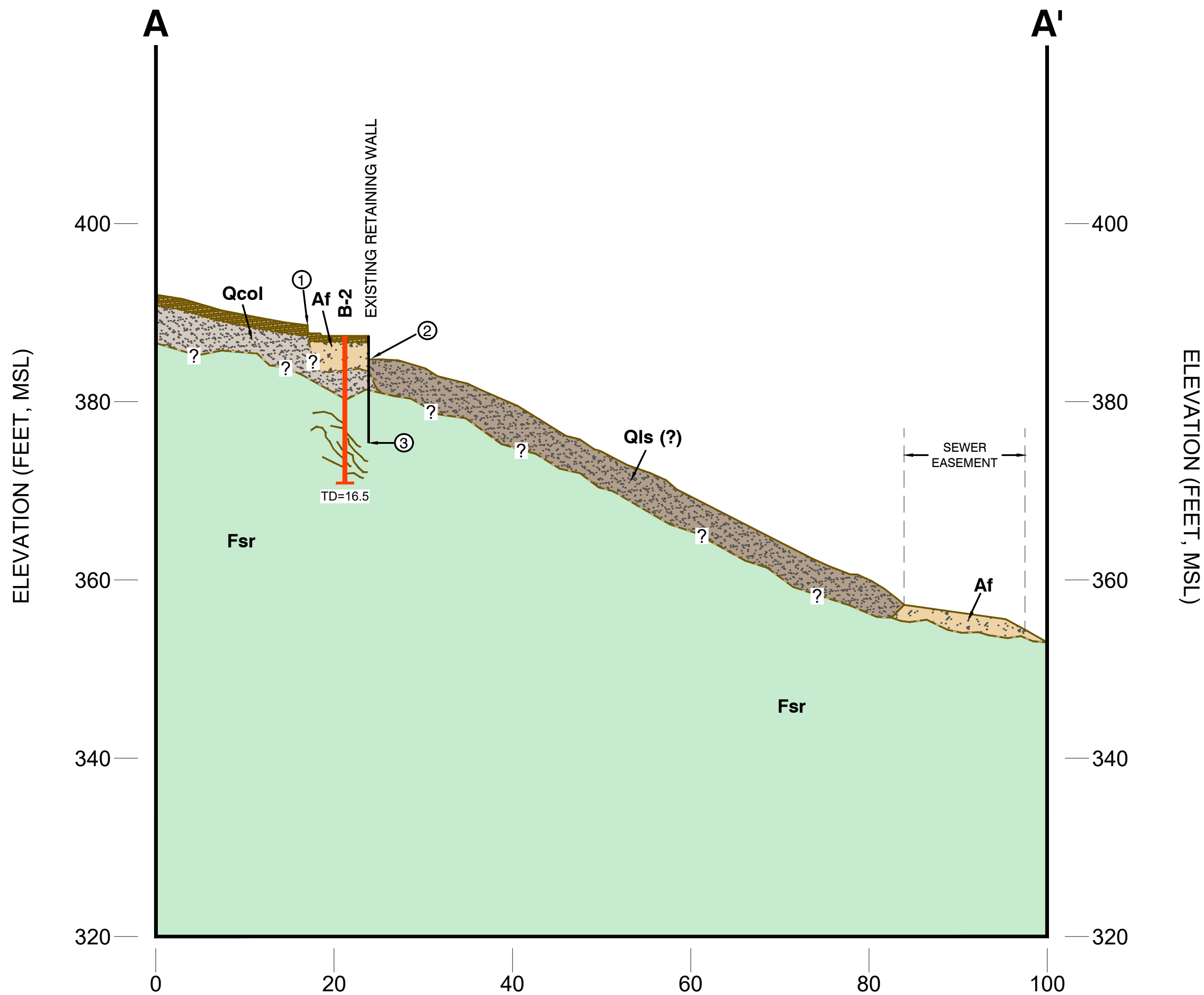
403150001 | 02/18

LEGEND

- Af — ARTIFICIAL FILL
- Qls — LANDSLIDE DEBRIS
- Qcol — COLLUVIUM
- Fsr — FRANCISCAN COMPLEX
- ? — GEOLOGIC CONTACT; QUERIED WHERE UNCERTAIN
- B-2, TD=16.5 — SOIL BORING, TD=TOTAL DEPTH IN FEET

NOTES

1. 4 " OF VERTICAL DISPLACEMENT.
2. 12" GAP BELOW BOTTOM OF TIMBER LAGGING AND GROUND.
3. TOTAL LENGTH OF PIERS 12 FEET, EMBEDDED 10 FEET.



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NOTE: DIMENSIONS, DIRECTIONS AND LOCATIONS ARE APPROXIMATE

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FIGURE 5

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

CROSS SECTION A-A' - AREA 1

MILLS CANYON SEWER ACCESS ROAD REPAIRS
CITY PROJECT NO. 85090
BURLINGAME, CALIFORNIA
403150001 I 02/18

LEGEND

Qcol COLLUVIUM

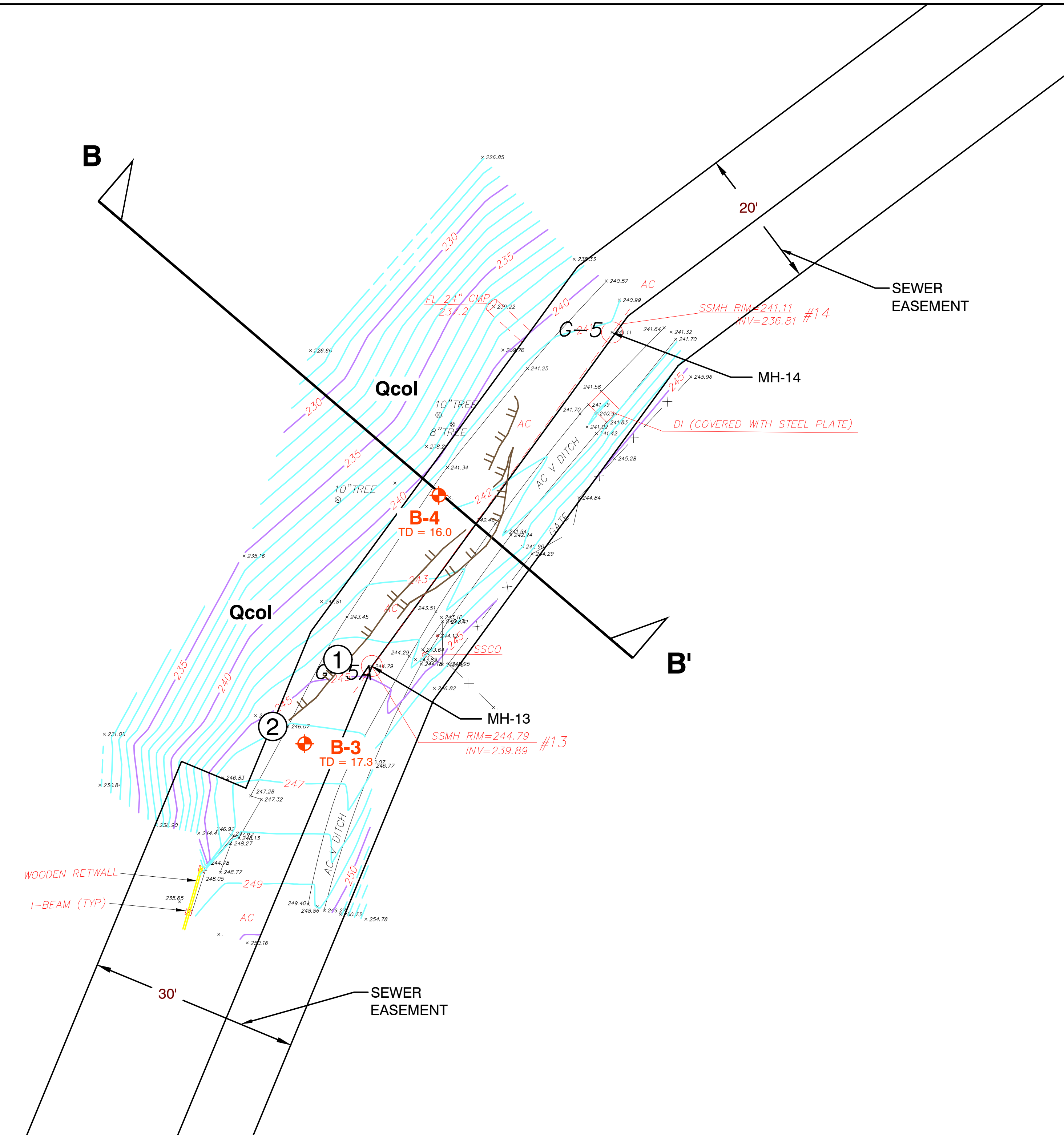
GROUND CRACK

B-3 TD=17.3 BORING LOCATION
TD=TOTAL DEPTH IN FEET

B B' CROSS SECTION

NOTES:

1. 8" OF LATERAL DISPLACEMENT (01/22/2018)
2. NO GROUND CRACKS BEYOND PAVEMENT



NOTE: DIMENSIONS, DIRECTIONS AND LOCATIONS ARE APPROXIMATE | REFERENCE: ALEXANDER & ASSOCIATES, INC., 2018

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FIGURE 6

Ninyo & Moore
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PRELIMINARY GEOLOGIC MAP - AREA 2
MILLS CANYON SEWER ACCESS ROAD REPAIRS
CITY PROJECT NO. 85090
BURLINGAME, CALIFORNIA
403150001 | 02/18

LEGEND

Af — ARTIFICIAL FILL

Qal — ALLUVIUM

Qcol — COLLUVIUM

Fsr — FRANCISCAN COMPLEX

?— GEOLOGIC CONTACT; QUERIED WHERE UNCERTAIN

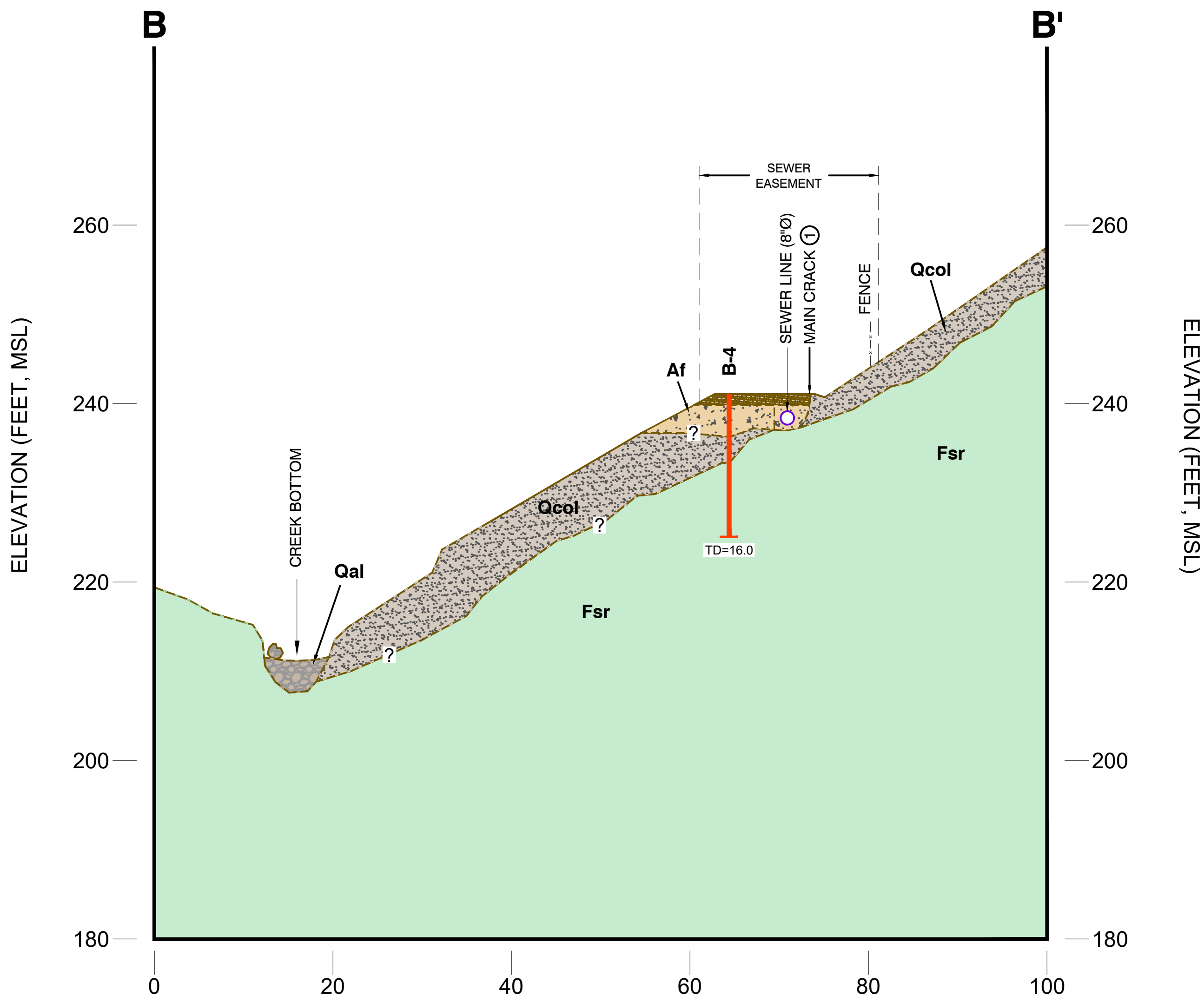
B-4 TD=16.0 — BORING TD=TOTAL DEPTH IN FEET

NOTES

1 HORIZONTAL SEPARATION UP TO 5", BUT ONLY ~1/2" OF VERTICAL SEPARATION.

FEET

0 20 40



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NOTE: DIMENSIONS, DIRECTIONS AND LOCATIONS ARE APPROXIMATE

FIGURE 7

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

CROSS SECTION B-B' - AREA 2

MILLS CANYON SEWER ACCESS ROAD REPAIRS
CITY PROJECT NO. 85090
BURLINGAME, CALIFORNIA
403150001 I 02/18

LEGEND

Qcol COLLUVIUM

GROUND CRACK

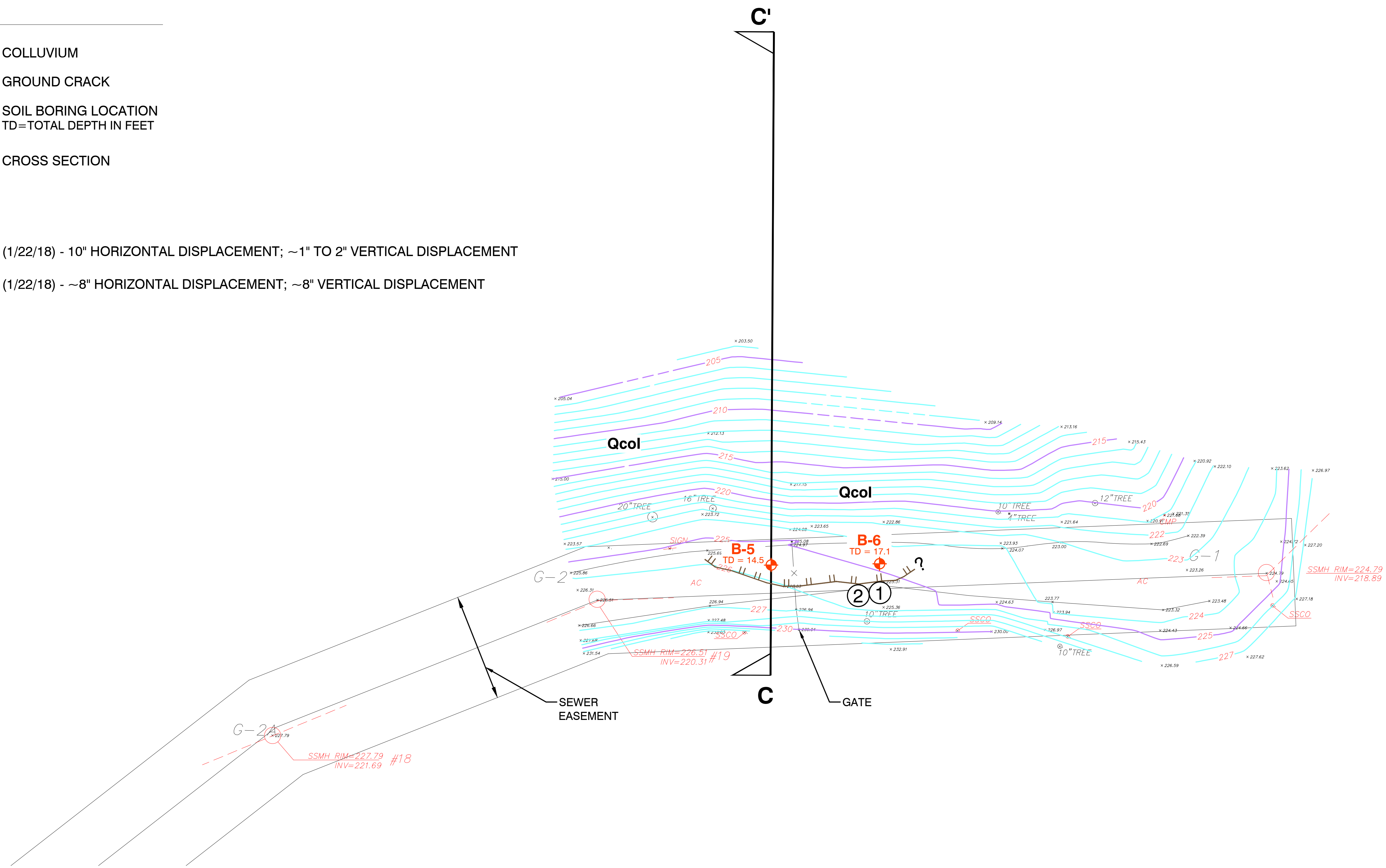
B-5 TD=14.5 SOIL BORING LOCATION
TD=TOTAL DEPTH IN FEET

C C' CROSS SECTION

NOTES:

① (1/22/18) - 10" HORIZONTAL DISPLACEMENT; ~1" TO 2" VERTICAL DISPLACEMENT

② (1/22/18) - ~8" HORIZONTAL DISPLACEMENT; ~8" VERTICAL DISPLACEMENT



NOTE: DIMENSIONS, DIRECTIONS AND LOCATIONS ARE APPROXIMATE | REFERENCE: ALEXANDER & ASSOCIATES, INC., 2018

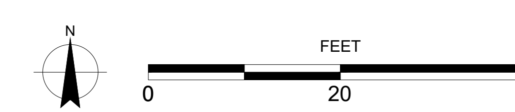


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FIGURE 8

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

PRELIMINARY GEOLOGIC MAP - AREA 3

MILLS CANYON SEWER ACCESS ROAD REPAIRS
CITY PROJECT NO. 85090
BURLINGAME, CALIFORNIA
403150001 | 02/18

LEGEND

Af — ARTIFICIAL FILL

Qal — ALLUVIUM

Qcol — COLLUVIUM

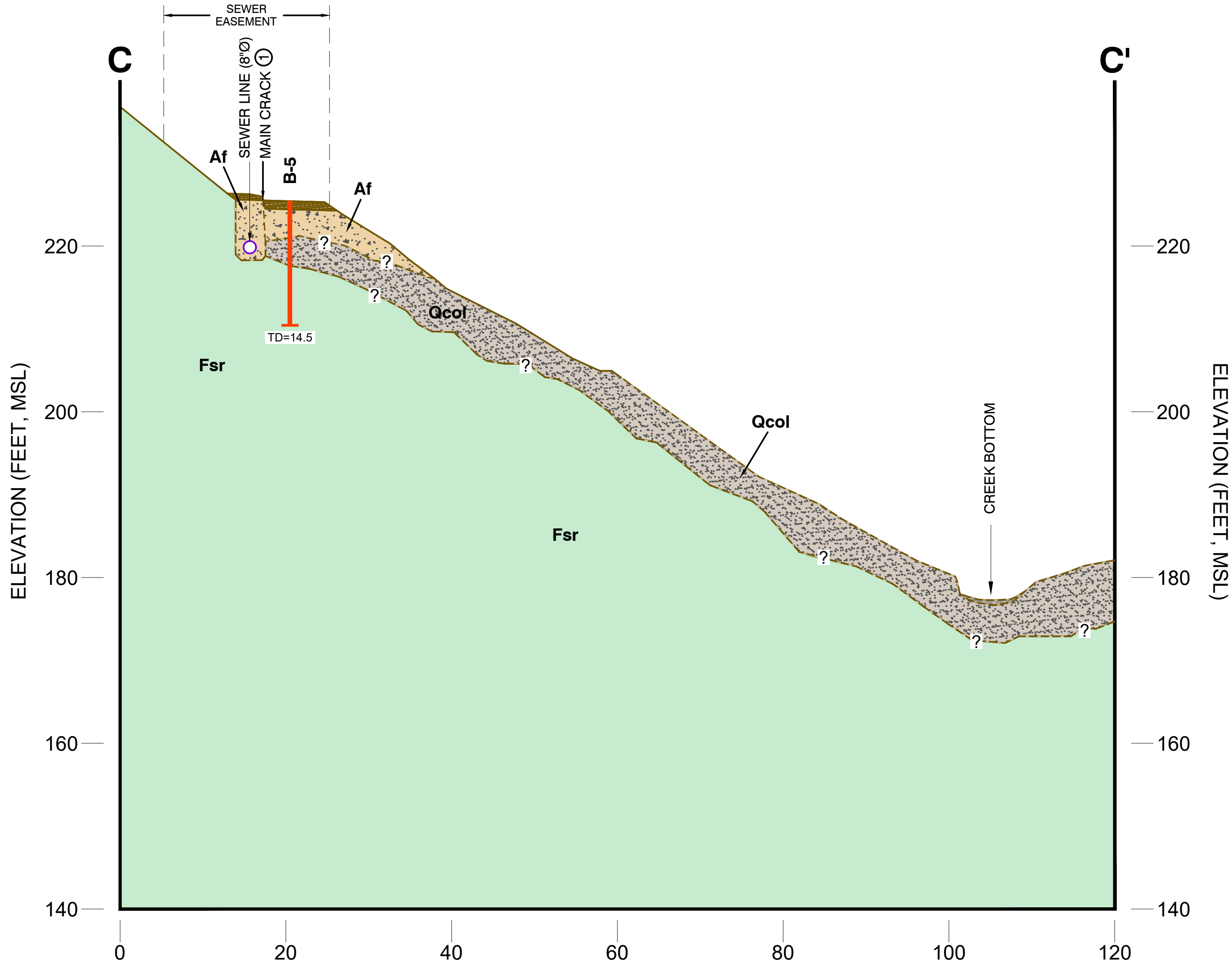
Fsr — FRANCISCAN COMPLEX

GEOLOGIC CONTACT; QUERIED WHERE UNCERTAIN

B-5 TD=14.5 — BORING TD=TOTAL DEPTH IN FEET

NOTES

1. HORIZONTAL SEPARATION UP TO 5"; VERTICAL SEPARATION UP TO 7".



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NOTE: DIMENSIONS, DIRECTIONS AND LOCATIONS ARE APPROXIMATE

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FIGURE 9

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

CROSS SECTION C-C' - AREA 3

MILLS CANYON SEWER ACCESS ROAD REPAIRS
CITY PROJECT NO. 85090
BURLINGAME, CALIFORNIA
403150001 | 02/18

APPENDIX A

Boring Logs

APPENDIX A

BORING LOGS

Field Procedure for the Collection of Disturbed Samples

Disturbed soil samples were obtained in the field using the following methods.

Bulk Samples

Bulk samples of representative earth materials were obtained from the exploratory borings. The samples were bagged and transported to the laboratory for testing.

The Standard Penetration Test (SPT) Sampler

Disturbed drive samples of earth materials were obtained by means of a Standard Penetration Test sampler. The sampler is composed of a split barrel with an external diameter of 2 inches and an unlined internal diameter of 1-3/8 inches. The sampler was driven into the ground 12 to 18 inches with a 140-pound hammer free-falling from a height of 30 inches in general accordance with ASTM D 1586. The blow counts were recorded for every 6 inches of penetration; the blow counts reported on the logs are those for the last 12 inches of penetration. Soil samples were observed and removed from the sampler, bagged, sealed and transported to the laboratory for testing.

Field Procedure for the Collection of Relatively Undisturbed Samples

Relatively undisturbed soil samples were obtained in the field using the following methods.

The Modified Split-Barrel Drive Sampler

The sampler, with an external diameter of 3.0 inches, was lined with 6-inch long, thin brass liners with inside diameters of approximately 2.4 inches. The sample barrel was driven into the ground with the weight of a hammer in general accordance with ASTM D 3550. The driving weight was permitted to fall freely. The approximate length of the fall, the weight of the hammer, and the number of blows per foot of driving are presented on the boring log as an index to the relative resistance of the materials sampled. The samples were removed from the sample barrel in the brass liners, sealed, and transported to the laboratory for testing.

BORING LOG EXPLANATION SHEET

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	
0								Bulk sample.
								Modified split-barrel drive sampler.
								No recovery with modified split-barrel drive sampler.
								Sample retained by others.
								Standard Penetration Test (SPT).
5								No recovery with a SPT.
			XX/XX					Shelby tube sample. Distance pushed in inches/length of sample recovered in inches.
								No recovery with Shelby tube sampler.
								Continuous Push Sample.
								Seepage.
10								Groundwater encountered during drilling.
								Groundwater measured after drilling.
							SM	MAJOR MATERIAL TYPE (SOIL): Solid line denotes unit change.
							CL	Dashed line denotes material change.
								Attitudes: Strike/Dip b: Bedding c: Contact j: Joint f: Fracture F: Fault cs: Clay Seam s: Shear bss: Basal Slide Surface sf: Shear Fracture sz: Shear Zone sbs: Shear Bedding Surface
15								
20								The total depth line is a solid line that is drawn at the bottom of the boring.

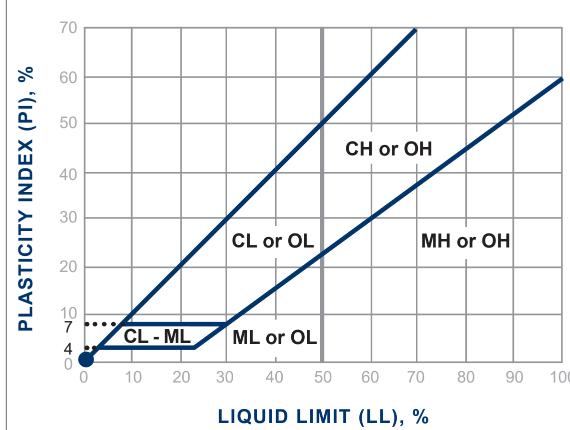
Soil Classification Chart Per ASTM D 2488

Primary Divisions			Secondary Divisions: Group Symbol	Secondary Divisions: Group Name
COARSE-GRAINED SOILS more than 50% retained on No. 200 sieve	**GRAVEL** more than 50% of coarse fraction retained on No. 4 sieve	CLEAN GRAVEL less than 5% fines	GW	well-graded GRAVEL
			GP	poorly graded GRAVEL
		GRAVEL with DUAL CLASSIFICATIONS 5% to 12% fines	GW-GM	well-graded GRAVEL with silt
			GP-GM	poorly graded GRAVEL with silt
			GW-GC	well-graded GRAVEL with clay
			GP-GC	poorly graded GRAVEL with
		GRAVEL with FINES more than 12% fines	GM	silty GRAVEL
			GC	clayey GRAVEL
			GC-GM	silty, clayey GRAVEL
	SAND 50% or more of coarse fraction passes No. 4 sieve	CLEAN SAND less than 5% fines	SW	well-graded SAND
			SP	poorly graded SAND
		SAND with DUAL CLASSIFICATIONS 5% to 12% fines	SW-SM	well-graded SAND with silt
			SP-SM	poorly graded SAND with silt
			SW-SC	well-graded SAND with clay
			SP-SC	poorly graded SAND with clay
		SAND with FINES more than 12% fines	SM	silty SAND
			SC	clayey SAND
			SC-SM	silty, clayey SAND
FINE-GRAINED SOILS 50% or more passes No. 200 sieve	**SILT and CLAY** liquid limit less than 50%	INORGANIC	CL	lean CLAY
			ML	SILT
			CL-ML	silty CLAY
		ORGANIC	OL (PI > 4)	organic CLAY
			OL (PI < 4)	organic SILT
	SILT and CLAY liquid limit 50% or more	INORGANIC	CH	fat CLAY
			MH	elastic SILT
		ORGANIC	OH (plots on or above "A"-line)	organic CLAY
			OH (plots below "A"-line)	organic SILT
	Highly Organic Soils		PT	Peat

Grain Size

Description		Sieve Size	Grain Size	Approximate Size
Boulders		> 12"	> 12"	Larger than basketball-sized
Cobbles		3 - 12"	3 - 12"	Fist-sized to basketball-sized
Gravel	Coarse	3/4 - 3"	3/4 - 3"	Thumb-sized to fist-sized
	Fine	#4 - 3/4"	0.19 - 0.75"	Pea-sized to thumb-sized
Sand	Coarse	#10 - #4	0.079 - 0.19"	Rock-salt-sized to pea-sized
	Medium	#40 - #10	0.017 - 0.079"	Sugar-sized to rock-salt-sized
	Fine	#200 - #40	0.0029 - 0.017"	Flour-sized to sugar-sized
Fines		Passing #200	< 0.0029"	Flour-sized and smaller

Plasticity Chart



Apparent Density - Coarse-Grained Soil

Apparent Density	Spooling Cable or Cathead: SPT (blows/foot)	Spooling Cable or Cathead: Modified Split Barrel (blows/foot)	Automatic Trip Hammer: SPT (blows/foot)	Automatic Trip Hammer: Modified Split Barrel (blows/foot)
Very Loose	≤ 4	≤ 8	≤ 3	≤ 5
Loose	5 - 10	9 - 21	4 - 7	6 - 14
Medium Dense	11 - 30	22 - 63	8 - 20	15 - 42
Dense	31 - 50	64 - 105	21 - 33	43 - 70
Very Dense	> 50	> 105	> 33	> 70

Consistency - Fine-Grained Soil

Consis-tency	Spooling Cable or Cathead: SPT (blows/foot)	Spooling Cable or Cathead: Modified Split Barrel (blows/foot)	Automatic Trip Hammer: SPT (blows/foot)	Automatic Trip Hammer: Modified Split Barrel (blows/foot)
Very Soft	< 2	< 3	< 1	< 2
Soft	2 - 4	3 - 5	1 - 3	2 - 3
Firm	5 - 8	6 - 10	4 - 5	4 - 6
Stiff	9 - 15	11 - 20	6 - 10	7 - 13
Very Stiff	16 - 30	21 - 39	11 - 20	14 - 26
Hard	> 30	> 39	> 20	> 26

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USCS METHOD OF SOIL CLASSIFICATION

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0								ASPHALT CONCRETE: Approximately 5 inches thick.
								AGGREGATE BASE: Approximately 6 inches thick.
							SC	FILL: Retaining wall drain rock, fine gravel.
			15	15.2	100.4			COLLUVIUM: Brown, moist, loose, clayey SAND; contact, sand above, gravel below.
							GC	Brown, moist, loose, clayey GRAVEL; angular coarse gravel composed of sandstone and metashale.
			39					BEDROCK: Brown, moist, highly weathered METABASALT; weak rock.
10								
			50/5"	8.2	131.7			Gray, moist, weathered METASHALE; weak rock.
			50/4"					
								Total Depth = Drilling and sampling refusal at 16.8 feet.
								Backfilled with cement grout upon completion.
20								
30								
40								

DATE DRILLED 11/29/2017 BORING NO. B-1

GROUND ELEVATION 389' ± (MSL) SHEET 1 OF 1

METHOD OF DRILLING 4" SSA, B-24 Truck Mounted, California Geotech

DRIVE WEIGHT 140 LBS (cathead) DROP 30 INCHES

SAMPLED BY RH LOGGED BY RH REVIEWED BY DCS

Notes:
Groundwater, though not encountered at the time of drilling, may rise to a higher level due to seasonal variations in precipitation and several other factors as discussed in the report.

The ground elevation shown above is an estimation only. It is based on our interpretations of published maps and other documents reviewed for the purposes of this evaluation. It is not sufficiently accurate for preparing construction bids and design documents.

FIGURE A- 1

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MILLS CANYON SEWER ACCESS ROAD REPAIRS
BURLINGAME, CALIFORNIA

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0								ASPHALT CONCRETE: Approximately 5 inches thick.
								AGGREGATE BASE: Approximately 12 inches thick.
			8	15.8	109.4		GC	FILL: Brown and dark brown, moist, loose, clayey GRAVEL with sand; fine to coarse angular gravel.
			13				CL	COLLUVIUM: Brown, moist, loose, clayey SAND.
			39					BEDROCK: Grayish brown, moist, highly weathered METASHALE and METABASALT.
10								
			50/6"	5	132			Gray, moist, weathered METASHALE; weak rock.
			65					
								Total Depth = 16.5 feet. Drilling refusal at 15 feet. Backfilled with cement grout upon completion.
20								
30								
40								

DATE DRILLED 11/29/2017 BORING NO. B-2

GROUND ELEVATION 388' ± (MSL) SHEET 1 OF 1

METHOD OF DRILLING 4" SSA, B-24 Truck Mounted, California Geotech

DRIVE WEIGHT 140 LBS (cathead) DROP 30 INCHES

SAMPLED BY GL LOGGED BY GL REVIEWED BY DCS

Notes:

Groundwater, though not encountered at the time of drilling, may rise to a higher level due to seasonal variations in precipitation and several other factors as discussed in the report.

The ground elevation shown above is an estimation only. It is based on our interpretations of published maps and other documents reviewed for the purposes of this evaluation. It is not sufficiently accurate for preparing construction bids and design documents.

DATE DRILLED 11/29/2017 BORING NO. B-3

GROUND ELEVATION 340' ± (MSL) SHEET 1 OF 1

METHOD OF DRILLING 4" SSA, B-24 Truck Mounted, California Geotech

DRIVE WEIGHT 140 LBS (cathead) DROP 30 INCHES

SAMPLED BY GL LOGGED BY GL REVIEWED BY DCS

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0								ASPHALT CONCRETE: Approximately 4 inches thick.
							CL	AGGREGATE BASE: Approximately 5 inches thick.
			13					FILL: Dark brown and dark gray, moist, very stiff, sandy lean CLAY with gravel.
			23				CL	COLLUVIUM: Dark brown and dark gray, moist, very stiff, sandy lean CLAY with gravel.
			45					Dark gray; very stiff.
10								BEDROCK: Dark greenish gray, moist, highly weathered SERPENTINITE; weak rock.
			74					
			50/4"	9.3	118.7			
20								Total Depth = Sampling refusal at 17.3 feet. Drilling resufal at 16 feet. Backfilled with cement grout upon completion. Notes: Groundwater, though not encountered at the time of drilling, may rise to a higher level due to seasonal variations in precipitation and several other factors as discussed in the report. The ground elevation shown above is an estimation only. It is based on our interpretations of published maps and other documents reviewed for the purposes of this evaluation. It is not sufficiently accurate for preparing construction bids and design documents.
30								
40								

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION

DATE DRILLED 12/1/2017 BORING NO. B-4

GROUND ELEVATION 337' ± (MSL) SHEET 1 OF 1

METHOD OF DRILLING 4" SSA, B-24 Truck Mounted, California Geotech

DRIVE WEIGHT 140 LBS (cathead) DROP 30 INCHES

SAMPLED BY GL LOGGED BY GL REVIEWED BY DCS

DEPTH (feet)	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0					ASPHALT CONCRETE: Approximately 3.5 inches.
				CL	AGGREGATE BASE: Approximately 3 inches.
	14				FILL: Brown, moist, loose, clayey SAND; trace gravel, some roots.
	19	15.8	97.3		
				CL	COLLUVIUM: Brown, moist, loose, clayey SAND; trace gravel, some roots.
10	55				BEDROCK: Greenish gray, moist, weathered SERPENTINITE with seams of metashale.
	89	11.1	121.3		
	50/0"				
20					
30					
40					

Total Depth = Drilling and sampling refusal at 16 feet.

Backfilled with cement grout upon completion.

Notes:
Groundwater, though not encountered at the time of drilling, may rise to a higher level due to seasonal variations in precipitation and several other factors as discussed in the report.

The ground elevation shown above is an estimation only. It is based on our interpretations of published maps and other documents reviewed for the purposes of this evaluation. It is not sufficiently accurate for preparing construction bids and design documents.

FIGURE A - 4

Ninyo & Moore
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MILLS CANYON SEWER ACCESS ROAD REPAIRS
BURLINGAME, CALIFORNIA

DATE DRILLED 12/2017 BORING NO. B-5

GROUND ELEVATION 238' ± (MSL) SHEET 1 OF 1

METHOD OF DRILLING 4" SSA, B-24 Truck Mounted, California Geotech

DRIVE WEIGHT 140 LBS (cathead) DROP 30 INCHES

SAMPLED BY GL LOGGED BY GL REVIEWED BY DCS

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0							CL	ASPHALT: Approximately 3 inches thick. AGGREGATE BASE: Approximately 3.5 inches thick.
		■	33	15.7	100			FILL: Brown, moist, very stiff, lean CLAY; trace sand; trace angular sandstone gravel.
		■	32				CL	COLLUVIUM: Brown, moist, very stiff, lean CLAY; trace sand; trace angular sandstone gravel.
		■	50/6"	12.1	104.7			BEDROCK: Light brown, moist, weathered SANDSTONE.
10								
		■	50/1"	6.4	112.1			
								Total Depth = Sampling and drilling refusal at 14.5 feet.
								Backfilled with cement grout.
20								
30								
40								

Notes:
Groundwater, though not encountered at the time of drilling, may rise to a higher level due to seasonal variations in precipitation and several other factors as discussed in the report.

The ground elevation shown above is an estimation only. It is based on our interpretations of published maps and other documents reviewed for the purposes of this evaluation. It is not sufficiently accurate for preparing construction bids and design documents.

FIGURE A - 5

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

MILLS CANYON SEWER ACCESS ROAD REPAIRS
BURLINGAME, CALIFORNIA

DATE DRILLED 12/1/2017 BORING NO. B-6

GROUND ELEVATION 234' ± (MSL) SHEET 1 OF 1

METHOD OF DRILLING 4" SSA, B-24 Truck Mounted, California Geotech

DRIVE WEIGHT 140 LBS (cathead) DROP 30 INCHES

SAMPLED BY GL LOGGED BY GL REVIEWED BY DCS

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0							CL	FILL: Dark brown, moist, very stiff, lean CLAY.
			20					
			28				CL	COLLUVIUM: Dark brown, moist, very stiff, lean CLAY.
			12					Yellowish brown; stiff; trace angular sandstone gravel.
10								
								BEDROCK: Brown, dry to moist, weathered SANDSTONE.
			50/5"					
			50/1"	10.3	96.2			
								Total Depth = Sampling refusal at 17.1 feet. Backfilled with cement grout upon completion. Notes: Groundwater, though not encountered at the time of drilling, may rise to a higher level due to seasonal variations precipitation and several other factors as discussed in the report. The ground elevation shown above is an estimation only. It is based on our interpretations of published maps and other documents reviewed for the purposes of this evaluation. It is not sufficiently accurate for preparing construction bids and design documents.
20								
30								
40								

APPENDIX B

Laboratory Testing

APPENDIX B

LABORATORY TESTING

Classification
Soils were visually and texturally classified in accordance with the Unified Soil Classification System (USCS) in general accordance with ASTM D 2488-00. Soil classifications are indicated on the logs of the exploratory borings in Appendix A.

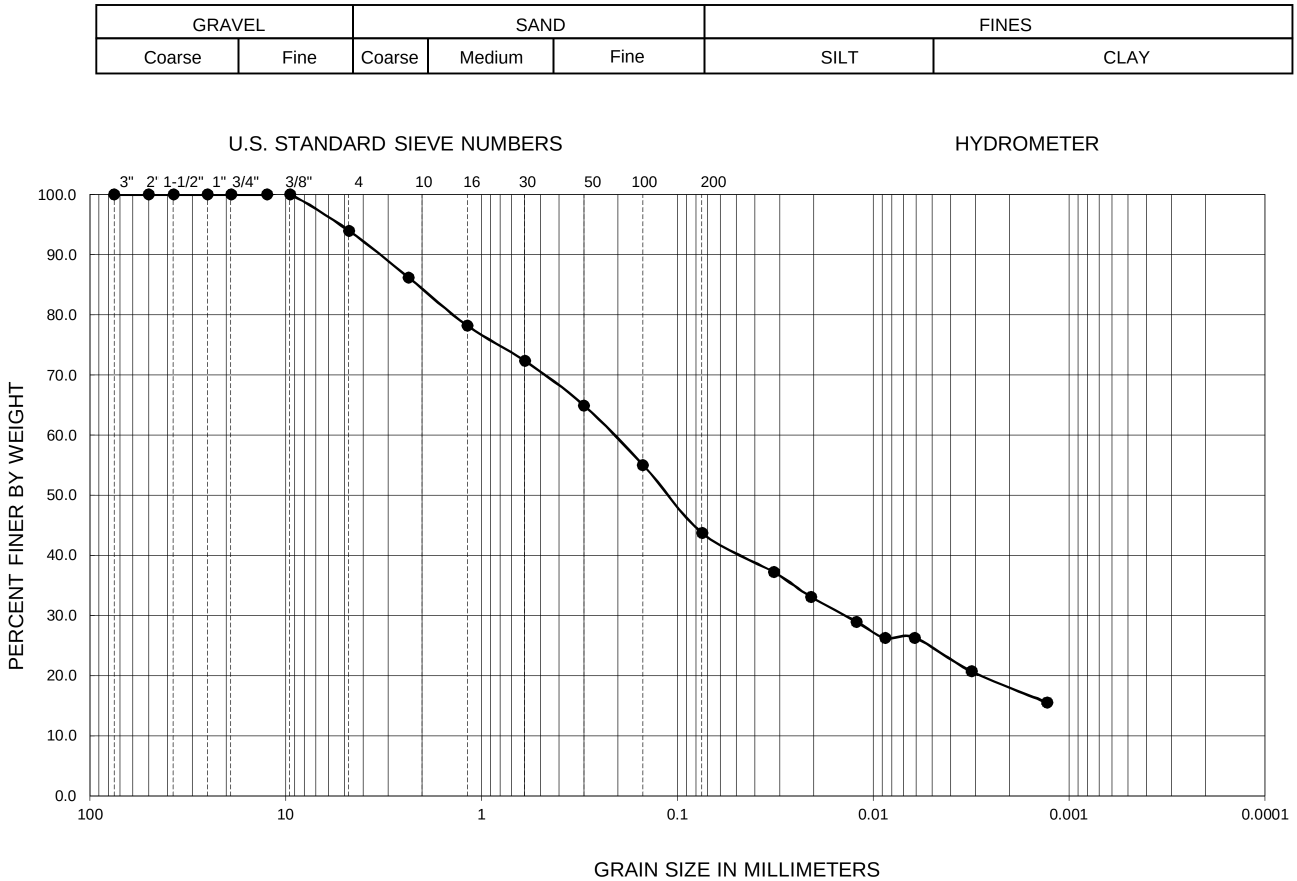
In-Place Moisture and Density Tests
The moisture content and dry density of relatively undisturbed samples obtained from the exploratory borings were evaluated in general accordance with ASTM D 2937-04. The test results are presented on the logs of the exploratory borings in Appendix A.

Gradation Analysis
A gradation analysis test was performed on selected representative soil samples in general accordance with ASTM D 422. The grain-size distribution curves are shown on Figures B-1 through B-4. These test results were utilized in evaluating the soil classifications in accordance with the USCS.

Atterberg Limits
Tests were performed on selected representative fine-grained soil samples to evaluate the liquid limit, plastic limit, and plasticity index in general accordance with ASTM D 4318. These test results were utilized to evaluate the soil classification in accordance with the USCS. The test results and classifications are shown on Figure B-5.

Direct Shear Tests
Direct shear tests were performed on relatively undisturbed and remolded samples in general accordance with ASTM D 3080 to evaluate the shear strength characteristics of selected materials. The samples were inundated during shearing to represent adverse field conditions. The results are shown on Figures B-6 through B-10.

Unconfined Compression Test
An unconfined compression test was performed on relatively undisturbed samples in general accordance with ASTM D 2166. The test results are shown on Figure B-11.



Symbol	Sample Location	Depth (ft)	Liquid Limit	Plastic Limit	Plasticity Index	D_{10}	D_{30}	D_{60}	C_u	C_c	Passing No. 200 (%)	USCS
●	B-2	4.5-5.0	34	13	21	-	0.02	0.21	-	-	44	SC

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422

FIGURE B-1

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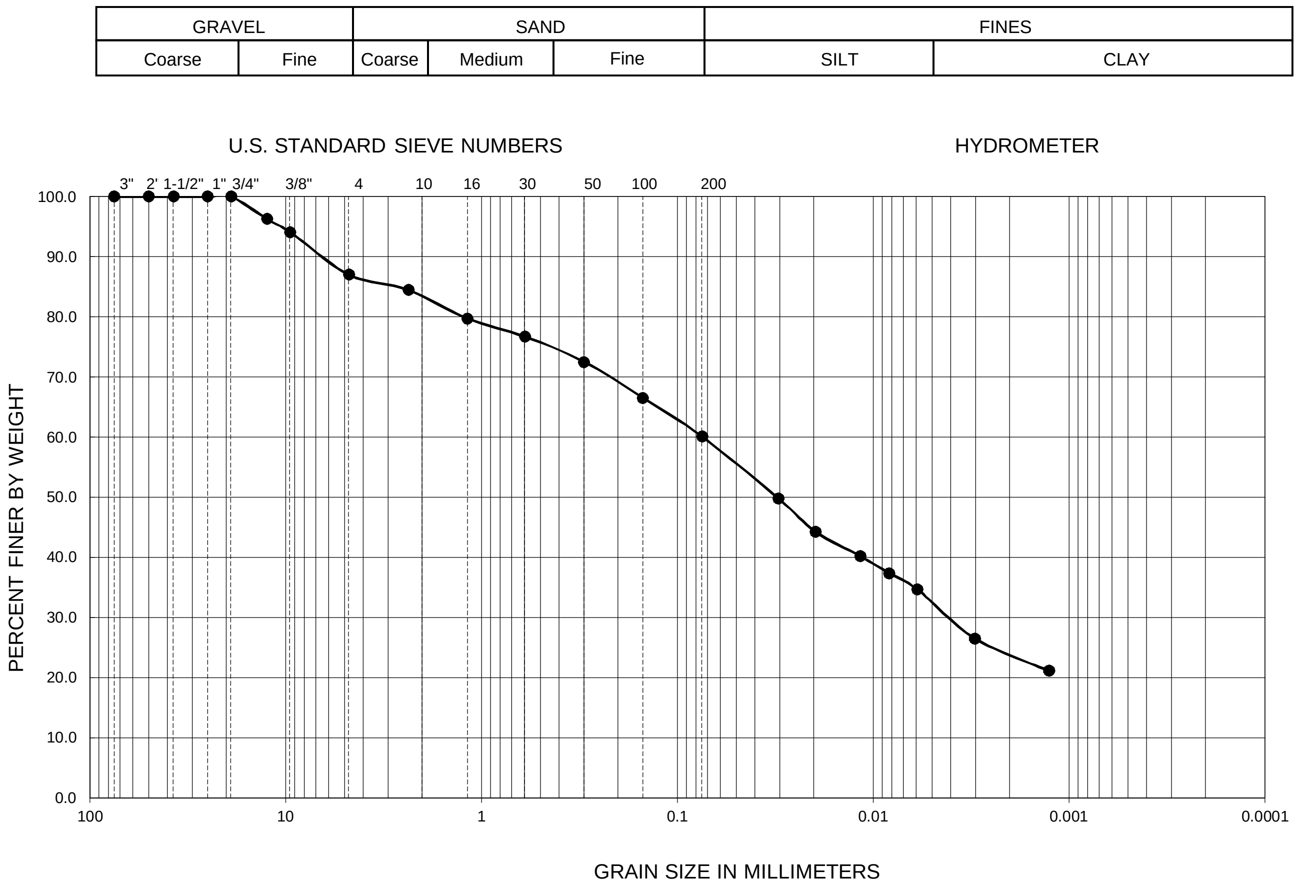
GRADATION TEST RESULTS
MILLS CANYON SEWER ACCESS ROAD REPAIRS
BURLINGAME, CALIFORNIA
403150001 |2/18



Symbol	Sample Location	Depth (ft)	Liquid Limit	Plastic Limit	Plasticity Index	D_{10}	D_{30}	D_{60}	C_u	C_c	Passing No. 200 (%)	USCS
●	B-3	3.0-3.5	37	16	21	-	0.00	0.08	-	-	60	CL

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422

FIGURE B-2

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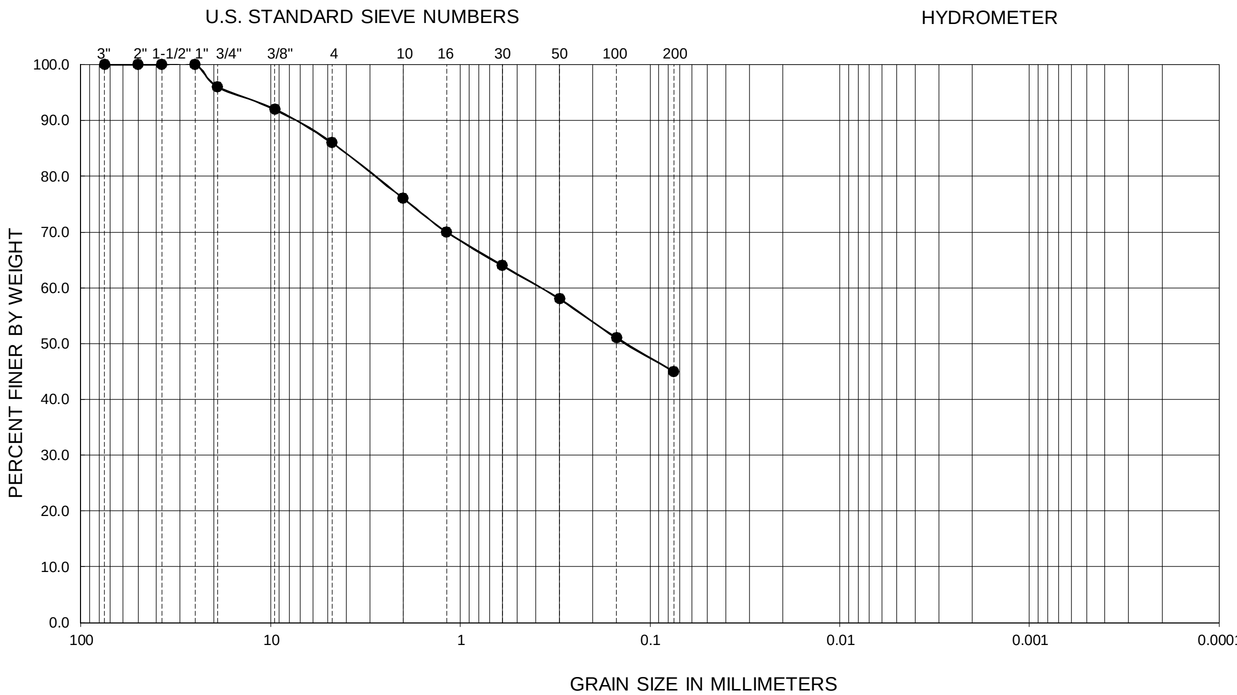
GRADATION TEST RESULTS
MILLS CANYON SEWER ACCESS ROAD REPAIRS
BURLINGAME, CALIFORNIA
403150001 | 2/18

GRAVEL		SAND			FINES	
Coarse	Fine	Coarse	Medium	Fine	SILT	CLAY



Symbol	Sample Location	Depth (ft)	Liquid Limit	Plastic Limit	Plasticity Index	D_{10}	D_{30}	D_{60}	C_u	C_c	Passing No. 200 (percent)	USCS
●	B-4	1.0-5.0	--	--	--	--	--	0.40	--	--	45	SC

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422

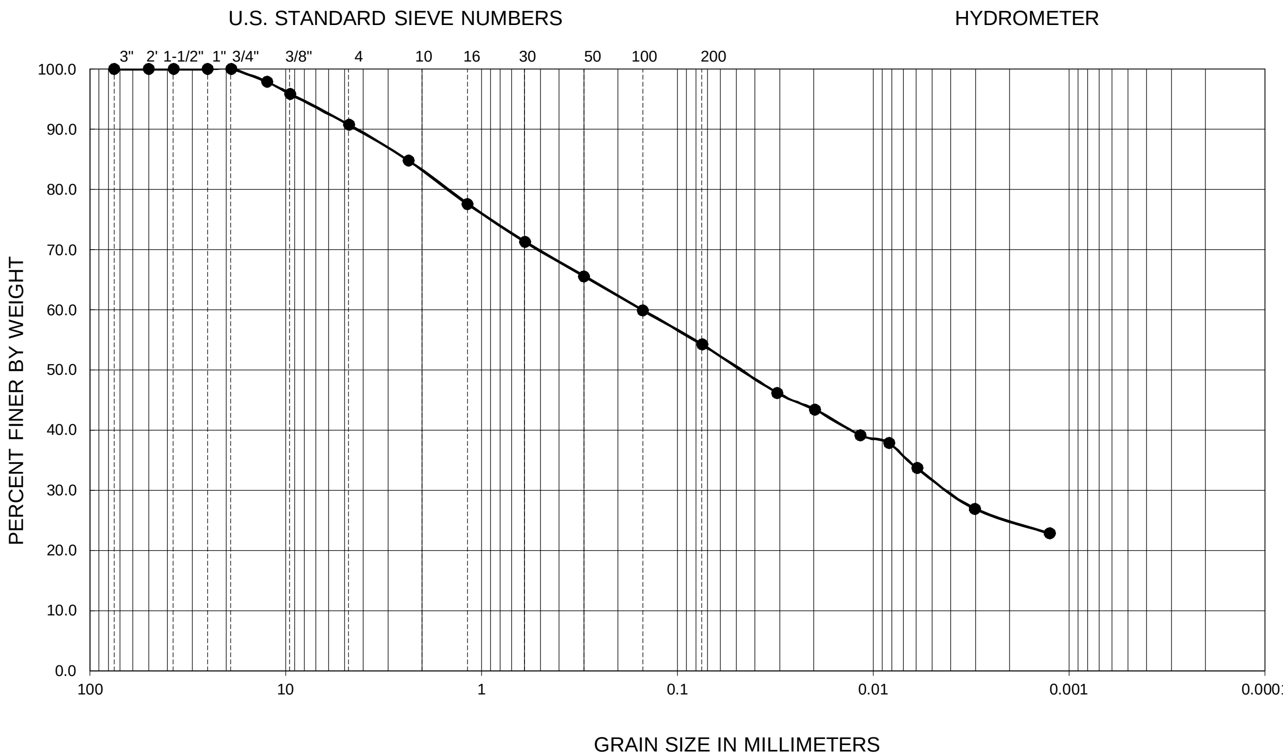
FIGURE B-3

GRADATION TEST RESULTS

MILLS CANYON SEWER ACCESS ROAD REPAIRS
BURLINGAME, CALIFORNIA
403150001 | 2/18

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

GRAVEL		SAND			FINES	
Coarse	Fine	Coarse	Medium	Fine	SILT	CLAY



Symbol	Sample Location	Depth (ft)	Liquid Limit	Plastic Limit	Plasticity Index	D_{10}	D_{30}	D_{60}	C_u	C_c	Passing No. 200 (%)	USCS
●	B-6	3.0-3.5	48	20	28	-	0.00	0.16	-	-	54	CL

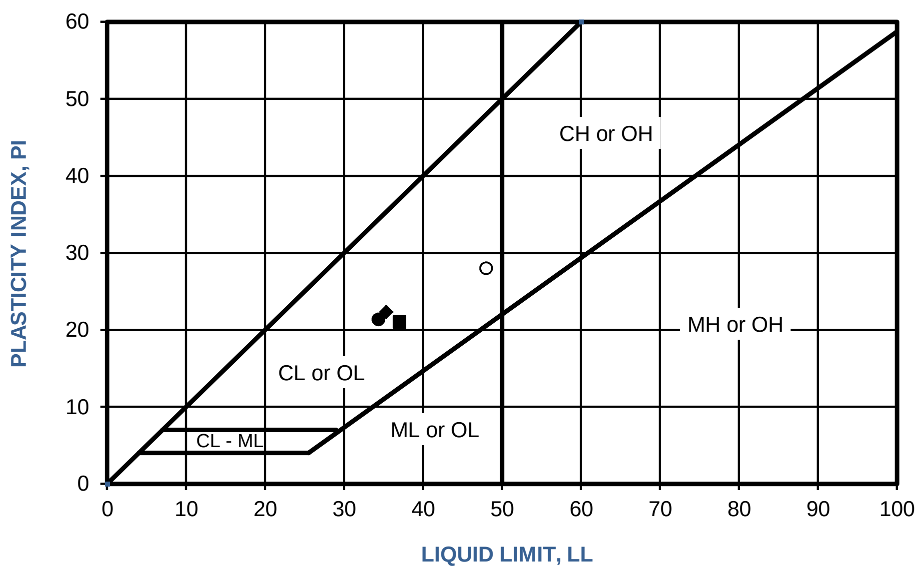
PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422

FIGURE B-4

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

GRADATION TEST RESULTS
MILLS CANYON SEWER ACCESS ROAD REPAIRS
BURLINGAME, CALIFORNIA
403150001 |2/18

SYMBOL	LOCATION	DEPTH (ft)	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX	USCS CLASSIFICATION (Fraction Finer Than No. 40 Sieve)	USCS
●	B-2	4.5-5.0	34	13	21	CL	CL
■	B-3	3.0-3.5	37	16	21	CL	CL
◆	B-3	9.5-10.0	35	13	22	CL	CL
○	B-6	3.0-3.5	48	20	28	CL	CL



PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 4318

FIGURE B-5

Ninyo & Moore
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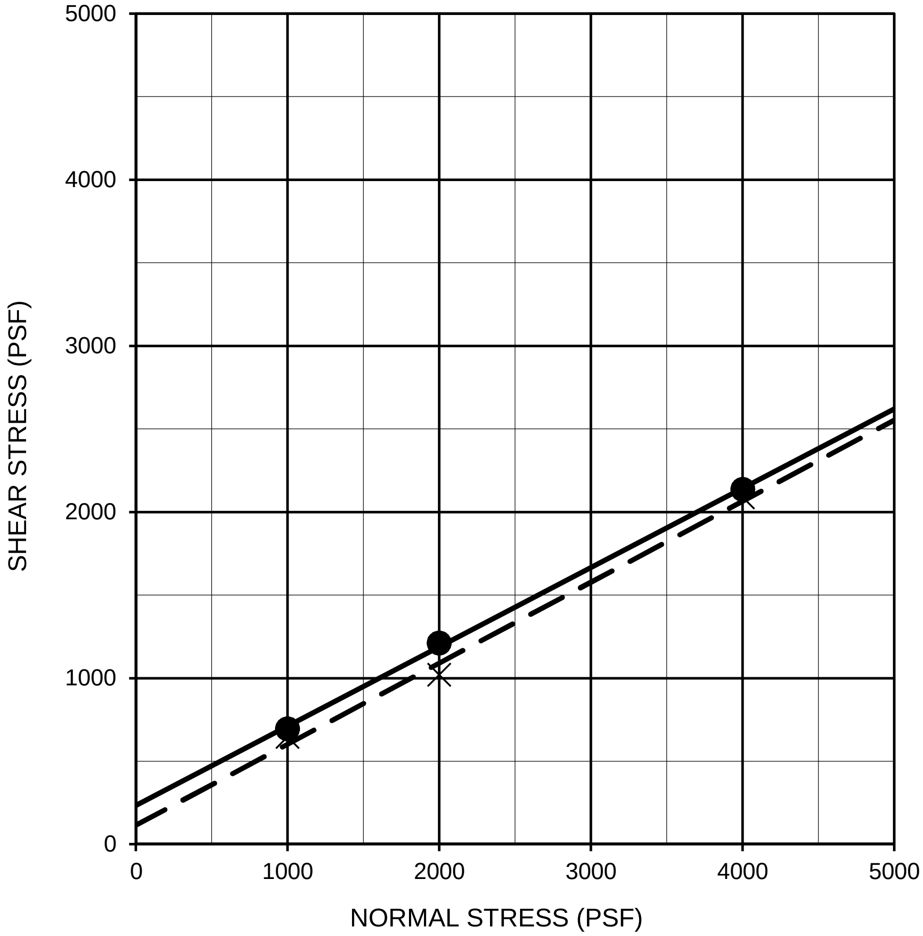
ATTERBERG LIMITS TEST RESULTS

MILLS CANYON SEWER ACCESS ROAD REPAIRS
BURLINGAME, CALIFORNIA



Description	Symbol	Sample Location	Depth (ft)	Shear Strength	Cohesion (psf)	Friction Angle (degrees)	Soil Type
METABASALT	——●——	B-1	9.5-10.0	Peak	230	26	METABASALT
METABASALT	– – X – –	B-1	9.5-10.0	Ultimate	110	26	METABASALT

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 3080

FIGURE B-6

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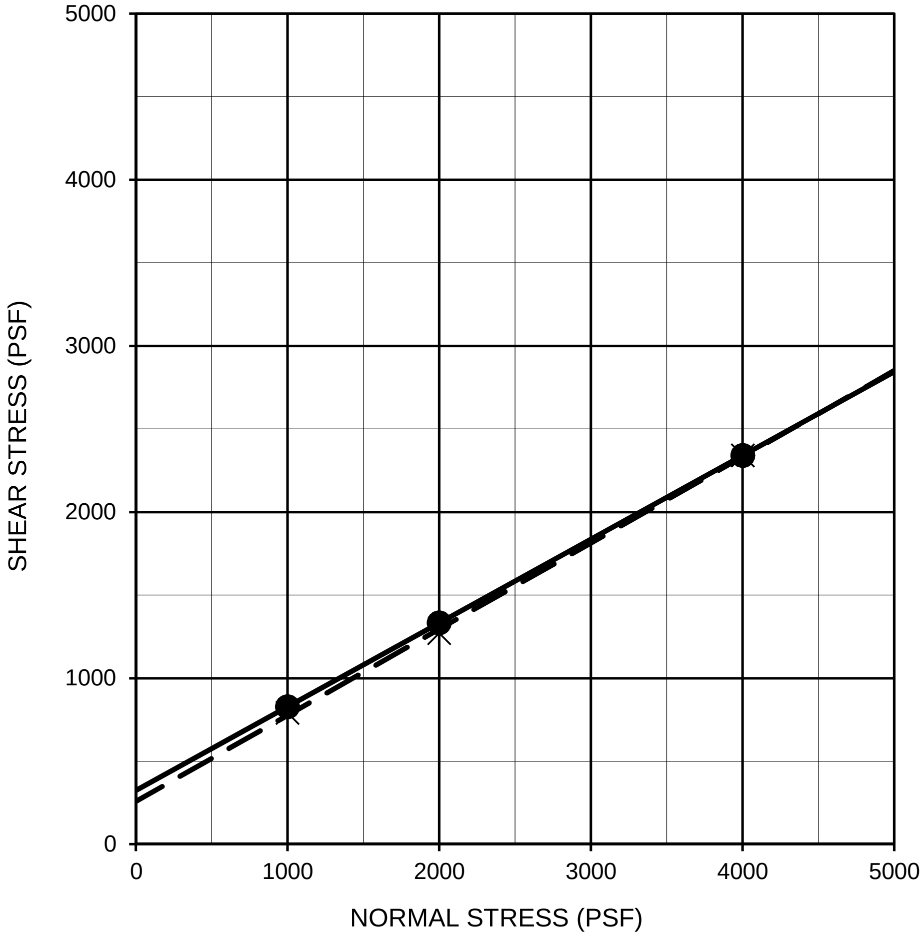
DIRECT SHEAR TEST RESULTS

MILLS CANYON SEWER ACCESS ROAD REPAIRS
BURLINGAME, CALIFORNIA

403150001 |2/18



Description	Symbol	Sample Location	Depth (ft)	Shear Strength	Cohesion (psf)	Friction Angle (degrees)	Soil Type
Lean CLAY	——●——	B-3	4.5-5.0	Peak	320	27	CL
Lean CLAY	– – X – –	B-3	4.5-5.0	Ultimate	260	27	CL

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 3080

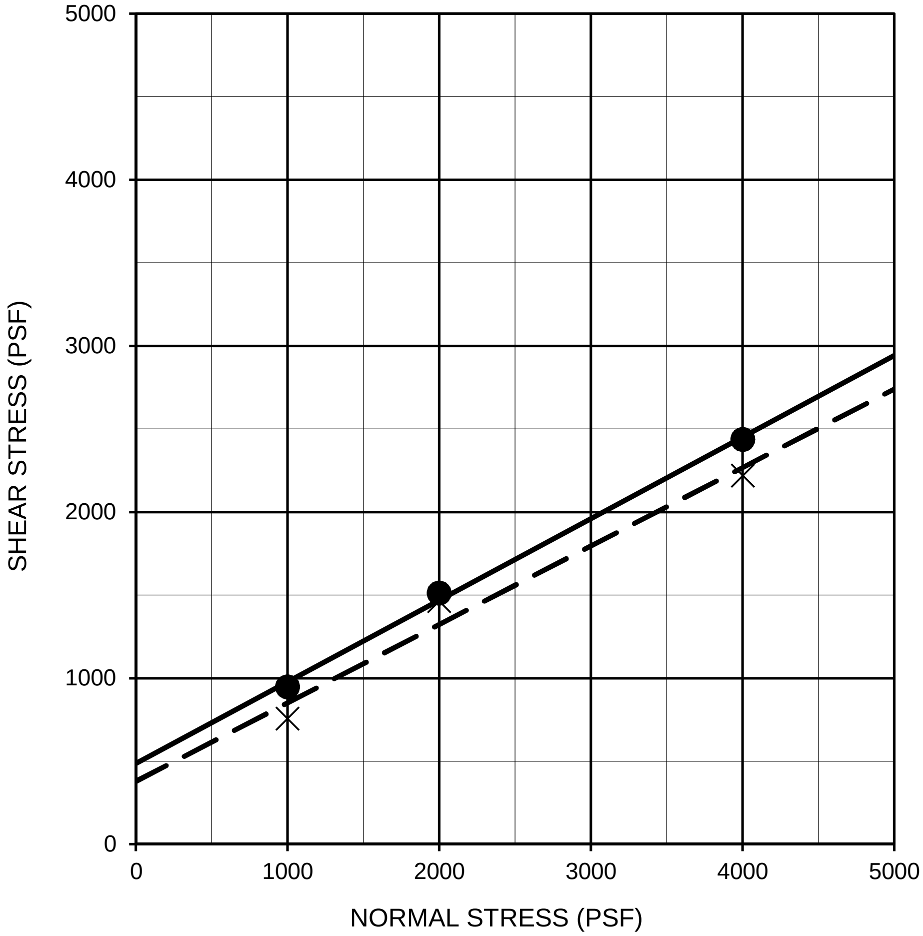
FIGURE B-7

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

DIRECT SHEAR TEST RESULTS

MILLS CANYON SEWER ACCESS ROAD REPAIRS
BURLINGAME, CALIFORNIA

403150001 |2/18



Description	Symbol	Sample Location	Depth (ft)	Shear Strength	Cohesion (psf)	Friction Angle (degrees)	Soil Type
SERPENTINITE	—●—	B-4	9.5-10.0	Peak	490	26	SERPENTINITE
SERPENTINITE	– – X – –	B-4	9.5-10.0	Ultimate	380	25	SERPENTINITE

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 3080

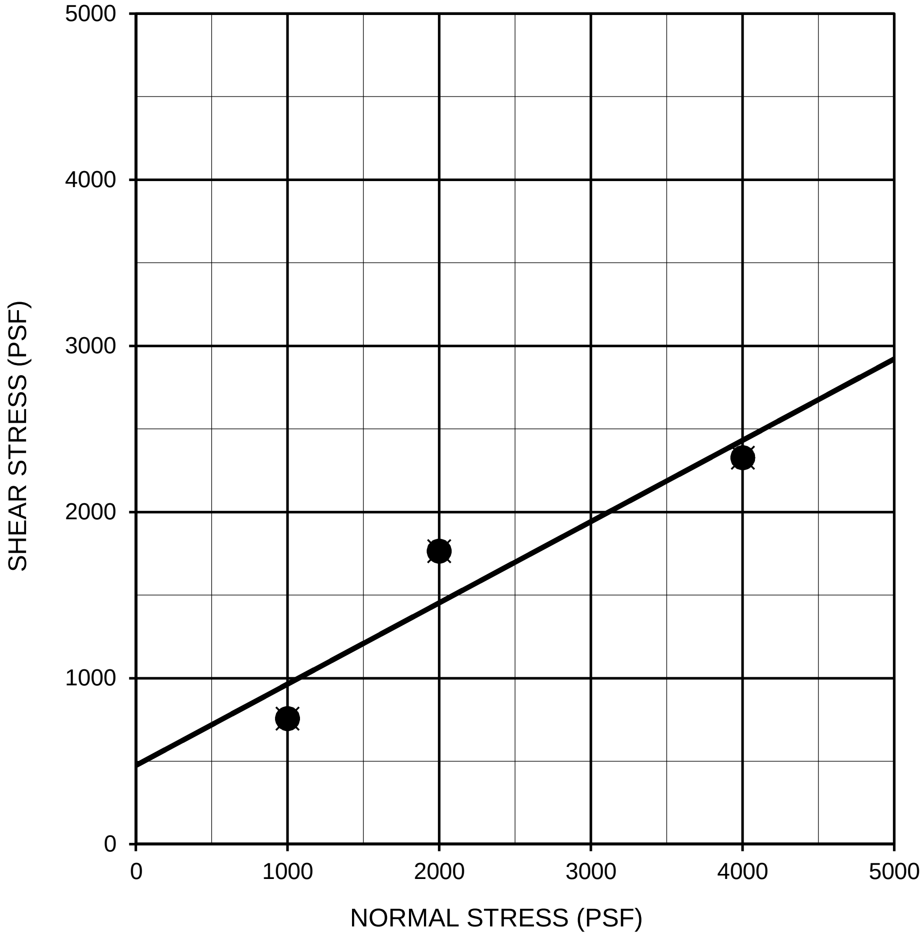
FIGURE B-8

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

DIRECT SHEAR TEST RESULTS

MILLS CANYON SEWER ACCESS ROAD REPAIRS
BURLINGAME, CALIFORNIA

403150001 |2/18



Description	Symbol	Sample Location	Depth (ft)	Shear Strength	Cohesion (psf)	Friction Angle (degrees)	Soil Type
Lean CLAY	——●——	B-6	4.5-5.0	Peak	470	26	CL
Lean CLAY	– – X – –	B-6	4.5-5.0	Ultimate	470	26	CL

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 3080

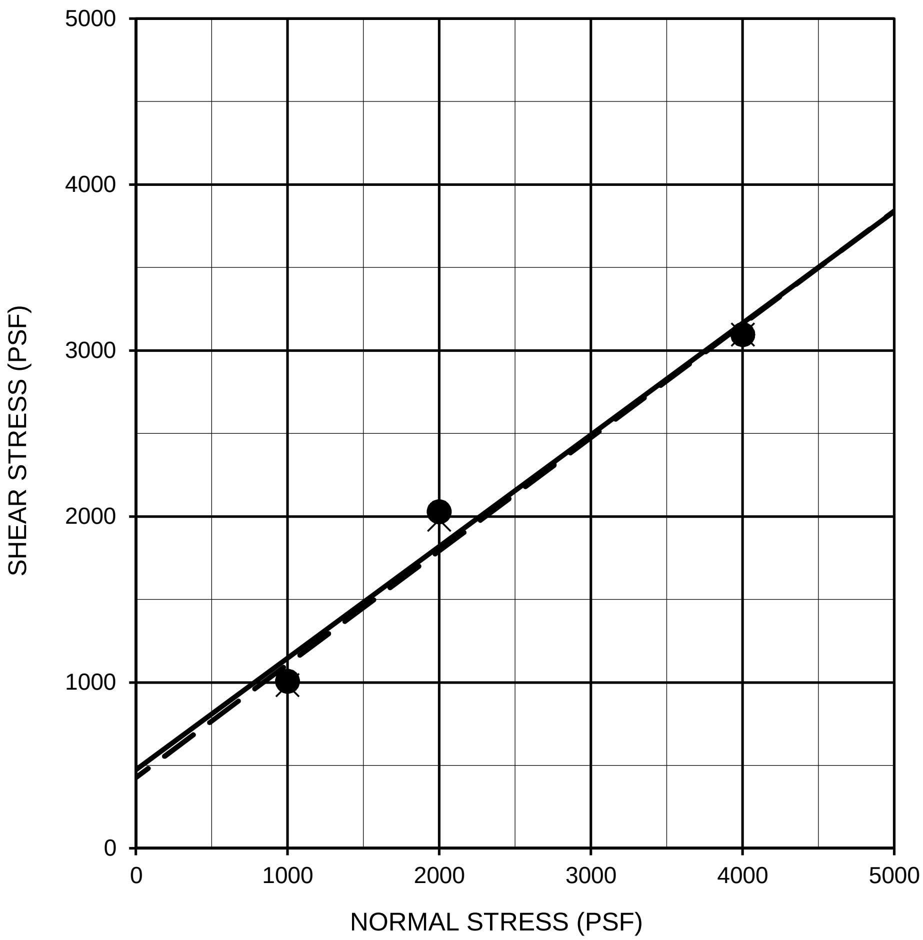
FIGURE B-9

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

DIRECT SHEAR TEST RESULTS

MILLS CANYON SEWER ACCESS ROAD REPAIRS
BURLINGAME, CALIFORNIA

403150001 |2/18



Description	Symbol	Sample Location	Depth (ft)	Shear Strength	Cohesion (psf)	Friction Angle (degrees)	Soil Type
SANDSTONE	——●——	B-6	14.0-14.5	Peak	470	34	SANDSTONE
SANDSTONE	– – X – –	B-6	14.0-14.5	Ultimate	430	34	SANDSTONE

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 3080

FIGURE B-10

DIRECT SHEAR TEST RESULTS

MILLS CANYON SEWER ACCESS ROAD REPAIRS
BURLINGAME, CALIFORNIA

403150001 |2/18

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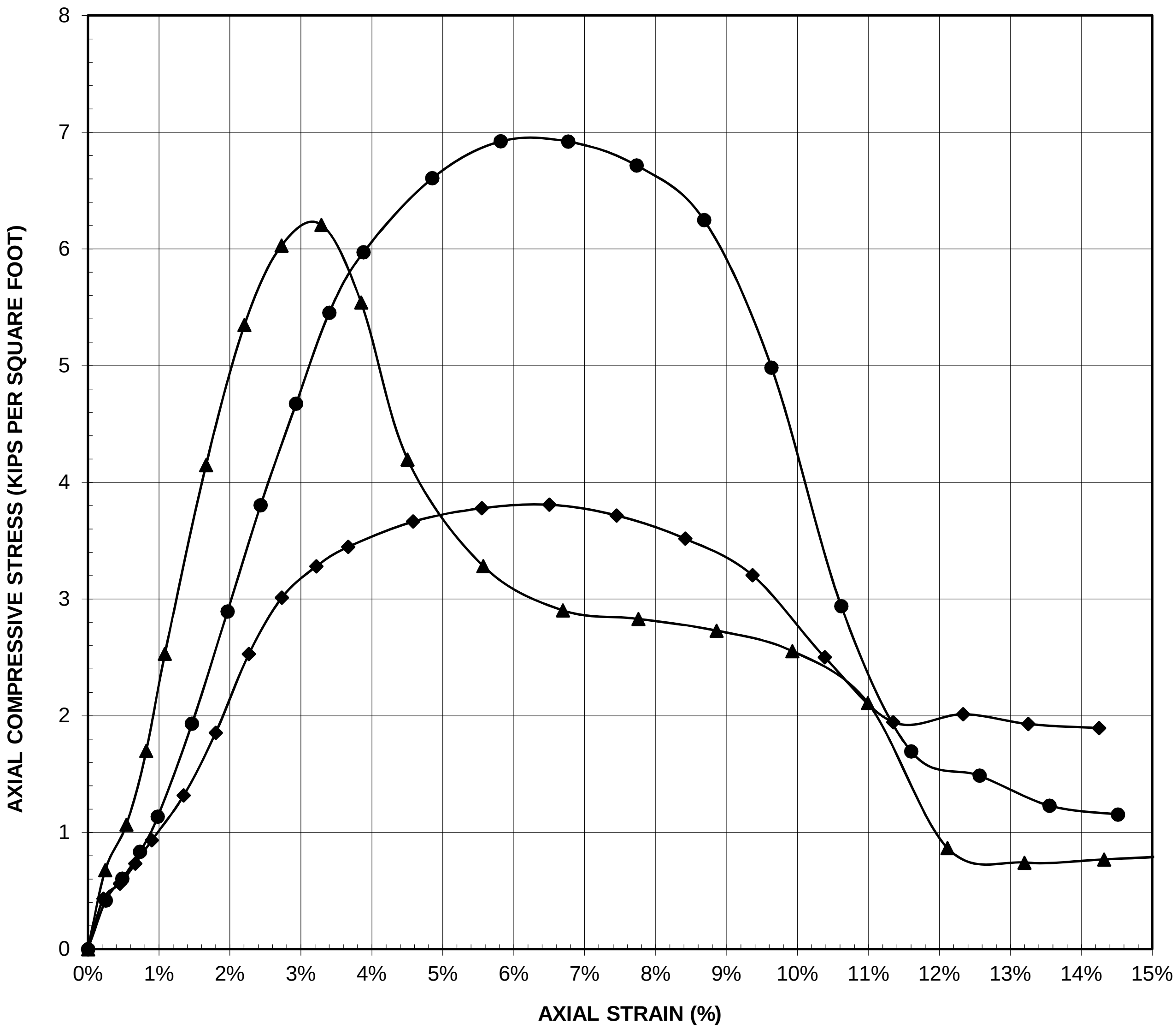

SYMBOL	DESCRIPTION	SOIL TYPE	SAMPLE LOCATION	SAMPLE DEPTH (ft.)	MOISTURE CONTENT w, (%)	DRY DENSITY γ_d, (pcf)	STRAIN RATE (%/min.)	UNDRAINED SHEAR STR s_u, (ksf)
◆	Highly weathered METASHALE and METABASALT		B-2	9.5-10.0	16.5	112.0	0.95	1.90
●	Highly weathered SERPENTINITE		B-3	14.5-15.0	11.3	126.1	0.97	3.46
▲	Weathered SANDSTONE		B-6	14.5-15.0	11.5	126.1	1.10	3.10
■								

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 2166

FIGURE B-11

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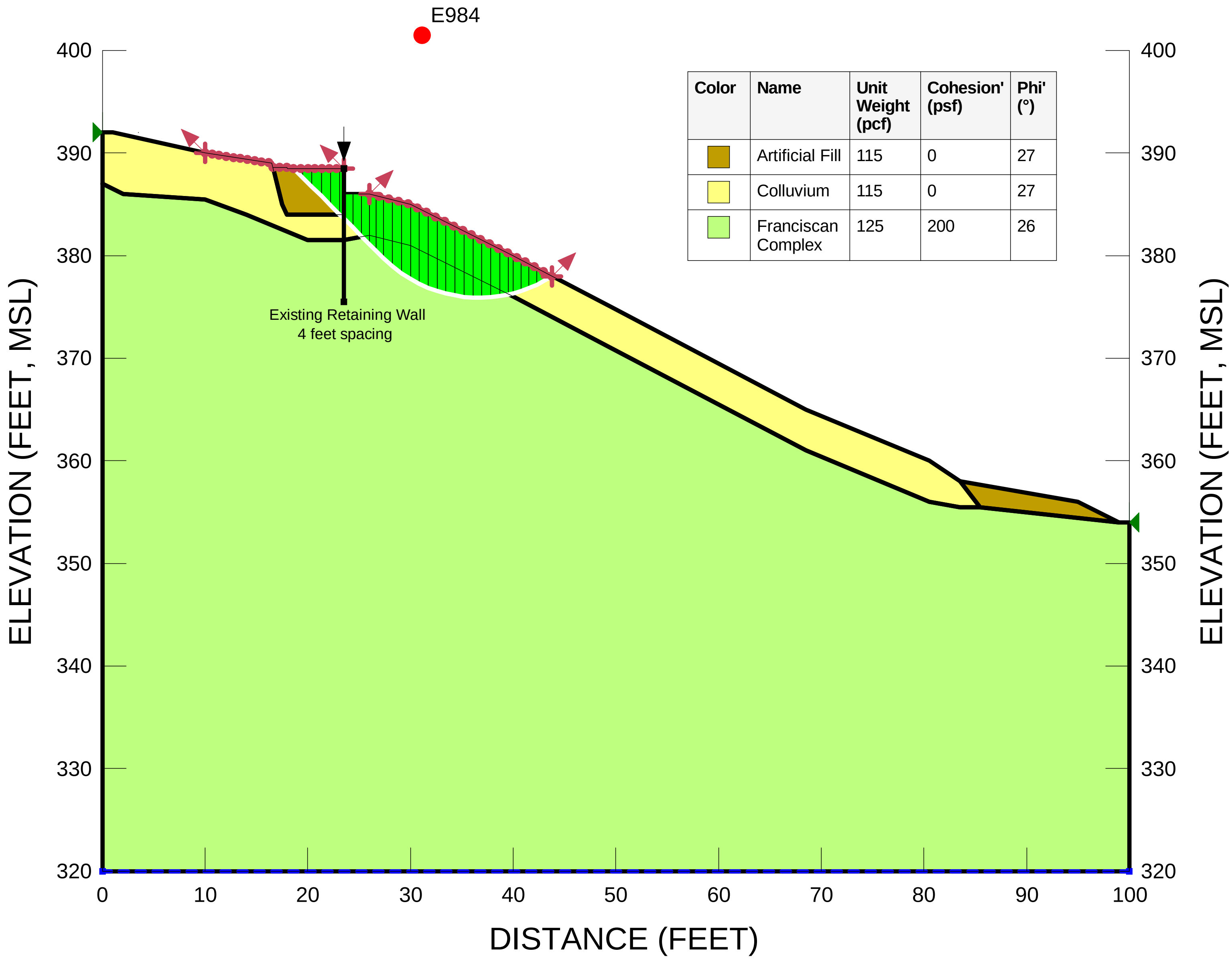
UNCONFINED COMPRESSION RESULTS

MILLS CANYON SEWER ACCESS ROAD REPAIRS
BURLINGAME, CALIFORNIA

APPENDIX C

Calculations

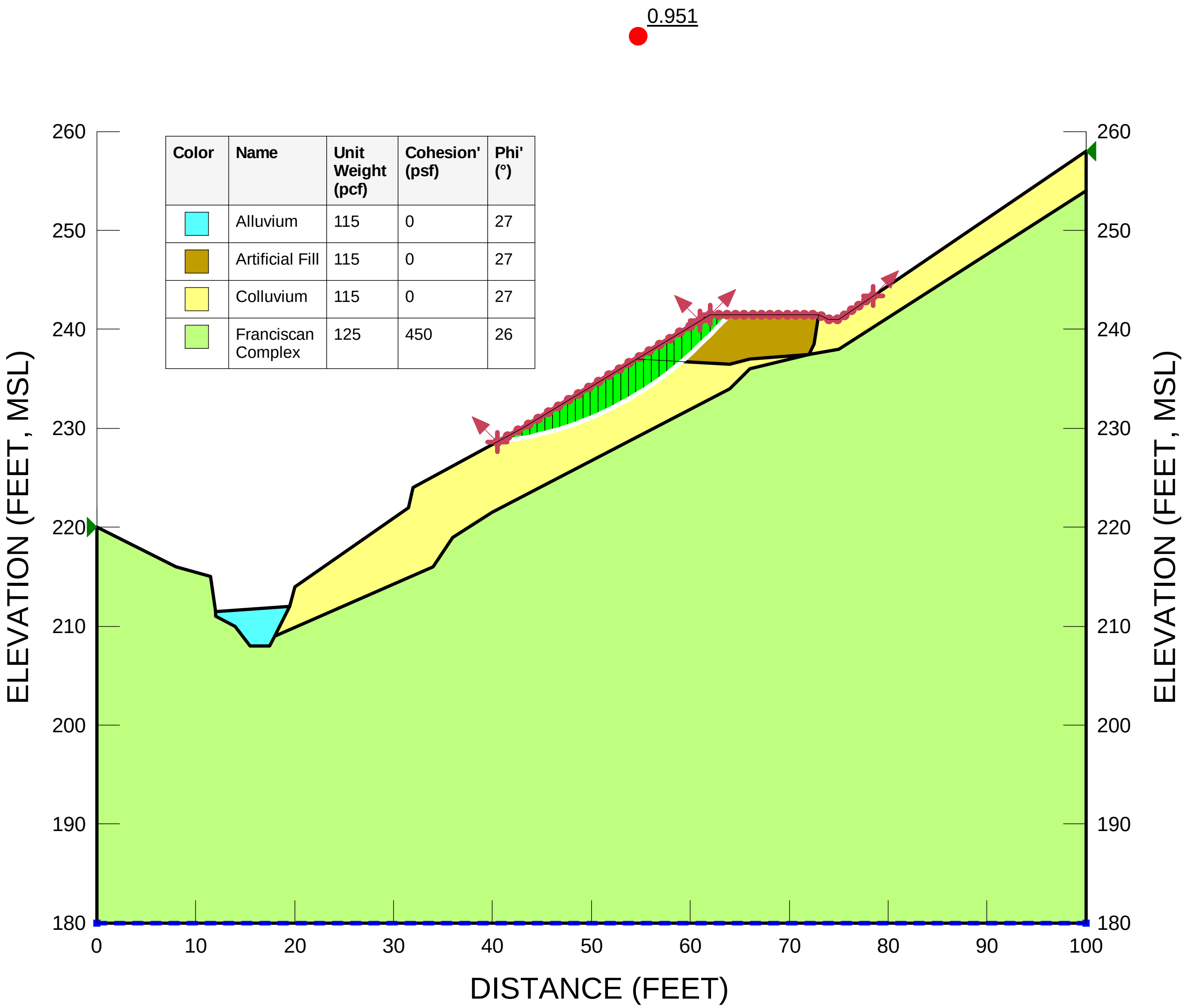


Color	Name	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
	Artificial Fill	115	0	27
	Colluvium	115	0	27
	Franciscan Complex	125	200	26

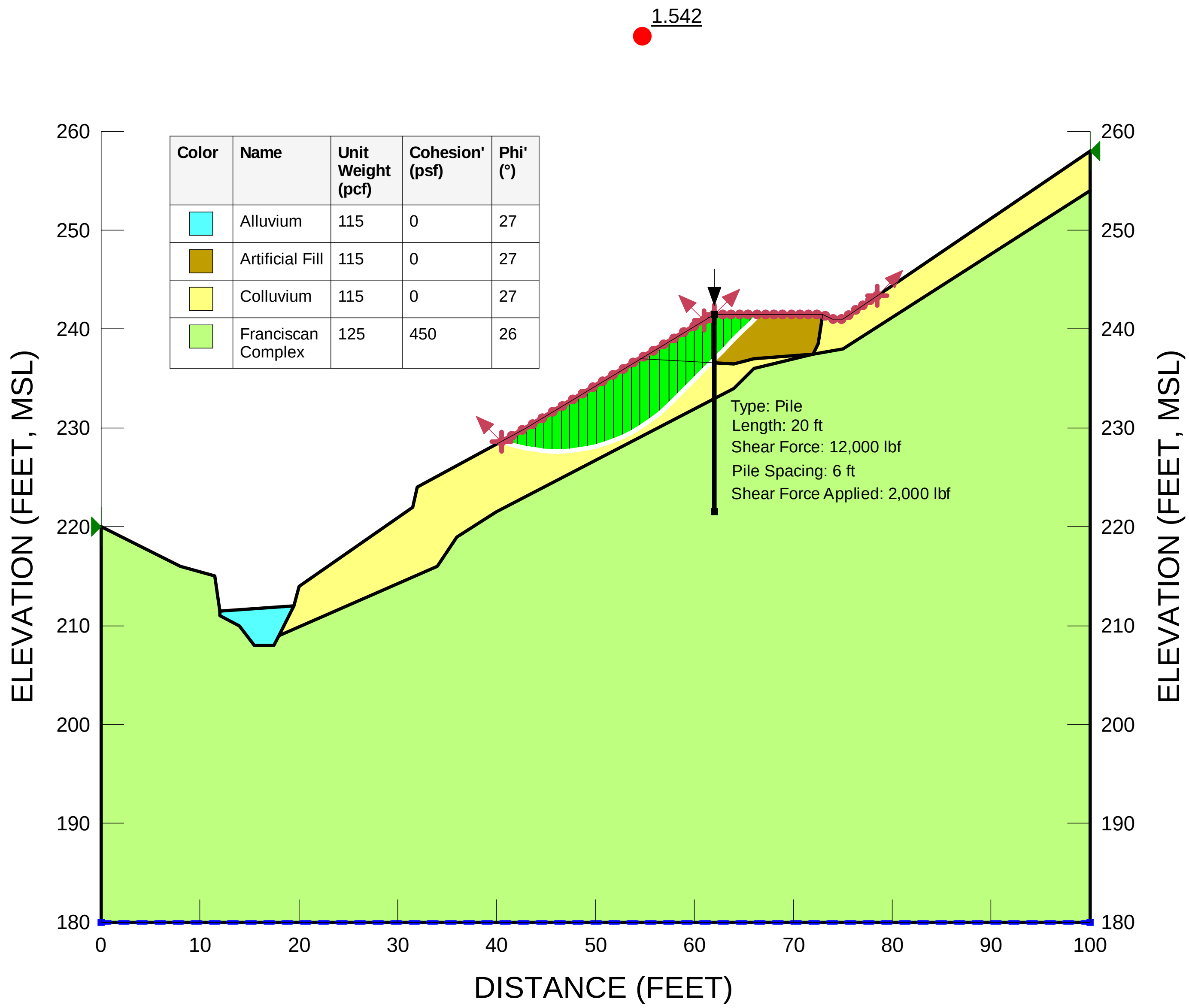
SLOPE STABILITY ANALYSIS (CROSS-SECTION A-A')

MILLS CANYON SEWER ACCESS ROAD REPAIRS

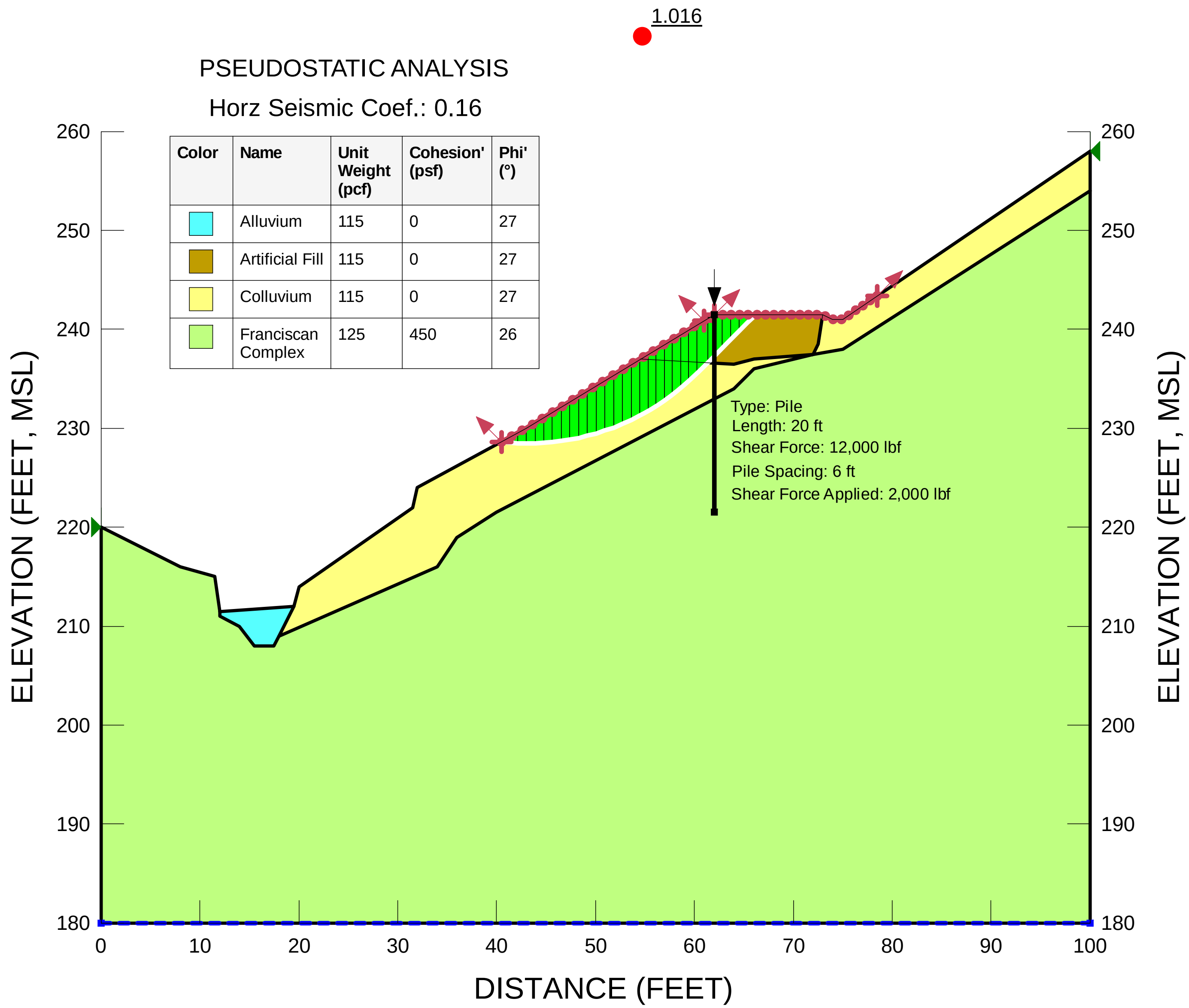
403150001 | 1/18 SCALE: 1:180



SLOPE STABILITY ANALYSIS (CROSS-SECTION B-B')
MILLS CANYON SEWER ACCESS ROAD REPAIRS
403150001 | 1/18 SCALE: 1:200



SLOPE STABILITY ANALYSIS (CROSS-SECTION B-B')	
MILLS CANYON SEWER ACCESS ROAD REPAIRS	
403150001 | 1/18	SCALE: 1:200

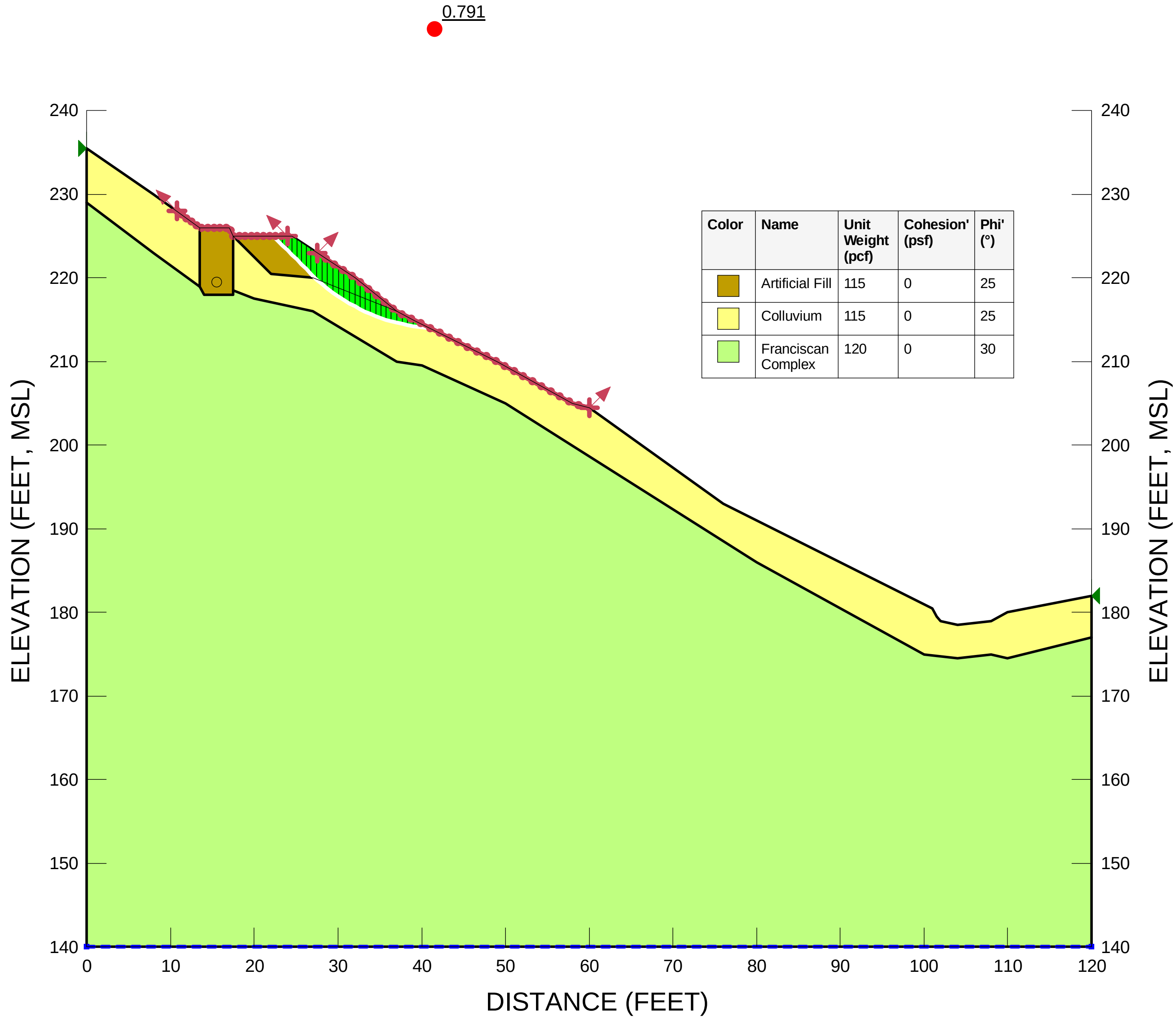


Color	Name	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
	Alluvium	115	0	27
	Artificial Fill	115	0	27
	Colluvium	115	0	27
	Franciscan Complex	125	450	26

SLOPE STABILITY ANALYSIS (CROSS-SECTION B-B')

MILLS CANYON SEWER ACCESS ROAD REPAIRS

403150001 | 1/18 SCALE: 1:200



Color	Name	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
	Artificial Fill	115	0	25
	Colluvium	115	0	25
	Franciscan Complex	120	0	30

SLOPE STABILITY ANALYSIS (CROSS-SECTION C-C')

MILLS CANYON SEWER ACCESS ROAD RAPAIRS

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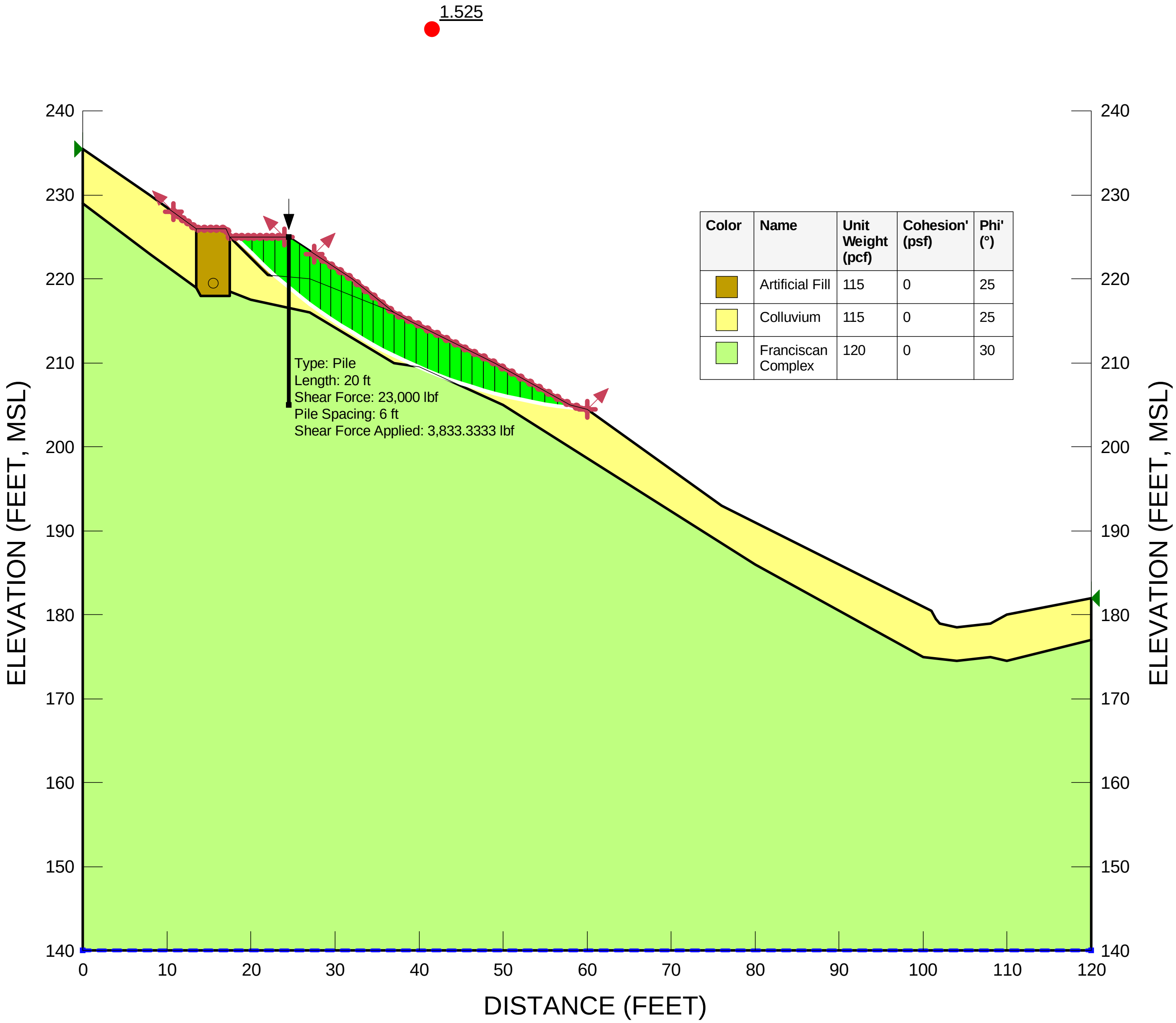
SCALE: 1:230

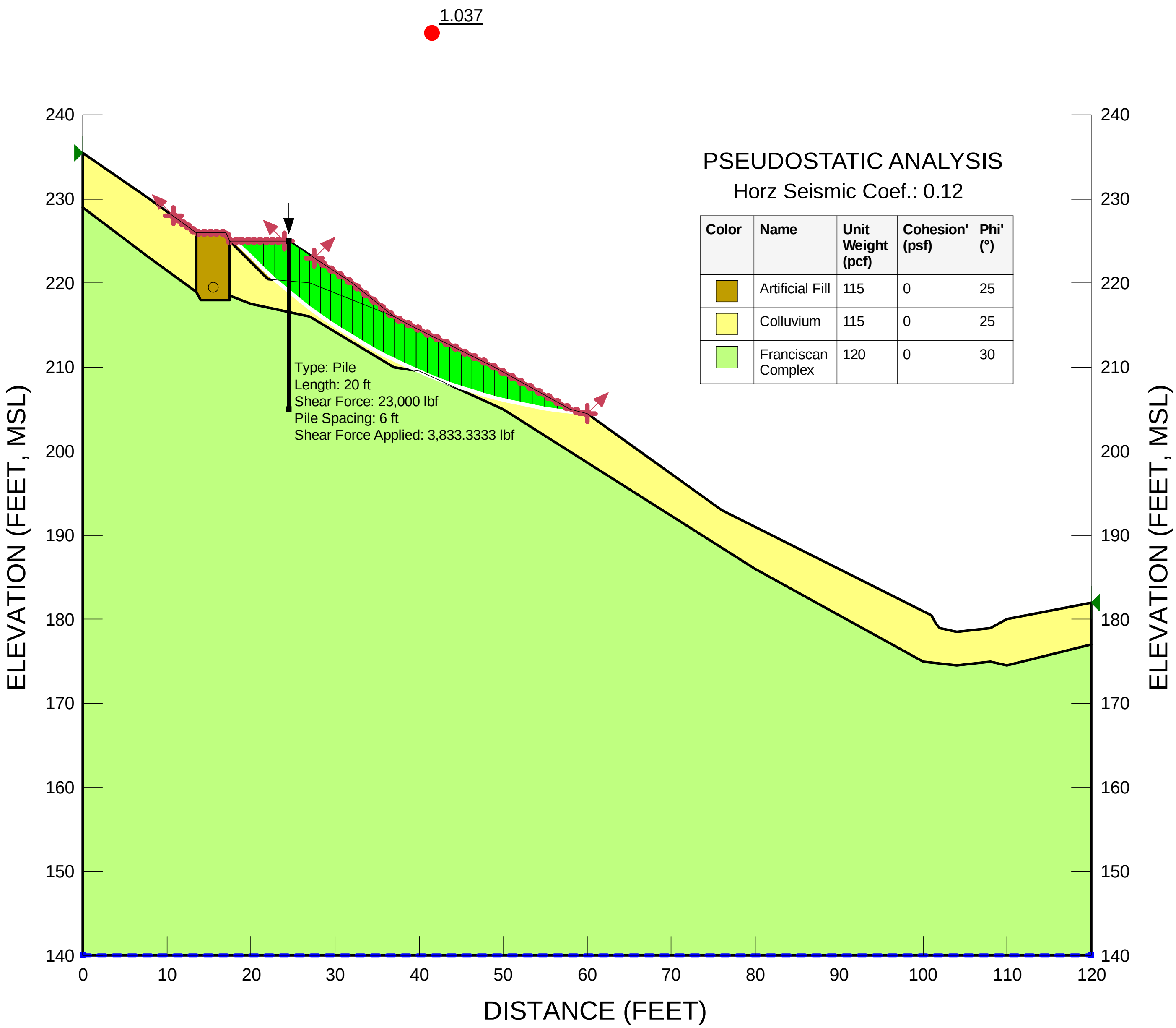


Color	Name	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
	Artificial Fill	115	0	25
	Colluvium	115	0	25
	Franciscan Complex	120	0	30

SLOPE STABILITY ANALYSIS (CROSS-SECTION C-C')

MILLS CANYON SEWER ACCESS ROAD RAPAIRS

403150001 | 1/18 SCALE: 1:230



Color	Name	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
	Artificial Fill	115	0	25
	Colluvium	115	0	25
	Franciscan Complex	120	0	30

SLOPE STABILITY ANALYSIS (CROSS-SECTION C-C')

MILLS CANYON SEWER ACCESS ROAD RAPAIRS

403150001 | 1/18 SCALE: 1:230

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