GEOTECHNICAL REPLY TO PEER REVIEW,
PRELIMINARY CITY STORM DRAIN OUTFALL
SLOPE RETREAT ASSESSMENT,
& GEOTECHNICAL REPORT UPDATE STUDY

Proposed Glenview Terrace Residential Subdivision
2880 San Bruno Avenue
San Bruno, California

Prepared for:

New Shidai Development, LLC
1807 Broadway Street
San Francisco, California 94107
Attn: Mr. Jiang Zhan Lin

February 7, 2020
GEO Project No. 91-04747-A
2172.01.00

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New Shidai Development, LLC
1807 Broadway Street
San Francisco, California 94107

Attention: Mr. Jiang Zhan Lin

RE: GEOTECHNICAL REPLY TO PEER REVIEW,
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2880 San Bruno Avenue
San Bruno, California
GEO #91-04747-A (2172)

Ladies and Gentlemen:

INTRODUCTION

Earth Investigations Consultants, Inc. (EIC), merged with the current Geotechnical Engineer of Record, Geosphere Consultants, Inc. (Geosphere), in 2017. Accordingly, Geosphere has reviewed the EIC 2005, 2008, 2013, 2016 geologic and geotechnical reports prepared for various proposed residential development schemes on this property, and generally adopted their findings, conclusions, and recommendations for the currently proposed version of the Glenview Terrace project. We understand the City of San Bruno (City) Planning Department has provided all but the 2013 EIC geotechnical report to Geocon Consultants, Inc. (GC) for peer review, though listed in the Peer Review references. The 2013 EIC report is attached in Appendix A for GC review, as it represents the project geotechnical report that will be updated with pertinent design information contained in the Geotechnical Update and Supplemental Recommendations provided in Appendix B.

In our opinion, a reviewer of geologic and geotechnical reports treating fault location, relative activity, and appropriate setback for the a reach of the San Francisco Segment of the San Andreas Fault (SFPS) in northern San Mateo County would greatly benefit from, as a minimum, review of Lawson and others (1908, Appendix C), Raymond (1984), Wakabayashi (1990), and Hall and others (2001).

In the spirit of California Geological Survey Special Report 42 (SP-42), we submit Appendix D containing information from two case histories pertaining to proactive, rational administration/implementation habitable building siting in the SFPS, Town of Woodside reach (Plate D1), derived from the necessary cooperative effort between the Planning Department (lead agency) and their Consulting Geologist.

The following section presents our reply to comments delivered in a December 20, 2019, 6-page Geologic and Geotechnical Peer Review letter prepared by Geocon, Inc. (GC, Project No. E9138-04001) for the proposed project consisting of a residential subdivision comprised of 28 detached, 2-story wood-framed single-family homes and associated site infrastructure. This proposed project plan represents an expansion of previous versions of residential development proposed for this property (EIC, 2016).

For peer review clarity of both pertinent site and off-site geologic conditions, we provide supplemental illustrations (Figure 1; Plates 1-4). They are largely based on the project geotechnical auger borings and continuously-sampled geologic percussion soil probes contained in Appendix E (EIC, 2005, 2008, 2013, 2016; Geosphere, 2019). For the same objective, Appendix F contains all of the 2008 site-specific fault exploration trench logs by Romig, and EIC , and nearby off-site trench logs by BAGG (2003, 2007).

The requested geotechnical slope stability analysis report is contained in Appendix G.

On December 3, 2019 we received from the City of San Bruno Planning Department an advisory reporting 30 feet of slope retreat had occurred from a municipal storm drain failure on the steep fill slope adjacent to the west bound lane of San Bruno Avenue, approximately 320 feet east of the Proposed Project area (Appendix H; Plates 2 and 5). At the request of the of Senior Planner, Michael Smith, we conducted a preliminary geologic assessment of the erosion relative to potential impact to the project is found below in section entitled, *Preliminary Assessment of Potential Impact to Proposed Project from Erosion due to Recent Storm Erosion from Failed City Storm Drain Outfall.*

REPLY TO GEOLOGIC & GEOTECHNICAL PEER REVIEW COMMENTS

Reply to Geologic Comments

1. The 1971 magnitude 6.5 San Fernando Earthquake in Southern California resulted in 60 deaths (mainly from collapse of the Sylmar Veterans Administration Hospital), over $500 million in damage, and approximately 48 square miles of fault surface rupture (Proctor and others, 1972). Given the magnitude of earthquake devastation and significate area of fault surface rupture, the State Government moved swiftly to enact the Alquist-Priolo Special Studies Act (ASSAP, 1973) which effectively mandated the State Geologist to expedite preparation of Special Study Zone (SPZ) maps, later named and hereafter referred to as Earthquake Fault Zone (EFZ, 1994) maps. Known and inferred active, and potentially active faults were plotted onto U.S. Geological were plotted onto U.S. Geological Survey 7½-minute topographic base maps generally centered within a 500-foot wide buffer zone defining the respective boundary of the EFZ zone (originally 600 feet wide).

In response to the APSSA (i.e., California Geological Survey Special Report 42 (SP-42, 2018)), jurisdictions adopted the requirement to have geologic investigations to assure construction of any habitable building (human-occupied 2000 hours or more/year) within an EFZ is at least 50 feet from a known active fault trace. Accordingly, in the early 1970's jurisdictions in San Mateo County engaged consulting, in-house geologists, and those qualified geologists in academia, to prepare maps defining active and potentially (suspect) fault zones, which they routinely submitted to the State Geologist for subsequent revision to the initial EFZ map compilations. The current Montara EFZ map was published in 1982. The current Woodside EFZ map was published in 1974.

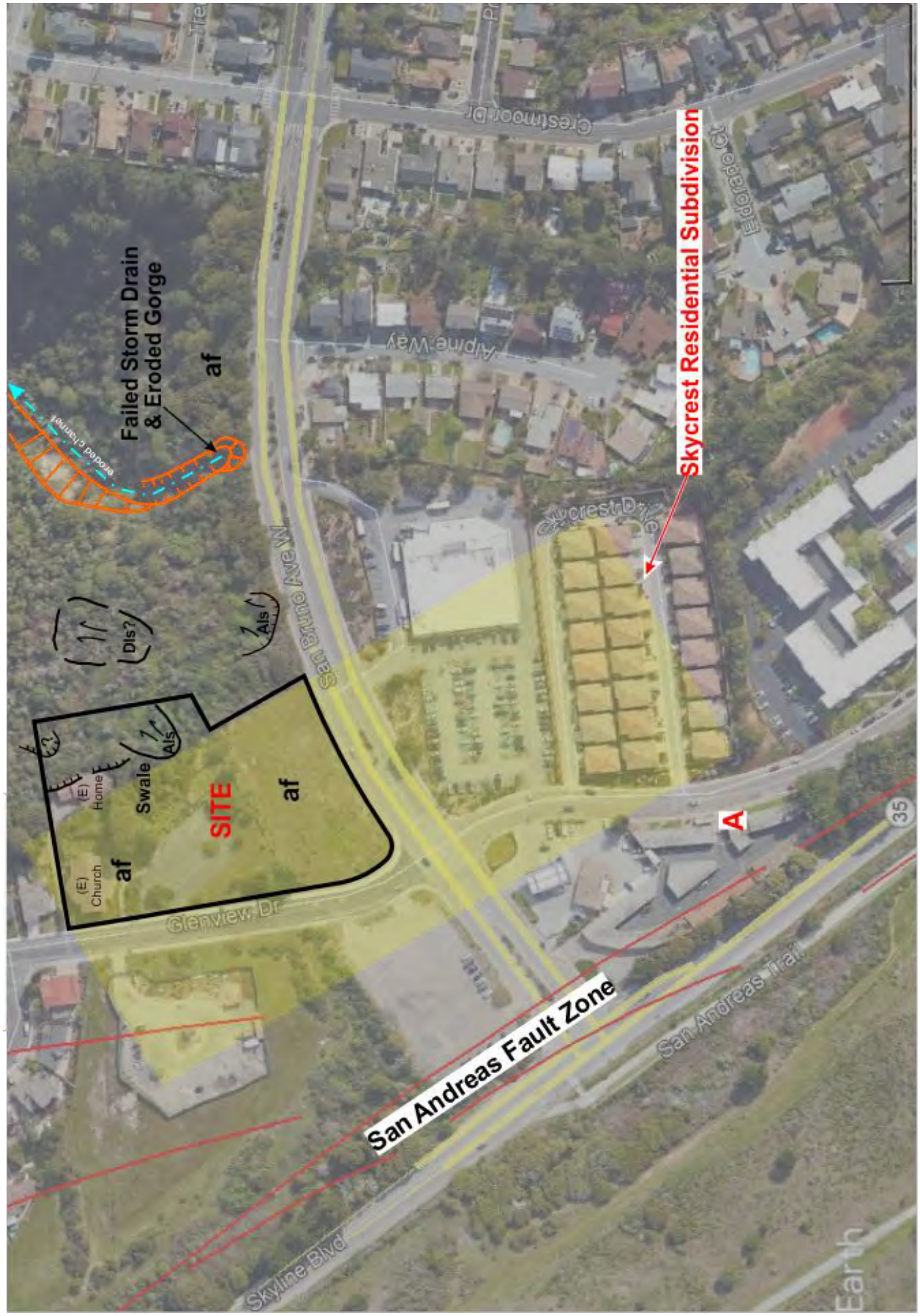

Figure 1. Vertical aerial view of N30°W projection of fault trench shadow projected across Site (yellow shade data from: BAGG, 2003; Romig Engineers, 2008; Google Earth, 2014). Fault rupture hazard clearance was approved by the City of San Bruno in the late 1980's for development for the Arco Self Storage facility office building (**A**). (**af** - Artificial Fill) (Fill distribution and landslides after Cotton, Shires, & Associates, Inc. , March 2008).

At least seven decades after the head-start provided by Lawson and others (1908) to the understanding of 1906 earthquake ground rupture from their invaluable observations, mapping, and detailed descriptions of highly perishable manifestations of surface rupture on the San Francisco Peninsula Segment of the San Andreas Fault (SFPS) shortly after the event, local lead agency planning departments, such as the Town of Woodside, Town of Portola Valley (Dickenson, 1970, 1973), and the County of San Mateo (F. Beach Leighton and Associates, 1971; William Cotton and Associates, 1980), began producing expedited geologic hazard maps showing traces of active and potentially active faults.

The basis for much of the mapping is based solely upon limited geologic site reconnaissance and photogeologic interpretations of new and evolving geomorphic evidence (e.g., linear and aligned landscape features of any kind sympathetic to the northwest trend of the of the SFPS) considered to be surface manifestations of late Quaternary fault ground deformation from recurrent vertical and right-lateral strike fault offsets (Wallace, 1990). For example, the SFPS in northern Woodside was derived from limited and often weak field and photogeologic evidence for defining the queried, continuous 1906 ground rupture. In contrast, actual mapping of 1906 rupture trace shortly after the earthquake (Lawson and others, 1908), defines the known SFPS just west of the proposed project area.

Dickenson's (1973) fault hazard mapping, apparently produced in less than a month's time period, was found to have been developed on the basis of weak, remotely-sensed geomorphic evidence that created an 800-foot wide EFZ comprised two divergent EFZ's containing potentially active faults mapped with variable degree of confidence (Appendix D; California Division of Mines and Geology, 1974). The EFZ on the northeast margin of the rift valley also contained faults located with various degree of certainty, including the queried trace of the 1906 rupture.

A geologic investigation by EIC (1994) that encountered the historic 1906 rupture approximately 250 feet from the Dickenson's queried trace, effectively created a rapid evolution of fault hazard investigation siting and scope as well as modification of potential fault surface rupture hazard setback criteria by the Town Geologist (Appendix D, Town Geologist review letter dated 11/21/2000). Further, in a 2012 fault investigation for habitable building setback containing a compilation of previous paleoseismic studies, EIC enabled the Town Geologist, under the auspices of Town of Woodside Planning Department, to effectively resolve the active SFPS in northern Woodside to a single trace associated with 1906 surface rupture as depicted on the Town Geologic and Geologic Hazard Maps (Cotton, Shires and Associates, 2012, 2017).

The case histories described above were instrumental in forming justification not only for revision of the Woodside Geologic Map but as importantly for inducing rational and progressive setback in the spirit of SP-42. It proves the essence of lead agency administration of SP-42 comes from engaging a qualified consulting geologist or in-house geologist with sufficient local knowledge of the geologic setting to implement habitable fault setback criteria (e.g., Woodside Town Geologist's Geomatrix Consultants and Cotton, Shires, and Associates, Inc.). Woodside's fault setback policies pertaining to habitable development in EFZ, in many cases, have greatly mitigated delays and the significant expense to building permit applicants associated with excessive trenching beyond a known Holocene or historically active fault, as has been recommended by GC. For example, current Town of Woodside habitable building setbacks are: 50 feet from a known active fault trace, and 125 feet from an inferred active fault trace.

This confidence in setback assignment for the SFPS has been largely derived from the founding work by Lawson and others (1908) followed by the many others referenced in this reply.
It is essential to recognize the standard of practice procedure for evaluation of potential fault rupture hazard setback has been highly influenced by projection of fault shadows derived from nearby paleoseismic investigations (e.g., Plates 2 and 3)**.**

APSSA, EFZ maps are an important planning guide for habitable building construction in potentially active fault zones where surface rupture could occur. It is important to keep in mind that the width of the respective EFZ maps are plotted with respect to what commonly is a significant buffer zone of 500 feet on either side of a potential active fault trace from known or potentially active faults. The spirit of SP-42 is to effectively maintain the mandate for safe habitable setbacks in California through consistent application of the SP-42 guidelines based on inclusion of current bodies of local geologic knowledge that, for example, in the Proposed Project area, indicate a 50-foot setback from the nearest mapped 1906 ground rupture is sufficient because of the narrow, straight fault trace that describes it, and it has been found to represent the locus for ground rupture during major earthquakes over all of Holocene time (Lawson, 1908; Solomon and Bahr, 1982; Pampeyan, 1981, 1986, 1995; Hall, 1984; and Hall and others, 2001). Similarly, if a habitable building is proposed within 125 feet of an inferred fault within the SPSA, which doesn't apply in the Proposed Project area, subsurface exploration should be considered.

2. The distinct northwest tectonic imprint expressed in the site area landscape and rock is a consequence of more than 140 million years of uninterrupted, east-dipping subduction that produced Franciscan Assemblage including high pressure-low temperature metamorphosed rock, and mélanges comprised of utterly chaotic geologic units in sheared matrix with a variety of included blocks (Hsu, 1969; Raymond, 1984; Wakabayashi, 1999). Occurrence of generally deformed clay seams having variable orientations within the Juro-Cretaceous mélange has been interpreted as deep ocean, soft sediment deformation prior to lithification in the, and/or high-subduction pressure forcing intrusion of plastic, if not fluid, black often sheared shale matrix containing rock fragments plucked from the adjoining county (Wakabayashi, 1999). To complicate the geologic setting, the mélange has been overprinted over the past 20 million years by high-angle east dipping northwest trending deformation associated with right-lateral northwest trending strike slip faulting on the SFPS that has also produced pervasive, variably-spaced, high angle northeast and southwest dipping clay seams associated with shearing in the active faulting zone, and also upwelling of clay gouge in the adjoining landscape by shifting of the fault (Hall, 1984).

The 1906 trace of the SFPS is at the base of aligned linear ridges that describe the east side of the relatively straight rift valley, expressed as aligned N30°-35°W ridges marked on the west side by other tectonic geomorphic features (Pampeyan, 1995).

The Proposed Project occupies the eastern side of one of the rounded, northwest-trending linear ridges that was mass-graded in the early-middle 1950's. Evidence of geomorphic evidence of faulting across the Proposed Project was absent on the basis of photogeologic interpretation of 1949 historic topographic mapping and 1943 aerial photographs interpreted for this property (Plate 1). Aside from the fault parallel orientation of the ridge itself, linear fault-parallel geomorphic features are absent. Instead, linear subparallel northeast-tending structurally-controlled seasonal drainage tributary San Bruno Creek features mark the east flank of the ridge.

Given literature treating mélange origin, age, and structure (i.e., Wakabayashi, 1991; Raymond, 1984), Romig's attempt to establish late Quaternary fault movement of the 14-foot wide shear zone with bounding, high-angle northwest trending clay seams observed in exploratory trench RT1 on the basis of *"relative shearing and weathering of the bedrock (sheared mélange)"* is misleading and unsupported. Nevertheless, Romig's confirmatory trenches RT2 and RT3, respectively located across strike of the shear zone approximately 40 and 80 feet projected northwestward from the RT1 exposure, indicate the sheared mélange and bounding clay seams are discontinuous (Appendix F). Moreover, there was no reported evidence of offset to the mélange contact or the overlying colluvium exposed in TR2 dated as 130,000 years old and occurring as a northeast trending swale cut into mélange with a mapped surface exposure in the graded headwaters in the middle of the Proposed Project (Plates 2 and 3).

Mélange containing a vertical zone of what EIC interpreted as subduction zone flow deformation of intruded clay or shale matrix exposed in EIC T-1 was not evidence of Holocene faulting for the same reasons described in the above paragraph (Appendix F). In our opinion, their discontinuous extent, and absence of geologic contact offset indicate they represent intruded, unsheared, high-angle, northwest and northeast-trending, wavy, pinch-swell clay–filled rock fabric derived from subduction zone. EIC's T-2, excavated in colluvium adjacent to RT2, was important because it exposed 130,000 year-old colluvium overlying mélange and occurring over a large area of the Proposed Project based upon the boring data (Plate 3). The colluvium was represented by several feet of soil profile development containing a dark brown Bt horizon with extensive clay film development. None of the clay seams in the mélange offset the base of the colluvium, proving that the clay seams had formed more than 130,000 years ago. It is well-known among Coast-Range geologists that clay seams are common in mélange as product of subduction, and are not considered evidence for Holocene surface fault rupture. The presence of similar mélange clay seams in areas in which formerly natural surface soils have been removed should not be mistaken as evidence for Holocene fault activity.

3. When taken as a whole, findings from the EIC investigations substantially demonstrate the Proposed Project is and will continue to be exposed to a low risk for future surface fault rupture from major earthquakes. EIC arrived at this conclusion in accordance with the prevailing standard of practice for fault surface rupture hazard on the northern reach of the SFPS. The site is at least 260 feet from the 1906 rupture trace and approximately 160 feet from the minor, eastern-most branch fault, and therefore satisfies the mandate of SP-42 for a habitable building setback of at least 50 feet. Absence of faulting determined by site-specific fault trenching in more than 140,000,000 year-old mélange terrane mantled by 130,000 year-old colluvium coupled with fault shadowing from nearby trenching within the past 17 years for now-developed, habitable building area, cover all but an approximately 5-foot gap between the eastern EFZ boundary and site-specific trenching (Figure 1; Plates 2 and 3). Thus, the recommendation of supplemental trenching is unwarranted.

Reply to Geotechnical Comments

1. That the 2013 geotechnical report was omitted from the submitted geologic and geotechnical documentation is unfortunate; however, GC references the report in their peer review letter as an apparent source of site information. For reference, a copy of that report is contained in Appendix A with supplemental recommendations in Appendix B for GC supplemental review.

In order to address GC comments concerning a lack of previous slope stability analyses performed on the eastern margin of the site, we performed three slope stability analyses at locations on the eastern margin of the proposed development area deemed critical based on past and existing performance of the steep eastern slope that descends approximately 100 feet to the entrenched upper reach of San Bruno Creek (Appendix G; Plate 5; Plate 2). The slope is underlain by Franciscan mélange which was found to have performed satisfactorily for support the dam foundations at San Andreas and Crystal Springs Reservoirs southeast of the site, in-spite of horizontal offset of up to 10 feet, with a component of vertical offset, from surface ground rupture during on the 1906 earthquake on the San Francisco Peninsula Segment of the San Andreas Fault, as determined from geologic and geotechnical seismic safety studies (Solomon and Bahr, 1982; Hall, 1984; Pampeyan, 1983, 1986; United States Committee on large Dams (USCOLD, 1992).

The results of the stability analyses, as summarized in Appendix G, show that under existing site conditions under static loading, the eastern slope had computed Factors of Safety (FoS) ranging from 1.4 to 3.9 between the north and south ends of the property, respectively. The lower (1.4) value was obtained at the location of Cross Section X-X'; however, this value represented the area downslope of the existing residence, outside of the limits of the proposed development property. Theoretical failure surfaces intersecting within the limits of the property all had FoS exceeding 1.5. Under design event seismic loading, FoS ranged between 0.7 and 1.3, suggesting that current stability of the northern half of the eastern slope may be marginal under seismic loading conditions, with seismic FoS values under 1.1 calculated at both Sections X-X' and Y-Y'.

When analyzed for the proposed new grading configuration, similar FoS values were obtained at each analyzed cross section, and with marginal slope stability calculated at only Section X-X', where potential seismic failure surfaces were calculated downslope of, but reaching near the property boundary. Therefore, in order to mitigate potential retrogression of seismically-induced slope failures into the limits of the property at the northeastern end of the development, we recommend adding a stitch pier system along the property line downslope of the new residential structures at this location. Assuming this stitch pier system is added to the property boundary at this location, and grading below the new residences along the top of the east boundary slope is accomplished in accordance with our geotechnical recommendations, we conclude the project is feasible from a perimeter slope stability standpoint. Recommendations for site grading as well as design of the stitch pier system are presented in Appendix B.

2. We discussed with the Civil Engineer our concern over high potential for deep instability from percolation of storm water on steep slopes on the northeast and southeast corners of the project area currently experiencing local slope instability. Pertinent geotechnical recommendations for storm drainage disposal are presented in Appendix B.

3. EIC's 2013 report has been reviewed by Geosphere and confirmed to represent a comprehensive geotechnical report containing appropriate conclusions pertaining to project feasibility, and valid design-level geotechnical recommendations for use in project Civil and Structural design. Current seismic parameters and other pertinent data to update the report are presented in Appendix B.

4. The prevailing standard of practice in San Mateo County mandates the project geotechnical consultant of record review the project plan set to verify plans have been prepared in general conformance with the project geotechnical report recommendations. Our experience with permitting in the City of San Bruno Building Department is a plan approval letter from the geotechnical engineer of record is required for geotechnical clearance leading to issuance of a building permit.

PRELIMINARY ASSESSMENT OF POTENTIAL IMPACT TO PROPOSED PROJECT FROM EROSION DUE TO RECENT STORM EROSION FROM FAILED CITY STORM DRAIN OUTFALL

Introduction

The task of undertaking a preliminary geologic assessment of potential adverse impact(s) to the Proposed Project from the December 3, 2019 slope erosion and storm drain failure was at the request of Mr. Michael Smith, Senior Planner, City of San Bruno. Our Engineering Geologist was present the morning of the event to aerial-drone document the conditions just after the City Maintenance and Public Works Departments had placed barricades across the affected slope. The bare earth LIDAR image map illustrating the approximate location and extent of slope erosion are presented on Plate 5. Figure 2 contains ground and aerial drone images taken on February 2, 2020 to document the status of near-final implementation of mitigation measures recommended to the City Public Works Department (Cotton, Shires and Associates, 2018).

Summary of Findings

Storm drain outfall has discharged storm water onto early 1950's undocumented reclamation fill placed in a broad seasonal drainage swale tributary to San Bruno Creek to accommodate construction of San Bruno Avenue. According to geologic mapping by Cotton, Shires and Associates (2018), the swale roughly coincides with the west flank of the erosional gully (gorge; Plate 5) and inferred depositional contact between Juro-Cretaceous Franciscan mélange and weak, highly erodible Pliocene marine Merced Formation. It is noteworthy, relative to slope stability of the southwestern bank of San Bruno Creek, that the eastern abutment of the 19th Century Crystal Springs Dam was found to have been unscathed by severe 1906 Earthquake ground shaking indicating the chaotic nature of the mélange supporting it has good resistance to earthquake-induced slope failure (USCOLD, 1992). This conclusion is supported by the general absence of reported and/or photogeologic evidence of global, bedrock instability affecting the site development area.

According to geologic mapping by Cotton, Shires and Associates (2008), it appears the recent catastrophic, southward retreat of the gully toward San Bruno Avenue, located approximate 300 feet from the proposed project, occurred during rainfall, but apparently after episodic retreat from decades of uncontrolled, concentrated stormwater discharge, as evidenced from reported gully formation and observed undermining exposed 24-inch diameter corrugated metal pipe (CMP) culvert protruding at least 8 feet from the steep headwall in 2008 (Cotton, Shires and Associates, 2018).

Historic maps and aerial photographs indicate the gullied outfall location, with an approximately 50-foot high, vertical headwall exposing native earth material to the confluence with San Bruno Creek, evolved from probably continuous storm water discharge onto the uniform approximately 1½H:1V fill slope. From experience of other similar and contemporary culvert systems constructed on the northern San Francisco Peninsula, we suspect appreciable subsurface seepage is, and continues to be, conveyed to the affected slope in the trench backfill containing the CMP draining a large area of the Crestmoor residential development area.

The slopes of Crestmoor Canyon are protected from surface erosion under current climatic conditions by the thick and pervasive woody vegetation. It precludes ease of access to San Bruno Creek, observation of the ground surface from the top of the canyon slopes, and photogeologic interpretation thereof. Fortuitous discovery of a 2007 bare earth hillshade LiDAR image overlay onto a Google Earth image covering the project area (Figure 1) effectively strips vegetation from the scene allowing geologic interpretation of historic slope conditions obscured by the vegetation that has existed below slope since before the 1943 vertical aerial photographs that comprise the stereogram on Plate 1. The gully is readily visible as a sinuous and serrated dark tone extending from San Bruno Avenue to confluence with San Bruno Creek marked by a bedrock landslide from undermining the northern creek bank. The upper to middle reach of the gully forms a distinct inside bend where runoff would impinge on the eastern side and another, opposing outside bend impinging on the lower middle segment. The upper reach is marked by two apparent, moderately dissected stream terraces. Terrace 1 appears to be at higher elevation but relatively concordant with Terrace 2 (Plate 5). The erosion pattern would then suggest Terrace 1 is an "ancestral" channel pirated by the entrenchment of the current channel draining the active stormwater discharge area; or the source location for historic runoff significant enough to mark the terrain as depicted on Plate 5 was from a significant source of runoff directed into the head of the swale adjacent to the segment of San Bruno Avenue between the current storm drain outfall and approximately 200 feet from the southeast corner of the proposed project area.

The presence of an earthen berm the top of the slope on the eastern side of the proposed development area, and geomorphic expression of surface erosion on the slope below suggest adverse concentrated runoff was directed to that area during mass grading of the ridge prior to drainage infrastructure for the adjoining commercial and residential developments nearly 70 years ago. These conditions would add light to the erosion conditions as described in the previous paragraph. Figure 1 and Plate 3 depict interpretations of slope morphology indicative of localized surficial erosion and landslide activity in the swale head mapped in the northeastern part of the proposed development area, on and below the eastern margin.

The mitigation being implemented, as judged from the site observations and the 2018 Site 1 Geotechnical Investigation appeared to be limited to:

- Installation of approximately 20 feet of approximately 24-inch diameter HDPE pipe connected to the pre-existing CMP, and placed against the steep gully headwall escarpment to discharge onto the lower section of headwall. Considerable erosion has occurred since installation of the new outfall pipe with a point of discharge approximately 20 feet below the new headwall. The position and orientation of the pipe outfall forces concentrated discharge against the curved transition from gully escarpment to west flank. In our opinion, the as-built condition of the stormwater discharge facility presents a low potential for adverse impact to site slope stability, but it is likely to induce rapid westerly undercutting of the west flank of the gully and consequent upslope and westerly headwall recession.
- Installation of a row of approximately 3-foot diameter concrete pier-reinforced double I-beam shear pins (AKA, stitch piers) spaced approximately 5 feet apart and extending at least 50 feet below the ground surface and spanning the existing gullying headway several feet as a measure to mitigate future headwall recession. Tiebacks were prescribed in the geotechnical report but evidence of their presence was not observed.

Photo 1. Areal image of storm drain outfall, including stitch piers (**A**), and new HDPE outfall pipe (**B**), at least 20 feet above the established drainage course (blue dot-dash line). Yellow hachured line denotes gully erosion from runoff since pipe was installed (**C**). Temporary erosion netting at top of slope (**D**).

Photo 2. Westerly view along top of slope line of double I-beam reinforced concrete stitch piers (**A**). Edge of west-bound lane of San Bruno Avenue on left of view. Temporary plastic tarp (**E**).

Fig. 2. February 2, 2020 observation of storm drain and slope retreat mitigation.

Conclusions

There was no observed evidence of significant gully erosion or known landslide activity that would exceed the scope of our recommended geotechnical mitigations to assure satisfactory performance over the projected design-life of the Proposed Project.

Existing erosion from City stormwater discharge is confined to the well-developed gully extending from the west-bound lane of San Bruno Avenue, approximately 300 feet east of the proposed project area. The recommended/observed as-built mitigations depicted on Figure 2 should stem upslope recession of the existing gully headwall, but we perceive gully erosion will continue downstream from the stitch pier headwall and laterally westward because of the observed position and orientation of the discharge point of the new HDPE segment of the outfall pipe. We strongly recommend the City seriously consider extending the point of storm drain outfall to at least the wider segment of gully channel reach, just upstream of the outside bend impingement, as a measure to mitigate potential westward erosion that will cause widening of the gully and likely expansion of headwall recession possibly beyond the area.

LIMITATIONS

We trust the information provided in this Reply to Geotechnical Peer Review, and the City requested supplemental section pertaining to geologic hazard assessment of gully erosion from the City storm drain outfall provides the information required at this time.

Should you have questions or need additional information, please contact the undersigned by email: cdare@geosphereinc.net or jbaldwin@geosphereinc.net at (650) 557-0262. We greatly appreciate the opportunity to be involved in this project.

Sincerely,
GEOSPHERE CONSULTANTS, INC.

Alex Lim, P.E., Q.S.P.
Project Engineer

Joel E. Baldwin, II, P.G., C.E.G.
Principal Engineering Geologist (Renewal date 2/28/21)

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Distribution: Efile to: Mr. Jiang Zhan Lin, New Shidai Development, LLC; Michael Smith, Senior Planner, City of San Bruno; Stan Panko, Panko Architects

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Appendix G – Slope Stability Analyses
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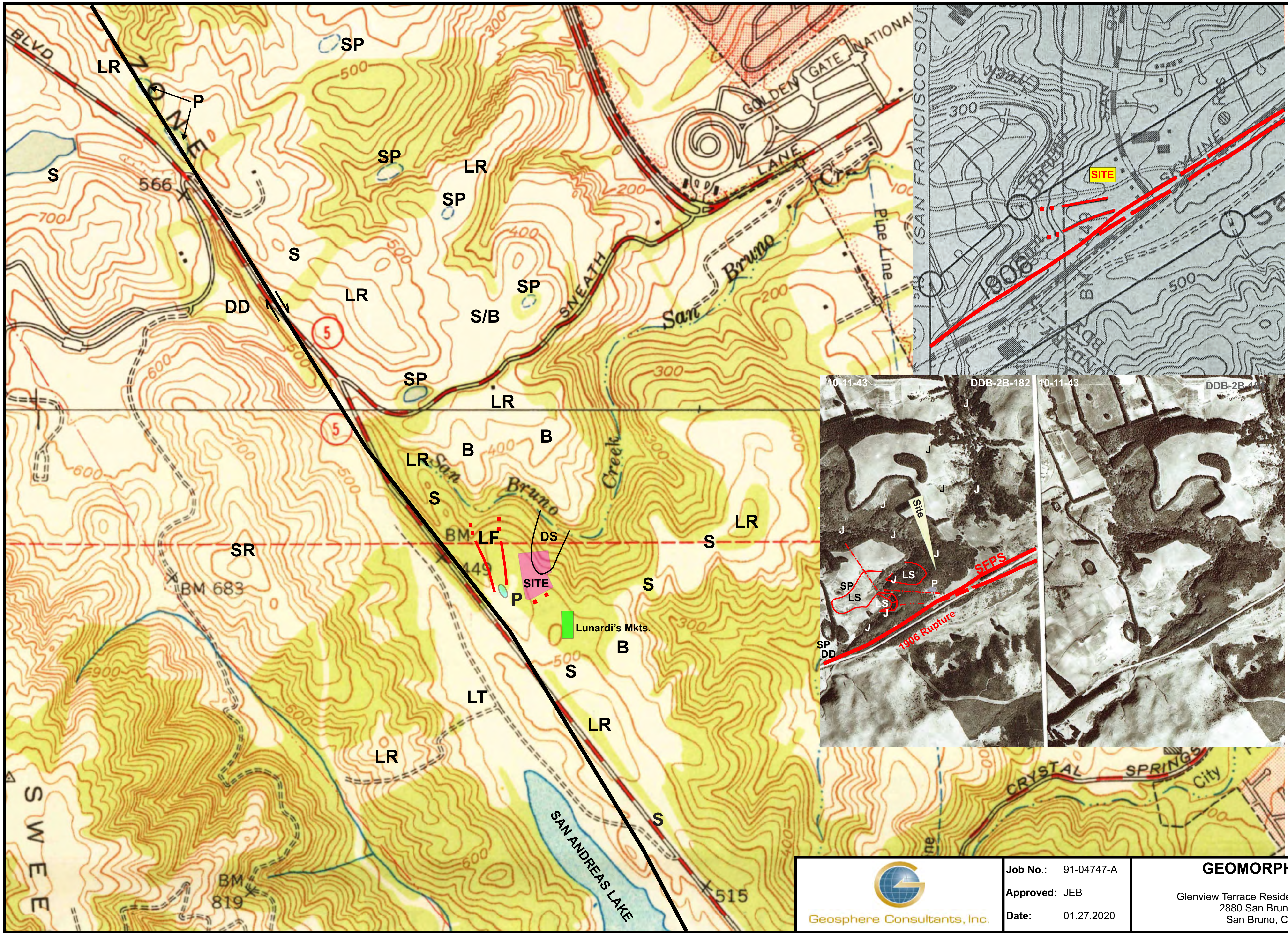


EXPLANATION

Historic Topographic Map

San Andreas Fault (1906 Earthquake rupture trace, after Pampeyan,1994)

GEOMORPHIC FEATURES

- **B** Bench
- **DD** Deflected Drainage
- **DS** Drainage Swale
- **LF** Linear Feature
- **LR** Linear Ridge
- **LT** Linear Trough
- **S** Historic Stock Pond
- **SP** Sag Pond

N

1000 ft.
Scale

Source: USGS (1949)

Montara Mt. Earthquake Fault Zone

Known active historic trace SFPS

Photogeologic lineament

Alquist Priolo EFZ boundary

N

1000 ft.
Scale

Stereogram

Linear notch (strong)

Linear notch (mod. strong)

- **J** Structure-controlled drainage
- **SP** Historic stock pond
- **S** Sag pond
- **LS** Landslide
- **DD** Deflected drainage

N

1700 ft.
Approx. Scale

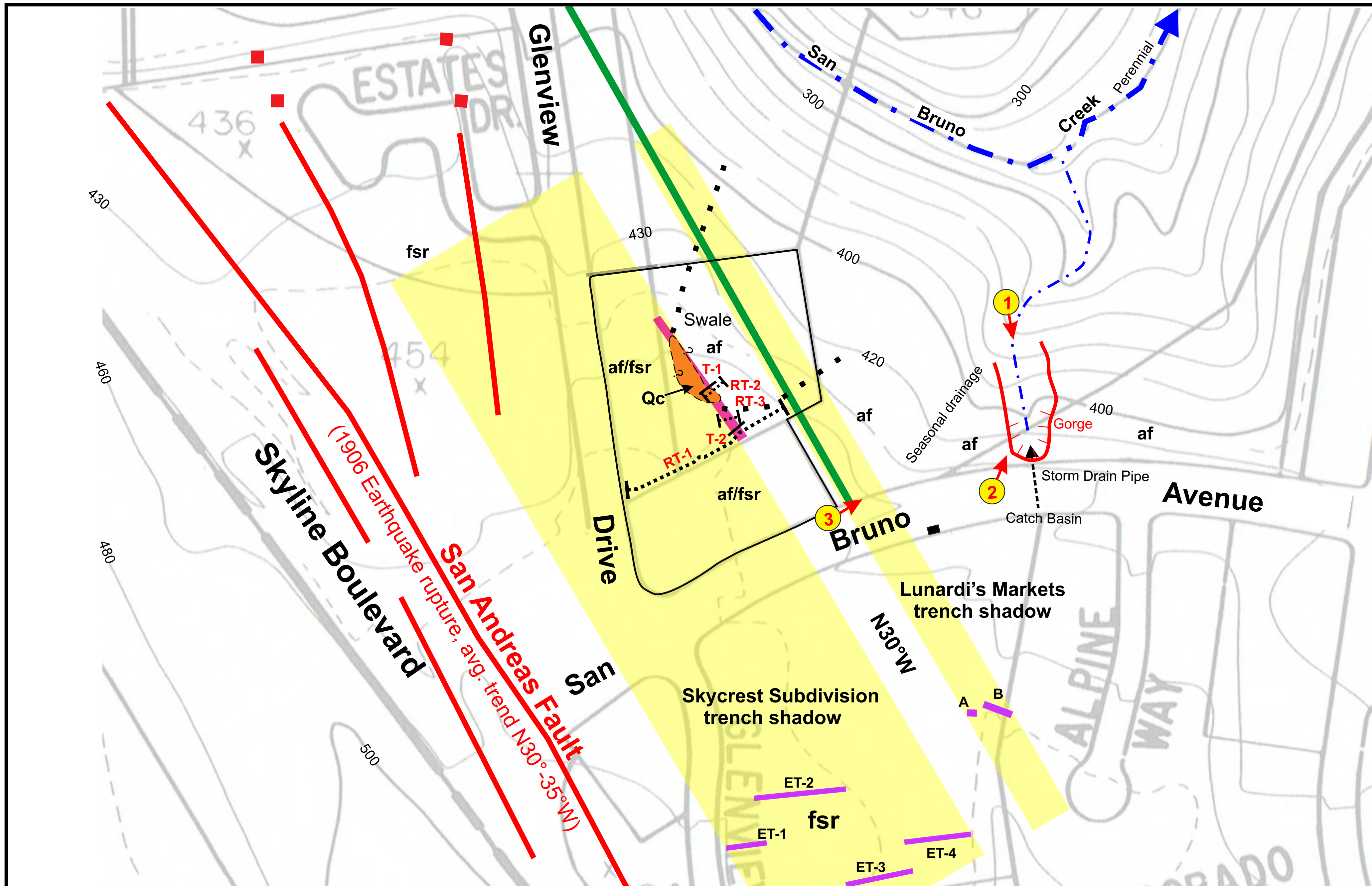
Geosphere Consultants, Inc.

Job No.:	91-04747-A
Approved:	JEB
Date:	01.27.2020

GEOMORPHIC MAP

Glenview Terrace Residential Development
2880 San Bruno Avenue
San Bruno, California

Plate
1



EXPLANATION

af Artificial fill mantling native earth (af/Qc)

Qc Pleistocene colluvium

fsr Juro-Cretaceous Franciscan sheared rock (melange)

Geologic Contact, inferred from boring data, dotted where concealed beneath fill

Active trace of San Andreas Fault (California Geological Survey, 1982)

Northeast boundary of Earthquake Fault Zone (California Geological Survey, 1982)

Inferred N35°W trace of potentially active fault (Romig Engineers, 2008)

ET-1, A — Approx. location of Paleoseismic Trench (BAGG: ET 1-4, 2003; A & B, 2007)

RT1-3 — Approx. location of Paleoseismic Trench (Romig Engineers, 2008)

T1-2 — Approx. location of Paleoseismic Trench (Earth Investigations Consultants, Inc., 2008)

N30°W Fault Shadow from nearby property exploratory trench exposures (BAGG, 2003, 2007)

Erosional gorge from storm drainage discharge onto bank of San Bruno Ck.

Channelized runoff

1 — Drone image 1 approx. line of sight

N

200 ft.
Scale
Contour Interval 20'
Source: San Mateo County Map Sheet 5C (1996)

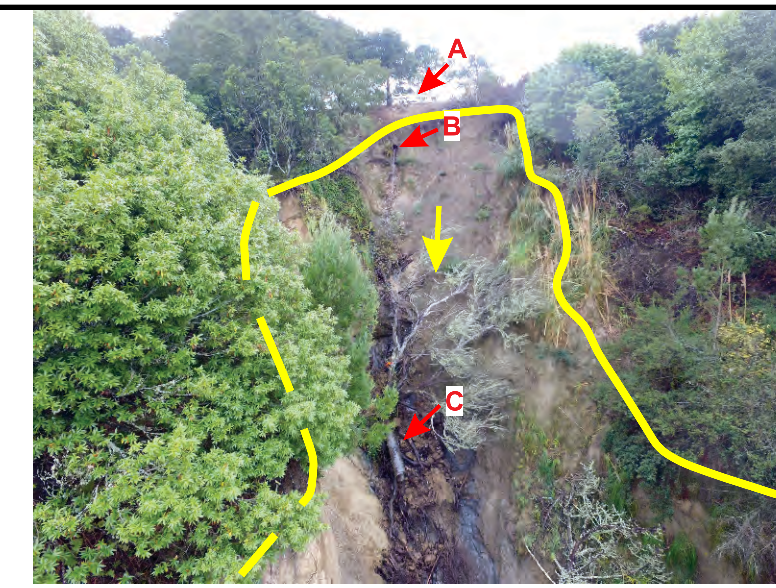
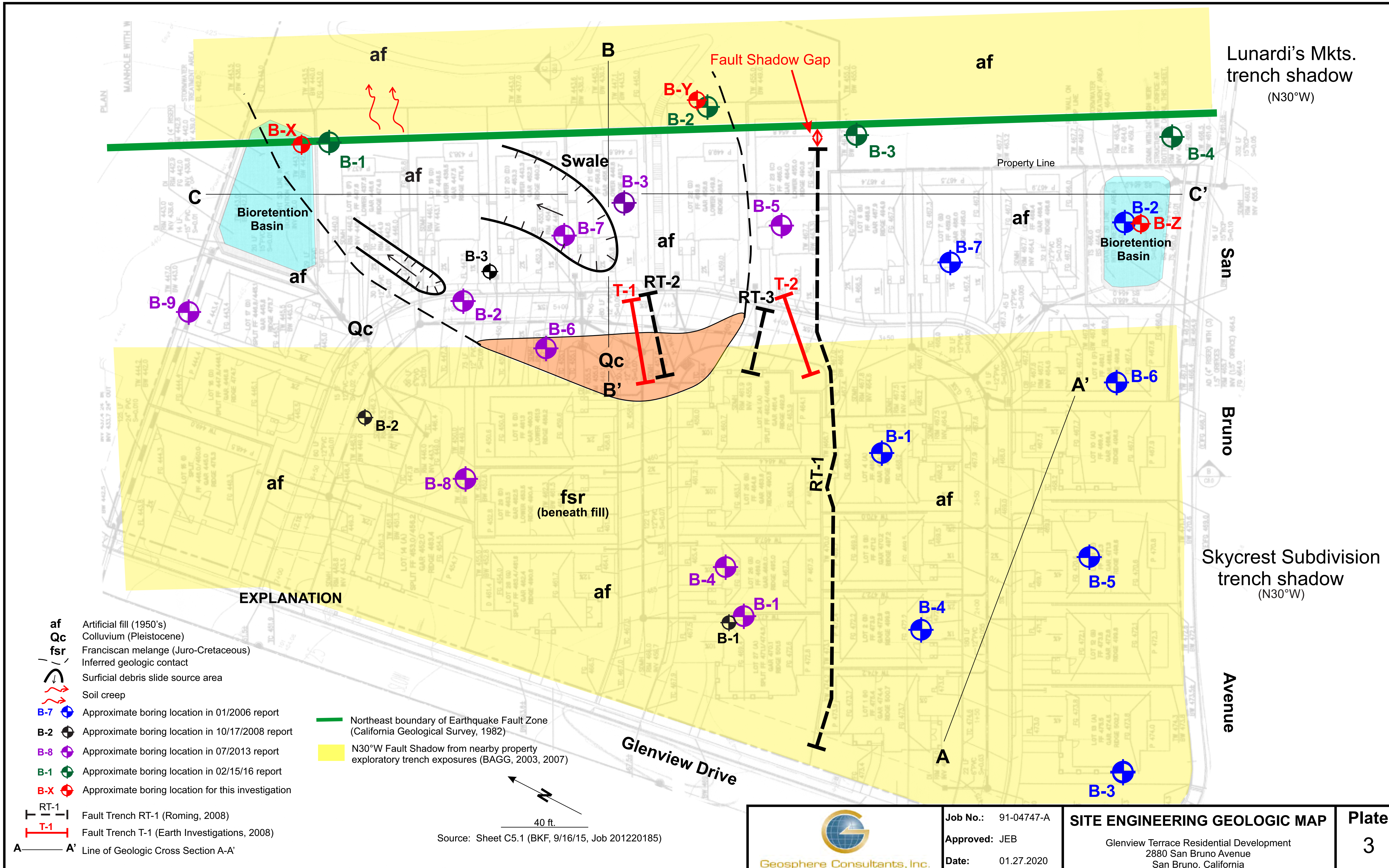


Image 1. South view of 12/04/19 erosional gorge & escarpment from storm drain pipe. A is edge of roadway; B is current CMP outfall; C is detached pipe segment.

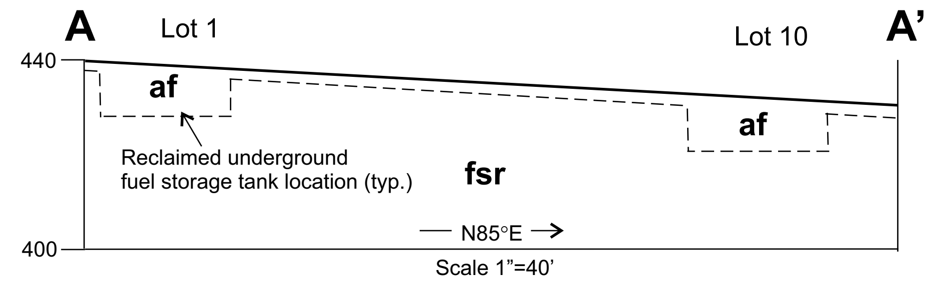
Image 2. North downstream view of confluence of gorge with San Bruno Creek.

Image 3. East view of headward erosion toward San Bruno Avenue. Arrow points to dilapidated asphalt roadway.

Geosphere Consultants, Inc.	Job No.: 91-04747-A Approved: JEB Date: 01.03.2020	**COMPOSITE GEOLOGIC HAZARD MAP** Glenview Terrace Residential Development 2880 San Bruno Avenue San Bruno, California	**Plate** 2

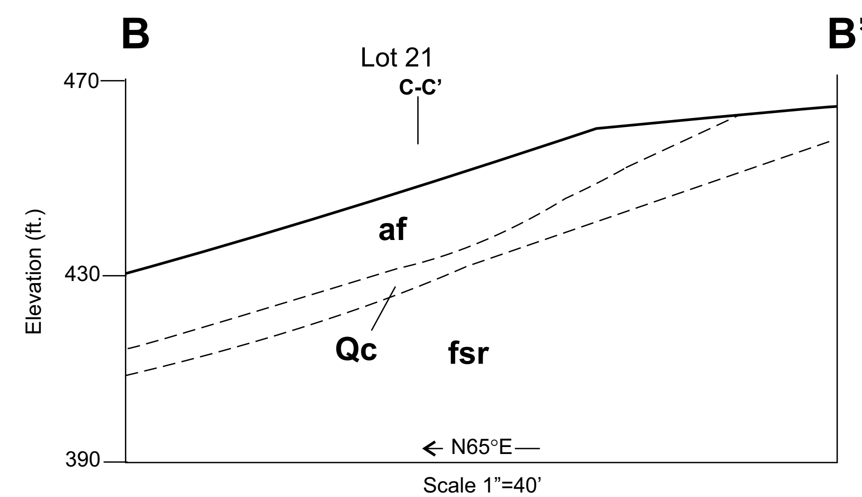


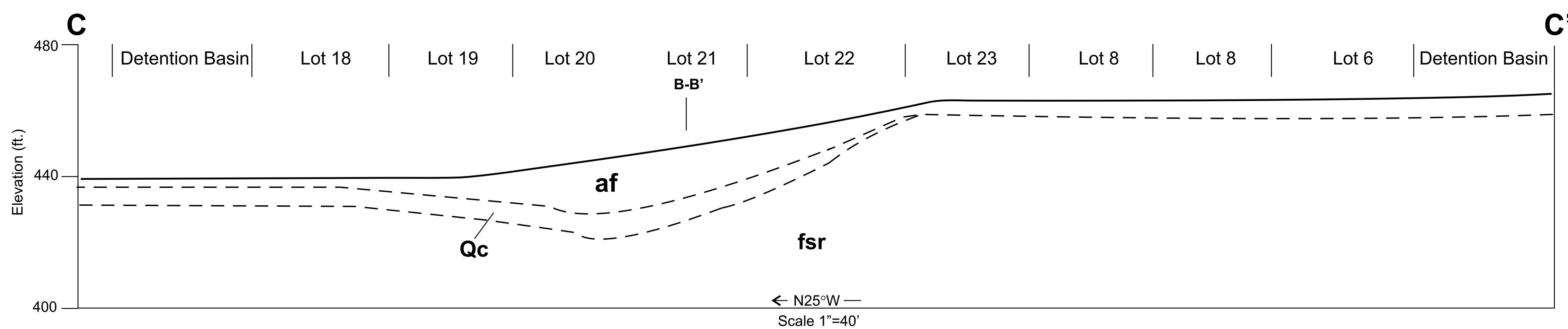
Lunardi's Mkts.
trench shadow
(N30°W)
Fault Shadow Gap
B
B'
C
C'
A
A'
af
Qc
fsr
(beneath fill)
Swale
Bioretention Basin
Bioretention Basin
Property Line
B-X
B-Y
B-Z
B-1
B-2
B-3
B-4
B-5
B-6
B-7
B-8
B-9
T-1
T-2
RT-1
RT-2
RT-3
San Bruno Avenue
Glenview Drive
Skycrest Subdivision
trench shadow
(N30°W)
EXPLANATION
af Artificial fill (1950's)
Qc Colluvium (Pleistocene)
fsr Franciscan melange (Juro-Cretaceous)
Inferred geologic contact
Surficial debris slide source area
Soil creep
B-7 Approximate boring location in 01/2006 report
B-2 Approximate boring location in 10/17/2008 report
B-8 Approximate boring location in 07/2013 report
B-1 Approximate boring location in 02/15/16 report
B-X Approximate boring location for this investigation
RT-1 Fault Trench RT-1 (Roming, 2008)
T-1 Fault Trench T-1 (Earth Investigations, 2008)
A A' Line of Geologic Cross Section A-A'
Northeast boundary of Earthquake Fault Zone
(California Geological Survey, 1982)
N30°W Fault Shadow from nearby property
exploratory trench exposures (BAGG, 2003, 2007)
N
40 ft.
Source: Sheet C5.1 (BKF, 9/16/15, Job 201220185)
Geosphere Consultants, Inc.
Job No.: 91-04747-A
Approved: JEB
Date: 01.27.2020
SITE ENGINEERING GEOLOGIC MAP
Glenview Terrace Residential Development
2880 San Bruno Avenue
San Bruno, California
Plate
3



EXPLANATION

af Undocumented fill (early 1950's)
Qc Colluvium (Pleistocene)
fsr Franciscan sheared rock (melangé)
Geologic contact inferred from boring data





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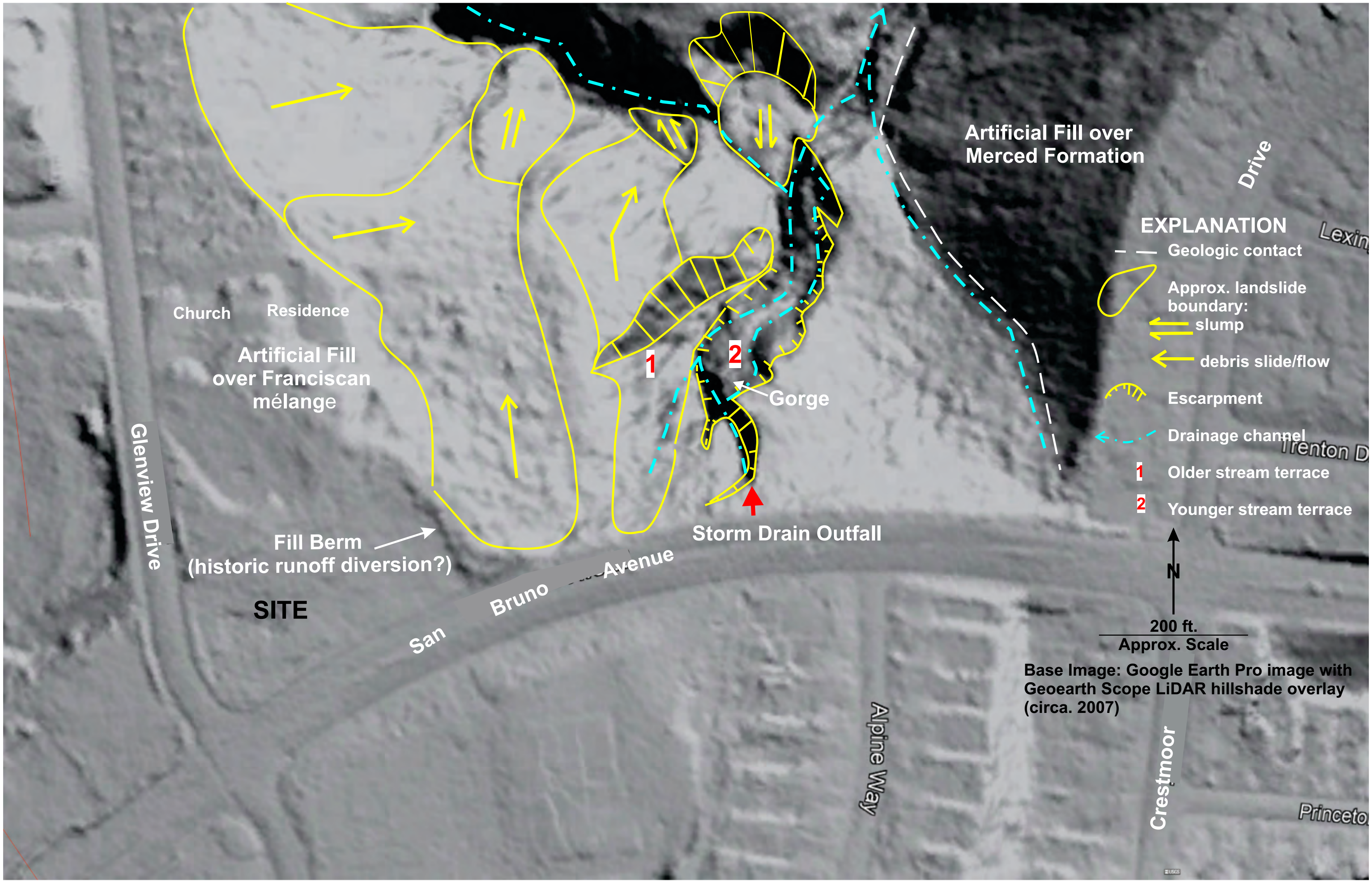
Job No.: 91-04747-A
Approved: JEB
Date: 01.27.2020

CROSS SECTIONS A-A' - C-C'

Glenview Terrace Residential Development
2880 San Bruno Avenue
San Bruno, California

Plate
4



Geosphere Consultants, Inc.	Job No.: 91-04747-A Approved: JEB Date: 01.21.2020	**EROSIONAL FEATURES** Glenview Terrace Residential Development 2880 San Bruno Avenue San Bruno, California	**Plate** 5

APPENDIX A

This appendix contains the Earth Investigations Consultants (EIC) 2013 geotechnial report for a previous site development plan.

GEOTECHNICAL INVESTIGATION

Proposed Glenview Terrace, Phase 2
850 Glenview Drive
San Bruno, California

Prepared for:
U.S.A. Guofu Investments, Inc.
475 El Camino Real, Room 218
Millbrae, California 94030

Attention: Mr. Guo Chen
August 4, 2013
Job 2479.02.00

Earth Investigations Consultants, Inc.
P.O. Box 795
Pacifica, California 94044
Phone 650-557-0262
earthinvestigations@comcast.net

August 4, 2013
Job 2479.02.00

U.S.A Guofu Investment, Inc.
475 El Camino Real
Millbrae, California 94030

Attention: Mr, Guo Chen

RE: GEOTECHNICAL INVESTIGATION
Proposed Glenview Terrace, Phase 2
850 Glenview Drive
San Bruno, California

Dear Mr. Chen:

In accordance with our *Agreement For Professional Services* (dated June 12, 2013) we have prepared the attached design-level geotechnical report for the proposed project.

We appreciate having the opportunity to participate with you on this project. Please contact our office if you have any questions or comments on our findings, conclusions, and recommendations.

Sincerely,

Earth Investigations Consultants, Inc.

Joel E. Baldwin, II
Engineering Geologist, CEG 1132

David W. Buckley
Civil Engineer, RCE 34386

JEB:DWB:jb:gi
Distribution: 3 bound copies and efile to Panko Architects

Geologists & Engineers
P.O. Box 795 ● Pacifica, CA 94044 ● (650) 557-0262 ● Fax (650) 557-0264 ● earthinvestigations@comcast.net

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INTRODUCTION

Location of Investigation and Proposed Project

This investigation was performed on the property currently occupied by Peace Lutheran Church, located at 850 Glenview Drive, San Bruno, California, near the intersection with San Bruno Avenue, in San Bruno, California (Plate 1, Vicinity Map). We understand the proposed project will entail construction of 17 detached, single family residence structures to be accessed by private streets off Glenview Drive (Plate 2, Site Plan). We understand the existing parsonage in the northeast corner of the property will be remodeled. The existing church, retaining walls and pavements will be removed. Grading will entail removal of existing undocumented fill will be removed and replaced as engineered fill, and construction of building pads. We anticipate application of site retaining walls to create the individual building pads.

Purpose of Investigation and Scope of Services

This investigation was undertaken to characterize the geotechnical setting and provide design-level geotechnical recommendations for the proposed project. It augments our previous geologic investigation, which found the site to not be constrained by potential fault rupture hazards (Earth Investigations Consultants, 2008).

The findings, conclusions, and recommendations in this report are based upon the following scope of services:

- Review of our geologic file for this project and other pertinent geologic and maps and geotechnical reports in our files pertinent to the proposed project. Plate 3 contains an excerpt of the regional geologic setting covering the site area;

- Engineering geologic mapping of the site onto the project site plan (Plate 4);
- Foundation soil profile characterization by advancing 9 borings on the site. Borings 1 through 3 obtained continuous samples of the earth materials with a 1 ½ -inch O.D., split spoon sampler advanced to practical refusal with a portable, gas-powered Wacker BHF 30S percussion hammer that imparts 35 ft. lbs. of axial force on the sampler at a rate of 1270 blows per minute. The borings, supervised, logged, and sampled by our field engineer, ranged in depth from 10 to 22 feet. Borings 4 though 9 were drilled with a truck-mounted, hollow-stem continuous flight auger. Relatively undisturbed soil samples were retrieved from the selected depths using Modified California and Standard Penetration (SPT) samplers. The samplers were driven with a 140-pound hammer free-falling from 30 inches. The number of drops (blows) required to advance the respective samplers at 6-inch intervals for a total of 18-inches are tabulated in SPT values in the Borings Logs. The blow counts were converted to Standard Penetration Test values using a multiplier of 0.8. Appendix A contains the Logs of Borings from this investigation (Plates A1-A6; Plates A7-A8 contain descriptions of the terms and symbols used on the logs. Plate A9 contains Logs of Trenches from our 2008 Engineering Geologic Investigation);
- ASTM laboratory testing of selected samples from the borings. Tests included moisture content, dry density, and pocket penetrometer unconfined compressive strength. The results of the lab tests are tabulated on the Boring Logs at the respective sample depths;
- Analysis of the data and preparation of this report. Plate 5 depicts Generalized Cross Sections A-A', B-B' and C-C' through selected parts of the project site derived from the soil exploration data.

PHYSICAL SETTING

Topography and Drainage

The area of this investigation is characterized by a terrace graded topographic surface bordered on the northeast by a steep, densely vegetated slope that drains to San Bruno Creek (Plates 1 and 2). There was no observed evidence of springs at the time of our site observations.

The site topography is a product of mass grading during the middle to late 1950's for improvement of San Bruno Avenue and Glenview Drive and late 1950's to early middle 1960's for development of the church facility. Historic aerial photographs indicate at least 20 feet of earth was removed from the dissected, linear ridgeline that originally characterized the site area. From the aerial photographs, it is apparent much of the investigation area is a cut surface with undocumented fill extending over the steep slope bordering the northeastern side. This slope descends eastward approximately 150 vertical feet to San Bruno Creek (Plate 1).

The top of the steep, undocumented fill slope has been subjected to uncontrolled runoff, which probably caused the two, relatively shallow debris slide scars mapped in the eastern part of the site (Plate 4). The larger debris slide scar is on the undeveloped slope and the smaller one is on the slope above the parsonage pad. We suspect this activity is more than a decade old given the relative roundness of the scars and density of vegetation covering the distal part. There was no apparent debris slide deposit observed on the slope.

Geology

The site occupies the middle of a semi-circular body of Juro-Cretaceous Franciscan assemblage sheared rock bounded on the east by a curvilinear, inferred trace of the Serra fault, and on the west by the San Andreas fault zone (Leighton and Associates, 1976; Pampeyan, 1994; Plate 3). There are abundant exposures in road cuts between the site and Highway 35 to the southwest, which exhibit randomly and complexly distributed, often hard blocks (knockers) of ultramafic rock, including serpentinite and silica carbonate rock, supported in a "matrix" of pulverized rock having the consistency of firm to stiff, usually plastic, gravelly and sandy clay and clayey sand. This rock is mapped as mélange. Bailey and others (1964) describe it as several hundred feet of soft, light to dark gray sheared shale, siltstone, and greywacke with various-size tectonic inclusions of Franciscan rock, on the order of 100 million years old.

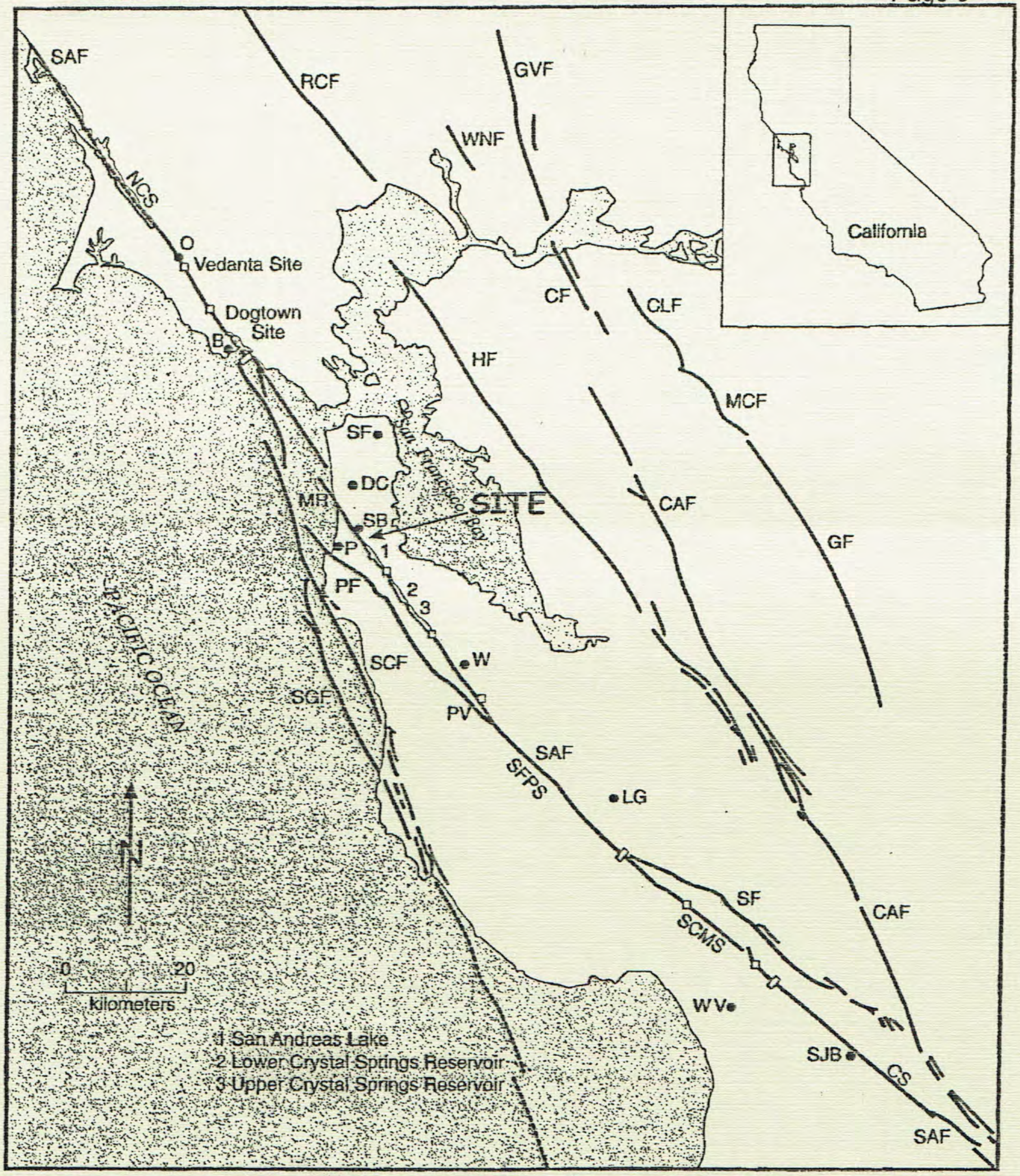
On the site, the sheared rock is mantled by a variable thickness of undocumented fill; thickest on the steep easterly slope beneath the parsonage and proposed Lots B8, C5 and D6-7. Several geologic investigation reports for nearby developments describe the bedrock as breccia comprised of: angular rock fragments, including calcareous shale and commonly sheared serpentinite supported by a plastic clay matrix (Bay Soils, Inc., 1978); serpentine and greywacke rock fragments and knockers in shale matrix in weathered to silty clay, greenstone and altered volcanic rocks with clay along remnant fracture surfaces (JCP, 1984); white breccia, serpentine, mylonite, clayey sand with greenstone fragments (Earth Systems Consultants, 1985); weathered greenstone, sandstone and shale similar to bedrock similar to that described in previous investigation (Earth Systems Consultants, 1989); clay matrix around greenstone or basalt with sign of shearing (Hallenbeck & Associates, 1989); sandstone inclusions in matrix of sheared rock, greenstone clasts in clay matrix, mélange of sheared clayey shale, and siltstone mixed and contorted with inclusions of sandstone and serpentinite in clay matrix, serpentinite mostly crushed and sheared with occasional hard blocks and cobbles with polished surfaces (BAGG, 2003); sheared silica carbonate fragments, altered serpentinite and highly sheared remnants of shale, and claystone in a clayey matrix (Romig Engineers, Inc., 2008).

Active Faults and Seismicity

The site lies in a seismically active region on the westerly side of a 60-mile wide northwest trending band of active and potentially active faults known as the San Andreas fault system (fig. 1). Faults in this zone are predominately right-lateral, strike slip faults that collectively have accommodated most of the relative motion between the Pacific Plate on the southwest side and the North American Plate on the northeast side, over the past 29 million years (Wallace, 1990). The site lies on the westerly margin of the North American Plate:

The San Andreas fault, with a landward extent of nearly 800 miles between the Imperial Valley and Mendocino coastline, is considered the most extensive and active of the faults in the San Andreas fault system (Jennings, 1994). In the site area, it is represented by the San Francisco Peninsula Segment (SFPS, fig. 1). Hall and others (2001) report it has an average right lateral slip rate of approximately 0.7 inches per year.

Historic crustal movement on the SFPS has been responsible for major central California earthquakes, including the magnitude 7.9, 1906, San Francisco earthquake and strong ground shaking in the site area (Lawson, 1908). Lawson (1908) described ground rupture in the 1906 earthquake as a continuous, gigantic mole track, up to 10 feet wide, sometimes branching out to several furrows over a width of up to 100 feet, with the typical occurrence of a straight



Geographic Locations: O=Olema, B=Bolinas, SF=San Francisco, DC=Daly City, P=Pacifica, W=Woodside, PV=Portola Valley, LG=Los Gatos, WV=Watsonville, SJB=San Juan Bautista, MR=Mussel Rock, SB=San Bruno
Faults: SAF=San Andreas, RCF=Rodgers Creek, WNF=West Napa, PF=Pilarcitos, CAF=Calaveras, GF=Greenville, SCF=Seal Cove, SGF=San Gregorio, SF=Sargent, MCF=Marsh Creek, CLF=Clayton, CF=Concord, GVF=Green Valley
Note: Open rectangles are the approximate location of segment boundaries of the Northern California section of the SAF and between the Northern California and Central California sections (NCS=North Coast, SFPS=San Francisco Peninsula. SCMS=Santa Cruz Mountains, CS=Central California; modified from WGCEP, 1990).

Figure 1. Map of the San Andreas fault system in San Francisco Bay area (after Hall and others, 2001)

line of raised sod blocks, broken and partly overturned along a North 35 West fault trace extending west of the site between Mussel Rock to San Andreas Lake.

The SFPS also produced a magnitude 5.3 earthquake in 1957 with weak ground shaking in the site area (Oakeshott, 1959). The magnitude 6.9, Loma Prieta earthquake, centered on the Santa Cruz Mountain segment (SCMS; fig. 1), was responsible for the magnitude 7, 1989 Loma Prieta earthquake with moderate to strong ground shaking in the site area (Plafker and Galloway, 1989). There was no apparent fault surface rupture associated with that event. Hall and others (2001) believe that SFPS was also responsible for a large earthquake in 1889.

The potentially active Serra fault, mapped just east of the site, separates the older Franciscan rocks from the younger Merced formation on the east side. The State Geologist considers it to be inactive (no movement in the past 11,000 years or Holocene time; California Division of Mines and Geology, 1981, 1982). However, Kennedy (2004) reports evidence for Holocene movement at Fort Funston.

Other potential sources for local, major earthquakes include the offshore trace of the San Gregorio-Seal Cove fault mapped approximately 5 miles to the southwest, and the Hayward and Calaveras faults respectively mapped approximately 18 and 26 miles to the northeast. Movement on the San Gregorio fault in 1926 is suspected of producing a magnitude earthquake in the Monterey Bay. Movement on the Hayward fault produced an estimated magnitude 7.1 earthquake in 1868, and movement on the Calaveras fault produced 2 magnitude 6.2 earthquakes in Morgan Hill (1911, and 1984) and the magnitude 6.3 Gilroy earthquake in 1897.

According to the Working Group (2008), there is a 63 percent chance of a 6.7 or greater earthquake occurring in the next 23. The potential for the aforementioned faults producing a major earthquake over this period is as follows: Calaveras fault, 3%; San Gregorio-Seal Cove fault, 6% San Andreas fault, 21%; and Hayward fault, 31%. In the event of a major earthquake, the site is expected to experience very strong to very violent ground shaking (Petersen and others, 1999).

Fault Rupture Hazard

We concluded from our 2008 geologic investigation there are not active fault traces constraining the proposed project area.

Liquefaction Hazard

Liquefaction potential is considered nil given the site is underlain at shallow depths by consolidated bedrock.

Landslide Hazard

There are no mapped or was there observed deep-seated bedrock landslides affecting the property. As described previously, we mapped two shallow debris slide scars caused by uncontrolled runoff across the steep undocumented fill slope in that area of the site. Potential for debris flow impact on the site is nil given is occupies the local topographic high of a ridge line

Flood Hazard

Flood hazard potential is considered nil given there are no active drainage courses on the site, or water impoundments above it (Ironside and Associates, undated).

SITE EXPLORATION

Site exploration logs contained in Appendix A indicate the site is underlain by Franciscan mélange, bedrock that is mantled by undocumented fill and native colluvial soil. The locations of the explorations are depicted on Plate 4. Explorations revealed that the fill consists of medium dense to very dense, silty sand with a variable content of clay and gravel. It occurs as slivers resting on colluvium in the northern half of the site and directly on bedrock in the east side. It is apparent that a seasonal drainage swale in area of proposed was buried by approximately up to 16 feet of undocumented fill beneath proposed Lots B8 and D7, with from 4 to 10 feet of fill beneath proposed Lots C5 and D6.

Colluvium separating undocumented fill from the bedrock is up to 10 feet thick at the Fault Trench 1 location at the head of the buried swale and intersection of the proposed streets (T-1; Plate 4). It occurs as medium dense, clayey sand, and stiff to hard sandy clay with a variable gravel content. The bedrock is mélange with sheared rock containing soft to moderately hard sandstone and serpentinite and very hard silica-carbonate rock.

The soils were found to be generally moist to damp. Ground water was not encountered in the borings.

DISCUSSION AND CONCLUSIONS

The results of this investigation indicate that the proposed residential subdivision is feasible from a geotechnical standpoint. There are no active faults or bedrock landslides constraining the proposed project layout. The site is underlain by stable mélange bedrock of the Franciscan assemblage mantled by a variable thickness of undocumented native colluvium and undocumented fill. Uncontrolled runoff has resulted in localized debris slides in the on the northeastern part of the site, however these features will be mitigated by remedial grading of the undocumented fill during mass grading.

It is our opinion that proposed residential buildings can be supported in competent native earth materials by spread footings or drilled piers. The following sections of this report provide pertinent design-level geotechnical recommendations for use by the project architect and engineers.

RECOMMENDATIONS

Seismic Design

The proposed structures should be designed for the following seismic design criteria derived from the subsurface exploration data and the 2010 California Building Code (CBC):

- Site Location: Latitude = 37.620 Longitude = -122.441
- Site Soil Class: C
- Spectral Response Acceleration Values: Ss = 2.236; S1 = 1.283; Fa = 1.0; Fv = 1.3; SMs = 2.236; SM1 = 1.668; SDs = 1.491; SD1 = 1.112;

Site Preparation, Grading and Compaction

Grading should be performed in the dry months. Grading will be required to develop the roadway alignment, house pads and driveways. We anticipate that grading balanced by using excavated earth material as engineered reinforced fill. We judge most of the site soils, including existing undocumented fill are acceptable provided they exhibit low plasticity, and free of organic and construction debris, as assessed by our field engineer during mass grading. Existing pavements can be ground up in place with the underlying baserock, for subsequent use in construction of interior slab capillary moisture breaks. Import soil, if required, should have a PI of 15 or less. The upper 6 inches of colluvium containing organic material should be avoided, but can be utilized for landscaping. Otherwise, it should be removed from the site. Voids resulting from site demolition, grubbing and tree removal should be backfilled with engineered fill, as described below.

All existing fill should be removed from proposed the proposed development areas. Exposed native colluvial soil surfaces should be scarified to a minimum depth of 8 inches, moisture condition to near optimum, and compacted to at least 90 percent relative to the ASTM D-1557 laboratory test procedure. Level benches should be graded on slopes greater than 7H:1V prior to soil scarification and compaction. Exposed native soil should be prepared as prepared as described above prior to fill placement. Fill should be placed onto the prepared native soil surface is loose lifts no greater than 8 inches thick, moisture conditioned to near optimum and compacted to at least 90 percent relative to the dry density determined from laboratory testing. Thinner lifts will be required in corners of the excavation with hand-operated compaction equipment.

Remedial grading of the buried swale in the eastern part of the site will require a properly drained key at the toe of the proposed buttress fill, graded with the base having a minimum slope of 2 percent into the hillside (refer to Plate 6 for specifications). Once the key is filled with compacted earth, level benches shall be but cut into competent native soil as the fill rises to the finished surface. Bench subdrainage requirements will be assessed during grading, however, for planning assume installation of a subdrain at the rear of every other bench elevation above the key subdrain. After the exposed, native soil surface is exposed, it should be scarified at least 8 inches deep, moisture conditioned to near optimum and compacted to at least 90 percent of the maximum dry density of the materials. Once the bench surface is prepared, soil suitable for engineered fill shall be placed in loose maximum 8-inch thick lifts, moisture conditioned to near optimum and compacted to at least 90 percent relative to the maximum dry density of the soil used. A reinforced concrete V-ditch shall be constructed at the top of the finished fill slope to intercept and redirect surface runoff to an approved discharge location.

As fill increases up a slope, level benches should be excavated to expose competent support material. Fill should not be placed on soft soil. The field engineer should assess the depth into bedrock of the key and benches, and the subdrainage requirements during grading. You should expect that subdrainage will be required for fills greater than 5 feet in thickness, and elsewhere near proposed improvements where there is perceived or active seepage.

The maximum fill slope shall be 2H:1V unless reinforced by an approved geogrid (i.e., Miragrid or equal), spaced horizontally 2 feet apart in the compacted fill. If granular soil angle of internal friction of at least 45 degrees, it may be possible to achieve a finished fill slope of 1H:1V. Preliminarily, we recommend that cut slopes in bedrock have a maximum gradient of 2H:1V, and 3H:1V in colluvium.

However, the actual material quality at the cut slope location will dictate maximum inclinations (e.g., a 5-foot high cut slope in massive sandstone may be acceptable with a finish slope of 1 ½H:1V). Steeper slopes should be retained by engineered retaining walls.

Utility Trenches

Vertical trench excavations up to 5 feet deep should be capable of standing with minimal bracing for short duration (less than 30 days). However, contractors should be alert to potential instability. Trench walls deeper than 5 feet should be cut and braced in accordance with the State of California Safety Ordinance treating excavations and trenches.

Utility trenches should be designed to prevent the transportation of water into the foundations, slabs or pavement subgrade soils. Care should be taken to assure that uncontrolled, concentrated runoff is not conducted toward the existing slopes. In particular, where utilities cross foundations, trenches should be plugged with compacted clayey soil for their full depth, and for a distance of at least 5 feet on either side of the foundations.

On-site, inorganic soil may be used as utility trench backfill. Special compaction of trench backfill will be necessary under and to within 3 feet of proposed structures, concrete slabs, asphalt pavements, and engineered fill. In these areas, backfill should be conditioned to approximately 3 percent above optimum and placed in horizontal lifts, each not exceeding 4 inches in loose thickness. Each layer should then be compacted to at least 90 percent MDD. The top 2 feet of trench backfill under pavements should be non-expansive, granular soil compacted to at least 90 percent MDD.

Foundations

It is acceptable to utilize footings for building and retaining wall foundation support provided they are embedded in bedrock or competent native soil with a minimum of 7 feet of horizontal confinement between the bottom, outer edge and nearest slope exceeding 3H:1V. Alternatively, drilled piers can be used for foundation support. Combination of footings and piers can be considered to achieve a safe setback, given both achieve bedrock. Otherwise, dissimilar foundations should be avoided.

Footings

The footings should be designed with the following geotechnical parameters:

- Footings should only be continuous. Isolated footings are not recommended;
- Footings should have a minimum width of 12 inches, and extend at least 18 inches into bedrock or competent native and engineered fill soil;
- Footings should be designed for an allowable bearing value of 3500 pounds per square foot (psf) for dead plus live loads in bedrock and 2500 psf in competent native and engineered fill soil. Both values should begin at the ground surface and increased by 1/3 to account for wind and seismic loads;
- Passive equivalent fluid pressure of 500 pounds per cubic foot (pcf) beginning at the bedrock surface, or 300 pcf in competent native and engineered fill soil beginning at the ground surface;
- Coefficient of friction at the base of the footing of 0.4 for bedrock and 0.3 for native and engineered fill soil.

Drilled Piers

Drilled, cast-in-place concrete piers should be at least 16 inches in diameter, and extend at least 12 feet deep. Piers on within 7 feet of a slope greater than 3H:1V should be drilled to a minimum depth of 15 feet. We recommend that the pier foundation be designed for an allowable skin friction value of 450 pounds per square foot (psf) beginning at a depth of 3 feet. The skin friction value should be increased by 1/3 to account for wind and seismic loads. End bearing of piers should be neglected in design because of the difficulty in cleaning out small diameter holes.

The piers should be designed for a passive equivalent fluid pressure of 400 pounds per cubic foot (pcf) acting over 1½ pier diameters beginning at the ground surface. Piers support on slopes greater than 3H:1V should begin at a depth of 3 feet.

Colluvium below a depth of 3 feet can be assigned a skin friction value of 350 psf and passive equivalent fluid pressure of 300 pcf acting over 1½ pier diameters.

The upper 3 feet of soil on slopes steeper than 3H:1V should be designed to resist a lateral creep load of 45 pcf.

Perimeter and interior piers should be interconnected by grade beams to avoid potential problems associated with isolated piers in expansive soils, and in seismically active areas.

In the event ground water is encountered in the pier holes, it may be necessary to remove standing water by the tremie method. If pier holes cave, it will be necessary to install casing to maintain the holes open until concrete is placed. Any concrete overpour will require removal prior to construction of grade beams.

Slabs-on-Grade

We recommend that the living spaces be designed with raised wood floors. Slab subgrades should be prepared as discussed in the *Site Preparation* section. In the unlikely event highly expansive soil is encountered in slab areas, it should be overexcavated at least 12 inches and replaced with non-expansive site or granular import soil. Slab subgrade moisture should be maintained 2 percent above optimum and should be approved by our field engineer prior concrete placement.

All concrete slabs should be a minimum of 5 inches thick, and reinforced with at least No. 4 bars spaced no greater than 12 inches apart, in both directions. Slabs should be underlain with a capillary moisture break consisting of at least 6 inches of clean, free-draining, crushed rock or gravel that drains to outlet, where practical. To mitigate migration of moisture vapor through the slabs, we recommend installation of an impermeable moisture vapor barrier (15 mil Stego wrap or better) between the clean crushed rock and the slab. It may be prudent to place an additional 2 inches of compacted clean sand over the membrane to protect it during construction, provided the sand thickness and separation from the steel remains uniform through concrete placement. The slabs should contain control joints to help control the distribution of cracking should it occur.

Pavements

Pavements should be placed on a uniform surface of bedrock or competent soil. Where pavements span a soil-bedrock interface, we recommend that the bedrock be over-excavated a minimum of 12 inches and replaced as engineered fill. Soil pavement subgrades and areas of overexcavation, prior to placement of fill, should be scarified to a depth of 8 inches and re-compacted to a minimum of 90 percent MDD and a moisture content of approximately 3 percent above optimum. Final pavement design will be dependent upon the anticipated traffic and the materials exposed at the subgrade levels. Table 1 defines preliminary, conservative pavement sections, in inches, for various traffic indices. Final pavement design can be evaluated after representative samples of exposed subgrade materials are provided for R-value testing.

Table 1. Preliminary Pavement Design

Traffic Index	Asphaltic Concrete	Aggregate Base	Aggregate Sub-Base
4.0	2.0"	8.5"	--
4.5	2.5"	9.0"	--
5.0	2.5"	10.0"	--
5.5	3.0"	11.5"	--
6.0	3.0"	6.0"	8.0"
6.5	3.5"	6.5"	9.0"
7.0	3.5"	7.0"	10.0"

Retaining Walls

The foundation type specified for slope conditions in *Foundation* section should support retaining walls. Retaining walls should be designed for an active equivalent fluid pressure of 45 pcf for level backfill and 60 pcf for backfill sloping up to maximum of 2H:1V. Any wall that is restrained from rotation should be designed to resist an additional uniform pressure of 100 psf. Walls supporting roadways should be designed to resist an additional surcharge of 1/3 the applied load acting on the top portion of the wall. Where seismic parameters are required, they should be designed for a pressure equal to 15H psf, where H is the height of the retained soil. The seismic component should be considered a load acting 0.5 times the wall height above the wall base. Add a uniform pressure of 250 psf to wall design If the basement wall will be subjected to passenger vehicle loading.

The pressures described above are contingent upon the basement walls being constructed with a backdrainage system. We recommend that the backdrain pipe be located at least 1 foot below the adjacent lowest grade to mitigate underseepage toward the basement house foundation. The backdrain should consist of a geosynthetic drainage mat (i.e., Miradrain 5000 or equivalent) integrated with a minimum 4-inch diameter, perforated SDR 35 PVC pipe (or better) in accordance with the manufacture's specifications, and sloped to drain to a sump and pump system designed by the project civil engineer.

We recommend that basement foundation walls be thoroughly waterproofed to prevent detrimental migration of moisture and potential development of unsightly precipitation on the wall face.

Retaining walls should be fully backdrained. Retaining wall backdrains should consist of either a geosynthetic drainage mat and properly placed perforated pipe (as specified by the manufacturer, i.e., Miradrain 5000 or equivalent), or of 4-inch diameter, high crush strength, perforated PVC pipe sloped to drain to outlet by gravity, and of clean, free draining crushed rock. If the crushed rock or gravel alternative is chosen, it should consist of a prism no less than 12 inches wide that extends to within 1 foot of the surface. The upper foot should be backfilled with compacted soil to exclude surface water. Drainrock should be separated from the soil by filter fabric. Retaining wall backdrainage should be directed to the storm drain. The uphill and sidehill foundation wall backdrain perforated pipes should be located at least 12 inches below the proposed crawl space or 8 inches below pavement sections and bottom of garage slab subgrades.

Retaining walls should be thoroughly waterproofed to prevent detrimental migration of moisture. Retaining walls will yield slightly during backfilling; therefore, walls should be backfilled prior to building on or adjacent to them.

We recommend that the ground surface behind retaining walls be sloped to drain in a positive manner so that ponding and erosion does not occur. Open, reinforced concrete lined gutters should be designed to provide surface drainage control behind the retaining walls. The gutter(s) should be tied to minimum 12-inch square, steel grate covered catch basin. In turn, the basin should be connected to a solid pipe that carries water from the catch basin to the storm drain. Surface water should not be diverted into subdrains.

Drainage

The project roadways should be provided with adequate catch basins designed by the project civil engineer for storm drainage control. Building areas should be provided with positive surface drainage gradients of at least 3 percent for a distance of at least 5 feet away from all structures. Driveways and paved parking areas should drain positively to the street storm drainage system or other approved location away from pavement subgrades and building foundations. It may be necessary to install properly sized area drains to achieve this.

We recommend that the house roofs be provided with gutters and downspouts. The downspouts should be connected to solid PVC pipes and these pipes should carry water to the street. Care should be taken to avoid discharge of drainpipes on slopes or adjoining private property.

Where water seepage is observed during rough grading, it would be necessary to install subdrainage to capture and reroute seepage to an appropriate discharge point away from the proposed development. The field engineer will assess the need for and distribution of subdrainage during rough grading.

We recommend installation of foundation drains on the uphill sides of proposed building foundations that are not foundation retaining walls, and closed depressions adjacent to pavements. The need for shallow subdrainage can be assessed during grading to remove seepage from potential areas of surface ponding. The subdrains should extend to a depth of at least 12 inches below the crawl space elevation and 8 inches below pavement sections. The subdrainage trench should be faced with filter fabric (Mirafi 140N or better), and the bottom sloped at least 2 percent in the direction of outfall. A minimum 4-inch diameter, rigid perforated drainpipe, laid holes down, should be placed at the bottom of the trench with a minimum slope of 2 percent to drain by gravity to an approved discharge point. The trench should then be filled to within 6 inches of the surface with ¾ -to 1 ½ -inch drainrock. Place filter fabric over the top of the drainrock and fill the balance of the trench with compacted site soil. The finished grade over the trenches should slope at least 3 percent away from the foundations. Areas where this is not feasible should be provided with a well-developed surface drainage catch basin seated in a well-defined ground depression having positive slopes to the inlet. Surface inlets should be at least 12 inches square.

All drainage material should conform to at least SDR-35 PVC, or Schedule 40 PVC material where heavy loads (i.e., vehicles) will be imposed on shallow drainage facilities. The perforated pipe leading from the subdrains should be connected to equivalent solid PVC pipes to carry water to an approved discharge point. For future monitoring and maintenance, cleanouts constructed of the approved PVC material shall be provided in buried drainlines at all bends greater than 45 degrees, and at distances not exceeding 50 feet.

While we believe that subdrainage will reduce soil moisture beneath buildings, it would be prudent to install wire-mesh reinforced, concrete ratproofing over the crawl space soils.

Landscaping and Erosion Control

Planting a dense tree canopy where practical can moderate desiccation of the soil surfaces of the project area. However, to mitigate potential effects of root growth under foundations, any proposed new trees should be planted at least a distance from the foundations equal to or greater than 1 ½ times the anticipated dripline of a mature tree. We suggest that you confirm this criterion with the landscape architect.

It is important to plan landscaping to reduce high-maintenance plantings adjacent to the foundations as they can promote infiltration and seepage of moisture into the foundation and crawl space soils. The landscape contractor should be made aware of the importance of these recommendations. Strict adherence is imperative.

Following construction, barren soil surface should be planted to reduce erosion and soil desiccation cracking.

SUPPLEMENTAL SERVICES

We recommend that we review the final foundation, grading and drainage plans for conformance with the intent of our recommendations. During construction, we should observe the rough and finished grading operations, foundation excavations prior to steel placement, and the installation of all drainage facilities prior to burial to ascertain that our recommendations are followed. Upon completion of the project, we should perform a site observation and report the results of our work in a final report. These services are outside the present scope and will be billed on a time and materials basis, in accordance with the fee schedule current at that time. These services will be performed only if we are provided with sufficient notice to perform the work. We do not accept responsibility for items that we are not notified to observe. We recommend that the Owner be responsible for notification, no less than 48 hours before the requested site visit.

INVESTIGATION LIMITATIONS

This report has been prepared in accordance with generally accepted geotechnical engineering principles and practices, and is in accordance with the standards and practices set by the geotechnical consultants in the area. This acknowledgment is in lieu of any warranties, either expressed or implied. We offer no guarantees.

Subsurface conditions could vary between those indicated by the test borings and interpreted from surface features. A representative from this office should be present to provide construction observation services, to observe the exposed geotechnical conditions, to modify recommendations, if necessary, and to ascertain that the project is constructed in accordance with the recommendations.

This report is submitted with the understanding that it is the responsibility of the Client (Owner) to ensure that the applicable provisions of the recommendations contained herein are made known to all design professionals involved with the project; that they are incorporated into the construction drawings; and that the necessary steps are taken to see that the contractor and subcontractors carry out such recommendations in the field.

If conditions different from those described in this report are encountered during construction, or if the project is revised, we should be notified immediately so that we may modify our recommendations, if warranted.

The practice of geotechnical engineering changes, and, therefore, we should be consulted to update this report if construction is not performed within 12 months.

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AERIAL PHOTOGRAPHS

Source: Soil Conservation Service, United States Department of Agriculture

Date	Job No	Flight Line	Frames	Scale
1943	DDB	2B	180-181	1:20,000
1956	DDB	1R	66-67	1:20,000

Source: Pacifica Aerial Surveys, Oakland, California

1946	AV 9	3	3- 4	1::23,600
1955	AV170	5	24-26	1:10000
1961	AV132	4	23-24	1:12,000

Source: United States Air Force (U.S. Geological Survey, Menlo Park)

1970	GS-VCMI	2	183-184	1:80,000

APPENDIX B

This appendix contains an update and supplemental recommendations to the 2013 EIC report.

APPENDIX B

Project Geotechnical Report Update & Supplemental Recommendations

Conclusions

Validity of Existing EIC Project Reports

Based on review of the currently proposed details for the Glenview Terrace residential development, it is our opinion the findings, conclusions, and recommendations in the EIC 2013 project geotechnical report (Appendix A), coupled with the updated conclusions and recommendations presented herein, provide sufficient geotechnical input for project design and construction.

Site and Off-Site Stability

On the basis of the supplemental slope erosion assessment presented following the Reply to Peer Review, we conclude the very dense woody vegetation covering provides effective erosion protection and surficial stability to the native slope between the eastern property boundary and San Bruno Creek. The undocumented fill currently involved in creep in the existing residence area, and locally elsewhere to the southeast corner of the property, will be effectively mitigated by remedial grading and stitch pier recommendations presented in Appendix A and below.

The active landslide by Cotton, Shires, and Associates (2008) depicted on Figure 1 coincides with the surface inflection of the northeast trending swale marked by surficial debris slide source areas observed on the east-central margin of proposed project area (Plate 3). However, the geologic section observed in exploratory trench T-1 (EIC, 2008) revealed no evidence of retrogressive, deep-seated landslide movement from either of the mapped features. Evidence of surficial soil creep, obscured by dense vegetation, was detected on the steeper slope segment extending beyond the eastern side of the proposed development area.

Updated Geotechnical Recommendations

Seismic Design Parameters

Previous seismic design parameters are updated for 2019 California Building Code (CBC). The proposed development (37.6195°N, 122.4409°W) should be designed in accordance with local design practice to resist the lateral forces generated by ground shaking associated with a major earthquake occurring on the San Andreas Fault and others within the greater Bay Area.

Based on the subsurface conditions encountered in our borings and our evaluation of the geology of the site, Site Class “C”, representative of very dense soil and soft rock averaged over the uppermost 100 feet of the subsurface profile, would be appropriate for this site. For seismic analysis of the proposed site in accordance with the seismic provisions of the 2019 CBC, we recommend the seismic ground motion values in Table 1 be used for design.

Table 1: Seismic Coefficients Based on 2019 CBC (per ASCE 7-16)

Item	Value	Source 2019 CBC Source[R1]	Source ASCE 7-16 Table/Figure[R2]
Site Class	**C**	**Section 1613.3.2**	**Table 20.3-1**
Mapped Spectral Response Accelerations			
Short Period, S_s	2.489 g		Figure 22-1
1-second Period, S_1	1.043 g		Figure 22-2
Site Coefficient, F_a	1.2	Table 1613.3.3(1)	Table 11.4-1
Site Coefficient, F_v	1.4	Table 1613.3.3(2)	Table 11.4-2
MCE (S_{MS})	2.986 g	Equation 16-37	Equation 11.4-1
MCE (S_{M1})	1.461 g	Equation 16-38	Equation 11.4-2
Design Spectral Response Acceleration			
Short Period, S_{DS}	1.991 g	Equation 16-39	Equation 11.4-3
1-second Period, S_{D1}	0.974 g	Equation 16-40	Equation 11.4-4
Peak Ground Acceleration (PGA_M)	1.276 g	-	Equation 11.8-1

R1 California Building Standards Commission (CBSC), "California Building Code," 2019 Edition.
R2 Applied Technology Council (ATC) Web Application.

Site Drainage

Please refer to the project geotechnical report (Appendix A) for acceptable project drainage measures.

We informed the Civil Engineer that concentrated stormwater infiltration at top-of-slope bio-retention basins proposed for the northeastern and southwestern corners of the property is inappropriate given the steep slopes that drain to San Bruno Creek are underlain with weak soils susceptible to erosion and surficial landsliding. Therefore, we recommend the basins designed as water-tight facilities that drain filters runoff to the municipal storm drain, if allowed by the Public Works Department. If outfall to the municipal storm drain is disallowed, then we recommend the required permeable bioretention basins be re-located at least 50 feet from the top of the slope steeper than 4H:1V descending to San Bruno Creek.

Lacking any other viable stormwater management system, a potentially geotechnical feasible alternative to surface retention may include metering filtered stormwater from the existing bioretention basin locations to dry wells designed by the Civil Engineer with pertinent geotechnical infiltration and setback input from Geosphere.

Remedial Grading

Supplemental to *Site Preparation, Grading and Compaction* recommendations presented in Appendix A, we recommend the existing undocumented fill, and any areas of potentially adverse/unstable surficial native soil underling existing fill exposed during mass grading on the northeastern part of the development area, and locally elsewhere, be removed and replaced as engineered fill up to the new proposed development site grades, as assessed by the Field Engineer. We judge the grading operation as described will mitigate existing and potential instability that exists in that area. Where required, as assessed by the Field Engineer, over-excavation of deleterious soils should extend a minimum of 10 feet beyond proposed structure foundations, or at least 5 feet beyond the soil-bedrock contact.

Excavated material meeting the specifications presented in Appendix A may be reused in structural fill. Engineered fill should be keyed and benched into the underlying competent soil or bedrock after the exposed support surface has been approved by the Engineering Geologist.

Foundations

Rigid Mat (Buildings) – *The project Architect and Structural Engineer can evaluate the feasibility of a reinforced concrete mat foundation for support of the proposed residential buildings. The mat foundation should be at least 12 inches thick, and gain support on a pad prepared as described in the Grading section presented in the August 4, 2013 report. The mat should be designed for a modulus of subgrade reaction of 150 pounds per cubic inch (pci), and capacity for an unsupported span of at least 8 feet and cantilever of 4 feet.*

Stitch Piers Retention System (Slope Stability Enhancement) – To enhance respective static and seismic Factors of Safety (Appendix G) and mitigate potential localized future slope regression by surface erosion, we recommend a row of stitch piers be installed on the eastern margin of the development area where a descending slope exceeds an inclination of 3H:1V. The Project Engineer should design the stitch pier retention system in accordance with the following geotechnical parameters:

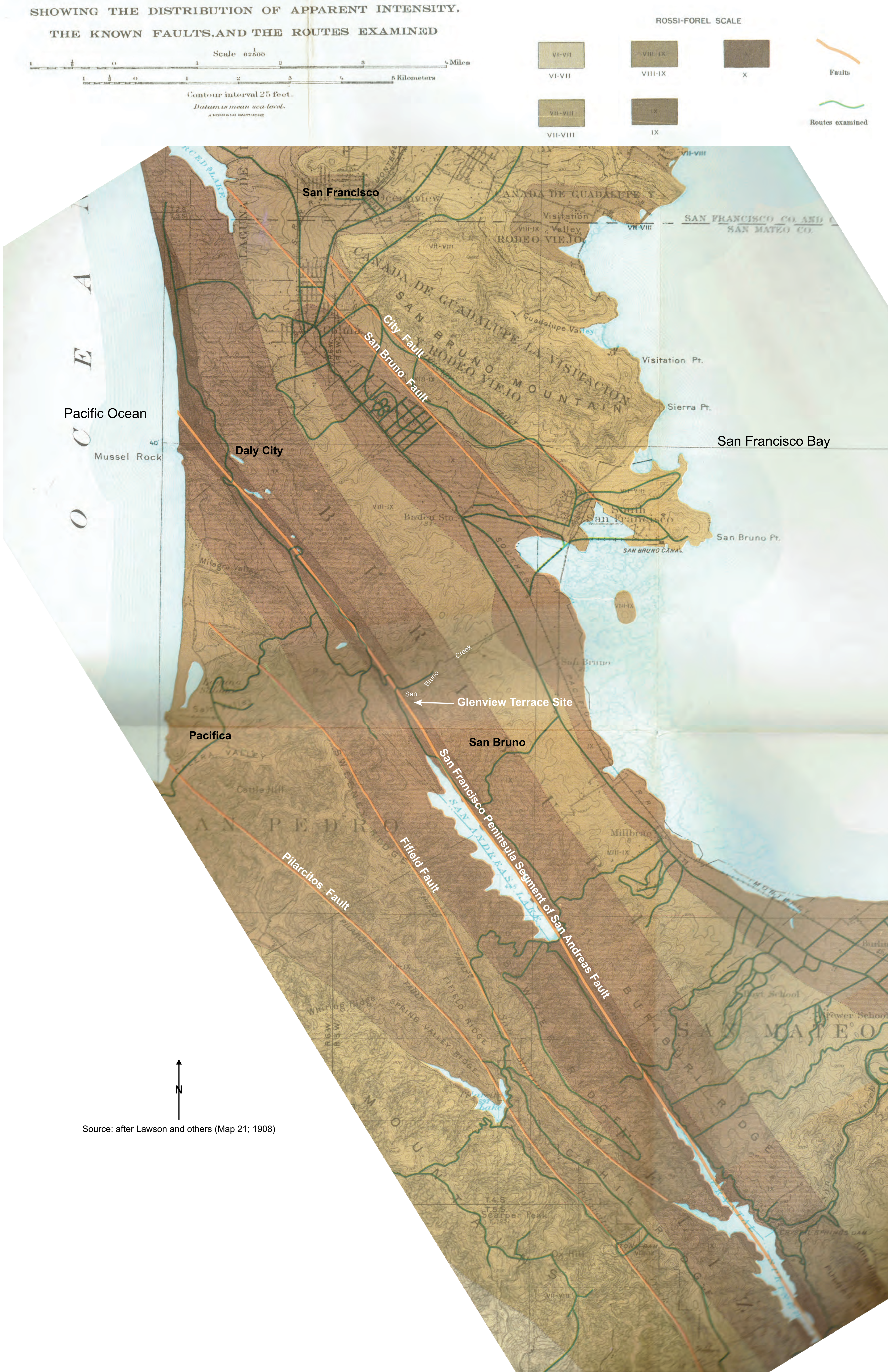
- Drilled cast-in-place reinforced concrete piers should have a minimum diameter of 24 inches, spaced 6 feet center to center, and extending a minimum of 5 feet into bedrock, as assessed by the field engineer at the outset of the pier drilling operation;
- Where a slope segment has not been remediated with engineered fill, we recommend piers be designed for a creep force of 45 pounds per cubic foot (pcf) applied over 1½ pier diameters in the upper 3 feet of the pier shaft;
- Lateral earth passive pressure of 500 pcf Equivalent Fluid Pressure acting over 1½ pier diameters (ignore contribution of pier length where lateral distance to exposed slope face is less than 10 feet);
- Skin friction of 450 pounds per square foot beginning 5 feet below the ground surface.
- Tops of the piers should be interconnected with a minimum 24-inch square, reinforced concrete grade beam.

Retaining Wall Recommendations

For seismic design, the uniform pressure value presented in the 2013 EIC report should be increased to 22H psf, where H equals the height of the retained soil. This seismic load should be added to the *unrestrained* condition for both restrained wall and unrestrained wall cases. That is, for a level backfill slope, the lateral active pressure under the design seismic loading should be 45 pcf Equivalent Fluid Pressure (EFP) plus a uniform pressure of 22H psf applied to the back of the wall through the retained height of the wall for both unrestrained and restrained cases.

APPENDIX C

This appendix contains an excerpt from the Lawson and others (1908) documentation of faults surface rupture from the 1906 Earthquake between Mussel Rock and Crystal Springs Reservoir.



Source: after Lawson and others (Map 21; 1908)

THE CALIFORNIA EARTHQUAKE OF APRIL 18, 1906

REPORT

OF THE

STATE EARTHQUAKE INVESTIGATION COMMISSION

IN TWO VOLUMES AND ATLAS

VOLUME I

BY

ANDREW C. LAWSON, CHAIRMAN

IN COLLABORATION WITH G. K. GILBERT, H. F. REID, J. C. BRANNER, H. W. FAIRBANKS, H. O. WOOD, J. F. HAYFORD AND A. L. BALDWIN, F. OMORI, A. O. LEUSCHNER, GEORGE DAVIDSON, F. E. MATTHES, R. ANDERSON, G. D. LOUDERBACK, R. S. HOLWAY, A. S. EAKLE, R. CRANDALL, G. F. HOFFMAN, G. A. WARRING, E. HUGHES, F. J. ROGERS, A. BAIRD, AND MANY OTHERS

VOLUME I, PART I

WASHINGTON, D. C.
PUBLISHED BY THE CARNEGIE INSTITUTION OF WASHINGTON
1908

Reprinted 1969

MUSSEL ROCK TO CRYSTAL SPRINGS LAKE.

Course of the fault-trace. — The point at which the fault-trace intersects the shore, on emerging from the ocean on the south side of the Golden Gate, is only approximately known. About 0.875 mile to the southeast of Mussel Rock, it has been located with precision at its intersection with the wagon road on the west side of the coastal ridge a little below its crest, and thence followed continuously for many miles. Projecting its course, there determined, in a northwesterly direction, it would pass out to sea in the midst of the large landslide which scars the coast immediately to the north of Mussel Rock, where the basal beds of the Merced series rest upon the older rocks. At the time of the earthquake there was an extensive movement of the landslide, and a tongue of landslide material, about 50 feet high and about 200 feet wide, was projected into the ocean across the narrow strip of beach.[1] This movement naturally obscured all evidence of the position of the fault-trace, which was doubtless overridden by the slide. All about the crest to the east of the landslide, and on its south side, the ground was greatly disturbed by fresh landslide cracks, scarps, and fissures, extending well back from the edge of its encircling cliffs. It appears to be probable that not only did the movement of the landslide obscure the evidence of the fault-trace, but also that the latter was here diffuse and scattered, and that the displacement was superficially taken up by the plasticity of the landslide material.

From the point southeast of the Mussel Rock slide where the fault-trace resumes its definite and easily recognizable character, to Crystal Springs Lake, our information regarding the course of the fault-trace and the earth movement on the fault is in part from observations made by Mr. Robert Anderson, and in part from observations recorded in a paper by Herman Schussler,[2] supplemented by the observations of Mr. H. O. Wood, Andrew C. Lawson, and others.

South of the road, at a point 0.875 mile southeast of Mussel Rock, begins the furrow which marks the surface path of the fault. The furrow as such does not cross the road to the north of this point. The side-hill slope, however, is very much fissured by landslide movements both above and below the road, and scarps are seen. From this point, the furrow runs uninterruptedly southeastward to the east side of the north end of San Andreas Lake, where, with a course of about S. 33° E., it passes beneath the waters of that reservoir. As it approaches the lake, the trace of the fault does not lie in the axis of the valley, but runs along its eastern side. It thence passes thru the lake on the northeast side, crossing a number of small promontories, to the east end of San Andreas dam; thence, with a course of S. 37° E., it traverses the east side of the valley between this dam and Lower Crystal Springs Lake, passes thru the latter and intersects the old dam between Upper and Lower Crystal Springs Lakes. Beyond this it skirts the southwest side of the upper lake, partly in the water and partly on the projecting points, and finally leaves the lake about a 0.25 mile from its end, for the stage of the water of April, 1906, having here a course of S. 40° E.

The mean course of the fault, as thus closely followed from the vicinity of Mussel Rock to the end of Upper Crystal Springs Lake, a distance of about 15 miles, is S. 36° 30′ E. But the trace is not a perfectly straight line. Between Mussel Rock and San Andreas

[1] On February 27, 1907, according to the observations of Mr. H. O. Wood, this projecting tongue of landslide had been entirely removed by the action of the waves, and alinement of the beach and sea-cliff had been reëstablished.

[2] The Water Supply of San Francisco before, during, and after the Earthquake of April 18, 1906, and the Subsequent Conflagration. New York, 1906.

dam, the trace of the fault is slightly concave to the straight line connecting these two points on the fault, the concavity being to the southwest. Between San Andreas dam and the end of Upper Crystal Springs Lake, the trace of the fault is again slightly concave to the straight line between these points, but is on the opposite side of the fault, the concavity here facing the northeast.

Characteristics of the fault-trace. — For this stretch of from 14 to 15 miles, Mr. Robert Anderson, who examined this territory under direction of Prof. J. C. Branner, describes the trace of the fault as marked by a belt of upturned earth resembling a gigantic mole-track. The rupture may be traced along every foot of the way when not below the waters of the lakes. It varies in width from 2 or 3 feet to 10 feet, but at times branches out into several furrows that include a space of 100 feet or more in width. Such branches sometimes join again after a short interval. Sometimes it forms a crack 2 or 3 feet wide and several feet deep, and in other places shows a vertical wall of soil on one side or the other, several feet high. The typical occurrence in turf-covered fields is a long, straight, raised line of blocks of sod broken loose and partly overturned. It is thus shown in plate 61A, B.

Associated with the fault fractures are many lateral cracks, extending away from the fault in a northward, or north slightly eastward, direction; that is, at an oblique angle to the northeast side. These cracks were especially abundant along the northeast side of the northern half of Crystal Springs Lake, and between there and San Andreas Lake. In places they run off every foot or few feet for a distance of 100 yards or more, and again they do not form for some distance. They vary in size from minute crevices in the earth to fractures a foot or more in width. Here and there they form lines of broken sod very like the main furrow in size, while they have a length of from a few feet to several hundred feet. At the great dam at the head of San Mateo Canyon, these cracks emerged from the lake and ran northward up on the hills for several hundred yards, breaking the fences where they crost. Plate 16A shows large lateral cracks of this description, already partly filled up, crossing a road that runs parallel to the fault at the upper end of Crystal Springs Lake. The main line of fracture is about 50 feet beyond the fence, and the cracks extend into the foreground at an angle of from 35° to 40° with the main fault-trace. The fence is pulled apart 40 inches in the two places which are shown in the photograph, and a total of 10 feet in ten different breaks in this locality, within a distance of 200 yards. Such lateral cracks as these were not noted on the southwest side of the fault.

The lateral cracks described above make an angle of 45° to the general line of the fault fracture. They appear to have been produced very much like the fracture lines in compression tests of building stones. There was evidently great pressure holding together the two faces along the fracture. A dam made of earth and rock divides Crystal Springs Lake into two parts. This dam crosses the fault-trace at right angles, and was offset but not badly cracked or injured by the movement. The fences that line the road were warped and their boards buckled thruout the distance across the dam. The earthquake rendered them too long for the distance from the hills on one side of the valley to those on the other. The inference is that a strong compression took place. The slicken-siding shown in plate 62A furnishes further evidence of compression. In the same way the heaving up of the sod into a long, raised mound, for most of the extent of the furrow, suggests lateral pressure. The formation of cracks a few inches to 2 or 3 feet wide in places along the furrow seems to contradict the theory of compression; but these are regarded as due to the irregular, crooked fracturing of the surface and the faulting of irregularities into juxtaposition with one another near the surface. The open cracks were never found to be of great extent, but were usually followed by stretches along which the earth was heaped up into a mound, as if by being prest together. The surface furrow indicates that there was a zone of crushing some 2 or 3 or more feet wide. Where a similar cross-section of the fault is viewed from the opposite direction, no such face is exhibited on the northeast side, but instead a mass of crusht earth projecting beyond its former position.

Offsets on fences, pipes, dams, etc. — About a mile southeast from the point near Mussel Rock where the furrow was first noted as a clearly defined feature, the fault-trace passes thru the trough of a well-marked saddle. This feature is more accentuated than similar features at other points along this portion of the rift, tho many such are found. Southeast from this saddle there is recognizable in the topography a distinct line of former movement, lying east of the fault. No furrow follows the line continuously, but an occasional

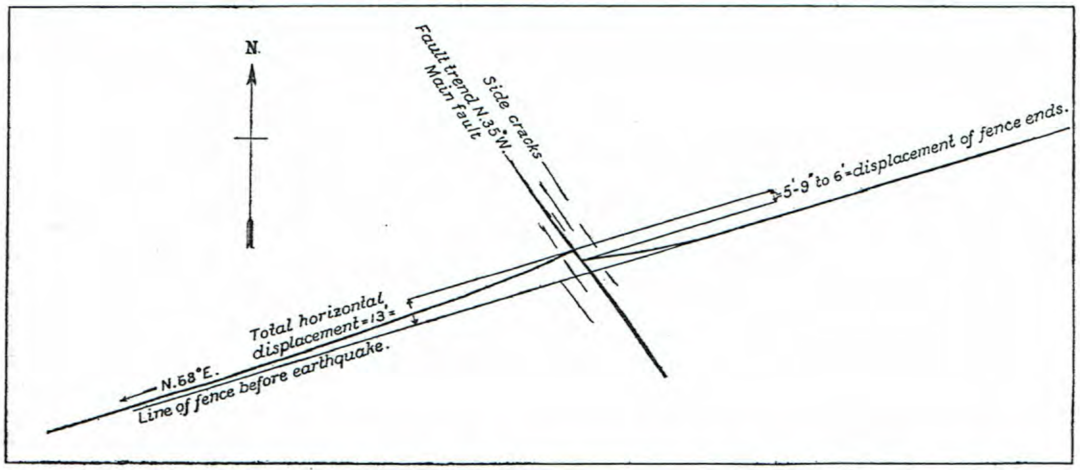


FIG. 29. — Offset fence southeast of Mussel Rock, showing distribution of deformation on either side of fault.

short fissure or crack runs along it for a little way. To the west of the place is a similar, but less well marked, topographic indication of a former movement. There is no evidence of any movement on this line at the time of the earthquake. At the point where the fault-trace crosses the road, less than half a mile farther on, the roadway and fence were broken, but the effects were so confused that the measure of the offset could not be determined. The apparent horizontal displacement was slight.

Still farther to the southeast, about 1.25 miles, the fault intersected a fence and not only caused it to be offset, but the intersection showed clearly the effect of the drag in the earth movement. The bearing of the fence is N. 68° E., so that it is approximately transverse to the line of the fault. On the west side of the latter, the fence suffered a displacement to the northwest of 13 feet from its former position, and this displacement was effected by a bending or curvature in the fence line extending westerly from the fault for a distance of over 200 feet. On the east side of the fault, the fence was bent away from its former position, *in the same direction*, about 7 or 7.25 feet, the bent portion extending easterly from the fault-trace about 45 feet. The two ends of the fence were thus offset on the line of the fault only 5.75 to 6 feet, altho the total displacement was 13 feet. The displacement is shown diagrammatically in fig. 29. At a point 330 yards beyond this, on the Rift, the fault-trace was found to be confined to a furrow about 6 feet wide, passing thru a little trough between an outcrop of Franciscan on the west and a fine conglomerate (Merced) on the east.

Nowhere along this portion of the fault-trace between the slide at Mussel Rock and San Andreas Lake was there observed any definite evidence of vertical displacement. There was a hint of slight upthrow on the western side, but it could not be tested by measurement. There were, in general, furrows on either side of the main fault, at various distances up to 200 feet. Some of these were persistent for considerable distances.

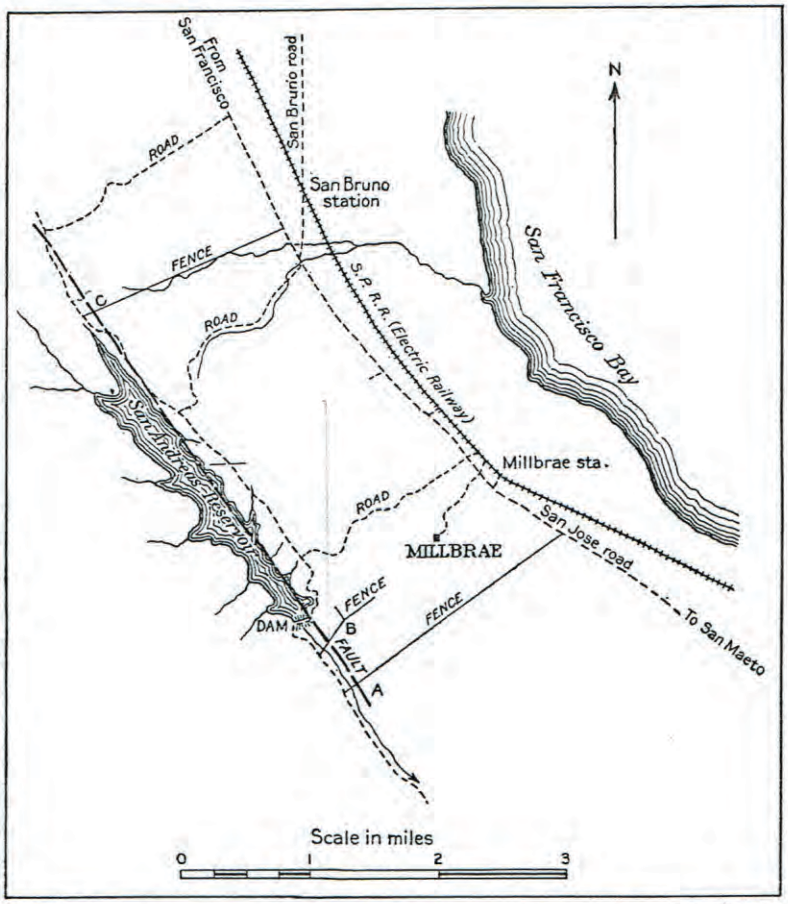


FIG. 30.—Index map showing positions of three fences, *A*, *B*, and *C*, the offsets of which are shown in figs. 31, 37, and 38.

About 2 miles from the upper end of San Andreas Lake the fault encounters the 30-inch, laminated, wrought-iron pipe of the Spring Valley Water Company, which prior to the earthquake conveyed the water from Pilarcitos Lake to San Francisco. The metal of the pipe is about 0.1875 inch thick and coated with asphaltum. The pipe is buried in the soil at a depth of 3 to 4 feet. The point of intersection is near Small Frawley Canyon. Here the course of the pipe swings from a northwesterly to a more northerly course, and the fault consequently intersects it at an acute angle. At the point of intersection, the pipe was obliquely sheared apart and telescoped upon itself, effecting a shortening of about 6 feet. The amount of the transverse offset involved in the shear was about half the diameter of the pipe. The portion north of the break was moved east and telescoped southerly. For 0.875 mile southeast of this point, the path of the fault lay on the northeast side of the pipe and nearly parallel to it, but a short distance away. About 220 yards southeast of the intersection, where the pipe, buried a few feet below the surface, ascends a rising slope, the pipe had completely collapsed for a distance of several yards, due doubtless to the establishment of a partial vacuum within the pipe by the sudden withdrawal of the water from the arch in the pipe at the time of the shock, owing either to the leakage below, or the propulsion of the water induced by the shock. (See plate 60B.)

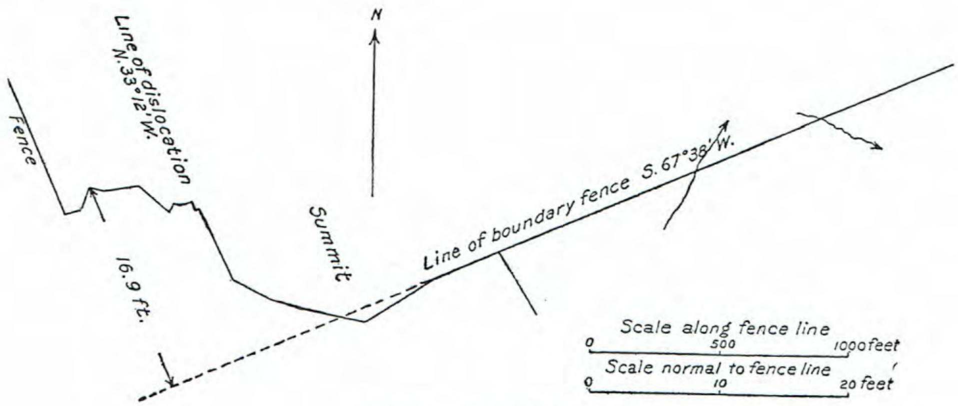


FIG. 31.—Fence C of fig. 30.

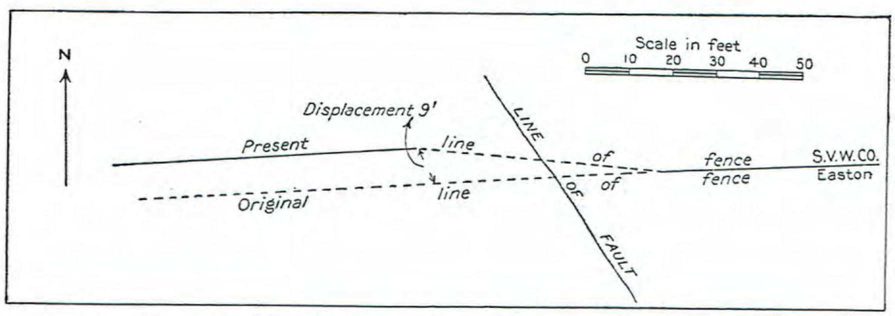


FIG. 32.—Dislocation of fence near San Andreas Lake. After H. Schussler.

At a point about a mile from the upper end of San Andreas Lake, the fault intersects a bend in the pipe at two places, and here again the pipe was telescoped. (See plate 60A.) The conditions at one of these intersections are thus described by Mr. Robert Anderson:

The pipe makes an angle of about 15° with the fault-trace, the end of the pipe on the north side of the fault running that much nearer the north. The ends of the pipe on opposite sides of the fracture were therefore thrust into each other. The furrow was at this place divided into several smaller ones, the disturbed zone covering an area of considerable width. The pipe was broken in three places within 100 feet. In one place it was telescoped 58 inches, as shown in plate 59B; in another 17 inches, and in a third, the one farthest north, 41 inches.

Near the head of the lake, the pipe was again intersected by the fault, with results described by Mr. Anderson as follows:

The pipe line runs almost parallel with the fracture, but slightly more to the west at this point, so that the acute angles made by the ends of the pipe with the furrow were in this case on opposite sides of the furrow to those in the two previous instances. In other words, the southeast end of the pipe was farther to the east than the southeast end of the furrow. The movement was in the same direction as before, therefore a pulling apart of the pipe took place instead of a compression. There occurred two breaks in the pipe (see plate 59A), the main one at the crossing of the fault, and the other 150 yards away on the northeast side of the fault, but very near it, the pipe being almost parallel with it. At the main break, the pipe was pulled apart 59 inches, and at the other one 21.5 inches, making a total displacement of 6.666 feet. The pipe was not quite parallel with the fault and therefore there was a slight offset, at right angles to its direction, of 4 inches at the main break and 2 inches at the minor one, or a total of 6 inches. A fence which crost the fault at the main break is offset 6.5 feet. (Plate 60c.)

The index map, fig. 30 (p. 95), indicates the position of three dislocated fences which were surveyed by R. B. Symington, C.E. The fences are marked *A*, *B*, *C*. One of these fences, *C*, near the upper end of San Andreas Lake, is nearly normal to the trace of the fault, and its deformation extends over a zone 1,200 feet wide, the total displacement aggregating 16.9 feet. Here, as usual, the portion on the southwest side of the fault moved relatively to the northwest, but there was a distinct drag on the northeast side in the same direction. (See fig. 31.)

The offsets in three other fences southeast of San Andreas Lake are shown in figs. 32, 33, and 34 and plates 60D and 61B.

Thruout this 2-mile stretch within which the pipe line nearly parallels the fault-trace, the path of the latter is strongly marked by a prominent furrow in the sod, with the usual diagonal cracks and variable width. This furrow lies on the northeast side of the

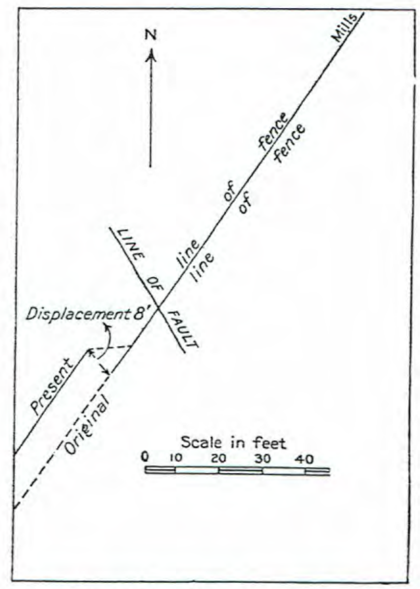


FIG. 33.—Dislocation of fence near San Andreas Lake. After H. Schussler.

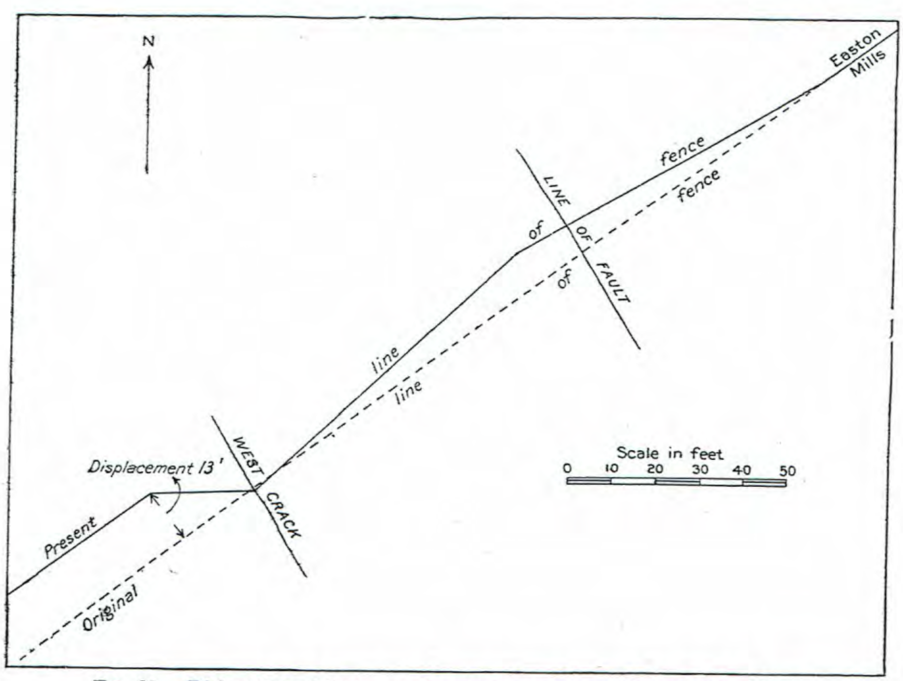


FIG. 34.—Dislocation of fence near San Andreas Lake. After H. Schussler.

lake for the first 0.875 of a mile of its length. It then enters the water (plate 61D) and follows the northeast side of the lake, a little distance from shore, to the San Andreas dam at the lower end of the lake. In this distance of nearly 2 miles, the fault-trace emerges from the water at a number of points where little capes project into the lake. The crossing of these capes by the fault-trace indicates that it follows a very straight course beneath the water of the lake. On the last of these promontories traversed by the fault, the main fault-trace has associated with it a number of auxiliary cracks. Between the main fault-trace and one of the diverging cracks, on the southwest side of the fault, is a brick and cement gate-well in connection with the tunnel which takes the waters from the lake toward Millbrae. This gate-well was circular in cross-section, the inside diameter being about 26 feet. The nearest point of the structure to the main fault-trace is within 5 feet.

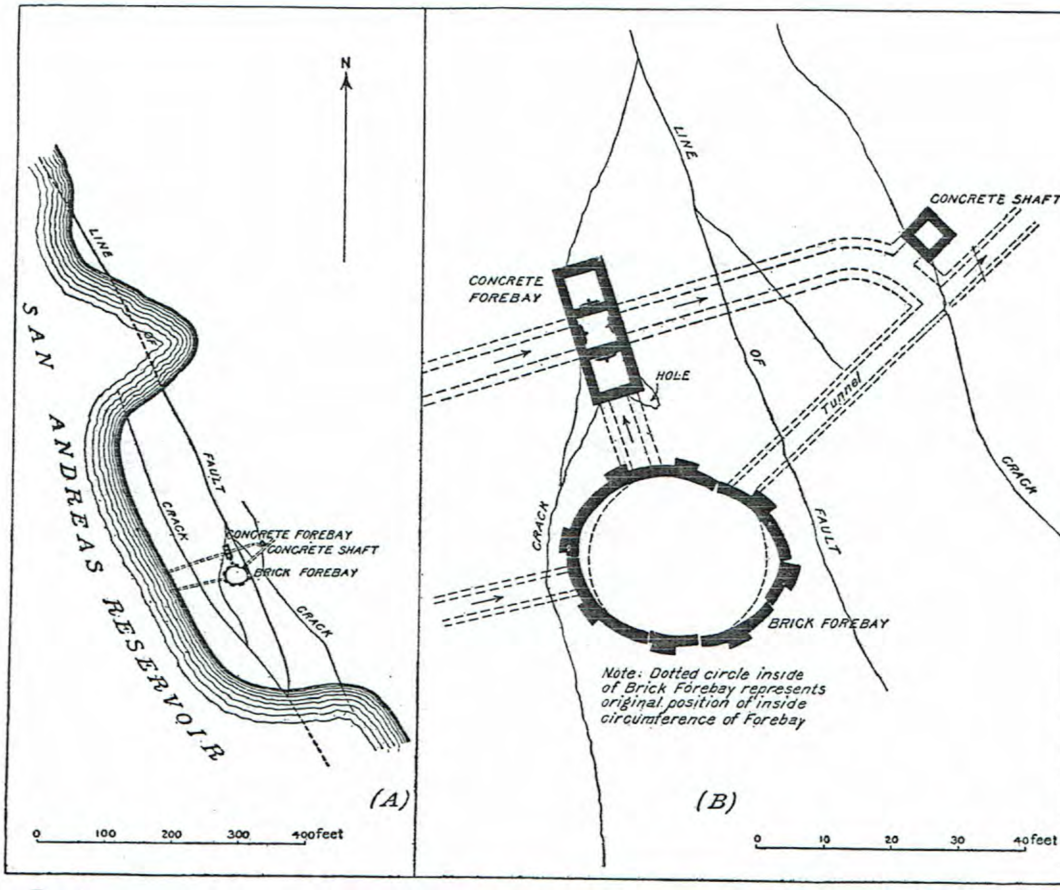


FIG. 35. — Main and auxilia y faults, San Andreas Lake. *A*. General Plan. *B*. Detail. After H. Schussler.

The walls are about a foot thick, and are strongly buttressed. As a result of the shock this gate-well was shattered and deformed so that it became oval in cross-section, the east and west diameter becoming 30 feet and the north and south diameter about 21 or 22 feet, as shown in the accompanying figure. A new concrete gate-well a few feet to the north, rectangular in cross-section and having three compartments, each 2.5 × 2.5 feet, was uninjured, altho on the line of the same branching crack. A concrete manhole 45 feet northeast of the damaged gate-well, also on an auxiliary crack, was similarly unaffected. (See fig. 35.)

At the San Andreas dam, the fault past thru a rocky knoll which serves as an abutment for the dam on both sides, the embankment being in 2 parts. The rocks were shattered and the road over the dam and the fence paralleling it were offset several feet in the usual direction. The ground here was traversed by several cracks, those on the south-

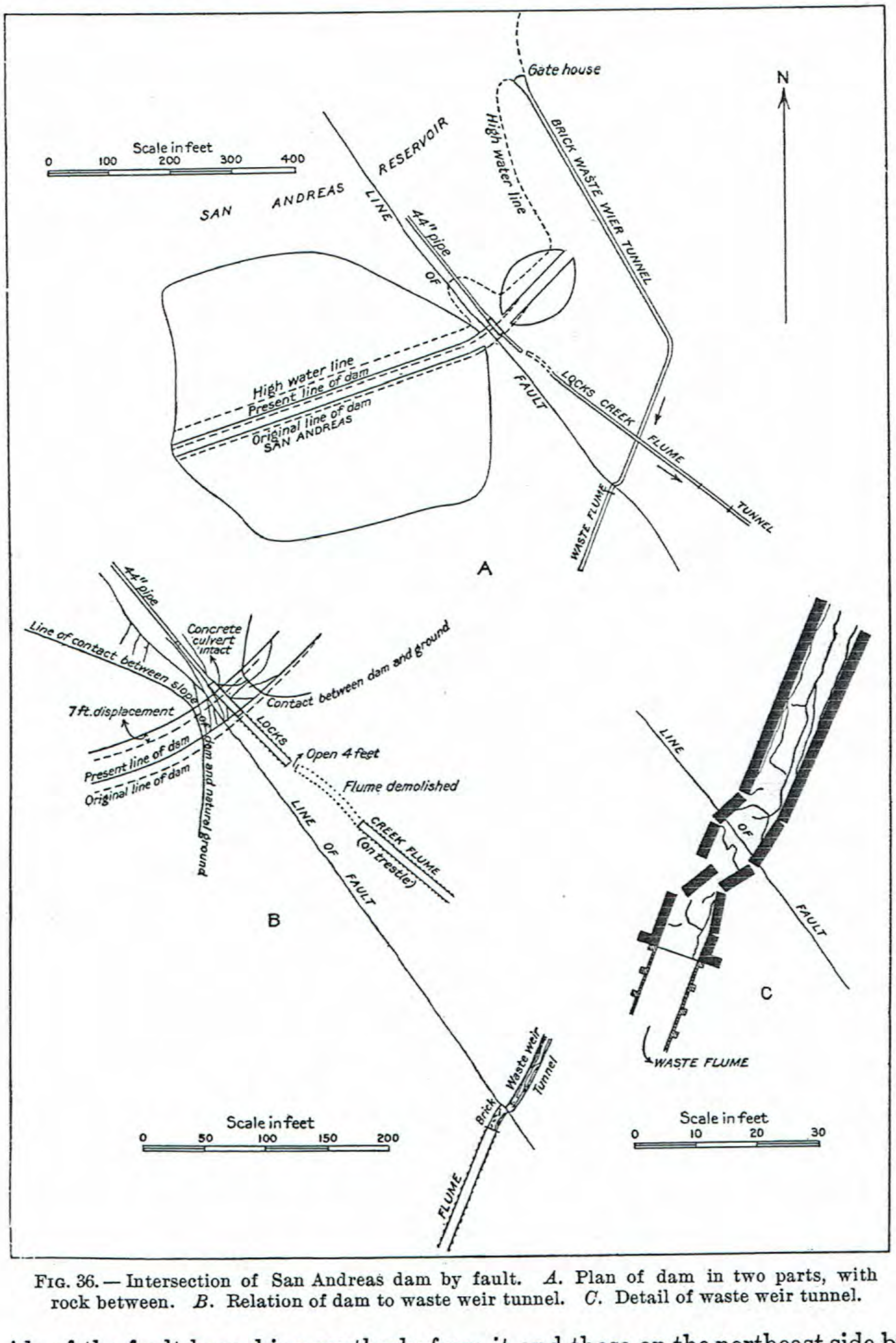


FIG. 36. — Intersection of San Andreas dam by fault. *A*. Plan of dam in two parts, with rock between. *B*. Relation of dam to waste weir tunnel. *C*. Detail of waste weir tunnel.

west side of the fault branching southerly from it and those on the northeast side branching northerly. Below the dam a heavy wooden flume on a trestle within 50 feet of the fault-trace was demolished for about 60 feet of its length.

About 125 yards below the dam the fault past thru the lower end of a massively built brick and cement waste weir tunnel. The inside diameter of the tunnel was about 7 or 8 feet and the walls were 17 inches thick. At the intersection of the fault within this structure, the latter was stove in and smashed in pieces for a distance of about 28 feet. The tunnel was offset about 5 feet. In the shattering of the brick work, the cracks and ruptures in no case followed the cement between the bricks, but broke across the latter; the cement and its adhesion to the bricks being stronger than the bricks themselves, altho the bricks were evidently carefully selected and of good quality. Several cracks traversed the tunnel longitudinally and obliquely to the northeast of the part that was demolished. (See fig. 36.)

About 550 yards below the San Andreas dam, the fault-trace crost a boundary fence between the estate of D. O. Mills and the property of the Spring Valley Water Company, causing an offset of about 10 feet. Here the deformation of the fence was distributed over a zone 300 feet wide in the direction of the fence, or about 250 feet in a direction normal to the trace of the fault. A survey of the dislocated fence made by R. B. Symington, C.E., is shown in fig. 37. Half a mile below the dam, the fault again crost the Pilarcitos pipe. A note by Mr. Anderson as to the conditions at this intersection is as follows:

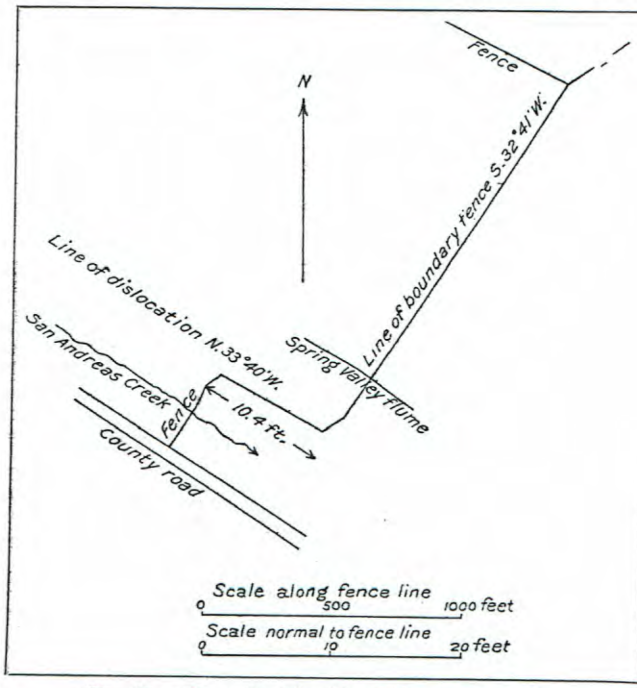


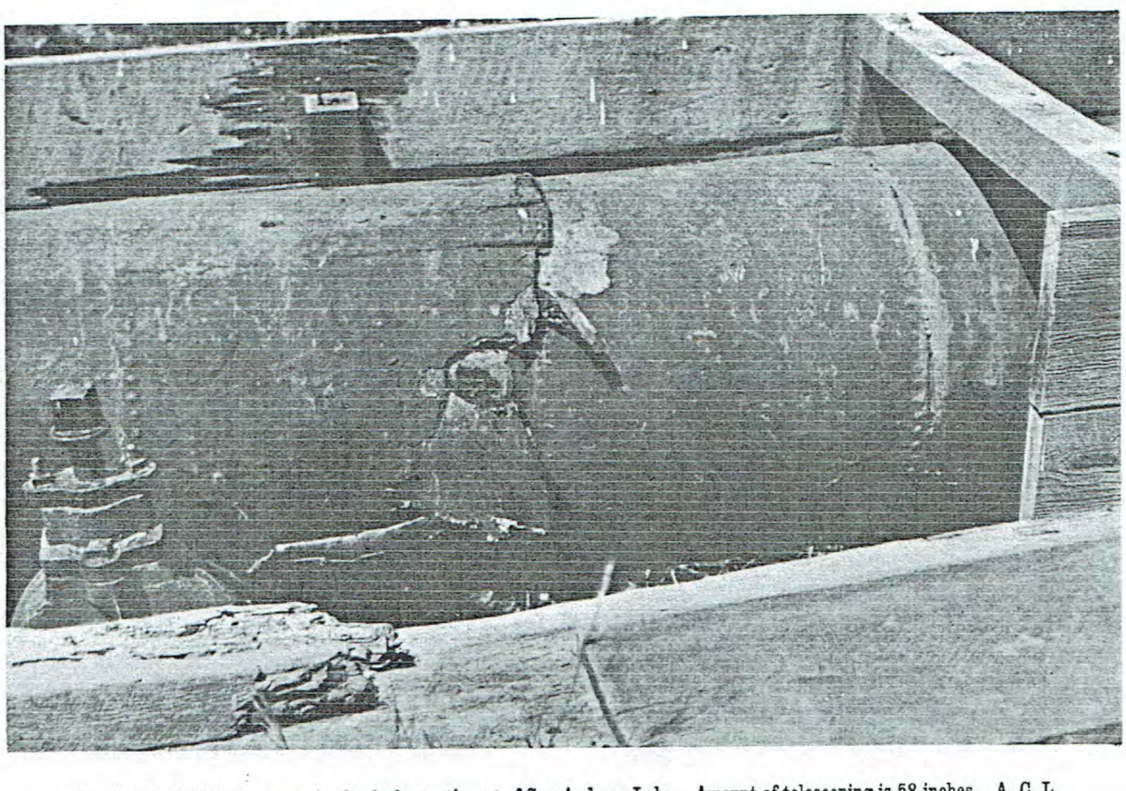
FIG. 37.—Fence *B* of fig. 30. Dislocated by fault.

It is a 2-foot pipe made of iron 1 inch thick. The fault broke it at an upward bend. An elbow at the bend was crusht by the compression and thrown down, while the two remaining ends were brought about 22 inches nearer together. At the same time they were faulted past each other a distance of 20 inches. The pipe runs N. 25° E., making an angle of 65° with the fracture, which here runs N. 40° W. The telescoping at this angle, being 22 inches, represents 52 inches of faulting.

In this neighborhood the fault crost a wire fence nearly normally, the line of which had been carefully established by a series of stone monuments. The fence marks the boundary between the estates of D. O. Mills and A. M. Easton. The deformation of the fence as shown in the accompanying diagram, fig. 38, from a survey by R. B. Symington, C.E., extended over a zone at least 2,200 feet wide. On the southwest side of the fault-trace, the fence was displaced to the northwest a distance of 9.3 feet, and on the northeast side it was displaced to the southeast 3.4 feet, making a total displacement of 12.7 feet and showing a slight drag close to the line of the fault. There were two parallel cracks representing the fault about 90 feet apart, and the chief displacement took place on the west crack.

About 0.625 mile farther southeast, near the upper end of Crystal Springs Lake, the fault crost another fence showing a displacement of 9 feet. About 0.25 mile southeast of this place, the fault crost the Locks Creek 44-inch pipe line. Regarding this intersection Mr. Anderson writes:

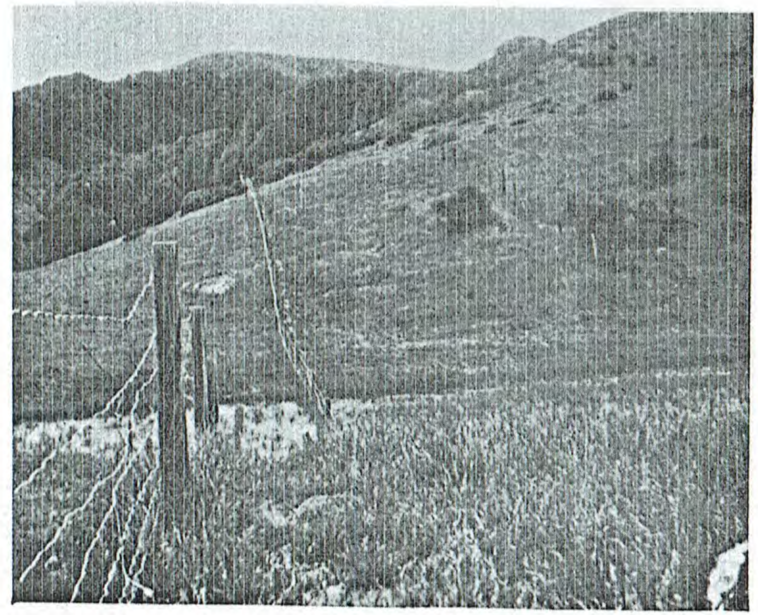
A. Rupture of 30-inch water-pipe by fault. Northwest of San Andreas Lake. A. C. L.

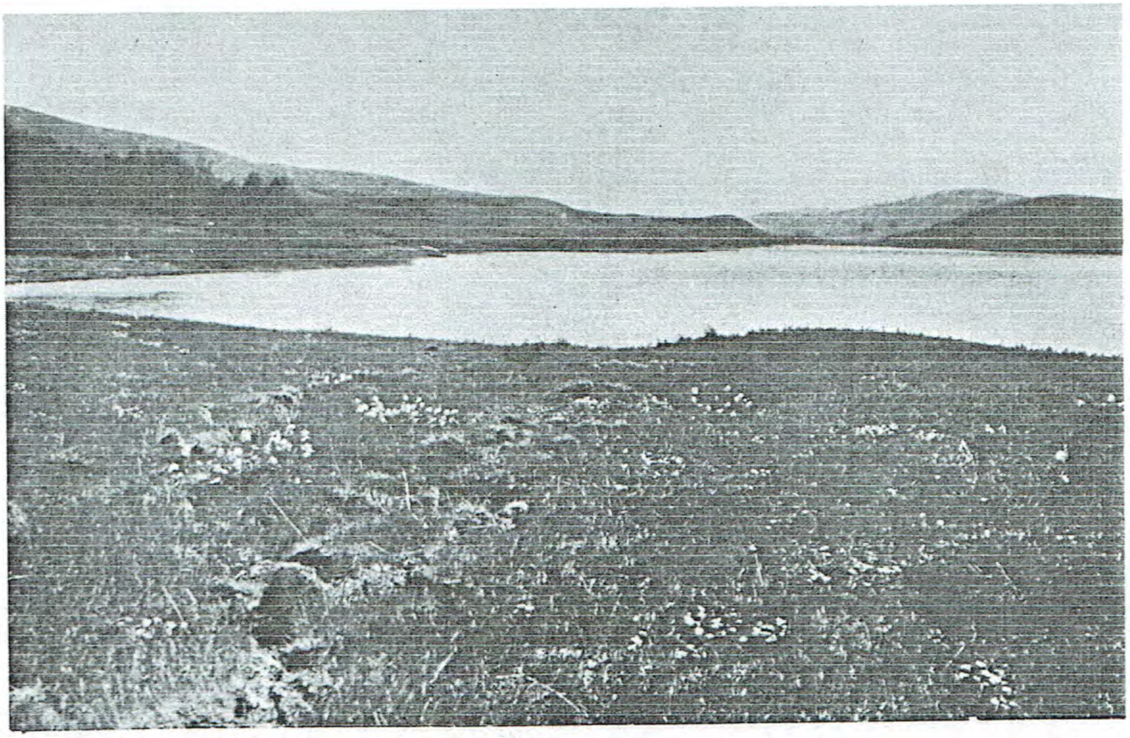


B. Thrust of 30-inch water-pipe by fault, northwest of San Andreas Lake. Amount of telescoping is 58 inches. A. C. L.

A. Offset in 30-inch water-pipe by fault. Northwest of San Andreas Lake. A. C. L.

B. Collapse of 30-inch water-pipe northwest of San Andreas Lake. R. L. H.





A. Fault-trace where it passes into San Andreas Lake. D.

B. Offset fence near Crystal Springs Lake. R. A.

A. Exposure of slicken-sided fault plane near north end of Crystal Springs Lake. R. A.

B. Offset of road by main fault near Searsville reservoir. Per J. C. B.

Just above the northern end of Crystal Springs Lake, a 44-inch water main made of iron 0.125 inch thick runs up the hill from the lake valley in a direction about N. 28° E. This line is buried all the way under several feet of soil. The fault crosses it at the base of the hill, in its N. 37° W. course, thus making an acute angle of 65° with the pipe line. At the intersection of the fault and the pipe line, the heavy rivets of the pipe were torn out all the way around at a section joint and the two sections were jammed into one another a distance of 4 feet 4 inches. In addition to the telescoping of this pipe, a slight change in course was induced, so that the northeast end trended one or two degrees more toward the east than the other end. This was shown by the fact that the broken ends did not fit into each other squarely. There was no lateral displacement, the whole movement having been taken up by the telescoping, but there was a bending of the pipes at the point of the break, as mentioned. The main part of the pipe, at a distance from the fault, must have moved with the land. At the fault-trace there was a bend amounting to one or two degrees. Supposing the bowing to be simple, this amount indicated that the land must have carried the pipe the distance represented by the telescoping, or about 10 feet, within 300 to 500 feet

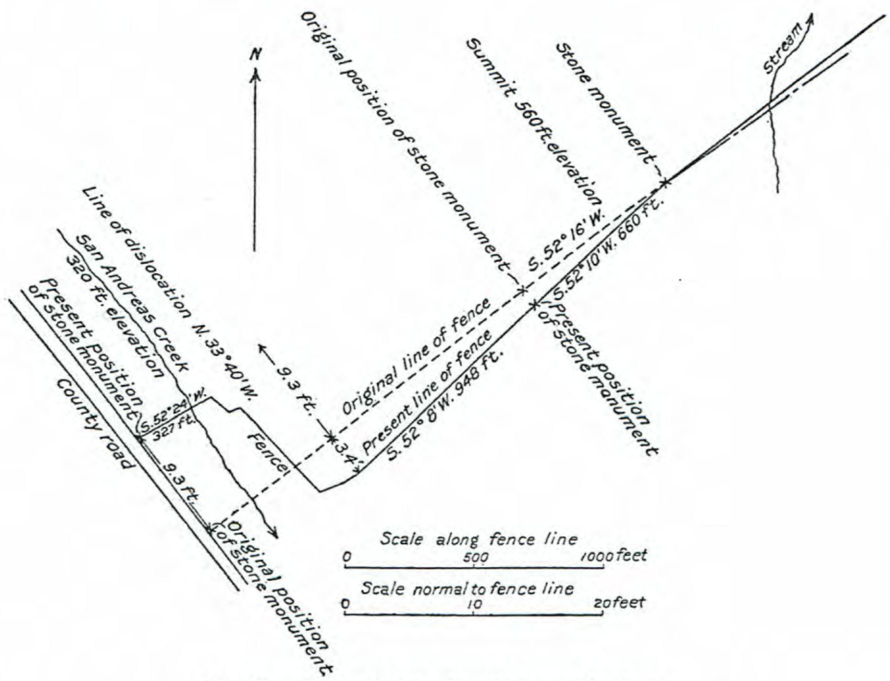


FIG. 38.—Fence *A* of fig. 30. Dislocated by fault.

of the fault on one side, and that beyond such a point the pipe must have preserved its normal course. As a matter of fact, this same pipe was broken on the northeast side of the fault about 400 feet further up the hill. The break occurred at the junction of 2 sections, the rivets having been sheared off and part of the rim torn away at the rivet holes. The ends were pulled apart 3.375 inches. Here the pipe resumed its former course, but owing to the slight amount of the pipe displayed by the excavation, it was impossible to see whether a return bend occurred or not. Beyond the break the direction was as before measured, approximately N. 28° E. No such break occurred on the southwest side of the fault. A crack was formed in the earth at right angles to the pipe for several yards on either side of the break.

The measurements of the engineers of the Spring Valley Water Company on the break and displacement of this pipe at the intersection above described by Mr. Anderson are given in the accompanying diagram, fig. 39.

About a mile southeast of the Locks Creek pipe line, the trace of the fault entered Crystal Springs Lake for the stage of water of April, 1906. At 2.5 miles farther due to settling of loosely accumulated or unsupported earth. For this reason no credence is given to the idea that an uplift or downthrow occurred along this part of the fault. This statement is based entirely on the evidence collected on the ground shortly after the earthquake and has nothing to do with the direction or amount of earlier displacement along this same fault-line. In some places an upward thrust seems to have taken place, as in the case of raising 7 pipes. This may, however, have been caused by wave-like movement in the ground near the surface or simply by the local heaving up of the ground as the result of compression.

APPENDIX D

This appendix contains an excerpt from the California Geological Survey 1941 Earthquake Fault Zone map (1974) for reference to locations oftwo attached case history studies in the Woodside, California reach of the San Francisco Segment of the San Andreas Fault (SFPS) pertaining to current Standard of Care geologic assessments of active faulting, and lead agency administration of habitable structure setback criteria in the spirit of guidelines presented in California Geological Survey Special Report 42 (SP-42, 2018).

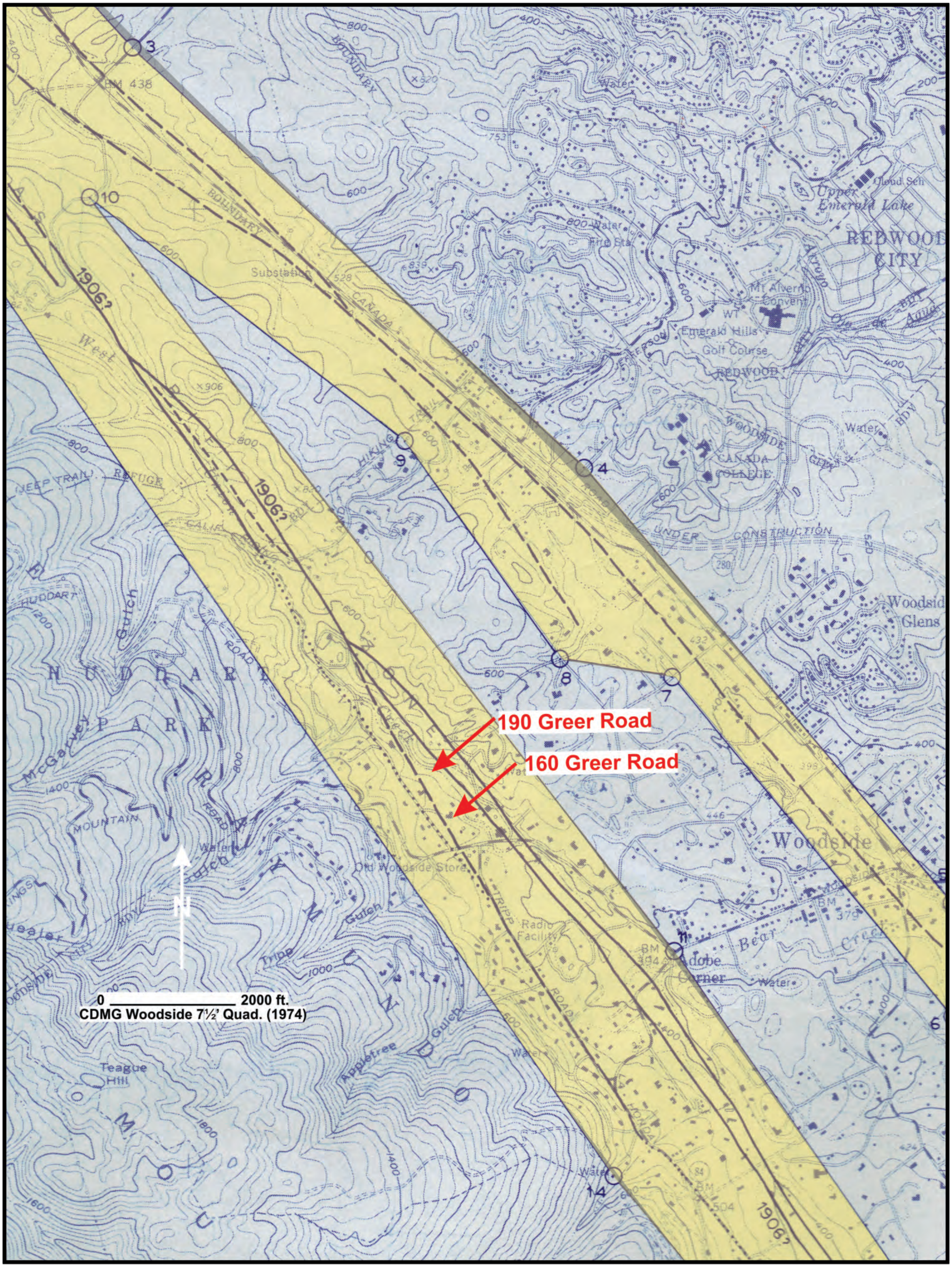


Plate D1. Woodside Quadrangle Earthquake Fault Zone Map (1974)

The Town of
Woodside

TOWN OF WOODSIDE

TOWN GEOLOGIST COMMENTS

Date of Review: November 21, 2000

Review by Town Geologist: RHW

Owner/Applicant: Burgess

Project Address: 190 Greer Road

Permit Application No.: NA

Deposit Account No.: **925**-UA5

Reference:

Earth Investigations Consultants (EIC), November 9, 2000, Engineering Geologic Evaluation, Active Faulting Across Residential Property, 190 Greer Road, Woodside, California, 6 p.

P.O. Box 620005
2955 Woodside Road
Woodside, CA 94062

Comments:

We understand that the conceptual plan is to demolish the existing residential development located in the northwest-central portion of the property, and construct a new residential development in the same area. Two traces of the San Andreas fault zone are mapped on the Town Geologic map traversing northwest-southeast, approximately parallel to Greer Road, through the northeast and southwest portions of the property. The active (1906) trace is mapped to the northeast near Josselyn Lane. With the current 110 foot building setbacks from these mapped traces, only a narrow, approximately 100-foot-wide zone through the center of the property is available for new residential construction. The existing residential development is partially located within the southwestern setback zone, and new residential construction, including additions and some remodeling, would generally not be allowed within this zone.

As discussed in the referenced report, two fault investigations have been performed on the adjacent property to the northwest (220 Greer Road) to evaluate the southwestern mapped trace. No evidence of faulting was encountered in these investigations. We concur with EIC that these trenches can be used to project an approximately 210 foot wide zone with no evidence of faulting southeast through the subject property as shown on Plate 2 of the referenced report. We also concur with the referenced report that although the evidence for the northeast trace is

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townhall@woodsidetown

questionable, this trace has not been sufficiently evaluated to conclude that it is not present.

Based on the data available at this time, we conclude that the southwestern portion of the subject property is suitable for residential development with respect to *potential fault ground rupture* from the southwest boundary of the "proximal trace setback (110')" to the "westerly boundary of area 'shadowed' by previous trenching" as shown on the Plate 2 of the reference report. Fault investigation(s) may be required for residential development in the northeast portion of the property, northeast of the southwest boundary of the "proximal trace setback (110') shown on Plate 2 of the reference report. Specific geologic and geotechnical requirements for future site development will be determined at the time a project specific application is submitted to the Town.

Comments forwarded by:

Dan Coughlin
Permits Technician

c: Richard Hyde, Trustee
Earth Investigations Consultants

GEOLOGICAL SERVICES
Active Fault Evaluation
160 Greer Road
Woodside, California

Prepared for:
Ms. Lina F. Crane
580 Whiskey Hill Road
Woodside, California 94062

Dated: April 18, 2012
Job 2447.01.00

Earth Investigations Consultants
P.O. Box 795
Pacifica, California 94044
Phone 650-557-0262
Fax 650-557-0264
earthinvestigations@comcast.net

EXECUTIVE SUMMARY

The attached report contains details of the site geological study pertaining to future habitable structure development constraints imposed by active earthquake faults that appear on the current version of the Town Geologic Map (Plate 1). The faults depicted were mapped in the early 1970's on the basis of geologic interpretation of surface features visible on stereo aerial photographs and from limited field reconnaissance. There have been some revisions based upon findings from geologic studies since the original mapping, but none sufficient to justify modification or deletion of the fault traces across the site. Hence, in accordance with State law, a jurisdiction is required to establish future habitable structure development setbacks on either side of the fault. A habitable building is one that is occupied by 2000 or more person-hours per year. In Woodside, the required minimum setback for an inferred fault trace is 125 feet. For a fault trace that has been established by exploration, the setback is 50 feet (note the respective setbacks are in addition to the width of the zone of active faulting measured in the exploratory trench exposure).

As illustrated on Plate 1, available space on the site for future habitable structure development is highly constrained by existing fault setbacks because at the time the map was prepared the faults were inferred (exact location unknown). Over the years, there have been numerous geologic investigations in an effort of limiting setback constraints to development. The purpose of this study was to review the existing geologic fault location reports in an effort to confirm the location, presence or absence of the faults mapped across the site.

In the following report, we present geologic findings from investigations on neighboring properties that indicate the western fault does not exist on the subject property. This finding has been accepted by the Town Geologist, Ted Sayre, of Cotton, Shires Associates, Inc. Deletion of the fault will be depicted on the revised geologic map. Plate 2 illustrates the conceptual revised map pertaining to deletion of the fault and related increase in available habitable building development area.

The exact position of the other fault trace on the site, near West Union Creek in the central part of the site, is unknown, hence 125-foot habitable building setbacks are required (Plate 2). It has been established as an active fault from exploratory trenching southeast of the site at 3301 Tripp Road, however there is a gap in previous trenching at 215 Josselyn Lane on the northwest side of the site (Plate 2). There was no report of faulting in that trenching, however, to assess the location of the trace on the site, it would be necessary to perform exploratory trenching that overlaps that neighboring trenching. If the fault is not discovered, then it, and the setbacks, would be removed. If the trenching encounters the fault, then 50-foot setbacks would be required, as depicted on Plate 3.

Earth Investigations Consultants

April 18, 2012
Job 2447.01.00

Lina F. Crane
c/o Mr. Ed Kahl
580 Whiskey Hill Road
Woodside, California
ed@edkahl.com

RE: GEOLOGICAL SERVICES
Active Fault Evaluation
160 Greer Road
Woodside, California

Dear Ms. Crane:

INTRODUCTION

Pursuant to your authorization, we have completed the referenced project located in the northwestern part of Woodside, California (Plate 1, Strip Map – San Andreas Fault Zone). The purpose of this study was to use available, existing geologic fault investigations to evaluate/confirm active fault setback constraints to future construction of habitable structures at the referenced property. According to the Alquist-Priolo Act 1973, a habitable structure is one occupied for 2000 or more person-hours per year (Hart, 1994; Bryant and Hart, 2007).

The scope of services undertaken to arrive at the findings and conclusions in this report was limited to field observations of the site area; photogeologic interpretation of the local reach of the right-lateral, strike slip San Andreas fault zone; and review of pertinent fault investigation reports obtained at the Town Hall and our office files.

Plate 1 represents the compilation of our work. It illustrates sites associated with useful fault investigation data. The Strip Map also depicts locations for each of the fault investigation sites applied to our analysis. To be useful in our analysis the trench locations should span the active fault trace, and the subsurface conditions illustrated on detailed logs prepared by a licensed geologist.

We submitted a draft of the map to the Town Geologist, Ted Sayre, who verbally expressed general agreement with the findings and conclusions. We understand

Geologists & Engineers
P.O. Box 795 ● Pacifica, CA 94044 ● (650) 557-0262 ● Fax (650) 557-0264

that you received an email confirmation of the their intention to delete from the revised Town Geologic Map the Distal Trace of the San Andreas fault, which extends across the westerly side of the site.

FINDINGS

San Andreas Fault Zone in Northwestern Woodside

The site lies on the westerly margin of the nearly 1000-foot wide right-lateral, strike slip San Andreas fault zone. The San Andreas fault zone is considered the tectonic boundary between the North American and Pacific Plates. Future major earthquake activity from movement along this fault zone is expected to occur (Working Group, 2008). Very strong ground shaking and fault ground rupture along the fault trace over a distance of 290 miles, from Cape Mendocino to San Juan Bautista, were associated with the 1906 earthquake. Reported ground rupture from the earthquake (Lawson, 1908) in parts of Woodside may have been similar to strike slip movement in the 1992 Mohave Desert Landers earthquake pictured below (from California Geological Survey, 2007).

The current version of the Town Geologic Map suggests that active fault movement during major earthquakes is distributed over a series of subparallel fault strands mapped by Dickenson (1973) on the basis or photogeologic interpretation and limited field reconnaissance of rift features marking the local landscape (Plate 1). The fault strands include the Master, Medial, and Distal traces. The Master trace on the east side of the zone is defined by two, subparallel traces that trend along West Union Creek between Woodside Road and Kings Mountain Road. The faults eventually converge as a single trace northwest of the intersection of Kings Mountain Road and Josselyn Lane; the Medial Trace, which is a single trace that projects across the upper middle part of the site near West Union Creek and eventually ends near the northern boundary of 179 Kings Mountain Road near the intersection of Tripp Road and Tripp Court; and the Distal Trace of two faults which converge as a singe trace at 220 Greer Road and extends across the western side of the site near Greer Road to also terminate at the northern boundary of 179 Kings Mountain Road. Note that symbolism in the Explanation on Plate 1 defines the relative certainty of fault location and rift features that marked the trace.

Geologic Investigations of the Master Trace

In 1992 fault exploratory trenching (Applied Earth Science Consultants, 1992) to locate the Master Trace instead encountered an unmapped active fault trace at the east side of 179 Kings Mountain Road without geologic evidence of active faulting at the mapped location of the Master Trace. Similarly, absence of evidence in trenching was reported across the mapped Master Trace at 335 King Mountain Road (Earth Investigations Consultants, 2010) and 3480 Woodside Road (Earth Investigations Consultants, 1997). However, at the northeast corner of 3480 Woodside Road there is a northwest trending pressure ridge tending to merge with the Master Trace where it intersects Woodside Road. The general trend of the unmapped active fault trace was further developed from trenching at 204 Josselyn Lane (Earth Investigations Consultants, 1994), 289 Kings Mountain Road (Upp Geotechnology, Inc. 1994), 360 Kings Mountain Road (Earth Investigations Consultants, 2010), and 340 Kings Mountain Road (Murray Engineers, Inc., 2010). The earliest fault investigation where the unmapped fault was exposed, in the northwest end of the area mapped on Plate 1, was where it merges with the Master Trace in the central part of what is locally known as the Greer Highlands approximately 1500 feet north of the site. There, Jo Crosby and Associates (1973) encountered in an exploratory fault trench a zone of faulting to the ground surface, with the fault zone containing steep, westerly dipping sheared Santa Clara mudstone juxtaposed with white ash (400,000 yr. old Rocklin Ash of Sarna-Wojcicki and others, 1985). Fault structures extending to the surface along all of the exposures where the unmapped fault was encountered is strong evidence that it represents the line of surface rupture in the 1906 earthquake (salmon-colored trace on Plate 1).

Geologic Investigations of the Medial Trace

The Medial Trace of the San Andreas fault was encountered in a trench excavated at 3301 Tripp Road southeast of the site in a closed depression with elevated ground water. The trench exposure was of terrace deposits offset by a foot or two in a relatively narrow zone. Trenching by JCP Engineers and Geologists (1990) at 215 Josselyn Lane, approximately 450 feet north of the site did not encounter evidence of faulting, however their trench distribution left a nearly 100-foot wide gap, which could conceivably contain an active fault. Unfaulted geologic exposures mapped by Upp Geotechnology, Inc. (2000) in the western bank of West Union Creek on 220 Greer Road, between the Medial and Distal traces. However this location did not span the gap in JCP's trenching.

Geologic Investigations of the Distal Trace

Trenching at 3573 Tripp Road (Purcell, Rhoades and Associates, 1985) and 179 King Mountain Road (Applied Earth Science Associates, 1990) across the projected Medial Trace southeast of the site found no evidence of faulting. Overlapping exploratory trenches by Jo Crosby and Associates (1973) and Upp Geotechnology, Inc. (2000) encountered unfaulted terrace deposits across the projected Distal fault trace. We, in 2000, prepared a report pertaining to the Distal trace citing absence of faulting in the aforementioned 1973 Jo Crosby and similar findings for an investigation at 460 Raymundo Drive across the Distal Trace by Phillip Frame (2000) as justification for the absence of the Distal Trace across and northwest of the western side of 190 Greer Road, to which the Town Geologist agreed (see attached memo of Town Geologist's comments).

CONCLUSIONS AND RECOMMENDED ACTION

The results of this investigation indicate the following:

Master Trace

Geologic investigations of the Master Trace have encountered no support of its existence. We recommend that the two traces between Woodside Road and at least Greer Road be excluded from the to-be-revised Town Geologic Map.

Medial Trace

The only geologic investigation found in our research for this trace, which projects across the east-central part of the site, was at 3301 Tripp Road, south of the site, where recent soil deformation with suspected, near-vertical shears in the soil profile extending to or near the ground surface to the east end of the exploratory trench, excavated in a closed depression. This data is indicative of an active fault.

Exploratory trenching at 215 Josselyn Lane on the north side of the site provided inadequate coverage of the fault trace. Setback requirements remain valid until additional exploration(s) across the projected trace are achieved.

Distal Trace

There is adequate geologic evidence to support the conclusion that this fault strand, extending across the western part of the site from the projected trace between the northern part of 179 Kings Mountain Road to Raymundo Drive, does not exist. It should be excluded from the to-be-revised Town Geologic Map. Attached is a previous determination of this fault trace by the Town Geologist in 2000.

Given the above conclusions, it is my opinion that the only fault setback habitable development on the site is associated with the Medial Trace mapped across the east-central part of the site (Plate 1).

Unmapped Trace

There is excellent geologic confirmation that fault rupture from the 1906 earthquake occurred along the unmapped fault trace (salmon-colored trace on

Plate 1) located uphill and from 200 to 400 feet east of the Master Trace. This fault should be mapped as active on the to-be-revised Town Geologic Map.

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AERIAL PHOTOGRAPHS

Source	Date	Job No.	Flight Line	Frames	Scale
USAF	4/19/70	GS-VCMI	1 & 2	31 &177,	1:80,700
U.S. Geological Survey, Menlo Park, California					
USDA	6/9/56	DDB	3R	48 &49	1:20,000
Soil Conservation. Service, Salt Lake City, Utah					
Pacific	6/3/55	AV 170	14	18 & 19	1:12,000
Aerial Survey, Oakland, California					

The following illustration and document are attached:

- Plate 1 – Strip-Map – San Andreas Fault
- Plate 2 – Recommended Town Geologic Map Revision
- Plate 3 – Fault Setback
- Town Geologist Comments, 190 Greer Road (2 pgs. dated November 21, 2000)

We trust that this report provides you with the information you require at this time. Please call if you have any questions.

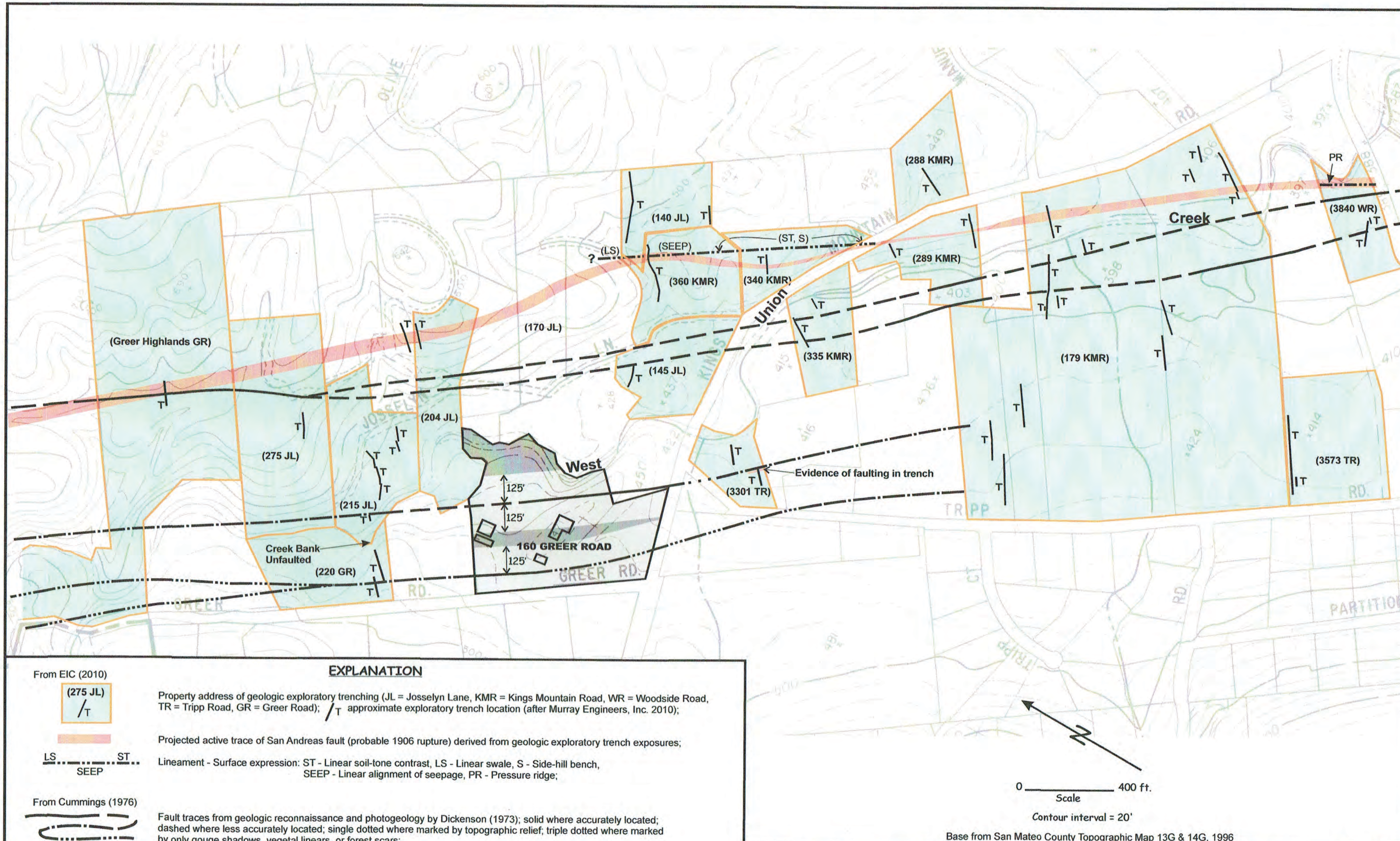
Very truly yours,

Earth Investigations Consultants

REGISTERED GEOLOGIST
JOEL E. BALDWIN, II
No. EG 1132
CERTIFIED ENGINEERING GEOLOGIST
STATE OF CALIFORNIA

Joel E. Baldwin. II
Principal Engineering Geologist, CEG 1132 (renewal date 2/28/13)

JEB:jb:gi
Distribution: 3 bound copied mailed and electronic file to addressee



Base from San Mateo County Topographic Map 13G & 14G, 1996

EXPLANATION

From EIC (2010)

(275 JL) /T — Property address of geologic exploratory trenching (JL = Josselyn Lane, KMR = Kings Mountain Road, WR = Woodside Road, TR = Tripp Road, GR = Greer Road); /T approximate exploratory trench location (after Murray Engineers, Inc. 2010);

Projected active trace of San Andreas fault (probable 1906 rupture) derived from geologic exploratory trench exposures;

LS ST SEEP — Lineament - Surface expression: ST - Linear soil-tone contrast, LS - Linear swale, S - Side-hill bench, SEEP - Linear alignment of seepage, PR - Pressure ridge;

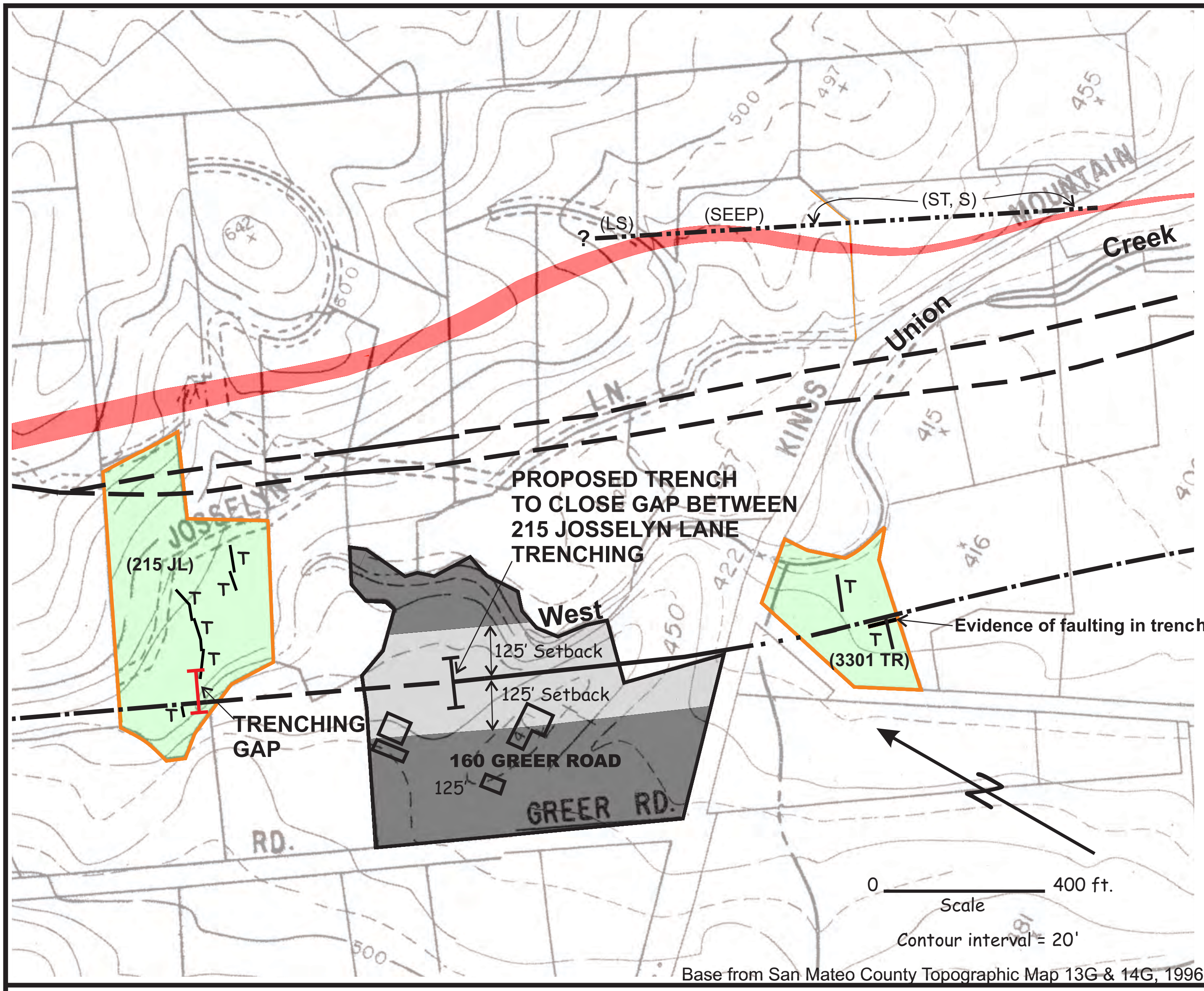
From Cummings (1976)

Fault traces from geologic reconnaissance and photogeology by Dickenson (1973); solid where accurately located; dashed where less accurately located; single dotted where marked by topographic relief; triple dotted where marked by only gouge shadows, vegetal linears, or forest scars;

Building locations at 160 Greer Road (from Town of Woodside 1" = 600' photogrammetric cadastral map (1967)

Fault constrained buildable area.

Earth Investigations Consultants	Job No. 2447.01.00 Approved Date 4/12/12	STRIP MAP San Andreas Fault Zone 160 Greer Road Woodside, California	Plate 1



Base from San Mateo County Topographic Map 13G & 14G, 1996

EXPLANATION

From EIC (2010)

(275 JL) T — Property address of geologic exploratory trenching (JL = Josselyn Lane, KMR = Kings Mountain Road, WR = Woodside Road, TR = Tripp Road, GR = Greer Road); T approximate exploratory trench location (after Murray Engineers, Inc. 2010);

Projected active trace of San Andreas fault (probable 1906 rupture) derived from geologic exploratory trench exposures;

LS ST SEEP — Lineament - Surface expression: ST - Linear soil-tone contrast, LS - Linear swale, S - Side-hill bench, SEEP - Linear alignment of seepage, PR - Pressure ridge;

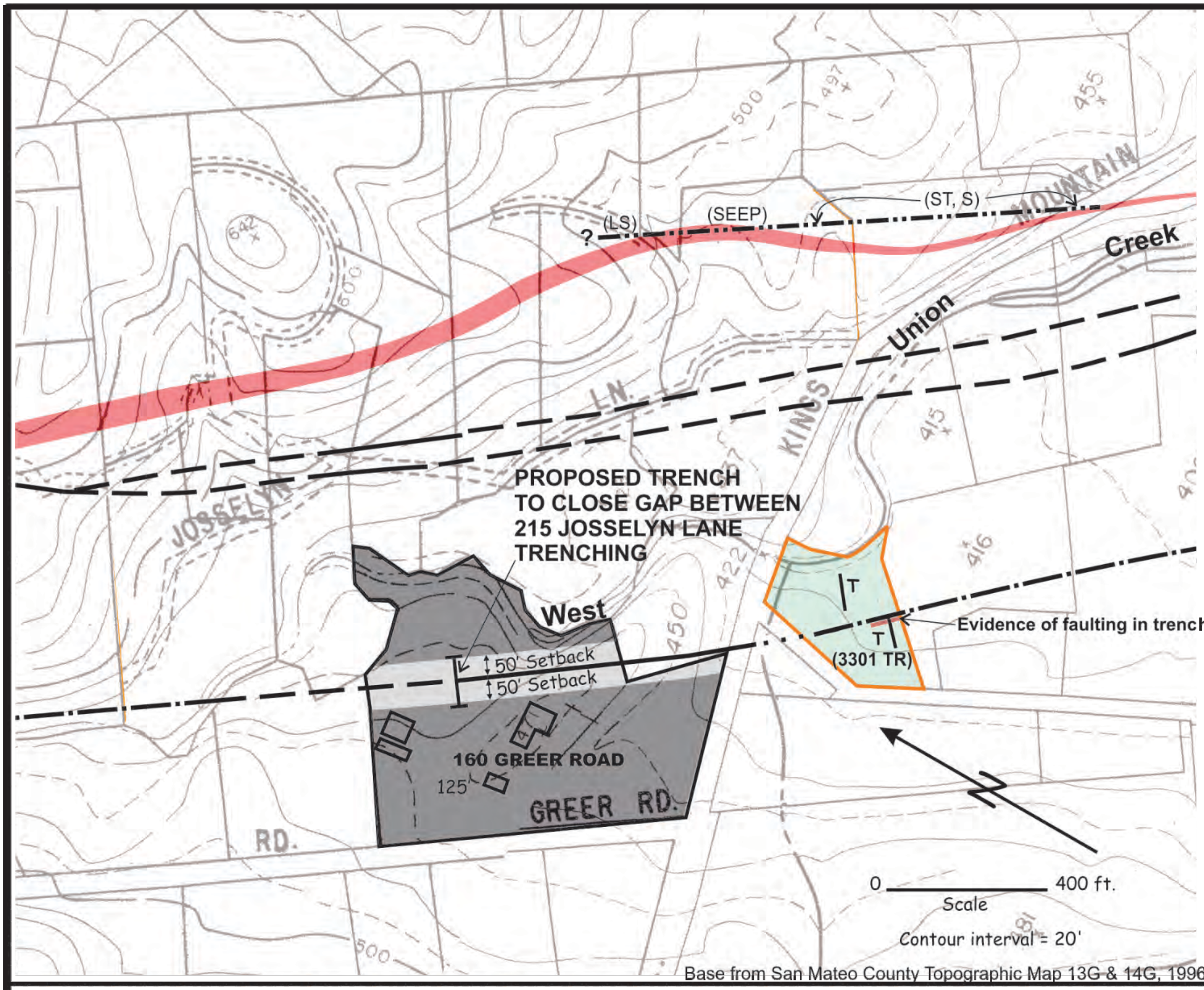
From Cummings (1976)

Fault traces from geologic reconnaissance and photogeology by Dickenson (1973; current Town of Woodside Geologic map); solid where accurately located; dashed where less accurately located; single dotted where marked by topographic relief; triple dotted where marked by only gouge shadows, vegetal linears, or forest scars;

Building locations at 160 Greer Road (from Town of Woodside 1" = 600' photogrammetric cadastral map (1967)

Fault constrained buildable area.

Earth Investigations Consultants	Job No. 2447.01.00 Date 4/12/12	RECOMMENDED TOWN GEOLOGIC MAP REVISION 160 Greer Road Woodside, California	Plate 2



EXPLANATION

From EIC (2010)

(275 JL) T — Property address of geologic exploratory trenching (JL = Josselyn Lane, KMR = Kings Mountain Road, WR = Woodside Road, TR = Tripp Road, GR = Greer Road); T approximate exploratory trench location (after Murray Engineers, Inc. 2010);

Projected active trace of San Andreas fault (probable 1906 rupture) derived from geologic exploratory trench exposures;

LS ST SEEP — Lineament - Surface expression: ST - Linear soil-tone contrast, LS - Linear swale, S - Side-hill bench, SEEP - Linear alignment of seepage, PR - Pressure ridge;

From Cummings (1976)

Fault traces from geologic reconnaissance and photogeology by Dickenson (1973); solid where accurately located; dashed where less accurately located; single dotted where marked by topographic relief; triple dotted where marked by only gouge shadows, vegetal linears, or forest scars;

Building locations at 160 Greer Road (from Town of Woodside 1" = 600' photogrammetric cadastral map (1967)

Fault constrained buildable area.

Earth Investigations Consultants	Job No. 2447.01.00 Date 4/12/12	FAULT SETBACK 160 Greer Road Woodside, California	Plate 3

APPENDIX E

This appendix contains the 29 logs of borings, including geologic soil probes, presented in EIC geologic and geotechnical reports (2006, 2008, 2013, 2016) and Geosphere Consultants, Inc. (2020) for various project layouts since 2006.

Plate 3 - Site Engineering Geologic Map
Plate E1 - Key to Borings
Plate E2 - Key to Soil Probes
Plate E3 - Rock Hardness Chart

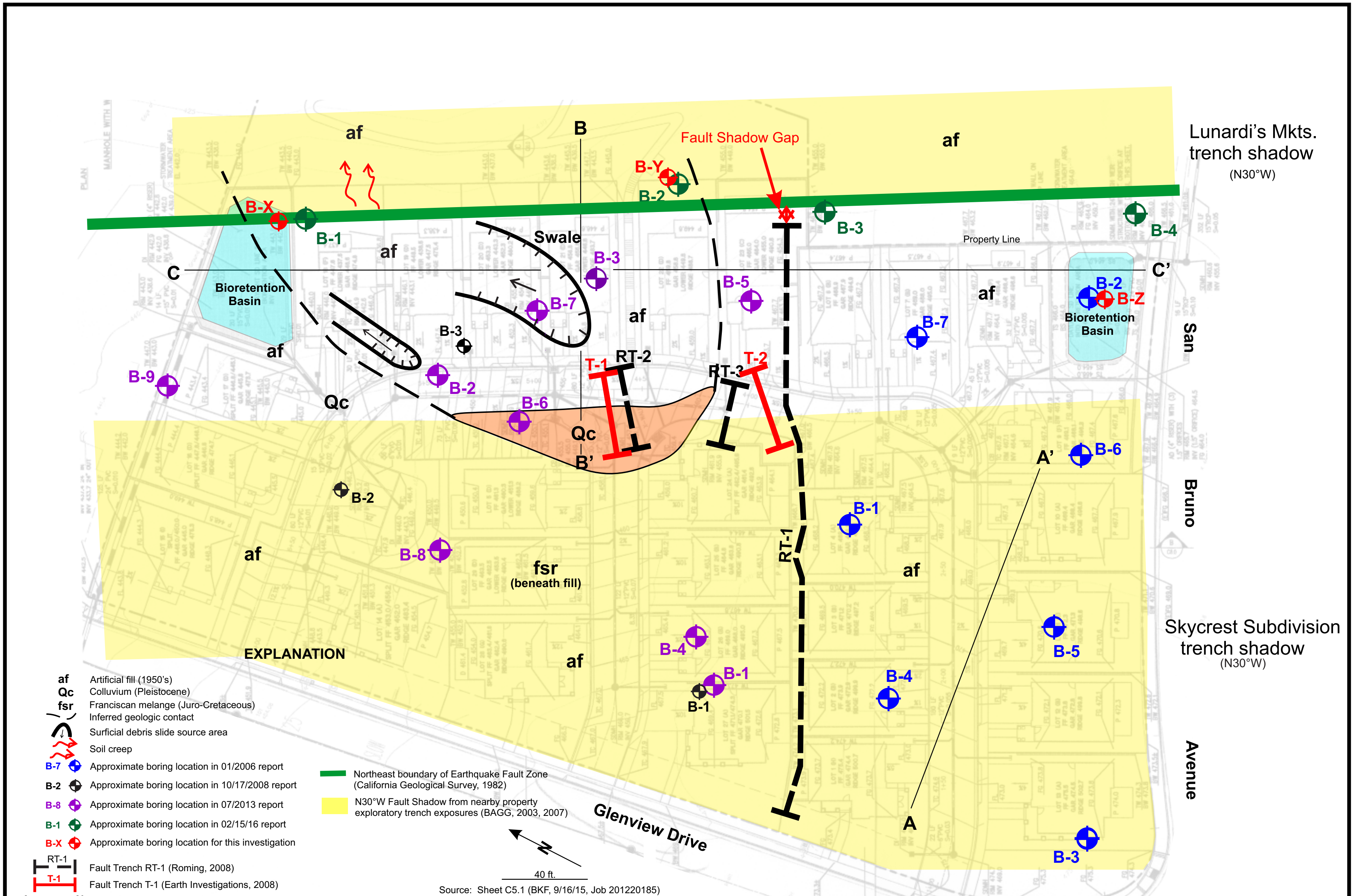
Earth Investigations Consultants, Job No. 2052.01.01, 10/2006
Plate E4 - Log of Boring 1
Plate E5 - Logs of Borings 2 & 3
Plate E6 - Logs of Borings 4 & 5
Plate E7 - Logs of Borings 6 & 7

Earth Investigations Consultants, Inc. Job No. 2271.01.00, 10/2008
Plate E8 - Logs of Borings 1 & 2 (Soil Probes)
Plate E9 - Log of Boring 3 (Soil Probe)

Earth Investigations Consultants, Inc. Job No. 2479.01.00, 08/2013
Plate E10 - Logs of Borings 1 & 2 (Soil Probes)
Plate E11 - Log of Boring 3 (Soil Probe)
Plate E12 - Logs of Borings 4 & 5
Plate E13 - Log of Boring 6
Plate E14 - Log of Boring 7
Plate E15 - Logs of Borings 8 & 9

Earth Investigations Consultants, Inc. Job No. 2479.02.00, 02/2016
Plate E16 - Logs of Borings 1 & 2 (Soil Probes)
Plate E17 - Logs of Borings 3 & 4 (Soil Probes)

Geosphere Consultants, Inc., 91-04747-A, 01/2020
Plate E18 - Log of Boring X
Plate E19 - Log of Boring Y
Plate E20 - Log of Boring Z



Geosphere Consultants, Inc.

Job No.: 91-04747-A
Approved: JEB
Date: 01.27.2020

SITE ENGINEERING GEOLOGIC MAP

Glenview Terrace Residential Development
2880 San Bruno Avenue
San Bruno, California

Plate 3

			GROUP SYMBOL	Secondary Divisions
COARSE GRAINED SOILS — MORE THAN HALF OF MATERIAL IS LARGER THAN NO. 200 SIEVE SIZE	GRAVELS — MORE THAN HALF OF COARSE FRACTION IS LARGER THAN NO. 4 SIEVE	CLEAN GRAVELS (LESS THAN 5% FINES)	GW	Well graded gravels, gravel-sand mixtures, little or no fines.
			GP	Poorly graded gravels or gravel-sand mixtures, little or no fines.
		GRAVEL WITH FINES	GM	Silty gravels, gravel-sand-silt mixtures, non-plastic fines.
			GC	Clayey gravels, gravel-sand-clay mixtures, plastic fines.
	SANDS — MORE THAN HALF OF COARSE FRACTION IS SMALLER THAN NO. 4 SIEVE	CLEAN SANDS (LESS THAN 5% FINES)	SW	Well graded sands, gravelly sands, little or no fines.
			SP	Poorly graded sands or gravelly sands, little or no fines.
		SANDS WITH FINES	SM	Silty sands, sand-silt mixtures, non-plastic fines.
			SC	Clayey sands, sand-clay mixtures, plastic fines.
FINE GRAINED SOILS — MORE THAN HALF OF MATERIAL IS SMALLER THAN NO. 200 SIEVE SIZE	SILTS AND CLAYS — LIQUID LIMIT IS LESS THAN 50%		ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity.
			CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays.
			OL	Orangic silts and organic silty clays of low plasticity.
	SILTS AND CLAYS — LIQUID LIMIT IS GREATER THAN 50%		MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic.
			CH	Inorganic clays of high plasticity, fat clays.
			OH	Organic clays of medium to high plasticity, organic silts.
HIGHLY ORGANIC SOILS			Pt	Peat and other highly organic soils.

U.S. Standard Series Sieve — Clear Square Sieve Openings

	200	40	10	4	3/4"	3"	12"
SILTS AND CLAY	SAND			GRAVEL		COBBLES	BOULDERS
	FINE	MEDIUM	COARSE	FINE	COARSE		

Grain Sizes

SAND AND GRAVELS	BLOWS/FOOT*
VERY LOOSE	0 -4
LOOSE	4 -10
MEDIUM DENSE	10 - 30
DENSE	30 - 50
VERY DENSE	OVER 50

Relative Density

SILTS AND CLAYS	STRENGTH **	BLOWS/FOOT*
VERY SOFT	0 - 1/4	0 - 2
SOFT	1/4 - 1/2	2 - 4
FIRM	1/2 - 1	4 - 8
STIFF	1 - 2	8 - 16
VERY STIFF	2 - 4	16 - 32
HARD	OVER 4	OVER 32

Consistency

* Number of blows of 140 pound hammer falling 30 inches to drive a split spoon, SPT sampler (ASTM D-1586)

** Unconfined compressive strength in tons/sq. ft. as determined by laboratory testing or approximated by the standard penetration test (ASTM D-1586), pocket penetrometer, torvane, or visual observation.

■ Sample location

☒ Grab sample

59 Total number of SPT blow counts for sampling interval. Bar graph represents individual 6-inch intervals for bottom 12 inches of 18-inch drive sample.

Unified Soil Classification System (ASTM D-2487)

Geosphere Consultants, Inc.

Job No.: 91-04747-A	**KEY TO BORINGS**	**Plate**
Approved: JEB	Glenview Terrace Residential Development 2880 San Bruno Avenue San Bruno, California	E1
Date: 01.21.2020		

Primary Divisions				GROUP SYMBOL	Secondary Divisions
COARSE GRAINED SOILS	MORE THAN HALF OF MATERIAL IS LARGER THAN NO. 200 SIEVE SIZE	GRAVELS MORE THAN HALF OF COARSE FRACTION IS LARGER THAN NO. 4 SIEVE	CLEAN GRAVELS (LESS THAN 5% FINES)	GW	Well graded gravels, gravel-sand mixtures, little or no fines.
				GP	Poorly graded gravels or gravel-sand mixtures, little or no fines.
			GRAVEL WITH FINES	GM	Silty gravels, gravel-sand-silt mixtures, non-plastic fines.
				GC	Clayey gravels, gravel-sand-clay mixtures, plastic fines.
		SANDS MORE THAN HALF OF COARSE FRACTION IS SMALLER THAN NO. 4 SIEVE	CLEAN SANDS (LESS THAN 5% FINES)	SW	Well graded sands, gravelly sands, little or no fines.
				SP	Poorly graded sands or gravelly sands, little or no fines.
			SANDS WITH FINES	SM	Silty sands, sand-silt mixtures, non-plastic fines.
				SC	Clayey sands, sand-clay mixtures, plastic fines.
FINE GRAINED SOILS	MORE THAN HALF OF MATERIAL IS SMALLER THAN NO. 200 SIEVE SIZE	SILTS AND CLAYS LIQUID LIMIT IS LESS THAN 50%		ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity.
				CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays.
				OL	Orangic silts and organic silty clays of low plasticity.
		SILTS AND CLAYS LIQUID LIMIT IS GREATER THAN 50%		MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic.
				CH	Inorganic clays of high plasticity, fat clays.
				OH	Organic clays of medium to high plasticity, organic silts.
HIGHLY ORGANIC SOILS				Pt	Peat and other highly organic soils.

Definition of Terms

U.S. Standard Series Sieve — **Clear Square Sieve Openings**

	200		40		10		4		3/4"		3"		12"	

SILTS AND CLAY	SAND			GRAVEL		COBBLES	BOULDERS
	FINE	MEDIUM	COARSE	FINE	COARSE		

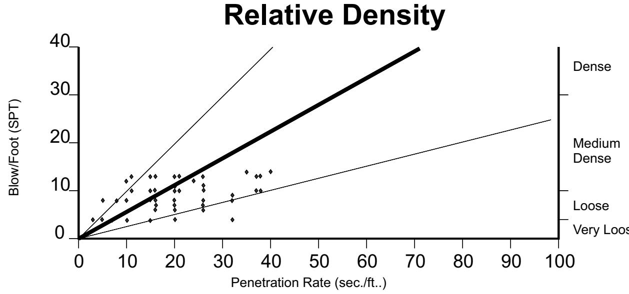
Grain Sizes

Unified Soil Classification System (ASTM D-2487)

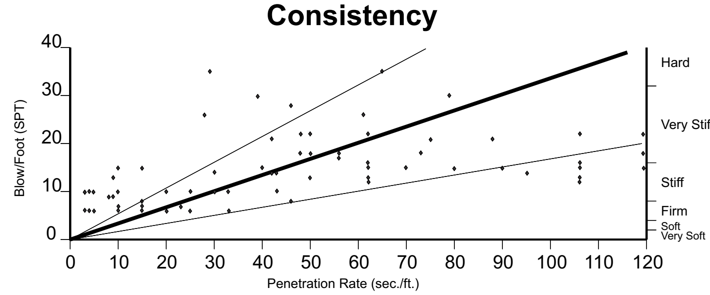
SAND AND GRAVELS	PENETRATION RATE*
VERY LOOSE	0 - 7
LOOSE	7 - 18
MEDIUM DENSE	18 - 53
DENSE	53 - 88
VERY DENSE	OVER 88

SILTS AND CLAYS	STRENGTH **	PENETRATION RATE*
VERY SOFT	0 - 1/4	0 - 6
SOFT	1/4 - 1/2	6 - 11
FIRM	1/2 - 1	11 - 23
STIFF	1 - 2	23 - 47
VERY STIFF	2 - 4	47 - 94
HARD	OVER 4	OVER 94

Relative Density



Consistency



* Seconds per foot, based on a portable percussion rig advancing a 1½-inch diameter split-spoon sampler with a force of 35 ft. lb. at a rate of 1270 blows per minute.

** Unconfined compressive strength in tons/sq. ft. as determined by laboratory testing or approximated by the standard penetration test (ASTM D-1586), pocket penetrometer, torvane, or visual observation.

Geosphere Consultants, Inc.	Job No.: 91-04747-A Approved: JEB Date: 01.21.2020	**KEY TO SOIL PROBES** Glenview Terrace Residential Development 2880 San Bruno Avenue San Bruno, California	**Plate** E2

ROCK HARDNESS CRITERIA

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

Subsurface Manual for Design and Construction of Foundations of Buildings, 1976
Published by American Society of Civil Engineers.

Geosphere Consultants, Inc.	Job No.: 91-04747-A Approved: JEB Date: 01.21.2020	**ROCK HARDNESS CHART** Glenview Terrace Residential Development 2880 San Bruno Avenue San Bruno, California	**Plate** E3

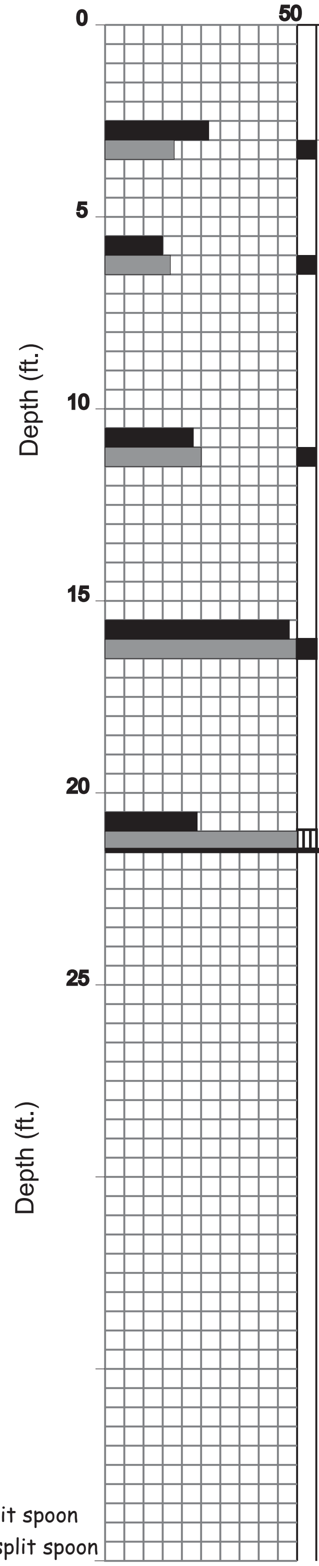
Earth Investigations Consultants
Job No. 2052.01.01
10/2006

BORING 1

Equipment: Truck Mounted CME-75 Auger

*Elevation: ~340'   Date: 2/2/05

Dry Density (pcf)	Moisture Content (%)	Blows/Foot	Depth (ft.)	USCS	Description
			0	SM	Dark yellowish brown Gravelly Silty SAND, damp, medium dense (FILL)
123.5	9.1	45		ROCK	Dark yellowish brown, dark brown and dark grey SHEARED ROCK, damp, very weathered, closely fractured, soft to moderately hard (BEDROCK)
123.2	11.5	32	5		
123.9	10.5	48	10		
137.2	5.9	98	15		
122.4	8.3	74/10"	20		
					Terminated at 21' 4"



EXPLANATION

* Arbitrary datum, see Plate 2

■ Modified California sampler, 3" O.D. split spoon

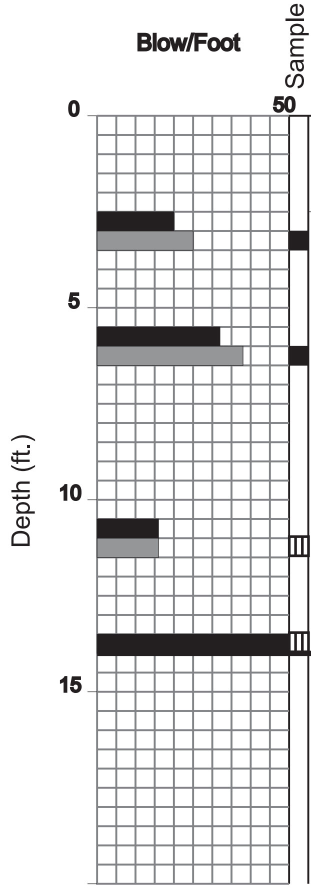
▥ Standard Penetration sampler, 2" O.D. split spoon

Earth Investigations Consultants	Job No. 2052.01.01 Date 10/16/06	LOG OF BORING 1 2880 San Bruno Avenue Burlingame, California	Plate E4

BORING 2

Equipment Truck Mounted CME-75 Auger

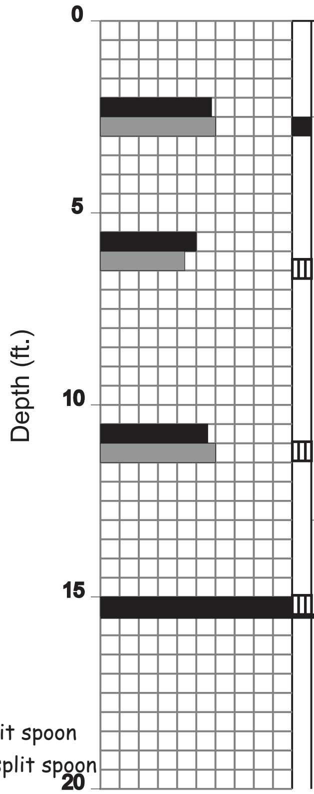
***Elevation** ~332′ **Date** 2/2/2005



			Depth	USCS	Description
			0	SM	Dark yellowish brown Silty SAND with Gravel, damp, medium dense (FILL)
118.6	10.1	45		ROCK	Dark yellowish brown, dark brown and dark grey SHEARED ROCK, damp, very weathered, closely fractured, soft to moderately hard (BEDROCK)
123.7	8.4	70	5		
114.3	7.5	32	10		
		50/2″			
			15		Terminated at 14′ 2″

BORING 3

***Elevation** ~343′ **Date** 2/2/2005



			Depth	USCS	Description
			0	SM	Dark yellowish brown Gravelly Silty SAND, damp, medium dense (FILL)
124.7	8.6	59		ROCK	Dark yellowish brown GREENSTONE, very weathered, closely fractured, soft to moderately hard (BEDROCK)
119.6	5.0	47	5		
114.5	8.6	58	10		
116.2	8.4	50/3″	15	ROCK	Dark grey SHEARED ROCK, veery weathered, closely fractured, soft to moderately hard (BEDROCK)
			20		Terminated at 15′ 9″

EXPLANATION

* Arbitrary datum, see Plate 2

Ground water elevation after drilling

Modified California sampler, 3″ O.D. split spoon

Standard Penetration sampler, 2″ O.D. split spoon

Earth Investigations Consultants	**Job No.** 2052.01.01 **Date** 10/16/06	**LOGS OF BORINGS 2 & 3** 2880 San Bruno Avenue Burlingame, California	**Plate** E5

BORING 4

Equipment Truck Mounted CME-75 Auger

***Elevation** ~342' **Date** 2/2/2005

Dry Density (pcf)	Moisture Content (%)	Blow/Foot
126.2	7.5	42
137.9	6.7	66
118.9	7.3	46

Blow/Foot 0–50; Depth (ft.) 0, 5, 10, 15

USCS	Description
SM	Dark yellowish brown Gravelly Silty SAND, damp, medium dense (FILL)
ROCK	Dark yellowish brown, GREENSTONE, very weathered, closely fractured, soft to moderately hard (BEDROCK)

Terminated at 11 ½ '

BORING 5

***Elevation** ~341' **Date** 2/2/2005

Dry Density (pcf)	Moisture Content (%)	Blow/Foot
125.4	7.3	74
		32
113.2	8.4	34
126.5	10.0	49

Depth (ft.) 0, 5, 10, 15, 20

USCS	Description
SM	Dark yellowish brown Gravelly Silty SAND, damp, medium dense (FILL)
ROCK	Dark yellowish brown GREENSTONE, very weathered, closely fractured, soft to moderately hard (BEDROCK) No recovery

Terminated at 16 ½ '

* Arbitrary datum, see Plate 2

Ground water elevation after drilling

Modified California sampler, 3" O.D. split spoon

Standard Penetration sampler, 2" O.D. split spoon

Earth Investigations Consultants

Job No. 2052.01.01

Date 10/16/06

LOGS OF BORINGS 4 & 5

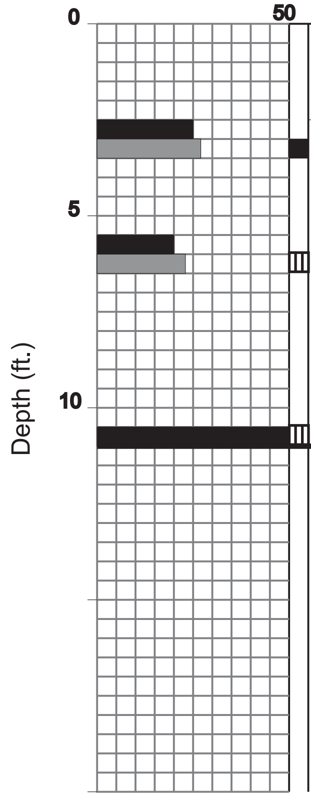
2880 San Bruno Avenue
Burlingame, California

Plate E6

BORING 6

Equipment: Truck Mounted CME-75 Auger
*Elevation: ~336' Date: 2/2/2005

Dry Density (pcf)	Moisture Content (%)	Blow/Foot	Depth (ft.)	USCS	Description
			0	SM	Dark yellowish brown Gravelly Silty SAND, damp, medium dense (FILL)
132.4	7.4	62		ROCK	Dark yellowish brown, GREENSTONE, very weathered, closely fractured, soft to moderately hard (BEDROCK)
118.7	5.4	43	5		
		50/5"	10		

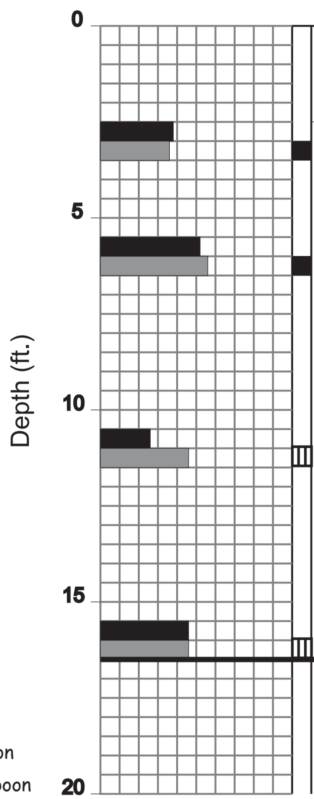


Terminated at 10' 11"

BORING 7

*Elevation: ~335' Date: 2/2/2005

Dry Density (pcf)	Moisture Content (%)	Blow/Foot	Depth (ft.)	USCS	Description
			0	SM	Dark yellowish brown Silty SAND with Gravel, damp, medium dense (FILL)
118.1	11.4	37		ROCK	Dark yellowish brown SHEARED ROCK, damp, very weathered, closely fractured, soft to moderately hard (BEDROCK)
123.0	9.5	54	5		
114.0	8.7	36	10		
117.7	6.8	46	15		



Terminated at 16 ½ '

EXPLANATION

* Arbitrary datum, see Plate 2
- Modified California sampler, 3" O.D. split spoon
- Standard Penetration sampler, 2" O.D. split spoon

Earth Investigations Consultants	Job No. 2052.01.01 Date 10/16/06	LOGS OF BORINGS 6 & 7 2880 San Bruno Avenue Burlingame, California	Plate E7

Earth Investigations Consultants, Inc.
Job No. 2271.01.00
10/2008

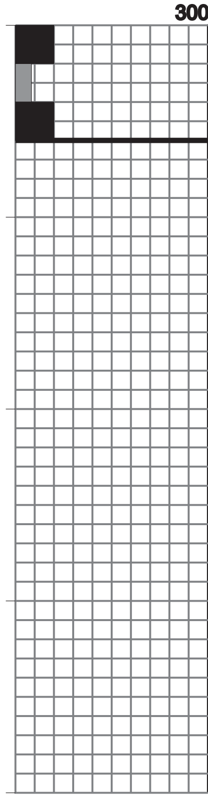
BORING 1 (Soil Probe)

Equipment Portable Percussion Rig

Elevation ~470' **Date** 8/14/2008

Dry Density (pcf)	Moisture Content (%)	Pocket Pen (tsf)	Depth (ft.)	Sample	USCS	Description
			0	Continuous	SM	Yellowish brown Gravelly Silty SAND, moist, medium dense (FILL)
124.9	7.1			Continuous	ROCK	Yellowish brown and dark yellowish brown SHEARED ROCK, very weathered, closely fractured, soft (BEDROCK)
			5			Terminated at 3'

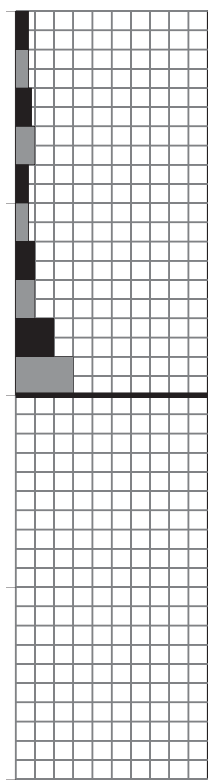
Penetration Rate (sec./ft.) 300



BORING 2 (Soil Probe)

Elevation ~457' **Date** 8/14/2008

Dry Density (pcf)	Moisture Content (%)	Pocket Pen (tsf)	Depth (ft.)	Sample	USCS	Description
			0	Continuous	SC	Yellowish brown and dark yellowish brown Gravelly Clayey SAND, moist, medium dense (FILL)
101.4	6.4					
107.2	9.1		5			
120.9	12.0			Continuous	SC	Dark brown Clayey SAND with Gravel, moist, medium dense (COLLUVIUM)
112.5	9.1		10		ROCK	Yellowish brown SANDSTONE, very weathered, closely fractured, soft (BEDROCK)
			15			Terminated at 10'



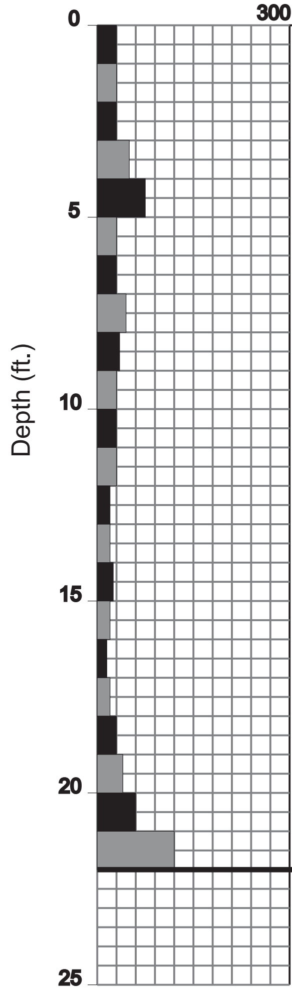
BORING 3 (Soil Probe)

Equipment Portable Percussion Rig

Elevation ~457' **Date** 8/14/2008

Dry Density (pcf)	Moisture Content (%)	Pocket Pen (tsf)
114.3	8.4	
123.9	6.5	
123.4	6.7	
118.8	6.7	
102.9	15.4	3.0
118.1	15.4	
117.9	11.8	

Penetration Rate (sec./ft.)



Sample	USCS	Description
Continuous	SC	Yellowish brown and dark yellowish brown Gravelly Clayey SAND, moist, medium dense (FILL)
	CL	Dark yellowish to greyish brown Sandy CLAY, damp, stiff (COLLUVIUM)
	ROCK	Yellowish brown and grey SHEARED ROCK, very weathered, closely fractured, soft (BEDROCK)

Terminated at 22'

Earth Investigations Consultants	Job No. 2271.01.00 Date 10/13/08	LOG OF BORING 3 (Soil Probe) 850 Glenview Drive San Bruno, California	Plate E9

BORING 1 (Soil Probes)

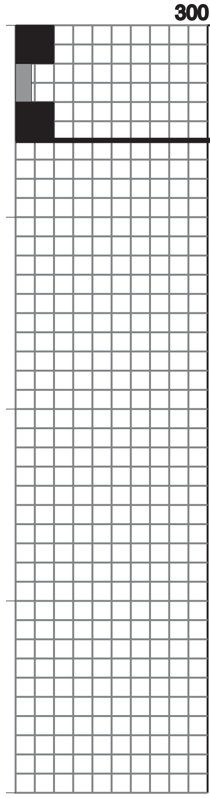
Equipment Portable Percussion Rig

Elevation✦ ~470' **Date** 8/14/2008

Dry Density (pcf)*	Moisture Content (%)	Pocket Pen (tsf)	Depth (ft.)	Sample	USCS	Description
			0	Continuous	SM	Yellowish brown Gravelly, Silty SAND, moist, medium dense (FILL)
124.9	7.1				ROCK	Yellowish brown and dark yellowish brown SHEARED ROCK, very weathered, closely fractured, soft (FRANCISCAN ASSEMBLAGE)

Terminated at 3'

Penetration Rate (sec./ft.) 300

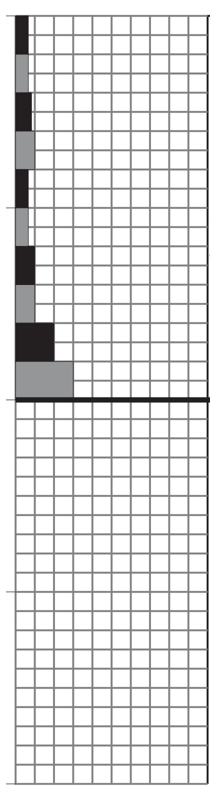


BORING 2 (Soil Probes)

Elevation✦ ~457' **Date** 8/14/2008

Dry Density (pcf)*	Moisture Content (%)	Pocket Pen (tsf)	Depth (ft.)	Sample	USCS	Description
			0	Continuous	SC	Yellowish brown and dark yellowish brown Gravelly, Clayey SAND, moist, medium dense (FILL)
101.4	6.4					
107.2	9.1		5			
120.9	12.0				SC	Dark brown Clayey SAND with Gravel, moist, medium dense (COLLUVIUM)
112.5	9.1		10		ROCK	Yellowish brown SANDSTONE, very weathered, closely fractured, soft (FRANCISCAN ASSEMBLAGE)

Terminated at 10'



Note to Reader:
Probes are from the EIC 2008 report; and are included in the 2013 report (Appendix A).

✦ Elevation from Plate 2 - Site Plan
* Disturbed sample

Earth Investigations Consultants	Job No. 2479.02.00 Date 8/4/13	LOGS OF BORINGS 1 & 2 (Soil Probes) 850 Glenview Drive San Bruno, California	Plate E10

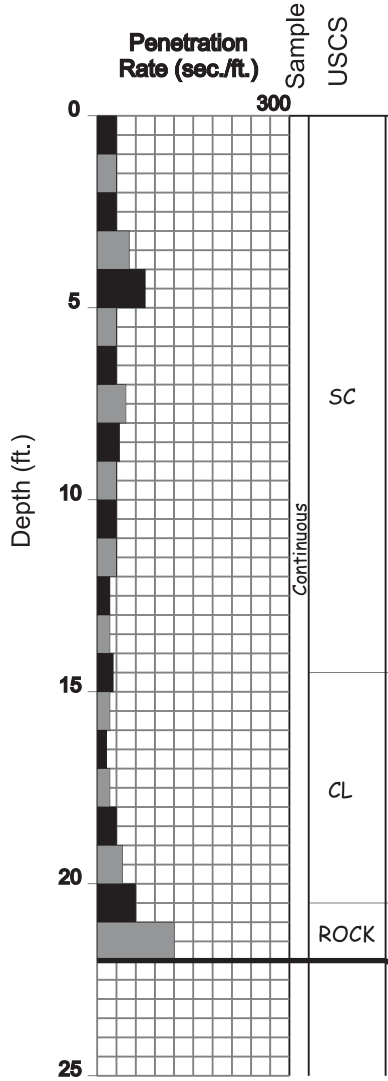
Earth Investigations Consultants, Inc.
Job No. 2479.02.00
08/2013

BORING 3 (Soil Probe)

Equipment Portable Percussion Rig

Elevation✦ ~457' **Date** 8/14/2008

Dry Density (pcf)*	Moisture Content (%)	Pocket Pen (tsf)
114.3	8.4	
123.9	6.5	
123.4	6.7	
118.8	6.7	
102.9	15.4	3.0
118.1	15.4	
117.9	11.8	



USCS	Description
SC	Yellowish brown and dark yellowish brown Gravelly, Clayey SAND, moist, medium dense (FILL)
CL	Dark yellowish to greyish brown Sandy CLAY, damp, stiff (COLLUVIUM)
ROCK	Yellowish brown and grey SHEARED ROCK, very weathered, closely fractured, soft (FRANCISCAN ASSEMBLAGE)

Terminated at 22'

Note to Reader:
Probes are from the EIC 2008 report; and are included in the 2013 report (Appendix A).

✦ Elevation from Plate 2 - Site Plan
* Disturbed sample

Earth Investigations Consultants

Job No. 2479.02.00

Date 8/4/13

LOG OF BORING 3 (Soil Probe)

850 Glenview Drive
San Bruno, California

Plate E11

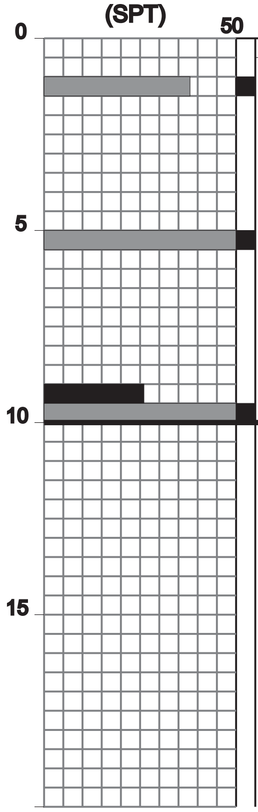
BORING 4

Equipment Truck-Mounted Flight Auger

Elevation✦ ~466′ **Date** 6/25/2013

Dry Density (pcf)	Moisture Content (%)	Blows/Foot (SPT)	USCS	Description
		38/3″	SM	Yellowish brown Silty SAND, moist, medium dense (FILL)
		50/4″	ROCK	Yellowish brown and dark yellowish brown SHEARED ROCK, very weathered, closely fractured, soft to moderately hard (FRANCISCAN ASSEMBLAGE)
121.9	7.8	76		

Terminated at 10′

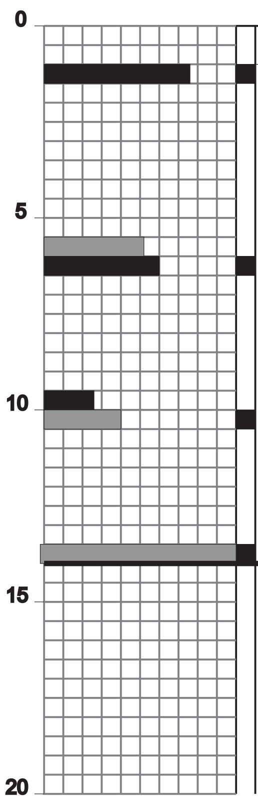


BORING 5

Elevation✦ ~467″ **Date** 6/25/2013

Dry Density (pcf)	Moisture Content (%)	Blows/Foot (SPT)	USCS	Description
		38/3″	SM	Yellowish brown Gravelly, Silty SAND, moist, medium dense (FILL)
113.9	6.7	56	ROCK	Dark yellowish brown and dark grey SHEARED ROCK, very weathered, closely fractured, soft to moderately hard (FRANCISCAN ASSEMBLAGE)
100.5	13.8	23		
		50/5″		

Terminated at 13′ 11″



✦ Elevation from Plate 2 - Site Plan

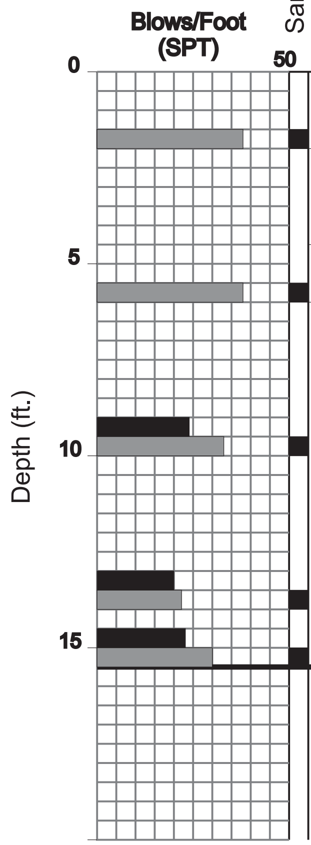
Earth Investigations Consultants	Job No. 2479.02.00 Date 8/4/13	LOGS OF BORINGS 4 & 5 850 Glenview Drive San Bruno, California	Plate E12

BORING 6

Equipment Truck-Mounted Flight Auger

Elevation✦ ~463′ **Date** 6/25/2013

Dry Density (pcf)	Moisture Content (%)	Blows/Foot (SPT)
111.4	9.3	38/3″
122.9	11.4	38/3″
108.2	14.3	57
		42
121.6	9.7	53



USCS	Description
SM	Brown Gravelly Silty SAND, moist, very dense (FILL)
SC	Dark yellowish brown Clayey SAND with Gravel, moist, very dense (COLLUVIUM)
CL	Dark yellowish brown Sandy CLAY with Gravel, damp, hard (COLLUVIUM)
ROCK	Dark yellowish brown and greyish brown SHEARED ROCK, very weathered, closely fractured, soft (FRANCISCAN ASSEMBLAGE)

Terminated at 15½′

✦ Elevation from Plate 2 - Site Plan

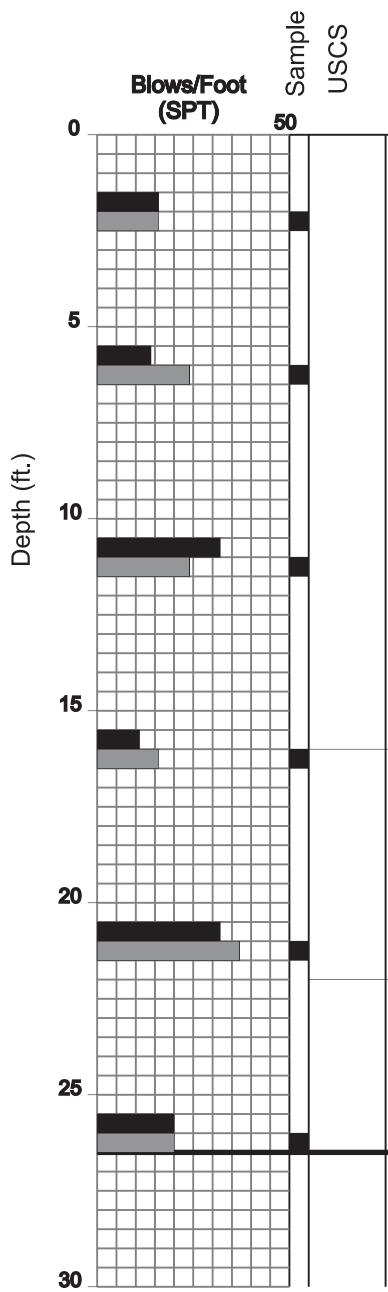
Earth Investigations Consultants	Job No. 2479.02.00 Date 8/4/13	LOG OF BORING 6 850 Glenview Drive San Bruno, California	Plate E13

BORING 7

Equipment Truck-Mounted Flight Auger

Elevation✦ ~457′ **Date** 6/25/2013

Dry Density (pcf)	Moisture Content (%)	Blows/Foot (SPT)	Depth (ft.)	Description
			0	Brown Silty SAND with Gravel, moist, dense (FILL)
113.8	6.1	32		color changes to dark yellowish brown, grades to Gravelly Silty SAND with Clay
118.9	5.7	38	5	
				color changes yellowish brown at 8′
		56	10	
104.8	14.5	27	15	Dark brown Sandy CLAY with Gravel, damp, very stiff to hard (COLLUVIUM)
				color changes to dark yellowish brown at 19′
106.5	18.1	69	20	
				Dark grey SHEARED ROCK, very weathered, closely fractured, soft (FRANCISCAN ASSEMBLAGE)
118.6	11.61	40	25	
				Terminated at 26½′



✦ Elevation from Plate 2 - Site Plan

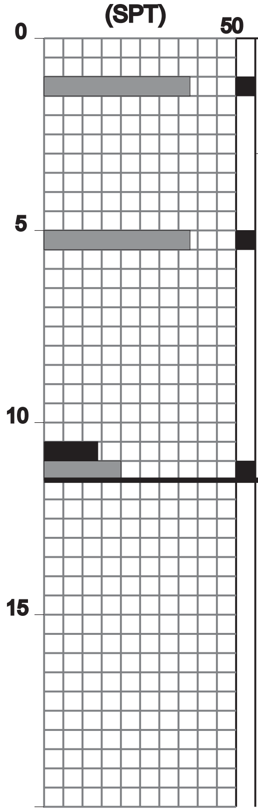
Earth Investigations Consultants	Job No. 2479.02.00 Date 8/4/13	LOG OF BORING 7 850 Glenview Drive San Bruno, California	Plate E14

BORING 8

Equipment: Truck-Mounted Flight Auger

Elevation✦: ~461′ Date: 6/25/2013

Dry Density (pcf)	Moisture Content (%)	Blows/Foot (SPT)	USCS	Description
120.4	5.3	38/5″		Yellowish brown Gravelly Silty SAND, moist, very dense (FILL)
		38/5″		Yellowish brown SANDSTONE, very weathered, closely fractured, hard (FRANCISCAN ASSEMBLAGE)
112.1	5.8	90/11″		

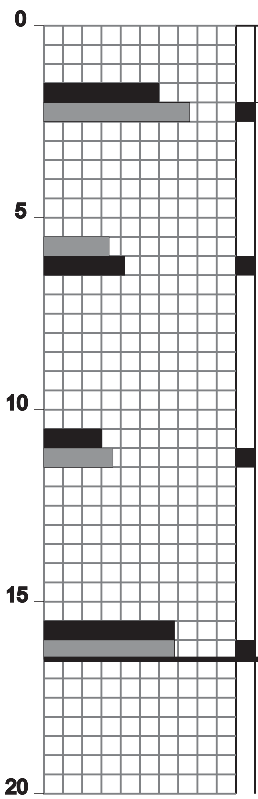


Terminated at 11′ 11″

BORING 9

Elevation✦: ~441″ Date: 6/25/2013

Dry Density (pcf)	Moisture Content (%)	Blows/Foot (SPT)	USCS	Description
112.1	11.5	68/9″	SC	Dark yellowish brown Clayey SAND with Gravel, moist, very dense (FILL)
110.6	11.3	38	ROCK	Dark yellowish brown and dark grey SHEARED ROCK, very weathered, closely fractured, soft (FRANCISCAN ASSEMBLAGE)
120.0	8.7	33		
133.4	9.8	68		



Terminated at 16½′

✦ Elevation from Plate 2 - Site Plan

Earth Investigations Consultants	Job No. 2479.02.00 Date 8/4/13	LOGS OF BORINGS 8 & 9 850 Glenview Drive San Bruno, California	Plate E15

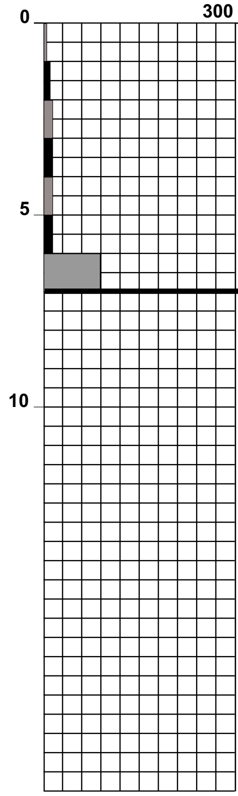
Earth Investigations Consultants, Inc.
Job No. 2479.02.00
02/2016

BORING 3 (Soil Probe)

Equipment Portable Percussion Rig

Elevation ~465' **Date** 01/15/16

Dry Density (pcf)	Moisture Content (%)	Pocket Pen (tsf)
124.5	8.8	
		>4.5
106.8	17.8	
		>4.5



Sample: Continuous

USCS: CL

Yellow brown Sandy CLAY with Gravel, damp, firm (FILL?)

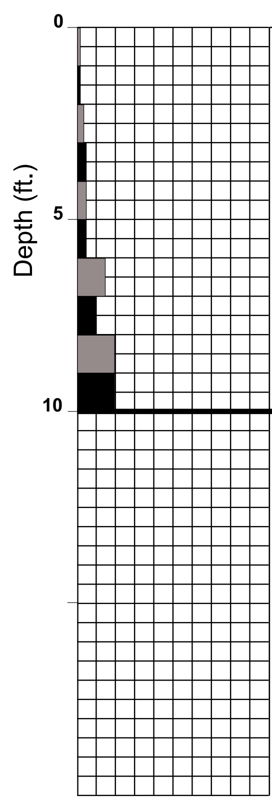
color changes to yellow brown & dark yellow brown @ 2 ½'

Refusal @ 7'

BORING 4 (Soil Probe)

Elevation ~465' **Date** 01/15/16

Dry Density (pcf)	Moisture Content (%)	Pocket Pen (tsf)
105.2	22.4	
		3.0
108.9	18.9	
		>4.5
119.1	9.0	>4.5



Sample: Continuous

USCS: CL

Yellow brown, dark yellow brown and grey Sandy CLAY with Gravel, damp, firm (FILL?)

USCS: ROCK

Dark brown SHEARED ROCK very weathered, closely fractured, soft

color changes to yellow brown @ 9'

Refusal @ 10'

Earth Investigations Consultants, Inc.	Job No. 2479.02.00 Approved By: JEB Date 02/13/16	**LOGS OF BORINGS 3 & 4 (Soil Probes)** 850 Glenview Drive, 2880 & 2890 San Bruno Avenue San Bruno, California	Plate E16

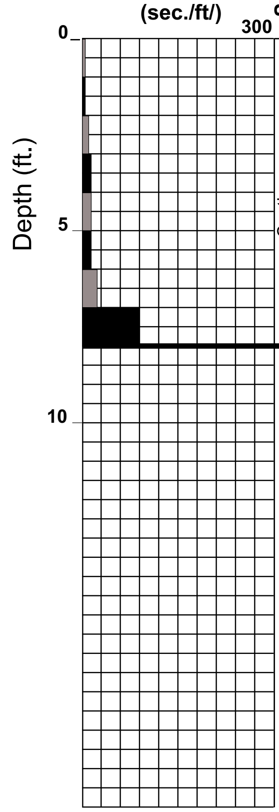
BORING 1 (Soil Probe)

Equipment: Portable Percussion Rig

Elevation: ~465' Date: 01/15/16

Dry Density (pcf)	Moisture Content (%)	Pocket Pen (tsf)	Depth (ft.)	Sample	USCS	Description
110.9	8.4		0–2.5	Continuous	CL	Dark yellow brown Gravelly Sandy CLAY, damp, firm (FILL)
			2.5–7	Continuous	SC	Yellow brown Gravelly Clayey SAND, moist, medium-dense (FILL?)
125.3	7.6		7–8	Continuous	ROCK	Yellow brown SHEARED ROCK(?) very weathered, closely fractured, moderately hard

Refusal @ 8'

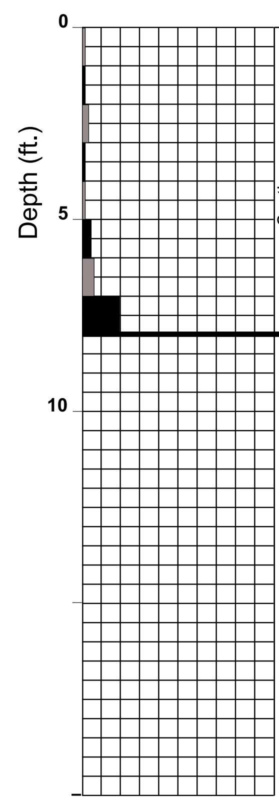


BORING 2 (Soil Probe)

Elevation: ~465' Date: 01/15/16

Dry Density (pcf)	Moisture Content (%)	Pocket Pen (tsf)	Depth (ft.)	Sample	USCS	Description
85.5	22.7		0–3	Continuous	SC	Yellow brown Gravelly Clayey SAND, damp, loose to medium dense (FILL)
115.0	16.1		3–6	Continuous	CL	Dark yellow brown Sandy CLAY, damp, firm (COLLUVIUM)
112.4	15.2	1.0	6–8	Continuous	ROCK	Dark yellow brown SHEARED ROCK(?), very weathered, closely fractured, soft

Refusal @ 8'



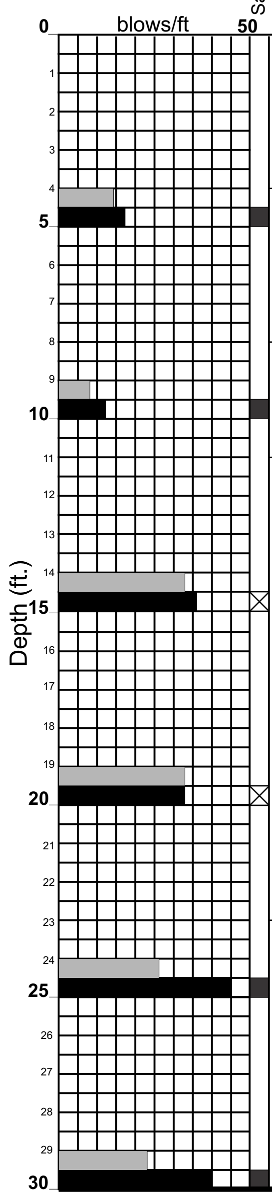
Earth Investigations Consultants, Inc.	Job No. 2479.02.00 Approved By: JEB Date 02/13/16	**LOGS OF BORINGS 1 & 2 (Soil Probes)** 850 Glenview Drive, 2880 & 2890 San Bruno Avenue San Bruno, California	Plate E17

Geosphere Consultants, Inc.
Job No. 91-04747-A
01/2020

BORING X

Equipment Hollow Stem Flight Auger

Elevation ~465 ft. **Date** 12/06/19



Depth (ft.)	-200 Sieve (%)	Dry Density (pcf)	Moisture Content %	Blows/Foot (SPT)	USCS	Description
0–4					SW	2½" AC/Grayish brown Gravelly SAND (Baserock) sewer pipe @ 3'
4–8 (sample @ 5)	54.3	81.7	40.6	31	MH	Yellow brown & dark yellow brow Sandy CLAY with Gravel, damp stiff (FILL)
8–11 (sample @ 10)	33.4	110.8	6.9	20	SC	Dark yellow brown Sandy CLAY with Gravel, damp, very stiff (COLLUVIUM)
11–23 (samples @ 15, 20)				70 66	ROCK	Yellow brown SANDSTONE, very weathered, closely fractured, moderately hard hard
23–30 (samples @ 25, 30)	19.4	126.4	9.7	70 63	ROCK	Yellow brown Sandy CLAYSTONE very weathered, closely fractured, moderately hard

Terminated @ 30'

Geosphere Consultants, Inc.

Job No.: 91-04747-A
Approved: JEB
Date: 01.21.2020

LOG OF BORING X

Glenview Terrace Residential Development
2880 San Bruno Avenue
San Bruno, California

Plate E18

BORING Y

Equipment Hollow Stem Flight Auger

Elevation ~465 ft. **Date** 12/06/19

Depth (ft.)	Blows/Foot (SPT)	USCS	Description
0–2		CL	Dark brown Sandy CLAY with Gravel, damp, firm (FILL)
2		ROCK	Yellow brown & dark yellow brown SHEARED ROCK, very weathered, closely fractured, moderately hard
4–5	38		hard
12			becomes grayish brown & dark grayish brown @ 12’
14–15	90		
19–20	50		
24–25	74		
29			Terminated @ 29’

Geosphere Consultants, Inc.

Job No.: 91-04747-A
Approved: JEB
Date: 01.21.2020

LOG OF BORING Y

Glenview Terrace Residential Development
2880 San Bruno Avenue
San Bruno, California

Plate E19

BORING Z

Equipment Hollow Stem Flight Auger

Elevation ~332 ft. **Date** 12/06/19

Depth (ft.)	Blows/Foot (SPT)	USCS	Description
0		CL	Dark brown Sandy CLAY with Gravel, damp, firm (FILL)
2½			becomes yellow brown and more Clayey @ 2½'
4	38/5"	ROCK	Yellow brown, dark yellow brown, & grayish brown SHEARED ROCK, very weathered, closely fractured, moderately hard
9	38		becomes gray & grayish brown @ 9'
14	50/5"		
19–20	76/11"		
23			becomes yellow brown @ 23'
29			Terminated @ 29'

Geosphere Consultants, Inc.

Job No.: 91-04747-A
Approved: JEB
Date: 01.21.2020

LOG OF BORING Z

Glenview Terrace Residential Development
2880 San Bruno Avenue
San Bruno, California

Plate E20

APPENDIX F

This appendix contains logs of site-specific fault exploration trench logs (Romig Engineers, 2008, and Earth Investigations Consultants, 2008), and nearby off-site fault exploration trench logs (BAGG, 2003, 2007).

BAGG, Job KENMR-02-01, 12/2003
- Plate 4 - Site Plan
- Plate 5 - Trench 1 Cross Section
- Plate 6 - Trench 2 Cross Section
- Plate 7 - Trench 3 Cross Section
- Plate 8 - Trench 4 Cross Section

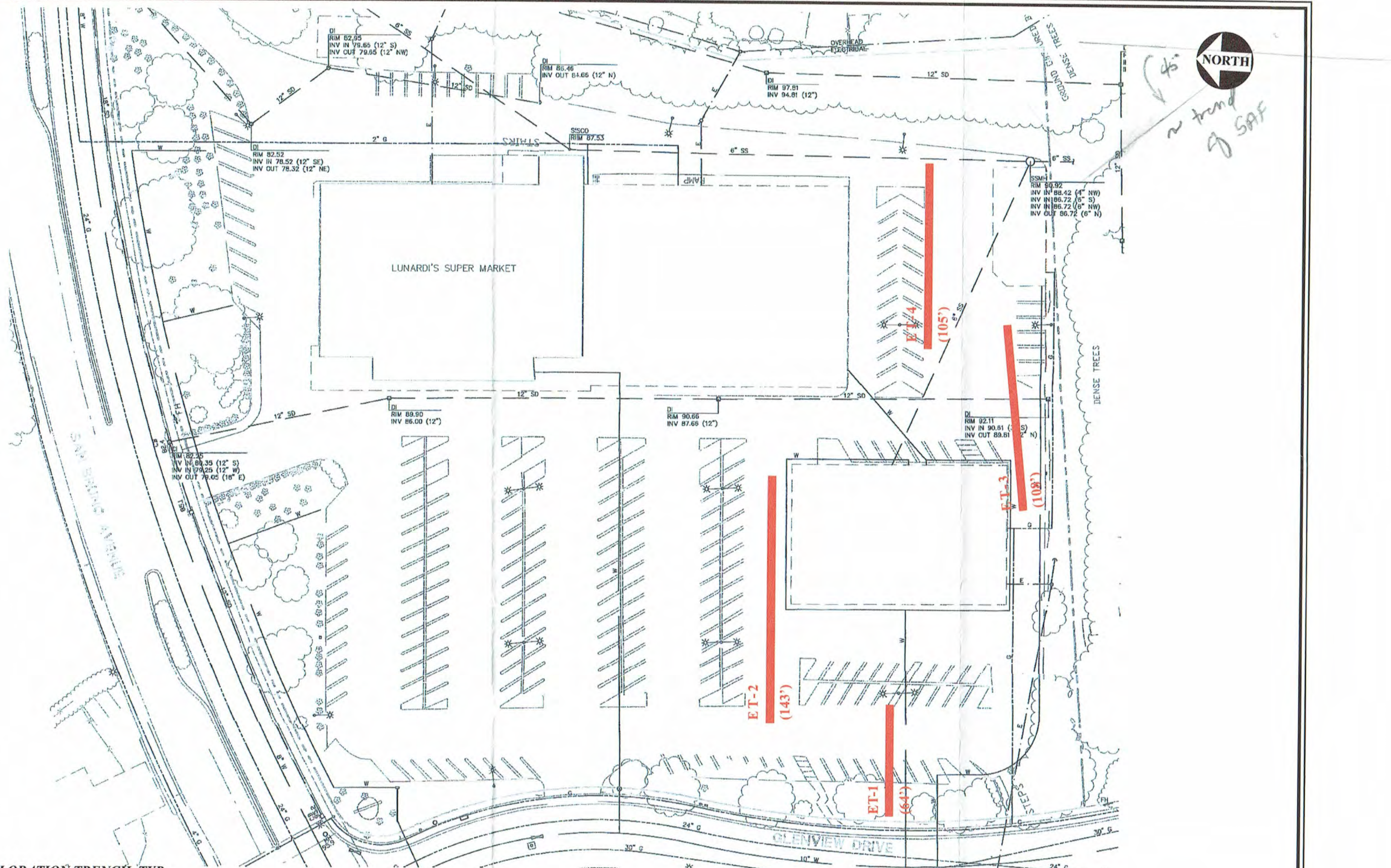
BAGG, Job SUTTI-01-00, 12/2007
- Plate 2 - Site Plan
- Plate 3 - Exploratory Trench Cross Section
- Plate 6 - Trench 2 Cross Section

Romig Engineers, Project No. 2178-1, 09/2008
- Figure 1 - Fault Exploration Trench Log TR-1
- Figure 2 - Fault Exploration Trench Logs TR-2 & TR-3
- Figure 3 - Descriptions of Materials Encountered in Trenches

Earth Investigations Consultants, Job No. 2271.01.00, 10/2008
- Plate A5 - Logs of Trenches

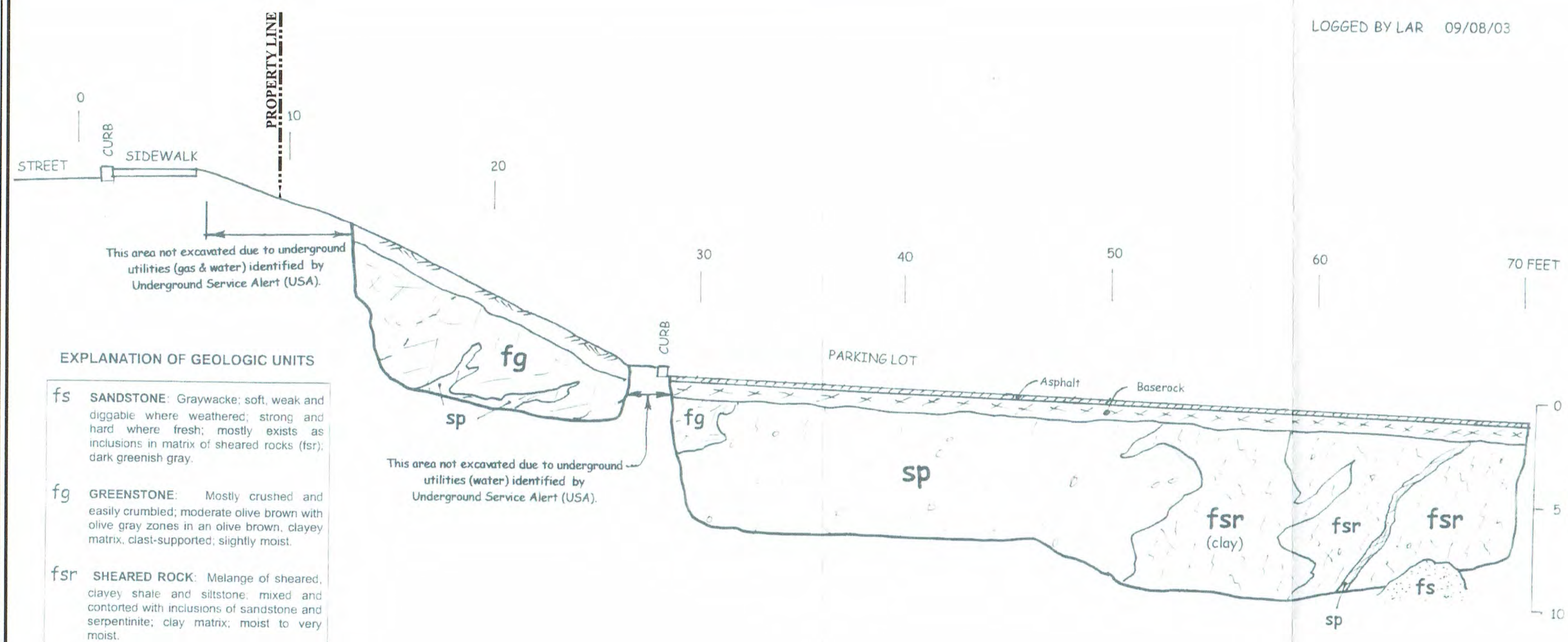
BAGG, Job KENMR-02-01, December 2003

- Plate 4 - Site Plan
- Plate 5 - Trench 1 Cross Section
- Plate 6 - Trench 2 Cross Section
- Plate 7 - Trench 3 Cross Section
- Plate 8 - Trench 4 Cross Section



APPROXIMATE LOCATION OF EXPLORATION TRENCH, TYP.

Base Map: *"Skycrest Shopping Center, Existing Utility Exhibit, San Bruno, San Mateo County, California,"* prepared by BKF Engineers/Surveyors/Planners, Job No. 20037060, dated July 29, 2003.	**SKY CREST CENTER** **SAN BRUNO AVENUE @ GLENVIEW DRIVE** **SAN BRUNO, CALIFORNIA**	BAGG	**SITE PLAN**			
			JOB NO. KENMR-02-01	SCALE 1" = 60'	DATE December 2003	PLATE 4



EXPLANATION OF GEOLOGIC UNITS

fs **SANDSTONE**: Graywacke; soft, weak and diggable where weathered; strong and hard where fresh; mostly exists as inclusions in matrix of sheared rocks (fsr); dark greenish gray.

fg **GREENSTONE**: Mostly crushed and easily crumbled; moderate olive brown with olive gray zones in an olive brown, clayey matrix, clast-supported; slightly moist.

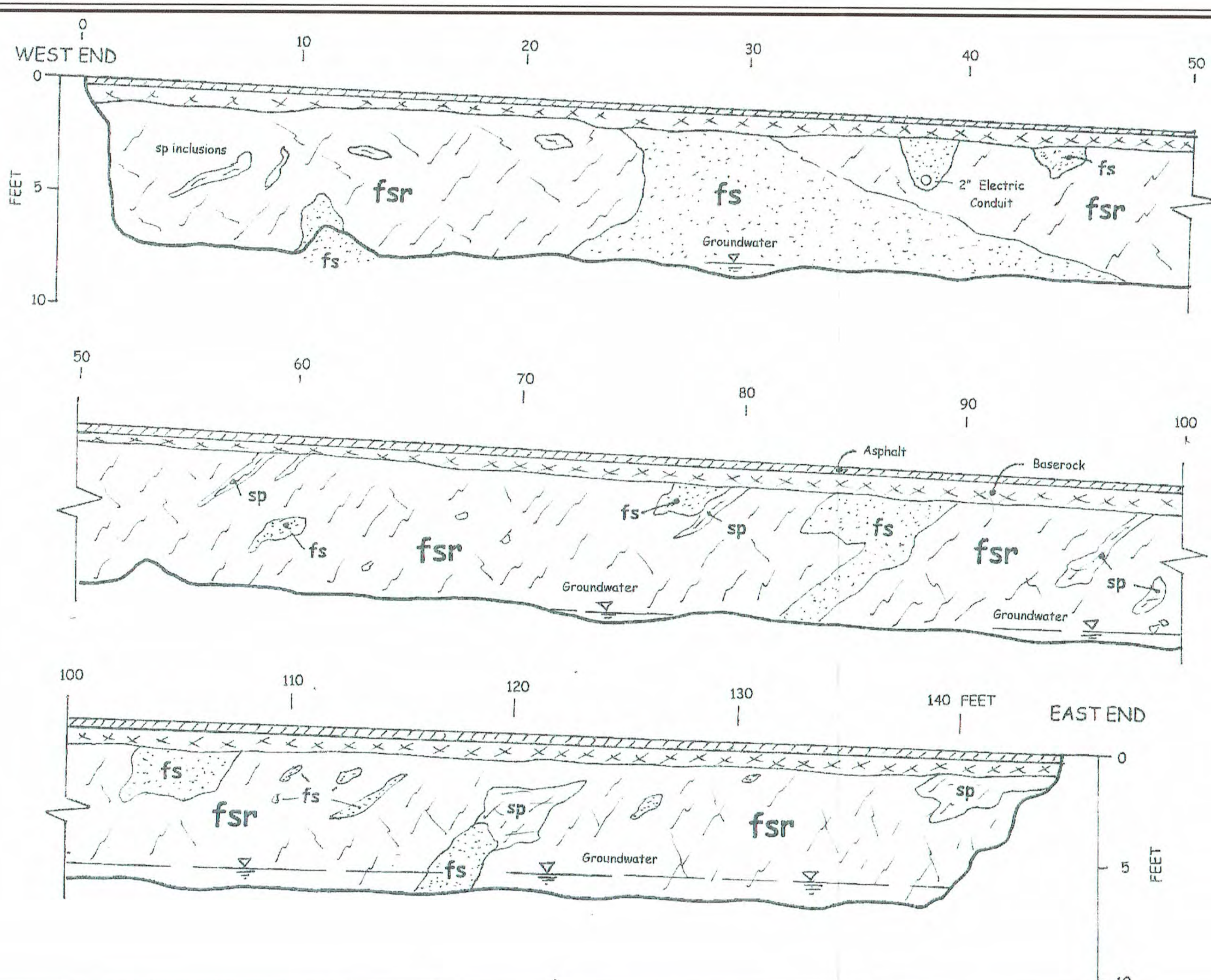
fsr **SHEARED ROCK**: Melange of sheared, clayey shale and siltstone; mixed and contorted with inclusions of sandstone and serpentinite; clay matrix; moist to very moist.

sp **SERPENTINITE**: Crushed and sheared; occasional hard blocks and cobbles with polished surfaces; many carbonate-rich coatings and nodules; blue-green; moist.

TRENCH ET-1

SKY CREST CENTER SAN BRUNO AVENUE @ GLENVIEW DRIVE SAN BRUNO, CALIFORNIA	BAGG	**TRENCH 1 CROSS SECTION**			
		JOB NO. KENMR-02-01	SCALE AS NOTED	DATE December 2003	PLATE 5

LOGGED BY LAR 09/16/03



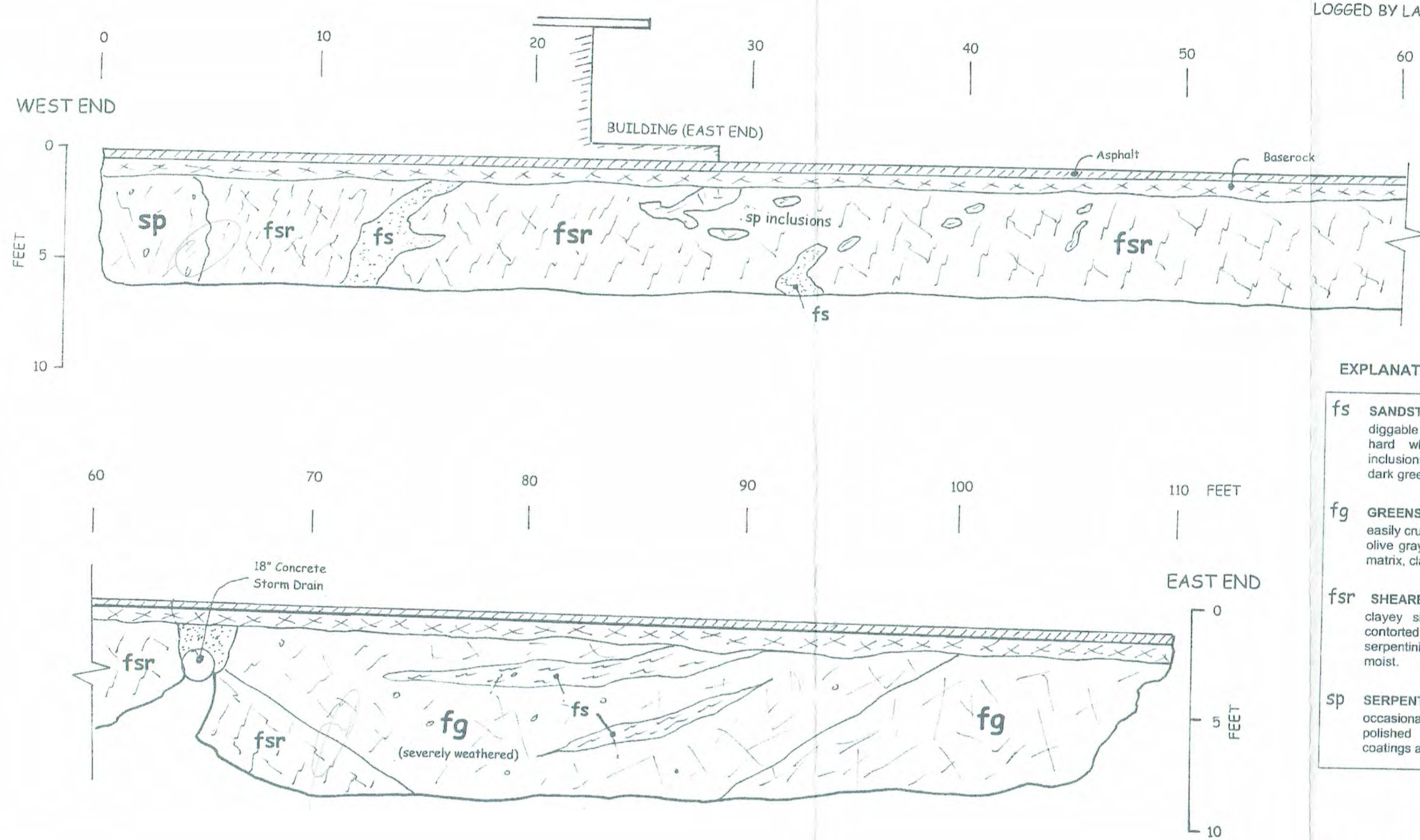
EXPLANATION OF GEOLOGIC UNITS

fs	**SANDSTONE:** Graywacke; soft, weak and diggable where weathered; strong and hard where fresh; mostly exists as inclusions in matrix of sheared rocks (fsr); dark greenish gray.
fg	**GREENSTONE:** Mostly crushed and easily crumbled; moderate olive brown with olive gray zones in an olive brown, clayey matrix, clast-supported; slightly moist.
fsr	**SHEARED ROCK:** Melange of sheared, clayey shale and siltstone; mixed and contorted with inclusions of sandstone and serpentinite; clay matrix; moist to very moist.
sp	**SERPENTINITE:** Crushed and sheared; occasional hard blocks and cobbles with polished surfaces; many carbonate-rich coatings and nodules; blue-green; moist.

TRENCH ET-2

SKY CREST CENTER SAN BRUNO AVENUE @ GLENVIEW DRIVE SAN BRUNO, CALIFORNIA	BAGG	TRENCH 2 CROSS SECTION			
		JOB NO. KENMR-02-01	SCALE AS NOTED	DATE December 2003	PLATE 6

LOGGED BY LAR 09/09/03



EXPLANATION OF GEOLOGIC UNITS

fs	**SANDSTONE:** Graywacke; soft, weak and diggable where weathered; strong and hard where fresh; mostly exists as inclusions in matrix of sheared rocks (fsr); dark greenish gray.
fg	**GREENSTONE:** Mostly crushed and easily crumbled; moderate olive brown with olive gray zones in an olive brown, clayey matrix, clast-supported; slightly moist.
fsr	**SHEARED ROCK:** Melange of sheared, clayey shale and siltstone; mixed and contorted with inclusions of sandstone and serpentinite; clay matrix; moist to very moist.
sp	**SERPENTINITE:** Crushed and sheared; occasional hard blocks and cobbles with polished surfaces; many carbonate-rich coatings and nodules; blue-green; moist.

TRENCH ET-3

SKY CREST CENTER
SAN BRUNO AVENUE @ GLENVIEW DRIVE
SAN BRUNO, CALIFORNIA

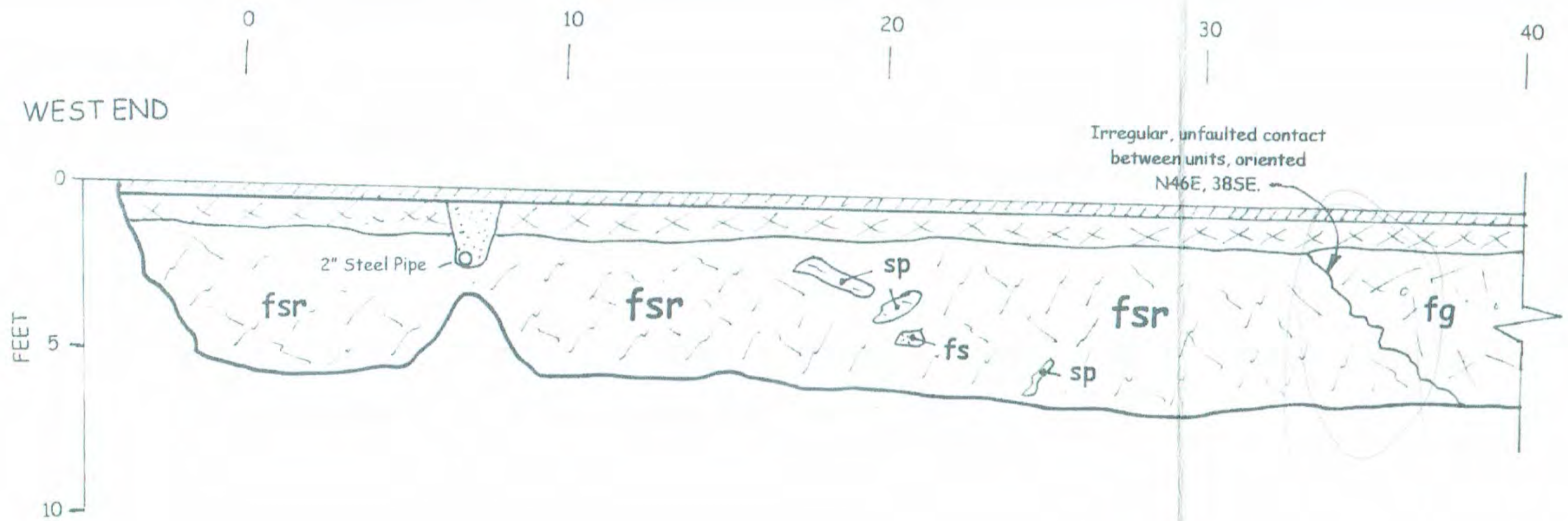
BAGG

TRENCH 3 CROSS SECTION

JOB NO.	SCALE	DATE	PLATE
KENMR-02-01	AS NOTED	December 2003	7

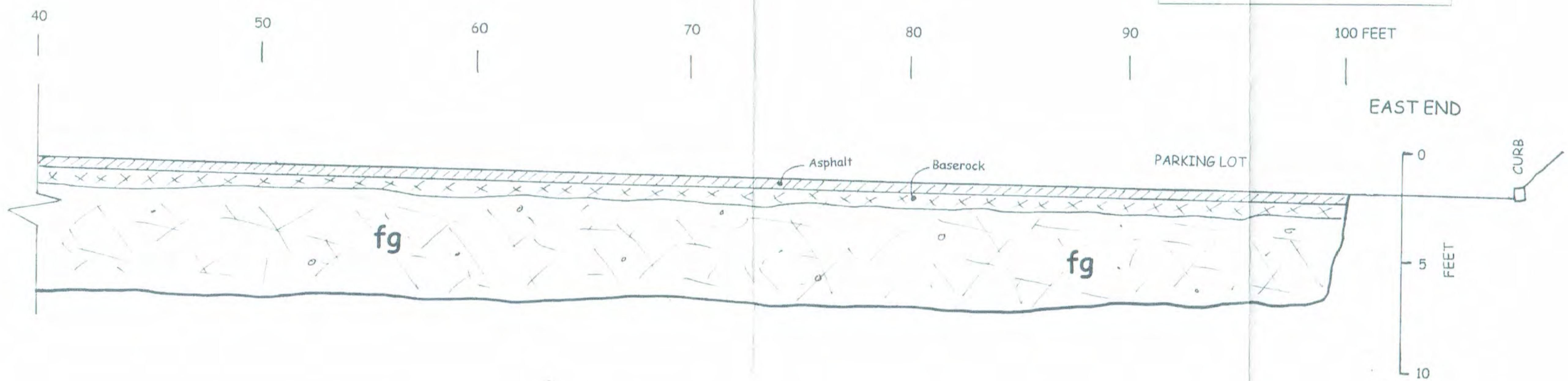
BAGG, Job SUTTI-01-00, December 2007

- Plate 2 - Site Plan
- Plate 3 - Exploratory Trench Cross Section
- Plate 6 - Trench 2 Cross Section



EXPLANATION OF GEOLOGIC UNITS

fs	**SANDSTONE:** Graywacke; soft, weak and diggable where weathered; strong and hard where fresh; mostly exists as inclusions in matrix of sheared rocks (fsr); dark greenish gray.
fg	**GREENSTONE:** Mostly crushed and easily crumbled; moderate olive brown with olive gray zones in an olive brown, clayey matrix, clast-supported; slightly moist.
fsr	**SHEARED ROCK:** Melange of sheared, clayey shale and siltstone; mixed and contorted with inclusions of sandstone and serpentinite; clay matrix; moist to very moist.
sp	**SERPENTINITE:** Crushed and sheared; occasional hard blocks and cobbles with polished surfaces; many carbonate-rich coatings and nodules; blue-green; moist.

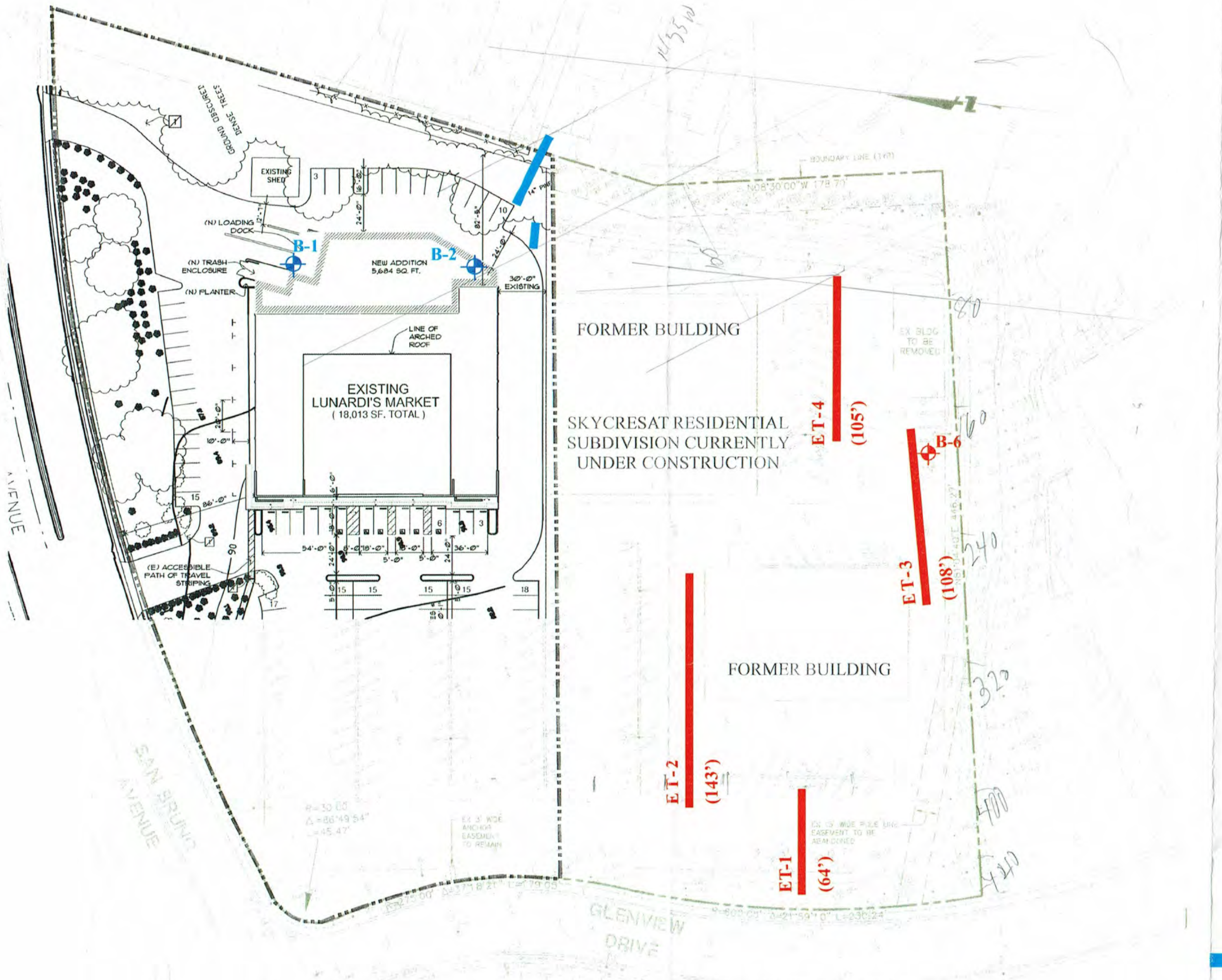


LOGGED BY LAR 09/10/03

TRENCH ET-4

SKY CREST CENTER SAN BRUNO AVENUE @ GLENVIEW DRIVE SAN BRUNO, CALIFORNIA	BAGG	**TRENCH 4 CROSS SECTION**	

JOB NO.	SCALE	DATE	PLATE
KENMR-02-01	AS NOTED	December 2003	8



BORING LOCATIONS, TYP. DRILLED FOR THIS INVESTIGATION

FAULT EXPLORATION TRENCH THAT WAS TRENCHED FOR THIS INVESTIGATION

FAULT EXPLORATION TRENCH (BAGG, 2003)

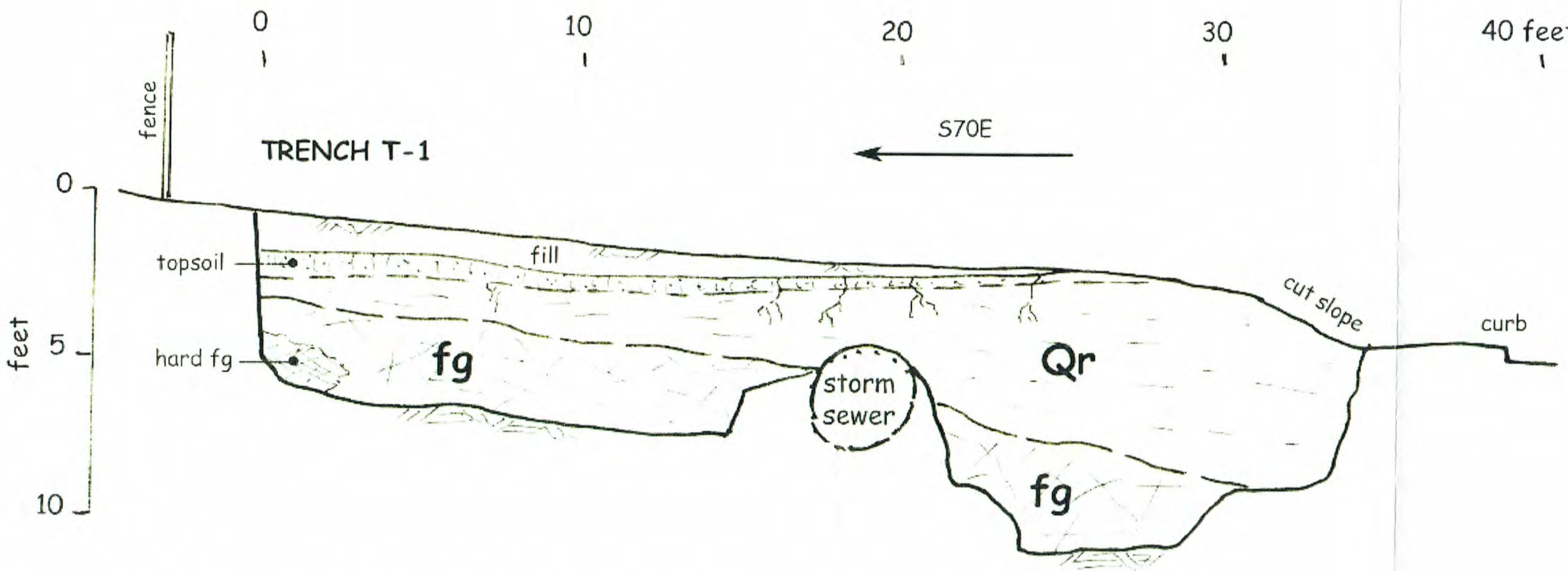
Base Map: *Skycrest, Existing Topography and Record Boundary*, prepared by BKF Engineers, Project: 20037060-10, Sheet C-1, dated July 23, 2004.

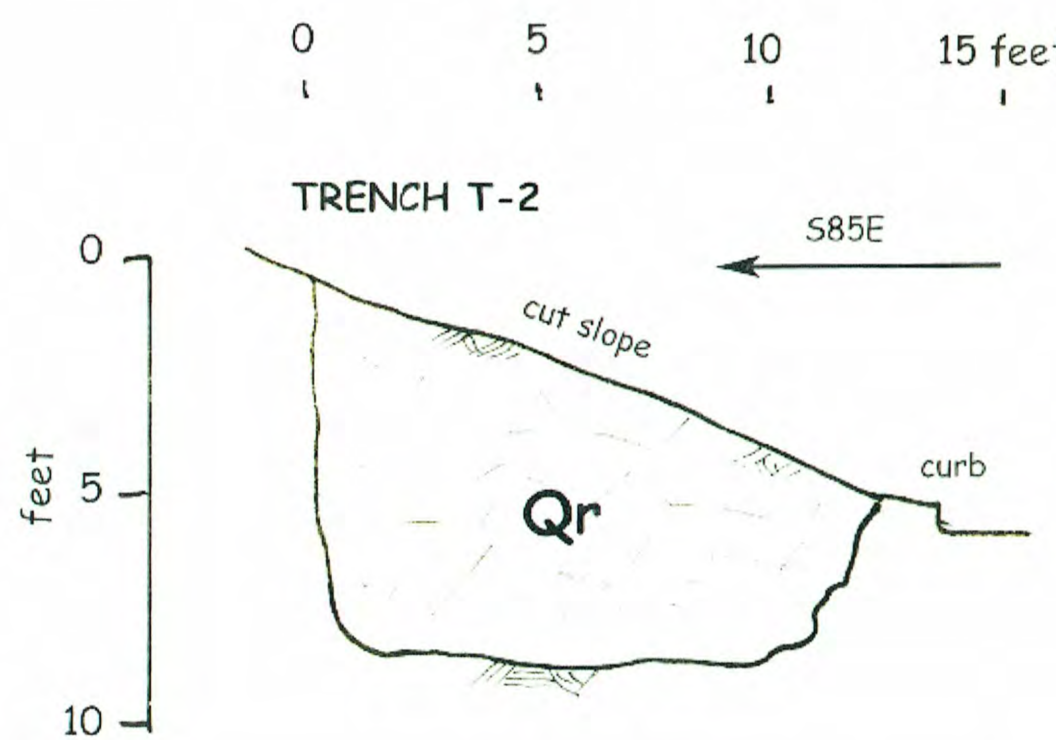
PROPOSED ADDITIONS TO LUNARDI'S MARKET
2801 SAN BRUNO AVENUE
SAN BRUNO, CALIFORNIA

BAGG

SITE PLAN

JOB NO.	SCALE	DATE	PLATE
SUTTI-01-00	1" = 80'	December 2007	2





EXPLANATION

Qr RESIDUAL SOIL: Decomposed greenstone fragments in moderate brown clay matrix (weathered-in-place greenstone bedrock).

fg GREENSTONE: Closely fractured; olive gray (5Y 3/2) with dusky brown coatings; mod. yel. brown clay fillings in fractures; decomposed fine roots in many fractures (Franciscan Complex bedrock).

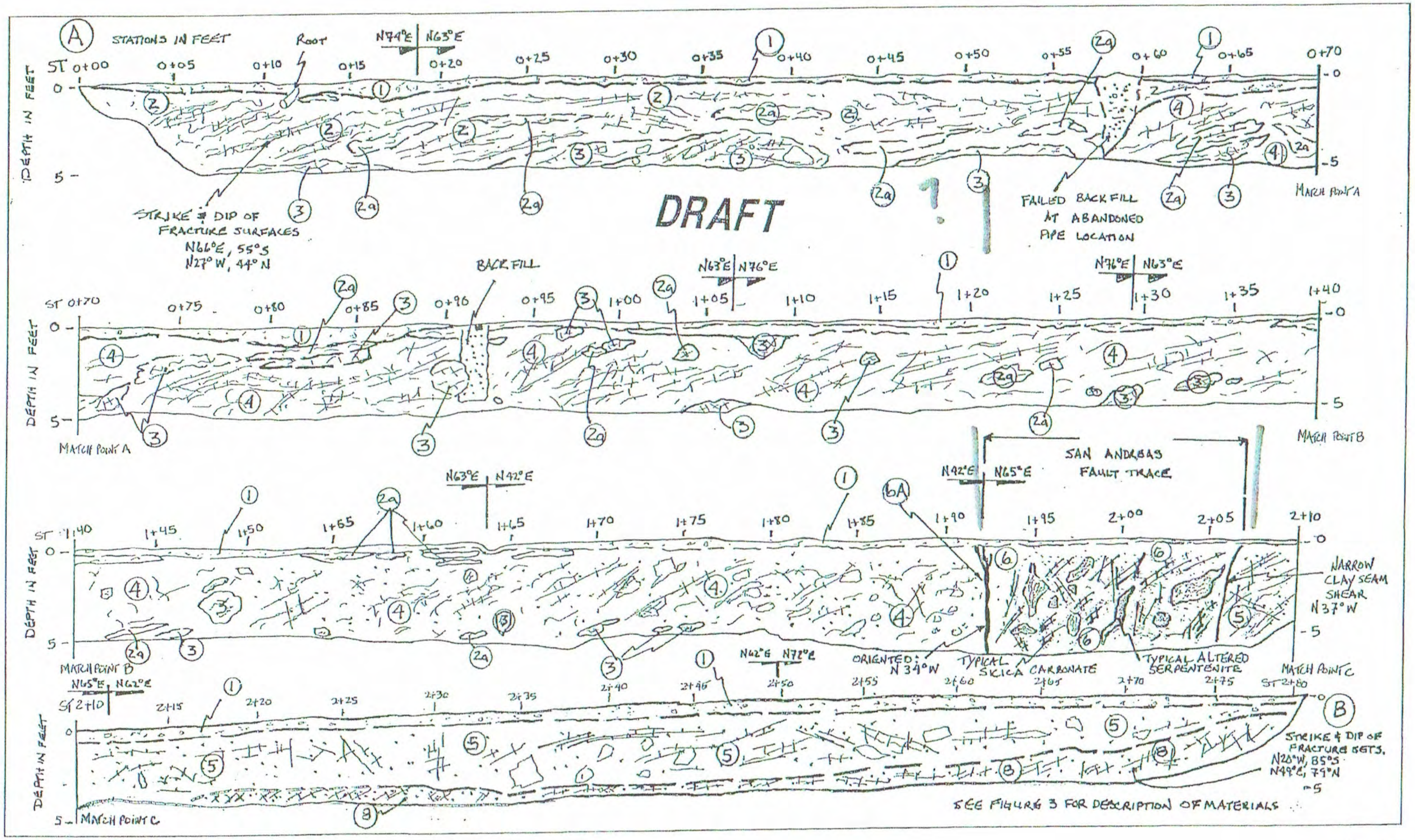
PROPOSED ADDITION TO LUNARDI'S MARKET 2801 SAN BRUNO AVENUE SAN BRUNO, CALIFORNIA	BAGG	EXPLORATORY TRENCH CROSS SECTION			
		JOB NO. SUTTI-01-00	SCALE 1" = 160'	DATE December	PLATE 3

Romig Engineers, Project No. 2178-1, 09/2008

Figure 1 - Fault Exploration Trench Log TR-1

Figure 2 - Fault Exploration Trench Logs TR-2 & TR-3

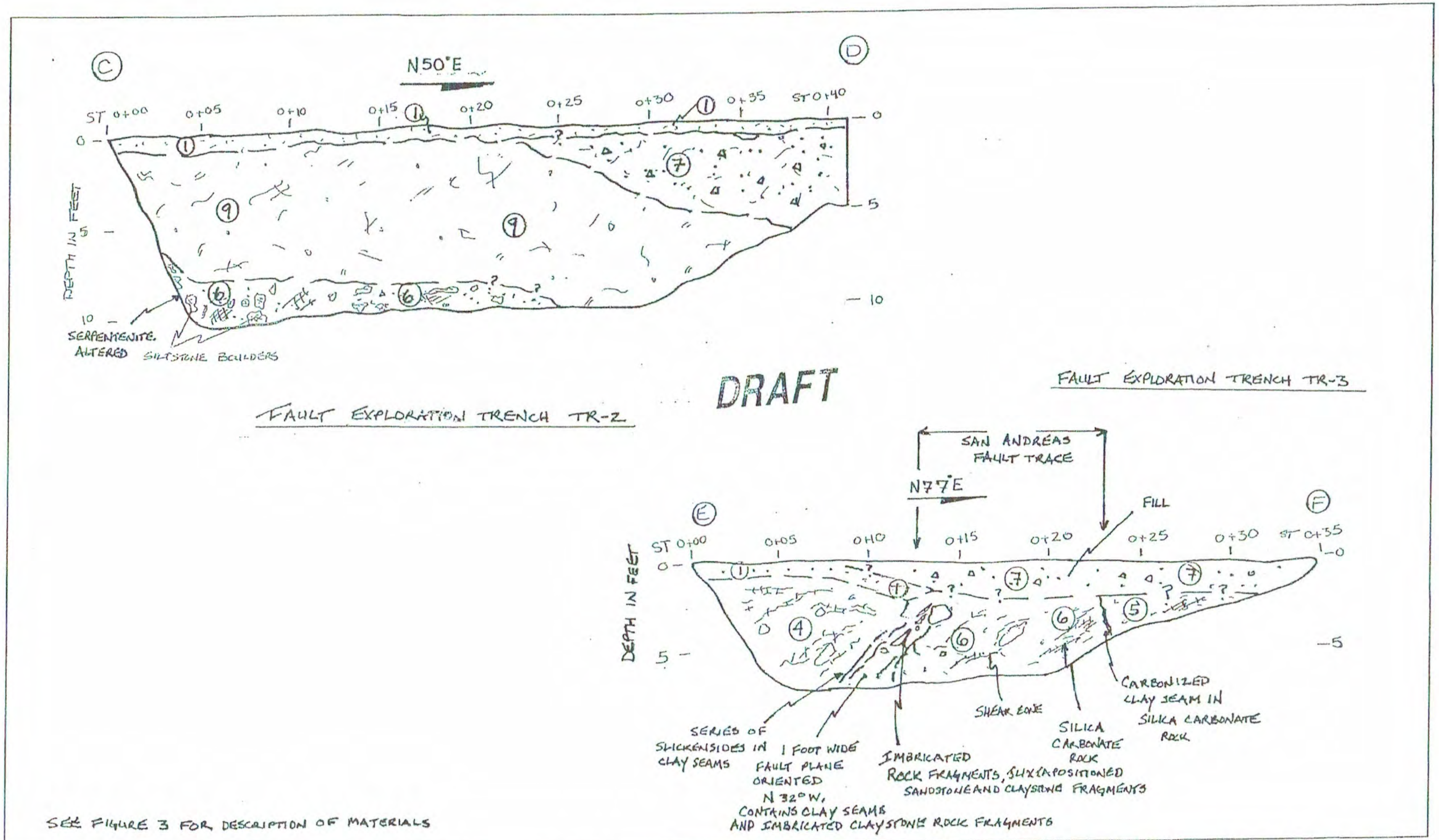
Figure 3 - Descriptions of Materials Encountered in Trenches



FAULT EXPLORATION TRENCH LOG TR-1
GOLDENWOOD
SAN BRUNO, CALIFORNIA

ROMIG ENGINEERS, INC.

FIGURE 1
SEPTEMBER 2008
PROJECT NO. 2178-1



FAULT EXPLORATION TRENCH LOGS TR-2 AND TR-3
GOLDENWOOD
SAN BRUNO, CALIFORNIA

FIGURE 2
SEPTEMBER 2008
PROJECT NO. 2178-1

ROMIG ENGINEERS, INC.

DESCRIPTION OF MATERIALS

(1) **Topsoil:** Brown, Sandy Lean Clay, slightly moist, low plasticity, fine to coarse sand, fine to medium angular to subrounded gravels up to 2-inch diameter, roots.

(2) **Franciscan Sheared Rock:** Tan to light brown, Fine-grained Sandstone with interbedded Siltstone, slightly moist, severely weathered, slightly friabe to firm along 1-2-inch fragments, fractured, tan and light brown to orange mottling, minor gravel and rock imbrication.

(2a) Multicolored, Sandy Lean Clay, moist, iron oxide staining, pervasively sheared, clay infill between fragments and fracture surfaces

(3) **Franciscan Complex:** Medium brown, Siltstone, slighlty moist, friable, soft, three fracture sets.

(4) **Franciscan Complex:** Light orange to light brown, Sheared Unit of Claystone, Sandstone, and Siltstone, moist, severely weathered, friable, iron oxide staining, somewhat blocky texture, some serpentinite.

(5) **Franciscan Complex:** Orange to brown, Highly Sheared Unit of Claystone and Sandstone in clay matrix, moist, friable.

(6) **Fault Gouge:** Light brown to orange brown, Silty/Sandy Lean Clay, moist, fine to medium sand matrix. Contains blocks of silica carbonate and altered serpentenite.

(6a) **Shear Plane:** Gray to dark brown, sheared, vertical 1-1 1/2-inch clay zone, serpentinite along plane. Contains serpentenite, claystone, silica carbonate, and sandstone fragments.

(7) **Fill:** Light brown to brown, Sandy/Gravelly Clay, slightly moist, fine to coarse sand, fine to medium gravel up to 3/4-inch, tan mottling, very stiff, roots.

(8) **Franciscan Complex:** Medium brown well graded sandstone, friable to indurated, contains three fracture sets, slightly moist.

(9) Brown Sandy Lean Clay, moist, low plasticity, stiff to very stiff, some fine to medium grained sand, contains small subrounded gravels and roots, some orange/tan mottling.

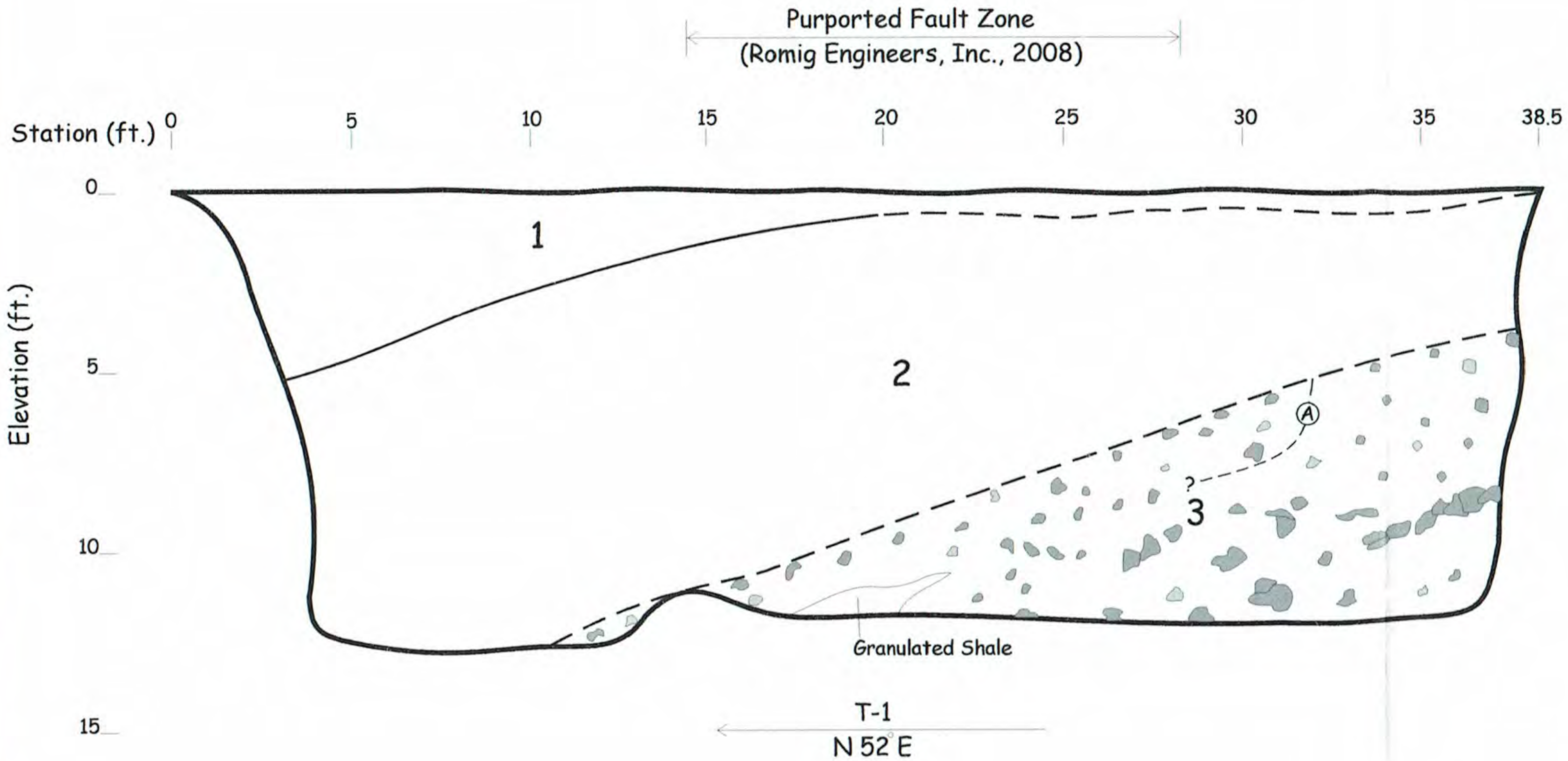
DRAFT

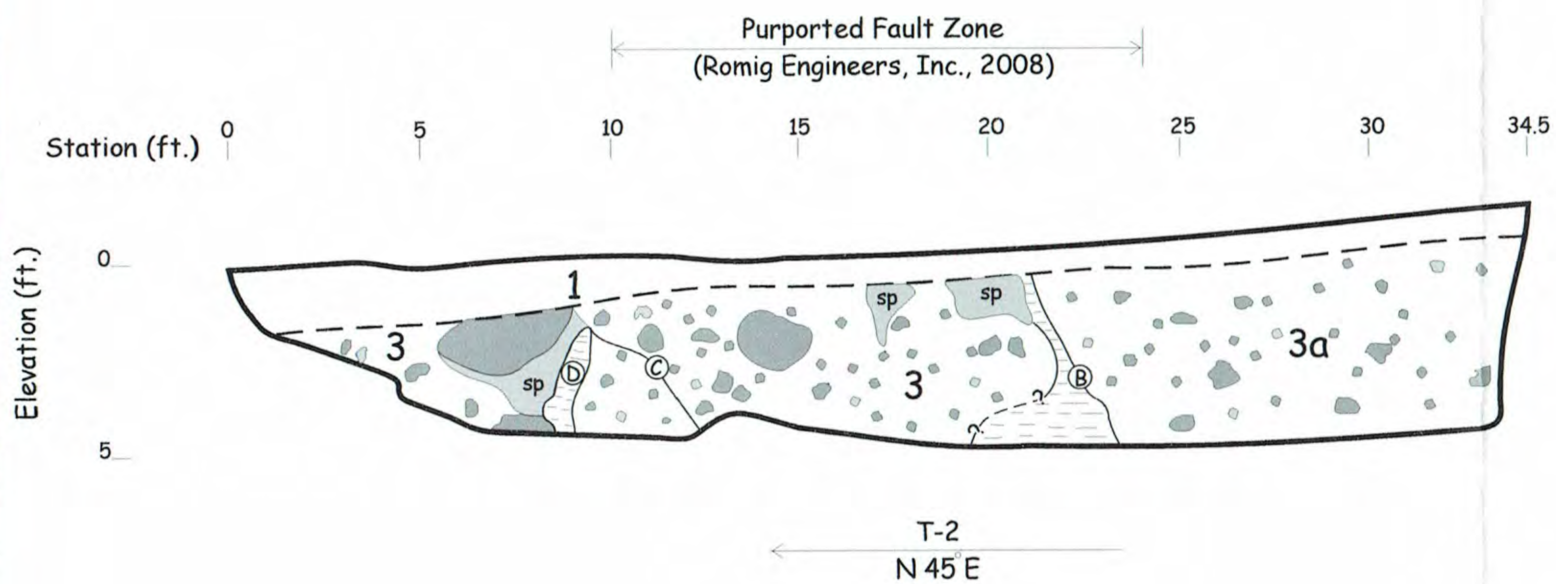
DESCRIPTIONS OF MATERIALS ENCOUNTERED IN TRENCHES
GOLDENWOOD
SAN BRUNO, CALIFORNIA

FIGURE 3
SEPTEMBER 2008
PROJECT NO. 2178-1

Earth Investigations Consultants, Job No. 2271.01.00, 10/2008

Plate A5 - Logs of Trenches





EXPLANATION

Symbol

1 — Unit

Unit contact; solid where sharp, dash where indistinct, queried where uncertain

Random of Silica-carbonate fragment to block size

sp — Random of Serpentinite fragment to block size

Gravelly Sandy CLAY

Unit

1. UNDOCUMENTED FILL: Light yellowish brown, Clayey, Silty SAND with occasional construction concrete debris to boulder size; loose; slightly moist; loose in upper foot to medium dense; trace roots in eastern part of TR-1.

2. COLLUVIUM: Dark brown in upper 1 foot to dark yellowish brown, Clayey, Gravelly, Silty SAND; angular to subrounded clasts of dark gray SILICA-CARBONATE, less discernable SERPENTINITE to 4 inches, and very pale brown, soft to moderately hard, fine-grained SANDSTONE; massive; porous; slightly moist in upper 2 feet moist to damp below, very stiff to hard (approx. age of 130,000 years; see Appendix B).

3. FRANCISCAN SHEARED ROCK: Mainly grayish and yellowish brown SERPENTINITE and dark brown SHALE(?) crushed/granulated to massive, stiff, fine Gravelly Sandy CLAY matrix supporting mainly angular to subrounded, randomly oriented, hard to soft, coarse gravel- to cobble-size fragments of dark olive brown SERPENTINITE and hard, dark olive gray, slightly vesicular SILICA-CARBONATE rock; locally contains (in TR-2) relative concentration of dark olive gray, soft to moderately hard, subangular to subrounded, SERPENTINITE boudinage(?) to 1-foot across, and angular, dense and very hard, dark olive gray, slightly vesicular, SILICA-CARBONATE blocks locally to 3 feet across.

3a FRANCISCAN SHEARED ROCK: Similar to 3 in texture and composition but lacks the larger serpentinite and silica-carbonate fragments.

Structures

T-1

A. Between Sta. 28.5 and 32 indistinct, curvilinear parting that when projected northeastward coincides with granulated shale at base of excavation between Sta. 17.5 and 21.5.

T-2

B. Approximately 1 inch thick, dark brown, stiff, unsheared Sandy CLAY containing angular to subrounded fine to medium gravel-size fragments (approx. 10%) of soft to moderately hard serpentinite and shale, and hard silica-carbonate rock that indistinctly widens on eastern side to approx. 3 feet. Bounded on west side by wavy parting oriented North 5 degrees East to North 5 degrees West.

C. Approximately 1/8-inch wide, East-West trending, stiff, unsheared CLAY seam.

D. Approximately 6-inch wide seam of dark olive green, stiff, unsheared fine to coarse, Sandy CLAY exhibiting cross-trench orientation of approximately North 25 degrees West.

Earth Investigations Consultants	Job No. 2271.01.00 Approved Date 10/13/08	LOGS OF TRENCHES 850 Glenview Drive San Bruno, California	Plate A5

APPENDIX G

Slope Stability Analysis

This appendix contains the input parameters and results of computerized slope stability analyses for three (3) critical slope segments for the Proposed Project, as well as laboratory test results from Cooper Testing Laboratory.

APPENDIX G
Slope Stability Analyses

Introduction

Static and seismic stability was characterized for three (3) critical slope segments on the eastern side of the project area (Plate G6, Slope Stability Line of Cross Sections Plan). The sections are graphically represented on the respective computer printouts (Plates G7 though G18). The stability analyses input data is based upon the project topographic site plan prepared by BKF Civil Engineers (dated August 8, 2016), and the relatively large volume of subsurface soil profile data compiled over the past decade by Earth Investigations Consultants (2006, 2008, 2013, 2016), and recently by Geosphere (2019). The following sections of this appendix discuss material strength parameters, seismic coefficient, ground water conditions, model description, and slope stability analyses results.

Analysis

Material Strength Parameters

Stability-model material strength parameters for subsurface materials encountered at the project site were derived from laboratory testing of samples collected from the 2019 Geosphere exploratory borings (Appendix E), published correlations of friction angle with laboratory testing data and effective normal stress (Stark, et al., 2005), and our experience with similar materials.

Sieve analysis and Atterberg Limits testing on samples at depths of 4.5, 9.5, and 24.5 feet in Boring 3 indicated measured clay contents of 14.4%, 13.1%, and 6.6%, and Plasticity Indices (PI) of 18, 14, and 14, respectively (Appendix E; Appendix G).

The shear strength parameters for the colluvium and bedrock were measured and interpreted from staged consolidated undrained triaxial (TXCU) tests performed on relatively undisturbed samples retrieved from depths of 14 and 21½ feet, respectively. Since no existing or past shear plane or failure surface was observed in the materials during our field exploration, we consider the measured strengths to be appropriate for the analysis without applying any reduction. The resulting strength parameters used in our analysis, as derived from our interpretation of the aforementioned data, are presented in Table 1.

Table 1 – Material Strength Parameters

		Strength Parameters	
Layer	**Unit Wt. (pcf)**	**Cohesion (psf)**	**Friction Angle (Deg.)**
New Fill	120	350	25
Old Fill	115 (Sat.)	50	28
Colluvium	126 (Sat.)	0	35
Bedrock	139	550	37

Seismic Coefficient (K_{EQ})

Based on the screening procedure developed by Bray and others (1998), we selected a seismic coefficient of 0.32g with an assumed allowable seismic displacement of 15 cm (6 inches) and a Mean Magnitude of 7.9 based on a return period of 10% in 50 years. According to California Geological Survey Special Report 117A (CGS, 2008), a Newmark displacement of less than 6 inches is unlikely to correspond to serious landslide movement and damage, which we consider to be appropriate for application to the subject site. For the stability analyses under seismic conditions, a minimum factor-of-safety for 1.1 is commonly required.

Model Ground Water

Ground water was absent in the subsurface explorations that span winter and summer seasons. However, we assigned a saturated unit weight to both Old Fill and Colluvium layers to model the presence of potential seasonal perched ground water within the surficial mantling the Franciscan mélange.

Model Description

To assess potential failure mechanisms of the slope at the project site, stability analyses were performed using *Slide 2018* Software developed by Rocscience, which computes the stability of hypothetical slip surfaces using two-dimensional, limit equilibrium methods. The program can be used to search for most critical surfaces, or the factor-of-safety (FoS) may be calculated for a specified surface. A FoS value of >1.0 generally indicates stability under the conditions used in the analysis. A value of 1.0 or less indicates that a slope is in a state of equilibrium or may fail. The Morgenstern-Price analysis method, which satisfies both force and moment-equilibrium, was used for stability evaluation.

The analyses include evaluation of the slope materials under both static and pseudo-static (earthquake shaking) conditions for the existing condition and the proposed new residential development. Data input for the analyses included model strength parameters for the existing fill and bedrock as provided in Table 1.

Our model for proposed development assumes that all artificial fill (af) and colluvium (Qc) layers will be reworked as engineered fill as presented in Appendices A and B.

<u>Slope Stability Analyses Results</u>

The results of the stability analyses are summarized on the following Tables 2 and 3.

Table 2 – Existing Slope Conditions		
Section	**Min Static FS**	**Min Pseudo-Static FoS**
X-X’	1.4	0.7
Y-Y’	2.4	1.0
Z-Z’	3.9	1.3

<table>
<tr><th colspan="3">Table 3 –Slope Conditions after Proposed Grading</th></tr>
<tr><th>Section</th><th>Min Static FS</th><th>Min Pseudo-Static FoS</th></tr>
<tr><td rowspan="2">X-X’</td><td>> 1.5 (Inside Development)</td><td>> 1.1 (Inside Development)</td></tr>
<tr><td>1.4
(Outside and Downslope of Development)</td><td>0.7
(Outside and Downslope of Development)</td></tr>
<tr><td>Y-Y’</td><td>2.2</td><td>1.2</td></tr>
<tr><td>Z-Z’</td><td>3.8</td><td>1.5</td></tr>
</table>

Stability Analysis Section X-X’

Section X-X transects the existing residence and the proposed bio-retention basin in the northeast corner where boring data indicates there is up to 11 feet of unconsolidated soil, consisting of 7 to 8 feet of artificial fill and up to 3 feet of colluvium resting on sheared rock (mélange; Plate G6; Appendix E). The grading plan suggests the finished grade will be approximately 4 feet higher than the existing condition on the northeastern part of the property. Reconnaissance observations revealed the existing retaining walls and the house foundation have been fortified by 16- to 18-inch diameter piers apparently to mitigate soil creep affecting the top of the very densely vegetated steep descending slope on the east side of the graded pad (Reply to Peer Review, Fig. 1). Documentation for the design and as-built construction of the fortifications were unavailable for our site characterization.

Our analyses indicate that under existing conditions in the absence of seismic shaking, the subsurface materials are generally stable (Plates G7, G8). All hypothetical failure surfaces intersecting the surface within the property exceeded a FoS of 1.5 (see Plate G6; in our graphical representation of all plates, only potentially slope failures under acceptable FoS are shown.) However, potential off-site failure surfaces far downslope had calculated FoS of 1.4, which is less than the FoS of 1.5 typically accepted for static conditions. Under pseudo-static conditions, a FoS less than 1 was calculated using the model strength parameters (Table 1) and a seismic coefficient of 0.32g. Table 2 and Plate G8 reveal potential failure-surface encroachment into the property with less than the generally-accepted minimum FoS of 1.1 for stable pseudo-static conditions. The least stable potential surficial failure surface, FoS of 0.72, was detected off-site, well downslope of the property. These findings results are consistent with mapped shallow landsliding from the San Bruno Creek drainage basin.

Based on our analysis, proposed grading within the actual limits of the development has acceptable static and pseudo-static FoS (Plates G13, G14), as all potential failure surfaces had computed FoS exceeding 1.5 and 1.1, respectively. However, as previously indicated for existing conditions and shown on Plates G13 and G14, minimum FoS for static and seismic conditions downslope of the development remain at 1.4 and 0.7, respectively. Since these conditions present potential for encroachment of shallow landsliding in the eastern margin of the development, in Appendix B we recommend seismic slope stability be enhanced by installing a stitch pier system on the property line in the northeastern part of the property (Plate G6). Provided our remedial grading and drainage recommendations in Appendices A and B are followed, we anticipate a low risk for adverse impact to the eastern side of the project area by surficial soil creep.

Stability Analysis Section Y-Y'

Section Y-Y' transects the middle-east margin of the development area at the headward part of a swale encountered in EIC exploratory trench T-1 (Appendix F), where there is up to 22 feet of unconsolidated soil comprised of 16 feet of artificial fill and 6 feet of colluvium resting on bedrock (mélange; Plate G6; Appendix E). The grading plan suggests the finished grade will be approximately 6 feet higher than existing conditions on the eastern perimeter.

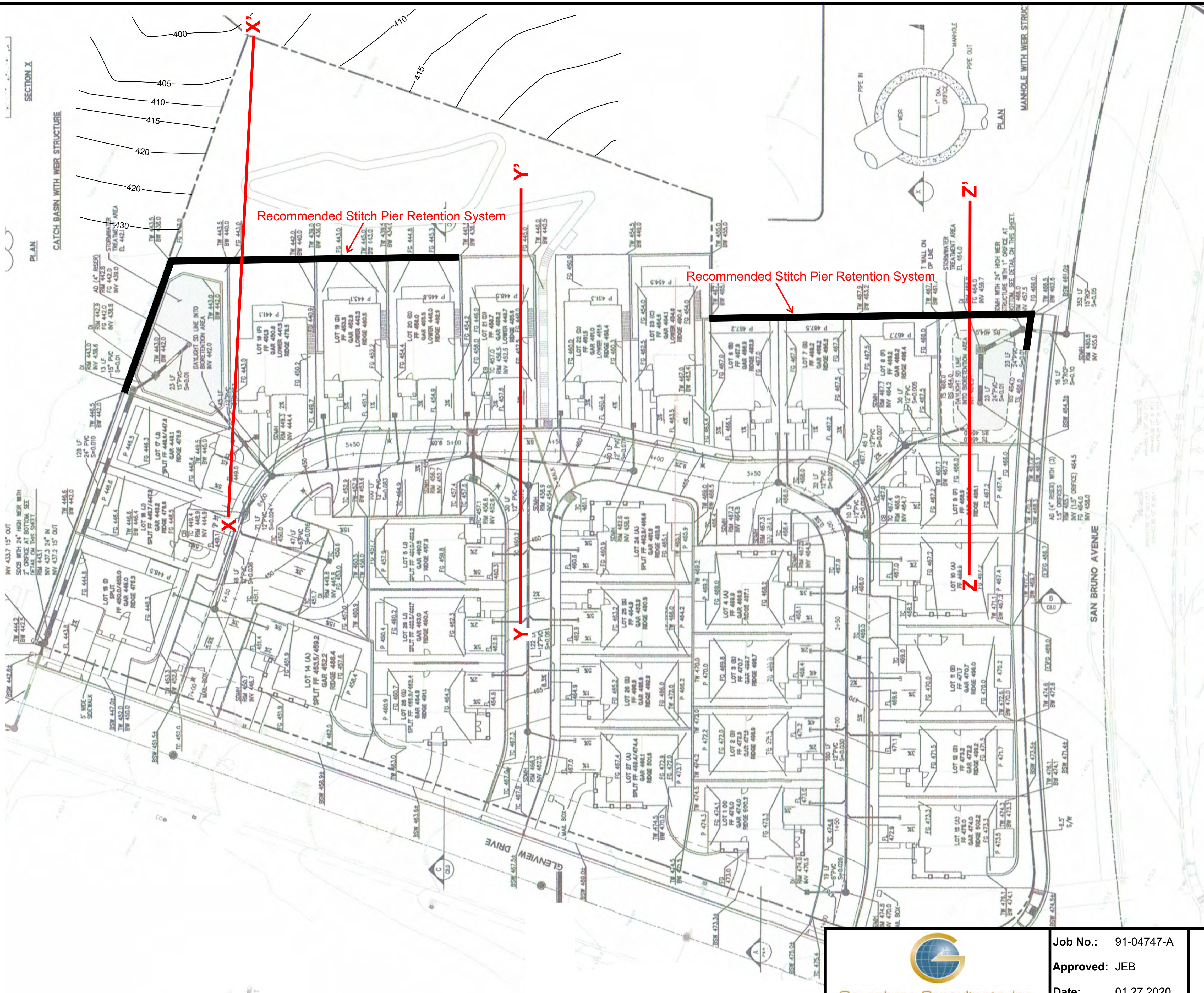
Our analysis indicates that subsurface materials are generally stable under the current condition, under pseudo-static load (Plates G9, G10). Our analysis also indicates that the proposed slope within the development has acceptable static and pseudo-static FoS when constructed in accordance with our remedial grading recommendations (Plates G15, G16). As stated above, grading and drainage recommendations for this project are intended to mitigate surficial soil creep on the eastern side of property.

Stability Analysis Section Z-Z'

Section Z-Z' transects the southeastern corner of the project area where the second bio-retention basin is sited. Boring data from this area indicates there is up to 6½ feet of artificial fill on mélange (Appendix E). The grading plan suggests the finished grade will be approximately 6 feet higher than existing condition on the eastern side. Our analysis indicates that subsurface materials are generally stable under current conditions and under pseudo-static load (Plates G11, G12). They also indicate that proposed slope has acceptable static and pseudo-static FoS when constructed in accordance with our remedial grading and drainage recommendations (Plates G17, G18; Appendices A and B). However, due to potential encroachment from retrogression of surficial landsliding on the slope below the proposed bio-retention basin in the southeast corner of the site, we have recommend seismic slope stability be enhanced from installation of a stitch pier retention system as depicted on Plate G6. Provided our remedial grading and drainage recommendations in Appendices A and B are followed, we anticipate a low risk for adverse impact to the eastern side of the project area by surficial soil creep.

Conclusions

Based on the results of our analyses, we judge that the proposed development is feasible, with an acceptable FoS for slope stability under both static and seismic loading conditions provided that our recommendations are followed. We judge the northeastern and southeastern portions of the property, typically where a slope of steeper than approximately 3:1 H:V exists, would benefit from a stitch pier retention system to mitigate potential encroachment into the proposed bio-retention basin locations by perceived off-site surficial slope instability (Plate G6). Geotechnical parameters for stitch pier design are presented in Appendix B.



EXPLANATION

X———X' Line of Slope Stability Section X-X'

50 ft.
Scale

Source: BKF Engineers
Tentative Tract Map TM13-001
Sheets C5.0 and C5.1
Dated 08.08.2016

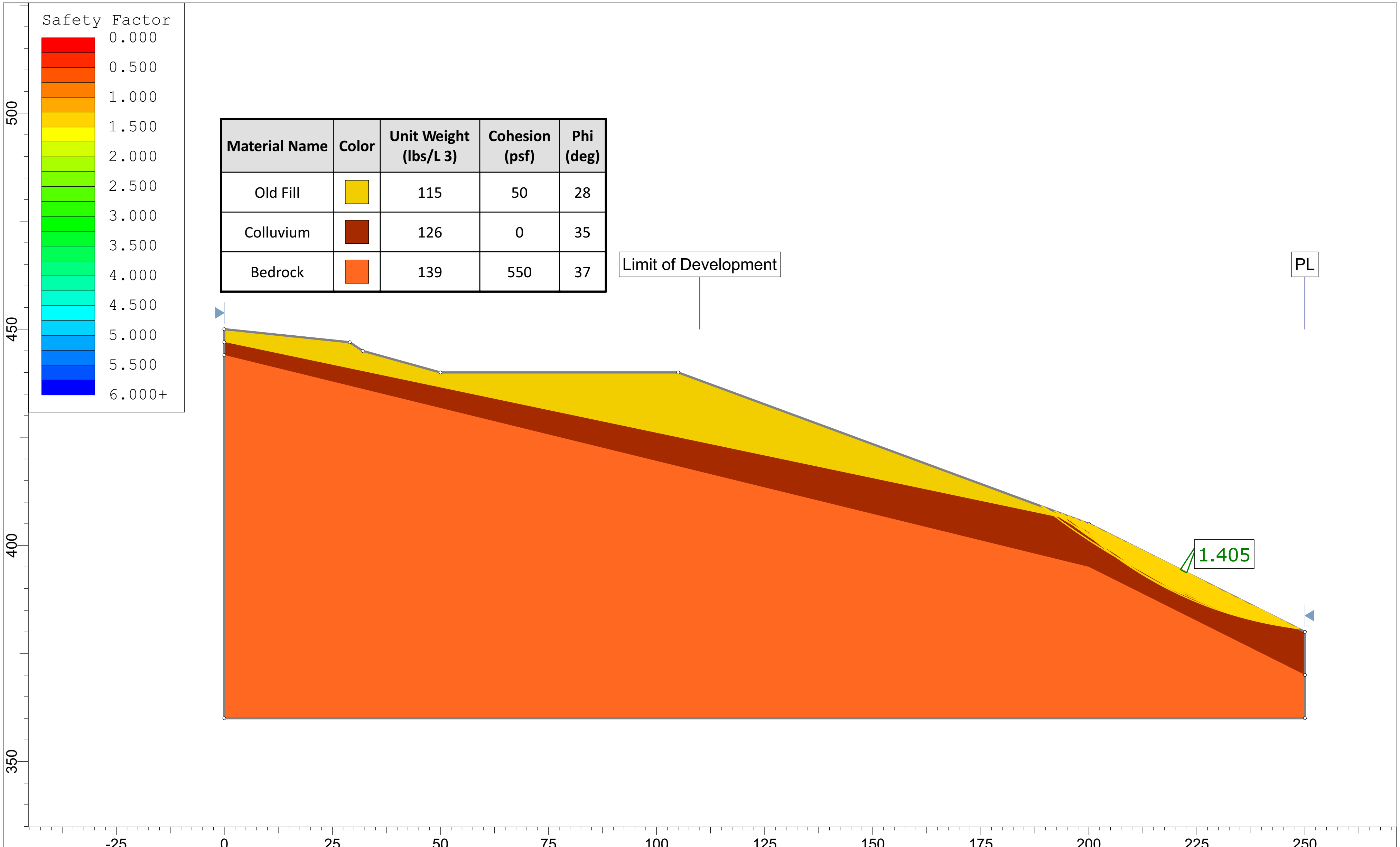
Geosphere Consultants, Inc.	Job No.: 91-04747-A Approved: JEB Date: 01.27.2020	**SLOPE STABILITY LINE OF CROSS SECTIONS PLAN** Glenview Terrace Residential Development 2880 San Bruno Avenue San Bruno, California	**Plate** G6

SLOPE STABILITY CROSS SECTIONS - EXISTING

Cross Section X-X’
Plate G7 - Static
Plate G8 - Seismic

Cross Section Y-Y’
Plate G9 - Static
Plate G10 - Seismic

Cross Section Z-Z’
Plate G11 - Static
Plate G12 - Seismic



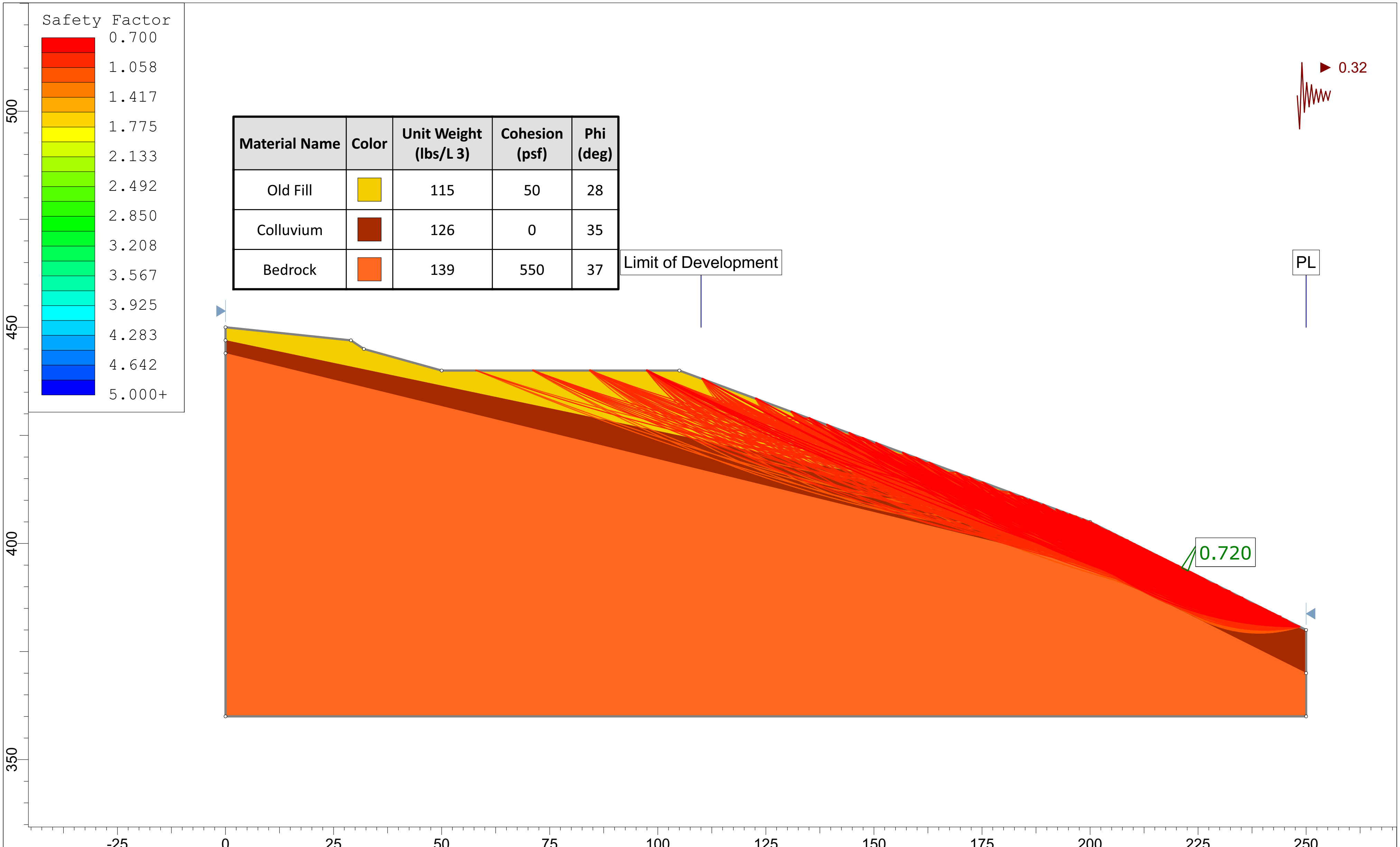
Material Name	Color	Unit Weight (lbs/L 3)	Cohesion (psf)	Phi (deg)
Old Fill		115	50	28
Colluvium		126	0	35
Bedrock		139	550	37

rocscience

SLIDEINTERPRET 8.030

Project	Glenview Terrace		
Analysis Description	**Existing Slope - Static - X-X'**		
Drawn By	AL	*Scale*	1:369
Company	Geosphere Consultants, Inc.		
Date	1/20/2020, 10:25:59 AM	*File Name*	Existing.slmd

Plate G7



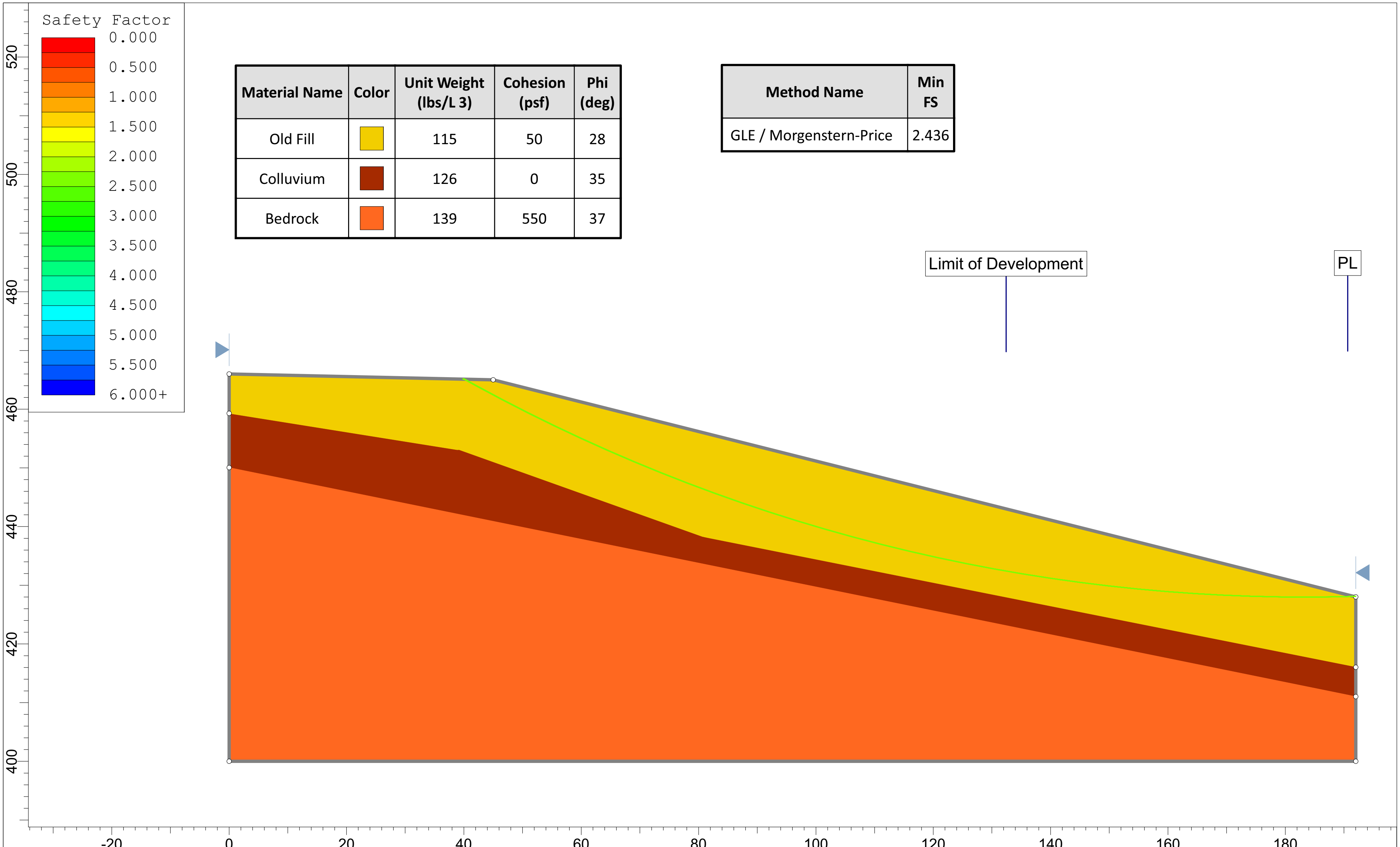
Material Name	Color	Unit Weight (lbs/L 3)	Cohesion (psf)	Phi (deg)
Old Fill		115	50	28
Colluvium		126	0	35
Bedrock		139	550	37

rocscience

SLIDEINTERPRET 8.030

Project	Glenview Terrace
Analysis Description	**Existing Slope - Seismic - X-X'**
Drawn By	AL
Scale	1:369
Company	Geosphere Consultants, Inc.
Date	1/20/2020, 1:25:24 PM
File Name	Existing.slmd

Plate G8



Material Name	Color	Unit Weight (lbs/L 3)	Cohesion (psf)	Phi (deg)
Old Fill		115	50	28
Colluvium		126	0	35
Bedrock		139	550	37

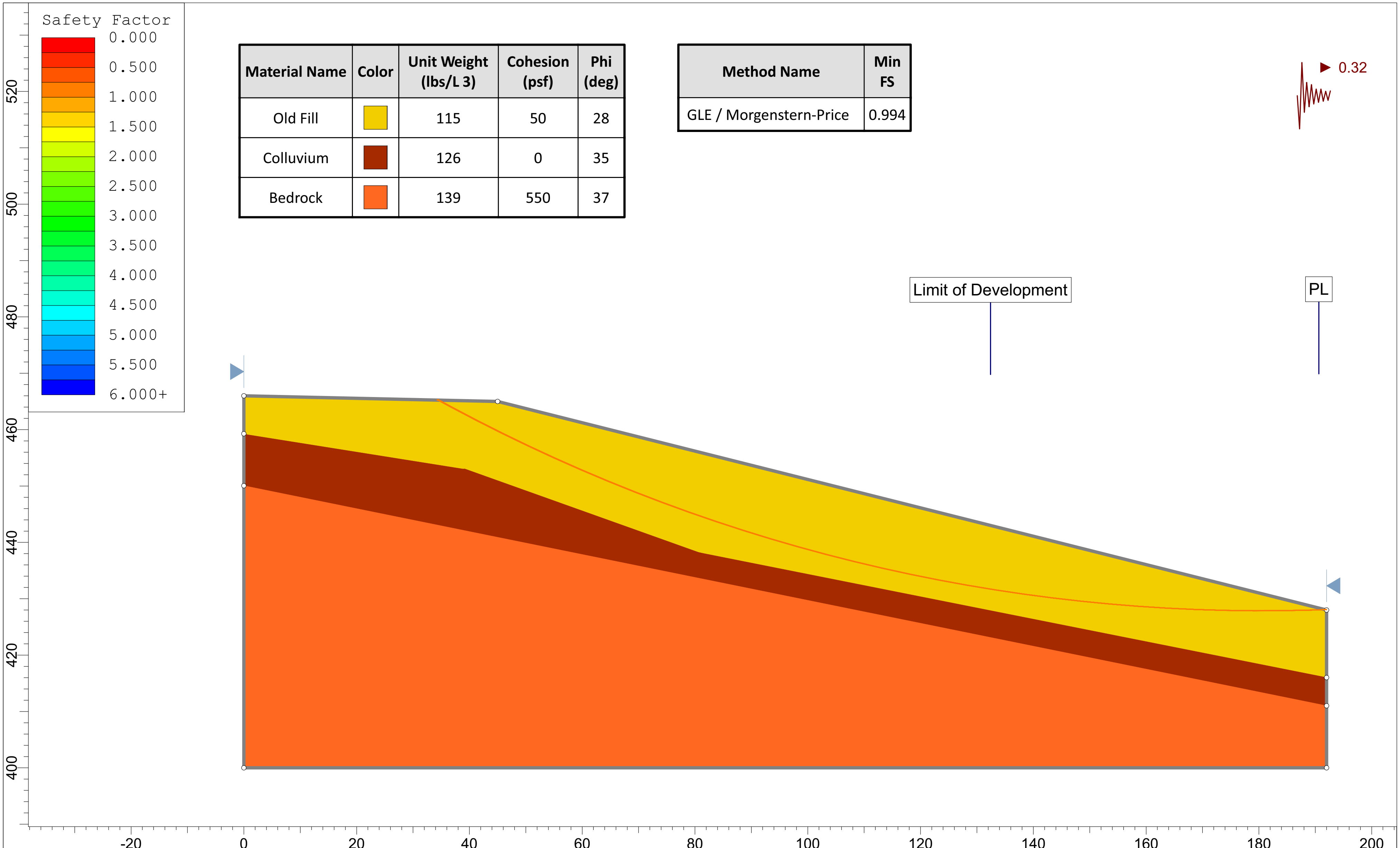
Method Name	Min FS
GLE / Morgenstern-Price	2.436

rocscience

SLIDEINTERPRET 8.030

Project	Glenview Terrace		
Analysis Description	**Existing Slope - Static - Y-Y'**		
Drawn By AL	*Scale* 1:271	*Company*	Geosphere Consultants, Inc.
Date	1/20/2020, 12:23:58 PM	*File Name*	Existing.slmd

Plate
G9



Material Name	Color	Unit Weight (lbs/L 3)	Cohesion (psf)	Phi (deg)
Old Fill		115	50	28
Colluvium		126	0	35
Bedrock		139	550	37

Method Name	Min FS
GLE / Morgenstern-Price	0.994

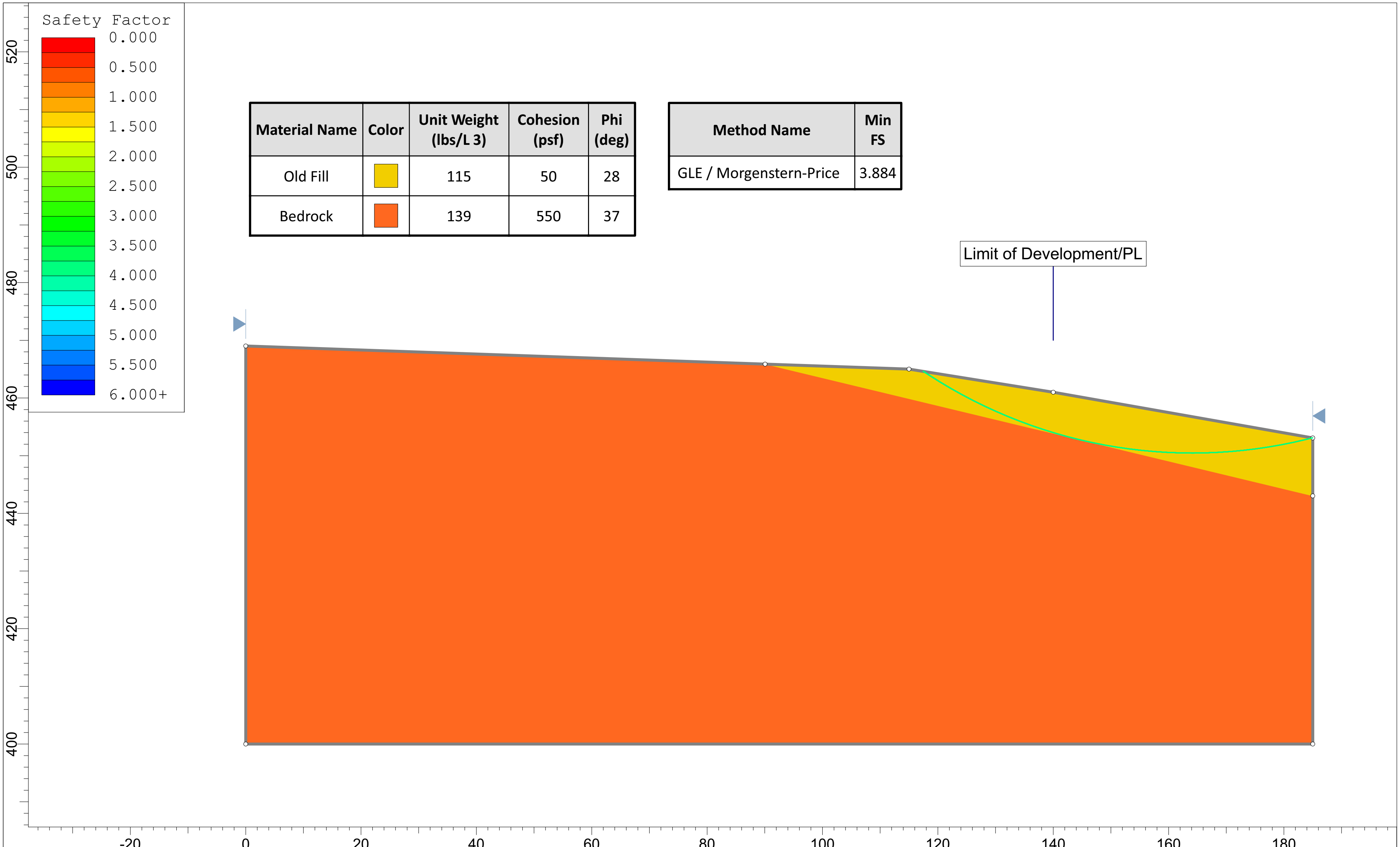
rocscience

SLIDEINTERPRET 8.030

Project	Glenview Terrace
Analysis Description	**Existing Slope - Seismic - Y-Y'**
Drawn By	AL
Scale	1:282
Company	Geosphere Consultants, Inc.
Date	1/20/2020, 1:25:54 PM
File Name	Existing.slmd

Plate

G10



Material Name	Color	Unit Weight (lbs/L 3)	Cohesion (psf)	Phi (deg)
Old Fill		115	50	28
Bedrock		139	550	37

Method Name	Min FS
GLE / Morgenstern-Price	3.884

rocscience

SLIDEINTERPRET 8.030

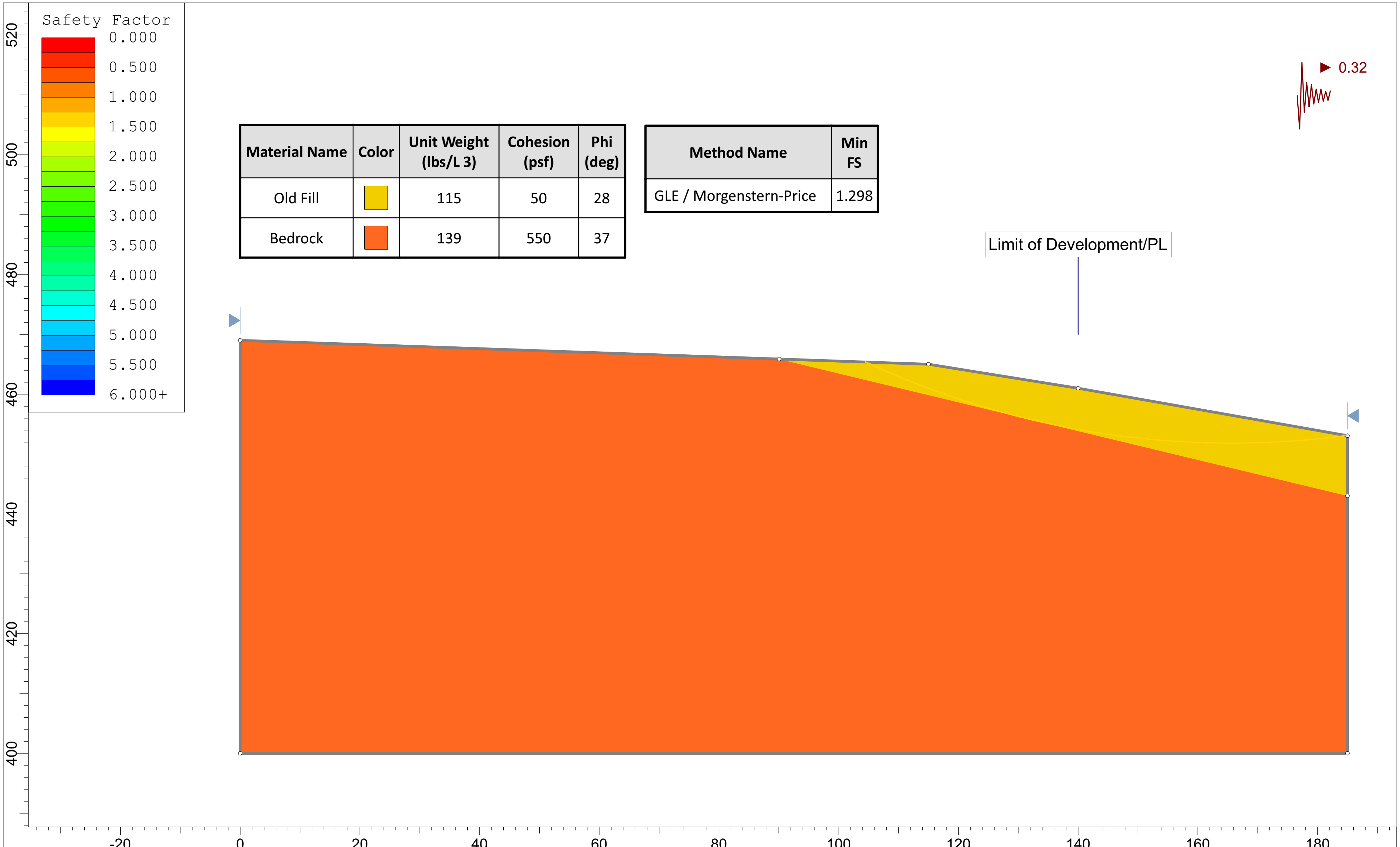
Project Glenview Terrace

Analysis Description **Existing Slope - Static - Z-Z'**

Drawn By AL | *Scale* 1:276 | *Company* Geosphere Consultants, Inc.

Date 2/9/2020, 12:41:33 PM | *File Name* Existing.slmd

Plate
G11



Material Name	Color	Unit Weight (lbs/L 3)	Cohesion (psf)	Phi (deg)
Old Fill		115	50	28
Bedrock		139	550	37

Method Name	Min FS
GLE / Morgenstern-Price	1.298

rocscience	*Project* Glenview Terrace		
	Analysis Description **Existing Slope - Static - Z-Z'**		
	Drawn By AL	*Scale* 1:266	*Company* Geosphere Consultants, Inc.
SLIDEINTERPRET 8.030	*Date* 2/9/2020, 12:41:33 PM		*File Name* Existing.slmd

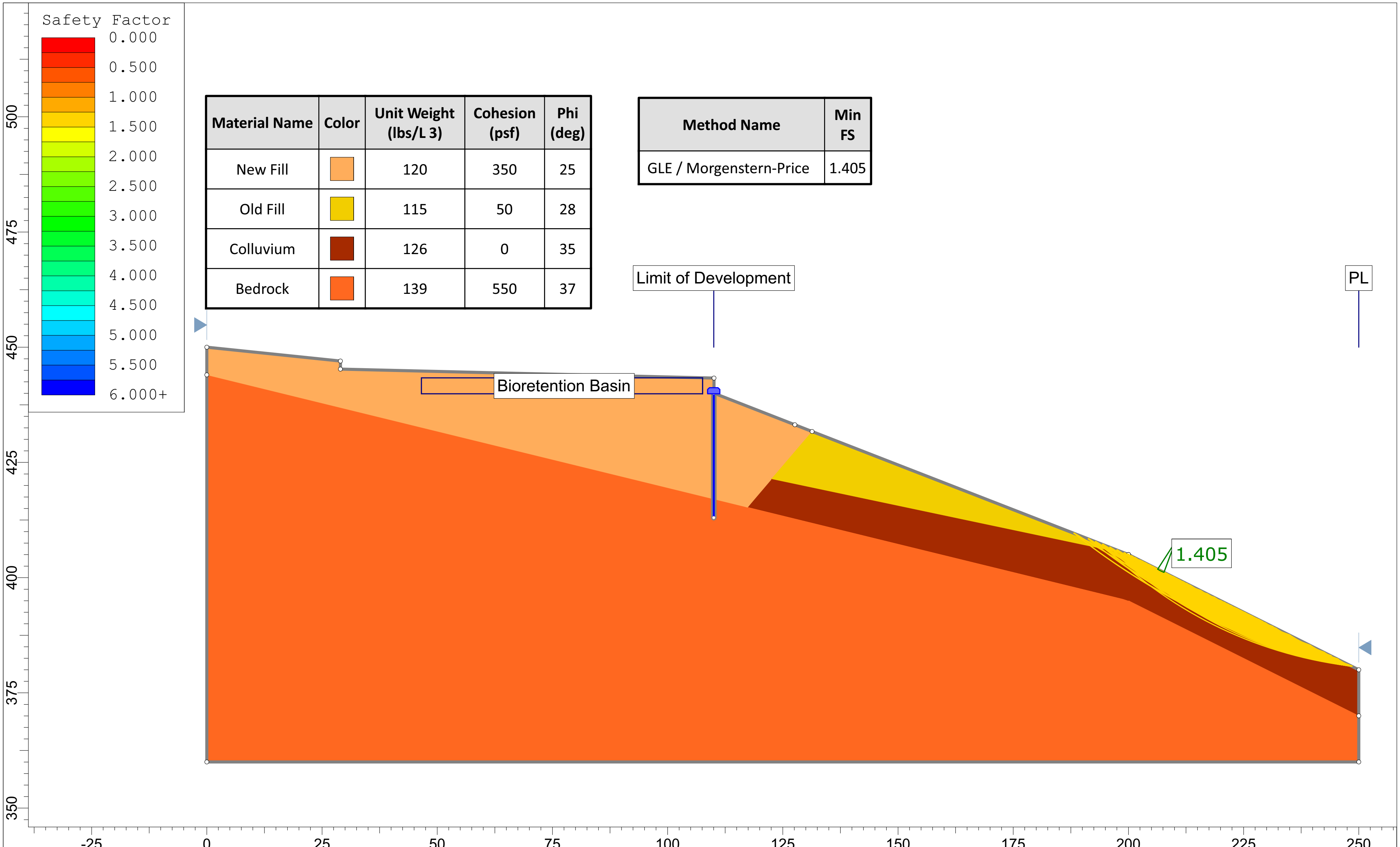
Plate G12

SLOPE STABILITY CROSS SECTIONS - PROPOSED

Cross Section X-X’
- Plate G13 - Static
- Plate G14 - Seismic

Cross Section Y-Y’
- Plate G15 - Static
- Plate G16 - Seismic

Cross Section Z-Z’
- Plate G17 - Static
- Plate G18 - Seismic



Material Name	Color	Unit Weight (lbs/L 3)	Cohesion (psf)	Phi (deg)
New Fill		120	350	25
Old Fill		115	50	28
Colluvium		126	0	35
Bedrock		139	550	37

Method Name	Min FS
GLE / Morgenstern-Price	1.405

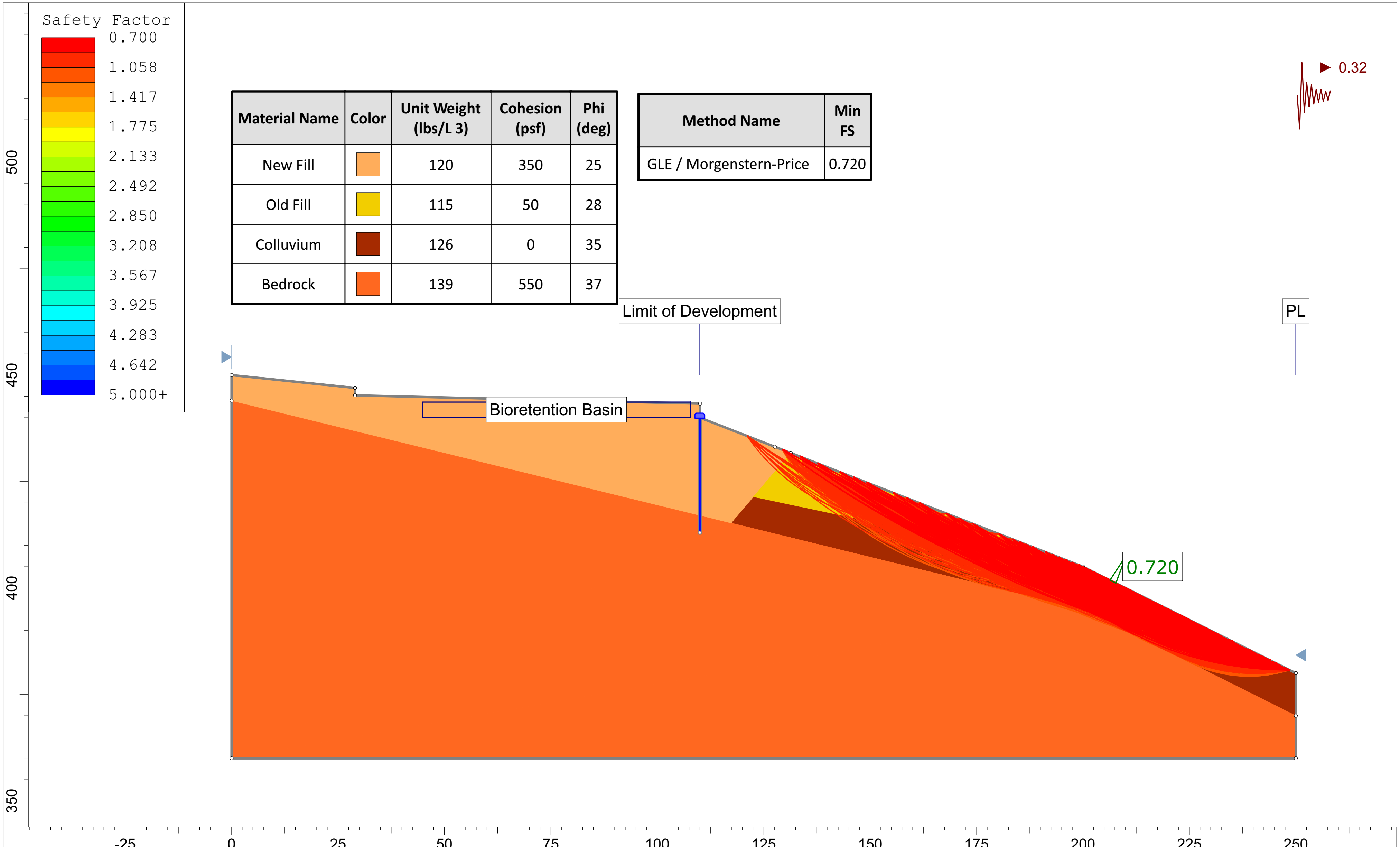
rocscience

SLIDEINTERPRET 8.030

Project	Glenview Terrace
Analysis Description	**Proposed Slope - Static - X-X'**
Drawn By	AL
Scale	1:346
Company	Geosphere Consultants, Inc.
Date	1/20/2020, 10:25:59 AM
File Name	Proposed.slmd

Plate

G13

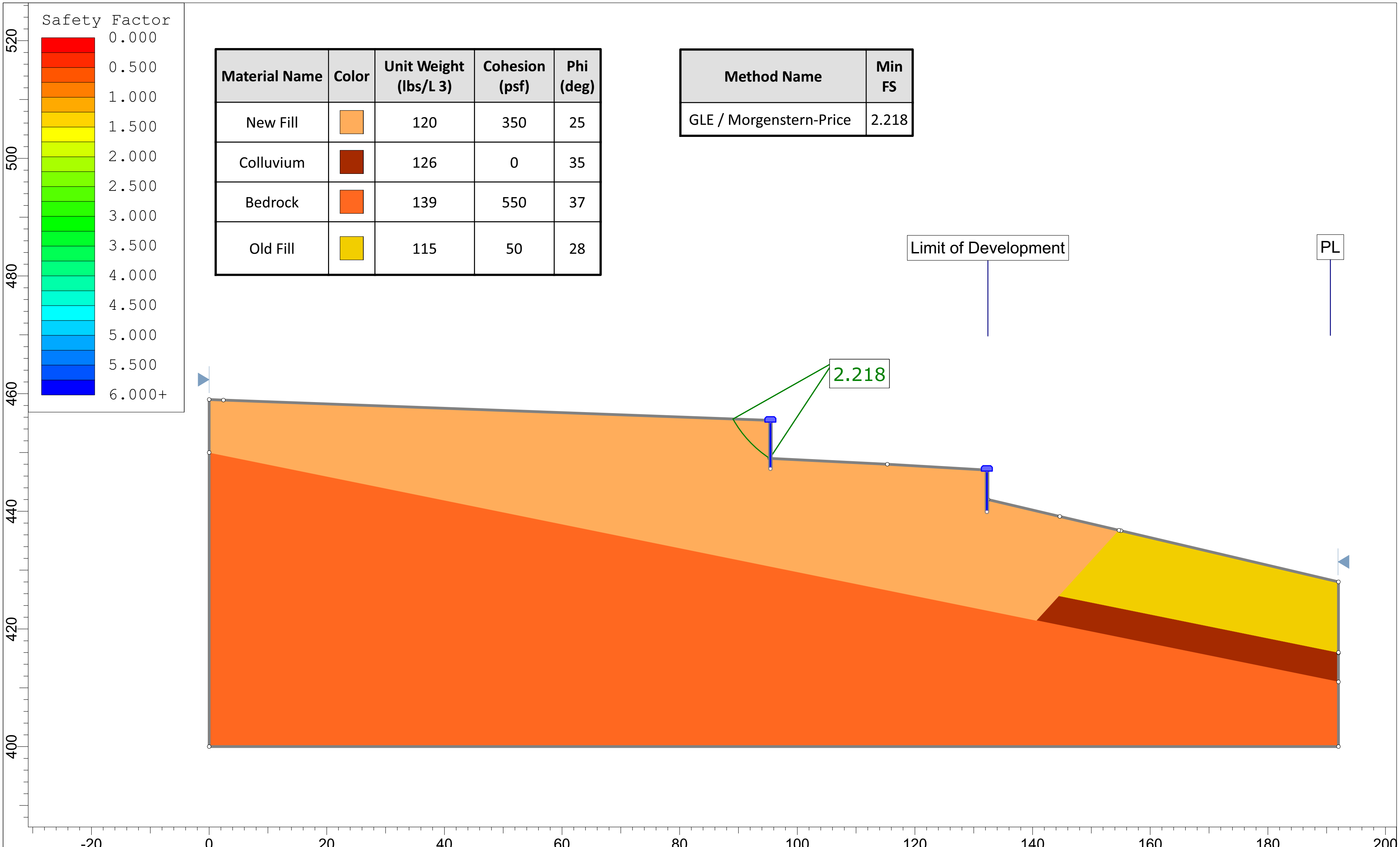


Material Name	Color	Unit Weight (lbs/L 3)	Cohesion (psf)	Phi (deg)
New Fill		120	350	25
Old Fill		115	50	28
Colluvium		126	0	35
Bedrock		139	550	37

Method Name	Min FS
GLE / Morgenstern-Price	0.720

rocscience	*Project*	Glenview Terrace	
	Analysis Description	**Proposed Slope - Seismic - X-X'**	
	Drawn By AL	*Scale* 1:374	*Company* Geosphere Consultants, Inc.
SLIDEINTERPRET 8.030	*Date* 1/20/2020, 10:25:59 AM		*File Name* Proposed.slmd

Plate G14



Material Name	Color	Unit Weight (lbs/L 3)	Cohesion (psf)	Phi (deg)
New Fill		120	350	25
Colluvium		126	0	35
Bedrock		139	550	37
Old Fill		115	50	28

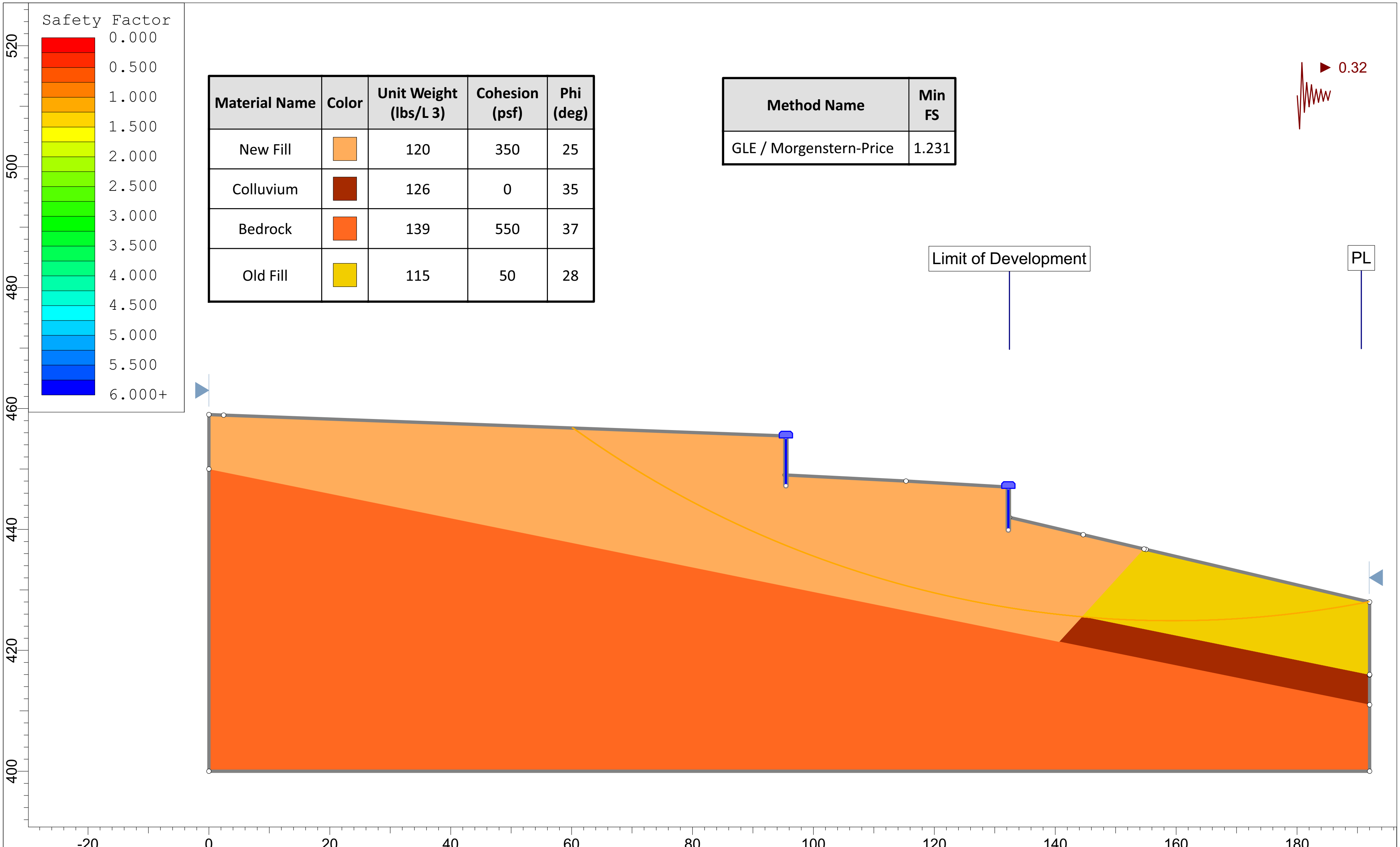
Method Name	Min FS
GLE / Morgenstern-Price	2.218

rocscience

SLIDEINTERPRET 8.030

Project	Glenview Terrace
Analysis Description	**Proposed Slope - Static - Y-Y'**
Drawn By	AL
Scale	1:271
Company	Geosphere Consultants, Inc.
Date	1/20/2020, 12:23:58 PM
File Name	Proposed.slmd

Plate
G15

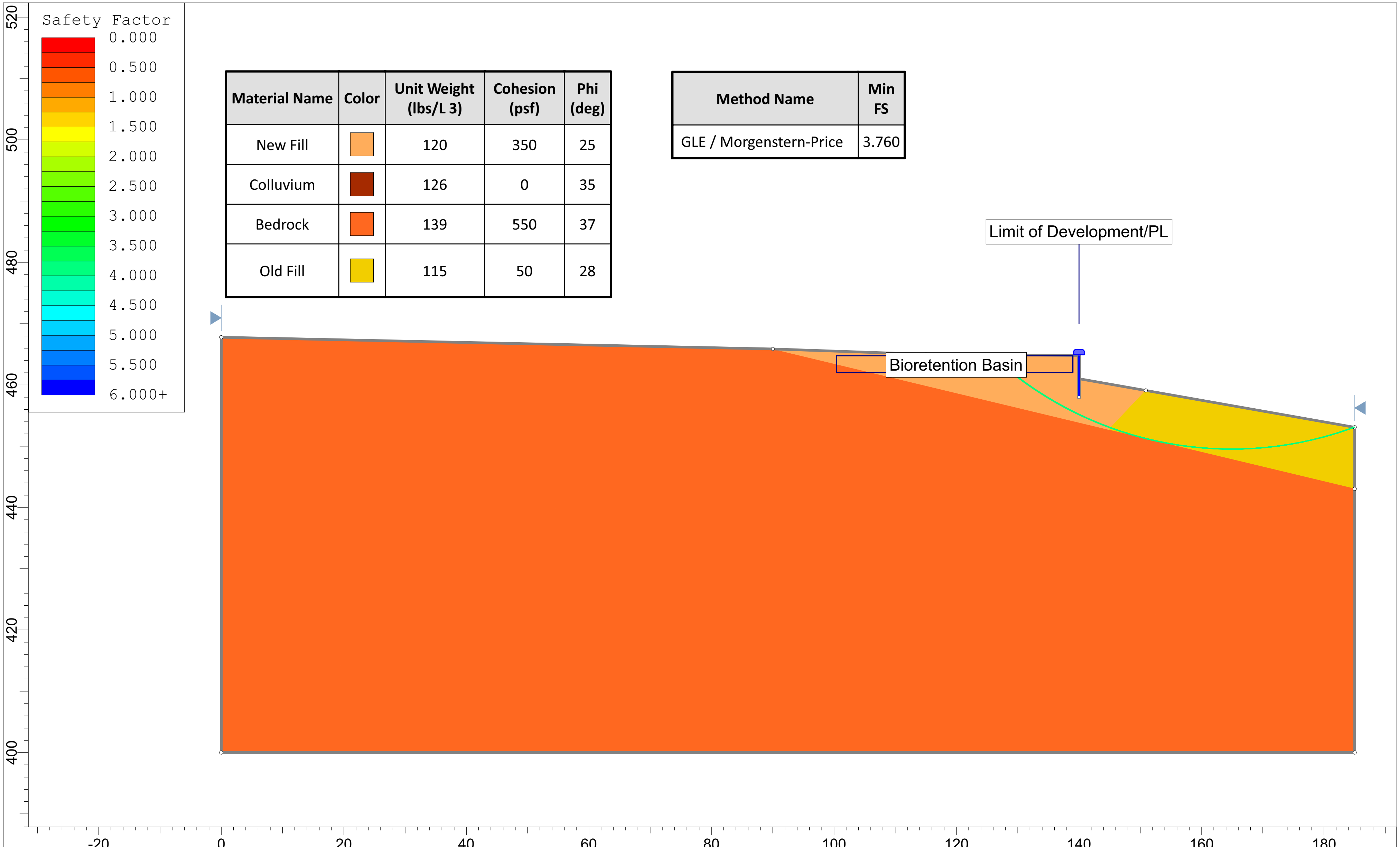


Material Name	Color	Unit Weight (lbs/L 3)	Cohesion (psf)	Phi (deg)
New Fill		120	350	25
Colluvium		126	0	35
Bedrock		139	550	37
Old Fill		115	50	28

Method Name	Min FS
GLE / Morgenstern-Price	1.231

rocscience	Project	Glenview Terrace	
	Analysis Description	**Proposed Slope - Seismic - Y-Y'**	
	Drawn By: AL	Scale: 1:263	Company: Geosphere Consultants, Inc.
SLIDEINTERPRET 8.030	Date: 1/20/2020, 12:23:58 PM		File Name: Proposed.slmd

Plate G16



Material Name	Color	Unit Weight (lbs/L 3)	Cohesion (psf)	Phi (deg)
New Fill		120	350	25
Colluvium		126	0	35
Bedrock		139	550	37
Old Fill		115	50	28

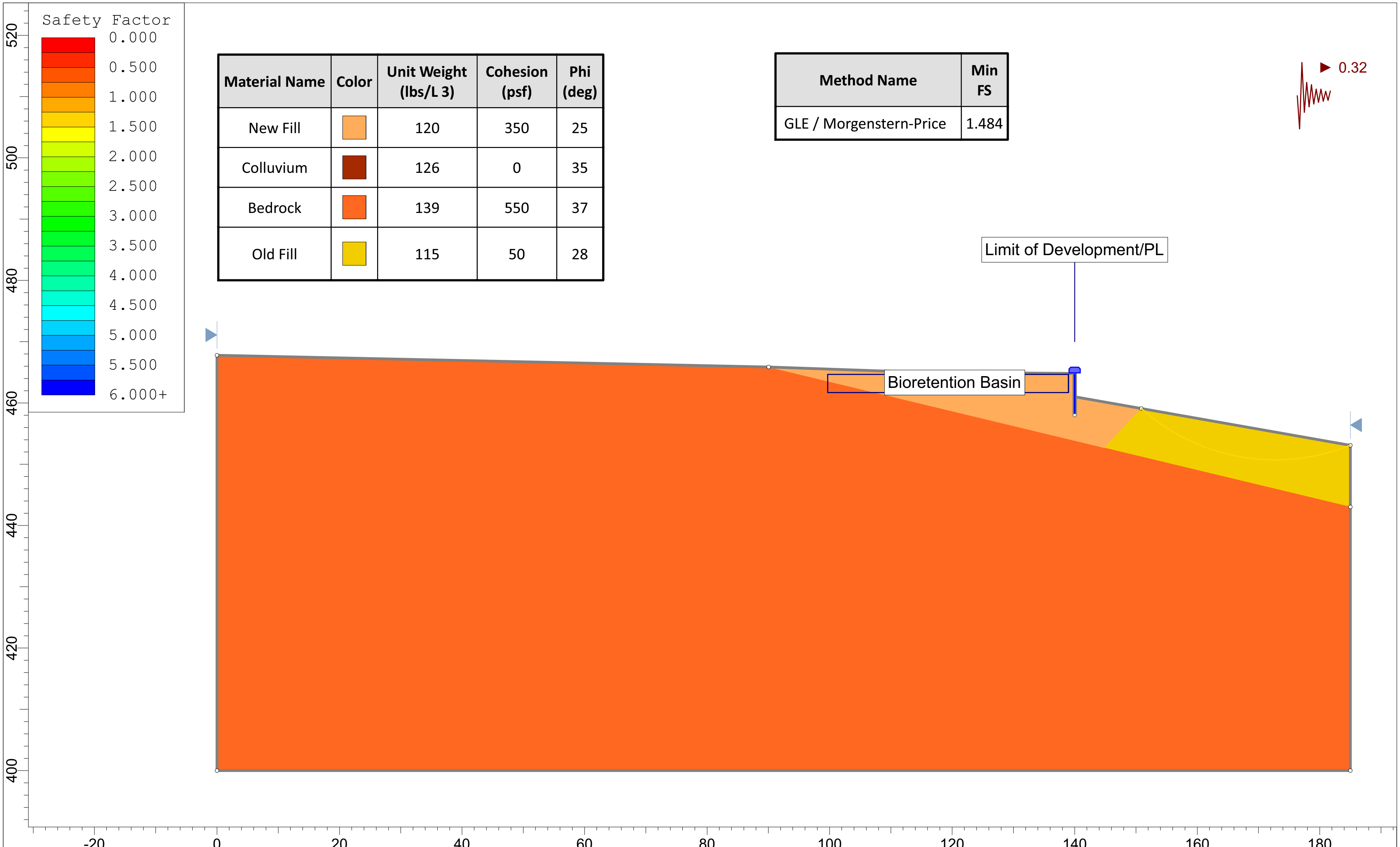
Method Name	Min FS
GLE / Morgenstern-Price	3.760

rocscience

SLIDEINTERPRET 8.030

Project	Glenview Terrace
Analysis Description	**Proposed Slope - Static - Z-Z'**
Drawn By	AL
Scale	1:260
Company	Geosphere Consultants, Inc.
Date	2/9/2020, 12:41:33 PM
File Name	Proposed.slmd

Plate G17



Material Name	Color	Unit Weight (lbs/L 3)	Cohesion (psf)	Phi (deg)
New Fill		120	350	25
Colluvium		126	0	35
Bedrock		139	550	37
Old Fill		115	50	28

Method Name	Min FS
GLE / Morgenstern-Price	1.484

rocscience

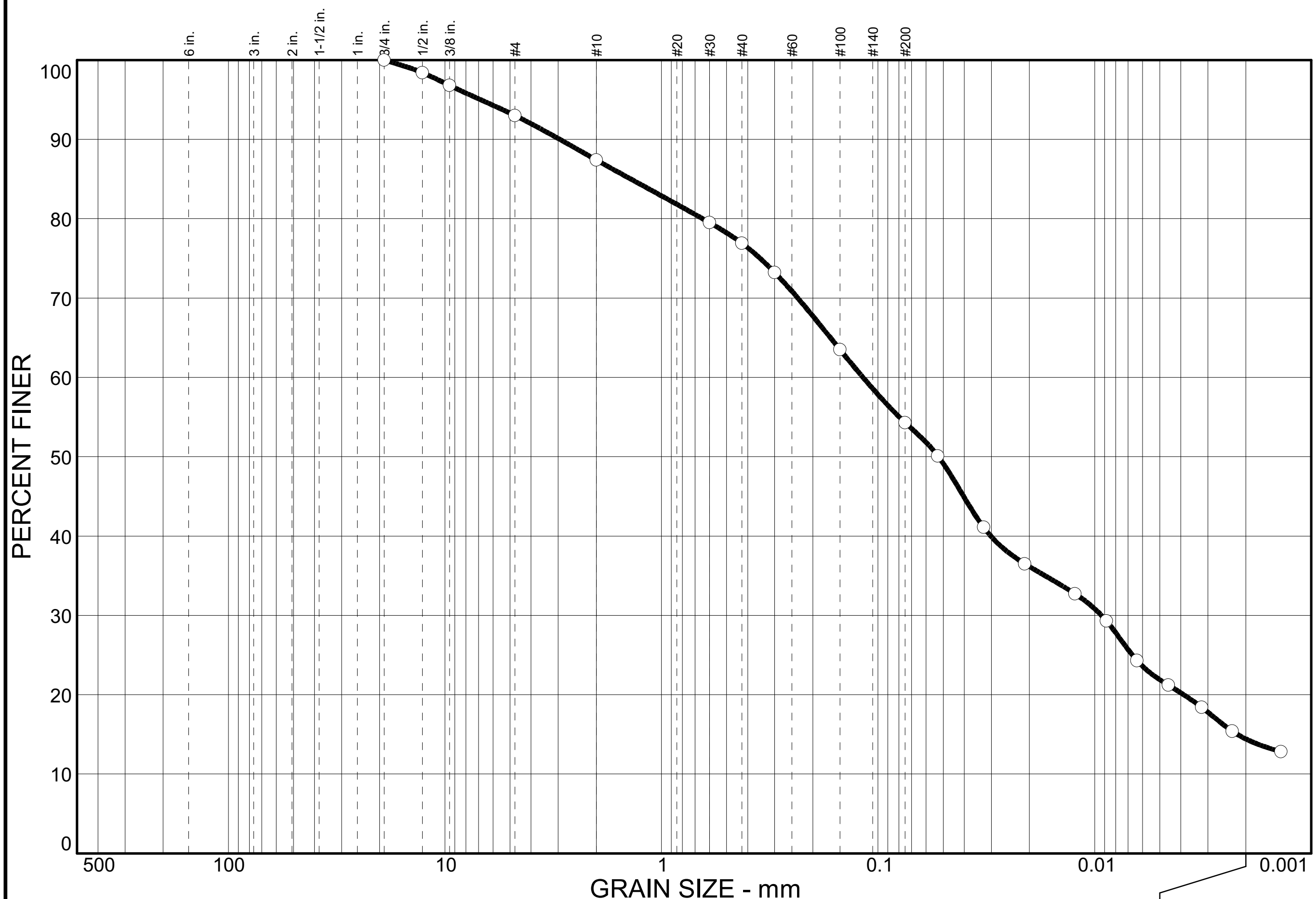
SLIDEINTERPRET 8.030

Project	Glenview Terrace
Analysis Description	**Proposed Slope - Seismic - Z-Z'**
Drawn By	AL
Scale	1:260
Company	Geosphere Consultants, Inc.
Date	2/9/2020, 12:41:33 PM
File Name	Proposed.slmd

Plate G18

Cooper Testing Laboratory

Particle Size Distribution Report



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	7.0	38.7	39.9	14.4

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3/4 in.	100.0		
1/2 in.	98.4		
3/8 in.	96.8		
#4	93.0		
#10	87.4		
#30	79.5		
#40	76.9		
#50	73.2		
#100	63.5		
#200	54.3		
#270	50.1		
0.0326 mm.	41.1		
0.0210 mm.	36.5		
0.0123 mm.	32.7		
0.0088 mm.	29.3		
0.0064 mm.	24.3		
0.0046 mm.	21.2		
0.0032 mm.	18.4		
0.0023 mm.	15.4		
0.0014 mm.	12.8		

* (no specification provided)

Soil Description

Dark Brown Sandy Elastic SILT

Atterberg Limits

PL= 35 LL= 53 PI= 18

Coefficients

D_{85}= 1.39 D_{60}= 0.118 D_{50}= 0.0527
D_{30}= 0.0093 D_{15}= 0.0022 D_{10}=
C_u= C_c=

Classification

USCS= MH AASHTO=

Remarks

Sample No.: **Source of Sample:** B-3 **Date:** 1/9/20
Location: **Elev./Depth:** 4.5'

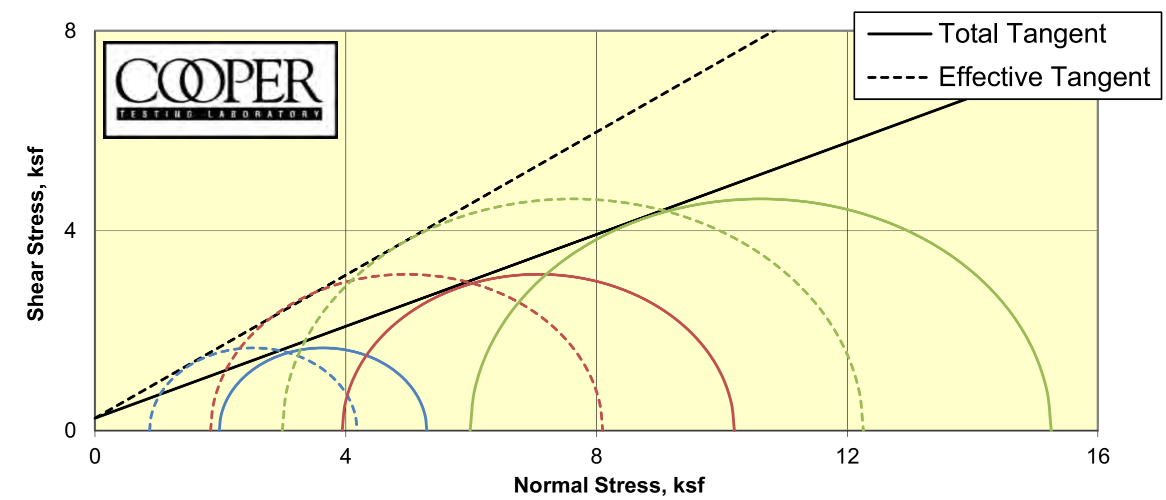
COOPER TESTING LABORATORY

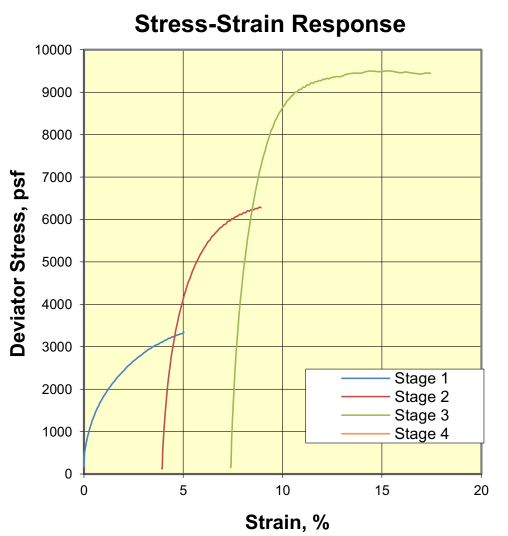
Client: Geosphere Consultants
Project: 91-04747-A

Project No: 724-213

Plate
G19

Staged Consolidated Undrained Triaxial Compression with Pore Pressure
ASTM D4767m



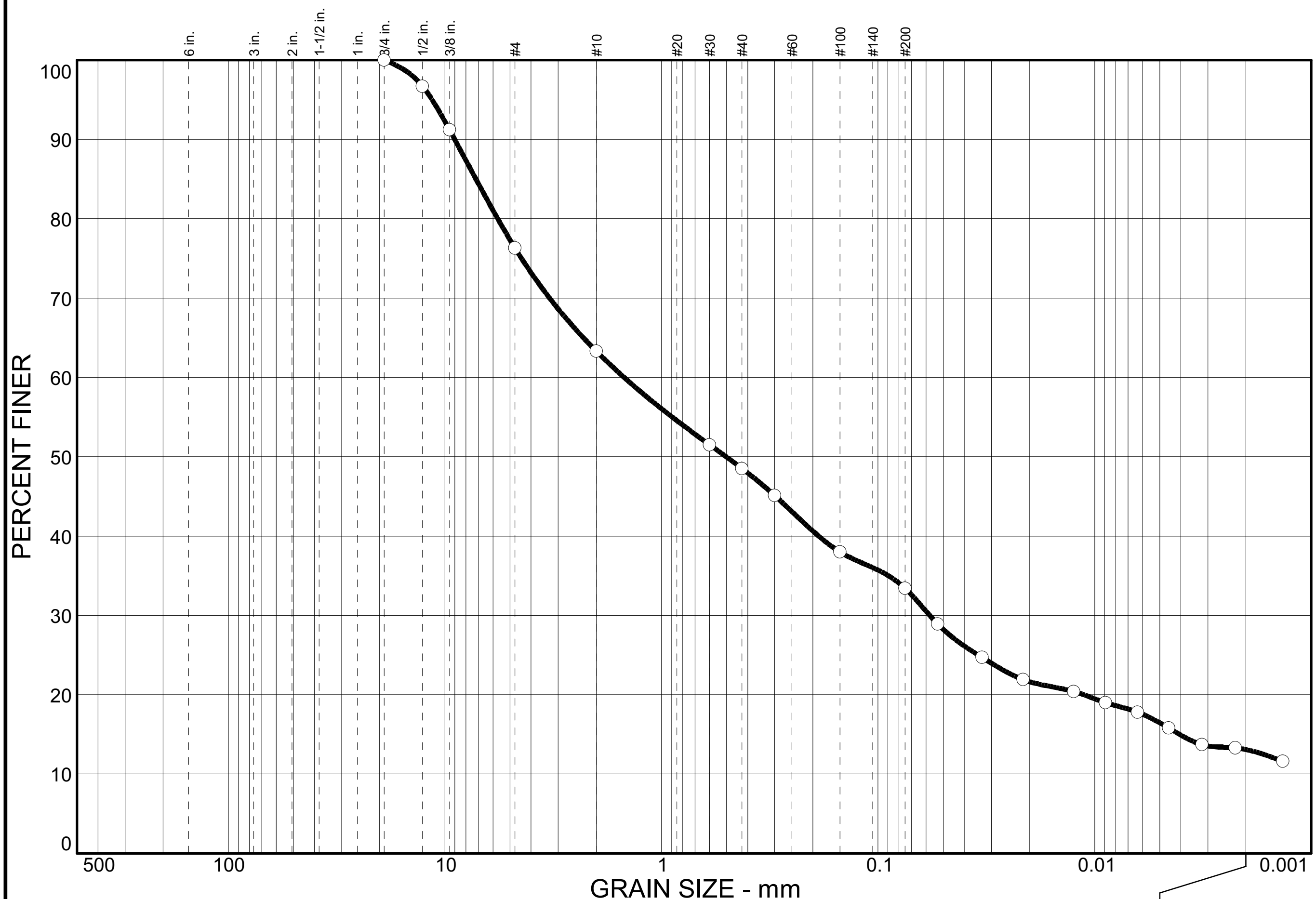


Stage	1	2	3	4
Boring	B-3			
Sample				
Depth	4.5			
Visual Description	Dark Brown Sandy Elastic SILT			
MC (%)	40.6			
Dry Density (pcf)	81.7			
Saturation (%)	99.8			
Void Ratio	1.138			
Diameter (in)	2.42			
Height (in)	5.04			
	Final			
MC (%)	36.9	34.4	32.5	
Dry Density (pcf)	86.0	89.0	91.5	
Saturation (%)	100.0	100.0	100.0	
Void Ratio	1.033	0.963	0.910	
Diameter (in)	2.37	2.38	2.39	
Height (in)	4.98	4.79	4.61	
Cell Pressure (psi)	93.7	107.7	121.6	
Back Pressure (psi)	79.9	80.3	80.1	
	Effective Stresses At:			
Strain (%)	5.0	5.0	5.0	
Deviator (ksf)	3.311	6.257	9.272	
Excess PP (psi)	7.7	14.6	20.8	
Sigma 1 (ksf)	4.180	8.099	12.258	
Sigma 3 (ksf)	0.869	1.842	2.987	
P (ksf)	2.525	4.971	7.622	
Q (ksf)	1.655	3.129	4.636	
Stress Ratio	4.809	4.397	4.105	
Rate (in/min)	0.0005	0.0005	0.0005	

CTL Number:	724-213		
Client Name:	Geosphere Consultants		
Project Name:			
Project Number:	91-04747-A		
Date:	01/06/20	By:	MD/DC
Total C	0.250	ksf	
Total phi	24.7	degrees	
Eff. C	0.250	ksf	
Eff. Phi	35.6	degrees	©

Plate G20

Particle Size Distribution Report



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	23.7	42.9	20.3	13.1

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3/4 in.	100.0		
1/2 in.	96.7		
3/8 in.	91.2		
#4	76.3		
#10	63.3		
#30	51.5		
#40	48.5		
#50	45.1		
#100	38.0		
#200	33.4		
#270	28.9		
0.0331 mm.	24.7		
0.0214 mm.	21.9		
0.0125 mm.	20.4		
0.0089 mm.	19.0		
0.0063 mm.	17.8		
0.0045 mm.	15.8		
0.0032 mm.	13.7		
0.0022 mm.	13.3		
0.0014 mm.	11.6		

* (no specification provided)

Soil Description

Dark Brown Lean Clayey SAND w/ Gravel

Atterberg Limits

PL= 18 LL= 32 PI= 14

Coefficients

D_{85}= 7.19 D_{60}= 1.49 D_{50}= 0.504
D_{30}= 0.0577 D_{15}= 0.0041 D_{10}=
C_u= C_c=

Classification

USCS= SC AASHTO=

Remarks

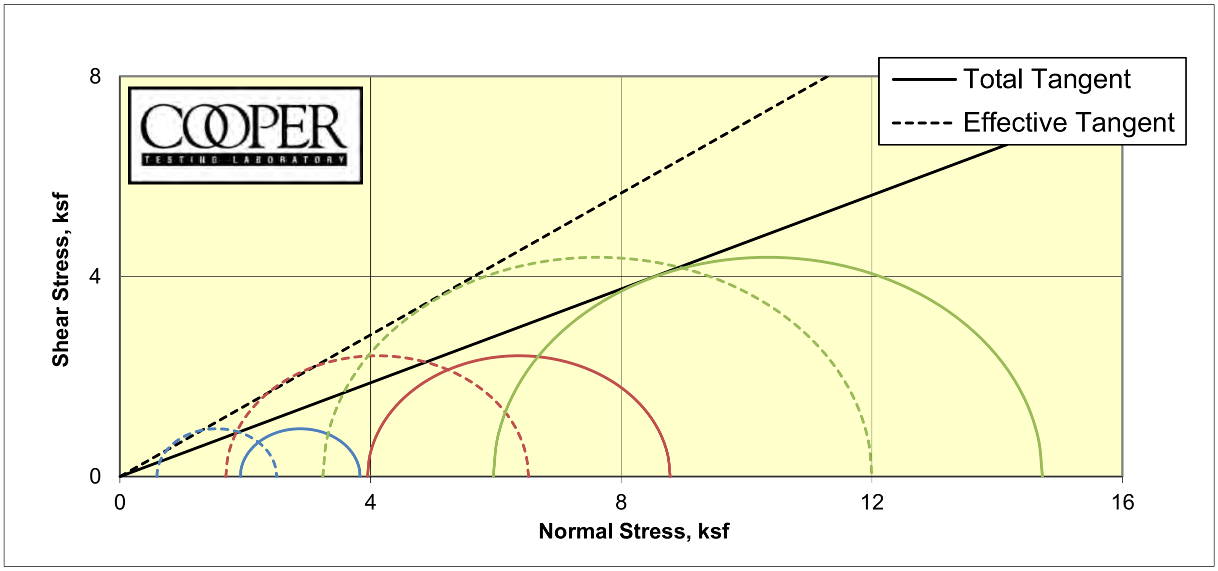
Sample No.: **Source of Sample:** B-3 **Date:** 1/10/20
Location: **Elev./Depth:** 9.5'

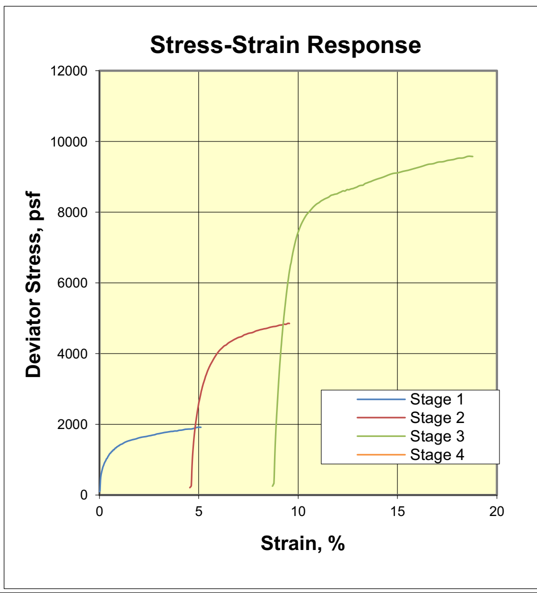
COOPER TESTING LABORATORY

Client: Geosphere Consultants
Project: 91-04747-A
Project No: 724-213

Plate G21

Staged Consolidated Undrained Triaxial Compression with Pore Pressure
ASTM D4767m





Stage	1	2	3	4
Boring	B-3			
Sample				
Depth	9.5			
Visual Description	Dark Brown Lean Clayey SAND w/ Gravel			
MC (%)	6.9			
Dry Density (pcf)	110.8			
Saturation (%)	38.4			
Void Ratio	0.465			
Diameter (in)	2.39			
Height (in)	5.01			
	Final			
MC (%)	15.0	13.5	12.6	
Dry Density (pcf)	116.8	120.1	122.3	
Saturation (%)	100.0	100.0	100.0	
Void Ratio	0.390	0.351	0.327	
Diameter (in)	2.35	2.37	2.40	
Height (in)	4.91	4.69	4.48	
Cell Pressure (psi)	103.7	117.8	131.6	
Back Pressure (psi)	90.4	90.3	90.3	
	Effective Stresses At:			
Strain (%)	5.0	5.0	5.0	
Deviator (ksf)	1.912	4.830	8.765	
Excess PP (psi)	9.3	15.7	18.9	
Sigma 1 (ksf)	2.499	6.519	12.002	
Sigma 3 (ksf)	0.588	1.689	3.237	
P (ksf)	1.543	4.104	7.619	
Q (ksf)	0.956	2.415	4.383	
Stress Ratio	4.253	3.860	3.708	
Rate (in/min)	0.0005	0.0005	0.0005	

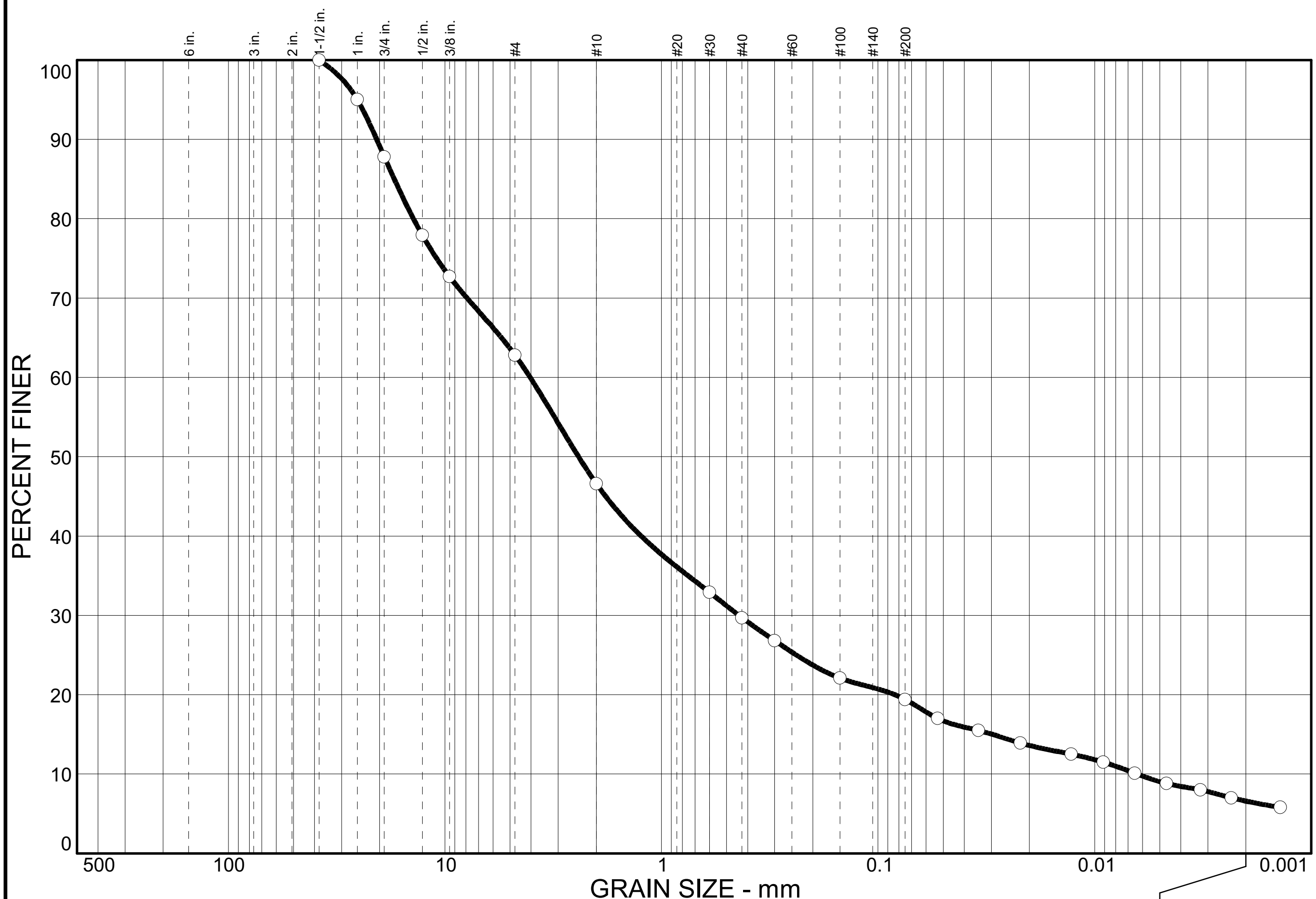
CTL Number:	724-213		
Client Name:	Geosphere		
Project Name:			
Project Number:	91-04747-A		
Date:	01/07/20	By:	MD/DC

Total C	**0.000**	**ksf**
Total phi	**25.1**	**degrees**
Eff. C	**0.000**	**ksf**
Eff. Phi	**35.3**	**degrees**

Remarks: On staged tests, there may appear to be a negative cohesion. This is more pronounced on granular materials. This is caused by the increased density of the second and/ or third stages due to the anisotropic conditions created by the staged procedure.

Plate G22

Particle Size Distribution Report



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	37.2	43.4	12.8	6.6

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1.5 in.	100.0		
1 in.	95.0		
3/4 in.	87.8		
1/2 in.	77.9		
3/8 in.	72.7		
#4	62.8		
#10	46.6		
#30	32.9		
#40	29.7		
#50	26.8		
#100	22.1		
#200	19.4		
#270	17.0		
0.0344 mm.	15.5		
0.0220 mm.	13.9		
0.0128 mm.	12.5		
0.0091 mm.	11.5		
0.0065 mm.	10.1		
0.0046 mm.	8.8		
0.0032 mm.	8.0		
0.0023 mm.	7.0		
0.0014 mm.	5.8		

* (no specification provided)

Soil Description

Dark Brown Lean Clayey SAND w/ Gravel

Atterberg Limits

PL= 18 LL= 32 PI= 14

Coefficients

D_{85}= 17.1 D_{60}= 4.04 D_{50}= 2.42
D_{30}= 0.439 D_{15}= 0.0297 D_{10}= 0.0064
C_u= 635.16 C_c= 7.50

Classification

USCS= SC AASHTO=

Remarks

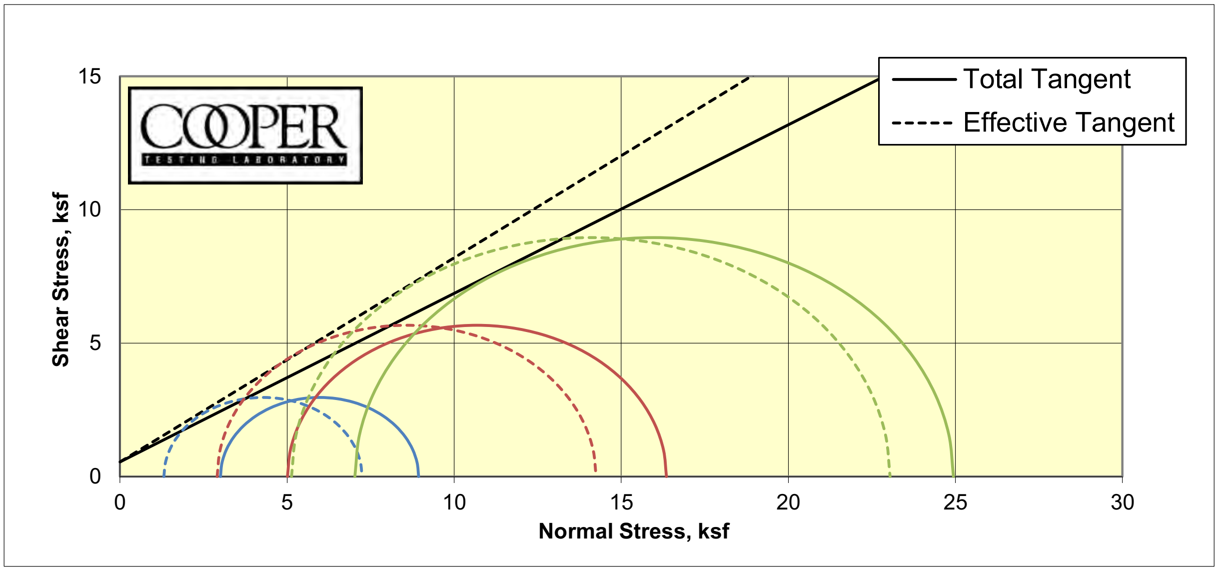
Sample No.: **Source of Sample:** B-3 **Date:** 1/9/20
Location: **Elev./Depth:** 24.5'

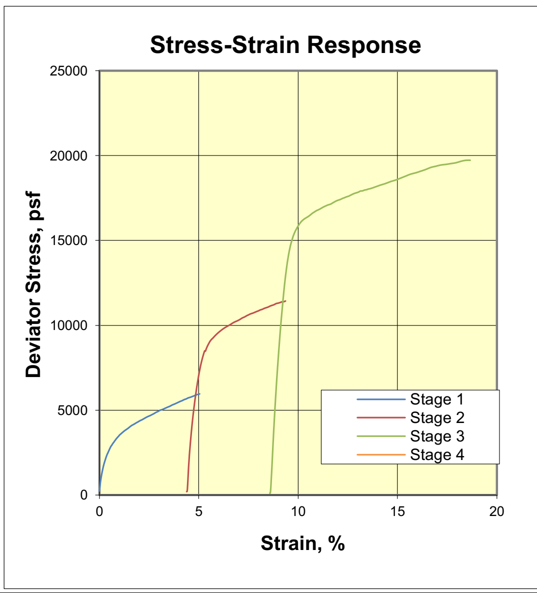
COOPER TESTING LABORATORY

Client: Geosphere Consultants
Project: 91-04747-A
Project No: 724-213

Plate G23

Staged Consolidated Undrained Triaxial Compression with Pore Pressure
ASTM D4767m





CTL Number:	724-213
Client Name:	Geosphere Consultants
Project Name:	
Project Number:	91-04747-A
Date:	01/06/20
By:	MD/DC

Total C	0.550	ksf
Total phi	32.3	degrees
Eff. C	0.550	ksf
Eff. Phi	37.4	degrees

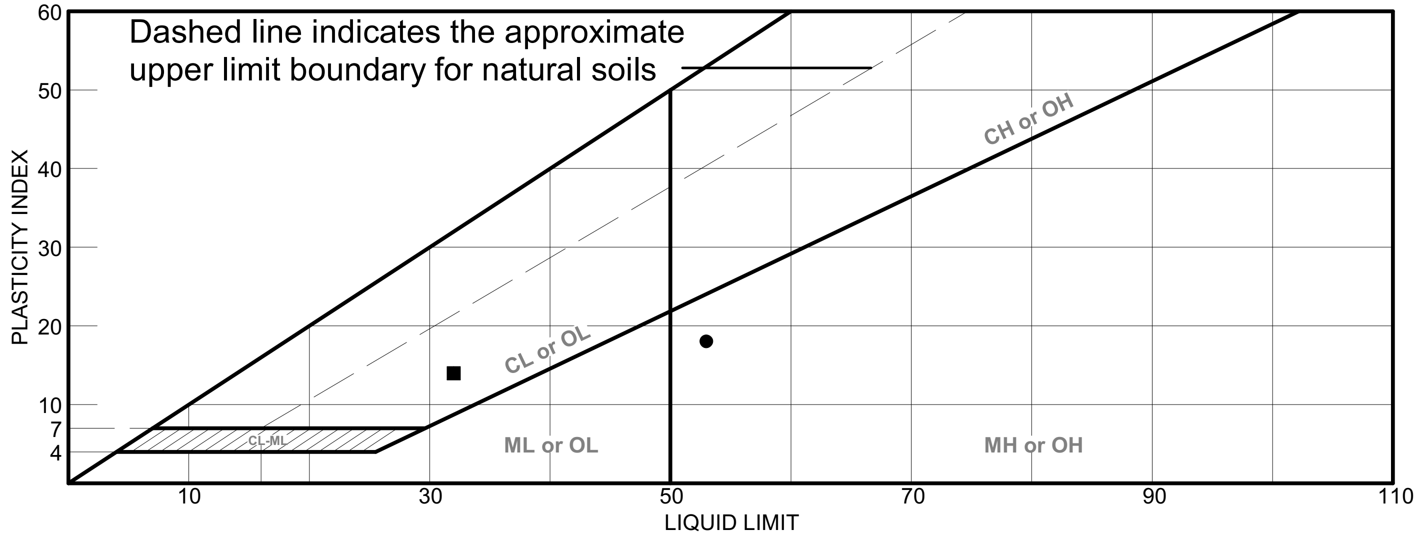
©

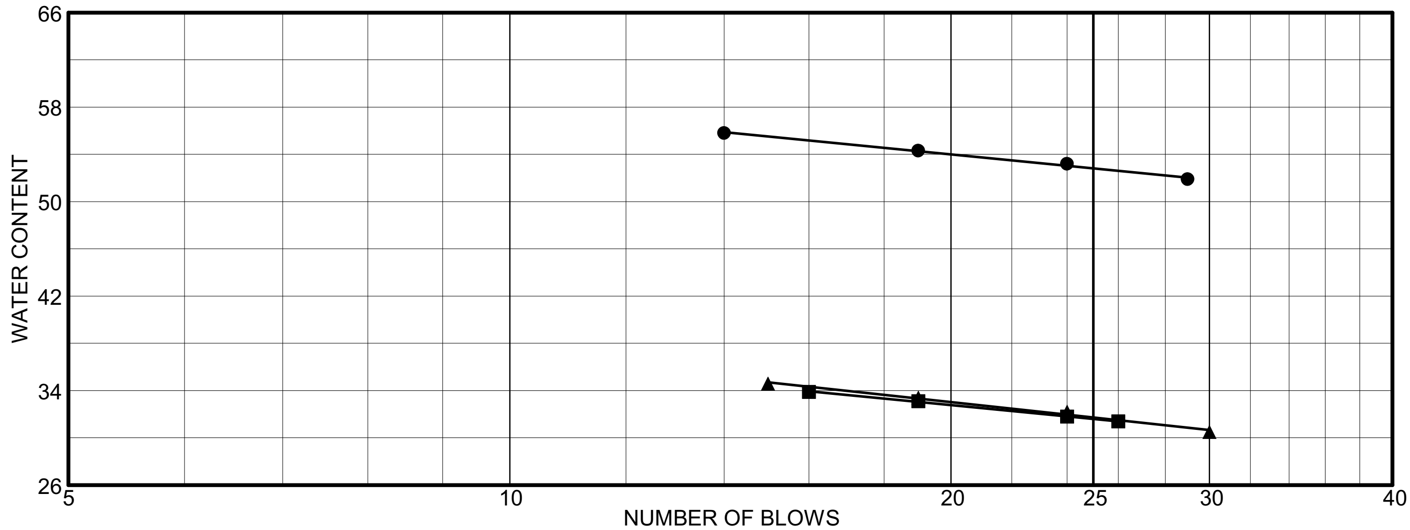
Remarks: On staged tests, there may appear to be a negative cohesion. This is more pronounced on granular materials. This is caused by the increased density of the second and/ or third stages due to the anisotropic conditions created by the staged procedure.

Stage	1	2	3	4
Boring	B-3			
Sample				
Depth	24.5			
Visual Description	Dark Brown Lean Clayey SAND w/ Gravel			
MC (%)	9.7			
Dry Density (pcf)	126.4			
Saturation (%)	74.8			
Void Ratio	0.358			
Diameter (in)	2.39			
Height (in)	5.01			
Final				
MC (%)	11.6	10.9	10.4	
Dry Density (pcf)	130.1	132.2	133.5	
Saturation (%)	100.0	100.0	100.0	
Void Ratio	0.320	0.299	0.286	
Diameter (in)	2.37	2.40	2.44	
Height (in)	4.96	4.74	4.54	
Cell Pressure (psi)	90.6	104.6	118.6	
Back Pressure (psi)	69.8	69.8	69.8	
Effective Stresses At:				
Strain (%)	5.0	5.0	5.0	
Deviator (ksf)	5.930	11.341	17.913	
Excess PP (psi)	11.8	14.6	13.2	
Sigma 1 (ksf)	7.243	14.247	23.045	
Sigma 3 (ksf)	1.312	2.906	5.131	
P (ksf)	4.277	8.576	14.088	
Q (ksf)	2.965	5.671	8.957	
Stress Ratio	5.518	4.903	4.491	
Rate (in/min)	0.0005	0.0005	0.0005	

Plate
G24

LIQUID AND PLASTIC LIMITS TEST REPORT





	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	Dark Brown Sandy Elastic SILT	53	35	18	76.9	54.3	MH
■	Dark Brown Lean Clayey SAND w/ Gravel	32	18	14	48.5	33.4	SC
▲	Dark Brown Lean Clayey SAND w/ Gravel	32	18	14	29.7	19.4	SC

Project No. 724-213 **Client:** Geosphere Consultants

Project: 91-04747-A

● **Source:** B-3 **Elev./Depth:** 4.5'

■ **Source:** B-3 **Elev./Depth:** 9.5'

▲ **Source:** B-3 **Elev./Depth:** 24.5'

Remarks:

●

■

▲

LIQUID AND PLASTIC LIMITS TEST REPORT

COOPER TESTING LABORATORY

Plate

G25

APPENDIX H

This appendix contains the City of San Bruno emergency notification for the storm rain outfall failure.

News Details

Local Emergency Declared Due to Soil Erosion

Update January 22, 2020

The City is working with Hillside Drilling to complete the installation and repair to the Canyon hillside. Hillside Drilling has mobilized their equipment in the area and has started the clearing and removing of bushes and potholing of existing utilities around the location where the permanent repair will be installed.

Due to the work plan scheduled for the week of January 20, 2020, there will be alterations to the lane closures for both west and east bound traffic. The changes will be required on Thursday, January 23 and Friday, January 24 to safely perform the work.

We appreciate your patience and use of caution around the area, while this work effort is underway.

Update January 14, 2020

City staff provided an update on the Crestmoor Canyon Slope Stability Project and emergency repair at the City Council meeting held on January 14, 2020. The update included information on the project progress, slope stabilization plan, cost estimate and project timeline. View the update by accessing either of the two following links:

Video (https://www.youtube.com/watch?v=Qfm4k8KHr50)

Presentation Slide Deck (https://www.sanbruno.ca.gov/civicax/filebank/blobdload.aspx?BlobID=31389)

Update January 9, 2020

City staff are continuing to work with the structural engineers and contractor to complete the repair to the hillside. The material for the repair has been ordered and is in process of being fabricated and prepared for installation. This process is expected to take approximately two weeks. Work is anticipated to begin during the last two weeks of January 2020.

The far right westbound travel lane of San Bruno Avenue will remain closed as a precaution, as well as the sidewalk in the area. We appreciate your patience, while this work effort is underway.

Update December 18, 2019

On Thursday, December 12, the City's hired contractor completed the installation of a 30" HDPE storm drain pipeline down the Cretsmoor Canyon hillside. At this time, no additional land movement has been observed.

City staff continue to work with the contractor on the design and permanent installation of tiebacks and wall to secure the hillside and roadway, and will also coordinate with PG&E. Completion of design and construction is expected to take approximately 8 weeks, depending on the weather.

The far right westbound travel lane of San Bruno Avenue will remain closed as a precaution, as well as the sidewalk in the area. We appreciate your patience, while this work effort is underway.

Update December 8, 2019

On December 3, a resolution was issued proclaiming a local emergency due to a landslide in the Crestmoor Canyon near W San Bruno Avenue between Glenview and Crestmoor Drives. City staff are continuously monitoring the slide and also working with geotechnical engineers and specialty contractors to make necessary immediate mitigating efforts.

Over the storms this weekend, staff reports no additional movement of the land. A permanent slope stabilization repair will be expedited to preserve the integrity of the hillside and roadway.

The far right westbound travel lane of San Bruno Avenue will remain closed as a precaution, as well as the sidewalk in the area.

Updated Statement and Landslide Markers Image (https://www.sanbruno.ca.gov/civicax/filebank/blobdload.aspx?t=54415.07&BlobID=31313)

December 3, 2019

Recent storms have caused a landslide that eroded approximately 30 feet of hillside along the western portion of San Bruno Avenue, between Crestmoor Drive and Glenview Drive. At present, approximately 10 feet of hillside remains between the edge of the slope and the sidewalk. No homes or private property are threatened by the soil erosion.

In response to the landslide, San Bruno City Manager Jovan D. Grogan issued a resolution proclaiming the existence of a local emergency on December 3, 2019. The proclamation allows for the City Manager to immediately acquire geotechnical, construction, and other services to mitigate the landslide in an effort to prevent the erosion from impacting the sidewalk and San Bruno Avenue.

The public should be advised that the sidewalk and the right travel lane in the west bound direction of San Bruno Avenue are closed for approximately 300 feet near Crestmoor Drive, until further notice.

Additional updates and information on the slope stabilization plan will be released as they become available.

Statement, with Landslide Images & Resolution Proclaiming Existence of a Local Emergency (https://www.sanbruno.ca.gov/civicax/filebank/blobdload.aspx?t=85196.23&BlobID=31289)