GEOTECHNICAL INVESTIGATION AND GEOLOGIC HAZARD EVALUATION
Dove Hill Assisted Living Community
4200 Dove Hill Road
San Jose, California

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1.0 INTRODUCTION

This report presents the results of the geotechnical investigation and geologic hazard evaluation performed by Langan Treadwell Rollo (Langan) for the Dove Hill Assisted Living Community Project at 4200 Dove Hill Road in San Jose, California. The subject property is north of Hassler Parkway and east of Highway 101, as shown on Figure 1.

The site encompasses approximately 21 acres; however, only a 3-acre portion of the site is slated for development. Based on our review of site topography[1], existing grades within the proposed development range between approximately Elevation 180 to 220 feet[2].

We understand that the existing structures will be demolished and replaced with two new buildings, designated as Buildings A and B; both buildings are proposed to be four stories above a podium level. According to the project plans, the finished floor of Buildings A and B will be at Elevations 187 and 207 feet, respectively. Cuts and fills on the order of approximately three feet are anticipated to achieve final grades for the two new building pads. A garden and recreational area are proposed upslope of Building B.

A previous geologic hazards evaluation and geotechnical engineering study titled *Dove Hill Assisted Living Community, APN 679-08-002/003; APN 679-09-001/002: (21± Acres), 4200 Dove Hill Road, San Jose, Santa Clara County, California* was prepared by E_2C, Incorporated (dated 3 September 2008). Subsurface data from this study was used in our investigation, and the approximate locations of the E_2C borings are shown on Figures 2 and 3. Boring logs and laboratory test data from this report are presented in Appendix A.

1 Caruso (2015). Electronic file provided by Salvatore Caruso Design Corporation titled, "ACAD-site-06 Survey Only Shown," dated 18 March 2015.

2 All elevations reference North American Vertical Datum of 1988 (NAVD 88).

2.0 SCOPE OF SERVICES

Our geotechnical investigation was performed in general accordance with the scope of services outlined in our proposal dated 8 October 2014. The scope consisted of reviewing previous site reports, advancing eight exploratory borings and performing engineering analyses to develop conclusions and recommendations regarding:

- anticipated subsurface conditions including groundwater levels;
- 2013 California Building Code (CBC) site classification, mapped values S_S and S_1, modification factors F_a and F_v and S_{MS} and S_{M1};
- site seismicity and potential for seismic hazards;
- appropriate foundation type(s) including deep foundations, as necessary;
- estimated range of capacities for the probable foundation type;
- anticipated settlement;
- corrosivity, with brief evaluation;
- construction considerations.

Our geologic hazard evaluation was performed concurrently with our geotechnical investigation. Our scope of services for the geologic hazard evaluation included:

- review of available geologic, subsurface and other technical data for the site and vicinity;
- review of stereo-paired aerial photographs;
- performing detailed site engineering geologic mapping;
- submitting soil and rock samples to determine the presence for asbestos-bearing serpentinite;
- preparing four idealized subsurface (Geologic) profiles;
- performing liquefaction analyses; and
- performing slope stability analyses for slopes adjacent to the proposed structures to determine stability under static and seismic conditions.

3.0 FIELD EXPLORATION AND LABORATORY TESTING

To evaluate surface and subsurface conditions, we advanced eight exploratory borings, submitted samples for laboratory testing and completed geologic mapping.

3.1 Exploratory Borings

To supplement the available subsurface data, we drilled eight test borings. The approximate locations of the borings are presented on Figures 2 and 3. Prior to performing the field investigation, we notified Underground Service Alert (USA) at least 48 hours prior to proceeding with our exploratory drilling. We also retained the services of a private utility locator to verify clearance of underground utilities.

On 25 March 2015, eight test borings, designated as B-1 through B-8, were drilled using a truck-mounted drill rig equipped with hollow stem augers, operated by Exploration Geoservices, Inc. The borings were drilled to a maximum depth of approximately 23.5 feet below the existing ground surface (bgs). Our geologists logged the soil conditions encountered in the borings and obtained samples for visual classification and laboratory testing. The logs of borings are presented on Figures B-1 through B-8 in Appendix B. The soil and rock encountered in the borings were classified in accordance with the Classification Chart presented on Figure B-9 and the physical properties criteria for rock descriptions for Figure B-10, respectively..

Soil samples were obtained using two different types of driven split-barrel samplers. The sampler types are as follows:

- Sprague & Henwood (S&H) sampler with a 3.0-inch outside diameter and 2.5-inch inside diameter, lined with steel or brass tubes with an inside diameter of 2.43 inches
- Standard Penetration Test (SPT) sampler with a 2.0-inch outside diameter and 1.5-inch inside diameter, without liners

The sampler types were chosen on the basis of soil type being sampled and desired sample quality for laboratory testing. In general, the S&H sampler was used to obtain samples in medium stiff to very stiff cohesive soil and the SPT sampler was used to evaluate the penetration resistance of sandy soil.

The SPT and S&H samplers were driven with a 140-pound, above-ground, downhole safety wireline hammer falling 30 inches. The samplers were driven up to 18 inches and the hammer blows required to drive the samplers every six inches of penetration were recorded and are presented on the boring logs. A "blow count" is defined as the number of hammer blows per six inches of penetration. The blow counts required to drive the S&H and SPT samplers were converted to approximate SPT N-values using factors of 0.6 and 1.0, respectively, to account for sampler type and hammer energy, and are shown on the boring logs. The blow counts used for this conversion were the last two blow counts.

Upon completion, the boreholes were backfilled with cement grout in accordance with the requirements of the SCVWD.

The soil cuttings from the borings were collected in 55-gallon drums, which were stored temporarily at the site, tested, and eventually transported off-site for proper disposal.

3.2 Reconnaissance Engineering Geologic Mapping

On 30 March 2015, our geologists completed detailed engineering geologic mapping of the site. The purpose of the mapping was to identify, characterize and evaluate site surface conditions and their potential impact on proposed site improvements. Using a recent site topographic survey, we documented the distribution of earth materials, rock outcrops, landslide features, slope inclinations, spring and seep locations, and the distribution of cuts and fills (Figure 4, Engineering Geologic Map).

3.3 Laboratory Testing

The samples recovered from the field investigation were examined to verify their soil classification, and representative samples were selected for laboratory testing. Soil samples were tested to measure moisture content, shear strength, plasticity (Atterberg Limits) and R-value. Results of the laboratory tests are included on the boring logs and in Appendix C. Laboratory tests from previous investigations by others are included in Appendix A.

To evaluate the corrosivity of the soil near the foundation subgrade, we performed corrosivity tests on samples obtained from the upper five feet. The corrosivity of the soil samples was evaluated by CERCO Analytical using the following ASTM Test Methods:

- Redox - ASTM D1498

- pH - ASTM D4972
- Resistivity (100% Saturation) – ASTM G57
- Sulfide – ASTM D4658M
- Chloride – ASTM D4327
- Sulfate – ASTM D4327

The laboratory corrosion test results and a brief corrosivity evaluation are presented in Appendix D.

In addition, we performed asbestos test, on selected samples of rock. The results are included in Appendix E

4.0 SITE AND SUBSURFACE CONDITIONS

4.1 Site Conditions

The subject property is located within the eastern foothills above San Jose, immediately adjacent to the northbound lanes of Highway 101. The site is accessed from Dove Hill Road, which intersects the Hellyer Avenue exit. Two main driveways lead to the existing residence from Dove Hill Road. The site is currently occupied by a two-story residential structure, barns and several auxiliary structures. The portion of the site immediately adjacent to Dove Hill Road is occupied by an industrial yard, with trailers and various types of machinery. A water tank is located approximately 550 feet upslope from the existing residence. The remainder of the property above the existing structures is open horse pasture.

Site topography is characterized by steep north- to west-facing hillslopes. Site slopes in the vicinity of the proposed improvements have been significantly altered by grading. A steep cut slope above the existing residence up to 35 feet high is unretained, and shows signs of surficial creep. The southern half of the site appears to have undergone significant quarrying/borrow excavations, resulting in steep cuts up to 1:1 (horizontal to vertical) inclinations. Areas of instability were noted on site cut slopes (Figure 4).

Previous grading activities in the vicinity of existing improvements have resulted in two relatively level building pads, flanked by moderately steep to steep cut slopes up to 34 degrees. Cut slopes above the lower building pad are covered in concrete rubble and timber debris.

Several areas of fill were observed on the slopes above the existing residence, and on the southern slopes above Dove Hill Road. Isolated areas of fill on site slopes were also noted, and are likely remnants of previous site grading activities.

A spring-fed horse trough was observed on the slope above proposed Building A. Areas that appeared to be relatively lush, green and muddy were noted around the site, indicating the likelihood for underground seeps that were not observed at the ground surface. These areas are depicted on Figure 4. Water-loving grasses and plants are indicated as phreatophytes.

4.2 Aerial Photo Review

We evaluated site conditions based on aerial photo interpretation. We used standard stereographic photograph interpretation techniques to map geologic-related features such as benches, tonal lineaments, linear vegetation features, seepage, depressions and other surface lineaments.

We have identified prominent surficial features on 14-pairs of aerial photographs spanning 1954 through 1990. The contact prints that we reviewed for investigation are included in Table 1.

The 1954 photographs show the property located at 4200 Dove Hill Road to lie at the base of a northern-facing hillslope. Access to the property was via an unimproved road leading east from (present-day) Hwy 101 and leading south toward the larger of two buildings. The site was within a drainage swale bound by hillslopes that form an amphitheater-shaped drainage upslope of the buildings on the site. What appears as farm buildings were constructed on an upper bench cut into the base of the north-facing slope. At about mid-slope and north of the buildings we identified a ditch cut along a contour into the hillside; dark tonal contrast in the photograph downslope below the ditch, suggest seepage and heavy vegetation growth. The hill to the south of the buildings appeared to have many "terracettes", perhaps created by cows or other domestic grazing animals. At this date, there were two bench levels with cuts upslope. The site was covered in grass with the exception of several trees and scattered bushes.

There is a gap in available high resolution aerial photography from 1954 to 1971. We observed the following conditions from the 1971 photographs. Several buildings and stables were added next to the larger of the two original buildings; the smaller building was removed, and a new access road constructed that approaches the property from the south along Hwy 101. Extensive grading of the north-facing hill slope was completed. The western nose of the hill was cut to accommodate the southern access road; the excavated material was pushed to the northeast. The north-facing slope was cut into and material was pushed northward into the swale. Another road was also cut leading from the base through the middle of the hill. The lower cut slope located near the center of the property was extended northward and a new building was erected at the base of the new cut-slope. Two small debris slides are evident on the outboard edge of the middle slope cut.

Two landslides coalesced to include the mid-slope road cut on the north-facing slope sometime between 1971 and 1974. These landslides left head scarps evident today that undermined the road and cut slope above.

The landslide deposits accumulated on the benched level area and appeared to be graded in the 1976 photos. A small pond impoundment was present in the excavated area east of the landslide deposits. This pond increased in size in the 1976 photos. A gentle low topographic area downstream of the pond collects surface runoff that flows over the top of the cut slope below, causing instability above the existing buildings. This is exacerbated by the increased pond size and general poor drainage of this benched/graded area.

Between 1982 and 1984, two smaller landslides occurred along the slope directly behind the largest building. Their head scarps were near the top of the slope and their toes appeared to have been either minimal, or possibly cleared away since emplacement. Also between the years 1982 and 1984, contrasting dark tonal areas on the northern hillside at two localities indicate seepage from hillslope springs. These areas do not appear in aerial photos after 1984.

Much of the material displaced by the cuts made in the north-facing slope appears to remain through the time of the most recent aerial photo (1999). Vegetation including small trees were observed to proliferate on the displaced fill. The upper half of the hillside to the east and northeast appears much darker than the lower half of the hill in the 1996 and 1999 photos. One possible cause for this darkening of the ground surface is a brush fire sometime between 1992 and 1996. Two new small slump landslides appear and to initiate in the cut slope above the buildings in 1992, and grow in dimensions by 1996.

TABLE 1
List of Aerial Photographs Reviewed

Date	Photo Number	Scale	Source
3/2/1954	AV-129-14-27,28	1:9,600	Pacific Aerial Surveys
9/28/1963	CIV-6DD-128,129	1:20,000	USGS
5/24/1965	SCL 19-44, 45	1:20,000	USGS
10/12/1971	AV-1006-18-20,21	1:12,000	Pacific Aerial Surveys
7/12/1974	11-171, 172 4	1:12,000	USGS
7/23/1980	AV-1905-17-15,16	1:12,000	Pacific Aerial Surveys
2/22/1981	3-140, 141 GS VEZR	1:24,000	USGS
4/30/1982	AV-2135-18-19,20	1:12,000	Pacific Aerial Surveys
6/28/1988	AV-3324-17-24,25	1:12,000	Pacific Aerial Surveys
7/23/1990	AV-3845-29-90,91	1:12,000	Pacific Aerial Surveys
7/22/1992	AV-4230-0131-79, 80	1:12,000	Pacific Aerial Surveys
7/31/1996	AV-5200-31-78,79	1:12,000	Pacific Aerial Surveys
8/20/1999	AV-6100-231-29,31	1:12,000	Pacific Aerial Surveys

4.3 Subsurface Conditions

According to published geologic maps, the site is mapped as underlain by Jurassic-age serpentinized harzburgite and dunite of the Silver Creek Block (Regional Geologic Map, Figure 5). Where explored, the near surface material encountered in the borings consists of undifferentiated colluvial and artificial fill deposits. Materials identified as potential artificial fill appear to have been derived from the colluvium. These deposits consist of very stiff, high plasticity clay to a depth of approximately four to nine feet below ground surface (bgs). Laboratory tests results indicate the near surface clay has high expansion potential[3] with plasticity indices ranging from 50 to 52. Samples of this material submitted for laboratory testing indicate that the undrained shear strengths of the near surface clay range from 1,900 to 5,140 pounds per square foot (psf).

Bedrock encountered underlying the surficial materials include sheared serpentinized harzburgite and dunite, with inclusions of mélange, sandstone and shale, to the maximum depth explored.

3 Highly expansive soil undergoes large volume changes with changes in moisture content.

Groundwater was measured at approximately 10.5 feet bgs, corresponding to Elevation 199 feet, in boring B-8. Groundwater was not encountered at borings B-1 through B-7. Seasonal fluctuations in rainfall influence groundwater levels and may cause several feet of variation.

Using the results of our exploratory drilling, geologic mapping as well as boring log data from the previous site investigation, we developed idealized subsurface profiles (Figures 6 through 9) to depict the general surface, geologic and subsurface conditions with respect to the proposed site improvements.

5.0 REGIONAL GEOLOGY, SEISMICITY AND GEOLOGIC HAZARDS

During a major earthquake, strong to violent ground shaking is expected to occur at the project site. Strong ground shaking during an earthquake can result in ground failure such as that associated with soil liquefaction,[4] lateral spreading,[5] cyclic densification,[6] landsliding, or seismically-induced landsliding. Each of these conditions, and other seismic hazards affecting the site, has been evaluated based on our literature review, field investigation and analysis, and are discussed in this section.

5.1 Regional Geology

The site is located in the Coast Ranges geomorphic province, which is characterized by northwest-southeast trending valleys and ridges. These are controlled by folds and faults that resulted from the collision of the Farallon and North American plates and subsequent shearing along the San Andreas fault. The bedrock in the site vicinity is mapped as Jurassic serpentinized harzburgite and dunite of the Silver Creek Block (Figure 5). This unit is characterized as mainly sheared serpentinite, but also includes massive serpentinized harzburgite. The unit was identified and mapped in the field as serpentinite and serpentinized dunite.

4 Liquefaction is a transformation of soil from a solid to a liquefied state during which saturated soil temporarily loses strength resulting from the buildup of excess pore water pressure, especially during earthquake-induced cyclic loading. Soil susceptible to liquefaction includes loose to medium dense sand and gravel, low-plasticity silt, and some low-plasticity clay deposits.

5 Lateral spreading is a phenomenon in which surficial soil displaces along a shear zone that has formed within an underlying liquefied layer. Upon reaching mobilization, the surficial blocks are transported downslope or in the direction of a free face by earthquake and gravitational forces.

6 Cyclic densification is a phenomenon in which non-saturated, cohesionless soil is densified by earthquake vibrations, causing ground-surface settlement.

The site is mapped within a zone designated with the potential for liquefaction and a zone designated as susceptible to earthquake-induced landsliding, according to the *State of California Seismic Hazard Zones Map of the San Jose East 7.5-Minute Quadrangle, Santa Clara County*, prepared by the California Geologic Survey (CGS, formerly the California Division of Mines and Geology), dated 17 January 2001 (Figure 10). According to published landslide maps (CGS, 2011), two landslides features are mapped on the north portion of the site, above proposed Building A. A large landslide is mapped in the southern half of the site, above proposed Building B and the proposed garden and recreational area.

5.2 Regional Seismicity

The major strands of active faults in the area are the San Andreas, San Gregorio, Hayward, and Calaveras faults. These and other faults of the region are shown on Figure 11. For each of the active faults within 100 kilometers (km) of the site, the distance from the site and estimated mean characteristic Moment magnitude[7] [2008 Working Group on California Earthquake Probabilities (WGCEP) (2008) and Cao et al. (2003)] are summarized in Table 1.

TABLE 2
Regional Faults and Seismicity

Fault Segment	Approximate Distance from Site (km)	Direction from Site	Mean Characteristic Moment Magnitude
Monte Vista-Shannon	9.1	Southwest	6.50
Total Calaveras	10	East	7.03
Total Hayward	19	North	7.00
Total Hayward-Rodgers Creek	19	North	7.33
N. San Andreas - Santa Cruz	21	Southwest	7.12
N. San Andreas (1906 event)	21	Southwest	8.05
N. San Andreas – Peninsula	21	Southwest	7.23
Zayante-Vergeles	27	Southwest	7.00
Greenville Connected	33	Northeast	7.00
Ortigalita	47	East	7.10
San Gregorio Connected	47	West	7.50
Mount Diablo Thrust	49	North	6.70
Monterey Bay-Tularcitos	49	Southwest	7.30

7 Moment magnitude is an energy-based scale and provides a physically meaningful measure of the size of a faulting event. Moment magnitude is directly related to average slip and fault rupture area.

Fault Segment	Approximate Distance from Site (km)	Direction from Site	Mean Characteristic Moment Magnitude
Great Valley 7	53	Northeast	6.90
Quien Sabe	56	Southeast	6.60
Great Valley 8	60	East	6.80
SAF - creeping segment (jl0.sa-creep, modified)	61	Southeast	6.70
Rinconada	68	South	7.50
Green Valley Connected	70	North	6.80
Great Valley 9	76	East	6.80
Great Valley 5, Pittsburg Kirby Hills	81	North	6.70
N. San Andreas - North Coast	87	Northwest	7.51

Figure 11 also shows the earthquake epicenters for events with magnitude greater than 5.0 from January 1800 through August 2014. Since 1800, four major earthquakes have been recorded on the San Andreas fault. In 1836 an earthquake with an estimated maximum intensity of VII on the Modified Mercalli (MM) scale (Figure 12) occurred east of Monterey Bay on the San Andreas fault (Toppozada and Borchardt 1998). The estimated Moment magnitude, $M_{w,}$ for this earthquake is about 6.25. In 1838, an earthquake occurred with an estimated intensity of about VIII-IX (MM), corresponding to a M_w of about 7.5. The San Francisco Earthquake of 1906 caused the most significant damage in the history of the Bay Area in terms of loss of lives and property damage. This earthquake created a surface rupture along the San Andreas fault from Shelter Cove to San Juan Bautista approximately 470 kilometers in length. It had a maximum intensity of XI (MM), a M_w of about 7.9, and was felt 560 kilometers away in Oregon, Nevada, and Los Angeles. The Loma Prieta earthquake occurred on 17 October 1989, in the Santa Cruz Mountains with a M_w of 6.9, approximately 29 km from the site.

In 1868 an earthquake with an estimated maximum intensity of X on the MM scale occurred on the southern segment (between San Leandro and Fremont) of the Hayward Fault. The estimated M_w for the earthquake is 7.0.

In 1861, an earthquake of unknown magnitude (probably a M_w of about 6.5) was reported on the Calaveras fault. The most recent significant earthquake on this fault was the 1984 Morgan Hill earthquake (M_w = 6.2).

The most recent earthquake to be felt in the Bay Area is the Napa earthquake, which occurred on 24 August 2014 with a M_w of 6.0. The earthquake epicenter is approximately 113 kilometers north of the site, and is believed to have occurred within the Napa fault system.

The 2007 WGCEP at the U.S. Geologic Survey (USGS) predicted a 63 percent chance of a magnitude 6.7 or greater earthquake occurring in the San Francisco Bay Area in 30 years. More specific estimates of the probabilities for different faults in the Bay Area are presented in Table 2.

TABLE 3
WGCEP (2008) Estimates of 30-Year Probability of a Magnitude 6.7 or Greater Earthquake

Fault	Probability (percent)
Hayward-Rodgers Creek	31
N. San Andreas	21
Calaveras	7
San Gregorio	6

5.3 Fault Rupture

In addition to the faults indicated in Table 2, According to the USGS Quaternary Fault and Fold Database (2006), the site is located approximately 1.1 miles southwest of the potentially active Silver Creek fault, and 0.5 miles northeast of the northern terminus of a potentially active, unnamed fault. The potentially active Piercy fault is located approximately one mile to the southwest.

The Silver Creek fault trends northwest-southeast. It is a steeply west-dipping reverse fault that has been identified as potentially active and capable of a significant seismic activity based on geomorphic and paleoseismic evidence presented by Hitchcock and Brankman (2002). Outcrop evidence shows that the Silver Creek fault dips westward, making it distinct from the nearby Quinby and Evergreen eastward-dipping reverse faults. Hitchcock and Brankman (2002) present structural, geologic, and geomorphic evidence that the Silver Creek fault is part of the Foothills Thrust Fault System and is therefore influenced by the restraining bend in the San Andreas fault within the Santa Cruz Mountains.

The Piercy fault is one of several potentially active, northwest-southeast trending, east-dipping reserve faults that run through the eastern Santa Clara Valley. Rapid uplift of the East Bay structural domain (1.5 ±0.5 mm/yr) has been accommodated in part by the Piercy fault.

We evaluated the risk of fault rupture at the site associated with active or potentially active fault traces. Historically, ground surface displacements closely follow the trace of geologically young faults. Based on our study and geologic mapping, we conclude the site is not within an Earthquake Fault Zone, as defined by the Alquist-Priolo Earthquake Fault Zoning Act and no known active or potentially active faults exist on the site. Therefore, we judge the risk of surface faulting at the site is low. However, in a seismically active area, the remote possibility exists for future faulting in areas where no faults were previously mapped.

5.4 Liquefaction

The western half of the site is within a zone designated with the potential for liquefaction, in the official hazards map titled *State of California Seismic Hazard Zones, San Jose East Quadrangle*, prepared by the California Geologic Survey (CGS), dated 17 January 2001 (Figure 10). Specifically, the map shows the site is in an area "where historic occurrence of liquefaction, or local geological, geotechnical and groundwater conditions indicate a potential for permanent ground displacements such that mitigation as defined in Public Resources Code Section 2693 (c) would be required."

The borings indicate the site is underlain by clay above bedrock. Because the soil encountered above bedrock consists of high plasticity clay, we judge the liquefaction potential as low.

5.5 Lateral Spreading

Because of the clayey nature of the soils overlying bedrock, lack of liquefiable deposits at the site, and lack of an open face above a channel or waterway in the vicinity of the proposed improvements, we conclude that the potential for lateral spreading at the site is low.

5.6 Cyclic Densification

Cyclic densification refers to seismically-induced differential compaction of non-saturated granular material (sand and gravel above the groundwater table) caused by earthquake vibrations. Because the soil above the groundwater level is clay, we conclude that cyclic densification is negligible.

5.7 Landsliding

According to the CGS Seismic Hazard Zone Report for the San Jose East Quadrangle (2001) the site is located on a slope in the western portion of the Silver Creek Hills where the combination of dissected hills and weak rocks has produced widespread and abundant landslides (California Department of Conservation, 2000). Published mapping indicates the site is within a zone where landslides have occurred, as identified by the official seismic hazards map of the area (Figure 10).

Two landslides were mapped identified in the field on the slopes in the northern half of the property above proposed Building A, designated as Qlso (old landslide) and Qlsd (dormant landslide), north to south (Figure 4). The old landslide is characterized as a shallow translational slide, with abundant bedrock boulders within the slide mass. A spring is located near the toe of the slide.

The dormant landslide is characterized as a shallow earth flow, likely confined to the surficial materials overlying bedrock. The slide appears as a very subtle feature identified by slightly elevated, uneven topography extending downslope, with tonal differences in vegetation relative that growing on the adjacent slopes.

During our aerial photo review, we identified an area of instability in the southern half of the property, on the slopes in the previous borrow/quarry area. Cut slopes above the proposed garden/recreation area appear to have failed in the past. Cut slope inclinations were measured in the field to be generally 1:1 (H:V), and occurred within bedrock materials. Very steep cut slope inclinations and the sheared nature of site bedrock, which is highly susceptible to weathering, likely contributed to instability in this area. Areas of instability observed during our aerial photo review are indicated on Figure 4.

Slip-outs were also observed in the cut slope above the existing residence, which was excavated into colluvial materials. Slope inclinations within the cut face range between 23 and 34 degrees. Cut slope failure behind the house was noted in the 1984 photos.

5.8 Expansive Soil

Surficial materials mantling the bedrock were determined through laboratory testing to have plasticity indices ranging from 50 to 52. Dessication cracks were also observed on the ground surface at the site, in areas underlain by clayey artificial fill and colluvium. We conclude that the potential for expansive soil to impact the proposed improvements is high.

5.9 Asbestos-Bearing Bedrock

Serpentinite bedrock is exposed in cut slopes within and around both parcels. According to CGS Note 14, serpentinite is primarily composed of one or more of the three magnesium silicate minerals: lizardite, chrysotile and antigorite. Chrysotile often occurs in fibrous veinlets in serpentinite, and is the most common type of asbestos. Lizardite and antigorite do not form asbestos fibers. Because serpentinite often contains some asbestos and exposure to asbestos fibers have potential human health consequences, testing was conducted on site bedrock materials to determine if they are asbestos-bearing.

Asbestos is typically a concern when it becomes airborne with potential for being inhaled. Mitigation during construction usually requires dust control. Any excavated asbestos containing material must be properly disposed. Encapsulation of asbestos containing materials that remain on-site is usually sufficient for health and safety of the public.

CERCO Analytical performed tests on serpentinite bedrock samples to evaluate the presence of asbestos. Select samples were analyzed by the Air Resources Board's Method 435, Determination of Asbestos Content of Serpentine Aggregate. The results of the tests are presented in Appendix E. The tests indicate up to 0.25 percent asbestos. Appropriate measures such as dust control and disposal may be required.

6.0 DISCUSSION AND CONCLUSIONS

We conclude the proposed development is feasible from a geotechnical standpoint, provided the recommendations presented in this report are incorporated into the project plans and implemented during construction. The primary geotechnical issues for this project include:

- the presence of near surface expansive soil;
- selection of an appropriate foundation system to support the building loads; and
- the potential for slope displacements during moderate to large earthquakes on slopes above the proposed development and long term creep.

Our conclusions regarding these and other geotechnical issues are discussed in the remainder of this section.

6.1 Expansive Soil Considerations

The existing near-surface soil has high expansion potential. Moisture fluctuations in near-surface expansive soil could cause the soil to expand or contract resulting in movement and potential damage to improvements that overlie them. Furthermore, highly plastic soil tend to creep downslope over-time. Potential causes of moisture fluctuations include drying during construction, and subsequent wetting from rain, capillary rise, landscape irrigation, poor drainage and type of plant selection.

At-grade improvements, including slabs and concrete flatwork, should be designed and constructed to resist the effects of the expansive soil. These effects can be mitigated by moisture conditioning the expansive soil and providing select, non-expansive fill below interior and exterior slabs and supporting foundations below the zone of severe moisture change.

An alternative to importing select fill includes lime treatment of the near surface soil. Lime stabilization of the subgrade of exterior flatwork and pavement may be a cost-effective means of improving on-site soils for use as non-expansive fill.

If the surface soil becomes wet, it may be difficult to compact during the winter unless it is dried. If required, the soil can be mixed with lime to aid in drying and compaction. Lime can also reduce the swell potential and increase the shear strength of the soil; however the amount and type of lime needed should be determined by the contractor and laboratory test results indicating the plasticity index (PI) of the treated soil should be provided.

Expansive soils may also contribute to slope instability in cut or fill slopes exceeding 2:1 (horizontal to vertical). Permanent cut and fill slopes composed of expansive materials should not exceed 3:1 (horizontal to vertical).

6.2 Foundations and Settlement

The proposed building locations are underlain by variable subsurface conditions, with about 1 to 10 feet of expansive soil, which indicates some fill above bedrock

The variable depth to bedrock and thickness of existing expansive soil within the building footprint can result in differential settlement under the building loads. To reduce the potential for differential settlement of foundations and differential movement as a result of wetting and drying cycles resulting from expansive soil, we conclude foundations for the proposed buildings should gain support in the bedrock underlying the expansive soil. Where rock is encountered at or near the subgrade level, the structure can be supported on spread footings bearing in rock. Where the bedrock depth is impractical for shallow foundations, drilled piers extending into rock may be used to support the structure. We anticipate that footings and drilled piers bottomed in rock will settle less than an inch. Because of the presence of highly expansive soil, a void, such as collapsible forms, should be created below the bottom of grade beams to isolate the grade beam from the ground and potential uplift sources.

Approximate foundation zones were developed using the results of our field investigation and are shown on Figure 13. Additional investigation consisting of exploratory pits, or drilled piers can be performed during the initial stages of construction to further define the transition zone between footings and drilled piers (shown in yellow on Figure 13). It is therefore important that the foundation design and construction documents allow for transition from one foundation type to the other as field conditions dictate.

Where edges of the buildings will extend over the existing slopes, we conclude drilled piers should be used to support the building. If a retaining wall is constructed along the edge of the building it should be supported on either shallow footings bottomed in bedrock or drilled piers.

6.3 Slope Stability Analysis

In general, the natural slopes at the site do not appear to exhibit signs of deep-seated landsliding, and no deep-seated slides were noted during our aerial photo review. Two landslide features in the northern half of the property appear to be shallow, with the dormant earth flow likely confined to surficial soils over bedrock. The existing more recent slope stability issues in the south half of the slope appear to have been confined to steeply graded slopes.

The existing steep cut slope behind the existing residential structure and proposed Building B has experienced instability in the past. This slope should be graded to an inclination of 3:1 or flatter. A retaining structure gaining support in the underlying bedrock could be used to reduce the amount of earthwork necessary to flatten the existing slope.

We understand large cuts and fills are proposed. Retaining walls may be needed to provide support at the toe of slopes, where cuts are made into the slope. Retaining walls should be designed for the appropriate earth pressures, which will depend on the slope inclination and backfill material.

For this study, we performed evaluations of the stability of the slopes closest to the proposed buildings. Figures 6 through 9 present idealized subsurface profiles (Section A-A', B-B', C-C' and D-D') of the slopes that we analyzed. The locations of the sections are shown on Figures 2 and 3. The slope stability analyses were performed using the computer program Slope/W (2010), which is a fully integrated slope stability analysis program. Slope/W uses the Morgenstern-Price method to search for the most critical surface and to calculate factor of safety[8].

The engineering properties of the fill, colluvium, landslide deposits and bedrock materials were developed based on the results of our field exploration and laboratory testing programs, and published values for the geologic units from the California Geologic Survey (CGS) Seismic Hazard Zone Report for the San Jose East Quadrangle (California Department of Conservation, 2000). A summary of engineering properties for the different material types used in our slope stability analysis is presented in Table 4.

TABLE 4
Engineering Properties used in Slope Stability Analyses

Material Description	Total Unit Weight (pcf)	Effective Strength Parameters	
		Effective Cohesion, C' (psf)	Effective Internal Friction, ϕ' (degrees)
Fill	125	240	28
Colluvium	125	240	28
Landslide Deposits	125	240	28
Bedrock	131	645	34

8 The factor of safety is the ratio of the resistance to sliding over the slide force. The higher the factor of safety, the more resistance the slope has to failure. Typically, a slope with a static factor of safety greater than 1.5 and a seismic factor of safety of 1.1 with a seismic coefficient of 0.1 to 0.15 is considered stable (CDMG Special Publication 117).

We used a pseudo-static approach to evaluate the seismic slope stability of these subsurface profiles. In this method of analysis, an earthquake is represented by an equivalent horizontal static force. This seismic force is modeled by applying a horizontal ground acceleration (a horizontal seismic coefficient) multiplied by the mass of the potential slide material. The magnitude of this equivalent horizontal seismic coefficient, which takes into account the geometry of the failure plane and average ground acceleration, was estimated to be half of the estimated peak ground acceleration for a given seismic event.

For our analyses we considered the following:

- a peak ground acceleration of 0.6g's from the Maximum Considered Earthquake ground motion (see Section 7.7)
- a moment magnitude 7.3 earthquake on the Hayward fault.

By modifying the horizontal seismic coefficient within each stability run (using the SLOPE/W program) we obtained the magnitude of the horizontal seismic coefficient that corresponds to a seismic factor of safety equal to 1.0. The corresponding horizontal seismic coefficient is referred to as the yield acceleration for that profile. Specifically, we evaluated the yield acceleration for each of the four profiles, where the pseudo-static evaluation resulted in a factor of safety is equal to 1.0.

To evaluate the magnitude of the slope movement, we computed slope deformations during a seismic event using the Makdisi and Seed (1978) simplified method. We determined the yield acceleration needed to lower the factor of safety to 1.0 for each section; the amount of deformation was computed by comparing the yield acceleration with the expected accelerations caused by the earthquake.

The results of our analyses indicate the existing slopes are stable or may exhibit negligible permanent slope displacements upslope of the proposed improvements. However, the slope above proposed Building A could exhibit permanent slope displacements of about 4 inches. The results of our slope stability analyses and seismic slope displacements are summarized in Table 5, below for existing conditions. The location of the critical slope failure surfaces for each of the profiles evaluated are provided in Appendix F.

TABLE 5
Slope Stability Results – Existing Conditions

Section	Static Factor of Safety	Yield Acceleration (g's)	Estimated Deformation (centimeters)
A-A' lower slope	2.70	0.72	<1
A-A' middle slope	5.90	0.93	<1
A-A' upper slope	2.10	0.56	<1
B-B'	2.40	0.45	<1
C-C'	1.48	0.18	10
D-D' lower slope	2.90	0.88	<1
D-D' upper slope	2.20	0.43	<1

Current plans are to grade the proposed building pads and outlying access roads and sidewalks. Grading will consist of cut and fill to level out areas for the proposed buildings. If the development is graded as proposed, the results of our analyses indicate similar results as the existing conditions. The results of our slope stability analyses and seismic slope displacements, which include the proposed grading, are summarized in Table 6. The location of the critical slope failure surfaces for each of the profiles evaluated are provided in Appendix F.

TABLE 6
Slope Stability Results with Proposed Grading

Section	Depth of Cut (feet)	Static Factor of Safety	Yield Acceleration (g's)	Estimated Deformation (centimeters)
A-A' lower slope	0 to 2	3.57	0.74	<1
A-A' middle slope	0 to 2	5.31	0.90	<1
B-B'	0 to 2	1.83	0.43	<1
C-C'	0 to 2	1.48	0.19	10
D-D' lower slope	0 to 2	3.20	0.75	<1
D-D' upper slope	0 to 2	2.21	0.41	<1

Our analysis along Section C-C' indicates a rotation failure is likely. With a rotational failure, the slope will slide along a failure surface and come to rest at a more stable configuration. At Section C-C', we estimate deformations of about 10 cm (4 inches) in a zone of about 50 to 100 feet upslope from proposed Building A. Furthermore deformations associated with slope displacements should decrease with distance from the top of slope. Utilities and flatwork near the failure plane may be affected; however, because Building A is about 50 feet from the toe of slope, slope movement should not adversely impact the building.

6.4 Excavation and Shoring

We understand that portions of the site will be cut into the existing slopes. The existing slope southwest of Building B will be cut into for parking and roadways with a finished elevation at approximately 195 feet, approximately 12 feet below the proposed finished floor elevation at Building B of 207 feet. Additionally, cuts into existing slopes along Dove Hill Road, south of the site will range from approximately 8 to 12 feet below the existing grade at the top of the slope. These excavations will need to be permanently retained.

The soil to be excavated consists predominantly of clay, which can be excavated with conventional earth-moving equipment such as loaders and backhoes. We anticipate that bedrock will be encountered within the excavations, especially at the eastern portion of Building B and the northern portion of Building A. Where bedrock is present within the planned depth of excavation, the contractor will need to select equipment that is capable of excavating and removing blocks of potentially very hard to very strong rock. Excavations deeper than five feet that will be entered by workers should be shored or sloped in accordance with the Occupational Safety and Health Administration (OSHA) standards (29 CFR Part 1926).

If there is insufficient space to slope the sides of the excavations, shoring will be required. Considering the anticipated excavation depths and the expected soil/rock conditions, we conclude that soldier-pile-and-lagging shoring systems are suitable for this project. A soldier-pile-and-lagging system consists of steel soldier beams placed in vertical predrilled holes that are backfilled with concrete and wood lagging between the soldier beams as the excavation proceeds.

Depending on the height of the shoring system, lateral restraint such as tiebacks may be required. Tiebacks will extend significant distances into the soil and rock behind the wall, and if they will be incorporated into a permanent retention system, use of deep foundations, utilities, and trees may need to be restricted or used cautiously in areas behind the wall. Alternatively soil nails could be used for portions of excavations in rock. For permanent retention systems, double-corrosion protection will be required for tiebacks and all other system components.

6.5 Corrosion Potential

Because corrosive soil can adversely affect underground utilities and foundation elements, laboratory testing was performed to evaluate the corrosivity of the near surface soil.

CERCO Analytical performed tests on soil samples to evaluate corrosion potential to buried metals and concrete. The results of the tests are presented in Table 7 and Appendix D.

TABLE 7

Summary of Corrosivity Test Results

Test Boring	Sample Depth (feet)	pH	Sulfates (mg/kg)	Resistivity (ohms-cm)	Redox (mV)	Chlorides (mg/kg)
B-3-1	3	8.26	86	660	340	22
B-1-2	6	8.25	110	780	330	19

Based upon resistivity measurements, the soil samples tested are classified as "corrosive" to buried iron, steel, cast iron, ductile iron, galvanized steel and dielectric coated steel or iron. The chemical analysis indicates reinforced concrete and cement mortar coated steel may be affected by the corrosivity of the soil. To protect reinforcing steel from corrosion, adequate coverage should be provided as required by the building code.

7.0 RECOMMENDATIONS

Recommendations for site preparation, foundation support, floor slabs, retaining walls, seismic design and other issues are presented in the following sections of this report.

7.1 Site Preparation

Demolition in areas to be developed may include removal of existing pavement and underground obstructions, including foundations of former structures. Any vegetation and organic topsoil should be stripped in areas to receive new site improvements. Stripped organic soil can be stockpiled for later use in landscaped areas, if approved by the owner and architect; organic topsoil should not be used as compacted fill.

Excavated asphalt or concrete may be crushed to provide recycled construction materials, including sand, free-draining crushed rock, and Class 2 aggregate base (AB), provided it is acceptable from an environmental standpoint. Where crushed rock will be used beneath vapor retarders and in other applications where free-draining materials are required, it should have no greater than six percent of material passing the 3/8-inch sieve and meet the other requirements presented in Section 7.3. Where recycled Class 2 AB will be used beneath pavements, it should meet requirements of the Caltrans Standard Specifications. Recycled Class 2 AB that does not meet the Caltrans specifications should not be used beneath City streets, but it is acceptable for use as select fill within building pads and beneath concrete flatwork, provided it meets the requirements for select fill as presented later in this section.

Existing underground utilities beneath areas to receive new improvements should be removed or abandoned in-place by filling them with grout. The procedure for in-place abandonment of utilities should be evaluated on a case-by-case basis, and will depend on location of utilities relative to new improvements. However, in general, existing utilities within four feet of final grades should be removed, and the resulting excavation should be properly backfilled based on the recommendations presented in this section.

Prior to placing fill, the subgrade exposed after stripping and site clearing, as well as other portions of the site that will receive new fill or site improvements, should be scarified to a depth of at least eight inches, moisture-conditioned to above optimum moisture content, and compacted to at least 90 percent relative compaction[9]. An exception to this general procedure is within any proposed vehicle pavement areas, where the upper six inches of the pavement subgrade should be compacted to at least 95 percent relative compaction regardless of expansion potential.

Heavy construction equipment should not be allowed directly on the final subgrade. The clay exposed at the foundation level may be susceptible to disturbance under construction equipment loads. If the subgrade is disturbed during the rainy season, it may be necessary to

[9] Relative compaction refers to the in-place dry density of soil expressed as a percentage of the maximum dry density of the same material, as determined by the ASTM D1557-07 laboratory compaction procedure.

place a minimum 12-inch working pad consisting of crushed rock on top of the subgrade or lime treating the upper 12 inches of the subgrade to winterize it. Any select fill placed during grading should meet the following criteria:

- be free of organic matter
- contain no rocks or lumps larger than three inches in greatest dimension
- have a low expansion potential (defined by a liquid limit of less than 40 and plasticity index lower than 12)
- have a low corrosion potential[10]
- be approved by the geotechnical engineer.

All fill placed beneath improvements should meet the criteria for select fill previously discussed in this section. Alternatively native soil may be used, if it is lime treated and meets the liquid limit and plasticity requirements for select fill. All select fill should be moisture-conditioned to above optimum moisture content, placed in horizontal lifts not exceeding eight inches in loose thickness, and be compacted to at least 90 percent relative compaction, except for fill that is placed within the proposed pavement areas. In these situations, the upper six inches of the soil subgrade, all select fill and aggregate baserock materials should be compacted to at least 95 percent relative compaction. Where used, sand containing less than 10 percent fines (particles passing the No. 200 sieve) should also be compacted to at least 95 percent relative compaction. Samples of on-site and proposed import fill materials should be submitted to the geotechnical engineer for approval at least three business days prior to use at the site.

7.1.1 Lime Treatment

To winterize the site, the upper 12 inches of the existing surface soil in building pads may be lime treated. We recommend that at least 5 percent lime by weight of the soil be used to treat at least the upper 12 inches of native soil for at-grade structures; additional thickness can be added to meet the select fill requirements, if needed. The lime treatment should extend at least five feet beyond building footprints except where hardscape areas are planned; landscape areas should not be lime treated because the lime treated soil may make it difficult for the

10 Low corrosion potential is defined as a minimum resistivity of 2,000 ohms-cm and maximum sulfate and chloride concentrations of 250 parts per million.

plants to survive. If it is intended to treat the native soil with lime to meet the selected fill criteria, the contractor should evaluate the amount and type of lime necessary to reduce the PI and provide confirming laboratory test results.

7.1.2 Quality Control of Lime Treatment

Lime treatment of fine-grained soils generally includes site preparation, application of lime, mixing, compaction, and curing of the lime treated soil. Field quality control measures should include checking the depth of lime treatment, degree of pulverization, lime spread rate measurement, lime content measurement, and moisture content and density measurements, and mixing efficiency. Quality control may also include laboratory tests for unconfined compressive strength tests on representative samples.

The lime treatment process should be designed by a contractor specializing in its use and who is experienced in the application of lime in similar soil conditions. Based on our experience with lime treatment, we judge that the specialty contractor should be able to treat the highly expansive on-site material to produce a non-expansive fill for building subgrade.

If the lime treatment is selected, we recommend that the specialty contractor prepare a treatment specification for our review prior to construction.

7.1.3 Fill Slopes

Where fill is planned along existing slopes, such as behind and around new retaining walls, the fill should be keyed and benched into the slope to reduce the potential for differential settlement and movement of the fill. Prior to placement of fill, the exposed subgrade should be scarified, moisture-conditioned, and compacted as previously discussed in Section 7.1. If the final fill surface will be sloped, we recommend the fill slope be overbuilt by placing and compacting horizontal lifts of fill as previously described in Section 7.1. Subsequently, the fill slope should be cut back to achieve the proper slope inclination.

We recommend that fill slopes be designed to have a maximum slope inclination of 3:1 (horizontal to vertical). At the toe of the proposed fill slope, a keyway should be installed to interconnect the new fill material into the existing strata. The keyway should be at least five feet wide at the base and extend at least two feet into competent soil or rock or at least 15 percent of the overall slope height, whichever is greater. The side slopes of the keyways should not be steeper than 1:1.

Where new fill is placed over existing slopes that are steeper than 5:1, the fill should be benched as the fill operation proceeds upslope. These benches will provide horizontal surfaces for the placement and compaction of the fill and reduce the effects of downward creep of the soil. Benches should be a maximum of five feet high and should expose competent soil or rock along the base of the bench.

The face of fill slopes should be planted with deep-rooted vegetation and covered by an erosion control blanket to reduce the potential for surface erosion. We recommend using a biodegradable erosion control blanket (North American Green SC150 or equivalent erosion control material that is acceptable to the Geotechnical Engineer) on the slope face that has been disturbed by grading. The biodegradable erosion control blanket should be installed in accordance with the manufacturer's specifications.

To limit the concentration of surface water on slopes, areas upslope of the cut or fill slope should be graded to drain away from these slopes. As an alternative, V-ditches or curbs and gutters should be placed at the crest of these slopes to capture and control surface water and re-direct it away from the slope.

7.1.4 Cut Slopes

We recommend that temporary cut slopes in fill or native soil over five feet high be graded no steeper than 1:1. Temporary cuts in bedrock may be made vertical; however, the height of any vertical segment should not exceed six feet unless shoring is used. If poor rock quality or adverse bedding is present, cuts in rock should be flattened and/or retained using temporary shoring. The safety of workers and equipment in or near excavations is the responsibility of the contractor. The contractor should be familiar with the most recent OSHA Trench and Excavation Safety standards.

If cut slopes will be permanent, the native soil should be graded no steeper than 3:1 (horizontal to vertical). Unretained cuts in bedrock may be graded as steep as 2:1, depending on the rock fracturing, hardness, and weathering. If poor rock quality or adverse bedding is present, rock slopes should be flattened and/or retained using soil nails.

We should review plans for temporary and permanent cut slopes prior to construction. During construction, we should observe cut slopes to verify the inclinations are appropriate for the conditions encountered. It is the responsibility of the contractor to maintain safe and stable slopes during construction. During wet weather, runoff should be prevented from running down slopes and from entering excavations, especially off of the level recreation area.

7.2 Foundations

We recommend the proposed buildings be supported on spread footings where bedrock is encountered at or near the subgrade level, and on drilled piers extending into bedrock where bedrock is too deep to be practically reached by the footings. The following sections present our recommendations for footing and pier foundations.

7.2.1 Spread Footings

Where it is practical to reach bedrock by excavating for the footings (we estimate this to be a depth of up to about 5 feet), the proposed structures can be supported on spread footings. Footings should be embedded at least three feet below the lowest adjacent grade where soil is present and a minimum of one foot into bedrock. Footings bearing on bedrock may be designed for a maximum allowable bearing pressure of 10,000 pounds per square foot (psf) for dead plus live loads, which can be increased by one-third for total loads, including wind and/or seismic loads. These values include factors of safety of at least 2.0 and 1.5 for dead plus live loads and total loads, respectively.

Lateral loads on footings can be resisted by a combination of passive resistance acting against the vertical faces of the footings and friction along the bases of the footings. Passive resistance may be calculated using uniform pressures of 2,000 psf for clay and 3,000 psf for bedrock. The upper foot of soil or rock should be ignored unless it is confined by slabs or pavement. Frictional resistance at the base of the footings should be computed using a friction coefficient of 0.4. These values include a factor of safety of about 1.5. Passive resistance should not be used for foundation elements on existing slopes unless the face of the footing is at least 7 feet from the slope face, measured horizontally.

The footing excavations should be free of standing water, debris, and disturbed materials prior to placing concrete. If disturbed, highly weathered, or decomposed bedrock is encountered at the bottom of footing excavations, the excavations should be deepened to expose more competent bedrock, as determined by the geotechnical engineer. We should check foundation excavations prior to placement of reinforcing steel to confirm suitable bearing material is present.

If overexcavation is required to reach bedrock or to remove unsuitable rock, the overexcavation may be backfilled to the design bottom of footing using lean concrete. The lean concrete should have a minimum unconfined compressive strength of 50 pounds per square inch.

7.2.2 Drilled Piers

Drilled piers bottomed in bedrock should be designed to derive their axial capacity from skin friction. Drilled piers will gain support in skin friction in the clay layer and rock. Because of the presence of near surface expansive soil, the skin friction in the upper five feet should be ignored. Below this depth, we recommend an ultimate skin friction of 1,000 pounds per square foot (psf) in the clay and 2,000 psf in rock. Piers should extend at least 5-feet into rock. Appropriate factors of safety should be used.

Piers should be spaced at least three diameters on center to avoid vertical capacity reduction due to group effects.

Piers will provide lateral resistance from passive pressure acting on the upper portion of the piers and from their structural rigidity. Lateral resistance of piers will depend on the pier diameter, pier head condition (restrained or unrestrained), allowable deflection of the pier top, and the bending moment resistance of the piers. We have performed lateral load analyses for isolated, 18- and 24-inch-diameter piers for a deflection of 0.5 inch at the pier head. We assumed that the pier head is at the ground surface and on level ground. The results of our analyses are presented in Table 8.

TABLE 8

Lateral Capacities for ½-Inch Pile Top Deflection

Pier Diameter (inches)	Free-Head Condition				Fixed-Head Condition			
	Lateral Capacity (kips)	Maximum Moment (kip-ft)	Depth To Maximum Moment (ft)	Depth to Zero Deflection (ft)	Lateral Capacity (kips)	Maximum Moment (kip-ft)	Depth To Maximum Moment (ft)	Depth to Zero Deflection (ft)
18	35	110	6	14	70	290	0	12
24	60	260	8	16	130	650	0	14

Structural loads are presently not available for the proposed development. Once building loads and grading plans are available, the settlement estimates can be refined. However, properly constructed drilled piers should have a total settlement less than one inch, with less than ½ inch of differential settlements between columns, under static conditions.

Drilled piers should be installed by a qualified contractor with demonstrated experience in this type of foundation. Concrete placement should start upon completion of the drilling and clean out. Concrete should be placed from the bottom up in a single operation using a tremie and/or a pumper pipe. The tremie pipe should be maintained at least 5 feet below the upper surface of the concrete during casting of the piers.

Piers should be connect by grade beams. Because of the presence of expansive soil, a void, such as a collapsible form, should be present below the bottom of grade beams to isolate the grade beam from the ground and potential uplift forces. Additional grade beams should be continuous around the building perimeter to reduce the migration of water beneath the building, which could cause heaving of the expansive soil.

7.3 Floor Slab

The buildings will be supported on a combination of spread footings bearing on rock drilled drilled piers; therefore, settlement of the building should be negligible; however, moisture fluctuations in near-surface expansive soil could cause the soil to expand or contract. We recommend a 30-inch-thick layer of select fill be used beneath a slab on grade floor. The selected fill may be eliminated, if structural slab spanning between pier caps is used in lieu of a slab on grade.

Since moisture fluctuations in near-surface expansive soil could cause the soil to expand or contract, the entrances to the building should be designed as hinged slabs. The hinge slab should be dowelled into the building to prevent from moving upward if soil swells.

Moisture is likely to condense on the underside of the ground floor slabs, even though they will be above the design groundwater level. Consequently, a moisture barrier should be considered if movement of water vapor through the slabs would be detrimental to its intended use. A typical moisture barrier consists of a capillary moisture break and a water vapor retarder.

The capillary moisture break should consist of at least four inches of clean, free-draining gravel or crushed rock. The vapor retarder should meet the requirements for Class C vapor retarders stated in ASTM E1745-97. The vapor retarder should be placed in accordance with the requirements of ASTM E1643-98. These requirements include overlapping seams by six inches, taping seams, and sealing penetrations in the vapor retarder. The vapor retarder should be covered with two inches of sand to aid in curing the concrete and to protect the vapor retarder during slab construction. The capillary break and sand should not be counted as part of the select fill. The particle size of the gravel/crushed rock and sand should meet the gradation requirements presented in Table 9.

TABLE 9
Gradation Requirements for Capillary Moisture Break

Sieve Size	Percentage Passing Sieve
Gravel or Crushed Rock	
1 inch	90 – 100
3/4 inch	30 – 100
1/2 inch	5 – 25
3/8 inch	0 – 6
Sand	
No. 4	100
No. 200	0 – 5

The sand overlying the membrane should be dry at the time concrete is cast. Excess water trapped in the sand could eventually be transmitted as vapor through the slab. If rain is forecast prior to pouring the slab, the sand should be covered with plastic sheeting to avoid wetting. If the sand becomes wet, concrete should not be placed until the sand has been dried or replaced.

Concrete mixes with high water/cement (w/c) ratios result in excess water in the concrete, which increases the cure time and results in excessive vapor transmission through the slab. Therefore, concrete for the floor slab should have a low w/c ratio - less than 0.50. If approved by the project structural engineer, the sand can be eliminated and the concrete can be placed directly over the vapor retarder, provided the w/c ratio of the concrete does not exceed 0.45 and water is not added in the field. If necessary, workability should be increased by adding plasticizers. In addition, the slab should be properly cured.

Before the floor covering is placed, the contractor should check that the concrete surface and the moisture emission levels (if emission testing is required) meet the manufacturer's requirements.

7.4 Shoring Design

Excavations to construct retaining walls the site may be open cut and/or temporarily shored. Excavations that will be deeper than five feet and will be entered by workers should be shored or sloped in accordance with CAL-OSHA standards (29 CFR Part 1926). It is the responsibility of the contractor to determine the safe excavation slopes; however, we recommend temporary cuts greater than 5 feet be no steeper than 1:1 (horizontal:vertical).

If the planned excavations cannot be sloped because of space limitations, shoring will be required to retain the excavation sides. We estimate excavations may be as deep as about 8 to 10 feet. If the shoring will be used as part of a permanent retention system, all system components should be double-corrosion protected and the shoring design should incorporate a factor of safety consistent with permanent structures.

Cantilevered shoring should be designed for the active earth pressures presented in Table 10 (see section 7.5). These values are considered appropriate for an active condition, which assumes that some movement of the supported soil is tolerable. For intermediate slope inclinations, the values presented in Table 10 may be interpreted. If movement of the soil is not acceptable, then the at-rest pressures presented in Table 10 should be used. For shoring consisting of soldier beams and lagging, the active and at-rest earth pressures should be assumed to act over the full width of the shoring above the excavation and over one soldier beam width below the excavation.

Passive resistance can be computed using a uniform pressure of 2,000 psf for clay and 3,000 psf for bedrock. These values include a factor of safety of about 1.5 for temporary shoring design. For beams spaced at least three shaft diameters, center-to-center, the passive resistances can be assumed to act over three soldier beam[11] widths.

The soldier beams should extend below the excavation bottom a minimum of five feet and be sufficient to achieve lateral stability and resist the downward loading of the tiebacks.

If traffic occurs within 10 feet of the shoring, a uniform surcharge load of 100 psf should be added to the design. An increase in lateral design pressure for the shoring may be required where heavy construction equipment or stockpiled materials are within a distance equal to the shoring depth. Construction equipment should not be allowed within five feet from the edge of the excavation unless the shoring is specifically designed for the appropriate surcharge. The increase in pressure should be computed after the surcharge loads are known. The anticipated deflections of the shoring system should be estimated to check if they are acceptable.

The shoring system should be designed by a licensed civil engineer experienced in the design of retaining systems, and installed by an experienced shoring specialty contractor. The shoring engineer should be responsible for the design of temporary shoring in accordance with applicable regulatory requirements. Control of ground movement will depend as much on the timeliness of installation of lateral restraint as on the design. We should review the shoring plans and a representative from our office should observe the installation of the shoring. If temporary tiebacks are need to restrain the shoring, we can provide supplemental recommendations.

7.5 Retaining Wall Design

If the walls support expansive soil, we recommend designing retaining/below grade walls for at-rest lateral pressures presented in Table 10. Because the site is in a seismically active area, the design should also be checked for seismic conditions. Under seismic loading conditions, there will be an added seismic increment that should be added to active earth pressures (Sitar et al. 2012). We used the procedures outlined in Sitar et al. (2012) and the peak ground acceleration based on the DE ground motion level (see Section 7.7) to compute the seismic

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11 The soldier beam width is defined as the diameter of the drilled hole for beams backfilled with structural concrete with an unconfined compressive strength of at least 50 pounds per square inch (psi).

pressure increment. We recommend the walls be designed for the more critical of at-rest pressures or total pressure (active plus seismic pressure increment). Below-grade walls backfilled with native clay should be designed for the equivalent fluid weights and pressures presented in Table 10.

TABLE 10

Earth Pressures to Wall with Native Soil Backfill (Drained Conditions)

Slope inclination	Static Conditions		Seismic Conditions[1]
	Unrestrained Walls – Active (pcf[3])	Restrained Walls – At-rest (pcf)	Total Pressure – Active Plus Seismic Pressure Increment (pcf)
Level[2]	45	65	70
3:1	60	80	85
2:1	70	100	95

Notes:

1. For seismic conditions, the more critical condition of either at-rest pressure or active pressure plus a seismic pressure increment should be checked.
2. Applicable to walls that are backdrained to prevent the buildup of hydrostatic pressure.
3. pcf = pounds per cubic foot

If open cuts are made for the below-grade walls and select fill meeting the requirements discussed in Section 7.1 is used as backfill, then the walls may be designed using the earth pressures presented in Table 11.

TABLE 11

Below-Grade Wall Design Earth Pressures with Select Fill Backfill (Drained Conditions)

Slope inclination	Static Conditions		Seismic Conditions[1]
	Unrestrained Walls – Active (pcf[3])	Restrained Walls – At-rest (pcf)	Total Pressure – Active Plus Seismic Pressure Increment (pcf)
Level[2]	35	55	60
3:1	45	65	70
2:1	55	75	80

Notes:

1. For seismic conditions, the more critical condition of either at-rest pressure or active pressure plus a seismic pressure increment should be checked.
2. Applicable to walls that are backdrained to prevent the buildup of hydrostatic pressure.
3. pcf = pounds per cubic foot

If surcharge loads occur above an imaginary 45-degree line projected up from the bottom of a retaining wall, a surcharge pressure should be included in the wall design. If this condition exists, we should be consulted to estimate the added pressure on a case-by-case basis.

Where truck traffic will pass within 10 feet of retaining walls, temporary traffic loads should be considered in the design of the walls. Traffic loads may be modeled by a uniform pressure of 100 pounds per square foot applied in the upper 10 feet of the walls.

The lateral earth pressures recommended are for the drained condition and are applicable to walls that are backdrained to prevent the buildup of hydrostatic pressure. One acceptable method for backdraining the walls is to place a prefabricated drainage panel against the back side of the wall. The drainage panel should extend down to the base of the wall or the design groundwater elevation (design groundwater elevations are discussed in Section 6.3) to a perforated PVC 20 collector pipe. The pipe should be surrounded on all sides by at least four inches of Caltrans Class 2 permeable material (Caltrans Standard Specifications Section 68-1.025). We should check the manufacturer's specifications regarding the proposed prefabricated drainage panel material to verify that it is appropriate for the intended use. An acceptable alternative is to backdrain the wall with Caltrans Class 2 material at least one foot wide, extending down to the base of the wall. A perforated PVC pipe should be placed at the bottom of the gravel, as described for the first alternative. The pipe in either alternative should be sloped to drain into an appropriate outlet. We should check the manufacturer's specifications for the proposed drainage panel material to verify it is appropriate for its intended use.

Wall backfill should be compacted to at least 90 percent relative compaction using light compaction equipment. Wall backfill with less than 10 percent fines, or deeper than five feet, should be compacted to at least 95 percent relative compaction for its entirety. If heavy equipment is used, the wall should be appropriately designed to withstand loads exerted by the equipment and/or temporarily braced.

7.6 Pavement Sections

The following subsections present recommendations for asphalt pavement and concrete pavement sections.

7.6.1 Asphalt Pavements

The State of California flexible pavement design method was used to develop the recommended asphalt concrete pavement sections. We expect the final soil subgrade in asphalt-paved areas will generally consist of the on-site clay soil. We selected an R-value of 5 for design to account for the highly expansive soil.

For our calculations, we assumed a Traffic Index (TI) of 4.5 for automobile parking areas with occasional trucks, and 6 and 7 for driveways and truck-use areas; these TIs should be confirmed by the project civil engineer. Table 9 presents our recommendations for asphalt pavement sections.

TABLE 9
Pavement Section Design

TI	Asphaltic Concrete (inches)	Class 2 Aggregate Base R = 78 (inches)
4.5	3.5	7.0
6	5.0	10.0
7	6.0	12.0

Pavement components should conform to the current Caltrans Standard Specifications. The upper six inches of the soil subgrade in pavement areas should be moisture-conditioned to above optimum and compacted to at least 95 percent relative compaction and rolled to provide a smooth non-yielding surface. Aggregate base should be compacted to at least 95 percent relative compaction.

Subdrains should be installed along the backsides of curbs adjacent to landscaped areas to reduce infiltration of irrigation water beneath the pavement.

7.6.2 Concrete Pavements and Exterior Slabs

Differential ground movement due to expansive soil and settlement will tend to distort and crack the pavements and exterior improvements such as courtyards and sidewalks. Periodic repairs and replacement of exterior improvements should be expected during the life of the project. Mastic joints or other positive separations should be provided to permit any differential movements between exterior slabs and the buildings.

We recommend exterior concrete flatwork be underlain by at least 4-inches of Class 2 AB and 12 inches of select fill. Where select fill will need to be placed, the subgrade should be scarified 12-inches, moisture conditioned to at least three percent above optimum, compacted to at least 88 percent relative compaction to provide a smooth, non-yielding surface. Class 2 AB should be compacted to at least 95 percent relative compaction.

Where rigid pavement is required, for loading and service areas, we recommend a minimum of six inches of concrete for medium traffic and a minimum of eight inches of concrete for heavy traffic. The upper six inches of subgrade should be compacted to at least 95 percent relative compaction and should provide a smooth, non-yielding surface. Loading and service areas should be underlain by at least six inches of Class 2 aggregate base compacted to 95 percent relative compaction and provide a smooth, non-yielding surface. Aggregate base material should conform to the current State of California Department of Transportation (Caltrans) Standard Specifications.

7.7 2013 CBC Mapped Values

For seismic design in accordance with the provisions of 2013 California Building Code/ASCE 7-10, we recommend the following:

- Risk Targeted Maximum Considered Earthquake (MCE_R) S_s and S_1 of 1.5g and 0.6g, respectively

- Site Class C
- Site Coefficients F_A and F_V of 1.0 and 1.3
- MCE_R spectral response acceleration parameters at short periods, S_{MS}, and at one-second period, S_{M1}, of 1.5g and 0.78g, respectively
- Design Earthquake (DE) spectral response acceleration parameters at short period, S_{DS}, and at one-second period, S_{D1}, of 1.0g and 0.52g, respectively
- Peak ground acceleration, PGA_M of 0.5g

7.8 Utilities and Utility Backfill

The corrosivity results provided in Appendix D of this report should be reviewed and corrosion protection measures used if needed. We recommend a corrosion engineer be retained when detailed corrosion protection recommendations are needed.

Utility trenches should be excavated a minimum of four inches below the bottom of pipes or conduits and have clearances of at least four inches on both sides. Where necessary, trench excavations should be shored and braced to prevent cave-ins and/or in accordance with safety regulations. If trenches extend below the groundwater level, it will be necessary to temporarily dewater them to allow for placement of the pipe and/or conduits and backfill.

To provide uniform support, pipes or conduits should be bedded on a minimum of four inches of sand or fine gravel. After pipes and conduits are tested, inspected (if required), and approved, they should be covered to a depth of six inches with sand or fine gravel, which should then be mechanically tamped to at least 90 percent relative compaction. If fill with less than 10 percent fines is used, the entire depth of the fill should be compacted to at least 95 percent relative compaction. Jetting of trench backfill should not be permitted. Special care should be taken when backfilling utility trenches in pavement areas. Poor compaction may cause excessive settlements resulting in damage to the pavement section.

Where utility trenches backfilled with sand or gravel enter the building pads, an impermeable plug consisting of native clay or lean concrete, at least five feet in length, should be installed at the building line. Further, where sand- or gravel-backfilled trenches cross planter areas and pass below asphalt or concrete pavements, a similar plug should be placed at the edge of the pavement. The purpose of these plugs is to reduce the potential for water to become trapped in trenches beneath the building or pavements. This trapped water can cause heaving of soils beneath slabs and softening of subgrade soil beneath pavements.

7.9 Site Drainage

Positive surface drainage should be provided around the buildings to direct surface water away from the building foundations. To reduce the potential for water ponding adjacent to the buildings, we recommend the ground surface within a horizontal distance of five feet from the buildings be designed to slope down and away from the buildings with a surface gradient of at least two percent in unpaved areas and one percent in paved areas. In addition, roof downspouts should be discharged into controlled drainage facilities to keep the water away from the foundations.

7.10 Landscaping

The use of water-intensive landscaping around the perimeter of the buildings should be avoided to reduce the amount of water introduced to the subgrade. Irrigation of landscaping around the buildings should be limited to drip or bubbler-type systems. Trees with large roots or have high water demand should also be avoided since they can dry out the soil beneath foundations and cause settlement. The purpose of these recommendations is to avoid large differential moisture changes adjacent to the foundations, which have been known to cause significant differential movement over short horizontal distances in expansive soil, resulting in cracking of slabs and architectural damage.

To reduce the potential for irrigation water entering the pavement section, vertical curbs adjacent to landscaped areas should extend through any aggregate base and at least six inches into the underlying soil. In heavily watered areas, such as lawns, it may also be necessary to install a subdrain behind the curb to intercept excess irrigation water.

7.11 Bioretention Systems

Bioretention areas are landscaping features used to treat stormwater runoff within a development site. They are commonly located in parking lot islands and landscape areas. Surface runoff is directed into shallow, landscaped depressions, which usually include mulch and a prepared soil mix. Typically, the filtered runoff is collected in a perforated underdrain beneath the bioretention system and returned to the storm drain system. For larger storms, runoff is generally diverted past the bioretention areas to the storm drain system.

The soil within a bioretention system should typically have an infiltration rate sufficient to draw down any pooled water within 48 hours after a storm event. Based on the "Bioretention Manual" prepared by The Prince George's County (2007), the infiltration rate of the bioretention soil is recommended to exceed ½ inch per hour; cohesionless soils like sand meet this criterion. Cohesive soils like clay and silts do not meet the infiltration rate requirement and are considered unsuitable in a bioretention system, particularly when they are expansive. For areas where there are unsuitable in-situ soils, the bioretention system can be created by importing a suitable soil mix and providing an underdrain. Based on our observation of the soil at the site, the existing fill and in-situ clays are relatively impervious and do not meet the infiltration rate requirements. The bioretention system will need to be constructed with suitable imported soil and include an underdrain system.

Underdrains are typically at the invert of the bioretention system to intercept water that does not infiltrate into the surrounding soils. Underdrains consist of a perforated PVC pipe surrounded by two to three inches of Class 2 Permeable material (Caltrans Standard Specifications Section 68-2.02F(3)). The perforated PVC pipe cross-section area should be determined based on the desired hydraulic conductivity of the underdrain. The PVC pipe should be bedded on two to three inches of the Class 2 Permeable material.

Because of the presence of near surface expansive soil, bioretention systems should be set back a minimum of five feet from building foundations, slabs, concrete flatwork or pavements. Overflow from bioretention areas should be directed to the storm drain system away from building foundations and slabs.

Typically, the bottom of the bioretention system is recommended to be a minimum of two feet or more above the groundwater table.

8.0 GEOTECHNICAL SERVICES DURING CONSTRUCTION

Prior to construction, we should review the project plans and specifications to check their conformance with the intent of our recommendations. During construction, our field engineer should provide on-site observation and testing during site preparation, grading, placement and compaction of fill, and installation of building foundations. These observations will allow us to compare actual with anticipated soil conditions and to check that the contractor's work conforms to the geotechnical aspects of the plans and specifications.

9.0 LIMITATIONS

The conclusions and recommendations presented in this report apply to the site and construction conditions as we have described them and are the result of engineering studies and our interpretations of the existing geotechnical conditions. Actual subsurface conditions may vary. If any variations or undesirable conditions are encountered during construction, or if the proposed construction will differ from that described in this report, Langan Treadwell Rollo should be notified so that supplemental recommendations can be developed.

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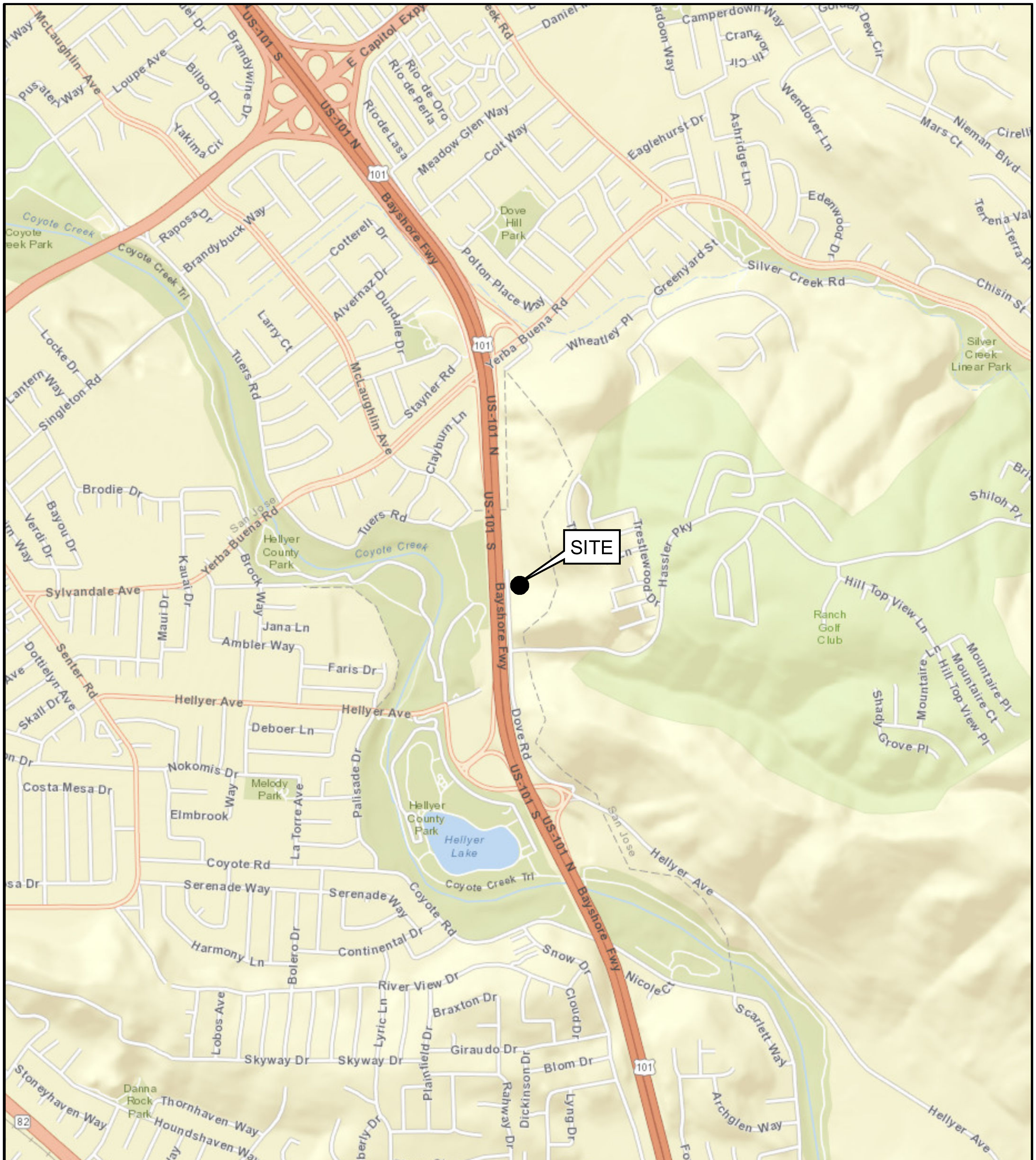
FIGURES



NOTES:

World street basemap is provided through Langan's Esri ArcGIS software licensing and ArcGIS online.
Credits: Sources: Esri, DeLorme, NAVTEQ, USGS, Intermap, iPC, NRCAN.

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Feet

N
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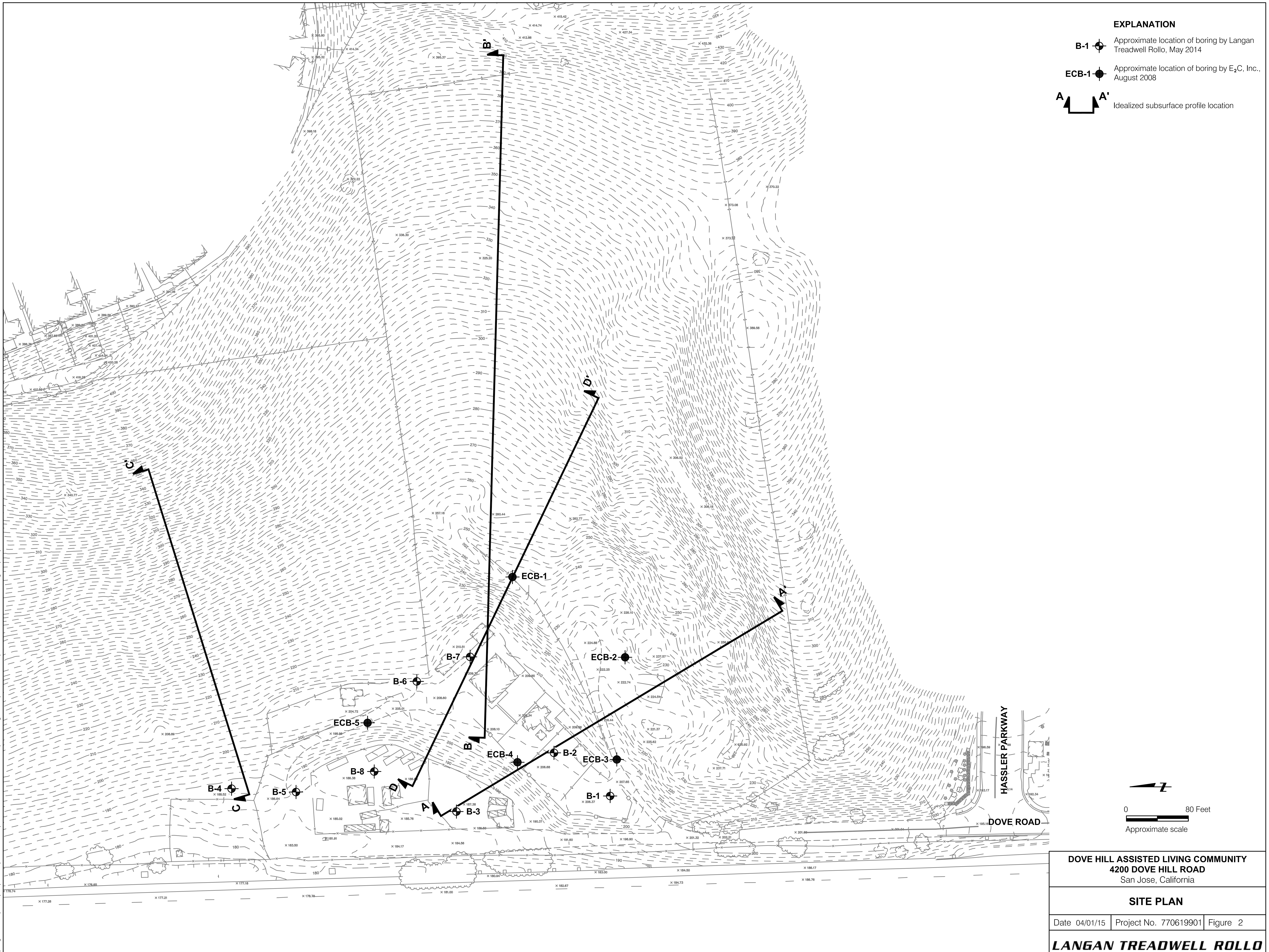
DOVE HILL ASSISTED LIVING COMMUNITY
4200 DOVE HILL ROAD
San Jose, California

SITE LOCATION MAP

LANGAN TREADWELL ROLLO

Date 04/01/15	Project No. 770619901	Figure 1

Path: \\langan.com\data\SJ\data9\770619901\ArcGIS\ArcMap_Documents\Site Location Map.mxd User: cyoung



DOVE HILL ASSISTED LIVING COMMUNITY
4200 DOVE HILL ROAD
San Jose, California

SITE PLAN

Date 04/01/15	Project No. 770619901	Figure 2

LANGAN TREADWELL ROLLO

Reference: Base map from a drawing title "--," by --, dated --.

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DOVE HILL ASSISTED LIVING COMMUNITY
4200 DOVE HILL ROAD
San Jose, California

SITE PLAN WITH
PROPOSED DEVELOPMENT

Date 05/26/15	Project No. 770619901	Figure 3

LANGAN TREADWELL ROLLO

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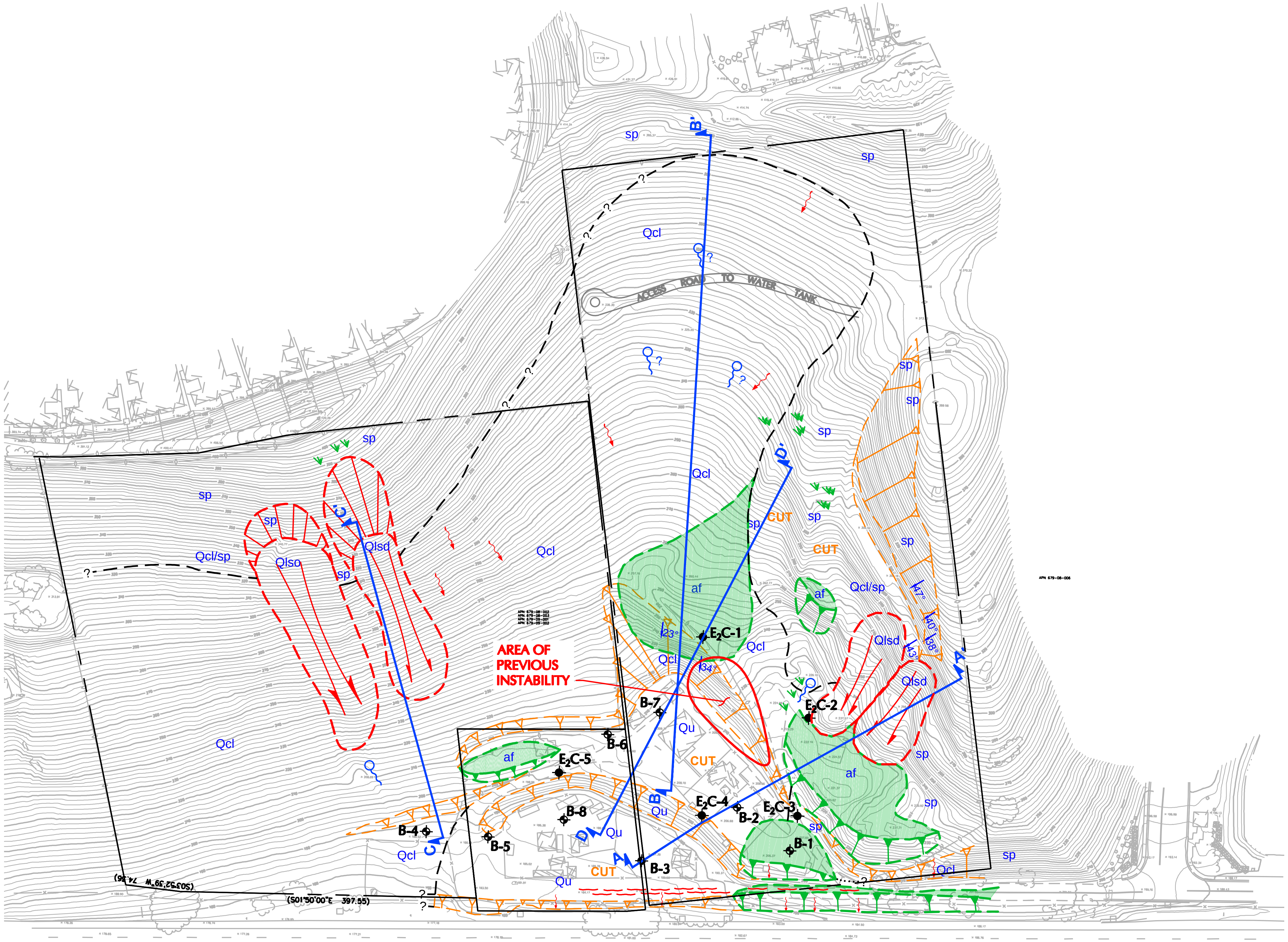


EXPLANATION

Geologic Units

af Artificial fill, shaded for definition

Qu Undifferentiated surficial deposits

Qls Landslide deposit

Qcl Colluvium

sp Serpentinite and serpentinized harzburgite and dunite

Key

Geologic contact; solid where certain, dashed where approximate, dotted where concealed, queried where uncertain

Fill/ Cut slope

Landslide, scarp area hachured, arrow in direction of downslope movement, conpressional toe with debris pushing over the original ground surface by a dash and semi-circle symbol;
Qlsd - dormant, Qlso - old

35° Slope inclination, as measured in field, with angle in degrees

Surficial soil creep, disturbed or hummocky surface

C C' Geologic cross section

B-3 Langan Treadwell Rollo exploratory boring

E_2C-1 Borings by E_2C Consultants

Phreatophytes

Distress cracks

Spring or seep, queried where uncertain

N

0 150 Feet

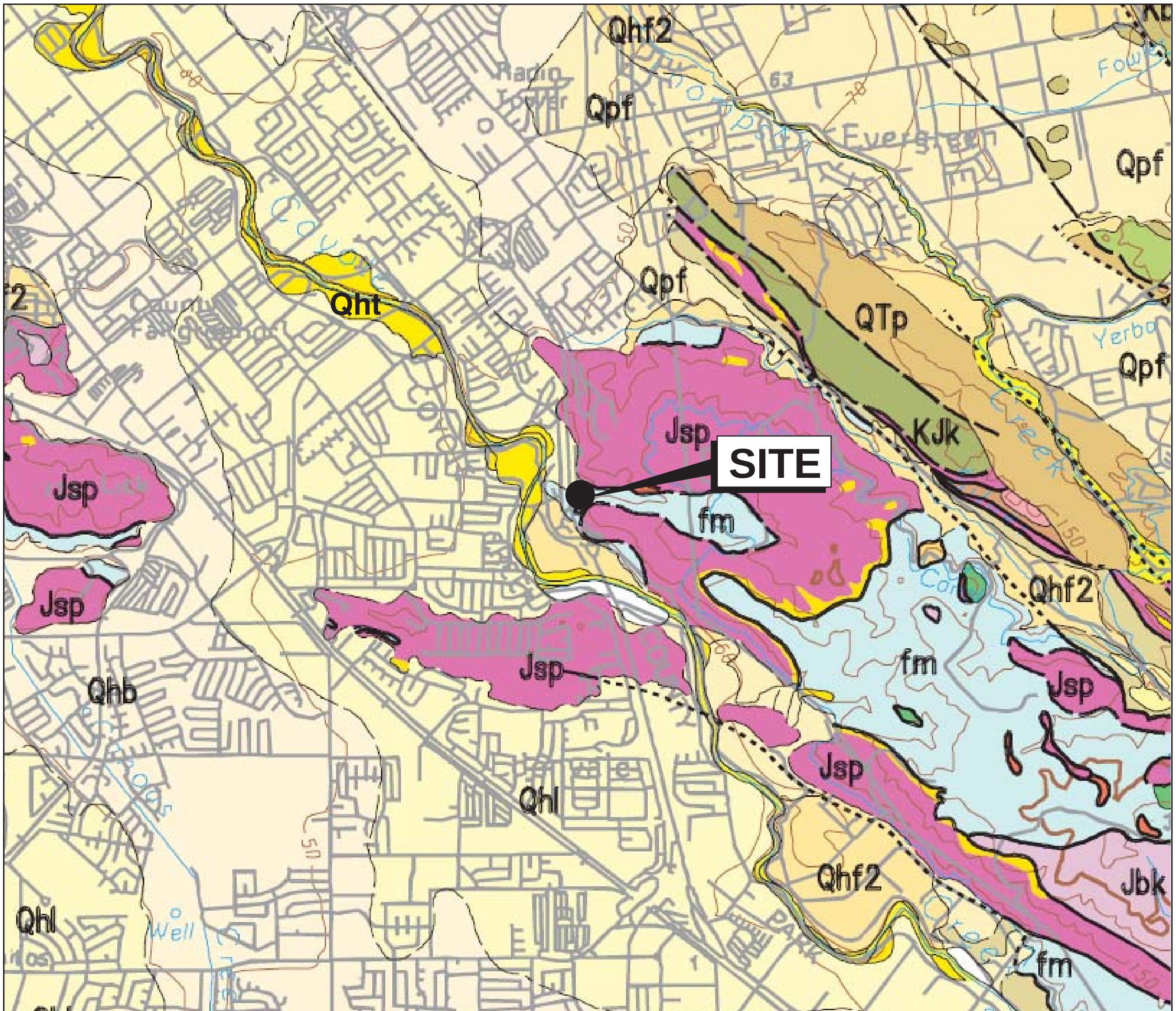
Approximate scale

DOVE HILL ASSISTED LIVING COMMUNITY
4200 Dove Hill Road
San Jose, California

ENGINEERING GEOLOGIC MAP

Date 05/26/15	Project No. 770619901	Figure 4

LANGAN TREADWELL ROLLO



Base map: Preliminary Geologic Map of the San jose 30 x 60-minute Quadrangle, California, by R. Graymer, 1999

EXPLANATION

Qhl	Levee deposits (Holocene)
Qhb	Basin deposits (Holocene)
Qht	Stream Terrace deposits (Holocene)
Qhf2	Alluvial Fan deposits Older (Holocene)
Qpf	Alluvial Fan deposits (Upper Pliestocene)
KJk	Knokville Formation (Lower Cretaceouse and Upper Jurassic)
Jsp	Serpentinized Harzburgite and dunite (Jurassic)
fm	Melange (Lower Tertiary and Upper Cretaceous
QTp	Packwood Gravels (Pleistocene and Pliocene)

Geologic contact
dashed where approximate
and dotted where concealed

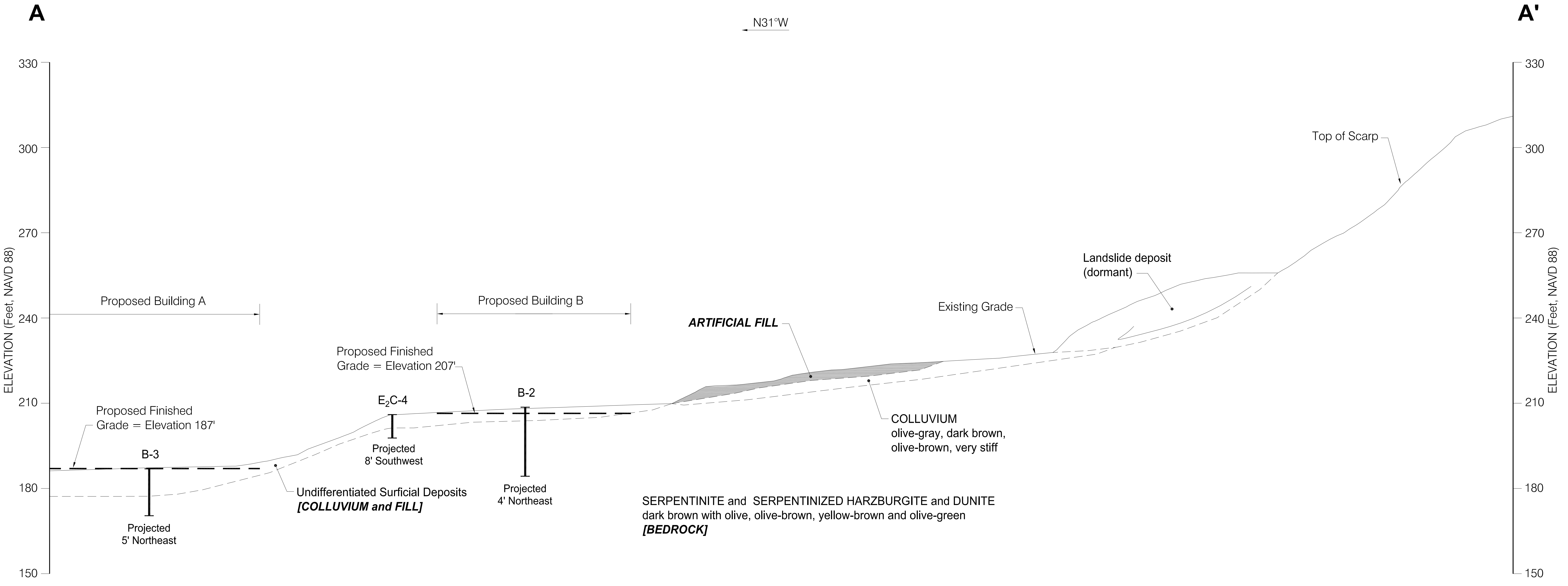
0 4000 Feet
Approximate scale

DOVE HILL ASSISTED LIVING COMMUNITY
4200 DOVE HILL ROAD
San Jose, California

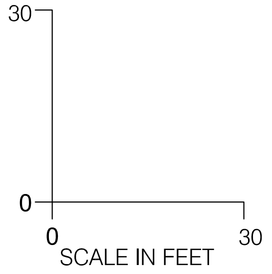
LANGAN TREADWELL ROLLO

REGIONAL GEOLOGIC MAP

Date 05/26/15p	Project No. 770619901	Figure 5



Notes:
1. The above profile represents a generalized subsurface cross section interpreted from widely spaced borings. Earth materials may vary in type, strength, and other important properties between points of exploration.



DOVE HILL ASSISTED LIVING COMMUNITY
4200 DOVE HILL ROAD
San Jose, California

IDEALIZED SUBSURFACE PROFILE
A-A'

Date 04/15/15	Project No. 770619901	Figure 6

LANGAN TREADWELL ROLLO

Notes:
1. The above profile represents a generalized subsurface cross section interpreted from widely spaced borings. Earth materials may vary in type, strength, and other important properties between points of exploration.

DOVE HILL ASSISTED LIVING COMMUNITY
4200 DOVE HILL ROAD
San Jose, California

IDEALIZED SUBSURFACE PROFILE
B-B'

Date 05/26/15	Project No. 770619901	Figure 7

LANGAN TREADWELL ROLLO

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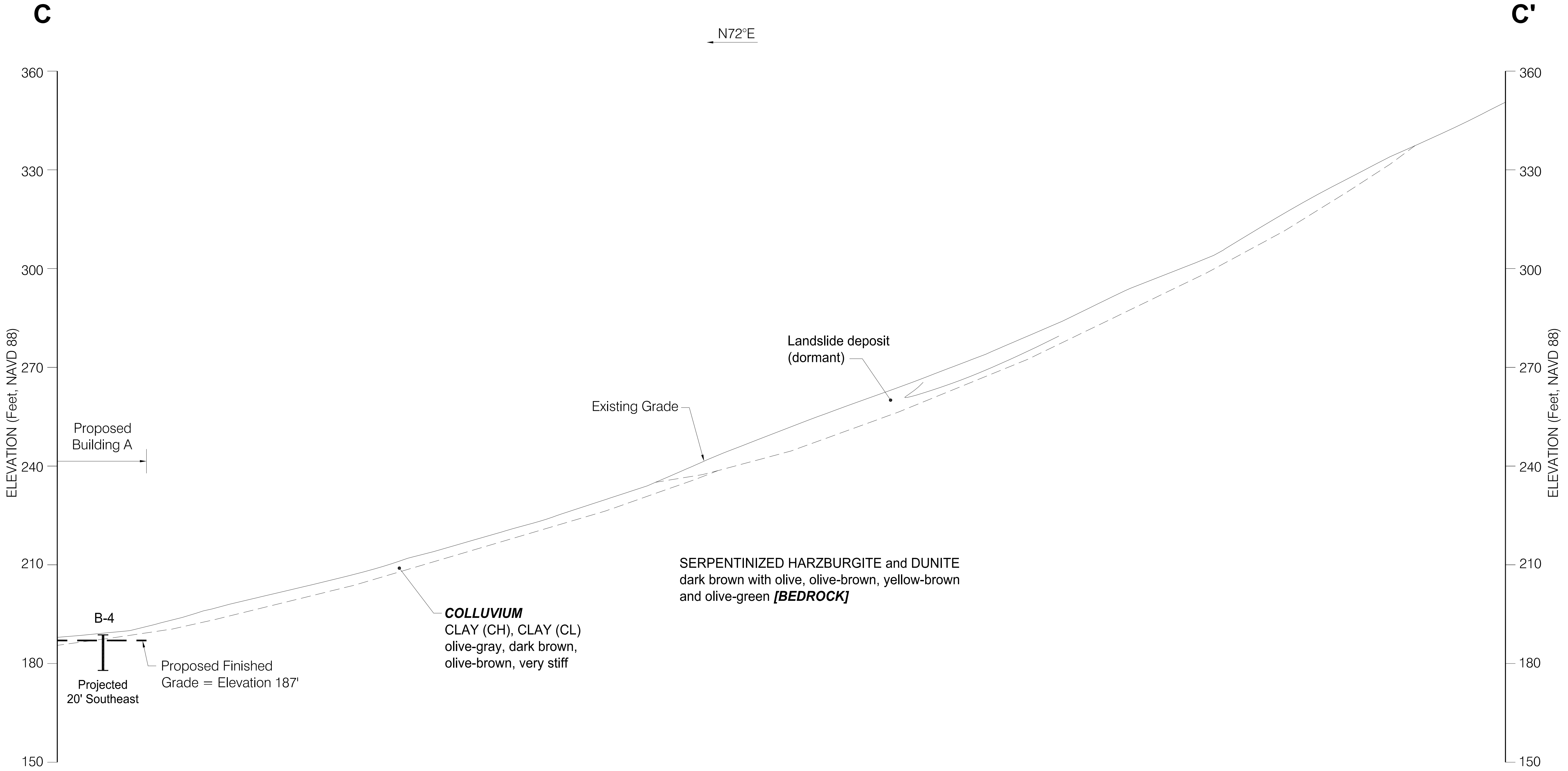
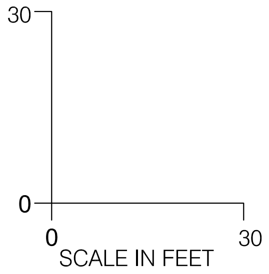


Notes:
1. The above profile represents a generalized subsurface cross section interpreted from widely spaced borings. Earth materials may vary in type, strength, and other important properties between points of exploration.



DOVE HILL ASSISTED LIVING COMMUNITY
4200 DOVE HILL ROAD
San Jose, California

IDEALIZED SUBSURFACE PROFILE C-C'

Date 04/15/15	Project No. 770619901	Figure 8

LANGAN TREADWELL ROLLO

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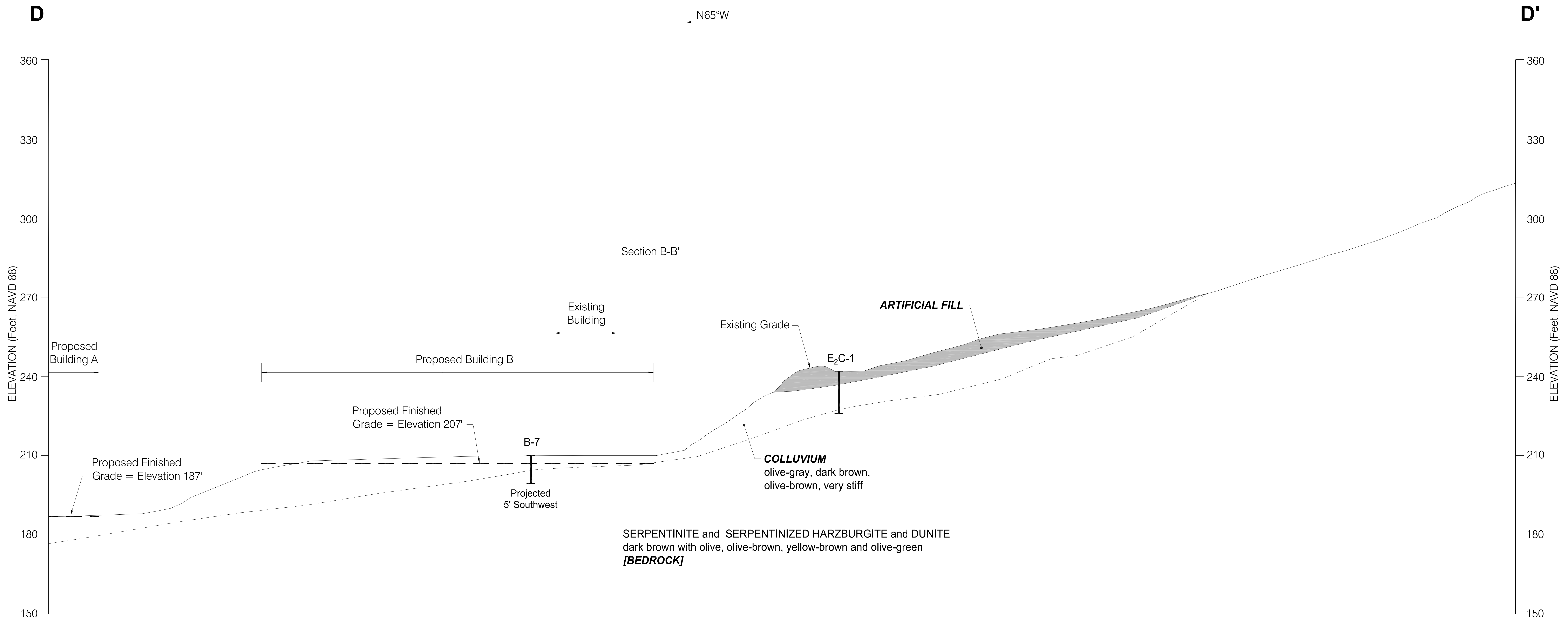
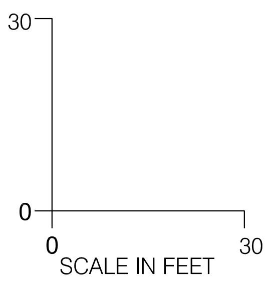


Notes:
1. The above profile represents a generalized subsurface cross section interpreted from widely spaced borings. Earth materials may vary in type, strength, and other important properties between points of exploration.



DOVE HILL ASSISTED LIVING COMMUNITY
4200 DOVE HILL ROAD
San Jose, California

IDEALIZED SUBSURFACE PROFILE D-D'

Date 04/15/15	Project No. 770619901	Figure 9

LANGAN TREADWELL ROLLO

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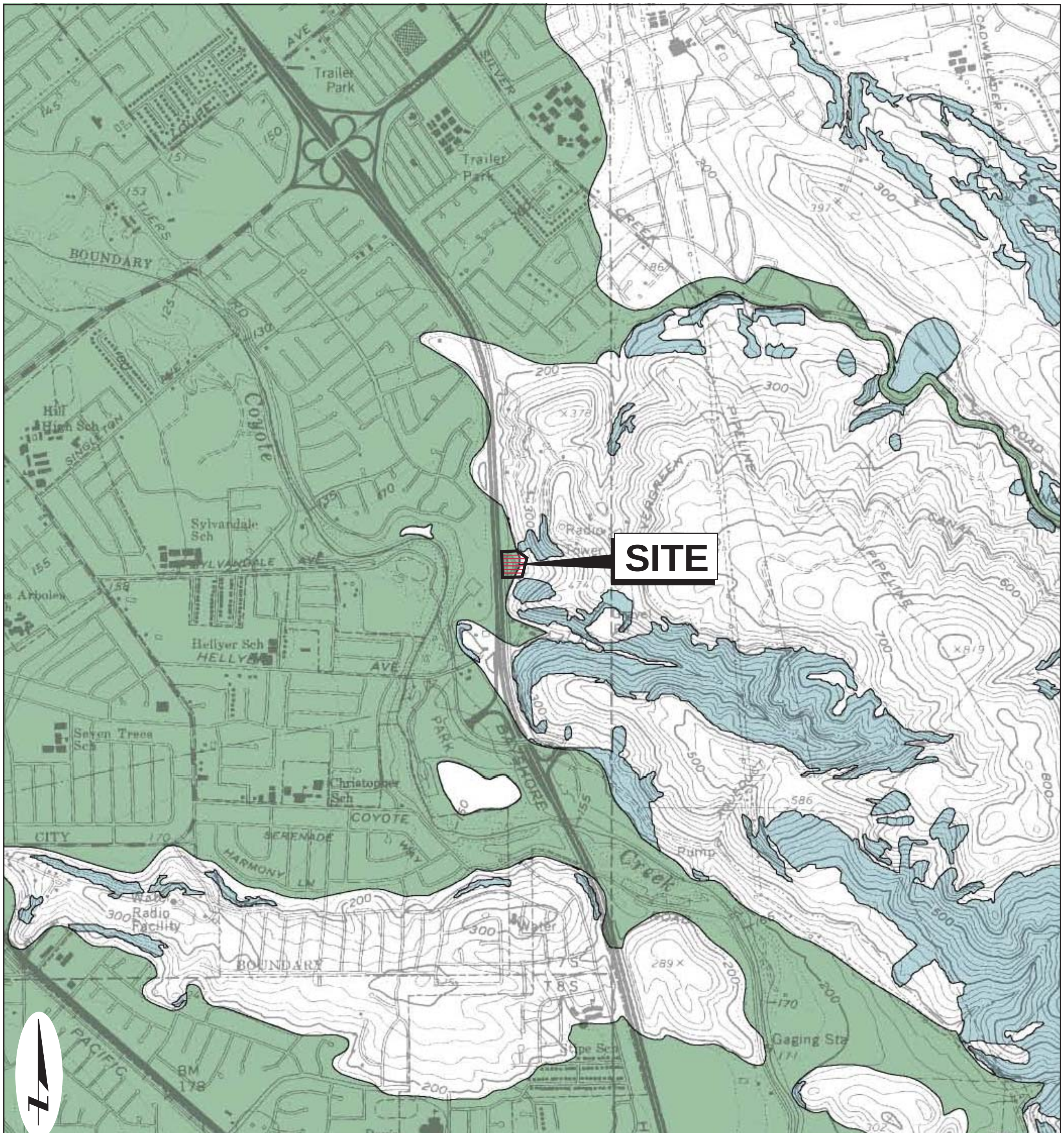


EXPLANATION

Liquefaction; Areas where historic occurence of liquefaction, or local topographic, geological, geotechnical, and subsurface water conditions indicate a potential for permanent ground displacements.

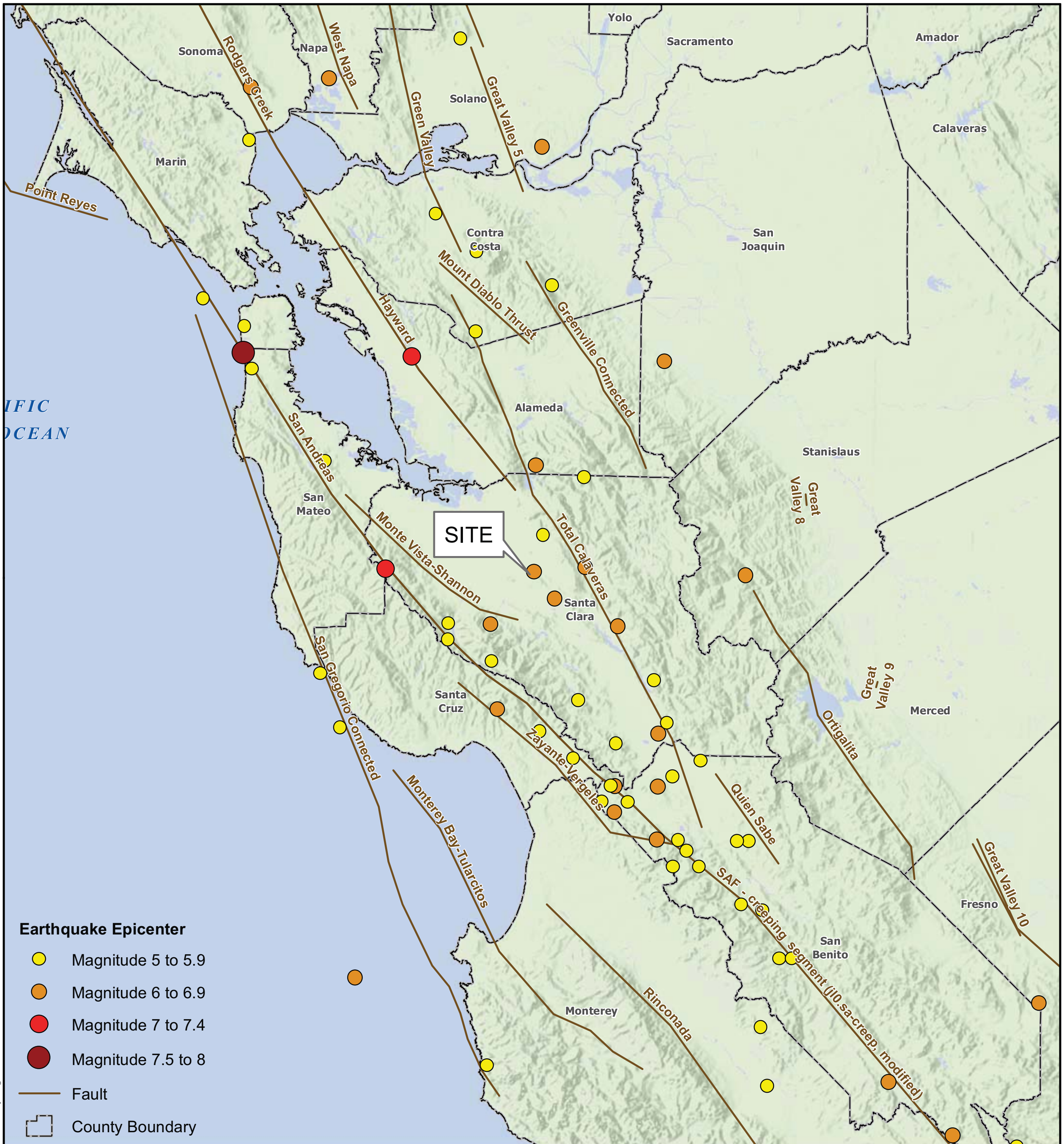
Earthquake-Induced Landslides; Areas where previous occurence of landslide movement, or local topographic, geological, geotechnical, and subsurface water conditions indicate a potential for permanent ground displacements.

0 2,000 4,000 Feet

Approximate scale

Reference:
State of California "Seismic Hazard Zones"
San Jose East Quadrangle
Released on January 17, 2001

DOVE HILL ASSISTED LIVING COMMUNITY **4200 DOVE HILL ROAD** San Jose, California	**REGIONAL SEISMIC HAZARD ZONES MAP**
LANGAN TREADWELL ROLLO	Date 05/26/15 | Project No. 770619901 | Figure 10



Notes:

1. Quaternary fault data displayed are based on a generalized version of USGS Quaternary Fault and fold database, 2010. For cartographic purposes only.
2. The Earthquake Epicenter (Magnitude) data is provided by the U.S Geological Survey (USGS) and is current through 08/26/2014.
3. Basemap hillshade and County boundaries provided by USGS and California Department of Transportation.
4. Map displayed in California State Coordinate System, California (Teale) Albers, North American Datum of 1983 (NAD83), Meters.

0 5 10 20
Miles

N

DOVE HILL ASSISTED LIVING COMMUNITY
4200 DOVE HILL ROAD
San Jose, California

LANGAN TREADWELL ROLLO

MAP OF MAJOR FAULTS AND EARTHQUAKE EPICENTERS IN THE SAN FRANCISCO BAY AREA

Date 4/1/2015	Project No. 770619901	Figure 11

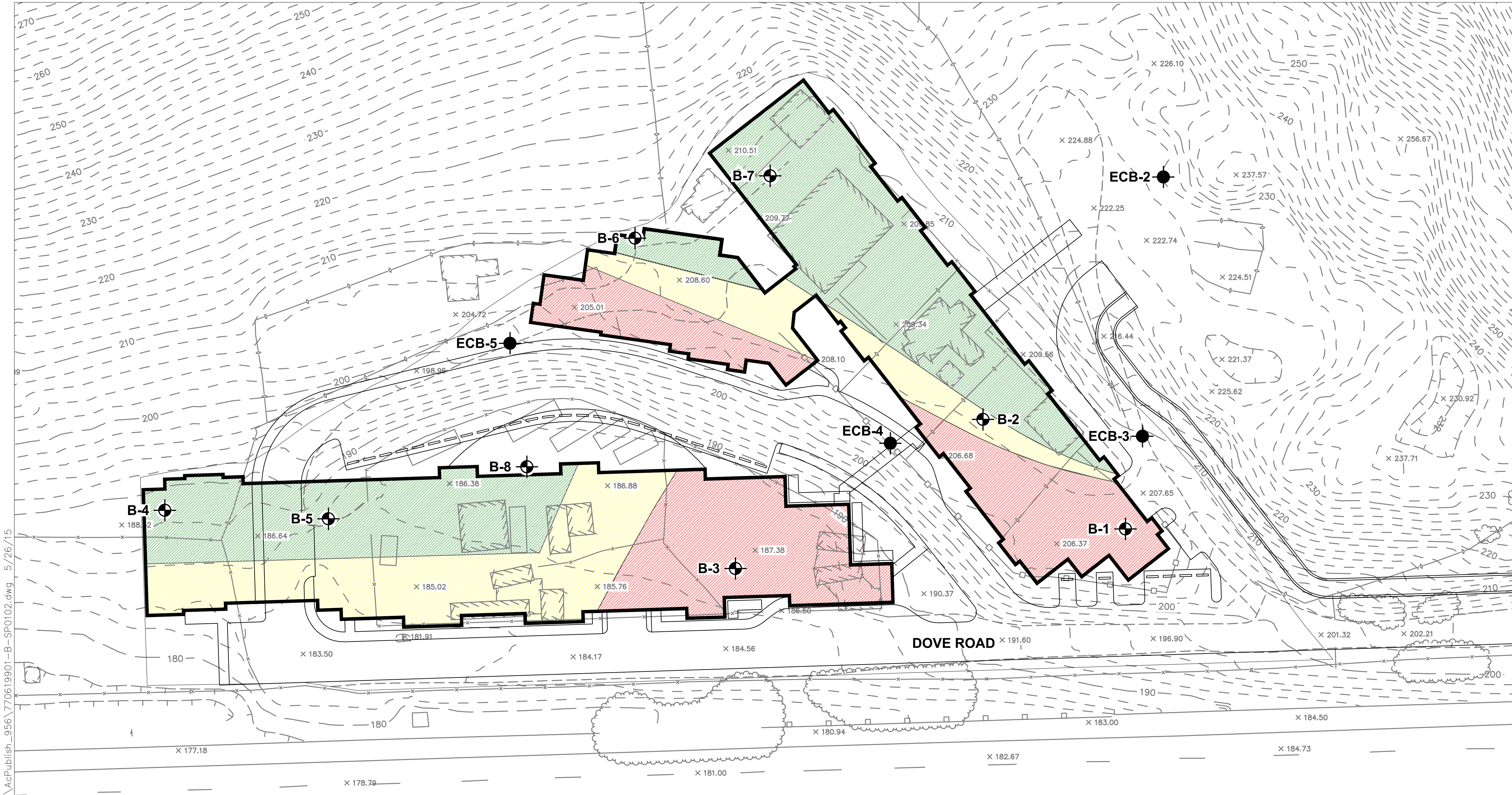
I	**Not felt by people, except under especially favorable circumstances. However, dizziness or nausea may be experienced.** Sometimes birds and animals are uneasy or disturbed. Trees, structures, liquids, bodies of water may sway gently, and doors may swing very slowly.
II	**Felt indoors by a few people, especially on upper floors of multi-story buildings, and by sensitive or nervous persons.** As in Grade I, birds and animals are disturbed, and trees, structures, liquids and bodies of water may sway. Hanging objects swing, especially if they are delicately suspended.
III	**Felt indoors by several people, usually as a rapid vibration that may not be recognized as an earthquake at first. Vibration is similar to that of a light, or lightly loaded trucks, or heavy trucks some distance away. Duration may be estimated in some cases.** Movements may be appreciable on upper levels of tall structures. Standing motor cars may rock slightly.
IV	**Felt indoors by many, outdoors by a few. Awakens a few individuals, particularly light sleepers, but frightens no one except those apprehensive from previous experience. Vibration like that due to passing of heavy, or heavily loaded trucks. Sensation like a heavy body striking building, or the falling of heavy objects inside.** Dishes, windows and doors rattle; glassware and crockery clink and clash. Walls and house frames creak, especially if intensity is in the upper range of this grade. Hanging objects often swing. Liquids in open vessels are disturbed slightly. Stationary automobiles rock noticeably.
V	**Felt indoors by practically everyone, outdoors by most people. Direction can often be estimated by those outdoors. Awakens many, or most sleepers. Frightens a few people, with slight excitement; some persons run outdoors.** Buildings tremble throughout. Dishes and glassware break to some extent. Windows crack in some cases, but not generally. Vases and small or unstable objects overturn in many instances, and a few fall. Hanging objects and doors swing generally or considerably. Pictures knock against walls, or swing out of place. Doors and shutters open or close abruptly. Pendulum clocks stop, or run fast or slow. Small objects move, and furnishings may shift to a slight extent. Small amounts of liquids spill from well-filled open containers. Trees and bushes shake slightly.
VI	**Felt by everyone, indoors and outdoors. Awakens all sleepers. Frightens many people; general excitement, and some persons run outdoors.** Persons move unsteadily. Trees and bushes shake slightly to moderately. Liquids are set in strong motion. Small bells in churches and schools ring. Poorly built buildings may be damaged. Plaster falls in small amounts. Other plaster cracks somewhat. Many dishes and glasses, and a few windows break. Knickknacks, books and pictures fall. Furniture overturns in many instances. Heavy furnishings move.
VII	**Frightens everyone. General alarm, and everyone runs outdoors.** People find it difficult to stand. Persons driving cars notice shaking. Trees and bushes shake moderately to strongly. Waves form on ponds, lakes and streams. Water is muddied. Gravel or sand stream banks cave in. Large church bells ring. Suspended objects quiver. Damage is negligible in buildings of good design and construction; slight to moderate in well-built ordinary buildings; considerable in poorly built or badly designed buildings, adobe houses, old walls (especially where laid up without mortar), spires, etc. Plaster and some stucco fall. Many windows and some furniture break. Loosened brickwork and tiles shake down. Weak chimneys break at the roofline. Cornices fall from towers and high buildings. Bricks and stones are dislodged. Heavy furniture overturns. Concrete irrigation ditches are considerably damaged.
VIII	**General fright, and alarm approaches panic.** Persons driving cars are disturbed. Trees shake strongly, and branches and trunks break off (especially palm trees). Sand and mud erupts in small amounts. Flow of springs and wells is temporarily and sometimes permanently changed. Dry wells renew flow. Temperatures of spring and well waters varies. Damage slight in brick structures built especially to withstand earthquakes; considerable in ordinary substantial buildings, with some partial collapse; heavy in some wooden houses, with some tumbling down. Panel walls break away in frame structures. Decayed pilings break off. Walls fall. Solid stone walls crack and break seriously. Wet grounds and steep slopes crack to some extent. Chimneys, columns, monuments and factory stacks and towers twist and fall. Very heavy furniture moves conspicuously or overturns.
IX	**Panic is general.** Ground cracks conspicuously. Damage is considerable in masonry structures built especially to withstand earthquakes; great in other masonry buildings - some collapse in large part. Some wood frame houses built especially to withstand earthquakes are thrown out of plumb, others are shifted wholly off foundations. Reservoirs are seriously damaged and underground pipes sometimes break.
X	**Panic is general.** Ground, especially when loose and wet, cracks up to widths of several inches; fissures up to a yard in width run parallel to canal and stream banks. Landsliding is considerable from river banks and steep coasts. Sand and mud shifts horizontally on beaches and flat land. Water level changes in wells. Water is thrown on banks of canals, lakes, rivers, etc. Dams, dikes, embankments are seriously damaged. Well-built wooden structures and bridges are severely damaged, and some collapse. Dangerous cracks develop in excellent brick walls. Most masonry and frame structures, and their foundations are destroyed. Railroad rails bend slightly. Pipe lines buried in earth tear apart or are crushed endwise. Open cracks and broad wavy folds open in cement pavements and asphalt road surfaces.
XI	**Panic is general.** Disturbances in ground are many and widespread, varying with the ground material. Broad fissures, earth slumps, and land slips develop in soft, wet ground. Water charged with sand and mud is ejected in large amounts. Sea waves of significant magnitude may develop. Damage is severe to wood frame structures, especially near shock centers, great to dams, dikes and embankments, even at long distances. Few if any masonry structures remain standing. Supporting piers or pillars of large, well-built bridges are wrecked. Wooden bridges that "give" are less affected. Railroad rails bend greatly and some thrust endwise. Pipe lines buried in earth are put completely out of service.
XII	**Panic is general.** Damage is total, and practically all works of construction are damaged greatly or destroyed. Disturbances in the ground are great and varied, and numerous shearing cracks develop. Landslides, rock falls, and slumps in river banks are numerous and extensive. Large rock masses are wrenched loose and torn off. Fault slips develop in firm rock, and horizontal and vertical offset displacements are notable. Water channels, both surface and underground, are disturbed and modified greatly. Lakes are dammed, new waterfalls are produced, rivers are deflected, etc. Surface waves are seen on ground surfaces. Lines of sight and level are distorted. Objects are thrown upward into the air.

DOVE HILL ASSISTED LIVING COMMUNITY
4200 DOVE HILL ROAD
San Jose, California

MODIFIED MERCALLI INTENSITY SCALE

LANGAN TREADWELL ROLLO

Date 04/01/15 | Project No. 770619901 | Figure 12



EXPLANATION

B-1 — Approximate location of boring by Langan Treadwell Rollo, May 2014

ECB-1 — Approximate location of boring by E_2C, Inc., August 2008

Approximate location of foundation consisting of footings bearing on rock

Approximate location of foundation transition zone where drilled piers may be required

Approximate location of foundation requiring drilled piers

0 80 Feet
Approximate scale

DOVE HILL ASSISTED LIVING COMMUNITY
4200 DOVE HILL ROAD
San Jose, California

APPROXIMATE FOUNDATION ZONE

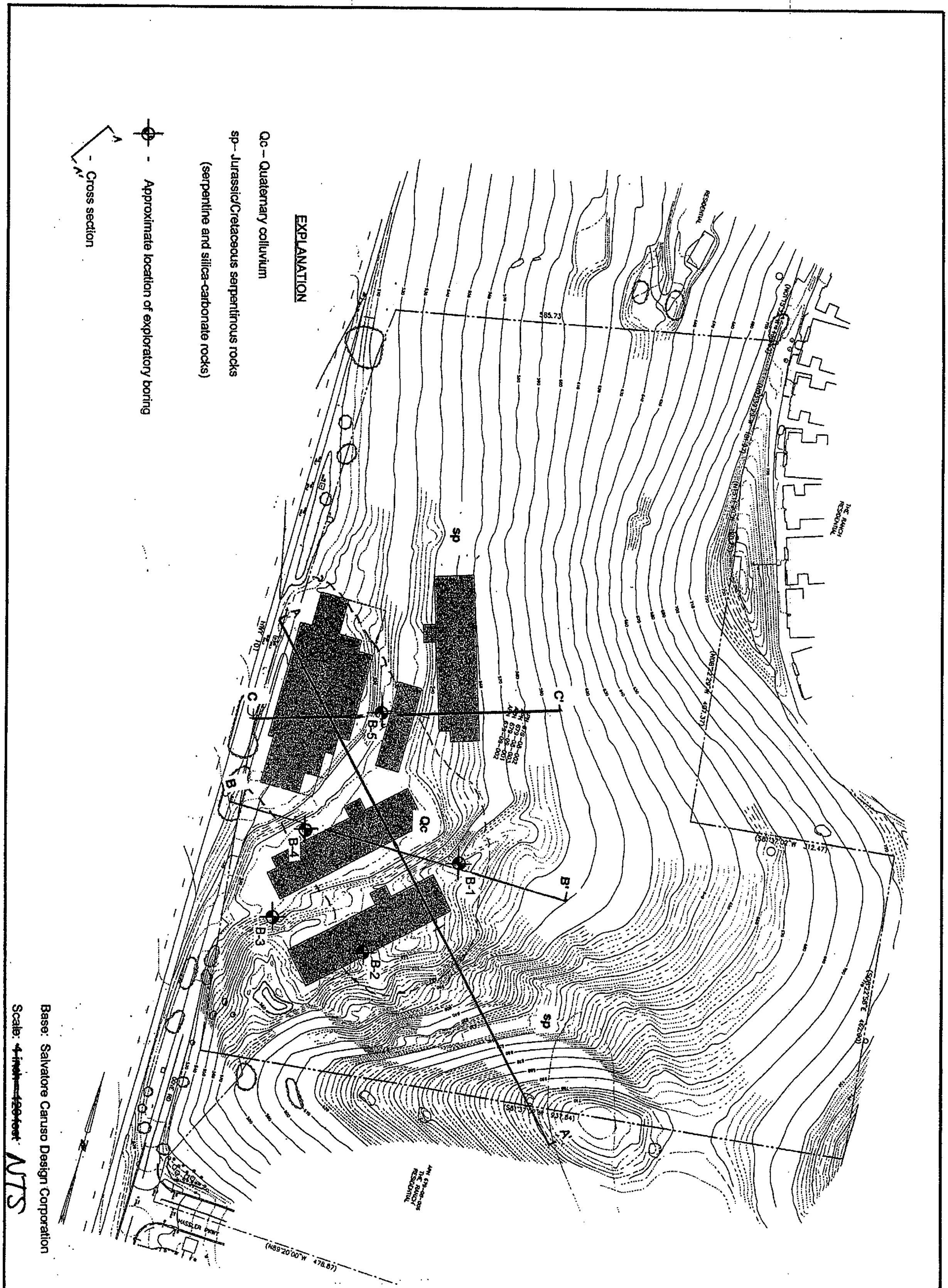
Date 05/26/15	Project No. 770619901	Figure 13

LANGAN TREADWELL ROLLO

Reference: Base map from a drawing title "--," by --, dated --.

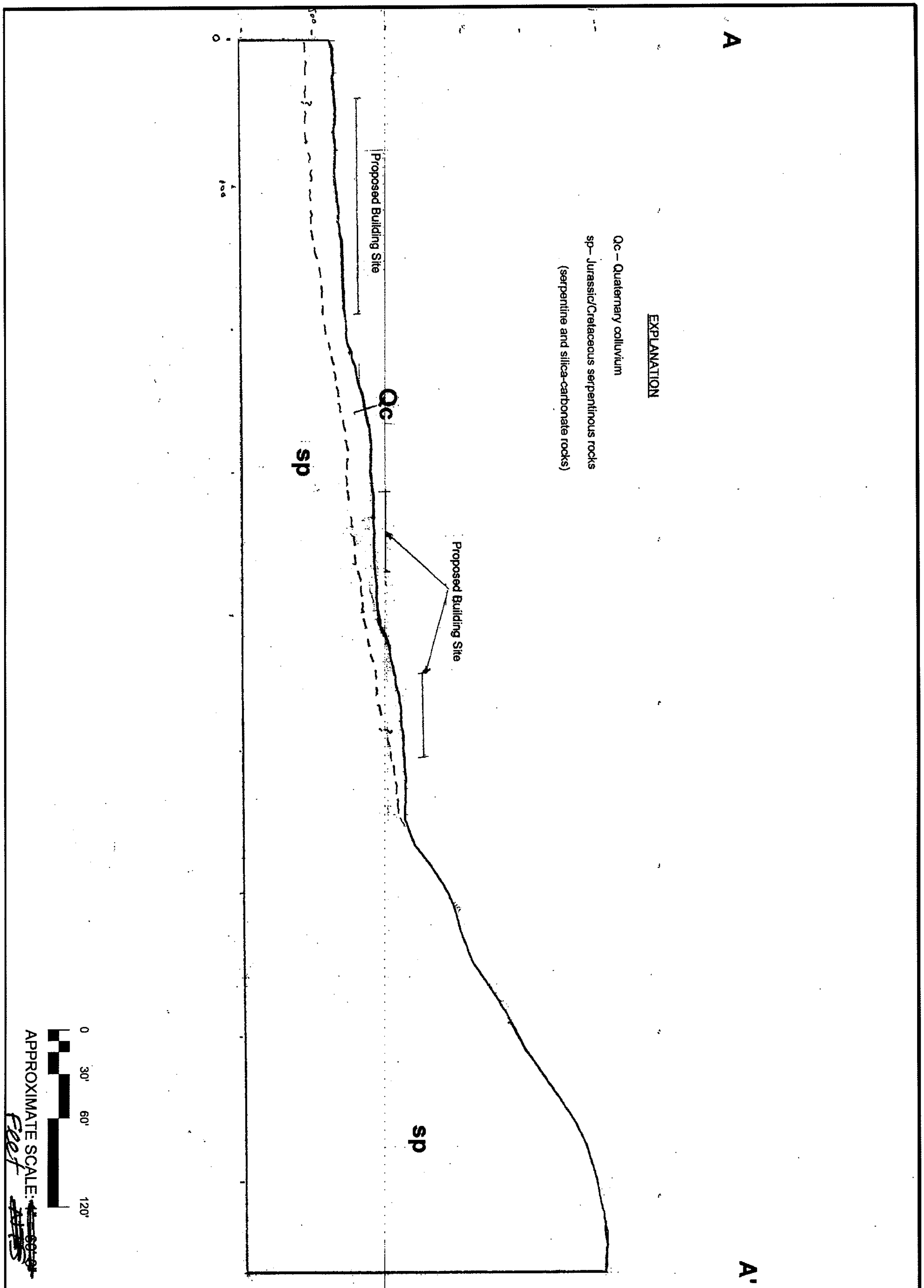
APPENDIX A

BORING LOGS AND LABORATORY TEST DATA FROM PREVIOUS INVESTIGATION



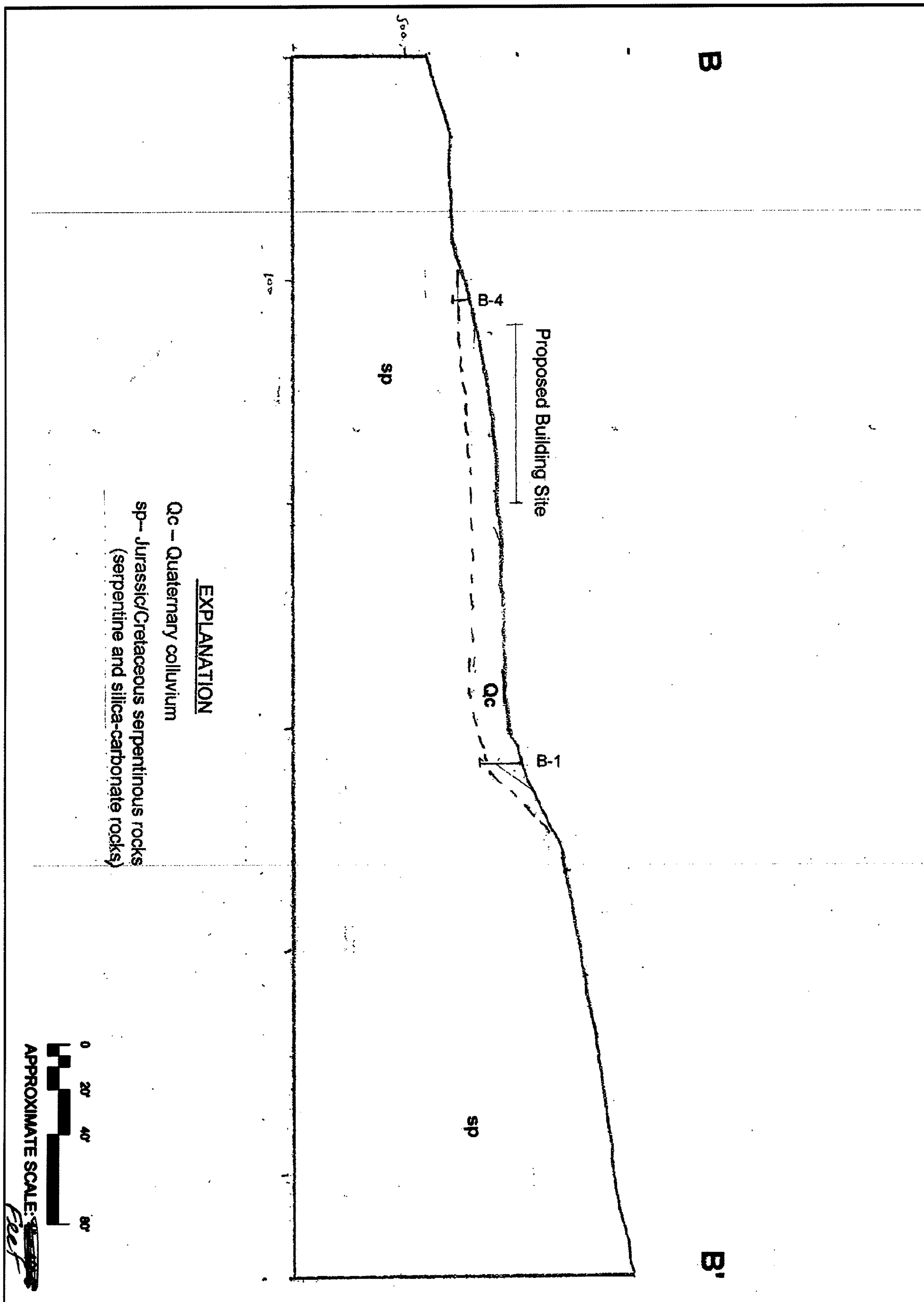
SITE PLAN AND LOCAL GEOLOGY

STRUCTURAL / ENVIRONMENTAL ENGINEERING CONSULTANTS 3016 SCOTT BOULEVARD SANTA CLARA, CALIFORNIA 95054-3323 TEL: 408.327.5700 FAX: 408.327.5707	Dove Hill Assisted Living Community 4200 Dove Hill Road San Jose, California	FILENAME: 2857-G DATE: August 2008 CHECK BY: DRAWN: JEB	FIGURE: 2



LOCAL GEOLOGY

STRUCTURAL / ENVIRONMENTAL ENGINEERING CONSULTANTS 3016 SCOTT BOULEVARD SANTA CLARA, CALIFORNIA 95054-3323 TEL: 408.327.5700 FAX: 408.327.5707	DOVE HILL ASSISTED LIVING COMMUNITY 4200 DOVE HILL ROAD SAN JOSE, CA	FILENAME: 2857-G DATE: AUG 2008 CHECKED BY: DRAWN: JEB	FIGURE: 5



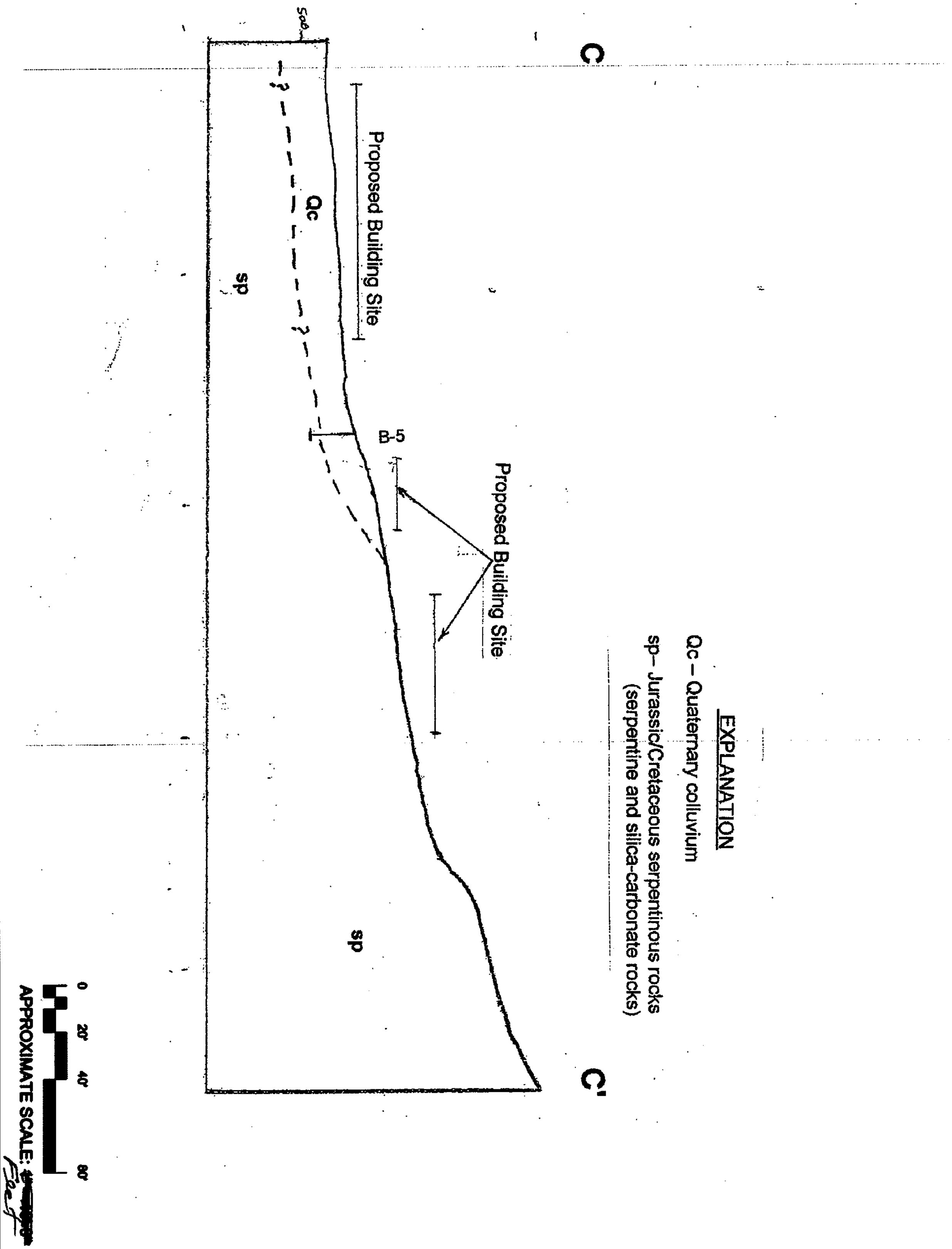
LOCAL GEOLOGY

STRUCTURAL / ENVIRONMENTAL
ENGINEERING CONSULTANTS
3016 SCOTT BOULEVARD
SANTA CLARA, CALIFORNIA 95054-3323
TEL: 408.327.5700 FAX: 408.327.5707

DOVE HILL ASSISTED LIVING COMMUNITY
4200 DOVE HILL ROAD
SAN JOSE, CA

FILENAME:	2857-G
DATE:	AUG 2008
CHECKED BY:	
DRAWN:	JEB

FIGURE:
6



LOCAL GEOLOGY

STRUCTURAL / ENVIRONMENTAL ENGINEERING CONSULTANTS 3016 SCOTT BOULEVARD SANTA CLARA, CALIFORNIA 95054-3323 TEL: 408.327.5700 FAX: 408.327.5707	DOVE HILL ASSISTED LIVING COMMUNITY 4200 DOVE HILL ROAD SAN JOSE, CA	FILENAME: 2857-G DATE: AUG 2008 CHECKED BY: DRAWN: JEB	FIGURE: 7

Environmental/Engineering Consultants
382 Martin Avenue
Santa Clara, California 95050-3112
Tel: 408.327.5700 FAX: 408.327.5707

BORING LOG B - 1

Project No. 2857SC01-G

Driller: Drake Drilling
Drill Rig: Mobile B-24
Depth to Groundwater: NGWE
Date Drilled: August 18, 2008
Logged by: JEB

Depth (ft.)	Symbol	Sample No.	Blows / Ft.	U.S.C.S Soil Group	Pocket Pen. (t.s.f.)	DESCRIPTION	% Moisture	Dry Density (p.c.f.)
0								
		1-1	6 9 14	CH		Silty Clay; dark grey, w/occasional serpentine clast, dry, very stiff	17.7	91.9
5		1-2	10 15 16	CH		Clay; black, plastic, slightly moist, very stiff to hard Phi = 25 degrees, C = 240 psf	40.2	89.5
10		1-3	11 17 23			Clay; brownish grey, slightly moist, hard	25.7	91.8
15		1-4	24 50	sp		Serpentine; olive, highly weathered, dry, hard	18.5	110.8
						Boring Terminated at 16 feet, Refusal @ 16 feet		
						No Groundwater Encountered		

Environmental/Engineering Consultants
382 Martin Avenue
Santa Clara, California 95050-3112
Tel: 408.327.5700 FAX: 408.327.5707

BORING LOG B-2

Project No. 2857SC01-G

Driller: Drake Drilling
Drill Rig: Mobile B-24
Depth to Groundwater: NGWE
Date Drilled: August 18, 2008
Logged by: JEB

Depth (ft.)	Symbol	Sample No.	Blows / Ft.	U.S.C.S Soil Group	Pocket Pen. (t.s.f.)	DESCRIPTION	% Moisture	Dry Density (p.c.f.)
0								
		2-1	21 24 15			Silty Clay; olive grey, w/ occasional highly weathered serpentine, slightly moist, very stiff, cobble of sandstone	6.2	114.1
5				CH				
		2-2	9 10 13			Silty Clay; olive grey, w/ occasional highly weathered serpentine, slightly moist, very stiff	14.7	116.7
10								
		2-3	9 15 50	sp		Serpentine; dark grey, moderately weathered, slightly moist, hard	12.8	126.3
						Boring Terminated at 13.5 feet, Refusal @ 13.5 feet No Groundwater Encountered		

Environmental/Engineering Consultants
382 Martin Avenue
Santa Clara, California 95050-3112
Tel: 408.327.5700 FAX: 408.327.5707

BORING LOG B - 3

Project No. 2857SC01-G

Driller Drake Drilling
Drill Rig Mobile B-24
Depth to Groundwater NGWE
Date Drilled August 18, 2008
Logged by JEB

Depth (ft.)	Symbol	Sample No.	Blows / Ft.	U.S.C.S Soil Group	Pocket Pen. (t.s.f.)	DESCRIPTION	% Moisture	Dry Density (p.c.f.)
0								
		3-1	14 26 31	sp		Serpentine; olive grey, highly weathered, dry, hard	10.8	123.2
5		3-2	29 50	sp		Serpentine; olive yellow, moderately weathered, dry, hard	19.1	104.1
						Boring Terminated at 6 feet, Refusal @ 6 feet		
						No Groundwater Encountered		

Environmental/Engineering Consultants
382 Martin Avenue
Santa Clara, California 95050-3112
Tel: 408.327.5700 FAX: 408.327.5707

BORING LOG B - 4

Project No. 2857SC01-G

Driller: Drake Drilling
Drill Rig: Mobile B-24
Depth to Groundwater: NGWE
Date Drilled: August 18, 2008
Logged by: JEB

Depth (ft.)	Symbol	Sample No.	Blows / Ft.	U.S.C.S Soil Group	Pocket Pen. (t.s.f.)	DESCRIPTION	% Moisture	Dry Density (p.c.f.)
0								
		4-1	7 12 25	CH		Clay; black,calcareous, expansive, dry, hard	17.3	100.1
5		4-2	50	sp		Serpentine; olive, dry, very hard	6.8	96.5
						Boring Terminated at 5.5 feet, Refusal @ 5.5 feet		
						No Groundwater Encountered		

Environmental/Engineering Consultants
382 Martin Avenue
Santa Clara, California 95050-3112
Tel: 408.327.5700 FAX: 408.327.5707

BORING LOG B - 5

Project No. 2857SC01-G

Driller: Drake Drilling
Drill Rig: Mobile B-24
Depth to Groundwater: NGWE
Date Drilled: August 18, 2008
Logged by: JEB

Depth (ft.)	Symbol	Sample No.	Blows / Ft.	U.S.C.S Soil Group	Pocket Pen. (t.s.f.)	DESCRIPTION	% Moisture	Dry Density (p.c.f.)
0								
		5-1	9 15 19	CH		Clay; black, w/occasional fine angular clast, dry, hard	----	----
5		5-2	11 17 23			As Above	25.3	----
10		5-3	13 18 22			Silty Clay; black, slightly moist, hard	25.3	94.3
15		5-4	24 50	sp		Serpentine; olive, dry, very hard	10.8	127.1

Boring Terminated at 16 feet, Refusal @ 16 feet

No Groundwater Encountered

TABLE A1

Summary of Moisture, Density, Swell and Direct Shear Testing

		In-Place Conditions		Direct Shear Testing	
		Moisture Content	Dry Density	Angle of Internal Friction	Unit Cohesion
Sample No.	Depth feet	(% dry wt)	p.c.f.	(degrees)	p.s.f.
1-1	2-2.5	17.7	91.9		
1-2	6-6.5	40.2	89.5	28	240
1-3	11-11.5	25.7	91.8		
1-4	15.5-16	18.5	110.8		
2-1	2-2.5	6.2	114.1		
2-2	8-8.5	14.7	116.7		
2-3	13-13.5	12.8	126.3		
3-1	2-2.5	10.8	123.2		
3-2	5.5-6	19.1	104.1		
4-1	2-2.5	17.3	100.1		
4-2	5-5.5	6.8	96.5		
5-1	2-2.5	*	*		
5-2	6-6.5	25.3	*		
5-3	11-11.5	23.5	94.3		
5-4	15-15.5	10.8	127.1		

* Disturbed sample or Atterberg Limits test

TABLE A2

Summary of Laboratory Atterberg Limits Test Results

Sample No.	Depth ft.	Description of Soil	Atterberg Limits: Liquid Limit %	Atterberg Limits: Plasticity Index (P.I.)
5-1	2-2.5	Black clay (CH)	73	45

ES GEOTECHNOLOGIES

File No.: BA-2181-01 Date: 8/25/2008

UNIT DENSITIES AND MOISTURE CONTENT

ASTM D2937 & D2216

Job Name: E2C - 2857G - Dove Hill Rd.

Sample Location	Soil Description (feet)	Unit Dry Density (pcf)	Moisture Content (%)	USCS Group Symbol
1-1	Very Dark Gray Silty Clay w/ fine gravel	91.9	17.7	CH/GC
1-3	Dark Gray Silty Clay w/ fine gravel	91.8	25.7	CH/GC
1-4	Grayish Brown Sandy Clay w/ fine gravel	110.8	18.5	SP/SC
2-1	ıyish Brown Sandy Silt mix w/ medium gravel (8(	114.1	6.2	GC
2-2	:k Grayish Brown Sandy Silty Clay mix w/fine gra	116.7	14.7	CL
2-3	Dark Greenish Gray Sandy Clay w/ fine gravel	126.3	12.8	SP/SC
3-1	Dark Gray Coarse Sandy Clay w/ fine gravel	123.2	10.8	GC
3-2	(ellowish Brown Coarse Sandy Clay w/ fine grave	104.1	19.1	GC/GP
4-1	Black Sily Clay	100.1	17.3	CH
4-2	Brown Coarse Sand w/ fine gravel	96.5	6.8	GC
5-2	Black Coarse Sandy Clay		25.3	ML
5-3	Black Coarse Sandy Clay w/ fine gravel	94.3	23.5	ML
5-4	Dark Gray Sandy Clay w/ fine gravel	127.1	10.8	ML

ES GEOTECHNOLOGIES

File No.: BA-2181-01

August 25, 2008

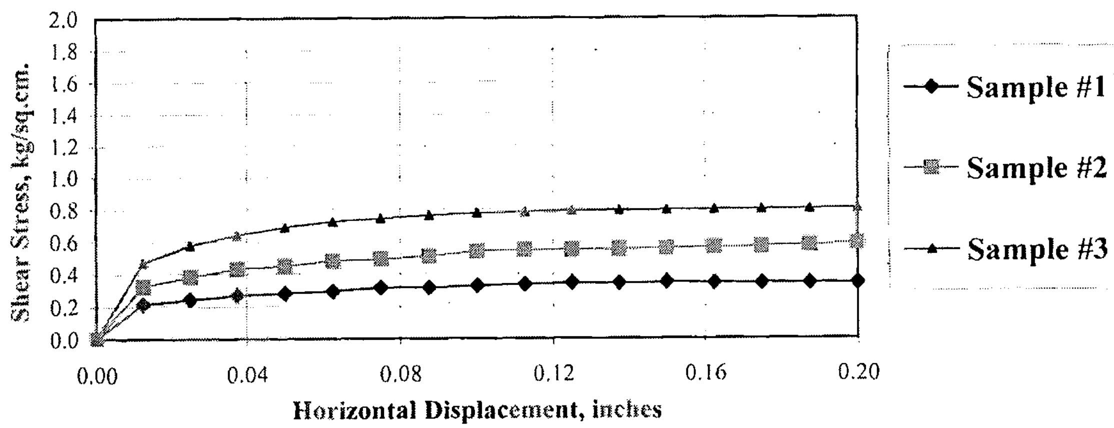
DIRECT SHEAR

ASTM D 3080-90 (modified for unconsolidated, undrained conditions)

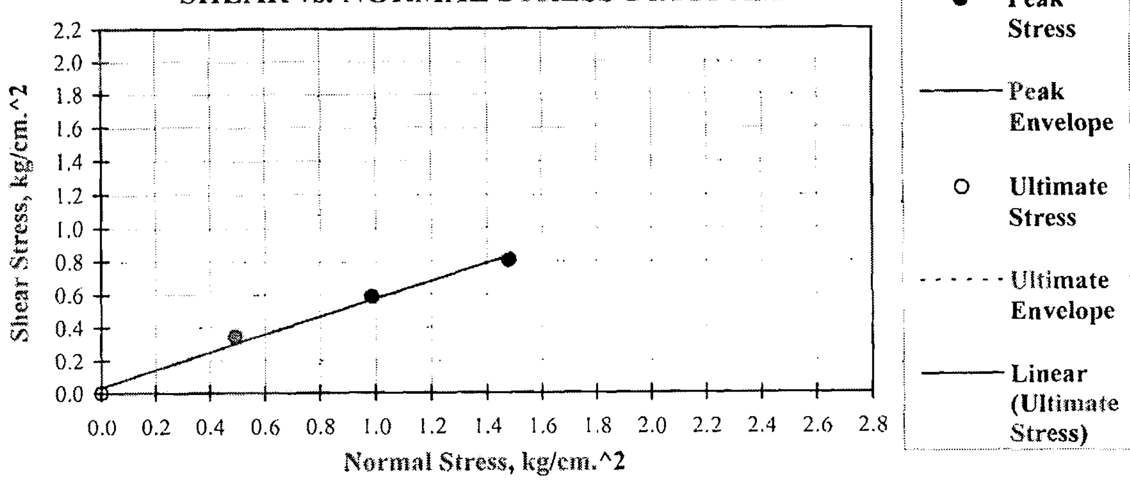
Project Name: E2C - 2857G - Dove Hill Rd.
File Number: BA-2181-01
Soil Description: Very Dark Brown Silty Clay
Sample : 1-2

Dry Density: 89.5 pcf
Mosture Content: 40.2 %
Peak Friction Angle (Ø): 25°
Cohesion (c): 0.118 kg/cm^2 (240 psf)
Ultimate Friction Angle (Ø): 28°

Sample No.	1	2	3	Average
Initial				
Dry Density, pcf	85.7	90.4	92.4	89.5
Moisture Content at Test, %	44.2	39.0	37.5	40.2
Saturation, %	100	100	100	100.0
At Test				
Moisture Content, %	44.2	39.0	37.5	40.2
Saturation, %	125	123	125	124
Normal Stress, kg/cm^2	0.49	0.99	1.48	
Peak Stress, kg/cm^2	0.34	0.59	0.80	
Ultimate Stress, kg/cm^2	0.34	0.59	0.80	



SHEAR vs. NORMAL STRESS DIAGRAM



ES GEOTECHNOLOGIES

File No.: BA-2181-01

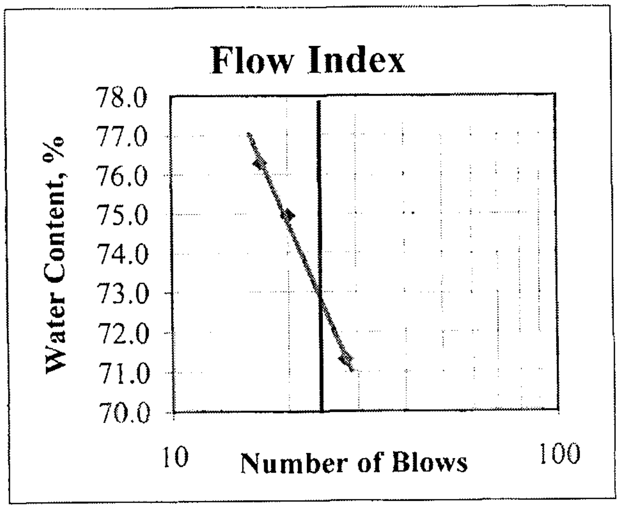
August 25, 2008

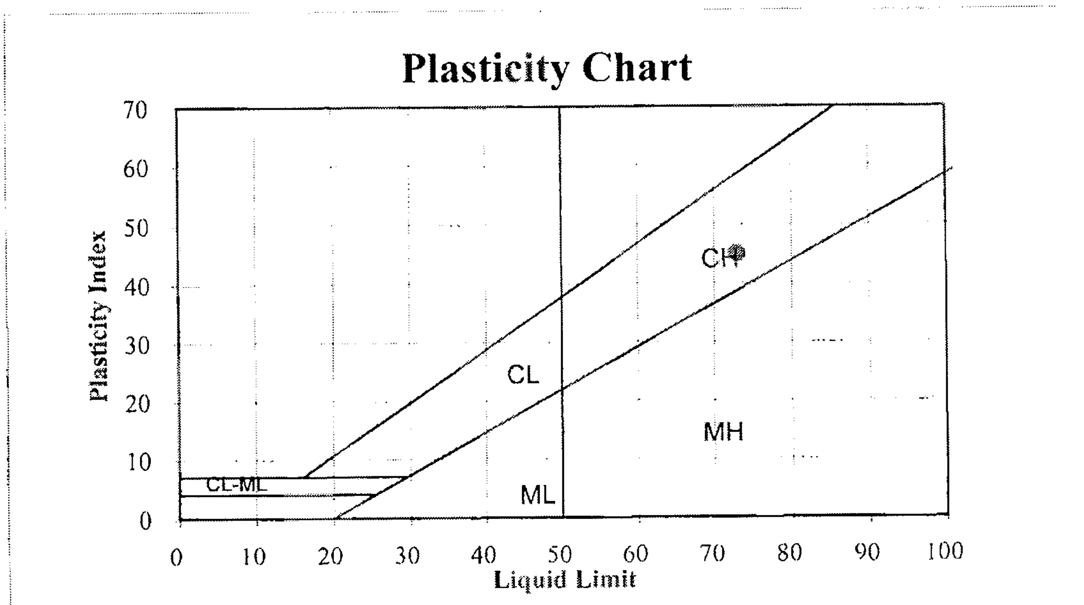
PLASTICITY INDEX

ASTM D-4318

Job Name: E2C - 2857G - Dove Hill Rd.
Sample ID: 5-1
Soil Description: Dark Gray Silty Clay

DATA SUMMARY				TEST RESULTS	
Number of Blows:	28	20	17	**LIQUID LIMIT**	**73**
Water Content, %	71.3	74.9	76.3	**PLASTIC LIMIT**	**28**
Plastic Limit:	28.2	27.4		**PLASTICITY INDEX**	**45**





APPENDIX B

LOG OF BORINGS

PROJECT: **DOVE HILL ASSISTED LIVING COMMUNITY**
4200 DOVE HILL ROAD
San Jose, California

Log of Boring B-1

Boring location:	See Site Plan, Figure 2
Date started:	3/25/15
Date finished:	3/25/15
Drilling method:	Hollow Stem Auger
Hammer weight/drop:	140 lbs./30 inches
Hammer type:	Rope & Cathead
Sampler:	Sprague & Henwood (S&H), Standard Penetration Test (SPT)
Logged by:	S. Magallon/R. Ward

Ground Surface Elevation: 207.2 feet[2]

DEPTH (feet)	SAMPLES: Sampler Type	Blows/ 6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	LABORATORY TEST DATA: Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
0					2 inches topsoil						
1				CH	CLAY (CH) olive-gray, very stiff, moist						
3	S&H	20 17 31	29		LL = 69, PI = 47, see Figure C-4 TxUU Test, see Figure C-1	TxUU	420	3,490		20.0	100
6	S&H	24 27 30	34		dark brown with olive mottling, hard						
8	S&H	22 18 14	19		very stiff						
9				CL	SANDY CLAY with GRAVEL (CL) olive-brown, very stiff, dry, fine sand, fine gravel						
10	S&H	44 50/ 4.5"	30/ 4.5"		SERPENTINITE dark brown with olive and light brown, moderately hard, deeply weathered, strong, highly sheared with white talc veins, isolated chert inclusion						
15	SPT	21 45 50	95								

Boring terminated at a depth of 16.5 feet below ground surface.
Boring backfilled with cement grout.
Groundwater not encountered during drilling.

[1] S&H and SPT blow counts for the last two increments were converted to SPT N-Values using factors of 0.6 and 1.0, respectively to account for sampler type and hammer energy.
[2] Elevations based on NAVD 88.

LANGAN TREADWELL ROLLO

Project No.: 770619901
Figure: B-1

TEST GEOTECH LOG 770619901 EDIT PDF FILE #.GPJ TR.GDT 5/26/15

PROJECT: **DOVE HILL ASSISTED LIVING COMMUNITY**
4200 DOVE HILL ROAD
San Jose, California

Log of Boring B-2

Boring location: See Site Plan, Figure 2
Logged by: S. Magallon/R. Ward
Date started: 3/25/15 Date finished: 3/25/15
Drilling method: Hollow Stem Auger
Hammer weight/drop: 140 lbs./30 inches Hammer type: Rope & Cathead
Sampler: Sprague & Henwood (S&H), Standard Penetration Test (SPT)

DEPTH (feet)	Sampler Type	Blows/ 6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
					Ground Surface Elevation: 208.2 feet[2]						
0–1					2 inches topsoil						
1–4.5				CH	CLAY (CH) olive-gray, very stiff, moist						
3	S&H	12 17 21	23		LL = 77 PI = 50, see Figure C-4						
4.5–16.2					SERPENTINITE dark brown with olive, yellow-brown and olive green, moist, low hardness, friable, deeply weathered, soft, plastic, decomposed, pulverized to soil-like consistency						
6	S&H	14 21 23	26								
8	S&H	24 45 50/ 4"	57/ 10"								
10–11	SPT	21 43 41	84		frequent veinlets						
15–16	SPT	16 40 50/ 2.5"	90/ 8.5'		polished surfaces, highly sheared						

Boring terminated at a depth of 16.2 feet below ground surface.
Boring backfilled with cement grout.
Groundwater not encountered during drilling.

[1] S&H and SPT blow counts for the last two increments were converted to SPT N-Values using factors of 0.6 and 1.0, respectively to account for sampler type and hammer energy.
[2] Elevations based on NAVD 88.

LANGAN TREADWELL ROLLO

Project No.: 770619901 Figure: B-2

PROJECT: **DOVE HILL ASSISTED LIVING COMMUNITY**
4200 DOVE HILL ROAD
San Jose, California

Log of Boring B-3

Boring location: See Site Plan, Figure 2

Logged by: S. Magallon/R. Ward

Date started: 3/25/15 | Date finished: 3/25/15

Drilling method: Hollow Stem Auger

Hammer weight/drop: 140 lbs./30 inches | Hammer type: Rope & Cathead

Sampler: Sprague & Henwood (S&H), Standard Penetration Test (SPT)

Ground Surface Elevation: 186.9 feet[2]

DEPTH (feet)	Sampler Type	Blows/6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
0					2 inches Aggregate Bae (AB)						
1–2				CH	CLAY (CH) dark brown, very stiff, moist						
3	S&H	15, 16, 25	25	CH							
5–6	S&H	11, 13, 15	17	CH	LL = 74, PI = 55, see Figure C-4; TxUU Test, see Figure C-2	TxUU	720	5,140		19.8	107
8	S&H	16, 19, 29	29	CH							
8.3–9.5				CL	CLAY with SAND (CL) olive-brown, very stiff, moist, fine sand, trace coarse sand						
10	S&H	31, 50	30/6"		SERPENTINITE olive-brown with olive-green and yellow-brown, moist, low hardness, friable, deeply weathered, decomposed, breaks down to soil-like consistency, root and rootlets						
15	SPT	16, 22, 34	56								

Boring terminated at a depth of 16.5 feet below ground surface.
Boring backfilled with cement grout.
Groundwater not encountered during drilling.

[1] S&H and SPT blow counts for the last two increments were converted to SPT N-Values using factors of 0.6 and 1.0, respectively to account for sampler type and hammer energy.
[2] Elevations based on NAVD 88.

LANGAN TREADWELL ROLLO

Project No.: 770619901 | Figure: B-3

TEST GEOTECH LOG 770619901 EDIT PDF FILE #.GPJ TR.GDT 5/26/15

PROJECT:	DOVE HILL ASSISTED LIVING COMMUNITY 4200 DOVE HILL ROAD San Jose, California	Log of Boring B-4

Boring location: See Site Plan, Figure 2

Logged by: S. Magallon/R. Ward

Date started: 3/25/15 | Date finished: 3/25/15

Drilling method: Hollow Stem Auger

Hammer weight/drop: 140 lbs./30 inches | Hammer type: Rope & Cathead

Sampler: Sprague & Henwood (S&H), Standard Penetration Test (SPT)

DEPTH (feet)	Sampler Type	Sample	Blows/ 6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION Ground Surface Elevation: 188.7 feet[2]	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
					CL	CLAY (CL) brown, moist, trace fine sand, trace fine gravel						
1						SERPENTINIZED GREENSTONE olive to olive-brown, low hardness, friable, deeply weathered						
2	S&H		38 50/ 3"	30/ 3"								
3												
4												
5	SPT		50/ 4.5"	50/ 4.5"								
6						moderately weathered, zones of moderately strong and moderately hard greenstone						
7												
8	SPT		50/ 5"	50/ 5"								
9												
10	SPT		39 50/ 4"	50/ 4"								
11												
12												
13												
14												
15												
16												
17												
18												
19												
20												
21												
22												
23												
24												
25												
26												
27												
28												
29												
30												

Boring terminated at a depth of 10.8 feet below ground surface.
Boring backfilled with cement grout.
Groundwater not encountered during drilling.

[1] S&H and SPT blow counts for the last two increments were converted to SPT N-Values using factors of 0.6 and 1.0, respectively to account for sampler type and hammer energy.
[2] Elevations based on NAVD 88.

LANGAN TREADWELL ROLLO

Project No.: 770619901 | Figure: B-4

TEST GEOTECH LOG 770619901 EDIT PDF FILE #.GPJ TR.GDT 5/26/15

PROJECT: **DOVE HILL ASSISTED LIVING COMMUNITY**
4200 DOVE HILL ROAD
San Jose, California

Log of Boring B-5

Boring location: See Site Plan, Figure 2

Logged by: S. Magallon/R. Ward

Date started: 3/25/15 Date finished: 3/25/15

Drilling method: Hollow Stem Auger

Hammer weight/drop: 140 lbs./30 inches Hammer type: Rope & Cathead

Sampler: Sprague & Henwood (S&H), Standard Penetration Test (SPT)

LABORATORY TEST DATA

DEPTH (feet)	Sampler Type	Sample	Blows/ 6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION — Ground Surface Elevation: 185.7 feet[2]	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
0–2	BULK					SERPENTINITE olive-green with tan, white, dark green, low hardness, friable, deeply weathered, bedrock breaks down to soil-like consistency\						
2.5	S&H		50/ 5"	30/ 5"		R-Value Test, see Figure C-6						
5	S&H		50/ 4.5"	30/ 4.5"		Particle Size Analysis, see Figure C-5				29.6	20.4	
6.5						SERPENTINITE olive with yellow-brown mottling, moist, low hardness, friable, deeply weathered, moderately hard, greenstone inclusions, oxidized, pulverized to soil-like consistency						
8	SPT		19 36 30	66								
10.5	SPT		18 38 50/ 5.5"	88/ 11.5"		decrease in greenstone fragments						
15.5	SPT		18 22 32	54								
18						MELANGE dark gray to dark blue-green, glauconitic clay, low hardness, friable, completely sheared to soil-like consistency, polished surfaces, moist						
20.5	SPT		20 22 28	50								
22.5	SPT		26 20 28	48		decrease in moisture, white seams						

Boring terminated at a depth of 23.5 feet below ground surface.
Boring backfilled with cement grout.
Groundwater not encountered during drilling.

[1] S&H and SPT blow counts for the last two increments were converted to SPT N-Values using factors of 0.6 and 1.0, respectively to account for sampler type and hammer energy.
[2] Elevations based on NAVD 88.

LANGAN TREADWELL ROLLO

Project No.: 770619901

Figure: B-5

TEST GEOTECH LOG 770619901 EDIT PDF FILE #.GPJ TR.GDT 5/26/15

PROJECT:	DOVE HILL ASSISTED LIVING COMMUNITY 4200 DOVE HILL ROAD San Jose, California	Log of Boring B-6 PAGE 1 OF 1

Boring location: See Site Plan, Figure 2

Logged by: S. Magallon/R. Ward

Date started: 3/25/15 | Date finished: 3/25/15

Drilling method: Hollow Stem Auger

Hammer weight/drop: 140 lbs./30 inches | Hammer type: Rope & Cathead

Sampler: Sprague & Henwood (S&H), Standard Penetration Test (SPT)

DEPTH (feet)	Sampler Type	Sample	Blows/ 6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION Ground Surface Elevation: 209.5 feet[2]	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
0						2 inches topsoil						
1						CLAY (CH) dark brown, very stiff, moist						
3	S&H		12 13 21	20	CH	LL = 83, PI = 52, see Figure C-4					34.9	86
5	S&H		17 50/ 6"	30/ 6"	CL	CLAY with SAND (CL) olive-brown, hard, moist, fine to medium sand, trace fine angular gravel						
7						SERPENTINITE light olive-brown, green, low hardness, friable, deeply weathered, talc seams, sheared fabric						
8	S&H		44 50/ 4.5"	30/ 4.5"								
10	SPT		40 50/ 6"	50/ 6"		completely weathered to soil-like consistency						
12	SPT		50/ 4"	50/ 4"								

Boring terminated at a depth of 12.3 feet below ground surface.
Boring backfilled with cement grout.
Groundwater not encountered during drilling.

[1] S&H and SPT blow counts for the last two increments were converted to SPT N-Values using factors of 0.6 and 1.0, respectively to account for sampler type and hammer energy.
[2] Elevations based on NAVD 88.

LANGAN TREADWELL ROLLO

Project No.: 770619901 | Figure: B-6

PROJECT:	DOVE HILL ASSISTED LIVING COMMUNITY 4200 DOVE HILL ROAD San Jose, California	Log of Boring B-7 PAGE 1 OF 1

Boring location: See Site Plan, Figure 2

Logged by: S. Magallon/R. Ward

Date started: 3/25/15 | Date finished: 3/25/15

Drilling method: Hollow Stem Auger

Hammer weight/drop: 140 lbs./30 inches | Hammer type: Rope & Cathead

Sampler: Sprague & Henwood (S&H), Standard Penetration Test (SPT)

DEPTH (feet)	SAMPLES: Sampler Type	Sample	Blows/ 6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION Ground Surface Elevation: 210 feet[2]	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
						2 inches Aggregate Base (AB)						
1					CH	CLAY (CH) dark brown, very stiff, moist, trace- to coarse sand						
2												
3	S&H		10 12 15	16								
4						TxUU Test, see Figure C-3	TxUU	420	1,900		27.9	93
5	S&H		33 50/ 3"	30/ 3"		SERPENTINITE olive-gray with olive and yellow-brown mottling, soft, friable, deeply weathered, white talc seams						
6												
7						SANDSTONE and SHALE olive to gray sandstone, scattered mica grains, low hardness, friable to weak, dark grayish-brown, friable shale, moderately indurated, clast of hard, very fine-grained sandstone, somewhat serpentinized near contact with serpentinite						
8	SPT		41 50/ 4.5"	50/ 4.5"								
9												
10	SPT		50/ 5.5"	50/ 5.5"								
11												
12												
13												
14												
15												
16												
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28												
29												
30												

Boring terminated at a depth of 10.5 feet below ground surface.
Boring backfilled with cement grout.
Groundwater not encountered during drilling.

[1] S&H and SPT blow counts for the last two increments were converted to SPT N-Values using factors of 0.6 and 1.0, respectively to account for sampler type and hammer energy.
[2] Elevations based on NAVD 88.

LANGAN TREADWELL ROLLO

Project No.: 770619901 | Figure: B-7

TEST GEOTECH LOG 770619901 EDIT PDF FILE #.GPJ TR.GDT 5/26/15

PROJECT:	DOVE HILL ASSISTED LIVING COMMUNITY 4200 DOVE HILL ROAD San Jose, California	Log of Boring B-8

Boring location:	See Site Plan, Figure 2
Logged by:	S. Magallon/R. Ward
Date started:	3/25/15
Date finished:	3/25/15
Drilling method:	Hollow Stem Auger
Hammer weight/drop:	140 lbs./30 inches
Hammer type:	Rope & Cathead
Sampler:	Sprague & Henwood (S&H), Standard Penetration Test (SPT)

Ground Surface Elevation: 186.5 feet[2]

DEPTH (feet)	Sampler Type	Sample	Blows/ 6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
0						2 inches Aggregate Base (AB)						
1					CL	SANDY CLAY (CL) brown, moist, fine sand						
2						SERPENTINIZED GREENSTONE olive-brown, low hardness, weak, deeply weathered, zones of moderately hard, moderately strong greenstone, white veining (unidentified mineral), quartz veins, manganese staining						
3	S&H		50/ 3"	30/ 3"								
5	SPT		50/ 5"	50/ 5"								
8	SPT		50/ 4"	50/ 4"								
10						▽ (03/25/15, 5:05 p.m.)						
12						hard, strong, wet						
12.5	SPT		50/ 1.5"	50/ 1.5"		▽ (03/25/15, 4:50 p.m.)						
13–30												

Boring terminated at a depth of 12.5 feet below ground surface.
Boring backfilled with cement grout.
Groundwater encountered at 12.5 feet below ground surface during drilling.

[1] S&H and SPT blow counts for the last two increments were converted to SPT N-Values using factors of 0.6 and 1.0, respectively to account for sampler type and hammer energy.
[2] Elevations based on NAVD 88.

LANGAN TREADWELL ROLLO

Project No.: 770619901

Figure: B-8

TEST GEOTECH LOG 770619901 EDIT PDF FILE #.GPJ TR.GDT 5/26/15

UNIFIED SOIL CLASSIFICATION SYSTEM

Major Divisions		Symbols	Typical Names
Coarse-Grained Soils (more than half of soil > no. 200 sieve size	**Gravels** (More than half of coarse fraction > no. 4 sieve size)	**GW**	Well-graded gravels or gravel-sand mixtures, little or no fines
		GP	Poorly-graded gravels or gravel-sand mixtures, little or no fines
		GM	Silty gravels, gravel-sand-silt mixtures
		GC	Clayey gravels, gravel-sand-clay mixtures
	Sands (More than half of coarse fraction < no. 4 sieve size)	**SW**	Well-graded sands or gravelly sands, little or no fines
		SP	Poorly-graded sands or gravelly sands, little or no fines
		SM	Silty sands, sand-silt mixtures
		SC	Clayey sands, sand-clay mixtures
Fine -Grained Soils (more than half of soil < no. 200 sieve size)	**Silts and Clays** LL = < 50	**ML**	Inorganic silts and clayey silts of low plasticity, sandy silts, gravelly silts
		CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, lean clays
		OL	Organic silts and organic silt-clays of low plasticity
	Silts and Clays LL = > 50	**MH**	Inorganic silts of high plasticity
		CH	Inorganic clays of high plasticity, fat clays
		OH	Organic silts and clays of high plasticity
Highly Organic Soils		**PT**	Peat and other highly organic soils

GRAIN SIZE CHART

Classification	Range of Grain Sizes: U.S. Standard Sieve Size	Range of Grain Sizes: Grain Size in Millimeters
Boulders	Above 12"	Above 305
Cobbles	12" to 3"	305 to 76.2
Gravel	3" to No. 4	76.2 to 4.76
coarse	3" to 3/4"	76.2 to 19.1
fine	3/4" to No. 4	19.1 to 4.76
Sand	No. 4 to No. 200	4.76 to 0.075
coarse	No. 4 to No. 10	4.76 to 2.00
medium	No. 10 to No. 40	2.00 to 0.420
fine	No. 40 to No. 200	0.420 to 0.075
Silt and Clay	Below No. 200	Below 0.075

Unstabilized groundwater level

Stabilized groundwater level

SAMPLE DESIGNATIONS/SYMBOLS

Sample taken with Sprague & Henwood split-barrel sampler with a 3.0-inch outside diameter and a 2.43-inch inside diameter. Darkened area indicates soil recovered

Classification sample taken with Standard Penetration Test sampler

Undisturbed sample taken with thin-walled tube

Disturbed sample

Sampling attempted with no recovery

Core sample

Analytical laboratory sample, grab groundwater

Sample taken with Direct Push sampler

Sonic

SAMPLER TYPE

C	Core barrel
CA	California split-barrel sampler with 2.5-inch outside diameter and a 1.93-inch inside diameter
D&M	Dames & Moore piston sampler using 2.5-inch outside diameter, thin-walled tube
O	Osterberg piston sampler using 3.0-inch outside diameter, thin-walled Shelby tube
PT	Pitcher tube sampler using 3.0-inch outside diameter, thin-walled Shelby tube
S&H	Sprague & Henwood split-barrel sampler with a 3.0-inch outside diameter and a 2.43-inch inside diameter
SPT	Standard Penetration Test (SPT) split-barrel sampler with a 2.0-inch outside diameter and a 1.5-inch inside diameter
ST	Shelby Tube (3.0-inch outside diameter, thin-walled tube) advanced with hydraulic pressure

DOVE HILL ASSISTED LIVING COMMUNITY
4200 DOVE HILL ROAD
San Jose, California

LANGAN TREADWELL ROLLO

CLASSIFICATION CHART

Date 04/01/15 | Project No. 770619901 | Figure B-9

I FRACTURING

Intensity	Size of Pieces in Feet
Very little fractured	Greater than 4.0
Occasionally fractured	1.0 to 4.0
Moderately fractured	0.5 to 1.0
Closely fractured	0.1 to 0.5
Intensely fractured	0.05 to 0.1
Crushed	Less than 0.05

II HARDNESS

1. **Soft** - reserved for plastic material alone.
2. **Low hardness** - can be gouged deeply or carved easily with a knife blade.
3. **Moderately hard** - can be readily scratched by a knife blade; scratch leaves a heavy trace of dust and is readily visible after the powder has been blown away.
4. **Hard** - can be scratched with difficulty; scratch produced a little powder and is often faintly visible.
5. **Very hard** - cannot be scratched with knife blade; leaves a metallic streak.

III STRENGTH

1. **Plastic** or very low strength.
2. **Friable** - crumbles easily by rubbing with fingers.
3. **Weak** - an unfractured specimen of such material will crumble under light hammer blows.
4. **Moderately strong** - specimen will withstand a few heavy hammer blows before breaking.
5. **Strong** - specimen will withstand a few heavy ringing hammer blows and will yield with difficulty only dust and small flying fragments.
6. **Very strong** - specimen will resist heavy ringing hammer blows and will yield with difficulty only dust and small flying fragments.

IV WEATHERING - The physical and chemical disintegration and decomposition of rocks and minerals by natural processes such as oxidation, reduction, hydration, solution, carbonation, and freezing and thawing.

D. Deep - moderate to complete mineral decomposition; extensive disintegration; deep and thorough discoloration; many fractures, all extensively coated or filled with oxides, carbonates and/or clay or silt.

M. Moderate - slight change or partial decomposition of minerals; little disintegration; cementation little to unaffected. Moderate to occasionally intense discoloration. Moderately coated fractures.

L. Little - no megascopic decomposition of minerals; little of no effect on normal cementation. Slight and intermittent, or localized discoloration. Few stains on fracture surfaces.

F. Fresh - unaffected by weathering agents. No disintegration of discoloration. Fractures usually less numerous than joints.

ADDITIONAL COMMENTS:

V CONSOLIDATION OF SEDIMENTARY ROCKS: usually determined from unweathered samples. Largely dependent on cementation.

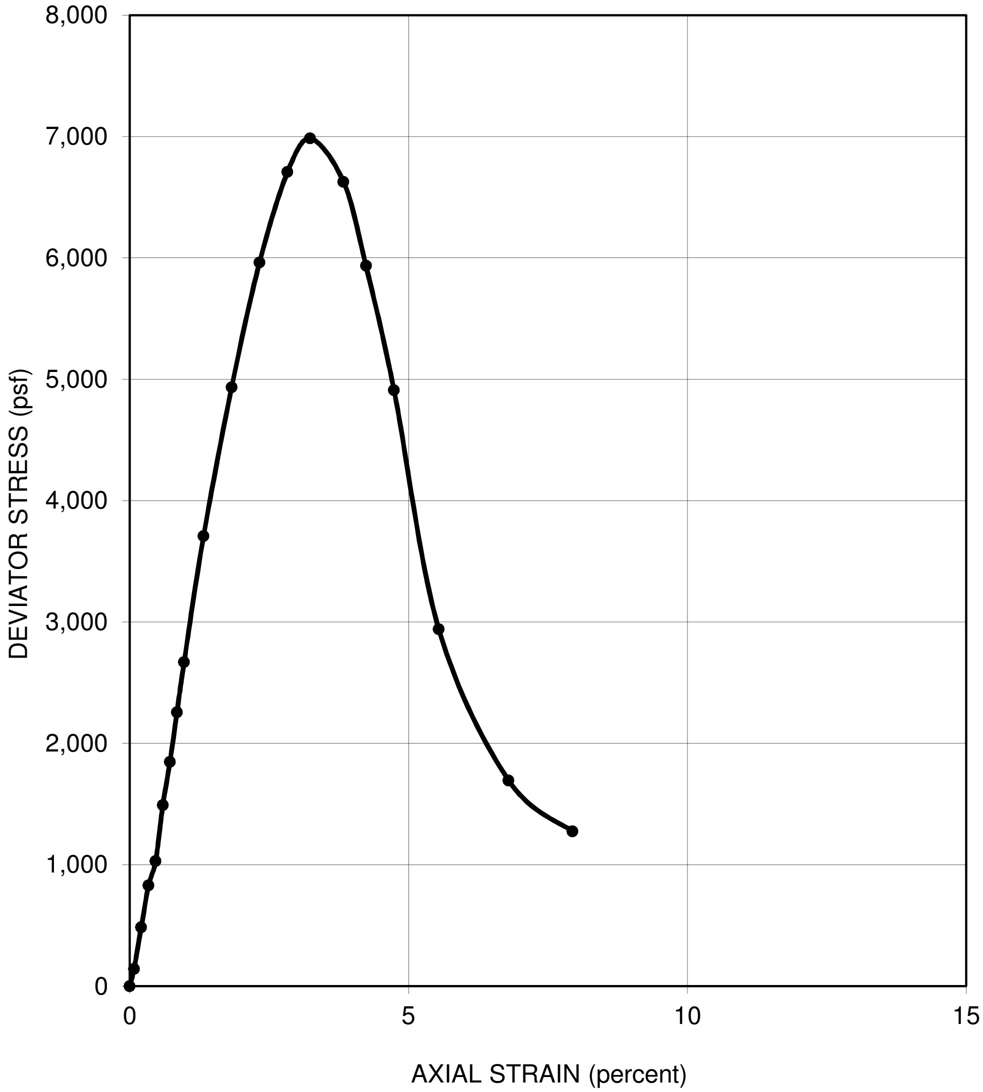
U = unconsolidated
P = poorly consolidated
M = moderately consolidated
W = well consolidated

VI BEDDING OF SEDIMENTARY ROCKS

Splitting Property	Thickness	Stratification
Massive	Greater than 4.0 ft.	very thick-bedded
Blocky	2.0 to 4.0 ft.	thick bedded
Slabby	0.2 to 2.0 ft.	thin bedded
Flaggy	0.05 to 0.2 ft.	very thin-bedded
Shaly or platy	0.01 to 0.05 ft.	laminated
Papery	less than 0.01	thinly laminated

APPENDIX C

LABORATORY DATA



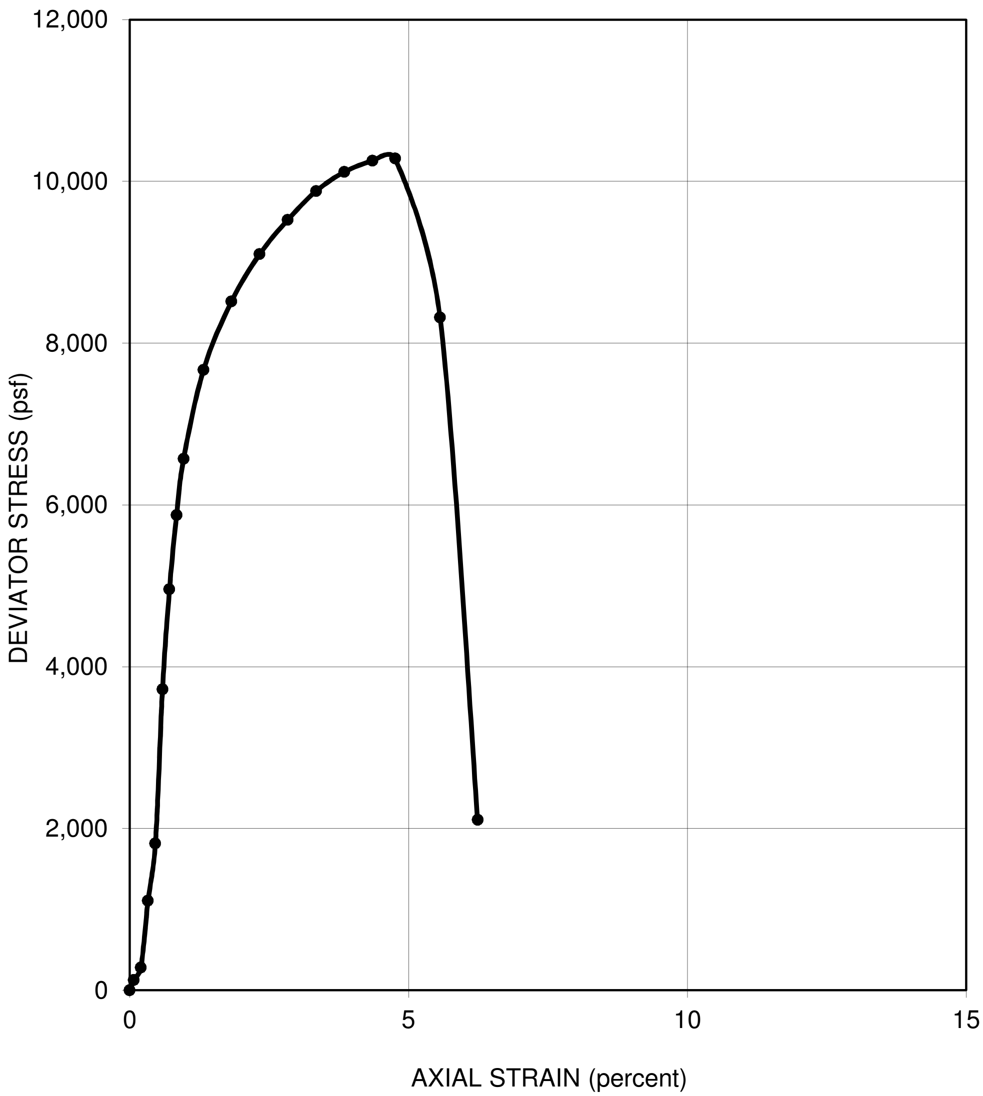
SAMPLER TYPE	Sprague & Henwood			SHEAR STRENGTH	3,490	psf
DIAMETER (in.)	2.4	HEIGHT (in.) 6.0		STRAIN AT FAILURE	3.2	%
MOISTURE CONTENT		20.0	%	CONFINING PRESSURE	420	psf
DRY DENSITY		100	pcf	STRAIN RATE	0.50	% / min
DESCRIPTION	CLAY (CH), olive-gray				SOURCE	B-1 at 3.5 feet

DOVE HILL ASSISTED LIVING COMMUNITY 4200 DOVE HILL ROAD
San Jose, California

UNCONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TEST

LANGAN TREADWELL ROLLO

Date: 04/07/15 | Project: 770619901 | Figure: C-1



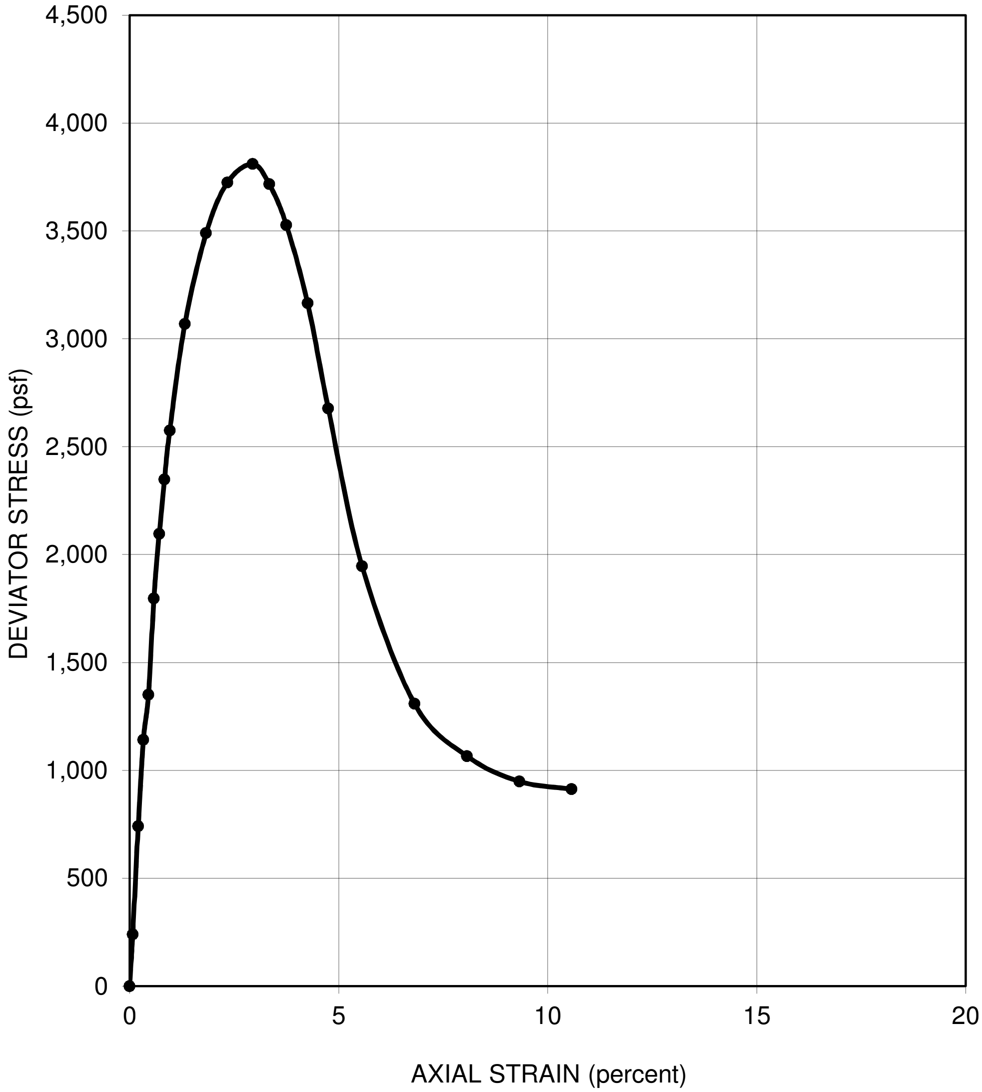
SAMPLER TYPE	Sprague & Henwood			SHEAR STRENGTH	5,140	psf
DIAMETER (in.)	2.4	HEIGHT (in.) 5.7		STRAIN AT FAILURE	4.8	%
MOISTURE CONTENT		19.8	%	CONFINING PRESSURE	720	psf
DRY DENSITY		107	pcf	STRAIN RATE	0.75	% / min
DESCRIPTION	CLAY (CH), dark brown				SOURCE	B-3 at 6 feet

DOVE HILL ASSISTED LIVING COMMUNITY 4200 DOVE HILL ROAD
San Jose, California

UNCONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TEST

LANGAN TREADWELL ROLLO

Date: 04/07/15 | Project: 770619901 | Figure: C-2



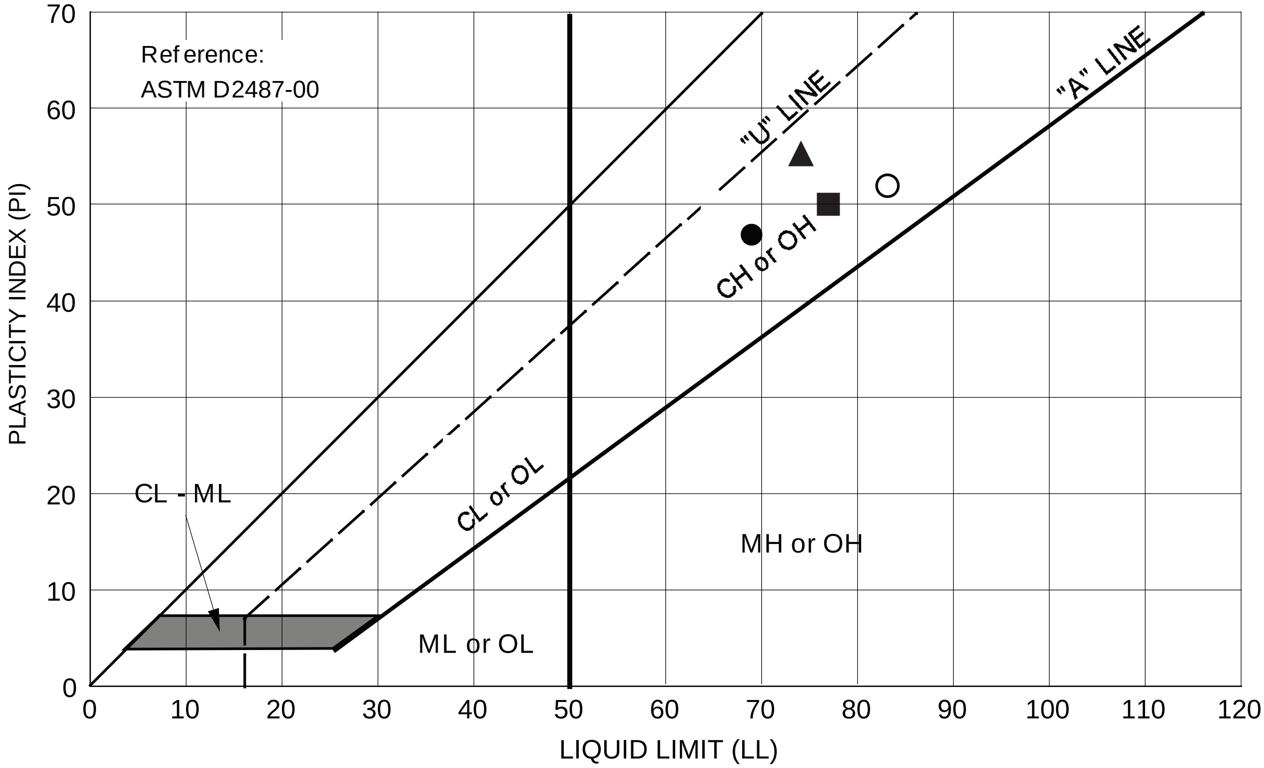
SAMPLER TYPE	Sprague & Henwood			SHEAR STRENGTH	1,900	psf
DIAMETER (in.)	2.4	HEIGHT (in.)	5.7	STRAIN AT FAILURE	2.9	%
MOISTURE CONTENT		27.9	%	CONFINING PRESSURE	420	psf
DRY DENSITY		93	pcf	STRAIN RATE	0.75	% / min
DESCRIPTION	CLAY (CH), dark brown				SOURCE	B-7 at 3.5 feet

DOVE HILL ASSISTED LIVING COMMUNITY 4200 DOVE HILL ROAD
San Jose, California

UNCONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TEST

LANGAN TREADWELL ROLLO

Date: 04/07/15	Project: 770619901	Figure: C-3



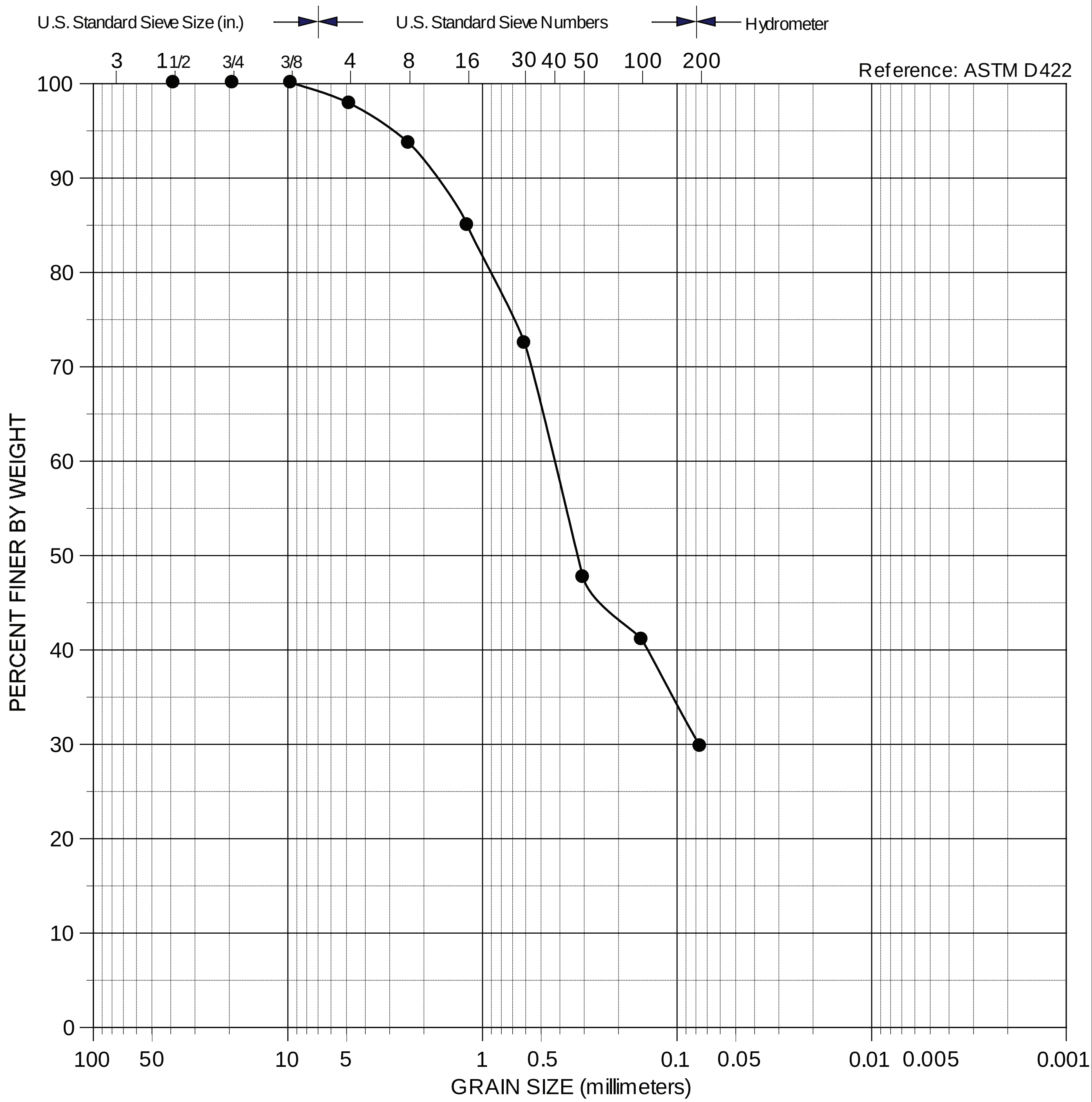
Symbol	Source	Description and Classification	Natural M.C. (%)	Liquid Limit (%)	Plasticity Index (%)	% Passing #200 Sieve
●	B-1 at 3.5 feet	CLAY (CH), olive-gray	20.0	69	47	--
■	B-2 at 2.5 feet	CLAY (CH), olive-gray	--	77	50	--
▲	B-3 at 6 feet	CLAY (CH), dark brown	19.8	74	55	--
○	B-6 at 3.5 feet	CLAY (CH), dark brown	--	83	52	--

DOVE HILL ASSISTED LIVING COMMUNITY
San Jose, California

LANGAN TREADWELL ROLLO

PLASTICITY CHART

Date 04/17/15 | Project No. 770619901 | Figure C-4



% Gravel		%Sand			% Fines	
Coarse	Fine	Coarse	Medium	Fine	Silt	Clay

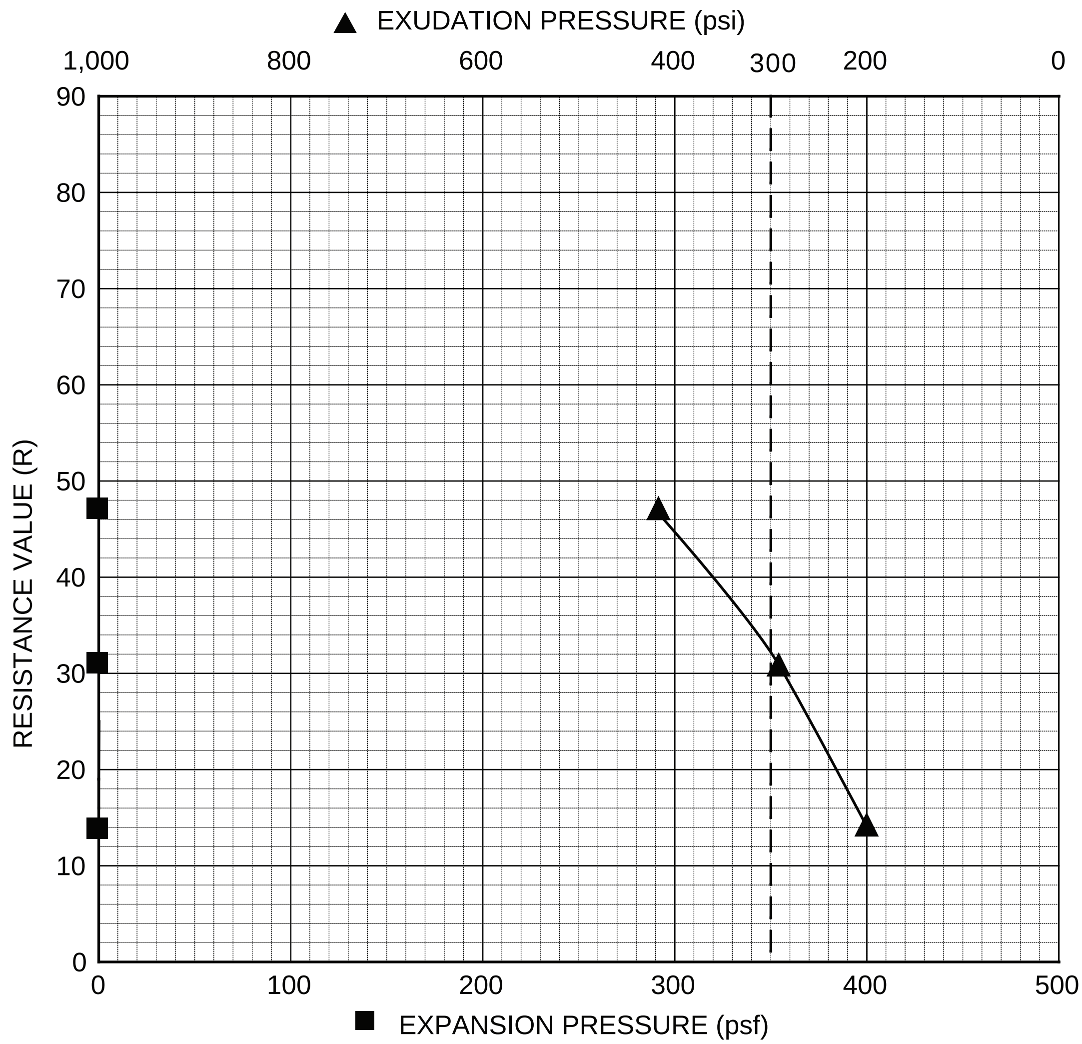
Symbol	Sample Source	Classification
●	B-5 at 5 feet	SERPENTINITE, olive-green with tan, white, dark green

DOVE HILL ASSISTED LIVING COMMUNITY
San Jose, California

LANGAN TREADWELL ROLLO

PARTICLE SIZE ANALYSIS

Date 04/17/15	Project No. 770619901	Figure C-5



Specimen ID:	A	B	C	D
Water Content (%)	23.1	22.1	21.2	--
Dry Density (pcf)	99.1	101.0	101.8	--
Exudation Pressure (psi)	199	296	423	--
Expansion Pressure (psf)	0.00	0.00	0.00	--
Resistance Value (R)	14	31	47	--

Sample Source	Sample Description	Sand Equivalent	Expansion Pressure	R value
B-5 at 0-2.5 feet	SERPENTINITE, olive-green with tan, white, dark green	--	--	32

DOVE HILL ASSISTED LIVING COMMUNITY
San Jose, California

LANGAN TREADWELL ROLLO

RESISTANCE VALUE TEST DATA

Date 04/17/15 | Project No. 770619901 | Figure C-6

APPENDIX D

CORROSIVITY ANALYSES WITH BRIEF EVALUATION

California State Certified Laboratory No. 2153

CERCO analytical
1100 Willow Pass Court, Suite A
Concord, CA 94520-1006
925 **462 2771** Fax. 925 **462 2775**
www.cercoanalytical.com

10 April, 2015

Job No. 1504008
Cust. No.10727

Mr. Matt Lattin
Langan Treadwell Rollo
555 Montgomery Street, Suite 1300
San Francisco, CA 94111

Subject: Project No.: 770619901.700.310
Project Name: Dove Hill
Corrosivity Analysis – ASTM Methods

Dear Mr. Lattin:

Pursuant to your request, CERCO Analytical has analyzed the soil samples submitted on April 01, 2015. Based on the analytical results, this brief corrosivity evaluation is enclosed for your consideration.

Based upon the resistivity measurements, both samples are classified as "corrosive". All buried iron, steel, cast iron, ductile iron, galvanized steel and dielectric coated steel or iron should be properly protected against corrosion depending upon the critical nature of the structure. All buried metallic pressure piping such as ductile iron firewater pipelines should be protected against corrosion.

The chloride ion concentrations ranged from 19 to 22 mg/kg and are determined to be insufficient to attack steel embedded in a concrete mortar coating.

The sulfate ion concentrations ranged from 79 to 86 mg/kg and are determined to be insufficient to damage reinforced concrete structures and cement mortar-coated steel at these locations.

The pH of the soils ranged from 8.25 to 8.26, which does not present corrosion problems for buried iron, steel, mortar-coated steel and reinforced concrete structures.

The redox potentials ranged from 330 to 340-mV, which are indicative of potentially "slightly corrosive" soils resulting from anaerobic soil conditions.

This corrosivity evaluation is based on general corrosion engineering standards and is non-specific in nature. For specific long-term corrosion control design recommendations or consultation, please call *JDH Corrosion Consultants, Inc. at (925) 927-6630.*

We appreciate the opportunity of working with you on this project. If you have any questions, or if you require further information, please do not hesitate to contact us.

Very truly yours,
CERCO ANALYTICAL, INC.

Cheryl McMillen for

J. Darby Howard, Jr., P.E.
President

JDH/jdl
Enclosure

California State Certified Laboratory No. 2153

CERCO analytical
1100 Willow Pass Court, Suite A
Concord, CA 94520-1006
925 462 2771 Fax. 925 462 2775
www.cercoanalytical.com

Client: Langan Treadwell Rollo
Client's Project No.: 770619901.700.310
Client's Project Name: Dove Hill
Date Sampled: 25-Mar-15
Date Received: 1-Apr-15
Matrix: Soil
Authorization: Chain of Custody

Date of Report: 10-Apr-2015

Job/Sample No.	Sample I.D.	Redox (mV)	pH	Conductivity (umhos/cm)*	Resistivity (100% Saturation) (ohms-cm)	Sulfide (mg/kg)*	Chloride (mg/kg)*	Sulfate (mg/kg)*
1504008-001	B-1-2 @ 6'	330	8.25	-	780	-	19	79
1504008-002	B-3-1 @ 3'	340	8.26	-	660	-	22	86
Method:		ASTM D1498	ASTM D4972	ASTM D1125M	ASTM G57	ASTM D4658M	ASTM D4327	ASTM D4327
Reporting Limit:		-	-	10	-	50	15	15
Date Analyzed:		8-Apr-2015	8-Apr-2015	-	6-Apr-2015	-	7-Apr-2015	7-Apr-2015

* Results Reported on "As Received" Basis

N.D. - None Detected

Cheryl McMillen

Cheryl McMillen
Laboratory Director

Quality Control Summary - **All laboratory quality control parameters were found to be within established limits**

1504008 10727

Chain of Custody

1100 Willow Pass Court
Concord, CA 94520-1006
925 462 2771
Fax: 925 462 2775
CERCO analytical

Job No.	CU#		Client Project I.D.	Schedule	Analyte	Date Sampled	Date Due
77061990l	700	310	Dove Hill			3/25/15	standard

Full Name: Matt Lattin
Langan Treadwell Rollow
Company: San Jose
email: mlattin@langan.com
Fax: sjang@langan.com
Phone: 415-955-5266
Cell ☒
Sample Source

ASTM w/Brief Evaluation — ANALYSIS

Lab No.	Sample I.D.	Date	Time	Matrix	Contain.	Size	Preserv.	Qty.	Redox Potential	pH	Sulfate	Chloride	Resistivity-100% Saturated		Brief Evaluation							
001	B-1-2-6ft	3/25/15		S					x	x	x	x	x		x							
002	~~B-2~~ B-3-1-3ft	3/25/15		S					x	x	x	x	x		x							

MATRIX
DW - Drinking Water
GW - Ground Water
SW - Surface Water
WW - Waste Water
Water
SL - Sludge
S - Soil
Product

ABBREVIATIONS
HB - Hosebib
PV - Petcock Valve
PT - Pressure Tank
PH - Pump House
RR - Restroom
GL - Glass
PL - Plastic
ST - Sterile

SAMPLE RECEIPT
Total No. of Containers
Rec'd Good Cond/Cold
Conforms to Record
Temp. at Lab -°C
Sampler

	Date	Time
Relinquished By: [signature]	3/31/15	2pm
Received By: [signature]	4/1/15	1400
Relinquished By:		
Received By:		
Relinquished By:		
Received By:		

Comments:
THERE IS AN ADDITIONAL CHARGE FOR METAL/POLY TUBES

APPENDIX E

ASBESTOS ANALYSIS RESULTS

Forensic Analytical Laboratories

Final Report

Bulk Asbestos Material Analysis

(Air Resources Board Method 435, June 6, 1991)

McCampbell Analytical, Inc.
Account Payable
1534 Wilow Pass Rd

Pittsburg, CA 94565

Client ID: A31409
Report Number: N007055
Date Received: 04/02/15
Date Analyzed: 04/09/15
Date Printed: 04/09/15

Job ID/Site: 1504040 - 770619901; 700 310 Dave Hill

PLM Report Number: N/A

FALI Job ID: A31409
Total Samples Submitted: 3
Total Samples Analyzed: 3

Sample Preparation and Analysis:

Samples were analyzed by the Air Resources Board's Method 435, Determination of Asbestos Content of Serpentine Aggregate. Samples were ground to 200 particle size in the laboratory. Approximately 1 pint was retained for analysis. Samples were prepared for observation according to the guidelines of Exception I and Exception II as defined by the 435 Method. Samples which contained less than 10% asbestos were prepared for observation according to the point count technique as defined by the 435 Method. This analysis was performed with a standard cross-hair reticle.

Sample ID	Lab Number	Layer Description
B-1-5-15ft	11626379	**Grey Soil**

Visual Estimation Results:

Matrix percentage of entire: 100
Visual estimation percentage: **None Detected**
Asbestos type(s) detected: None Detected

Comment: This result meets the requirements of Exception I as defined by the 435 Method.

Sample ID	Lab Number	Layer Description
B-4-4-10ft	11626380	**Grey Soil**

Visual Estimation Results:

Matrix percentage of entire: 100
Visual estimation percentage: **None Detected**
Asbestos type(s) detected: None Detected

Comment: This result meets the requirements of Exception I as defined by the 435 Method.

Sample ID	Lab Number	Layer Description
B-6-7.5ft	11626381	**Grey Soil**

Visual Estimation Results:

Matrix percentage of entire: 100
Visual estimation percentage: **None Detected**
Asbestos type(s) detected: None Detected

Comment: This result meets the requirements of Exception I as defined by the 435 Method.

Tad Thrower

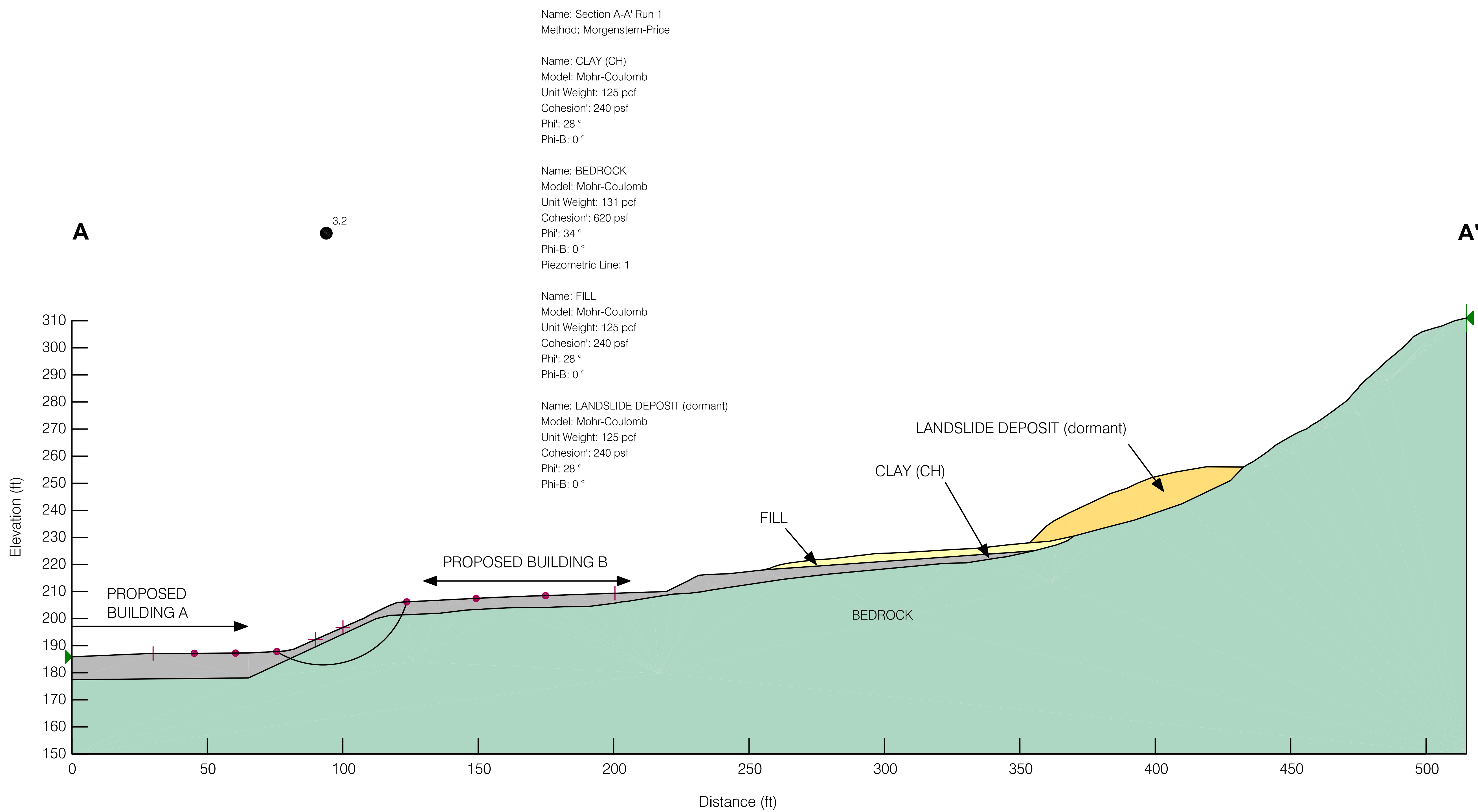
Tad Thrower, Laboratory Supervisor, Hayward Laboratory

Note: Limit of Quantification (LOQ) = 0.25%. Trace denotes the presence of asbestos below the LOQ. ND = None Detected.

3777 Depot Road, Suite 409, Hayward, CA 94545 / Telephone: (510) 887-8828 (800) 827-FASI / Fax: (510) 887-4218

APPENDIX F

SLOPE STABILITY EVALUATIONS



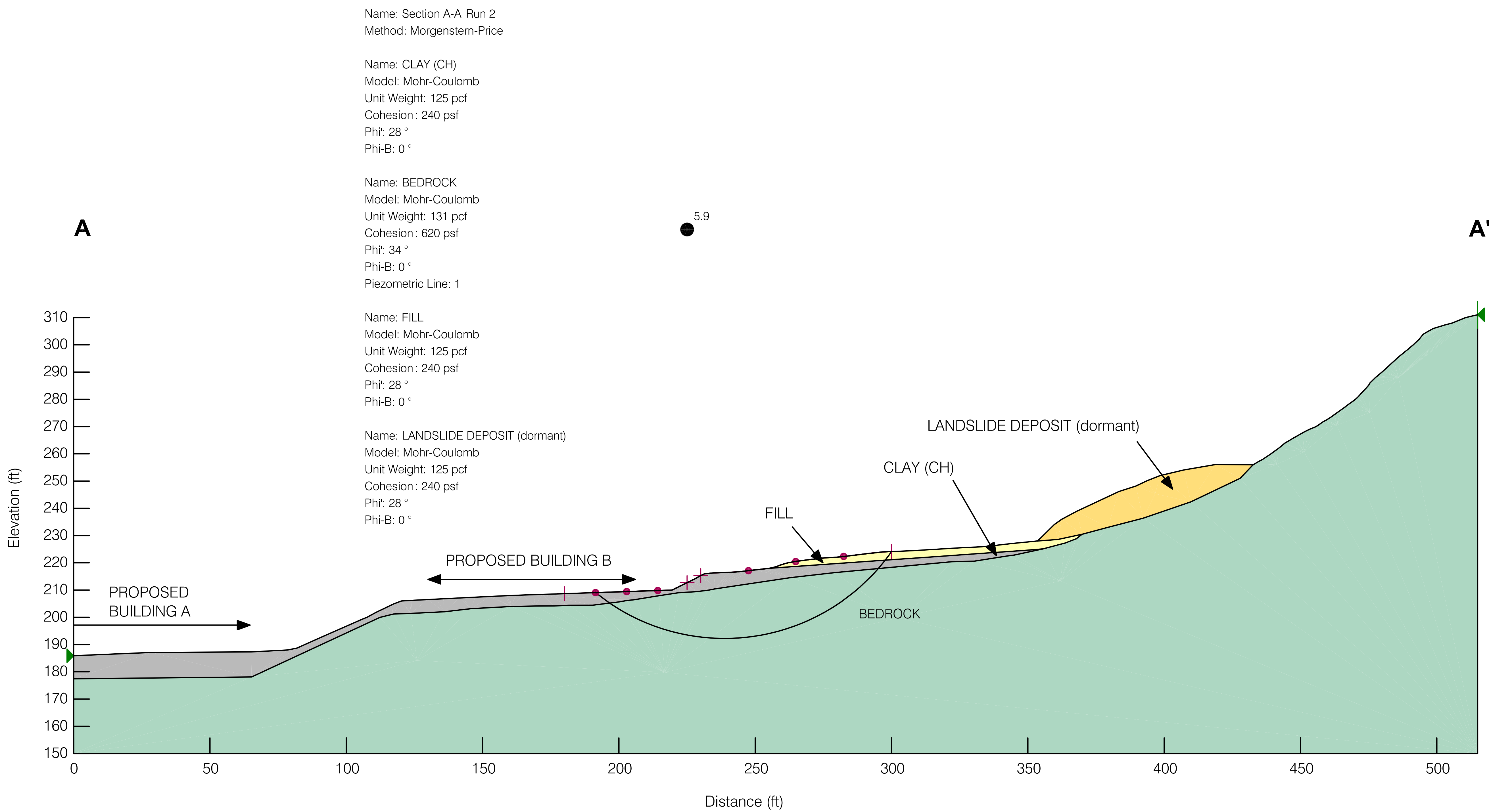
DOVE HILL ASSISTED LIVING COMMUNITY
4200 DOVE HILL ROAD
San Jose, California

SLOPE STABILITY ANALYSIS A-A'
EXISTING CONDITIONS - LOWER SLOPE

Date 05/07/15	Project No. 770619901	Figure F-1

LANGAN TREADWELL ROLLO

\\langan.com\data\SJ\data9\770619901\Cadd Data - 770619901\2D-DesignFiles\Geotechnical\770619901-B-PF0103.dwg 5/26/15



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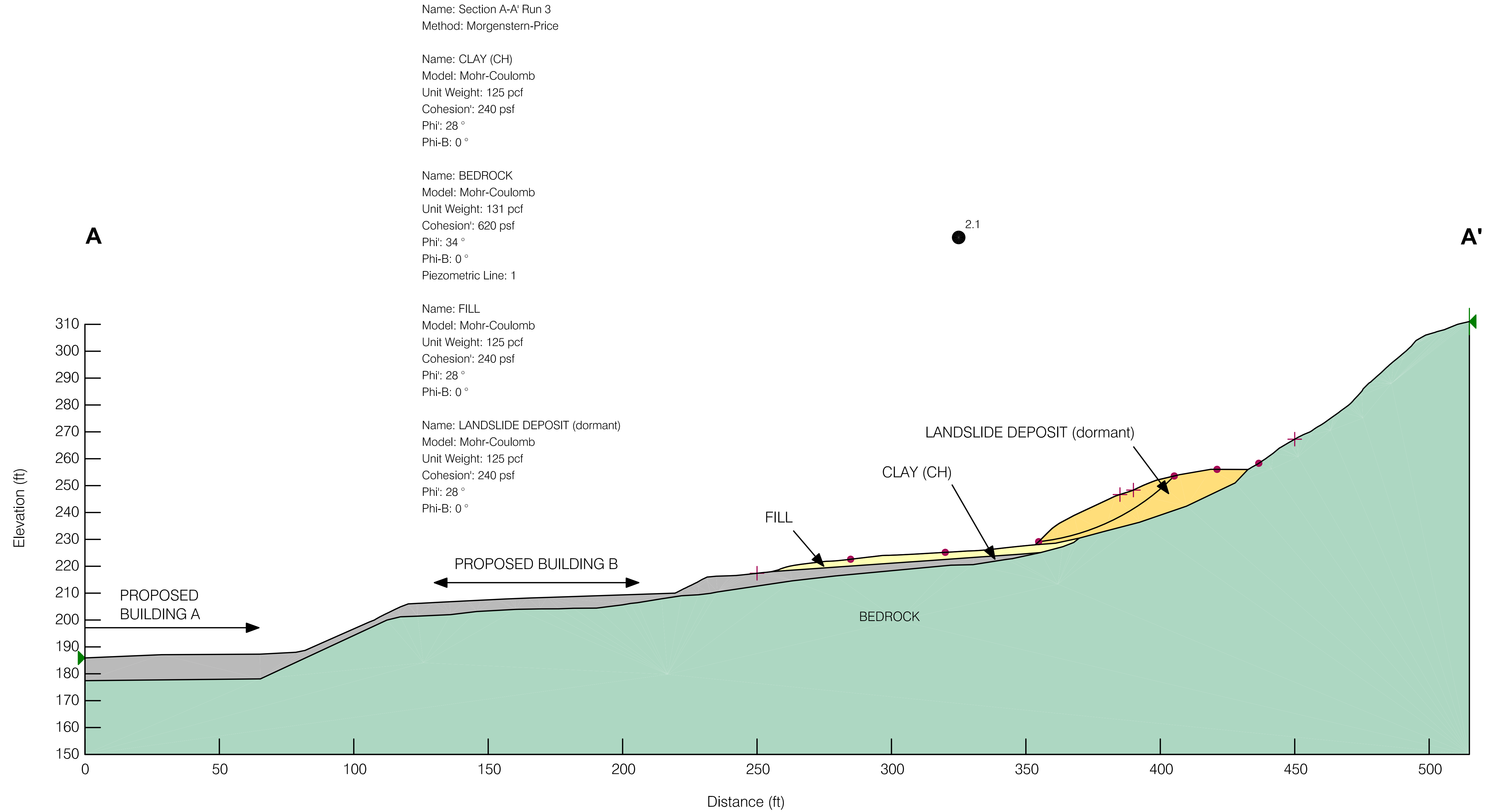
4200 DOVE HILL ROAD
San Jose, California

SLOPE STABILITY ANALYSIS A-A'
EXISTING CONDITIONS - MIDDLE SLOPE

Date 05/07/15	Project No. 770619901	Figure F-2

LANGAN TREADWELL ROLLO



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4200 DOVE HILL ROAD
San Jose, California

SLOPE STABILITY ANALYSIS A-A'
EXISTING CONDITIONS - UPPER SLOPE

Date 05/07/15	Project No. 770619901	Figure F-3

LANGAN TREADWELL ROLLO

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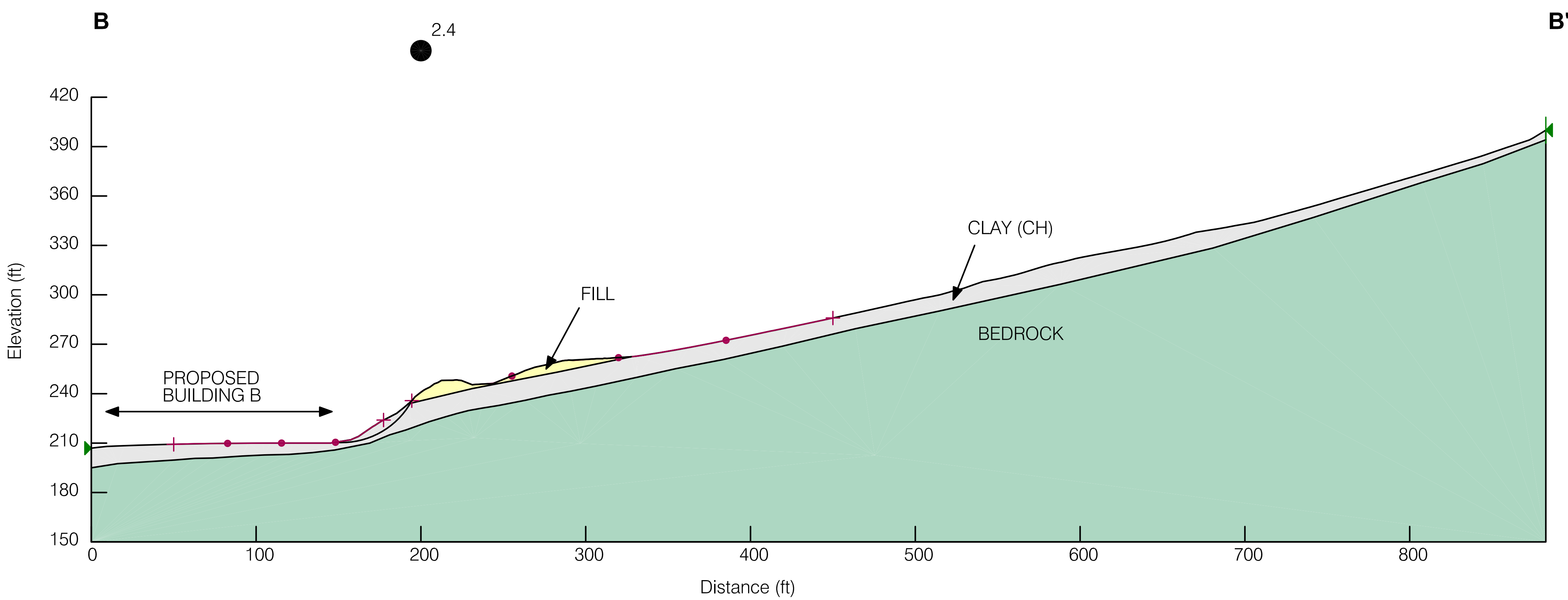


Name: Section B-B' Run 1
Method: Morgenstern-Price
Horz Seismic Coef.: 0

Name: CLAY (CH)
Model: Mohr-Coulomb
Unit Weight: 125 pcf
Cohesion': 240 psf
Phi': 28 °
Phi-B: 0 °

Name: BEDROCK
Model: Mohr-Coulomb
Unit Weight: 131 pcf
Cohesion': 620 psf
Phi': 34 °
Phi-B: 0 °
Piezometric Line: 1

Name: Fill
Model: Mohr-Coulomb
Unit Weight: 125 pcf
Cohesion': 240 psf
Phi': 28 °
Phi-B: 0 °
Piezometric Line: 1

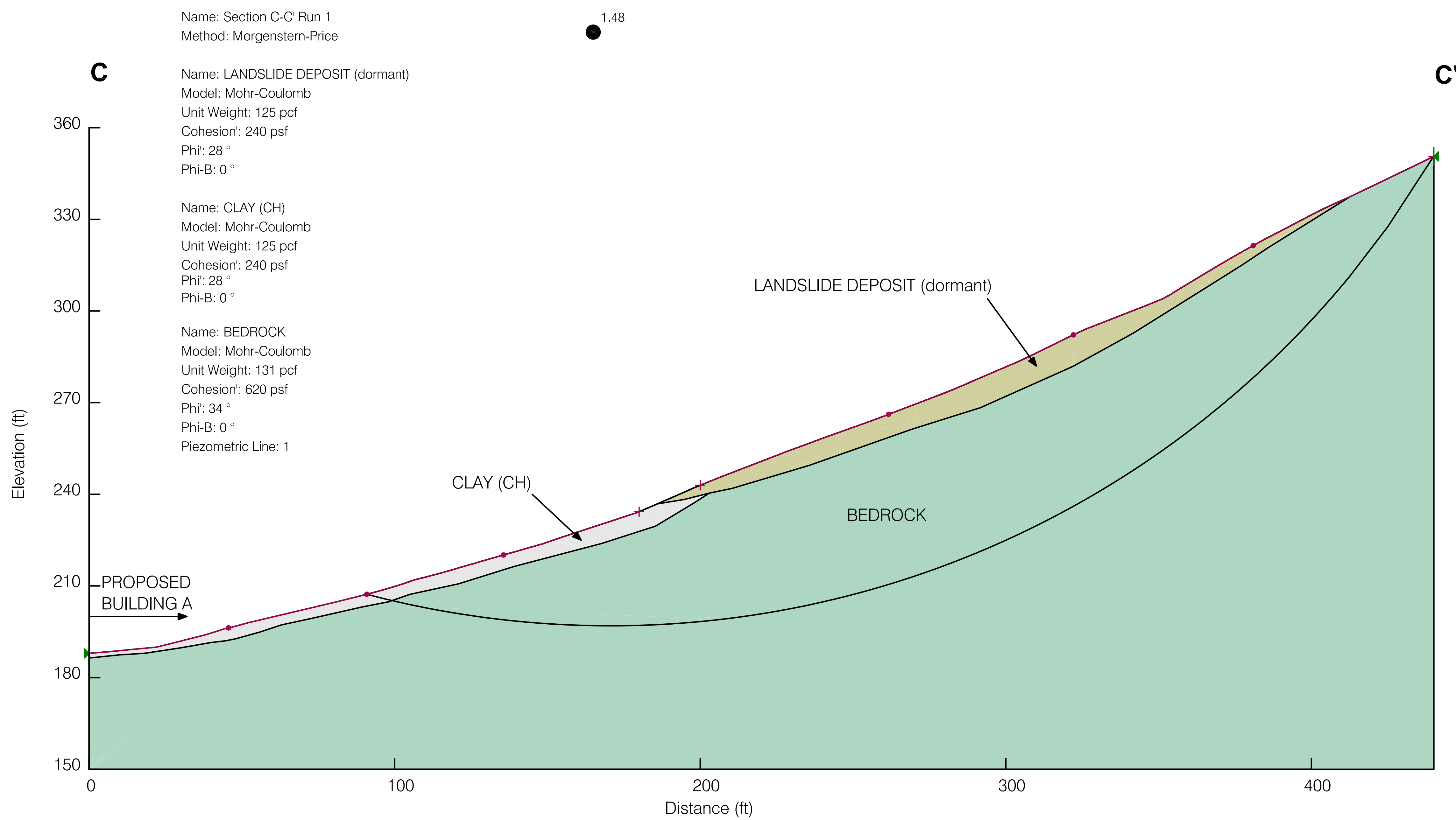
DOVE HILL ASSISTED LIVING COMMUNITY
4200 DOVE HILL ROAD
San Jose, California

SLOPE STABILITY ANALYSIS B-B'
EXISTING CONDITIONS

Date 05/07/15	Project No. 770619901	Figure F-4

LANGAN TREADWELL ROLLO

\\langan.com\data\SJ\data9\770619901\Cadd Data - 770619901\2D-DesignFiles\Geotechnical\770619901-B-PF0103.dwg 5/07/15



DOVE HILL ASSISTED LIVING COMMUNITY
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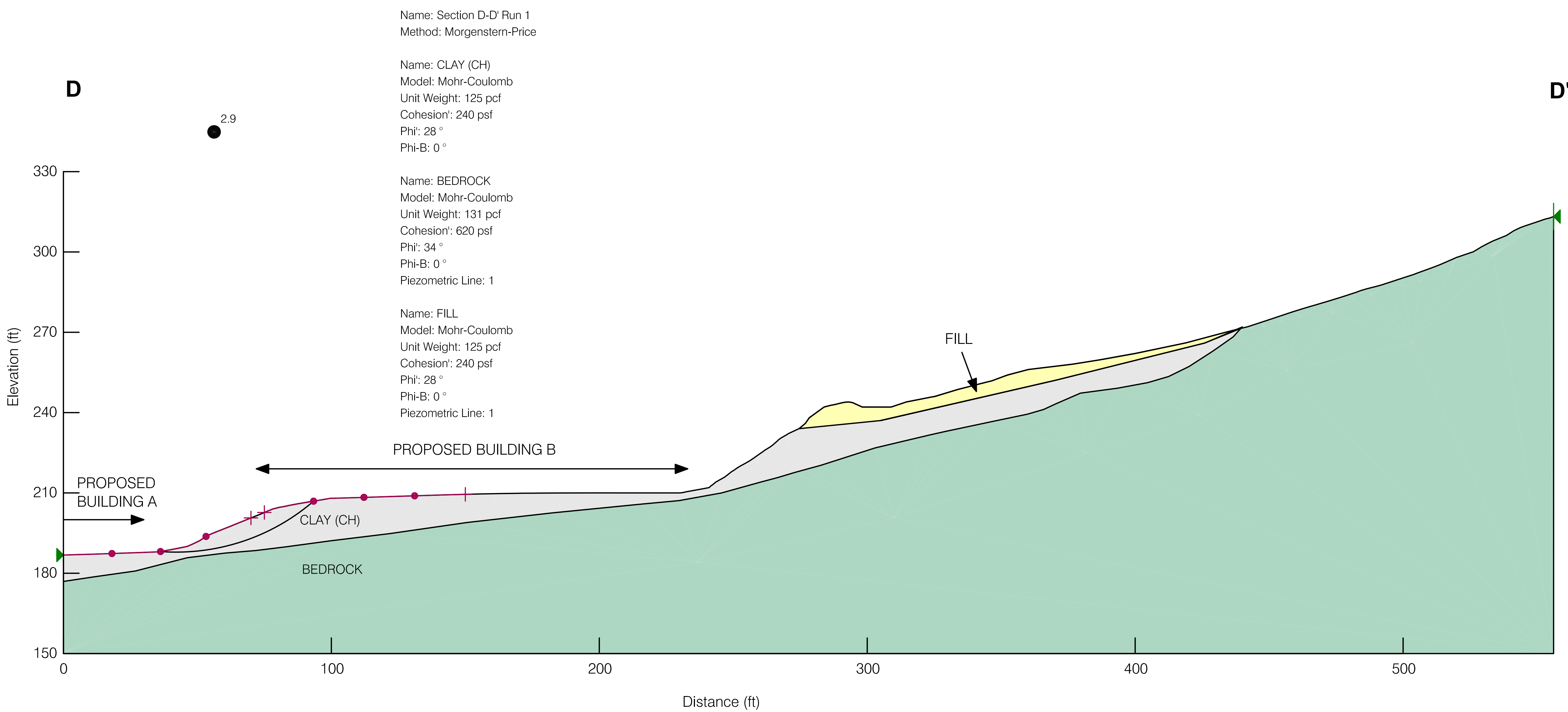
San Jose, California

SLOPE STABILITY ANALYSIS C-C'
EXISTING CONDITIONS

Date 05/07/15	Project No. 770619901	Figure F-5

LANGAN TREADWELL ROLLO

\\langan.com\data\SJ\data9\770619901\Cadd Data - 770619901\2D-DesignFiles\Geotechnical\770619901-B-PF0103.dwg 5/08/15



DOVE HILL ASSISTED LIVING COMMUNITY
4200 DOVE HILL ROAD
San Jose, California

SLOPE STABILITY ANALYSIS D-D'
EXISTING CONDITIONS - LOWER SLOPE

Date 05/07/15	Project No. 770619901	Figure F-6

LANGAN TREADWELL ROLLO

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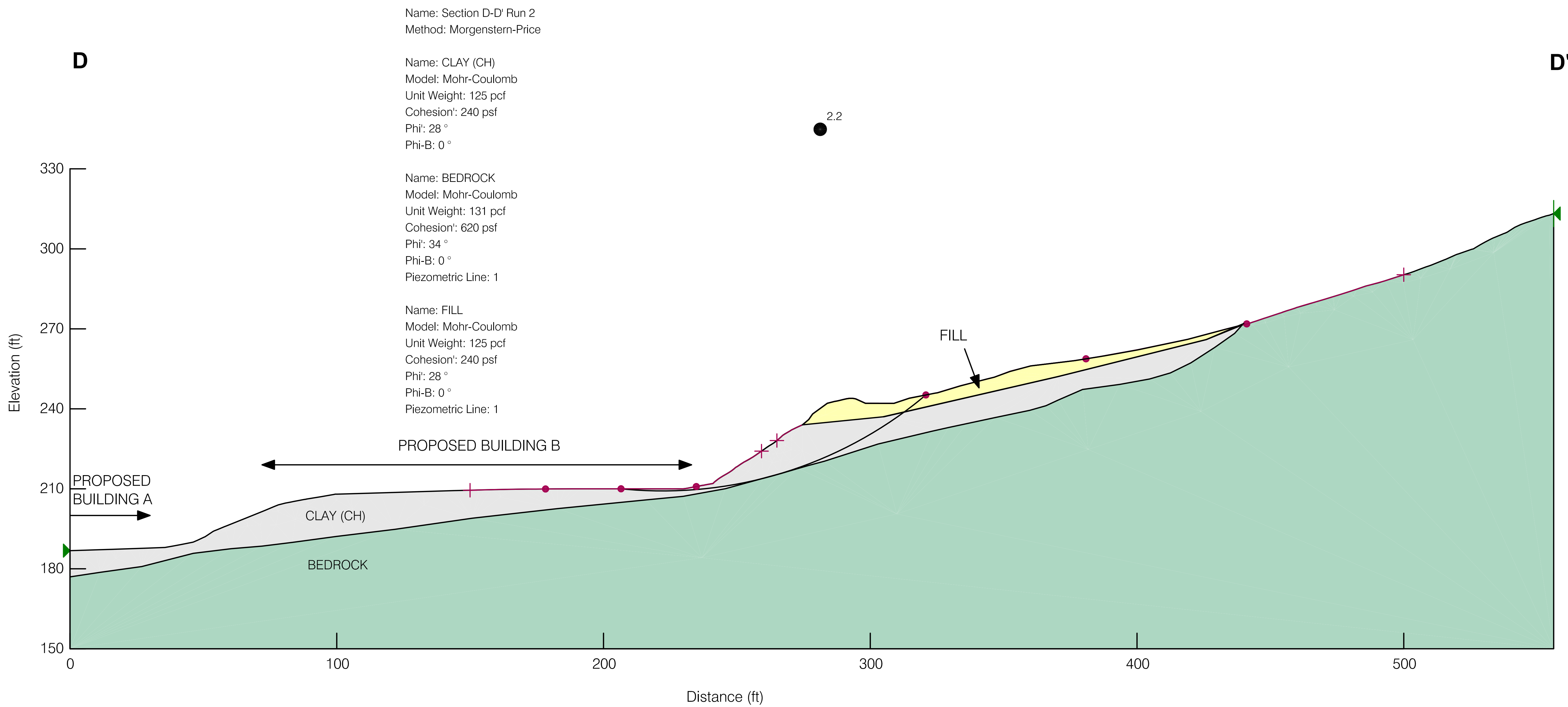

DOVE HILL ASSISTED LIVING COMMUNITY
4200 DOVE HILL ROAD
San Jose, California

SLOPE STABILITY ANALYSIS D-D'
EXISTING CONDITIONS - UPPER SLOPE

Date 05/07/15	Project No. 770619901	Figure F-7

LANGAN TREADWELL ROLLO

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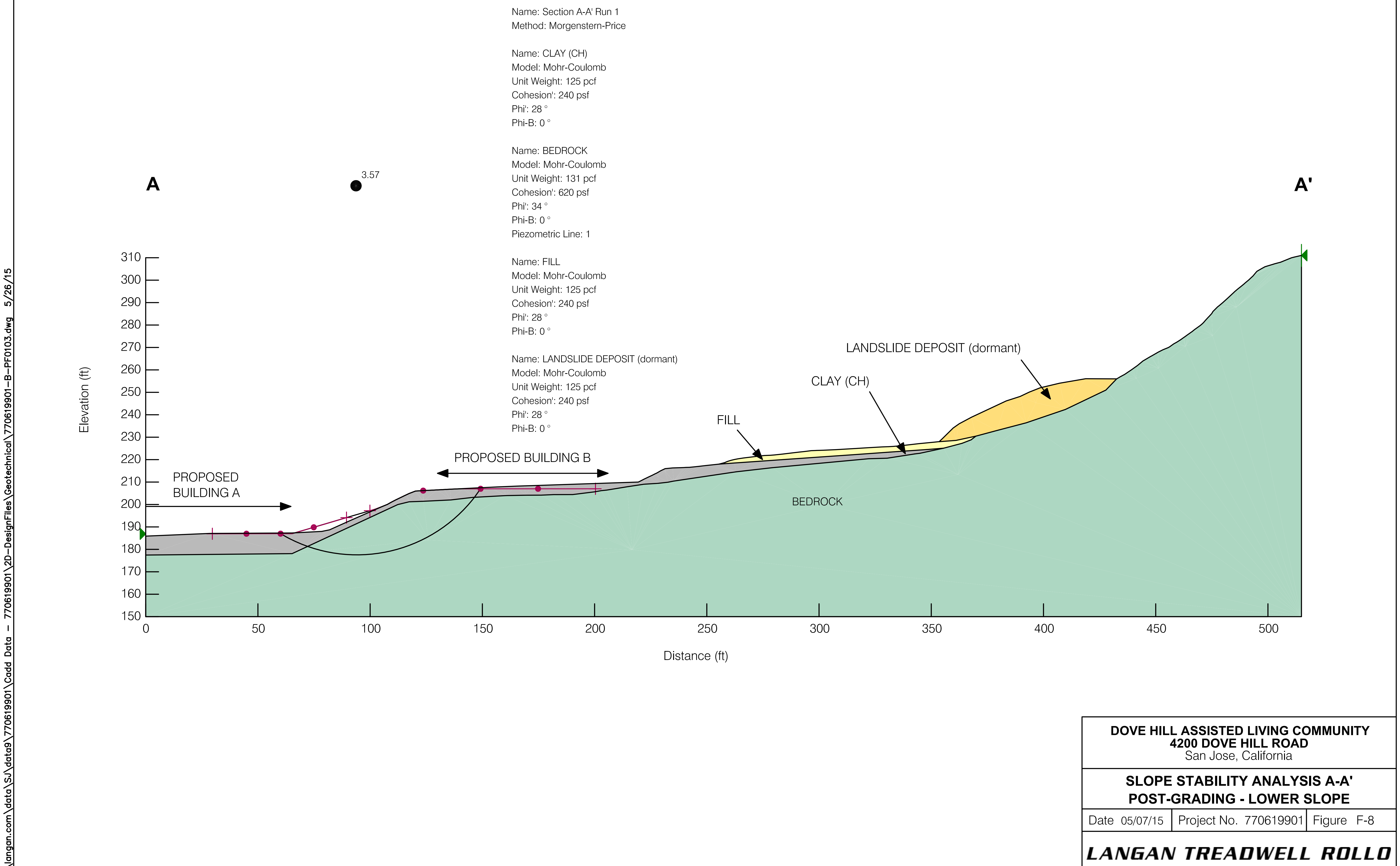

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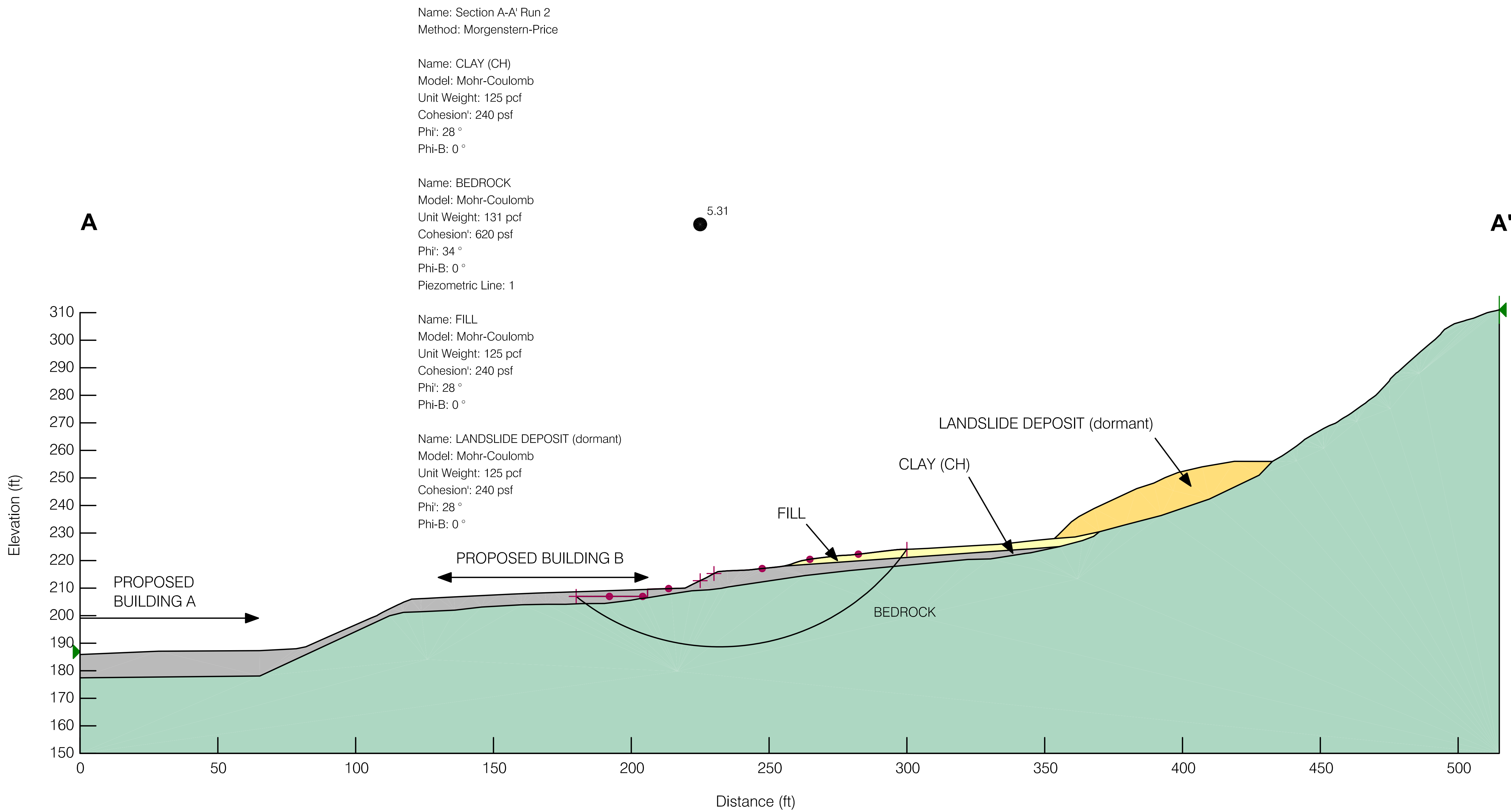
4200 DOVE HILL ROAD
San Jose, California

SLOPE STABILITY ANALYSIS A-A'
POST-GRADING - LOWER SLOPE

Date 05/07/15	Project No. 770619901	Figure F-8

LANGAN TREADWELL ROLLO



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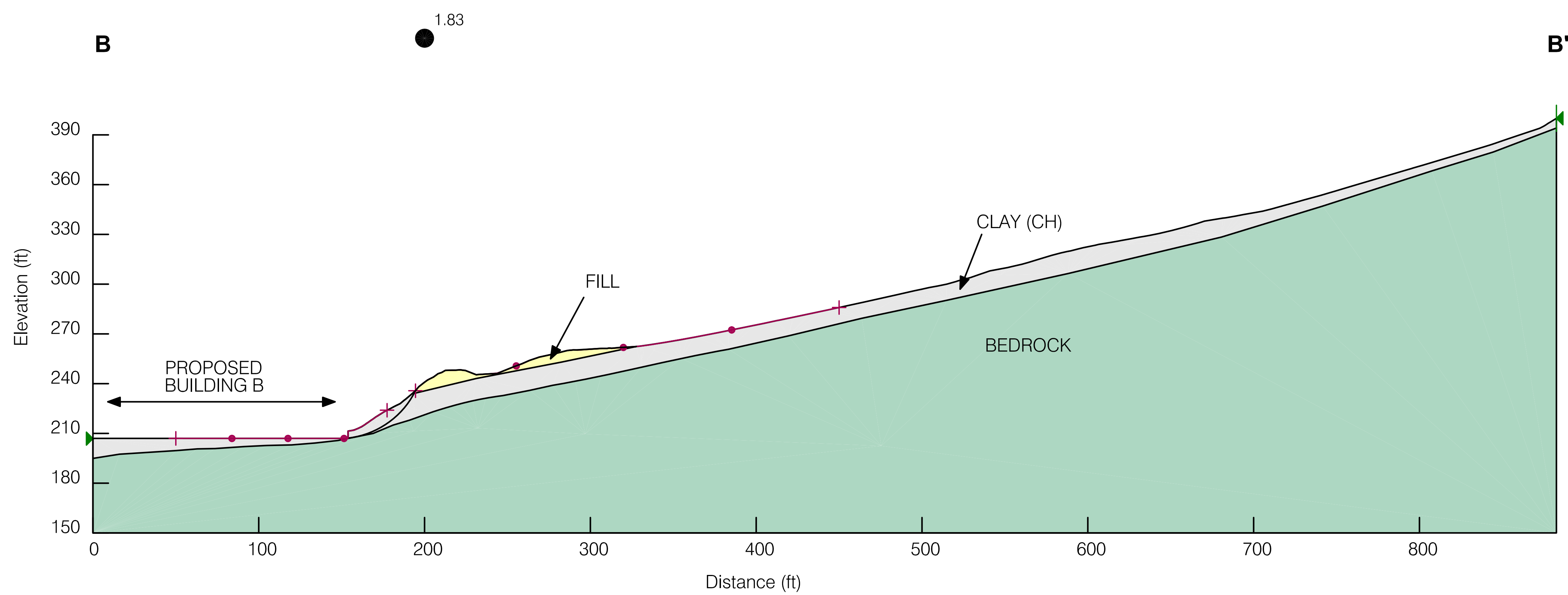
4200 DOVE HILL ROAD
San Jose, California

SLOPE STABILITY ANALYSIS A-A'
POST-GRADING - MIDDLE SLOPE

Date 05/07/15	Project No. 770619901	Figure F-9

LANGAN TREADWELL ROLLO



Name: Section B-B' Run 1
Method: Morgenstern-Price
Horz Seismic Coef.: 0

Name: CLAY (CH)
Model: Mohr-Coulomb
Unit Weight: 125 pcf
Cohesion': 240 psf
Phi': 28 °
Phi-B: 0 °

Name: BEDROCK
Model: Mohr-Coulomb
Unit Weight: 131 pcf
Cohesion': 620 psf
Phi': 34 °
Phi-B: 0 °
Piezometric Line: 1

Name: Fill
Model: Mohr-Coulomb
Unit Weight: 125 pcf
Cohesion': 240 psf
Phi': 28 °
Phi-B: 0 °
Piezometric Line: 1

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San Jose, California

SLOPE STABILITY ANALYSIS B-B'
POST-GRADING

Date 05/07/15	Project No. 770619901	Figure F-10

LANGAN TREADWELL ROLLO

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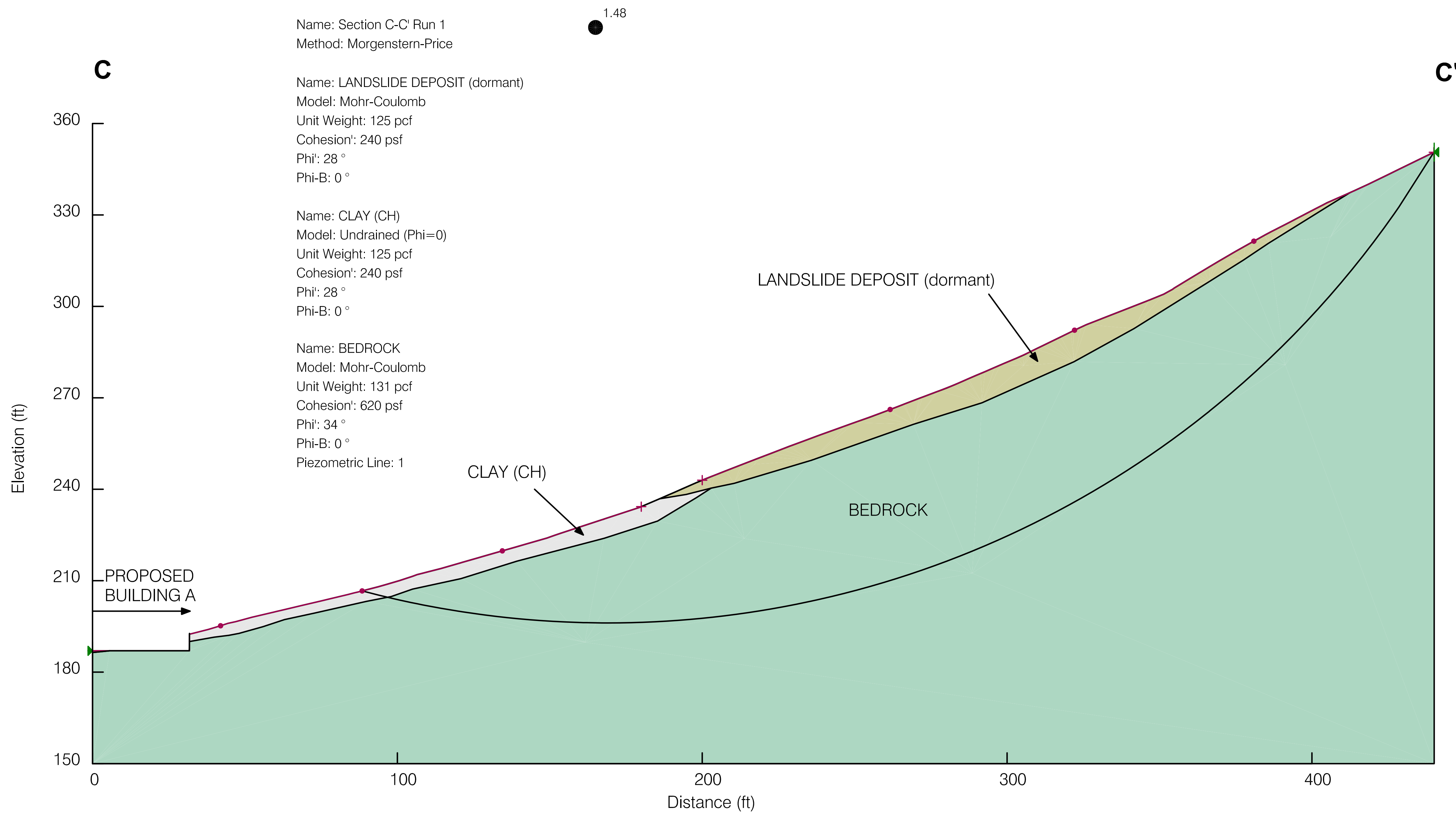

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San Jose, California

SLOPE STABILITY ANALYSIS C-C'
POST-GRADING

Date 05/07/15	Project No. 770619901	Figure 11

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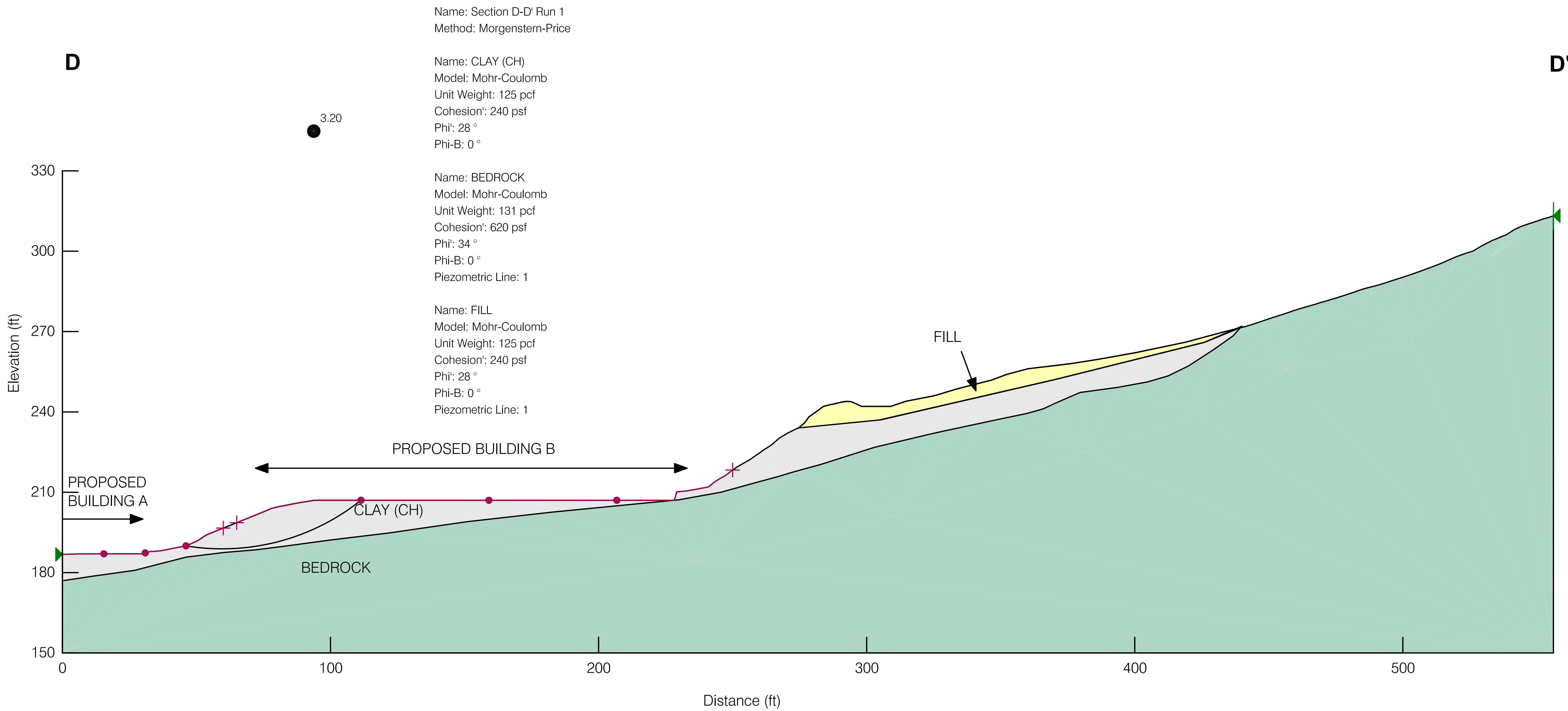

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4200 DOVE HILL ROAD
San Jose, California

SLOPE STABILITY ANALYSIS D-D'
POST-GRADING - LOWER SLOPE

Date 05/07/15	Project No. 770619901	Figure F-12

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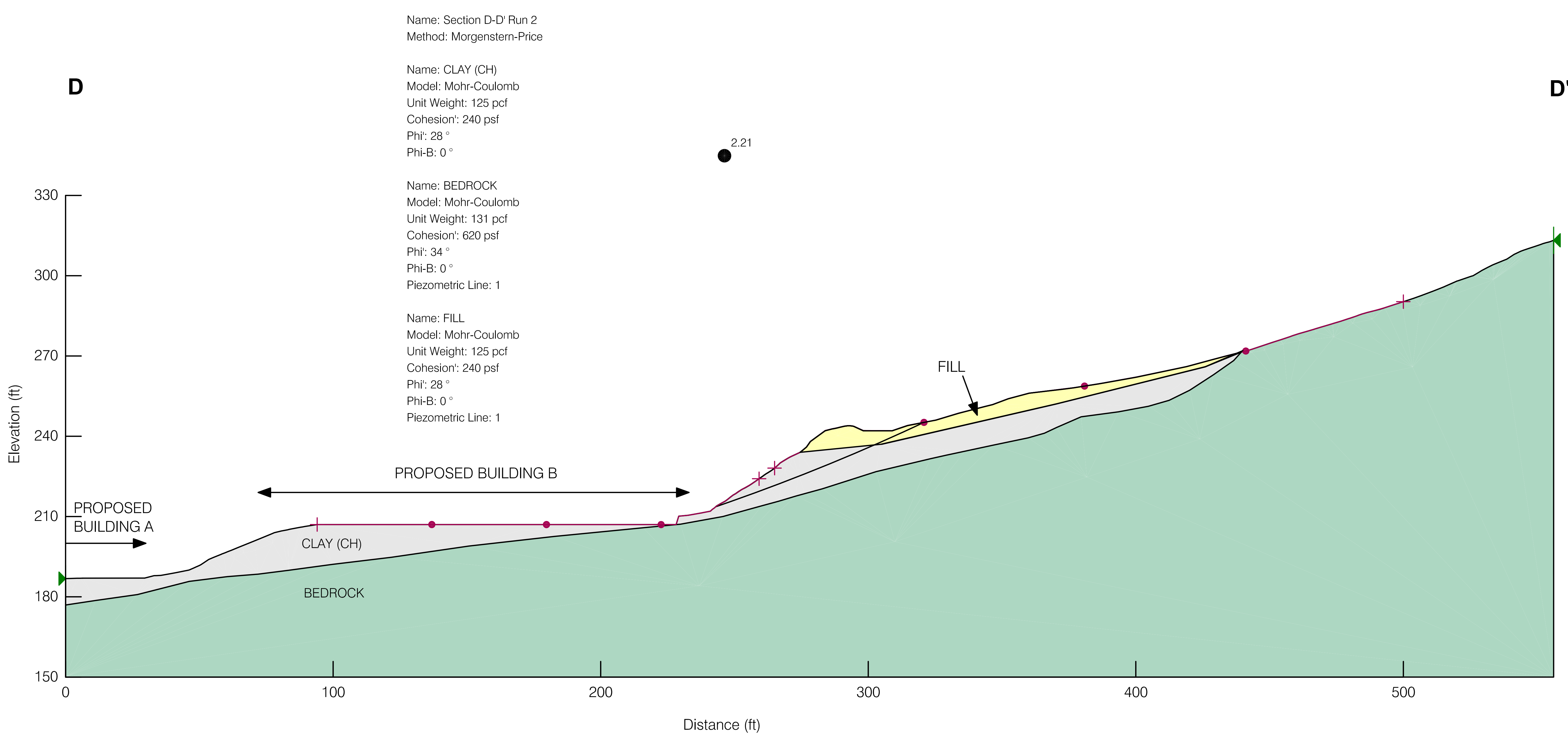

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San Jose, California

SLOPE STABILITY ANALYSIS D-D'
POST-GRADING - UPPER SLOPE

Date 05/07/15	Project No. 770619901	Figure F-13

LANGAN TREADWELL ROLLO

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