PRELIMINARY GEOTECHNICAL INVESTIGATION

PROPOSED MULTI-FAMILY RESIDENTIAL DEVELOPMENT
HAWTHORNE BOULEVARD AND VIA VALMONTE
TORRANCE, CALIFORNIA

GEOCON
WEST, INC.

GEOTECHNICAL
ENVIRONMENTAL
MATERIALS

PREPARED FOR

REYLENN PROPERTIES, LLC
SOLANA BEACH, CALIFORNIA

PROJECT NO. A9201-06-01C

MARCH 3, 2016

GEOCON
WEST, INC.
GEOTECHNICAL ■ ENVIRONMENTAL ■ MATERIALS

Project No. A9201-06-01C
March 3, 2016

Mr. Derek Empey
Reylenn Properties, LLC
444 South Cedros Avenue
Solana Beach, California 92705

Subject: PRELIMINARY GEOTECHNICAL INVESTIGATION
PROPOSED MULTI-FAMILY RESIDENTIAL DEVELOPMENT
HAWTHORNE BOULEVARD AND VIA VALMONTE
TORRANCE, CALIFORNIA

Dear Mr. Empey:

In accordance with the terms and conditions of our Professional Services Agreement dated January 28, 2015, we have performed a Preliminary Geotechnical Investigation for the proposed residential development at the subject property located south and west of Via Valmonte and Hawthorne Boulevard in the City of Torrance, California. The accompanying report presents the findings of our study, and our conclusions and recommendations pertaining to the geotechnical aspects of proposed design and construction. Based on the results of our investigation, it is our opinion that the site can be developed as proposed, provided the recommendations of this report are followed and implemented during design and construction.

Due to the preliminary nature of the project at this time, the recommendations presented herein should also be considered preliminary. Additional analyses will be required in order to provide comprehensive geotechnical recommendations for design and construction.

If you have any questions regarding this report, or if we may be of further service, please contact the undersigned.

Very truly yours,

GEOCON WEST, INC.

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PRELIMINARY GEOTECHNICAL INVESTIGATION

1.0 PURPOSE AND SCOPE

This report presents the results of our preliminary geotechnical investigation for the proposed residential development at the vacant property located south of Via Valmonte and west of Hawthorne Boulevard in the city of Torrance, California (see Vicinity Map, Figure 1). The purpose of the investigation was to evaluate subsurface soil and geologic conditions underlying the site and, based on conditions encountered, to provide conclusions and recommendations pertaining to the geotechnical aspects of design and construction.

Due to the preliminary nature of the project at this time, the recommendations presented herein should also be considered preliminary. Additional analyses will be required in order to provide comprehensive geotechnical recommendations for design and construction.

The scope of this investigation included a literature review, site reconnaissance, field exploration, laboratory testing, engineering analysis, and the preparation of this report. We have also concurrently performed a fault rupture hazard investigation at the site. The results of the fault rupture hazard investigation are presented in a separate report dated January 21, 2016 (Geocon, 2016).

Our field exploration was performed between July 14, 2015 and July 24, 2015 and included drilling seventeen large-diameter bucker auger borings utilizing a truck mounted LM-60 Lo-Drill and a truck-mounted E-Z Bore bucket auger drilling machine. The large-diameter borings were advanced to approximate depths between 11 feet and 111½ feet below the existing ground surface. Four of the large-diameter borings were downhole logged by a Certified Engineering Geologist (CEG). In addition, six 4-inch diameter borings were excavated utilizing manual augers and digging equipment to depths between 7 and 23½ feet beneath the existing ground surface. The approximate locations of the exploratory borings are depicted on the Geologic Map (see Figure 2A). A detailed discussion of the field investigation, including boring logs, is presented in Appendix A.

Laboratory tests were performed on selected soil samples obtained during the investigation to determine pertinent physical and chemical soil properties. Appendix B presents a summary of the laboratory test results.

The recommendations presented herein are based on analysis of the data obtained during the investigation and our experience with similar soil and geologic conditions. References reviewed to prepare this report are provided in the *List of References* section.

If project details vary significantly from those described above, Geocon should be contacted to determine the necessity for review and possible revision of this report.

2.0 SITE AND PROJECT DESCRIPTION

The property is an approximately 23.35-acre irregular-shaped parcel and is currently vacant. The project area is bounded by Via Valmonte on the north and west, Hawthorne Boulevard on the east, and a 200- to 250-foot-high, north-facing, former quarry slope on the south (referred to herein as Slope 3). The lower 120 to 180 vertical feet of Slope 3 has been graded to a generally uniform inclination ranging from approximately 48 to 50 degrees and exposes Miocene age sedimentary bedrock of the Monterey Formation. The upper elevations of the slope have undergone little to no grading and are inclined at gradients of less than 2:1 (horizontal to vertical). The elevation at the base of the north-facing slope ranges from approximately 200 to 220 feet (MSL) in the western portion of the site and ranges from approximately 240 to 245 feet (MSL) in the eastern portion of the site with a maximum elevation of approximately 460 feet (MSL) at the crest.

The site topography, north of the former quarry slope (Slope 3), has also been altered by previous diatomite and diatomaceous soil mining activities. As a result, the existing site topography generally slopes toward the center of the site which is a topographic low. The area of the topographic low was previously mined to approximately Elevation 150 MSL and later backfilled to create two level pads, the lower pad at approximately Elevation 190 to Elevation 220 and the upper pad at approximately Elevation 235 MSL to Elevation 245 MSL. Between 2½ and 50½ feet of artificial fill is present at the site. Existing slopes bounding the proposed development on the northwest (Slope 1) and east-northeast (Slope 2) are considered graded slopes (from past mining operations). Slope 1 ranges from approximately 40 to 80 feet in height and is inclined at gradients ranging from 1¼:1 to 1½:1 (horizontal to vertical). Slope 2 is approximately 50 feet in height and is inclined at gradients ranging from 2:1 to 1½:1 (horizontal to vertical).

Information regarding the proposed project was provided by the client and is preliminary in nature. It is our understanding that that the planned multi-family residential development will consist of multi-family residential buildings, flats, and community areas. The multi-family residential buildings will consist of four- and five-stories of residential units over 2 levels of parking, with proposed lowest floor elevation of approximately 195 feet Mean Sea Level (MSL). The three-story flats will be constructed in the northeastern portion of the site at an elevation of approximately 195 feet MSL. The proposed site development is shown on the Site Plan (see Figure 2B).

Based on the proposed site elevations, excavations on the order of 45 feet and fills on the order of 5 feet will be required for construction of the proposed development.

Based on the preliminary nature of the design at this time, wall and column loads were not available. It is anticipated that column loads for the proposed residential structures will be up to 600 kips, and wall loads will be up to 8 kips per linear foot. It is anticipated that the column loads for the proposed flats structures will be up to 300 kips, and the wall loads will be up to 4 kips per linear foot.

Due to the preliminary nature of the project at this time, the recommendations presented herein should also be considered preliminary. As the project proceeds, additional analyses will be required in order to provide comprehensive geotechnical recommendations for design and construction. Any changes in the design, location or elevation of any structure, as outlined in this report, should be reviewed by this office.

3.0 PRIOR INVESTIGATIONS

We previously performed a preliminary geotechnical evaluation for the site that consisted of a literature review, including review of published geologic maps and reports, historic topographic maps, available local groundwater level data, and recent nearby fault investigations. The results of the preliminary geotechnical evaluation were presented in a separate report dated November 11, 2014.

Also, a previous geotechnical investigation was performed at the site by Pacific Soils, Inc. and field explorations were performed at the site between February 8, 2005 and September 21, 2005. The results of the investigation were presented in a report dated June 4, 2008. We were provided with a copy of the field exploration logs for review. However, we were not provided a full copy of the prior Pacific Soils report.

Based on our review of the Pacific Soils report, four bucket auger borings and nine test pits were performed as part of their geotechnical investigation at the site. The bucket auger borings (B-101, B-102, B-201 and B-202) were advanced to depths up to approximately 111 feet below the existing ground surface and the test pits (EP-1 to EP-9) were advanced to a maximum depth of 17 feet beneath the existing ground surface. Pacific Soils also utilized boring data from a previous geotechnical investigation at the site by Western Laboratories (1996). The locations of the Pacific Soils (2008) explorations and Western Laboratories (1996) are shown on Figure 2A. Boring B-102 and test pits EP-5 through EP-9 do not provide geologic information pertinent to our investigation and therefore are not shown on Figure 2A.

Other documents we reviewed include the following:

- *Butcher Hill Topographic Survey,* Bolten Engineering Corporation, undated, Scale: 1” = 50’.
- *Topographic Map for Butcher Construction Company, 1976,* Scale: 1” = 40’
- *Geotechnical Boring Logs, Plates A-1 through A-8, Pacific Soils,* dated June 2008, W.O. 102568.
- *Western Laboratories Testing Results, Plates E-1 through E-7,* dated June 2008, W.O. 102568.
- *Western Laboratories Boring and Test Pit Logs, Plates D1 through D-13,* dated June 2008, W.O. 102568.
- *Western Laboratories, 1996, Addendum No. 2, Preliminary Geotechnical Engineering Report, Butcher Hill Residential Development, West of Hawthorne Boulevard, Between Via Valmonte and Rolling Hills Road, Torrance, California,* dated April 23, 1996, W.O. 93-1605.

- *Draft Environmental Impact Report, Butcher Mountain, Tentative Tract No. 51753, EAS94-0008, ZON94-0002, State Clearinghouse No. 95041055*, prepared for the City of Torrance Planning Department, Prepared by Envicom Corporation, dated August 11, 1995.
- *Final Draft Environmental Impact Report 76-11 for D. Butcher, Tentative Tract 33120,* Prepared by Environmental Division, Planning Department, City of Torrance, Dated July 1977.
- *Action Engineering Consultants, 1976, Proposed Residential Development, Southwest Corner of Hawthorne Boulevard and Via Valmonte Avenue in the City of Torrance, California,* Prepared for Butcher Construction Company, 2371 Torrance Boulevard, Torrance, California, Work Order 4H2-507-01, dated July 27, 1976.
- *John D. Merrill, CPG, 1976, Engineering Geology Report, Tentative Tract, Hawthorne Boulevard and Via Valmonte Street, Torrance,* California, for Action Engineering Consultants, Project 63487, dated August 4, 1976.
- *Environmental Impact Evaluation of Proposed Tract 26507 in the City of Torrance, Prepared for Mr. Philip Nicholson-Trustee,* Prepared by LANDCO, Land Consultants and Civil Engineers, dated April 18, 1972.
- *Site Plan* and *Parking Plan,* Withee Malcolm Architects, undated.

A complete list of the documents reviewed as part of this study is presented in the *List of References* section of this report.

4.0 GEOLOGIC SETTING

The site is located on the northern flank of the Palos Verdes Hills, the westernmost onshore uplift of the Peninsular Ranges geomorphic province. The sedimentary rock in the Palos Verdes Hills are folded and faulted into a dome-like structure with the north and south limbs dipping downward away from the central portion of the hills. The sedimentary rocks along the north side of the hills dip to the north at inclinations of approximately 17 to 86 degrees. The major geologic structure in the area is the northwest-trending Palos Verdes Fault Zone, a zone of right-lateral strike-slip and oblique-slip faults. The Palos Verdes Fault Zone is generally located along the northern edge of the Palos Verdes Hills between San Pedro Bay on the southeast and Santa Monica Bay on the northwest. The northeastern strand of the fault zone is the boundary between the uplifted Palos Verdes Hills to the south and the Los Angeles Basin to the north.

5.0 SOIL AND GEOLOGIC CONDITIONS

Based on our field investigation and published geologic maps of the area, the geologic materials exposed at the site include artificial fill, overburden soil, Pleistocene age marine sand, San Pedro Sand and Lomita Marl, and Miocene age sedimentary bedrock of the Monterey Formation. Detailed stratigraphic profiles are provided on the boring logs in Appendix A.

5.1 Artificial Fill (af)

Artificial fill was encountered in our field explorations to depths between 2½ and 50½ feet below existing ground surface. On the lower pad, the fill is shallowest near the base of the adjacent slopes and increases in thickness towards the central area of the site. On the slopes bounding the proposed development on the northwest (Slope 1) and east-northeast (Slope 2), the fill is approximately 2½ to 5½ feet thick. The artificial fill in the flat portion of the site generally consists of light to dark brown and yellowish brown sand, silty sand, clayey sand, with lesser amounts of gravelly sand, silt and clay. As observed in the large-diameter borings, the fill contains localized lifts of concentrated concrete, brick, and rock fragments (up to 22 inches in longest dimension) with localized pockets of debris such as wire, PVC pipe, plastic and metal debris. The artificial fill is characterized as slightly moist and loose to medium dense. The fill is the result of backfilling the previous mining pit, a process that has been on-going since the 1960s. Deeper fill may exist between excavations and in other portions of the site that were not directly explored.

5.2 Overburden Soil

Overburden soil was encountered within the upper five feet in boring B1, located at the top of the north-facing slope (Slope 3). The overburden soil was derived from in situ weathering of the underlying sedimentary bedrock and consists primarily of light gray sandy silt with varied amounts of gravel and roots. The soils are primarily dry and soft and are underlain by the sedimentary bedrock of the Monterey Formation.

5.3 Marine Sand (Qm)

Late Pleistocene age marine sand was encountered below the fill soils in hand augers HA1 through HA3 (on Slope 2) to a maximum depth of 15 feet. The marine sand generally consists of reddish brown fine to medium-grained sand and silty sand with lenses of coarse-grained sand and rounded gravel. As observed in our trenches, excavated for our concurrent fault investigation at the site, the marine sand is generally massive to horizontally bedded. The marine sand is characterized as slightly moist and loose to medium dense. Although weakly developed bedding is locally observed in this formation, this material is considered predominantly massive.

5.4 San Pedro Sand (Qsp)

The late Pleistocene age San Pedro Sand (Dibblee, 1999) underlies the fill on Slope 1, the marine sand on Slope 2, and the proposed building areas on the existing graded pads. The San Pedro Sand ranges from light gray to yellowish brown (where oxidized), fine- to coarse-grained sand that is generally massive to well-bedded, friable (uncemented) with local gravel-rich beds and some rounded cobbles. As observed in our trenches, excavated for our concurrent fault investigation at the site, the San Pedro Sand is generally massive but locally exhibits crudely stratified sand beds that dip to the north between 20 and 65 degrees. The sand is characterized as slightly moist and medium dense to dense. Although bedding is locally observed in this formation, this material is considered predominantly massive.

5.5 Lomita Marl (Qlm)

The mid-Pleistocene age Lomita Marl underlies the San Pedro Sand (Dibblee, 1999) and is locally exposed on the north-facing slope (Slope 3) along the southern project boundary. The Lomita Marl was not encountered in our explorations at the site. However, as observed in very limited exposures on Slope 3, the Lomita Marl is generally fossiliferous, calcareous, fine-grained sandstone and siltstone that is massive to poorly bedded and dips to the northeast at inclinations of 50 degrees or steeper (Woodring et al., 1946). The very limited exposures of the Lomita Marl are included in the Monterey Formation units shown on Figure 2A.

5.6 Monterey Formation Bedrock (Tm)

Sedimentary bedrock of the Valmonte Diatomite member of the Miocene age Monterey Formation was encountered in borings B1 and B2 and is exposed on the north-facing slope (Slope 3) along the southern site boundary. As encountered in the borings, the Valmonte Diatomite consists of interbedded white diatomaceous siltstone and sandstone and brown to yellow brown clayey siltstone with localized siliceous and cherty beds. As exposed on Slope 3, the bedrock is predominantly diatomaceous siltstone and sandstone with localized lenses of siliceous siltstone, fossiliferous sandstone, and cherty sandstone. The bedrock is thinly bedded with well-developed bedding and ranges from very soft (diatomaceous siltstone and sandstone beds) to medium hard (cherty and siliceous beds). The diatomaceous-rich portion of this formation is reported to be highly porous with low permeability, highly expansive, has poor slope stability, and is not suitable for fill material (City of Torrance, 2009).

Based on published geologic maps, previous geotechnical reports for the site, and downhole logging of large-diameter borings B1 and B2 at the site, the Monterey Formation bedrock dips to the northeast and northwest at inclinations of approximately 17 to 86 degrees. The bedding exposed on the slope face generally dips 50 to 68 degrees to the north.

6.0 GROUNDWATER

Based on a review of the Seismic Hazard Zone Report for the Torrance 7.5 Minute Quadrangle, Los Angeles County, California (California Division of Mines and Geology [CDMG], 1998), there is no reported data for the historically highest groundwater level in the immediate area.

The Los Angeles County Department of Public Works (LACDPW) has maintained various wells in the vicinity of the subject site over the past 50 years. The closest groundwater monitoring well to the site is Well No. 769 located approximately 1.4 miles to the northeast (LACDPW, 2015a). Data for this well is available for the monitoring period between 1946 and 1991. The monitoring data indicates that the depth to groundwater has fluctuated between the high and low measurements of 82.7 feet below the existing ground surface in October 2008 to 164.4 feet below the existing ground surface in April, 1971, respectively (LACDPW, 2015a). The most recent groundwater level measurement for Well No. 240A was measured in October, 2008 at a depth of 82.7 feet below the existing ground surface (LACDPW, 2015a).

Groundwater was not encountered in our borings drilled to a maximum depth of 61.5 feet beneath the existing ground surface within the proposed building area and 111½ feet beneath the existing ground surface at the top of Slope 3. Based on the lack of groundwater encountered in the previous explorations as well as the historic groundwater levels in nearby monitoring wells, groundwater is not anticipated to adversely impact the proposed development. However, it is not uncommon for groundwater levels to vary seasonally or for groundwater seepage conditions to develop where none previously existed, especially in impermeable fine-grained soils which are heavily irrigated or after seasonal rainfall. In addition, recent requirements for stormwater infiltration could result in shallower seepage conditions in the immediate site vicinity. Proper surface drainage of irrigation and precipitation will be critical for future performance of the project. Recommendations for drainage are provided in the S*urface Drainage* section of this report (see Section 8.21).

7.0 GEOLOGIC HAZARDS

7.1 Surface Fault Rupture

7.1.1 General

The numerous faults in Southern California include active, potentially active, and inactive faults. The criteria for these major groups are based on criteria developed by the California Geological Survey (CGS, formerly known as California Division of Mines and Geology [CDMG]) for the Alquist-Priolo Earthquake Fault Zone Program (Bryant and Hart, 2007). By definition, an active fault is one that has had surface displacement within Holocene time (about the last 11,000 years). A potentially active fault has demonstrated surface displacement during Quaternary time (approximately the last 1.6 million years), but has had no known Holocene movement. Faults that have not moved in the last 1.6 million years are considered inactive.

The site is not within a currently established Alquist-Priolo Earthquake Fault Zone for surface fault rupture hazards (CGS, 2016; Bryant and Hart, 2007). However, the site is located within a City of Torrance Fault Hazard Management Zone (City of Torrance, 2009). As previously discussed, we have performed a fault rupture hazard investigation for the proposed development in accordance with the requirements of the City of Torrance. The results of this investigation are presented under separate cover (Geocon, 2016).

7.1.2 Palos Verdes Fault

The results of our fault rupture hazard investigation (Geocon, 2016) indicate the active Palos Verdes Fault is located offsite, approximately 350 feet to the north. Active faults are not present at the site and the potential for surface fault rupture to occur at the site is considered very low. However, the site is located within a zone of minor shearing that is considered a consequence of either folding of the northeastern flank of the Palos Verdes Hills or a result of minor movement during periods of strong ground shaking or liquefaction prior to uplift of this portion of the peninsula. All the features observed are minor and considered Middle Pleistocene age.

Since the former mining operation has significantly altered the original ground surface at the site and the majority of the upper portion of the San Pedro Sand has been removed and replaced with up to 50 feet of fill soils, there is some uncertainty as to the potential for differential movement along these minor shears during a future earthquake. Therefore, we conclude there is a minor risk that a future earthquake may generate minor secondary slip along these features.

It is our opinion that a structural setback from these features is not warranted. However, mitigation measures are necessary to eliminate the potential for differential settlement that may adversely affect the proposed structures. These mitigation measures are presented in the Conclusions and Recommendations section of this report.

7.1.3 Other Nearby Faults

Other nearby active faults include the Cabrillo Fault and the Newport-Inglewood Fault Zone approximately 1.9 miles south and 7.5 miles northeast of the site, respectively. The active San Andreas Fault Zone is located approximately 52 miles northeast of the site (Ziony and Jones, 1989). The faults in the Los Angeles area and the general site vicinity are shown in Figure 4, Regional Fault Map.

The Compton Thrust underlies the site and the majority of the city of Torrance at depths greater than several kilometers (City of Torrance, 2009). Also, several buried thrust faults, commonly referred to as blind thrusts, underlie the Los Angeles Basin at depth. These faults are not exposed at the ground surface and are typically identified at depths greater than 3.0 kilometers. The October 1, 1987 M_w 5.9 Whittier Narrows earthquake and the January 17, 1994 M_w 6.7 Northridge earthquake were a result of movement on the Puente Hills Blind Thrust and the Northridge Thrust, respectively. These thrust faults and others in the Los Angeles area are not exposed at the surface and do not present a potential surface fault rupture hazard at the site; however, these deep thrust faults are considered active features capable of generating future earthquakes that could result in moderate to significant ground shaking at the site.

7.2 Seismicity

As with all of Southern California, the site has experienced historic earthquakes from various regional faults. The seismicity of the region surrounding the site was formulated based on research of an electronic database of earthquake data. The epicenters of recorded earthquakes with magnitudes equal to or greater than 5.0 in the site vicinity are depicted on Figure 5, Regional Seismicity Map. A partial list of moderate to major magnitude earthquakes that have occurred in the Southern California area within the last 100 years is included in the following table.

LIST OF HISTORIC EARTHQUAKES

Earthquake (Oldest to Youngest)	Date of Earthquake	Magnitude	Distance to Epicenter (Miles)	Direction to Epicenter
San Jacinto-Hemet area	April 21, 1918	6.8	78	E
Near Redlands	July 23, 1923	6.3	90	E
Long Beach	March 10, 1933	6.4	25	SE
Tehachapi	July 21, 1952	7.5	91	NW
San Fernando	February 9, 1971	6.6	42	N
Whittier Narrows	October 1, 1987	5.9	24	NE
Sierra Madre	June 28, 1991	5.8	38	NE
Landers	June 28, 1992	7.3	113	ENE
Big Bear	June 28, 1992	6.4	92	ENE
Northridge	January 17, 1994	6.7	31	NW

The site could be subjected to strong ground shaking in the event of an earthquake. However, this hazard is common in Southern California and the effects of ground shaking can be mitigated if the proposed structures are designed and constructed in conformance with current building codes and engineering practices.

7.3 Seismic Design Criteria

The following table summarizes summarizes site-specific design criteria obtained from the 2013 California Building Code (CBC; Based on the 2012 International Building Code [IBC] and ASCE 7-10), Chapter 16 Structural Design, Section 1613 Earthquake Loads. The data was calculated using the computer program *U.S. Seismic Design Maps*, provided by the USGS. The short spectral response uses a period of 0.2 second. The values presented below are for the risk-targeted maximum considered earthquake (MCE_R).

2013 CBC SEISMIC DESIGN PARAMETERS

Parameter	Value	2013 CBC Reference
Site Class	D	Table 1613.3.2
MCE_R Ground Motion Spectral Response Acceleration – Class B (short), S_S	1.730g	Figure 1613.3.1(1)
MCE_R Ground Motion Spectral Response Acceleration – Class B (1 sec), S_1	0.673g	Figure 1613.3.1(2)
Site Coefficient, F_A	1.0	Table 1613.3.3(1)
Site Coefficient, F_V	1.5	Table 1613.3.3(2)
Site Class Modified MCE_R Spectral Response Acceleration (short), S_{MS}	1.730g	Section 1613.3.3 (Eqn 16-37)
Site Class Modified MCE_R Spectral Response Acceleration – (1 sec), S_{M1}	1.010g	Section 1613.3.3 (Eqn 16-38)
5% Damped Design Spectral Response Acceleration (short), S_{DS}	1.153g	Section 1613.3.4 (Eqn 16-39)
5% Damped Design Spectral Response Acceleration (1 sec), S_{D1}	0.673g	Section 1613.3.4 (Eqn 16-40)

The table below presents the mapped maximum considered geometric mean (MCE_G) seismic design parameters for projects located in Seismic Design Categories of D through F in accordance with ASCE 7-10.

ASCE 7-10 PEAK GROUND ACCELERATION

Parameter	Value	ASCE 7-10 Reference
Mapped MCE_G Peak Ground Acceleration, PGA	0.720g	Figure 22-7
Site Coefficient, F_{PGA}	1.0	Table 11.8-1
Site Class Modified MCE_G Peak Ground Acceleration, PGA_M	0.720g	Section 11.8.3 (Eqn 11.8-1)

The Maximum Considered Earthquake Ground Motion (MCE) is the level of ground motion that has a 2 percent chance of exceedance in 50 years, with a statistical return period of 2,475 years. According to the 2013 California Building Code and ASCE 7-10, the MCE is to be utilized for the evaluation of liquefaction, lateral spreading, seismic settlements, and it is our understanding that the intent of the Building code is to maintain "Life Safety" during a MCE event. The Design Earthquake Ground Motion (DE) is the level of ground motion that has a 10 percent chance of exceedance in 50 years, with a statistical return period of 475 years.

Deaggregation of the MCE peak ground acceleration was performed using the USGS 2008 Probabilistic Seismic Hazard Analysis (PSHA) Interactive Deaggregation online tool. The result of the deaggregation analysis indicates that the predominant earthquake contributing to the MCE peak ground acceleration is characterized as a 6.90 magnitude event occurring at a hypocentral distance of 4.9 kilometers from the site.

Deaggregation was also performed for the Design Earthquake (DE) peak ground acceleration, and the result of the analysis indicates that the predominant earthquake contributing to the DE peak ground acceleration is characterized as a 6.66 magnitude occurring at a hypocentral distance of 17.4 kilometers from the site.

Conformance to the criteria in the above tables for seismic design does not constitute any kind of guarantee or assurance that significant structural damage or ground failure will not occur if a large earthquake occurs. The primary goal of seismic design is to protect life, not to avoid all damage, since such design may be economically prohibitive.

7.4 Liquefaction Potential

Liquefaction is a phenomenon in which loose, saturated, relatively cohesionless soil deposits lose shear strength during strong ground motions. Primary factors controlling liquefaction include intensity and duration of ground motion, gradation characteristics of the subsurface soils, in-situ stress conditions, and the depth to groundwater. Liquefaction is typified by a loss of shear strength in the liquefied layers due to rapid increases in pore water pressure generated by earthquake accelerations.

The current standard of practice, as outlined in the "Recommended Procedures for Implementation of DMG Special Publication 117, Guidelines for Analyzing and Mitigating Liquefaction in California" and "Special Publication 117A, Guidelines for Evaluating and Mitigating Seismic Hazards in California" requires liquefaction analysis to a depth of 50 feet below the lowest portion of the proposed structure. Liquefaction typically occurs in areas where the soils below the water table are composed of poorly consolidated, fine to medium-grained, primarily sandy soil. In addition to the requisite soil conditions, the ground acceleration and duration of the earthquake must also be of a sufficient level to induce liquefaction.

A review of the State of California Seismic Hazard Zone Map for the Torrance Quadrangle (CDMG, 1999) indicates that the site is not located in an area designated as “liquefiable”. In addition, according to the Safety Element of the City of Torrance General Plan (2009), the site is not located within an area identified as having a potential for liquefaction. Historic high groundwater levels in monitoring wells in the south Torrance area indicate groundwater levels have been below a depth of 80 feet for the last 70 years. Also, groundwater was not encountered in our borings drilled within the building area to a maximum depth of 61½ feet beneath the existing ground surface. Based on these considerations, it is our opinion that the potential for liquefaction and associated ground deformations beneath the site is very low.

7.5 Slope Stability

There are no known deep-seated landslides near the site, nor is the site in the path of any known or potential landslides. However, a steep north-facing slope (Slope 3) exists along the southern site boundary. This slope exposes well-bedded diatomaceous siltstone and sandstone of the Valmonte Member of the Monterey Formation and locally some massive to weakly bedded calcareous-rich sandstone and siltstone of Pleistocene age Lomita Marl. A review of the State of California Seismic Hazard Zone Map for the Torrance Quadrangle (CDMG, 1999) indicates this slope may have a potential for earthquake-induced landslides.

Slopes 1 and 2 range in height from 40 to 80 feet and are inclined at gradients ranging from 1¼:1 to 2:1 (horizontal to vertical). These slopes are underlain by San Pedro Sand and marine sand that are generally homogeneous formations and not considered bedded for the purposes of slope stability evaluation.

Slope 3 ranges from 200 to 250 feet in height. This former quarry slope has been generally graded to a uniform inclination ranging from 48 to 50 degrees and exposes Miocene age sedimentary bedrock of the Monterey Formation. Based on geologic mapping of surface exposures and downhole logging of large-diameter borings at the site, the Monterey Formation bedrock is highly fractured and generally dips in a consistent manner to the north. As shown on Figures 2A, and 3A, 3B, 3F and 3G (Geologic sections A, B, F, and G), the bedding exposed on the slope face generally dips 50 to 68 degrees to the north. This bedding orientation is favorable with respect to global stability, generally being inclined at angles steeper than the slope inclination or exhibiting a “dip slope” condition. Both these conditions are highly stable with respect to global stability. Furthermore, considering the well-developed bedding of the Monterey Formation, slope stability analysis consisting of planar analyses and wedge-failure analyses are considered appropriate for slope stability evaluation.

General Slope Stability Approach

In accordance with the current standard of practice, as outlined in the “Recommended Procedures for Implementation of DMG Special Publication 117, Guidelines for Analyzing and Mitigating Landslide Hazards in California” and “Special Publication 117A, Guidelines for Evaluating and Mitigating Seismic Hazards in California”, slopes should demonstrate the following conditions when analyzed for stability:

Global Static Stability - minimum factor of safety of 1.5

- Global Pseudo-Static Stability - minimum factor of safety of 1.0 at screening criteria
- Surficial Stability - minimum factor of safety of 1.5

Shear strength parameters were determined from laboratory direct shear tests on samples obtained during our field investigation. In addition, we reviewed the laboratory test results from Pacific Soils and Western Laboratories. Based on our review of the prior direct shear testing performed by Pacific Soils, we excluded the shear strength results from Pacific Soils boring b-101 at a depth of 50 and 95 feet. The data points for these two samples showed significant scatter and it is our opinion that a mohr-coulumb failure envelope cannot be confidently established for these tests.

The stability analyses presented herein were performed by selecting average strength parameters for each of the geologic units. The values used in the slope stability analyses are summarized in the table below.

SUMMARY OF SOIL PROPERTIES USED FOR SLOPE STABILITY ANALYSES

Material	Density (pcf)	Friction Angle (degrees)	Cohesion (psf)	Remarks
Engineered Fill	125	30	180	Saturated, Peak
Artificial Fill (af)	112	33	125	Saturated, Peak
Marine sand (Qm)	120	33	240	Saturated, Peak
San Pedro Sand (Qsp)	125	39	375	Saturated, Peak
Monterey Formation Bedrock (Tm)	75	35	200	Field Moisture, Residual

The Monterey Formation bedrock generally consists of siltstone, diatomaceous siltstone and sandstone, and clayey siltstone which are considered relatively impermeable materials and are non-waterbearing. No groundwater or water seepage was observed within the Monterey Formation bedrock during the current or prior explorations. Furthermore, it is our understanding that the portion of the property at the top of the slope will be dedicated as open space with no appreciable source of water that could inundate the hillside. Based on these considerations, it is our opinion that the use of field moisture shear strengths is appropriate given the lack of potential for the slope to become saturated.

Due to the relatively light load associated with existing and proposed structure located adjacent to the slopes, the structure loads are considered negligible with no appreciable impact to the slope stability analyses and were not incorporated into the analyses.

Pseudo-Static Approach

In accordance with the "Recommended Procedures for Implementation of DMG Special Publication 117, Guidelines for Evaluating and Mitigating Landslide Hazards in California (2002)" and the "Guidelines for Evaluating and Mitigating Seismic Hazards in California, Special Publication 117A (2008)", seismic slope performance may be evaluated by first applying a screening analysis based on the seismic coefficient (keq). The seismic coefficient (keq) is based on the maximum horizontal acceleration (MHAr) and ƒeq, a factor related to the seismicity of the site. Graphs to estimate the value of ƒeq based on slope displacement, earthquake magnitude, rupture distance, and MHAr are available in SP117A.

As recommended by SP117, seismic slope stability analysis shall be evaluated based on a maximum displacement of 5 centimeters where potential failure planes intersect stiff improvements (such as structures) and for ground motions corresponding to the Design Earthquake hazard level. The MHAr corresponding to the Design Earthquake hazard level was taken as two-thirds of the PGA_M . A value of 0.48g was used as MHAr in the analysis. As discussed in Section 7.3, the earthquake characterizing the DE peak ground acceleration is a 6.66 magnitude occurring at a hypocentral distance of 17.4 kilometers from the site.

Based on these input values, the seismic coefficient (keq) was evaluated as 0.24g and was used in the pseudo-static slope stability analysis. If the pseudo-static stability analysis results in a factor of safety of greater than 1.0, the slope passes the screening criteria and analysis of slope deformations is not required. If the resulting factor of safety is less than one, analyses to determine the estimated displacements are required.

Slope 1

Slope 1, located adjacent to Via Valmonte, ranges from approximately 40 to 80 feet in height, and is inclined at gradients ranging from 1¼:1 to 1½:1 (horizontal to vertical). This slope is prone to surficial instability as evidenced by four surficial failures observed during geologic mapping of the slope. The surficial failures range from 1 to 2 feet in thickness and are shown on Figure 2A.

Slope 1 was analyzed for gross static, pseudo-static, and surficial stability at Cross-Sections C-C’ and D-D’. Slope stability analyses were performed using the two-dimensional computer program *GeoStudio2007 Slope/W* by Geo-Slope International Ltd.

Slope 1 at Cross-Section C-C’

Slope 1 at Cross-Section C-C’ was analyzed using entry and exit with circular failure surfaces and using the GLE method.

Analysis of Slope 1 at Cross-Section C-C’ indicates a factor of safety of 1.712 for gross stability under static conditions. This exceeds the minimum required factor of safety of 1.5 and therefore the slope is considered stable with respect to gross stability under static conditions. The slope geometry, geologic structure, and calculated factor of safety are presented on Figure E1. The critical failure surface is shown on computer generated output.

Analysis of Slope 1 at Cross-Section C-C’ indicates a factor of safety of 1.137 for gross stability when subject to the seismic coefficient acceleration (keq). The factor of safety exceeds the minimum required factor of safety of 1.0. Therefore, the slope passes the screening criteria, is considered stable under pseudo-static loading, and analysis of slope deformations is not required. The slope geometry, geologic structure, and calculated factor of safety for are presented on Figure E2. The critical failure surface is shown on computer generated output.

Surficial slope stability calculations were performed for the artificial fill exposed on the slope face. The calculated factor of safety for surficial slope stability is approximately 1.0, which is less than the required minimum factor of safety of 1.5. The results of the surficial slope stability analysis indicate Slope 1 could be prone to surficial instability. It is our understanding this slope will not be graded as part of the currently proposed project and it is planned to leave this slope in its current configuration. Furthermore, two retaining walls are proposed to be constructed along Slope 1 (see Site Plan, Figure 2B). Based on this consideration, mitigation of the potential for surficial instability of Slope 1 to impact the proposed development can be achieved by incorporating a slough wall at the toe of the slope to prevent surficial failures from accumulating on-site. The slough wall should be setback from the toe of slope to provide sufficient room for maintenance and removal of any accumulated soils.

The surficial stability calculations are presented on Figures E3. Recommendations for the proposed slough wall and other mitigation measures for surficial instability of Slope 1 are presented in Section 8.15 of this report.

Slope 1 at Cross-Section D-D'

Slope 1 at Cross-Section D-D' was analyzed using entry and exit with circular failure surfaces and using the GLE method.

Analysis of Slope 1 at Cross-Section D-D' indicates a factor of safety of 1.578 for gross stability under static conditions. This exceeds the minimum required factor of safety of 1.5 and therefore the slope is considered stable with respect to gross stability under static conditions. The slope geometry, geologic structure, and calculated factor of safety are presented on Figure E4. The critical failure surface is shown on computer generated output.

Analysis of Slope 1 at Cross-Section D-D' indicates a factor of safety of 1.257 for gross stability when subject to the seismic coefficient acceleration (keq). The factor of safety exceeds the minimum required factor of safety of 1.0. Therefore, the slope passes the screening criteria, is considered stable under pseudo-static loading, and analysis of slope deformations is not required. The slope geometry, geologic structure, and calculated factor of safety for are presented on Figures E5. The critical failure surface is shown on computer generated output.

Surficial slope stability calculations were performed for the artificial fill exposed on the slope face. The calculated factor of safety for surficial slope stability is approximately 1.04, which is less than the required minimum factor of safety of 1.5. As previously discussed, this slope will not be graded as part of the currently proposed project. Mitigation of the potential for surficial instability of Slope 1 to impact the proposed development can be achieved by constructing a slough wall at the toe of the slope to prevent surficial failures from accumulating on-site. The slough wall should be setback from the toe of slope to provide sufficient room for maintenance and removal of any accumulated soils.

The surficial stability calculations are presented on Figures E6. Recommendations for the proposed slough wall and other mitigation measures for surficial instability of Slope 1 are presented in Section 8.15 of this report.

Slope 2

As a part of the proposed site development, Slope 2 will be removed for construction of the proposed flats. Therefore, stability analyses of Slope 2 are not necessary.

Slope 3

Slope stability analyses of Slope 3 were performed by a third-party consultant, GeoStabilization, Inc. A copy of the report prepared by GeoStabilization, Inc. is included herein as Appendix E. Based on the findings of their study, Slope 3 is considered stable under gross static and pseudo-static conditions; however, there is a potential for surficial instability consisting of sloughing and/or rockfall. Localized areas of surficial sloughing were observed during our geologic mapping of Slope 3 as evidenced by slough accumulation at the toe of the slope (see Geologic Map, Figure 2A). Based on our onsite observations and the results of the analysis performed by GeoStabilization, mitigation of surficial slope instability, in the form of sloughing and/or rockfall, should be incorporated into the site design. Mitigation measures are discussed in the *Rockfall Catchment and Slough Protection* section of this report (Section 8.15). The slope mitigation area is depicted on the Site Plan (see Figure 2B).

7.6 Earthquake-Induced Flooding

Earthquake-induced flooding is inundation caused by failure of dams or other water-retaining structures due to earthquakes. Based on a review of the Safety Element for the City of Torrance (2009), the site is not located within a potential inundation area for an earthquake-induced dam failure. The probability of earthquake-induced flooding is considered very low.

7.7 Tsunamis, Seiches, and Flooding

The site is not located within a coastal area. Therefore, tsunamis, seismic sea waves, are not considered a significant hazard at the site.

Seiches are large waves generated in enclosed bodies of water in response to ground shaking. No major water-retaining structures are located immediately up gradient from the project site. Flooding from a seismically-induced seiche is considered unlikely.

The site is not within a flood hazard zone as indicated by the Torrance General Plan (2009) and the County of Los Angeles (2014).

7.8 Oil Fields & Methane Potential

Based on a review of the California Division of Oil, Gas and Geothermal Resources (DOGGR) Oil and Gas Well Location Map W1-5, the site is not located within the limits of an oilfield and oil wells are not located in the immediate site vicinity. However, due to the voluntary nature of record reporting by the oil well drilling companies, wells may be improperly located or not shown on the location map and undocumented wells could be encountered during construction. Any wells encountered will need to be properly abandoned in accordance with the current requirements of the DOGGR.

Since the site is not located within the boundaries of a known oil field, the potential for the presence of methane or other volatile gases at the site is considered low. However, should it be determined that a methane study is required for the proposed development it is recommended that a qualified methane consultant be retained to perform the study and provide mitigation measures as necessary.

7.9 Subsidence

Subsidence occurs when a large portion of land is displaced vertically, usually due to the withdrawal of groundwater, oil, or natural gas. Soils that are particularly subject to subsidence include those with high silt or clay content. The site is not located within an area of known ground subsidence. No large-scale extraction of groundwater, gas, oil, or geothermal energy is occurring or planned at the site or in the general site vicinity. There appears to be little or no potential for ground subsidence due to withdrawal of fluids or gases at the site.

8.0 CONCLUSIONS AND RECOMMENDATIONS

8.1 General

8.1.1 It is our opinion that neither soil nor geologic conditions were encountered during this investigation that would preclude the construction of the proposed development provided the recommendations presented herein are followed and implemented during design and construction. This report should be considered "preliminary" and the geotechnical design parameters presented herein should be reviewed and updated once proposed grades are better established and the project progresses to a more finalized state.

8.1.2 Between 2½ and 50½ feet of existing artificial fill was encountered during the site investigation. The fill is the result of backfilling the previous mining pit, and deeper fill may exist between excavations and in other portions of the site that were not directly explored. Excavations for the proposed structures are anticipated to remove some, but not all, of the existing artificial fill. Based on our site exploration and laboratory testing, it is our opinion that the existing artificial fill is not suitable for direct support of proposed structures. The existing fill and site soils are suitable for re-use as engineered fill provided the recommendations in the *Grading* section of this report are followed (see Section 8.4).

8.1.3 The existing artificial fill contains localized lifts of concentrated concrete, brick, and rock fragments (up to 22 inches in longest dimension) with localized pockets of debris such as wire, PVC pipe, plastic and metal debris. If encountered, the debris trash should be exported from the site and should not be mixed with the fill soils. Generation of trash and oversized material (greater than 8 inches) should be anticipated. Oversized materials should be managed in accordance with the recommendations provided in the *Grading* section of this report (see Section 8.4).

8.1.4 As discussed in Section 7.1, minor shears considered secondary features associated with regional folding or past ground shaking were observed within the San Pedro Sand at the site. These secondary features are not considered active faults. However, it is recommended that special recommendations be implemented in order to mitigate the effects of small displacements that may occur along some of these secondary features as a result of an earthquake originating along the nearby Palos Verdes Fault. It is recommended that the proposed structures be supported on a blanket of engineered fill reinforced with geosynthetic materials. The reinforced engineered fill blanket will provide a ductile sublayer that can accommodate earthquake-induced ground displacement and minimize the displacements transferred to the structures. It is also recommended that the structures be decoupled from the reinforced engineered fill blanket through the placement of a of a double layer of polyolefin sheets sandwiched between layers of clean sand, placed immediately below the concrete slab-on-grade. Once the project proceeds to a more finalized state and a project structural engineer is available to provide input, additional analyses can be performed to evaluation of the required thickness of and number of geosynthetic layers in the reinforced engineered fill blanket.

8.1.5 Subsequent to the recommended grading, the proposed structures may be supported on a reinforced concrete mat foundation system. Recommendations for the design of a mat foundation system are provided in Section 8.8.

8.1.6 Retaining walls of up to 32 feet in height are anticipated as a part of the proposed project. Due to the potential for small displacements along minor shears observed in the San Pedro Sand, it is recommended that proposed retaining walls be backfilled with compressible material. Recommendations for the design of retaining walls are provided in Section 8.14.

8.1.7 Due to the nature of the proposed design and intent for a subterranean level, waterproofing of subterranean walls and slabs is suggested. Particular care should be taken in the design and installation of waterproofing to avoid moisture problems, or actual water seepage into the structure through any normal shrinkage cracks which may develop in the concrete walls, floor slab, foundations and/or construction joints. The design and inspection of the waterproofing is not the responsibility of the geotechnical engineer. A waterproofing consultant should be retained in order to recommend a product or method, which would provide protection to subterranean walls, floor slabs and foundations.

8.1.8 Excavations of up to 60 feet in vertical height are anticipated for construction of the proposed structures. It is anticipated that stable excavations can be achieved with sloping measures. Excavation recommendations are provided in the *Temporary Excavations* section of this report (Section 8.20).

8.1.9 Proposed fill slopes should be properly benched and keyed into competent native soil prior to the placement of engineered fill. Proposed cut slopes should be excavated into competent native soils or constructed with a stability fill. All slope and backcut excavations must be observed and approved in writing by the Geotechnical Engineer prior to placement of additional engineered fill. Recommendations for slope construction are provided in Section 8.5

8.1.10 Foundations for small outlying structures, such as block walls up to 6 feet high, planter walls or trash enclosures, which will not be tied to the proposed structure, may be supported on conventional foundations bearing on a minimum of 12 inches of newly placed engineered fill which extends laterally at least 12 inches beyond the foundation area. If excavation and compaction cannot be performed, such as adjacent to property lines, alternative recommendations will be required. Due to the preliminary nature of the project at this time, if required, alternative recommendations will be provided under separate cover as the project proceeds.

8.1.11 Where new paving is to be placed, it is recommended that all existing fill and unsuitable native soils be excavated and properly compacted for paving support. As a minimum, the upper twelve inches of soil should be scarified and property compacted for paving support. Paving recommendations are provided in *Preliminary Pavement Recommendations* section of this report (see Section 8.13).

8.1.12 Once the design and foundation loading configuration for the proposed structure proceeds to a more finalized plan, the recommendations within this report should be reviewed and revised, if necessary. Based on the final foundation loading configurations, the potential for settlement should be re-evaluated by this office.

8.1.13 Any changes in the design, location or elevation of improvements, as outlined in this report, should be reviewed by this office. Geocon should be contacted to determine the necessity for review and possible revision of this report.

8.2 Soil and Excavation Characteristics

8.2.1 The in-situ soil and bedrock can be excavated with moderate effort using conventional excavation equipment. Minor caving should be anticipated in unshored vertical excavations, especially where loose or granular fills are encountered. The bedrock is moderately to highly weathered and should be rippable with conventional equipment; however, concretions or well cemented layers may be encountered in the bedrock which could make excavation or drilling conditions difficult. Coring or jack-hammering may be required if concretions are encountered, and the contractor should be prepared for these conditions. All sloping cuts that expose bedrock must be inspected by a certified engineering geologist to verify the presence of out of slope bedding and to determine if there is a stability risk.

8.2.2 It is the responsibility of the contractor to ensure that all excavations and trenches are properly shored and maintained in accordance with applicable OSHA rules and regulations to maintain safety and maintain the stability of existing adjacent improvements.

8.2.3 All onsite excavations must be conducted in such a manner that potential surcharges from existing structures, construction equipment, and vehicle loads are resisted. The surcharge area may be defined by a 1:1 projection down and away from the bottom of an existing foundation or vehicle load. Penetrations below this 1:1 projection will require special excavation measures such as sloping or shoring. Excavation recommendations are provided in the *Temporary Excavations* section of this report (see Section 8.20).

8.2.4 The upper few feet of soils encountered during the investigation are considered to have a "low" (EI=33) expansive potential and are classified as "expansive" based on the 2013 California Building Code (CBC) Section 1803.5.3. Recommendations presented herein assume that the building foundations and slabs will derive support in these materials.

8.2.5 It is recommended that after finish pad grades have been achieved, laboratory testing of the subgrade soil at each building pad should be performed to reevaluate expansive potential. If laboratory test results indicate the presence of "highly expansive" soils at the building pad subgrade, additional recommendations may be required.

8.3 Minimum Resistivity, pH and Water-Soluble Sulfate

8.3.1 Potential of Hydrogen (pH) and resistivity testing as well as chloride content testing were previously performed on representative samples of soil to generally evaluate the corrosion potential to surface utilities. The tests were performed in accordance with California Test Method Nos. 643 and 422 and indicate that the soils are considered corrosive with respect to corrosion of buried ferrous metals on site. The results are presented in Appendix B (Figure B15) and should be considered for design of underground structures.

8.3.2 Laboratory tests were performed on representative samples of soil near the anticipated subterranean levels to measure the percentage of water-soluble sulfate content. Results from the laboratory water-soluble sulfate tests are presented in Appendix B (Figure B15) and indicate that the on-site materials possess "negligible" sulfate exposure to concrete structures as defined by 2013 CBC Section 1904.3 and ACI 318-11 Sections 4.2 and 4.3.

8.3.3 It is recommended that after finish pad grades have been achieved, laboratory testing of the subgrade soil at select lots should be performed to reevaluate the corrosivity characteristics.

8.3.4 Geocon West, Inc. does not practice in the field of corrosion engineering and mitigation. If corrosion sensitive improvements are planned, it is recommended that a corrosion engineer be retained to evaluate corrosion test results and incorporate the necessary precautions to avoid premature corrosion of buried metal pipes and concrete structures in direct contact with the soils.

8.4 Grading

8.4.1 Earthwork should be observed, and compacted fill tested by representatives of Geocon West, Inc.

8.4.2 A preconstruction conference should be held at the site prior to the beginning of grading operations with the owner, contractor, civil engineer, building official, and geotechnical engineer in attendance. Special soil handling requirements can be discussed at that time.

8.4.3 It shall be the sole responsibility of the contractor to provide adequate equipment and methods to accomplish the work in accordance with applicable grading codes or agency ordinances, the recommendations presented herein, and the approved grading plans.

8.4.4 The existing artificial fill, native soils, and bedrock encountered during exploration are suitable for re-use as an engineered fill, provided any encountered deleterious debris is removed. Pockets of trash and debris may be encountered within the deeper artificial fill. If encountered, the trash and debris should be exported from the site and should not be mixed with the fill soils. Generation of oversized material (greater than 8 inches) should be anticipated. Rocks larger than 8 inches but less than 4 feet in maximum dimension may be incorporated into the engineered fill. Placement of oversized material (larger than 8 inches) shall be limited to the area measured at least 15 feet horizontally from the nearest slope face and 10 feet below finish grade or 3 feet below the deepest utility, whichever is deeper. It is recommended that where non-building areas are available, placement of oversized material should be performed in these areas.

8.4.5 If bedrock is to be utilized as an engineered fill, it may be blocky and may have to be crushed and moisture conditioned prior to utilization.

8.4.6 All materials utilized as engineered fill should be well-blended to create a uniform fill material prior to placement and compaction within each building pad area or slope construction. Soils must be placed uniformly and at equal thickness at the direction of the Geotechnical Engineer (a representative of Geocon West, Inc.).

8.4.7 Grading should commence with the removal of all existing vegetation and existing improvements from the area to be graded. All existing underground improvements planned for removal should be completely excavated and the resulting depressions properly backfilled in accordance with the procedures described herein. Deleterious debris such as wood and root structures should be exported from the site and should not be mixed with the fill soils.

8.4.8 Asphalt and concrete should not be mixed with the fill soils unless approved by the Geotechnical Engineer. All existing underground improvements planned for removal should be completely excavated and the resulting depressions properly backfilled in accordance with the procedures described herein.

8.4.9 During grading operations, the Geotechnical Engineer (a representative of Geocon) should be onsite to observe that soil and geologic conditions do not differ significantly from those expected. If conditions are found to be variable, modification to the grading recommendations described herein should be implemented based on onsite observations. This may include deeper excavations to remove artificial fill or unsuitable soils, or reducing excavations where competent soil is encountered at shallower depths than anticipated.

8.4.10 It is recommended that the proposed structures be supported on a blanket of engineered fill reinforced with geosynthetic materials. The reinforced engineered fill blanket will provide a ductile sublayer that can accommodate earthquake-induced ground displacement and minimize the transfer of the displacements to the structures. It is also recommended that the structures be decoupled from the reinforced engineered fill blanket through the placement of a of a double layer of polyolefin sheets sandwiched between layers of clean sand, placed immediately below the concrete slab-on-grade. Once the project proceeds to a more finalized state and a project structural engineer is available to provide input, additional analyses can be performed to evaluation of the required thickness of and number of geosynthetic layers in the reinforced engineered fill blanket.

8.4.11 All excavations must be observed and approved in writing by the Geotechnical Engineer (a representative of Geocon), prior to placing fill, gravel, or construction materials. If determined to be excessively soft, additional removals or stabilization of the excavation bottom may be required in order to provide a firm working surface upon which engineered fill can be placed and heavy equipment can operate. If required, recommendations for stabilization measures can be provided under separate cover.

8.4.12 All fill and backfill soils should be placed in horizontal loose layers approximately 6 to 8 inches thick, moisture conditioned to optimum moisture content, and properly compacted to a minimum 90 percent of the maximum dry density in accordance with ASTM D 1557 (latest edition).

8.4.13 Although not anticipated for this project, all imported fill shall be observed, tested, and approved by Geocon West, Inc. prior to bringing soil to the site. Rocks larger than 8 inches in diameter shall not be used in the fill. If necessary, import soils used as structural fill should have an expansion index less than 20 and corrosivity properties that are equally or less detrimental to that of the existing onsite soils (see Figure B15).

8.4.14 Where the slope ratio of the existing ground or temporary backcut is steeper than 6:1 (horizontal:vertical), the ground should be benched into competent soil in accordance the illustration provided as Figure 6.

8.4.15 Where new paving is to be placed, it is recommended that all existing fill and unsuitable native soils be excavated and properly compacted for paving support. As a minimum, the upper twelve inches of soil should be scarified and compacted to at least 95 percent relative compaction where placing and compacting granular soils, and 92 percent relatively compaction where placing and compacting fine-grained soils for paving support. Paving recommendations are provided in *Preliminary Pavement Recommendations* section of this report (see Section 8.13).

8.4.16 It is recommended that flexible utility connections be utilized for all rigid utilities to minimize or prevent damage to utilities from minor differential movements. Utility trenches should be properly backfilled in accordance with the requirements of the Green Book (latest edition). The pipe should be bedded with clean sands (Sand Equivalent greater than 30) to a depth of at least one foot over the pipe, and the bedding material must be inspected and approved in writing by the Geotechnical Engineer (a representative of Geocon). The use of gravel is not acceptable unless used in conjunction with filter fabric to prevent the gravel from having direct contact with soil. The remainder of the trench backfill may be derived from onsite soil or approved import soil, compacted as necessary, until the required compaction is obtained. The use of minimum 2-sack slurry is also acceptable. Prior to placing any bedding materials or pipes, the excavation bottom must be observed and approved in writing by the Geotechnical Engineer (a representative of Geocon).

8.4.17 All trench and foundation excavation bottoms must be observed and approved in writing by the Geotechnical Engineer (a representative of Geocon), prior to placing bedding material, fill, steel, gravel or concrete.

8.5 Slope Construction

8.5.1 Fill slopes comprised of on-site materials should be constructed at a gradient of 2:1 or flatter. Fill slopes should be overbuilt by at least 3 feet measured perpendicular to the slope face and trimmed back to the tight fill core. This procedure is considered preferable to track-walking of slopes, as described in the following paragraph.

8.5.2 As an alternative, fill slope faces may be compacted by backrolling with a loaded sheepsfoot roller at vertical intervals not to exceed 4 feet, and should be track-walked at the completion of each slope such that the fill is compacted to a dry density of at least 90 percent of the laboratory maximum dry density and near or slightly above optimum moisture content to the face of the finished sloped.

8.5.3 Where the slope ratio of the existing ground or temporary backcut is steeper than 6:1 (horizontal:vertical), preparation for the construction of proposed slopes should include removal of all vegetation and unsuitable soils. Prior to the placement of engineered fill, the existing grade should be benched and keyed into competent native soil (see Figure 6). All backcut excavations must be observed and approved in writing by the Geotechnical Engineer prior to placement of engineered fill.

8.5.4 Cut slope excavations, including buttresses and shear keys, must be observed by the Geotechnical Engineer (a representative of Geocon) during grading operations to check that soil and geologic conditions do not differ significantly from those expected. Cut slopes which are excavated into soft or unsuitable soil or bedrock may require stabilization. Typically stabilization measures consist of drains and/or buttress fills. Stabilization design and details are depicted on Figure 7.

8.5.5 During the construction of buttresses, there is a risk that the temporary backcut slopes will become unstable. This risk can be reduced by grading the buttress fill in short segments and/or flattening the inclination of the temporary slope.

8.5.6 All graded slopes should be planted, drained, and property maintained to reduce erosion. It is recommended that finished slopes be planted as soon after completion of grading as possible. Planting on the slope stabilizes the surface and reduces the potential for erosion. It is further suggested that a jute or mesh product be placed on the slope face prior to planting; however, the planting of the slope should be performed at the direction of a qualified landscaping consultant.

8.6 Shrinkage

8.6.1 Shrinkage results when a volume of material removed at one density is compacted to a higher density. A shrinkage factor of between 10 and 20 percent should be anticipated when excavating and compacting the existing earth materials on the site to an average relative compaction of 92 percent.

8.6.2 Removal of existing underground structures or pockets of debris which are not considered suitable for reuse as engineered fill may increase the need for import soils.

8.6.3 If import soils will be utilized in the building pad or for slope construction, the soils must be placed uniformly and at equal thickness at the direction of the Geotechnical Engineer (a representative of Geocon West, Inc.). Soils can be borrowed from non-building and later replaced with imported soils.

8.7 Foundation Setback

8.7.1 The Building Code requires that foundations be sufficiently setback from an ascending or descending slope. The required setback from a descending slope is 1/3 the height of the descending slope with a minimum of 5 feet and a maximum of 40 feet measured horizontally from the exterior face of the foundation to the slope face. In lieu of relocating a structure to achieve the setback at the ground surface, foundations may be deepened as necessary to achieve the required setback.

8.7.2 The required setback from an ascending slope is 1/2 the height of the ascending slope with a maximum of 15 feet measured horizontally from the exterior face of the structure to the toe of the slope. Where a retaining wall is utilized the setback is measured from a projected toe of slope.

8.7.3 The required building setbacks should be understood and implemented into the orientation and location of the proposed structures by the project architect.

8.8 Mat Foundation Design

8.8.1 Subsequent to the recommended grading, it is recommended that a reinforced concrete mat foundation be utilized for support of the proposed on-grade and subterranean portions of the structure. The reinforced concrete mat foundation may derive support in the newly placed engineered fill.

8.8.2 The recommended maximum allowable bearing value is 5,000 pounds per square foot (psf). The allowable bearing pressure may be increased by up to one-third for transient loads due to wind or seismic forces.

8.8.3 It is recommended that a modulus of subgrade reaction of 150 pounds per cubic inch (pci) be utilized for the design of the mat foundation bearing in newly placed engineered fill. This value is a unit value for use with a one-foot square footing. The modulus should be reduced in accordance with the typical reduction factors when used with larger foundations.

8.8.4 The thickness of and reinforcement for the mat foundation should be designed by the project structural engineer.

8.8.5 It is recommended that the structures be decoupled from the reinforced engineered fill blanket through the placement of a of a double layer of polyolefin sheets sandwiched between layers of clean sand, placed immediately below the concrete slab-on-grade. Based on the intent of the polyolefin sheets, frictional resistance should not be assumed between the mat foundation and the subgrade soils.

8.8.6 Foundation excavations should be observed and approved in writing by the Geotechnical Engineer (a representative of Geocon West, Inc.), prior to the placement of reinforcing steel and concrete to verify that the exposed soil conditions are consistent with those anticipated. If unanticipated soil conditions are encountered, foundation modifications may be required.

8.8.7 Waterproofing of subterranean walls and slabs is recommended for this project. Particular care should be taken in the design and installation of waterproofing to avoid moisture problems, or actual water seepage into the structure through any normal shrinkage cracks which may develop in the concrete walls, floor slab, foundations and/or construction joints. The design and inspection of the waterproofing is not the responsibility of the geotechnical engineer. A waterproofing consultant should be retained in order to recommend a product or method which would provide protection to subterranean walls, floor slabs and foundations.

8.8.8 This office should be provided a copy of the final construction plans so that the recommendations presented herein could be properly reviewed and revised if necessary.

8.9 Foundation Settlement

8.9.1 The maximum expected total settlement for a structure supported on a conventional foundation system designed with the maximum allowable bearing value of 5,000 psf and deriving support in the recommended bearing material at the excavation bottom is estimated to be approximately less than 2 inches and occur below the heaviest loaded structural element. Settlement of the foundation system is expected to occur on initial application of loading. Differential settlement is expected to be less than 1 inch over a distance of 20 feet.

8.9.2 It is recommended that flexible utility connections be utilized for all rigid utilities to minimize or prevent damage to utilities from minor differential movements.

8.9.3 Once the design and foundation loading configurations for the proposed structures proceeds to a more finalized plan, the estimated settlements presented in this report should be reviewed and revised, if necessary. If the final foundation loading configurations are greater than the assumed loading conditions the potential for settlement should be reevaluated by this office.

8.10 Miscellaneous Foundations

8.10.1 Foundations for small outlying structures, such as block walls less than 6 feet in height, planter walls or trash enclosures, which will not be tied-in to the proposed structures, may be supported on conventional foundations bearing on a minimum of 12 inches of newly placed engineered fill. If excavation and compaction cannot be performed, such as adjacent to property lines, alternative foundation recommendations will be required. Alternative recommendations will be provided under separate cover as the project proceeds.

8.10.2 Miscellaneous foundations may be designed for a bearing value of 1,500 psf, and should be a minimum of 12 inches in width, 18 inches in depth below the lowest adjacent grade and 12 inches into the recommended bearing material. The allowable bearing pressure may be increased by up to one-third for transient loads due to wind or seismic forces.

8.10.3 Foundation excavations should be observed and approved in writing by the Geotechnical Engineer (a representative of Geocon West, Inc.), prior to the placement of reinforcing steel and concrete to verify that the excavations and exposed soil conditions are consistent with those anticipated.

8.11 Lateral Design

8.11.1 Resistance to lateral loading may be provided by friction acting at the base of foundations, slabs and by passive earth pressure. An allowable coefficient of friction of 0.35 may be used with the dead load forces in newly placed engineering fill, undisturbed marine sand, and San Pedro Sands; an allowable coefficient of friction of 0.4 may be used with the dead load forces in bedrock.

8.11.2 Passive earth pressure for the sides of foundations and slabs poured against newly placed engineering fill, undisturbed marine sand, and San Pedro Sands may be computed as an equivalent fluid having a density of 250 pounds per cubic foot (pcf) with a maximum earth pressure of 2,500 psf. Passive earth pressure for the sides of foundations and slabs poured against bedrock may be computed as an equivalent fluid having a density of 600 pcf with a maximum earth pressure of 6,000 pcf. When combining passive and friction for lateral resistance, the passive component should be reduced by one-third. A one-third increase in the passive value may be used for wind or seismic loads.

8.12 Exterior Concrete Slabs-on-Grade

8.12.1 Exterior concrete slabs-on-grade at the ground surface subject to vehicle loading should be designed in accordance with the recommendations in the *Preliminary Pavement Recommendations* section of this report (Section 8.13).

8.12.2 Exterior slabs, not subject to traffic loads, should be at least 4 inches thick and reinforced with No. 3 steel reinforcing bars placed 18 inches on center in both horizontal directions, positioned near the slab midpoint. Prior to construction of slabs, the upper 12 inches of subgrade should be moistened to two percent above optimum moisture content and properly compacted to at least 95 percent relative compaction, as determined by ASTM Test Method D 1557 (latest edition). Crack control joints should be spaced at intervals not greater than 10 feet and should be constructed using saw-cuts or other methods as soon as practical following concrete placement. Crack control joints should extend a minimum depth of one-fourth the slab thickness. The project structural engineer should design construction joints as necessary.

8.12.3 The moisture content of the slab subgrade should be maintained and sprinkled as necessary to maintain a moist condition as would be expected in any concrete placement.

8.12.4 The recommendations of this report are intended to reduce the potential for cracking of slabs due to settlement. However, even with the incorporation of the recommendations presented herein, foundations, stucco walls, and slabs-on-grade may exhibit some cracking due to minor soil movement and/or concrete shrinkage. The occurrence of concrete shrinkage cracks is independent of the supporting soil characteristics. Their occurrence may be reduced and/or controlled by limiting the slump of the concrete, proper concrete placement and curing, and by the placement of crack control joints at periodic intervals, in particular, where re-entrant slab corners occur.

8.13 Preliminary Pavement Recommendations

8.13.1 Where new paving is to be placed, it is recommended that all existing fill and soft soils be excavated and properly compacted for paving support. The client should be aware that excavation and compaction of all existing artificial fill and soft soils in the area of new paving is not required; however, paving constructed over existing uncertified fill or unsuitable soils may experience increased settlement and/or cracking, and may therefore have a shorter design life and increased maintenance costs. As a minimum, the upper 12 inches of paving subgrade should be scarified, moisture conditioned to two percent above optimum moisture content, and properly compacted to at least 95 percent relative compaction, as determined by ASTM Test Method D 1557 (latest edition).

8.13.2 The following pavement sections are based on an assumed R-Value of 20. Once site grading activities are complete an R-Value should be obtained by laboratory testing to confirm the properties of the soils serving as paving subgrade, prior to placing pavement.

8.13.3 The Traffic Indices listed below are estimates. Geocon does not practice in the field of traffic engineering. The actual Traffic Index for each area should be determined by the project civil engineer. If pavement sections for Traffic Indices other than those listed below are required, Geocon should be contacted to provide additional recommendations. Pavement thicknesses were determined following procedures outlined in the *California Highway Design Manual* (Caltrans). It is anticipated that the majority of traffic will consist of automobile and large truck traffic.

PRELIMINARY PAVEMENT DESIGN SECTIONS

Location	Estimated Traffic Index (TI)	Asphalt Concrete (inches)	Class 2 Aggregate Base (inches)
Automobile Parking and Driveways	4.0	3.0	4.0
Trash Truck & Fire Lanes	7.0	4.0	12.0

8.13.4 Asphalt concrete should conform to Section 203-6 of the "*Standard Specifications for Public Works* Construction" (Green Book). Class 2 aggregate base materials should conform to Section 26-1.02A of the "*Standard Specifications of the State of California, Department of Transportation*" (Caltrans). The use of Crushed Miscellaneous Base (CMB) in lieu of Class 2 aggregate base is acceptable. Crushed Miscellaneous Base should conform to Section 200-2.4 of the "*Standard Specifications for Public Works Construction*" (Green Book).

8.13.5 Unless specifically designed and evaluated by the project structural engineer, where exterior concrete paving will be utilized for support of vehicles, it is recommended that the concrete be a minimum of 6 inches of concrete reinforced with No. 3 steel reinforcing bars placed 18 inches on center in both horizontal directions. Concrete paving supporting vehicular traffic should be underlain by a minimum of 4 inches of aggregate base and a properly compacted subgrade. The subgrade and base material should be compacted to 95 percent relative compaction, as determined by ASTM Test Method D 1557 (latest edition).

8.13.6 The performance of pavements is highly dependent upon providing positive surface drainage away from the edge of pavements. Ponding of water on or adjacent to the pavement will likely result in saturation of the subgrade materials and subsequent cracking, subsidence and pavement distress. If planters are planned adjacent to paving, it is recommended that the perimeter curb be extended at least 12 inches below the bottom of the aggregate base to minimize the introduction of water beneath the paving.

8.14 Retaining Walls Design

8.14.1 The recommendations presented below are generally applicable to the design of rigid concrete or masonry retaining walls having a maximum height of 40 feet. In the event that walls higher than 40 feet are planned, Geocon should be contacted for additional recommendations.

8.14.2 Retaining wall foundations may be designed in accordance with the recommendations provided in the *Mat Foundation Design* section of this report (see Section 8.8).

8.14.3 Due to the potential for minor displacements along secondary shears observed in the San Pedro Sand, it is recommended that proposed retaining walls be backfilled with compressible material.

8.14.4 The wall pressures provided below assume that the proposed retaining walls will support newly placed engineered fill. This is based on the assumption that sloping techniques will be utilized for construction of the proposed walls, which would result in a wedge of engineered fill behind the retaining walls. If proposed retaining walls will support undisturbed soils, revised earth pressures will be required. This should be evaluated once temporary excavations are established.

8.14.5 Where engineered fill is to be retained, retaining walls should be designed utilizing the equivalent fluid pressures indicated in the following tables. Walls not restrained at the top (cantilevered) may utilize the active pressure. Restrained walls are those that are not allowed to rotate more than 0.001H (where H equals the height of the retaining portion of the wall in feet) at the top of the wall. Where walls are restrained from movement at the top the at-rest pressure should be considered.

Retaining Wall with Level Backfill Surface

HEIGHT OF WALL (FEET)	EQUIVALENT FLUID PRESSURE (Pounds Per Cubic Foot) (ACTIVE PRESSURE)	EQUIVALENT FLUID PRESSURE (Pounds Per Cubic Foot) (AT-REST PRESSURE)
Up to 15	40	60
Up to 25	47	68

Retaining Wall with Surcharge from 2:1 Slope

HEIGHT OF WALL (FEET)	EQUIVALENT FLUID PRESSURE (Pounds Per Cubic Foot) (ACTIVE PRESSURE)	EQUIVALENT FLUID PRESSURE (Pounds Per Cubic Foot) (AT-REST PRESSURE)
Up to 15	70	95
Up to 32	90	105

8.14.6 The wall pressures provided above assume that the retaining wall will be properly drained preventing the buildup of hydrostatic pressure. If retaining wall drainage is not implemented, the equivalent fluid pressure to be used in design of undrained walls is 90 pcf. The value includes hydrostatic pressures plus buoyant lateral earth pressures.

8.14.7 Where freeboard for sloughing and impact will be incorporated into retaining wall design, it is recommended that the freeboard be a minimum height of 18 inches and be designed for a pressure of 125 pcf.

8.14.8 Additional active pressure should be added for a surcharge condition due to slopes, vehicular traffic or adjacent structures and should be designed for each condition.

8.14.9 The wall pressures provided above assume that the retaining wall will be properly drained preventing the buildup of hydrostatic pressure. If retaining wall drainage is not implemented, the equivalent fluid pressure to be used in design of undrained walls is 90 pcf. The value includes hydrostatic pressures plus buoyant lateral earth pressures.

8.14.10 In addition to the recommended earth pressure, the upper ten feet of the retaining wall adjacent to the street or driveway areas should be designed to resist a uniform lateral pressure of 100 psf, acting as a result of an assumed 300 psf surcharge behind the shoring due to normal street traffic. If the traffic is kept back at least 10 feet from the shoring, the traffic surcharge may be neglected.

8.14.11 Seismic lateral forces should be incorporated into the design as necessary, and recommendations for seismic lateral forces are presented in Section 8.16.

8.15 Rockfall Catchment and Slough Protection (Slopes 1 and 3)

8.15.1 As discussed in the Slope Stability section of this report (Section 7.5), Slope 1 could be prone to surficial instability. Mitigation of the potential for surficial instability of Slope 1 to impact the proposed development can be achieved by incorporating a slough wall at the toe of the slope to prevent surficial failures from accumulating on-site. The slough wall should be setback at least 5 feet from the toe of slope to provide sufficient room for maintenance and removal of any accumulated soils. The wall should be a minimum of 24 inches in height and designed for impact utilizing a triangular distribution of pressure of 125 pcf. In addition, it is suggested that the face of Slope 1 be treated with a chemical binder applied to the exposed granular soils to improve surficial stability.

8.15.2 Based on the configuration of the Slope 3 and the observed accumulation of slough at the toe of this slope, as well as the results of the rock mechanics and rockfall analysis performed by GeoStabilization International (see Appendix D), there is a potential for sloughing and rockfall from Slope 3 to occur periodically as a result of future weathering, rainfall, and seismic activity. It is recommended that rockfall mitigation consisting of a catchment area and/or a rockfall containment barrier at the base of Slope 3 be incorporated into the site design. Furthermore, the analyses performed by GeoStabilization indicate that a horizontal setback of 40 feet, when combined with a rockfall catchment area or containment barrier as described below, will be sufficient to retain all potential rockfall.

8.15.3 Where sufficient space is available, a rockfall catchment area should be constructed adjacent to the structure at the base of Slope 3. As shown on Figure 8, the rockfall catchment area can be either flat or sloped and utilize a berm or containment barriers to retain potential slough or rockfall. Based on the analysis performed by GeoStabilization International (see Appendix D), rockfall catchment areas which will retain all potential rockfall are depicted as Details 1 and 2 on Figure 8. Where there is insufficient room to construct a rockfall catchment area, a rockfall containment barrier may be constructed. The rockfall containment barrier may consist of a typical concrete barrier (aka a jersey barrier) or a geosynthetically confined soil (GCS) wall. A GSC wall consists of soil reinforced with closely spaced geosyntetic reinforcement; is flexible, which allows it to absorb impact energy efficiently and without shattering; and has many options for wall facing. Where a rockfall containment barrier is incorporated into the catchment area (see Details 3A and 3B, Figure 8), the analyses performed by GeoStabilization indicate that the barrier should be designed to absorb an average kinetic energy of 20 kilojoules.

8.15.4 Where space is limited, a rockfall containment barrier may be constructed at the top of retaining walls to mitigate rockfall hazards generated from Slope 3. Based on the analysis performed by GeoStabilization International (see Appendix D), a rockfall containment barrier which will retain all potential rockfall is depicted as Detail 4 on Figure 8. Where a rockfall containment barrier is incorporated into the retaining wall design, the analyses performed by GeoStabilization indicate that the barrier should be designed to absorb an average kinetic energy on the order of 20 kilojoules.

8.15.5 A slough wall may also be installed to contain runout material and other debris from impacting the residences at the base of Slope 3. The wall should be a minimum of 24 inches in height and designed for impact utilizing a triangular distribution of pressure of 125 pcf.

8.15.6 The area upslope of slough walls and debris containment fences will require periodic evaluations and maintenance; accumulations of debris shall be removed on a periodic basis and after severe storms. It is recommended that the developer of the site create and enforce a written maintenance plan.

8.16 Dynamic (Seismic) Lateral Forces

8.16.1 The structural engineer should determine the seismic design category for the project in accordance with Section 1613 of the CBC. If the project possesses a seismic design category of D, E, or F, proposed retaining walls in excess of 6 feet in height should be designed with seismic lateral pressure (Section 1803.5.12 of the 2013 CBC).

8.16.2 A seismic load of 20 pcf should be used for design of walls that support more than 6 feet of backfill in accordance with Section 1803.5.12 of the 2013 CBC. The seismic load is applied as an equivalent fluid pressure along the height of the wall and the calculated loads result in a maximum load exerted at the base of the wall and zero at the top of the wall. This seismic load should be applied in addition to the active earth pressure. The earth pressure is based on half of two thirds of PGA_M calculated from ASCE 7-10 Section 11.8.3.

8.17 Retaining Wall Drainage

8.17.1 Retaining walls should be provided with a drainage system extended at least two-thirds the height of the wall. At the base of the drain system, a subdrain covered with a minimum of 12 inches of gravel should be installed, and a compacted fill blanket or other seal placed at the surface (see Figure 9). The clean bottom and subdrain pipe, behind a retaining wall, should be observed by the Geotechnical Engineer (a representative of Geocon), prior to placement of gravel or compacting backfill.

8.17.2 As an alternative, a plastic drainage composite such as Miradrain or equivalent may be installed in continuous, 4-foot-wide columns along the entire back face of the wall, at 8 feet on center. The top of these drainage composite columns should terminate approximately 18 inches below the ground surface, where either hardscape or a minimum of 18 inches of relatively cohesive material should be placed as a cap (see Figure 10).

8.17.3 Subdrainage pipes at the base of the retaining wall drainage system should outlet to an acceptable location via controlled drainage structures. Drainage should not be allowed to flow uncontrolled over descending slopes.

8.17.4 Moisture affecting below grade walls is one of the most common post-construction complaints. Poorly applied or omitted waterproofing can lead to efflorescence or standing water. Particular care should be taken in the design and installation of waterproofing to avoid moisture problems, or actual water seepage into the structure through any normal shrinkage cracks which may develop in the concrete walls, floor slab, foundations and/or construction joints. The design and inspection of the waterproofing is not the responsibility of the geotechnical engineer. A waterproofing consultant should be retained in order to recommend a product or method, which would provide protection to subterranean walls, floor slabs and foundations.

8.18 Soil Nail Wall Design

8.18.1 Soil nail walls consist of installing closely spaced steel bars (nails) into a slope or excavation in a top-down construction sequence. Following installation of a horizontal row of nails, drains, waterproofing, and wall reinforcing steel are placed and shotcrete applied to create a final wall.

8.18.2 Soil nail walls should not be considered a permanent design to support the seismic lateral loads and soil pressures on a building wall. Therefore, the proposed building should be designed to support the expected lateral loads.

8.18.3 The wall should be designed by an engineer familiar with the design of soil nail walls.

8.18.4 In general, ground conditions are moderately suited to soil nail wall construction techniques. However, localized gravel, cobble, and oversized material could be encountered in the existing materials that could be difficult to drill. Additionally, relatively clean sands may be encountered within the existing soil that may result in some raveling of the unsupported excavation.

8.18.5 A wall drain system should be incorporated into the design of the soil nail wall. Corrosion protection should be provided for the nails if the wall will be a permanent structure.

8.18.6 Testing of the soil nails should be performed in accordance with the guidelines of the Federal Highway Administration or similar guidelines. At least two verification tests should be performed to confirm design assumptions for each soil/rock type encountered. Verification tests nails should be sacrificial and should not be used to support the proposed wall. The bond length should be adjusted to allow for pullout testing of the verification nails to evaluate the ultimate bond stress. A minimum of 5 percent of the production nails should also be proof tested and a minimum of 4 sacrificial nails should be tested at the discretion of Geocon West, Inc. Consideration should be given to testing sacrificial nails with an adjusted bond length rather than testing production nails. Geocon West, Inc. should observe the nail installation and document the nail testing.

8.18.7 The soil strength parameters listed in the table below can be used in design of the soil nails. The bond stress is dependent on drilling method, diameter, and construction method. Therefore, the designer should evaluate the bond stress based on the existing soil conditions and the construction method.

SOIL STRENGTH PARAMETERS FOR SOIL NAIL WALLS

Description	**Unit Weight (Pounds Per Cubic Foot)**	**Friction Angle (Degrees)**	**Cohesion (Pounds Per Cubic Foot)**
Artificial Fill	112	33	125
San Pedro Sand (Qsp)	125	39	375

8.19 Elevator Pit Design

8.19.1 Based on the potential for secondary fault displacements, it is the intent of the geotechnical grading and foundation recommendations to decouple the proposed structures from the ground in order to minimize the earthquake-induced ground displacement transferred to the structures. In order to minimize the potential for ground displacement transferred to the structures as a result of the elevator pits, it is recommended that a cold-joint be used where the pits connect to the mat foundation.

8.19.2 The elevator pit slab and retaining wall should be designed by the project structural engineer. As a minimum the slab-on-grade for the elevator pit bottom should be at least 4 inches thick and reinforced with No. 4 steel reinforcing bars placed 16 inches on center in both horizontal directions, positioned near the slab midpoint. Elevator pit walls may be designed in accordance with the recommendations in the Mat Foundation Design and Retaining Wall Design sections of this report (see Sections 8.8 and 8.14).

8.19.3 Additional active pressure should be added for a surcharge condition due to sloping ground, vehicular traffic, or adjacent foundations and should be designed for each condition as the project progresses.

8.19.4 If retaining wall drainage is to be provided, the drainage system should be designed in accordance with the Retaining Wall Drainage section of this report (see Section 8.17).

8.19.5 It is suggested that the exterior walls and slab be waterproofed to prevent excessive moisture inside of the elevator pit. Waterproofing design and installation is not the responsibility of the geotechnical engineer.

8.20 Temporary Excavations

8.20.1 Excavations on the order of 60 feet in vertical height may be required during excavation and construction of the proposed subterranean level and foundations. The excavations are expected to expose artificial fill, marine sand, San Pedro Sands, and bedrock, which are suitable for vertical excavations. Due to the adverse bedding and jointing configuration of bedrock, excavations into bedrock should be sloped or shored in order to provide a stable excavation. All cut slopes should be observed by the Project Geologist (a representative of Geocon West, Inc.) during excavation. Where adverse bedding is encountered the bedrock may be trimmed along the angle of bedding.

8.20.2 Vertical excavations greater than 5 feet will require sloping, shoring, or other special excavation measures in order to provide a stable excavation. Where sufficient space is available, temporary unsurcharged embankments up to 15 feet high could be sloped back at a uniform 1:1 slope gradient or flatter. Slopes higher than 15 feet may be sloped back at a uniform 2:1 slope gradient or flatter. A uniform slope does not have a vertical portion.

8.20.3 Where sloped embankments are utilized, the top of the slope should be barricaded to prevent vehicles and storage loads at the top of the slope within a horizontal distance equal to the height of the slope. If the temporary construction embankments are to be maintained during the rainy season, berms are suggested along the tops of the slopes where necessary to prevent runoff water from entering the excavation and eroding the slope faces. Geocon personnel should inspect the soils exposed in the cut slopes during excavation so that modifications of the slopes can be made if variations in the soil conditions occur. All excavations should be stabilized within 30 days of initial excavation.

8.21 Surface Drainage

8.21.1 Proper surface drainage is critical to the future performance of the project. Uncontrolled infiltration of irrigation excess and storm runoff into the soils can adversely affect the performance of the planned improvements. Saturation of a soil can cause it to lose internal shear strength and increase its compressibility, resulting in a change in the original designed engineering properties. Proper drainage should be maintained at all times.

8.21.2 All site drainage should be collected and controlled in non-erosive drainage devices. Drainage should not be allowed to pond anywhere on the site, and especially not against any foundation or retaining wall. The site should be graded and maintained such that surface drainage is directed away from structures in accordance with 2013 CBC 1804.3 or other applicable standards. In addition, drainage should not be allowed to flow uncontrolled over any descending slope. Discharge from downspouts, roof drains and scuppers are not recommended onto unprotected soils within five feet of the building perimeter. Planters which are located adjacent to foundations should be sealed to prevent moisture intrusion into the soils providing foundation support. Landscape irrigation is not recommended within 5 feet of the building perimeter footings except when enclosed in protected planters.

8.21.3 Positive site drainage should be provided away from structures, pavement, and the tops of slopes to swales or other controlled drainage structures. The building pad and pavement areas should be fine graded such that water is not allowed to pond.

8.21.4 Landscaping planters immediately adjacent to paved areas are not recommended due to the potential for surface or irrigation water to infiltrate the pavement's subgrade and base course. Either a subdrain, which collects excess irrigation water and transmits it to drainage structures, or an impervious above-grade planter boxes should be used. In addition, where landscaping is planned adjacent to the pavement, it is recommended that consideration be given to providing a cutoff wall along the edge of the pavement that extends at least 12 inches below the base material.

8.22 Plan Review

8.22.1 Grading, foundation plans, and shoring should be reviewed by the Geotechnical Engineer (a representative of Geocon West, Inc.), prior to finalization to verify that the plans have been prepared in substantial conformance with the recommendations of this report and to provide additional analyses or recommendations.

LIMITATIONS AND UNIFORMITY OF CONDITIONS

1. The recommendations of this report pertain only to the site investigated and are based upon the assumption that the soil conditions do not deviate from those disclosed in the investigation. If any variations or undesirable conditions are encountered during construction, or if the proposed construction will differ from that anticipated herein, Geocon West, Inc. should be notified so that supplemental recommendations can be given. The evaluation or identification of the potential presence of hazardous or corrosive materials was not part of the scope of services provided by Geocon West, Inc.

2. This report is issued with the understanding that it is the responsibility of the owner, or of his representative, to ensure that the information and recommendations contained herein are brought to the attention of the architect and engineer for the project and incorporated into the plans, and the necessary steps are taken to see that the contractor and subcontractors carry out such recommendations in the field.

3. The findings of this report are valid as of the date of this report. However, changes in the conditions of a property can occur with the passage of time, whether they are due to natural processes or the works of man on this or adjacent properties. In addition, changes in applicable or appropriate standards may occur, whether they result from legislation or the broadening of knowledge. Accordingly, the findings of this report may be invalidated wholly or partially by changes outside our control. Therefore, this report is subject to review and should not be relied upon after a period of three years.

4. The firm that performed the geotechnical investigation for the project should be retained to provide testing and observation services during construction to provide continuity of geotechnical interpretation and to check that the recommendations presented for geotechnical aspects of site development are incorporated during site grading, construction of improvements, and excavation of foundations. If another geotechnical firm is selected to perform the testing and observation services during construction operations, that firm should prepare a letter indicating their intent to assume the responsibilities of project geotechnical engineer of record. A copy of the letter should be provided to the regulatory agency for their records. In addition, that firm should provide revised recommendations concerning the geotechnical aspects of the proposed development, or a written acknowledgement of their concurrence with the recommendations presented in our report. They should also perform additional analyses deemed necessary to assume the role of Geotechnical Engineer of Record.

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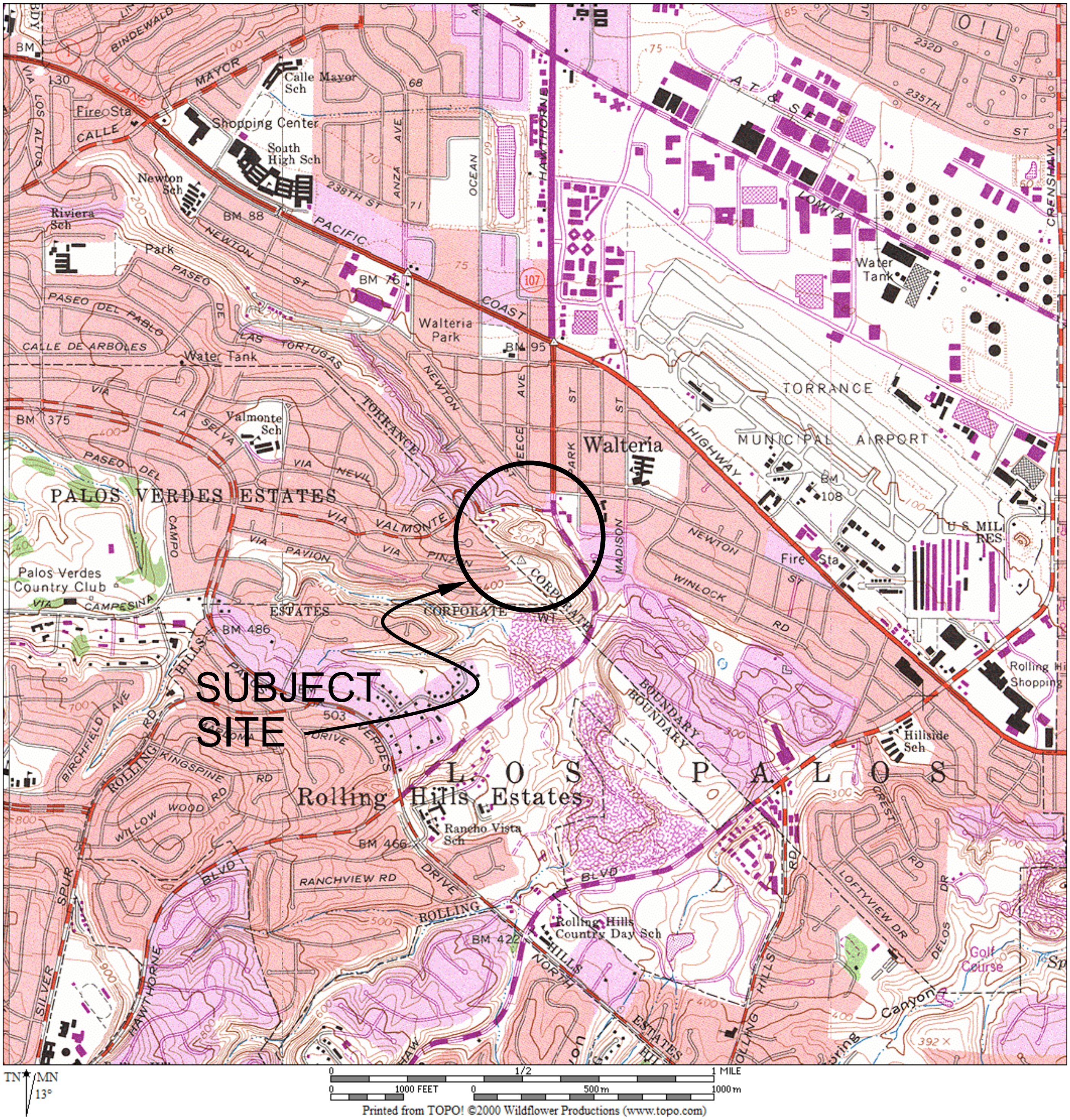

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REFERENCE: U.S.G.S. TOPOGRAPHIC MAPS, 7.5 MINUTE SERIES, TORRANCE, CA QUADRANGLE (1964, Photorevised 1981)

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VICINITY MAP

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TORRANCE, CALIFORNIA 90505

MARCH 2016	PROJECT NO. A9201-06-01C	FIG. 1

LEGEND

B15	Approximate Location of Large-Diameter Boring (GEOCON, 2015)
T-3	Approximate Location of Exploratory Trench (GEOCON, 2015)
HA6	Approximate Location of Hand Auger Boring (GEOCON, 2015)
B-101	Approximate Location of Boring (Pacific Soils, 2005)
EP-4	Approximate Location of Trench (Pacific Soils, 2004)
B-1	Approximate Location of Boring (Western Laboratories, 1993)
af	Artifical Fill
af/Qsp	Artificial Fill over Qsp
af/Qm and Qsp	Artificial Fill over Qm and Qsp
Qm	Marine Sand
Qsp	San Pedro Sand
Tm	Monterey Formation
– – –?—?	Geologic Contact - Dashed where approximate, querried where uncertain
– · · –	Property Line
G G'	Location of Geologic Section
18	Strike and Dip of Bedding
63	Strike and Dip of Joint
67	Strike and Dip of Bedding Plane Shear
	Erosion Gully
	Surficial Failure

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GEOLOGIC MAP

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MARCH 2016 | PROJECT NO. A9201-06-01C | FIG. 2A

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By: RMA | Checked By: SFK

SITE PLAN

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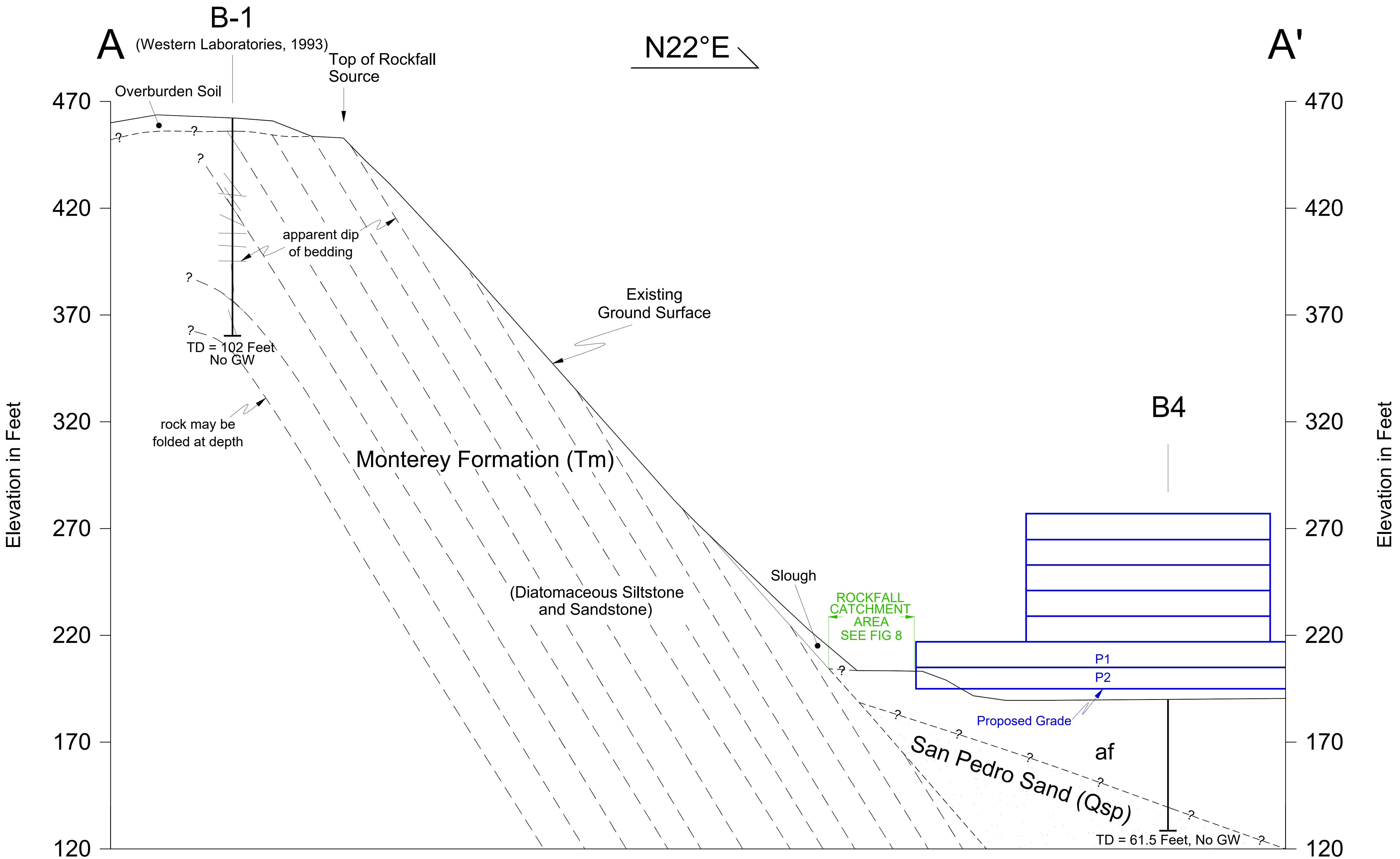
MARCH 2016 | PROJECT NO. A9201-06-01C | FIG. 2B



NOTES:

1. THE SECTIONS ARE BASED ON GEOLOGIC CONDITIONS AT BORING LOCATIONS AND AT SURFACE EXPOSURES MAPPED DURING THE INVESTIGATION. THE GEOLOGIC CONDITIONS BETWEEN SUCH LOCATIONS HAVE BEEN INTERPOLATED. LOCALIZED VARIATIONS COULD OCCUR. THE SECTIONS ARE INTENDED FOR DESCRIPTIVE PURPOSES ONLY.
2. SEE FIGURE 2 FOR LOCATION OF SECTIONS.
3. BEDDING DIPS STEEPER THAN SLOPE.

Scale (H&V): 1" = 50'

0 50' 100'

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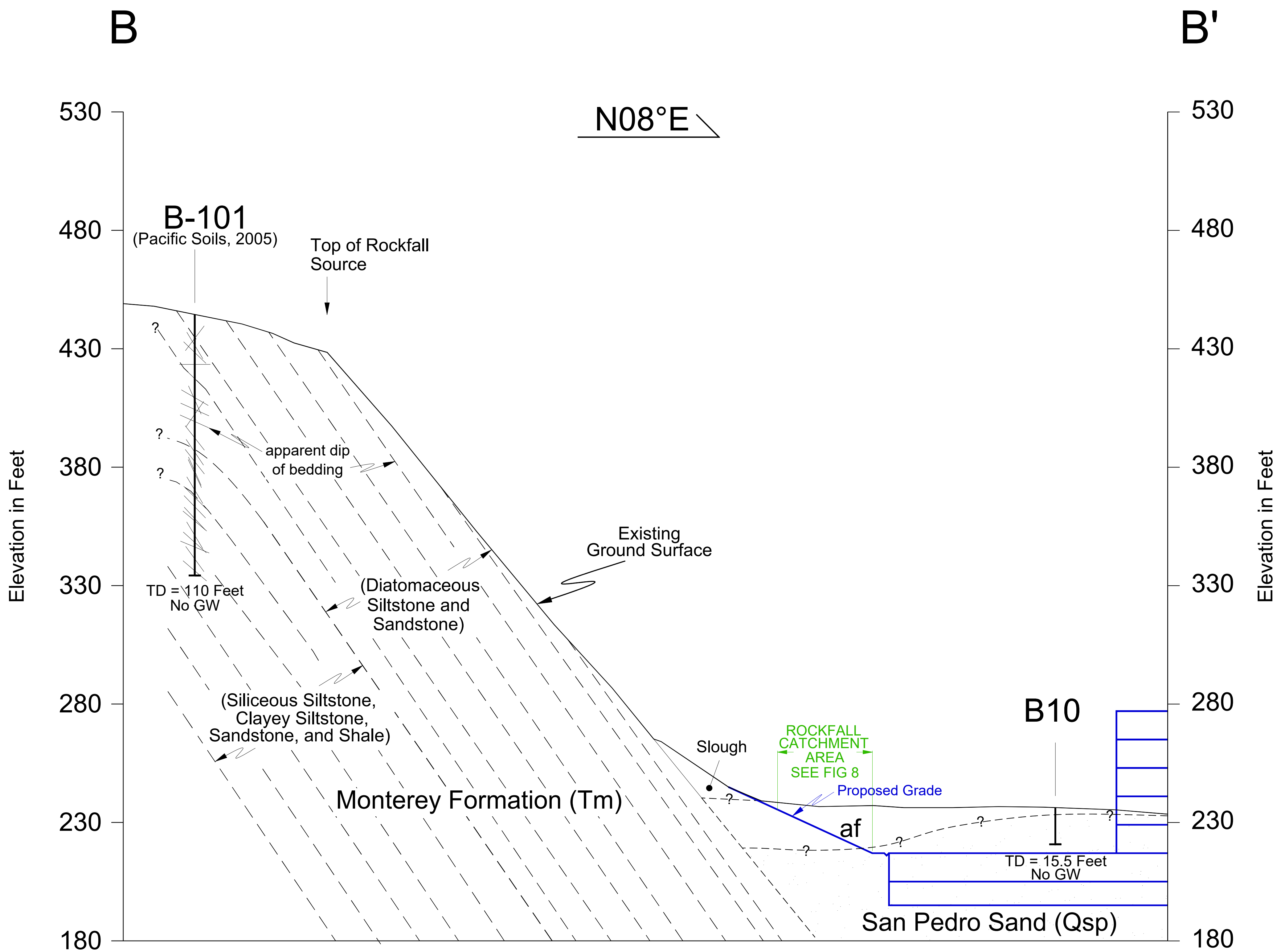
MARCH 2016
PROJECT NO. A9201-06-01C
FIG. 3A



NOTES:

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2. SEE FIGURE 2 FOR LOCATION OF SECTIONS.
3. BEDDING DIPS STEEPER THAN SLOPE.

Scale (H&V): 1" = 50'

0 50' 100'

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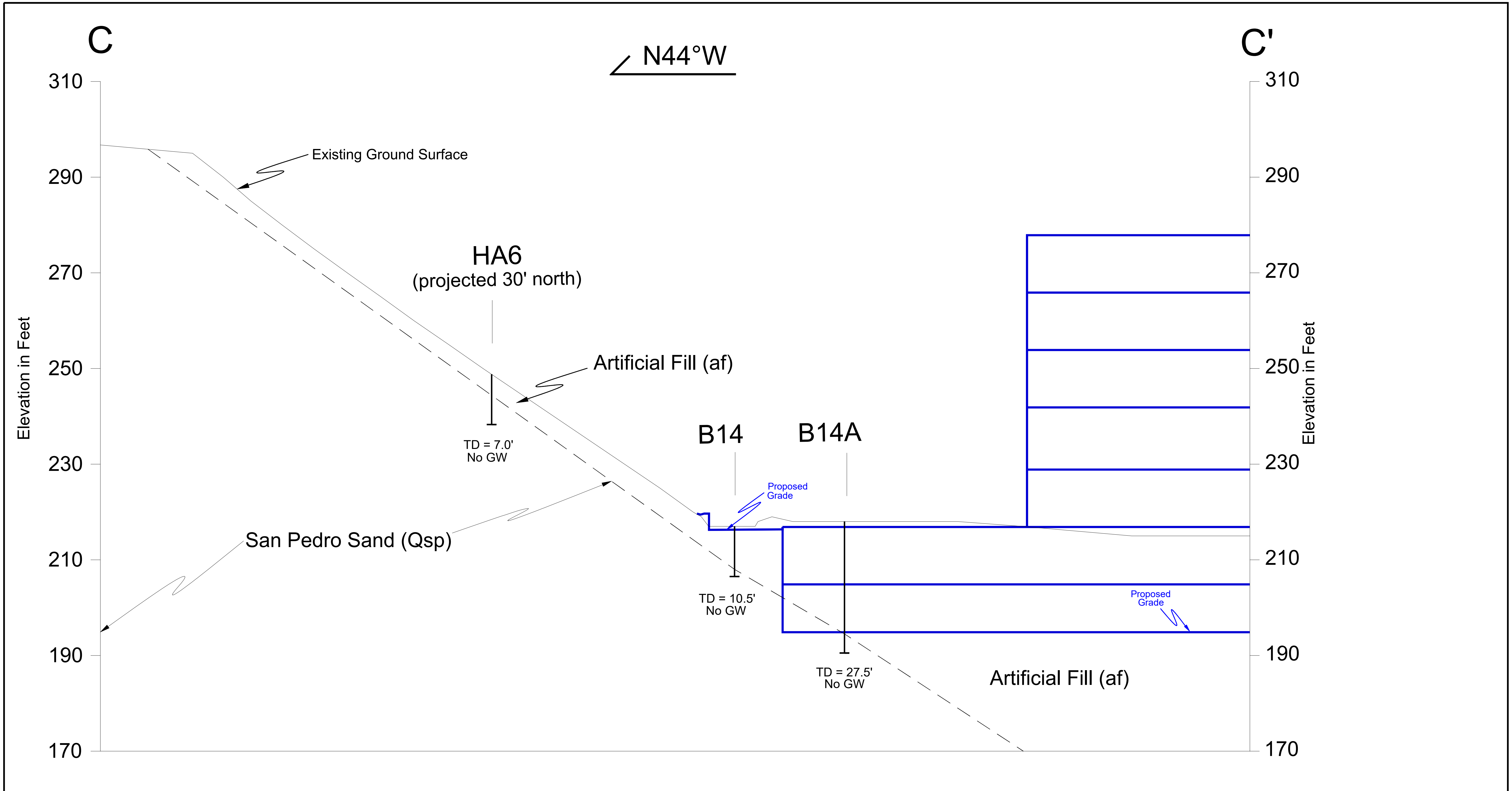
FIG. 3B



Scale (H&V): 1" = 20'

0 20' 40'

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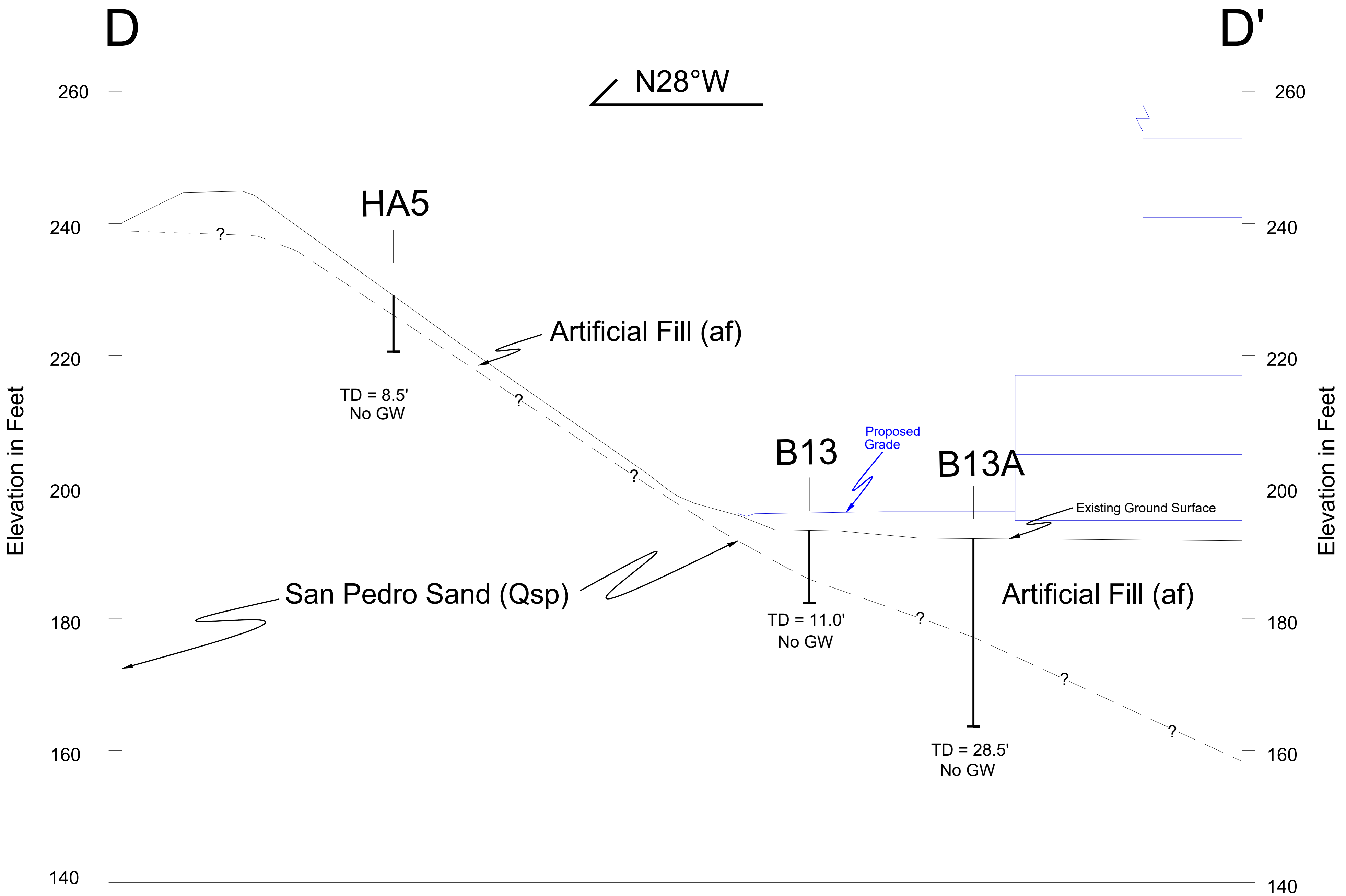
FIG. 3C



Scale (H&V): 1" = 20'

0 20' 40'

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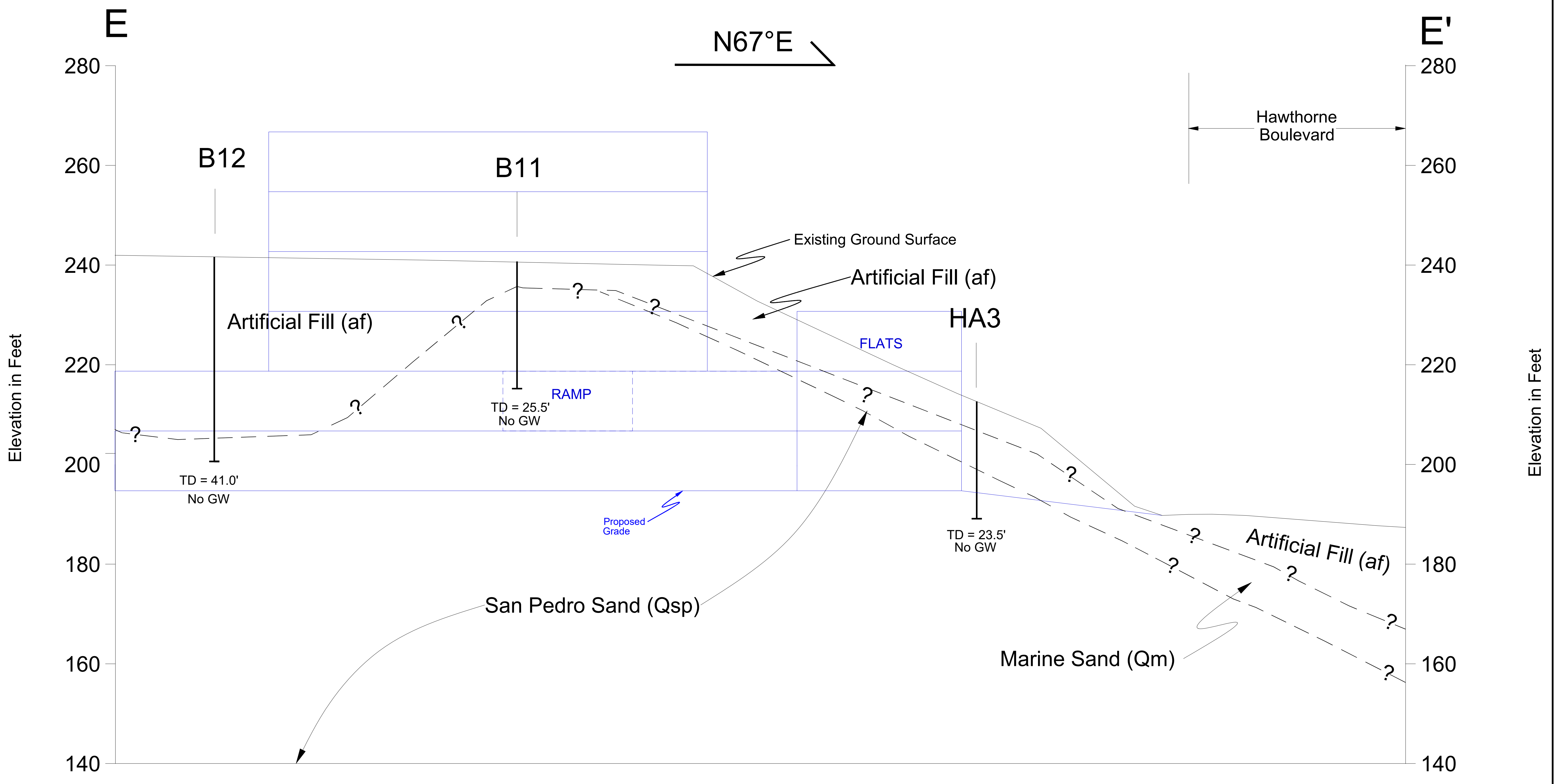
FIG. 3D



Scale (H&V): 1" = 20'

0 20' 40'

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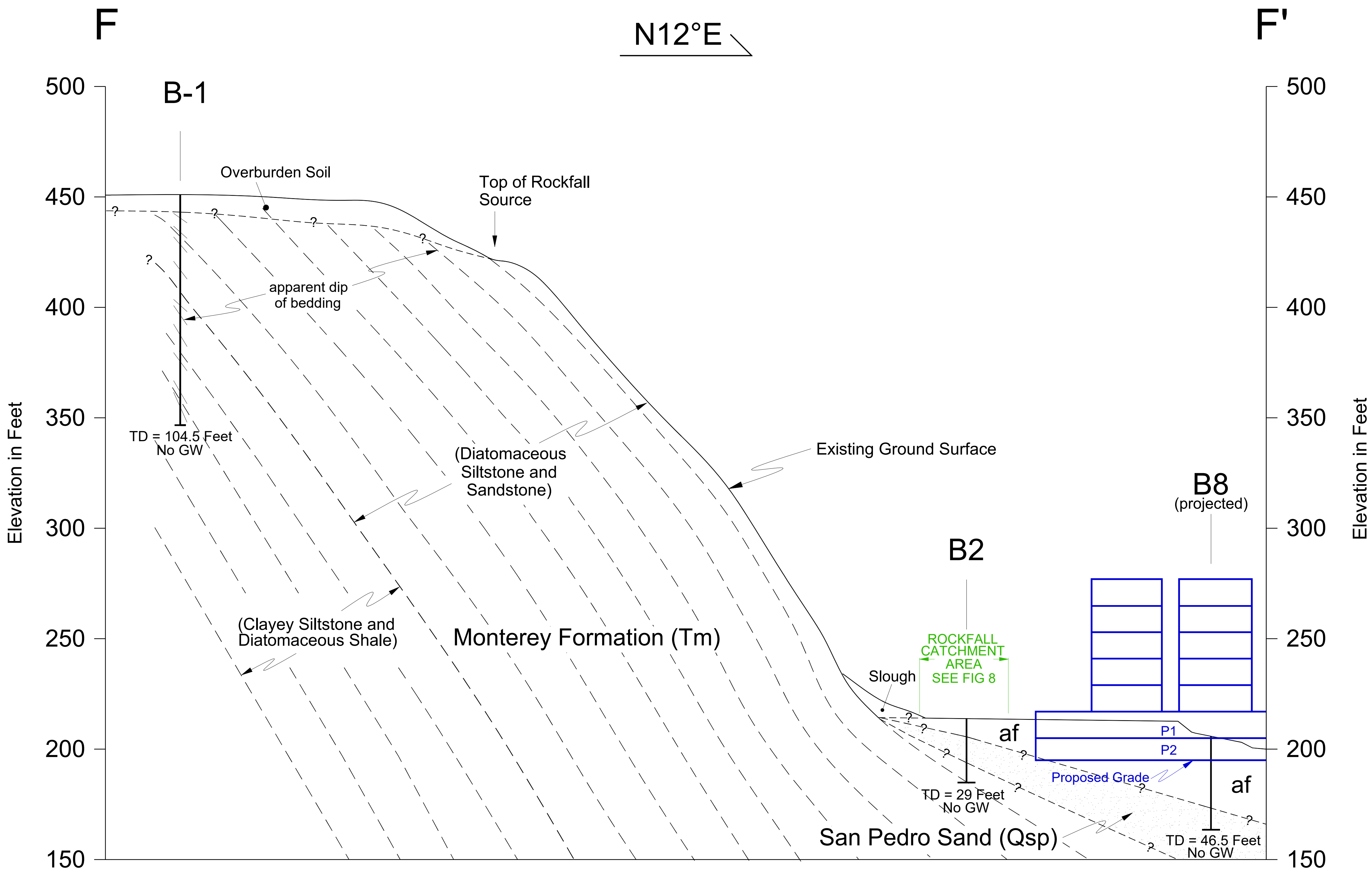
FIG. 3E



NOTES:

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2. SEE FIGURE 2 FOR LOCATION OF SECTIONS.
3. BEDDING DIPS AT SAME INCLINATION AS SLOPE

Scale (H&V): 1" = 50'

0 50' 100'

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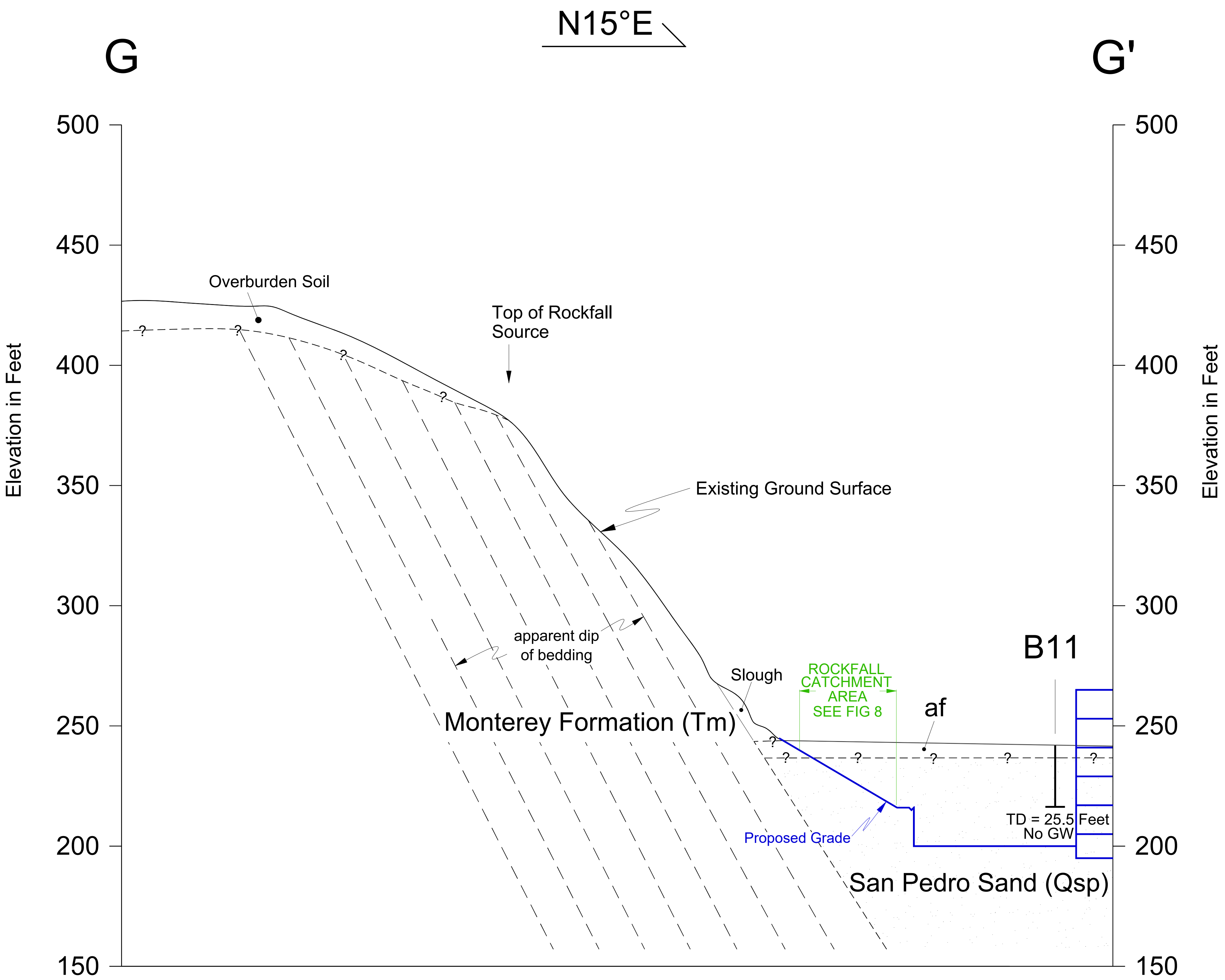
FIG. 3F



NOTES:

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2. SEE FIGURE 2 FOR LOCATION OF SECTIONS.
3. BEDDING DIPS STEEPER THAN SLOPE.

Scale (H&V): 1" = 50'

0 50' 100'

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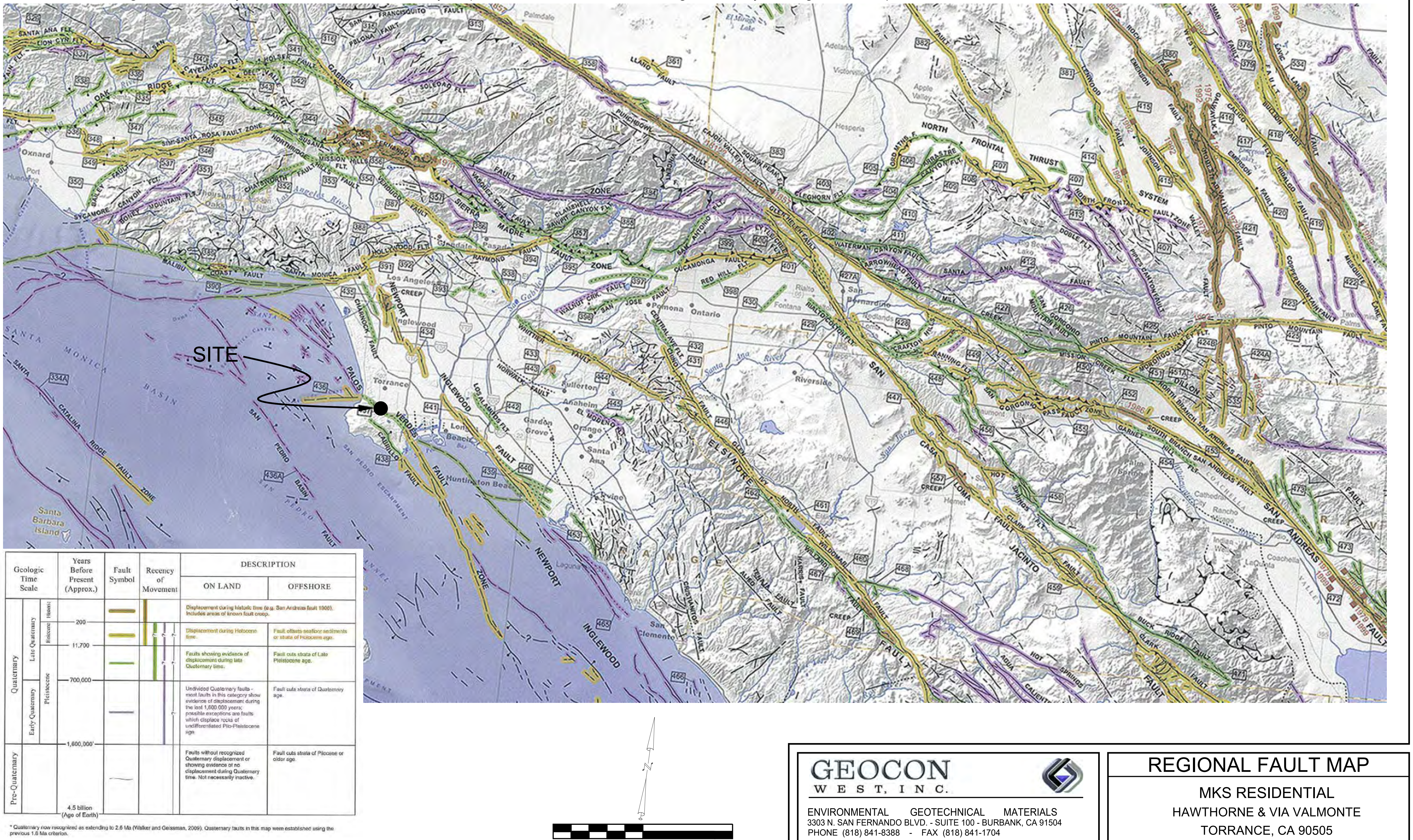
FIG. 3G

Reference: Jennings, C.W. and Bryant, W. A., 2010, Fault Activity Map of California, California Geological Survey Geologic Data Map No. 6.



Geologic Time Scale			Years Before Present (Approx.)	Fault Symbol	Recency of Movement	DESCRIPTION	
						ON LAND	OFFSHORE
Quaternary	Late Quaternary	Historic		▬		Displacement during historic time (e.g. San Andreas fault 1906). Includes areas of known fault creep.	
		Holocene	200	▬	? ? ?	Displacement during Holocene time.	Fault offsets seafloor sediments or strata of Holocene age.
		Pleistocene	11,700	▬	? ?	Faults showing evidence of displacement during late Quaternary time.	Fault cuts strata of Late Pleistocene age.
	Early Quaternary		700,000	▬	?	Undivided Quaternary faults - most faults in this category show evidence of displacement during the last 1,600,000 years; possible exceptions are faults which displace rocks of undifferentiated Plio-Pleistocene age.	Fault cuts strata of Quaternary age.
Pre-Quaternary			1,600,000* 4.5 billion (Age of Earth)	—		Faults without recognized Quaternary displacement or showing evidence of no displacement during Quaternary time. Not necessarily inactive.	Fault cuts strata of Pliocene or older age.

* Quaternary now recognized as extending to 2.6 Ma (Walker and Geissman, 2009). Quaternary faults in this map were established using the previous 1.6 Ma criterion.

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REGIONAL FAULT MAP

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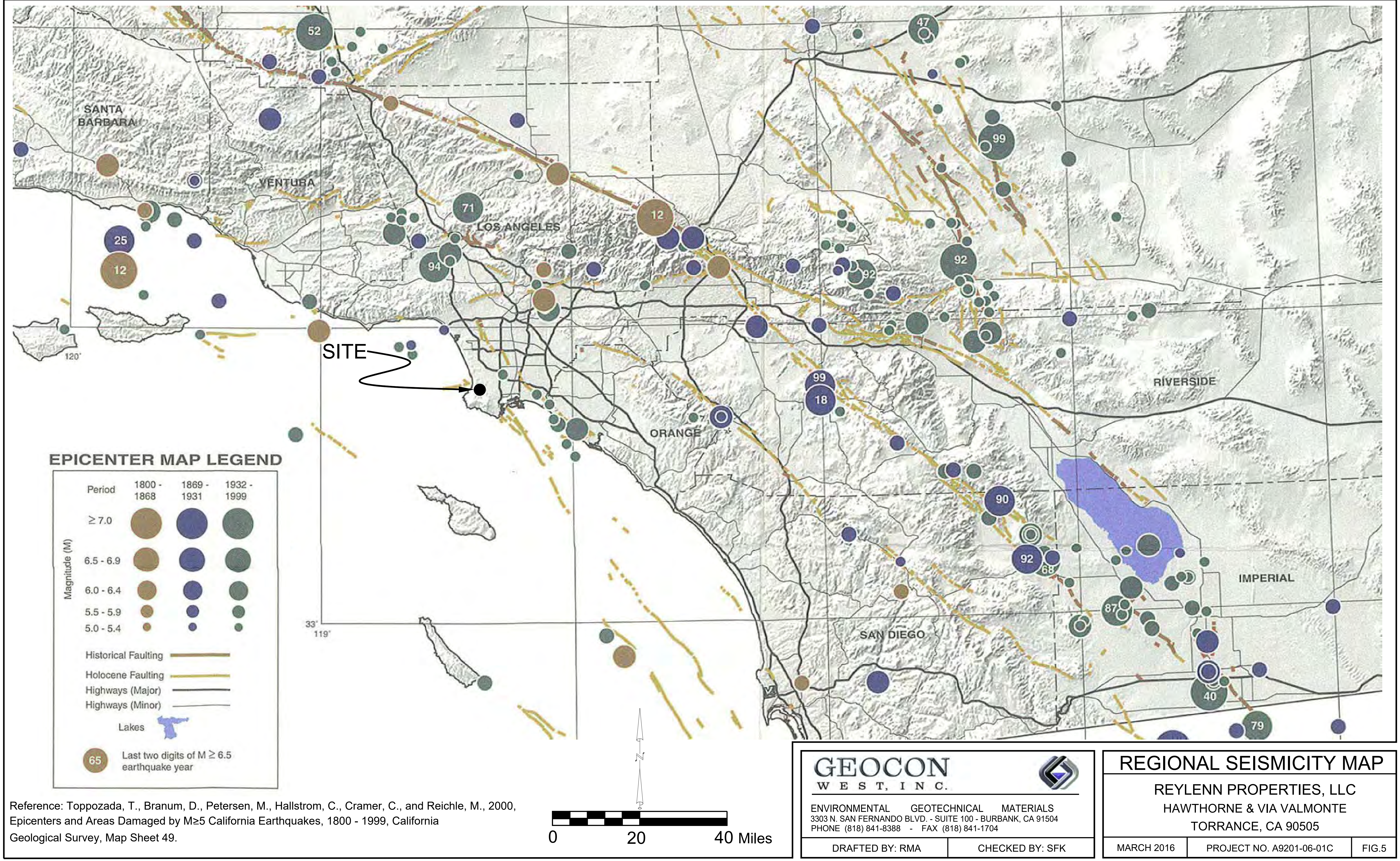
FIG. 4



Reference: Toppozada, T., Branum, D., Petersen, M., Hallstrom, C., Cramer, C., and Reichle, M., 2000, Epicenters and Areas Damaged by M≥5 California Earthquakes, 1800 - 1999, California Geological Survey, Map Sheet 49.

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REGIONAL SEISMICITY MAP

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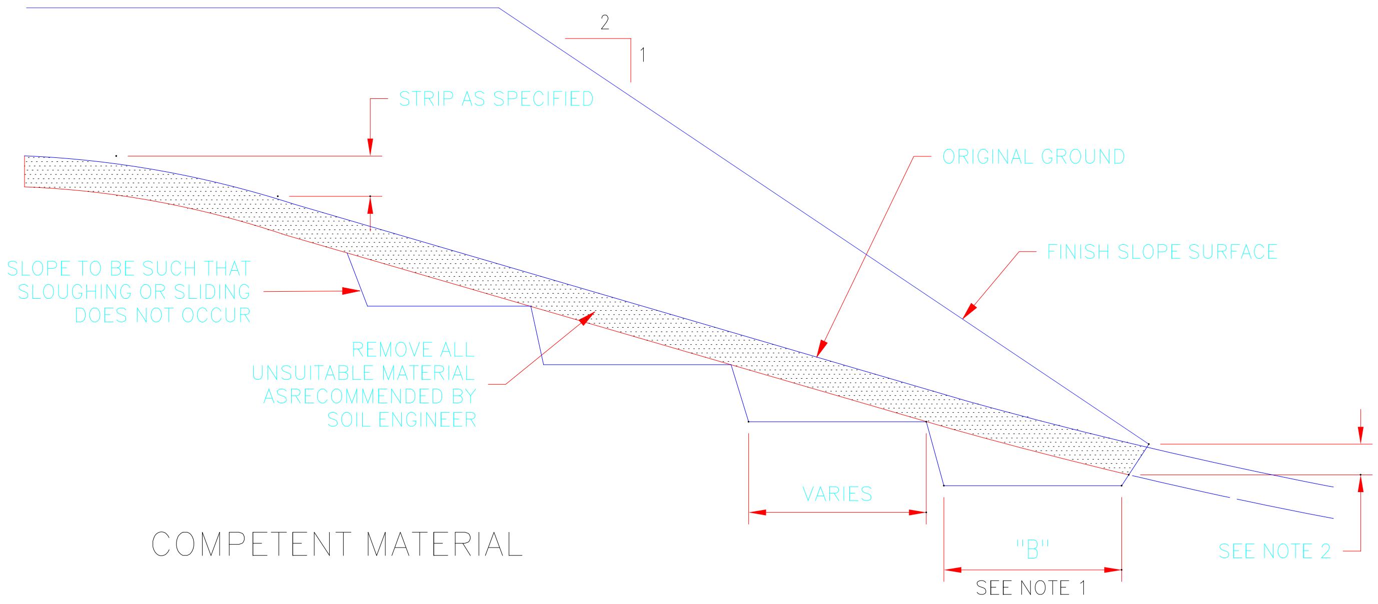

DETAIL NOTES:

(1) Key width "B" should be a minimum of 10 feet wide, or sufficiently wide to permit complete coverage with the compaction equipment used. The base of the key should be graded horizontal, or inclined slightly into the natural slope.

(2) The outside of the bottom key should be below the topsoil or unsuitable surficial material and at least 2 feet into competent material.

Where hard rock is exposed in the bottom of the key, the depth and configuration of the key may be modified as approved by the Geotechnical Engineer.

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BENCHED EXCAVATION DETAIL

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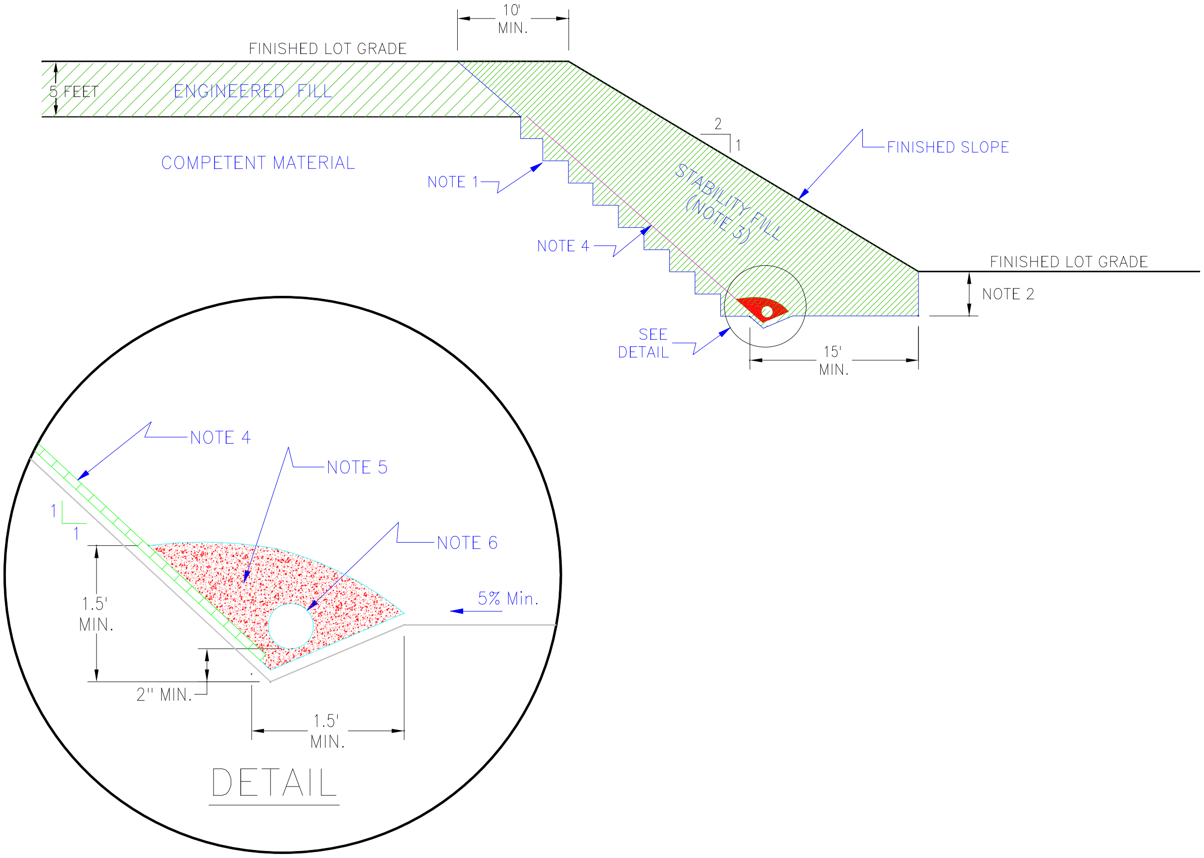
MARCH 2016 | PROJECT NO. A9201-06-01C | FIG. 6



NOTES

1 EXCAVATE BENCHED BACKCUT AT 1:1 INCLINATION

2 BASE OF BUTTRESS TO BE 3 FEET BELOW PAD GRADE, OR A MAXIMUM OF 15 FEET BELOW PAD GRADE SLOPING A MINIMUM 5% INTO SLOPE

3 BUTTRESS FILL TO BE COMPOSED OF PROPERLY COMPACTED ENGINEERED FILL

4 WHERE SEEPAGE IS ENCOUNTERED IN BACKCUT OR SLOPE HEIGHT EXCEEDS 15 FEET, CHIMNEY DRAINS ARE RECOMMENDED, CHIMNEY DRAINS TO BE APPROVED, PREFABRICATED DRAINS ARE CHIMNEY DRAIN PANELS (MIRIDRAIN 5000 OR EQUIVALENT) SPACED APPROXIMATELY 20 FEET CENTER TO CENTER AND 4 FEET WIDE

5 FILTER MATERIAL TO BE 1-INCH, OPEN-GRADED CRUSHED ROCK ENCLOSED IN APPROVED FILTER FABRIC

6 COLLECTOR PIPE TO BE 4-INCH MINIMUM DIAMETER, PERFORATED, THICK-WALLED PVC SCHEDULE 40 OR EQUIVALENT, AND SLOPED TO DRAIN AT 1 PERCENT MINIMUM TO APPROVED OUTLET

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STABILITY / BUTTRESS FILL DETAIL

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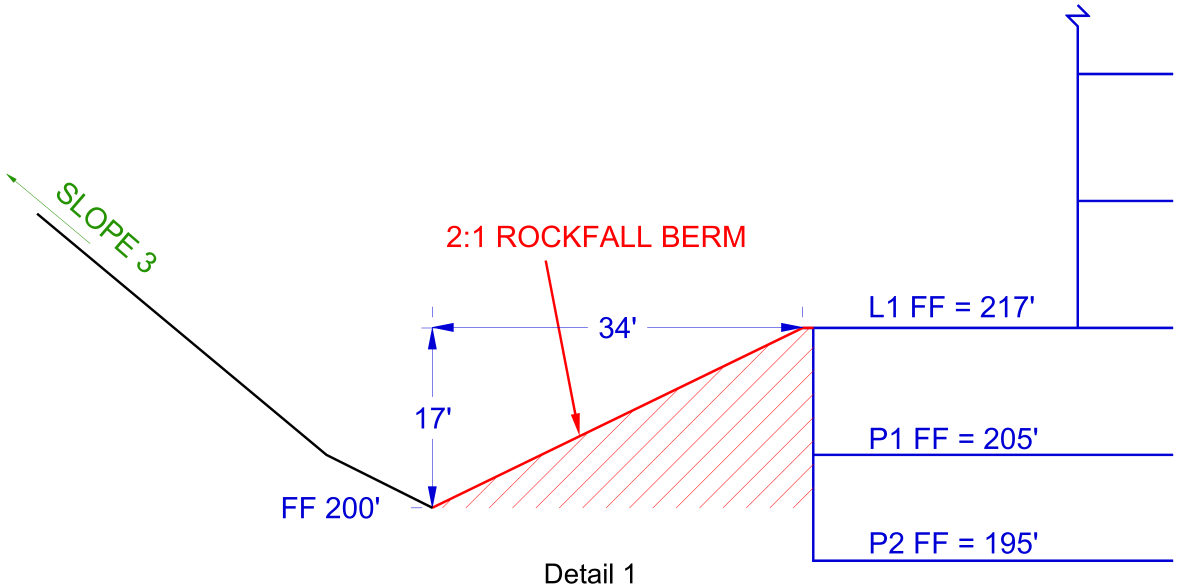
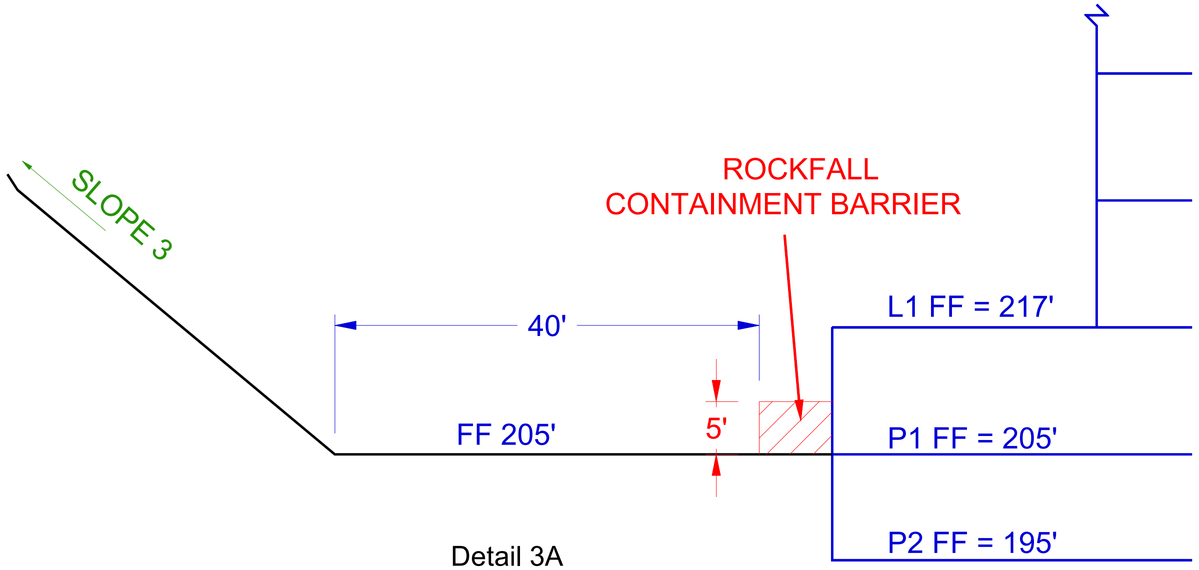
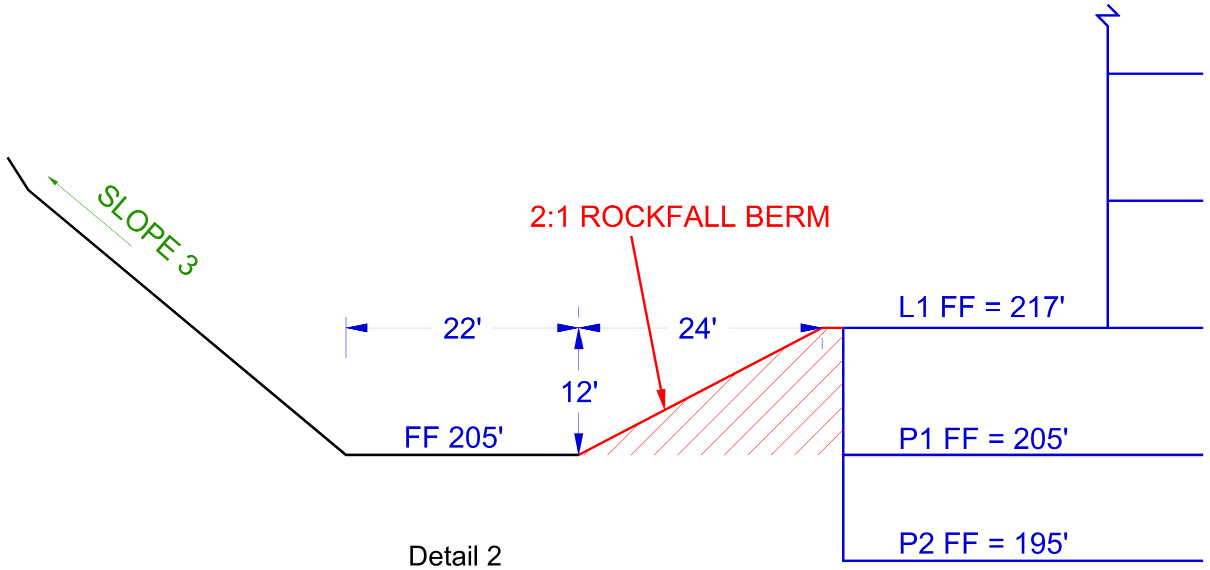
FIG. 7



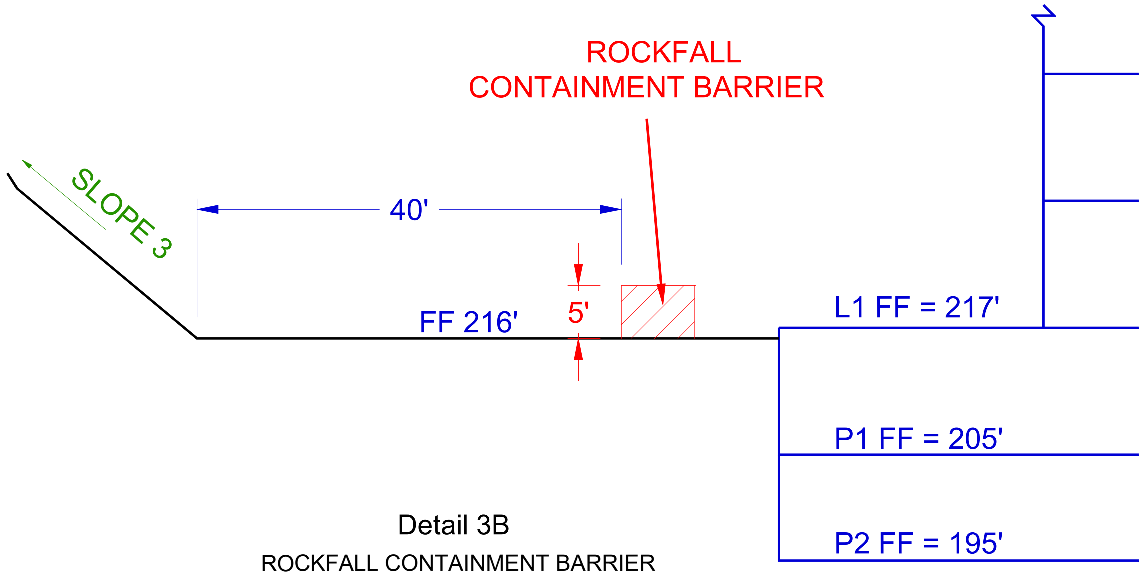
Detail 1
ROCKFALL CATCHMENT AREA
NO SCALE



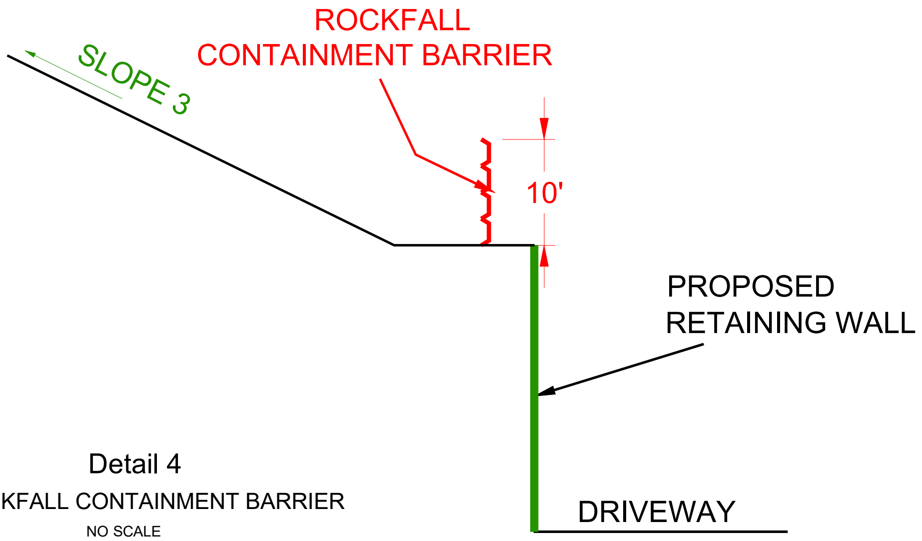
Detail 3A
ROCKFALL CONTAINMENT BARRIER
NO SCALE



Detail 2
ROCKFALL CATCHMENT AREA
NO SCALE



Detail 3B
ROCKFALL CONTAINMENT BARRIER
NO SCALE



Detail 4
ROCKFALL CONTAINMENT BARRIER
NO SCALE

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ROCKFALL CATCHMENT

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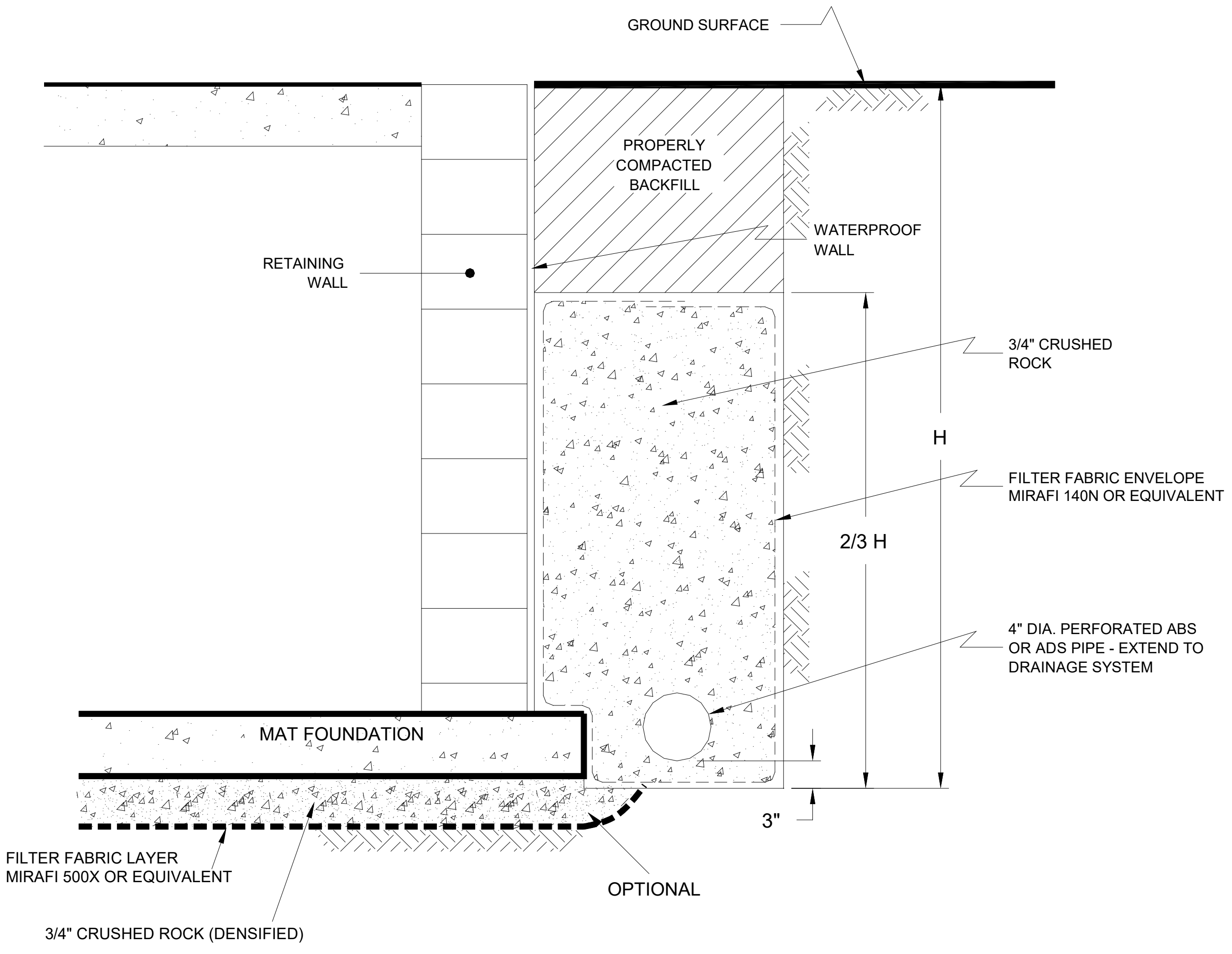
MARCH 2016	PROJECT NO. A9201-06-01C	FIG. 8



NO SCALE

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RETAINING WALL DRAIN DETAIL

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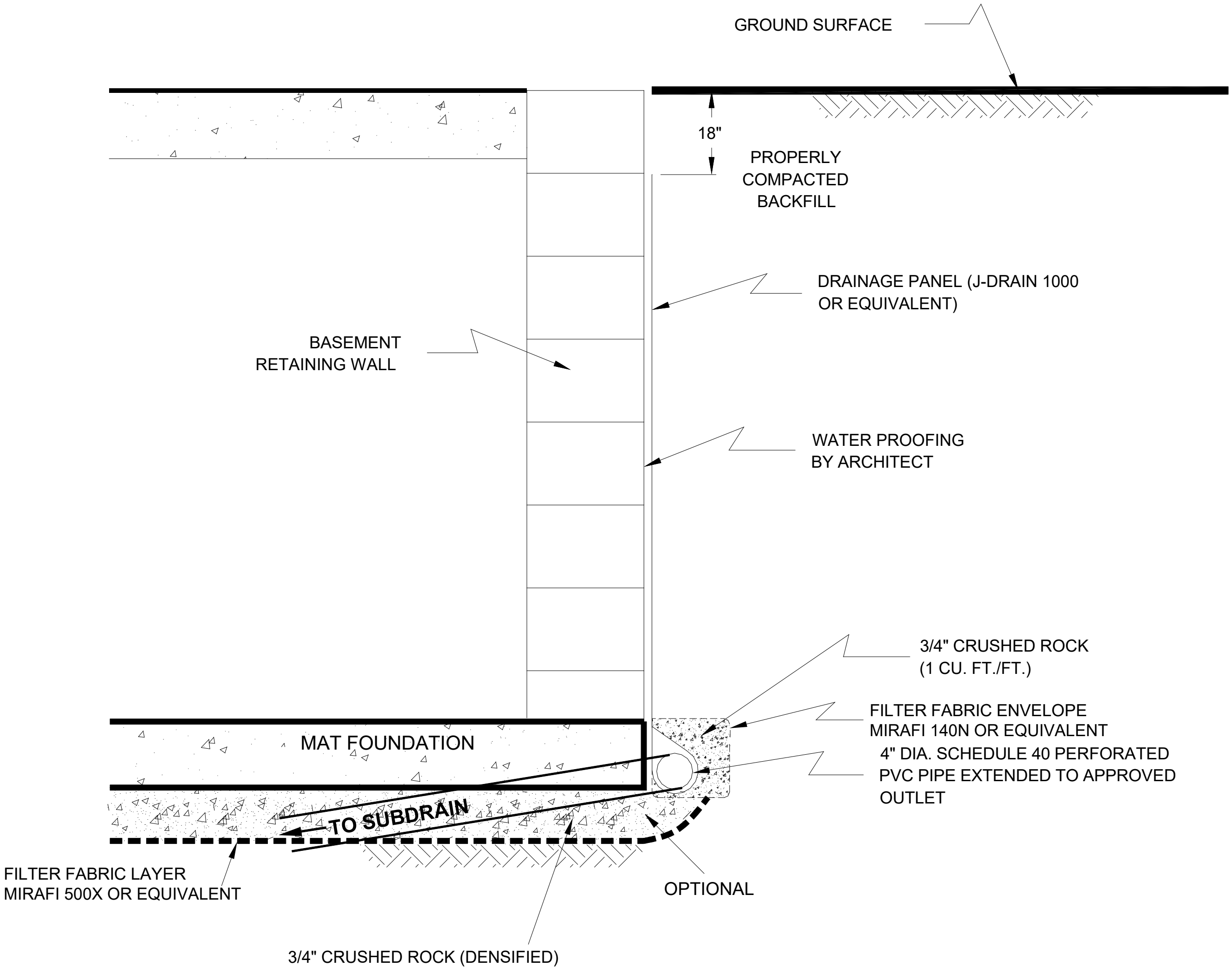
MARCH 2016 | PROJECT NO. A9201-06-01C | FIG. 9



NOTE: TOP OF DRAINAGE PANEL NOT MORE THAN
18 INCHES FROM GROUND SURFACE

NO SCALE

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RETAINING WALL DRAIN DETAIL

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APPENDIX A

APPENDIX A

FIELD INVESTIGATION

Our field exploration was performed between July 14, 2015 and July 24, 2015 and included drilling fifteen large-diameter bucker auger borings utilizing a truck mounted LM-60 Lo-Drill and a truck-mounted E-Z Bore bucket auger drilling machine. The borings were advanced to approximate depths of between 11 feet and 110 feet below the existing ground surface. Four of the large-diameter borings were downhole logged by a registered Certified Engineering Geologist (CEG). In addition, six 4-inch diameter borings were excavated utilizing manual augers and digging equipment to depths between 7 and 23½ feet beneath the existing ground surface. Representative and relatively undisturbed samples were obtained by driving a 3 inch, O. D., California Modified Sampler into the "undisturbed" soil mass. The California Modified Sampler was equipped with 1-inch high by 2 ³⁄₈-inch diameter brass sampler rings to facilitate soil removal and testing. Bulk samples were also obtained.

The soil conditions encountered in the borings were visually examined, classified and logged in general accordance with the Unified Soil Classification System (USCS). Logs of the borings are presented on Figures A1 through A23. The logs depict the soil and geologic conditions encountered and the depth at which samples were obtained.

PROJECT NO. A9201-06-01C

BORING 1

ELEV. (MSL.) **452'** DATE COMPLETED **7/14/15**

EQUIPMENT **BUCKET AUGER** BY: **RA**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					**OVERBURDEN SOIL (Qs)** Sandy Silt, very soft, dry, light gray, fine- to medium-grained, some coarse-grained, abundant rootlets, abundant calcium carbonate, gravel to 2".			
2								
	B1@2.5'					2		
4								
					- contact varies from 4.0-6.0', dips to north			
6	B1@5.5'				**MONTEREY FORMATION (TM)** Siltstone, very soft (H1), dry, yellowish brown, some fine-grained sandstone beds, thinly bedded to laminated, intensely fractured, intensely weathered, abundant gopher holes in upper 8 feet.	4		
8	B1@7.5'				- 7.5' soft (H2), pale yellowish brown, diatomaceous-rich	2		
10					- 10.0' Fracture: N12E, 85SE Bedding: N52E, 30NW			
	B1@10.5'				- 10.5' becoming more diatomaceous-rich, highly weathered	4		
12								
					- 13.0' some siliceous siltstone interbeds, very soft (H1) to soft (H2), thinly bedded, highly fractured Bedding: N40W, 42NE			
14								
					- 15.0' diatomaceous-rich, soft (H2), white, moderately to intensely fractured, moderately weathered			
16	B1@15.5'					6		
18					- 18.0' pale yellowish brown, intensely weathered, thinly bedded			
					- 19.0' some clayey siltstone interbeds Bedding: N39W, 47NE Fracture: N6E, 85SE			
20								
	B1@21'				- 21.0' soft (H2), highly weathered, diatomaceous content decreasing, laminated	6		
22								
24								
					Siltstone, siliceous, medium hard (H3), white, thinly bedded, moderately cemented, moderately to intensely fractured, some fractures open to 1/4". Bedding: N60E, 54NW.			
26	B1@26'					1	51.2	36.5
28					- 28.0' pale yellowish brown			

Figure A1,
Log of Boring 1, Page 1 of 4

SAMPLE SYMBOLS
☐ ... SAMPLING UNSUCCESSFUL
◧ ... STANDARD PENETRATION TEST
■ ... DRIVE SAMPLE (UNDISTURBED)
☒ ... DISTURBED OR BAG SAMPLE
◪ ... CHUNK SAMPLE
▼ ... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

PROJECT NO. A9201-06-01C

BORING 1

ELEV. (MSL.) 452' DATE COMPLETED 7/14/15

EQUIPMENT BUCKET AUGER BY: RA

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
30					- 30.0' soft (H2), increase in diatomaceous siltstone interbeds Bedding: N69W, 44NE Fracture: N19E, 84SE			
	B1@31'	■				5		
32					Clayey Siltstone, soft (H2), thinly bedded, moderately weathered.			
34								
					Siltstone, diatomaceous, white, laminated, slightly weathered.			
36	B1@36'	■				4		
					- 37.0' Bedding: N72W, 55NE Fracture: N9W, 88NE			
38								
					Clayey Siltstone, light brown, thinly bedded, moderately weathered, trace diatomaceous siltstone interbeds.			
40								
	B1@41'	■				4		
42								
					- 43.0' predominantly clayey siltstone, clay coatings on bedding, slightly polished, no striations Bedding: N58W, 42NE			
44					- 44.0' Bedding: N84E, 68NW			
					- 45.0' very pale brown, laminated, slightly weathered, moderately to intensely fractured			
46	B1@46'	■				6		
48								
50								
	B1@51'	■			- 51.0' becomes diatomaceous-rich	18	41.8	47.0
52					- 52.0' Bedding: N52E, 67NW - 52.5' very soft (H1)			
54								
56	B1@56'	■				15		
					- 57.0' Bedding: N82E, 65NW			
58					- 58.0' highly weathered, clay-coated surfaces common, pale brown			

Figure A1,
Log of Boring 1, Page 2 of 4

SAMPLE SYMBOLS			
	☐ ... SAMPLING UNSUCCESSFUL	◧ ... STANDARD PENETRATION TEST	■ ... DRIVE SAMPLE (UNDISTURBED)
	⊠ ... DISTURBED OR BAG SAMPLE	◪ ... CHUNK SAMPLE	▼ ... WATER TABLE OR SEEPAGE

BORING 1

ELEV. (MSL.) **452'** DATE COMPLETED **7/14/15**

EQUIPMENT **BUCKET AUGER** BY: **RA**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
60								
	B1@61' ■				- 60.5' laminated to thinly bedded, pale yellowish brown	16		
62								
					- 63.0' clay-coated bedding and fracture surfaces, highly weathered, highly fractured			
64								
					- 65.0' some fine-grained sandstone interbeds, yellowish brown Bedding: N88W, 62NE			
66	B1@66' ■					15		
68								
					- 69.0' Bedding: N88E, 57SE			
70								
	B1@71' ■					18		
72								
					- 73.5' very fine- to fine-grained sandstone interbeds, yellowish brown, thinly bedded, moderately weathered			
74								
					- 75.0' Bedding: N85W, 71NE			
76	B1@76' ■				- 76.0' laminated, slightly weathered	35		
78					- 78.0' some diatomaceous-rich siltstone beds, clay films along bedding			
80								
	B1@81' ■					25	54.7	50.2
82								
					- 83.0' some very fine-grained sandstone interbeds, trace oxidation staining Bedding: N52W, 51NE - 84.0' moderately weathered			
84								
86	B1@86' ■					20		
					- 87.0' moderately fractured, yellowish brown			
88								

Figure A1,
Log of Boring 1, Page 3 of 4

SAMPLE SYMBOLS
☐ ... SAMPLING UNSUCCESSFUL
◧ ... STANDARD PENETRATION TEST
■ ... DRIVE SAMPLE (UNDISTURBED)
▨ ... DISTURBED OR BAG SAMPLE
◪ ... CHUNK SAMPLE
▼ ... WATER TABLE OR SEEPAGE

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PROJECT NO. A9201-06-01C

BORING 1

ELEV. (MSL.) **452'** DATE COMPLETED **7/14/15**

EQUIPMENT **BUCKET AUGER** BY: **RA**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
90					- 90.0' diatomaceous-rich siltstone interbeds			
	B1@91' ■					20		
92					- 92.0' Bedding: N88W, 63NE			
94					Interbedded Clayey Siltstone and Diatomaceous Siltstone, soft (H2), brown to light brown, thinly bedded, moderately weathered.			
96	B1@96' ■					21		
98								
					- 99.0' Bedding: N44W, 74NE			
100					- 100.5' olive brown, some clay films along bedding			
	B1@101' ■					13		
102					- 102.0' Bedding: N9W, 36NE - 102.5' yellowish brown, slightly weathered, slightly fractured, trace very fine-grained sand			
104					- 104.0' brown, highly weathered			
106	B1@106' ☐				- 106.0' no recovery			
108					- 108.5' moderately fractured, slightly weathered			
110								
	B1@111' ■				Siltstone, diatomaceous, massive, soft (H2), some clayey siltstone interbeds, trace oxidation staining.	15	45.2	54.1

Total depth of boring: 111.5 feet
Overburden to 5 feet.
No groundwater encountered.
Backfilled with soil cuttings and tamped.

*Penetration resistance for Kelly Bar falling 12 inches.
Kelly Bar weights: 0-25 feet, 4,900 lbs, 25-50 feet, 3,400 lbs, 50-75 feet, 2,200 lbs, 70-100 feet, 1,200 lbs

Figure A1,
Log of Boring 1, Page 4 of 4

SAMPLE SYMBOLS			
	☐ ... SAMPLING UNSUCCESSFUL	◧ ... STANDARD PENETRATION TEST	■ ... DRIVE SAMPLE (UNDISTURBED)
	☒ ... DISTURBED OR BAG SAMPLE	◪ ... CHUNK SAMPLE	▼ ... WATER TABLE OR SEEPAGE

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GEOCON

PROJECT NO. A9201-06-01C

BORING 2

ELEV. (MSL.) **210'** DATE COMPLETED **7/16/15**

EQUIPMENT **BUCKET AUGER** BY: **RA**

DEPTH IN FEET	SAMPLE NO.	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0	BULK 0-2.5'		**ARTIFICIAL FILL (af)** Sandy Silt, soft, dry, gray, some diatomaceous siltstone fragments.			
2			Silty Sand, well-graded, medium dense, slightly moist, brown, some brick and concrete fragments.			
4	B2@3.5'		- 3.5' poorly graded, yellowish brown, fine- to medium-grained, some coarse-grained, some brick fragments (to 3"), rock fragments (to 6")	5		
6	BULK 5-7.5 B2@6'		- 6.0' some slate fragments (to 1"), some cobbles (to 4")	4	103.8	13.1
8	B2@8.5'		**SAN PEDRO SAND (Qsp)** Sand, poorly graded, medium dense, slightly moist, pale light brown, fine- to medium-grained, some silt, friable, massive, trace rounded cobbles, subrounded (to 3"), contract with fill flat.	P	96.3	8.0
10			- 10.0' caving			
	B2@11'		- 11.0' light brown, trace coarse-grained sand	P	105.1	3.2
12						
14	B2@13.5'	SP		2	106.4	2.8
			- 14.0' yellowish brown - 14.5' caving (approximately 8" into sidewall) - 15.0' increase in coarse-grained sand, some rounded gravel (to 2")			
16	B2@16'			4	104.0	4.5
18						
	B2@18.5'			2	--	--
20			**MONTEREY FORMATION (Tm)** Interbedded Clayey Siltstone and diatomaceous Siltstone, very soft (H1), brown, thinly bedded, intensely fractured. - 20.0' Bedding: N81W, 17NE			
	B2@21'			2		
22						
24	B2@23.5'		Claystone, very soft (H1), brown, thinly bedded to laminated, slightly weathered, intensely fractured.	3	108.2	5.4
26	B2@26'		- 26.0' some polished surfaces randomly oriented, no striations, laminated	5		
28						

A9201-06-01C BORING LOGS.GPJ

Figure A2,
Log of Boring 2, Page 1 of 2

SAMPLE SYMBOLS

☐ ... SAMPLING UNSUCCESSFUL
◧ ... STANDARD PENETRATION TEST
■ ... DRIVE SAMPLE (UNDISTURBED)
☒ ... DISTURBED OR BAG SAMPLE
◪ ... CHUNK SAMPLE
▼ ... WATER TABLE OR SEEPAGE

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PROJECT NO. A9201-06-01C

BORING 2

ELEV. (MSL.) **210'** DATE COMPLETED **7/16/15**

EQUIPMENT **BUCKET AUGER** BY: **RA**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
30		▨						
	B2@31' ■	▨			- 31.0' some diatomaceous siltstone interbeds	4		

Total depth of boring: 31.5 feet
Fill to 8 feet.
No groundwater encountered.
Backfilled with soil cuttings and tamped.

*Penetration resistance for Kelly Bar falling 12 inches.
Kelly Bar weights: 0-25 feet, 4,900 lbs, 25-50 feet, 3,400 lbs, 50-75 feet, 2,200 lbs, 70-100 feet, 1,200 lbs

Figure A2,
Log of Boring 2, Page 2 of 2

SAMPLE SYMBOLS

☐ ... SAMPLING UNSUCCESSFUL	◧ ... STANDARD PENETRATION TEST	■ ... DRIVE SAMPLE (UNDISTURBED)
⊠ ... DISTURBED OR BAG SAMPLE	◪ ... CHUNK SAMPLE	▼ ... WATER TABLE OR SEEPAGE

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BORING 3

ELEV. (MSL.) **201'** DATE COMPLETED **7/16/15**

EQUIPMENT **BUCKET AUGER** BY: **RA**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					**ARTIFICIAL FILL (af)** Silty Sand, well-graded, medium dense, slightly moist, brown, fine- to medium-grained, some subangular cobbles (to 6"), some brick debris.			
2					- 2.5' some clay, some concrete fragments (to 8")			
4	B3@3.5'					4		
6	B3@6'				- 6.0' poorly graded, rock fragments (to over 12"), fine- to medium-grained, some coarse-grained - 7.0' no rock fragments	7	125.1	6.0
8	B3@8.5'					4	117.0	9.5
					- 9.0' trace coarse-grained sand			
10					- 10.0' some cobbles (to 4")			
	B3@11'					3	111.8	6.2
12								
14	B3@13.5'				**SAN PEDRO SAND (Qsp)** Sand, poorly graded, medium dense, slightly moist, fine- to medium-grained, trace coarse-grained, friable.	3		
16	B3@16'				- 16.0' increase in coarse-grained sand	3	108.6	7.5
18	B3@18.5'					3	114.7	5.9
20				SP				
	B3@21'				- 21.0' yellowish brown, fine- to medium-grained, some coarse-grained	2	101.3	4.5
22					- 22.0' some subrounded gravel (to 1.5")			
24	B3@23.5'					1		
26	B3@26'					7		
28					- 28.0' weakly bedded, some oxidation staining along bedding			
					Total depth of boring: 28 feet Fill to 13.5 feet.			

Figure A3,
Log of Boring 3, Page 1 of 2

SAMPLE SYMBOLS			
	☐ ... SAMPLING UNSUCCESSFUL	◧ ... STANDARD PENETRATION TEST	■ ... DRIVE SAMPLE (UNDISTURBED)
	⊠ ... DISTURBED OR BAG SAMPLE	◪ ... CHUNK SAMPLE	▼ ... WATER TABLE OR SEEPAGE

GEOCON

PROJECT NO. A9201-06-01C

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	**BORING 3** ELEV. (MSL.) **201'** DATE COMPLETED **7/16/15** EQUIPMENT **BUCKET AUGER** BY: **RA**	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					MATERIAL DESCRIPTION			
					No groundwater encountered. Backfilled with soil cuttings and tamped. *Penetration resistance for Kelly Bar falling 12 inches. Kelly Bar weights: 0-25 feet, 4,900 lbs, 25-50 feet, 3,400 lbs, 50-75 feet, 2,200 lbs, 70-100 feet, 1,200 lbs			

Figure A3,
Log of Boring 3, Page 2 of 2

SAMPLE SYMBOLS			
	☐ ... SAMPLING UNSUCCESSFUL	◧ ... STANDARD PENETRATION TEST	■ ... DRIVE SAMPLE (UNDISTURBED)
	▨ ... DISTURBED OR BAG SAMPLE	◪ ... CHUNK SAMPLE	▼ ... WATER TABLE OR SEEPAGE

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GEOCON

PROJECT NO. A9201-06-01C

BORING 4

ELEV. (MSL.) **192'** DATE COMPLETED **7/16/15**

EQUIPMENT **BUCKET AUGER** BY: **RA**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0	BULK 0-5'				**ARTIFICIAL FILL (af)** Gravelly Sand, well-graded, medium dense, slightly moist, light brown, rock debris (to 8.5"), asphalt debris, some claystone fragments, friable.			
2								
4								
6	B4@6'					6	--	--
8								
10	BULK 10-15							
	B4@11'					4	88.0	20.4
12					Silty Sand, poorly graded, medium dense, slightly moist, very fine- to fine-grained. - 13.0' some rock fragments (to 7")			
14					- 14.0' some claystone fragments (to 9")			
16	B4@16'				Sand with Silt, poorly graded, medium dense, slightly moist, dark brown, fine- to medium-grained, some coarse-grained, some clay, rock fragments (to 6").	6	84.6	16.0
18					- 17.5' wood fragments (to 4") - 19.0' rock fragments (to 3")			
20					Clayey Sand, poorly graded, medium dense, slightly moist, fine- to medium-grained, some coarse-grained, some gravel (to 4").			
	B4@21'					4	104.4	14.9
22					- 22.0' large rock fragments (to 8") - 23.5' plastic tarp debris (to 3")			
24					Silty Sand, poorly graded, medium dense, slightly moist, yellowish brown, fine- to medium-grained, some clay, some gravel (to 2"), friable.			
26	B4@26'					7	107.3	10.8
	BULK 27-30'				Sand, poorly graded, medium dense, slightly moist, light gray, very fine- to medium-grained, friable.			
28								

Figure A4,
Log of Boring 4, Page 1 of 3

A9201-06-01C BORING LOGS.GPJ

SAMPLE SYMBOLS			
	☐ ... SAMPLING UNSUCCESSFUL	◧ ... STANDARD PENETRATION TEST	■ ... DRIVE SAMPLE (UNDISTURBED)
	☒ ... DISTURBED OR BAG SAMPLE	◪ ... CHUNK SAMPLE	▼ ... WATER TABLE OR SEEPAGE

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GEOCON

PROJECT NO. A9201-06-01C

BORING 4

ELEV. (MSL.) **192'** DATE COMPLETED **7/16/15**

EQUIPMENT **BUCKET AUGER** BY: **RA**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
30					- 29.5' some gravel (to 3")			
	B4@31'					3	114.1	3.7
32					- 31.5' some subrounded coarse-grained sand			
					- 33.0' some angular rock fragments (to 4")			
34								
36	B4@36'					6	111.6	3.5
					- 37.0' some cobbles (to 4")			
38								
40	BULK 40-45' B4@41'					7	99.6	4.2
42					- 41.5' rounded gravel (to 1"), increase in coarse-grained sand, decrease in gravel content			
44					- 44.0' trace subrounded cobbles (to 4")			
					- 45.0' some subrounded gravel (to 2")			
46					- 46.0' no recovery			
48								
50								
	B4@51'				**SAN PEDRO SAND (Qsp)** Sand, poorly graded, dense, slightly moist, pale brown, fine- to medium-grained, trace coarse-grained, weakly developed bedding, friable.	8	91.4	5.4
52					- 52.5' subrounded cobbles (to 3.5"), increase in coarse-grained sand			
54								
				SP				
56	B4@56'				- 56.0' medium-grained - 56.5' light gray, some coarse-grained sand, trace subangular cobbles, friable	7	96.1	3.8
58					- 57.5' some coarse-grained sand			

Figure A4,
Log of Boring 4, Page 2 of 3

SAMPLE SYMBOLS

☐ ... SAMPLING UNSUCCESSFUL
◧ ... STANDARD PENETRATION TEST
■ ... DRIVE SAMPLE (UNDISTURBED)
▨ ... DISTURBED OR BAG SAMPLE
◪ ... CHUNK SAMPLE
▼ ... WATER TABLE OR SEEPAGE

PROJECT NO. A9201-06-01C

DEPTH IN FEET	SAMPLE NO.		LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	**BORING 4** ELEV. (MSL.) **192'** DATE COMPLETED **7/16/15** EQUIPMENT **BUCKET AUGER** BY: **RA**	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
						MATERIAL DESCRIPTION			
60					SP				
	B4@61'	■			SW	Sand, well-graded, dense, slightly moist, light gray, subrounded to rounded coarse gravel, massive, friable.	11	101.3	1.4

Total depth of boring: 61.5 feet
Fill to 50.5 feet.
No groundwater encountered.
Backfilled with soil cuttings and tamped.

*Penetration resistance for Kelly Bar falling 12 inches.
Kelly Bar weights: 0-25 feet, 4,900 lbs, 25-50 feet, 3,400 lbs, 50-75 feet, 2,200 lbs, 70-100 feet, 1,200 lbs

Figure A4,
Log of Boring 4, Page 3 of 3

A9201-06-01C BORING LOGS.GPJ

SAMPLE SYMBOLS			
	☐ ... SAMPLING UNSUCCESSFUL	◧ ... STANDARD PENETRATION TEST	■ ... DRIVE SAMPLE (UNDISTURBED)
	⊠ ... DISTURBED OR BAG SAMPLE	◪ ... CHUNK SAMPLE	▼ ... WATER TABLE OR SEEPAGE

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GEOCON

PROJECT NO. A9201-06-01C

BORING 5

ELEV. (MSL.) 190' DATE COMPLETED 7/17/15

EQUIPMENT BUCKET AUGER BY: RA

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					**ARTIFICIAL FILL (af)** Silty Sand, well-graded, medium dense, dry, light brown, large rock fragments (to 6"), some claystone and siltstone fragments (to 12").			
2								
	B5@3'					5	102.0	18.6
4					- 3.5' sand with silt lens, well-graded, dry, yellowish brown, medium-grained, some coarse-grained, some gravel (to 2") - 4.5' large rock fragments (to 18"), some clay fragments mixed with sand			
6	B5@6'				Clayey Sand, well-graded, medium dense, dark gray, some rock fragments (to 2"), organic odor.	3	93.7	28.9
					- 7.0' rock fragments (to 9")			
8	B5@8'					3		
10					- 10.5' increase in gravel content, large rock fragments (to 6")			
	B5@11'					3	93.4	23.3
12					- 12.0' rock fragments (to 12")			
14					- 13.5' plastic PVC pipe - 14.5' large rock fragments (to 15")			
16	B5@16'				Silty Sand, well-graded, loose to medium dense, dark yellowish brown, some clay.	2	107.7	18.6
18					Sand, poorly graded, slightly moist, yellowish brown, fine- to medium-grained, some gravel (to 3"), friable.			
20								
	B5@21'				- 21.0' some subrounded gravel (to 3/4")	2	97.0	5.2
22					- 22.5' some subrounded cobbles (to 7")			
24								
26	B5@26'				**SAN PEDRO SAND (Qsp)** Sand, poorly graded, medium dense, slightly moist, yellowish brown, fine- to medium-grained, trace coarse-grained, massive, rounded gravel (to 2"), friable.	4		
28				SP				

Figure A5,
Log of Boring 5, Page 1 of 2

SAMPLE SYMBOLS

☐ ... SAMPLING UNSUCCESSFUL	◧ ... STANDARD PENETRATION TEST	■ ... DRIVE SAMPLE (UNDISTURBED)
▩ ... DISTURBED OR BAG SAMPLE	◪ ... CHUNK SAMPLE	▼ ... WATER TABLE OR SEEPAGE

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GEOCON

PROJECT NO. A9201-06-01C

BORING 5

ELEV. (MSL.) **190'** DATE COMPLETED **7/17/15**

EQUIPMENT **BUCKET AUGER** BY: **RA**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
30								
	B5@31' ■				- 31.0' some rounded coarse gravel	4	111.3	9.9
32								
				SP				
34								
36	B5@36' ■					7		

Total depth of boring: 36.5 feet
Fill to 26 feet.
No groundwater encountered.
Backfilled with soil cuttings and tamped.

*Penetration resistance for Kelly Bar falling 12 inches.
Kelly Bar weights: 0-25 feet, 4,900 lbs, 25-50 feet, 3,400 lbs, 50-75 feet, 2,200 lbs, 70-100 feet, 1,200 lbs

Figure A5,
Log of Boring 5, Page 2 of 2

A9201-06-01C BORING LOGS.GPJ

SAMPLE SYMBOLS			
	☐ ... SAMPLING UNSUCCESSFUL	◧ ... STANDARD PENETRATION TEST	■ ... DRIVE SAMPLE (UNDISTURBED)
	☒ ... DISTURBED OR BAG SAMPLE	◪ ... CHUNK SAMPLE	▼ ... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

GEOCON

PROJECT NO. A9201-06-01C

BORING 6

ELEV. (MSL.) **189'** DATE COMPLETED **7/16/15**

EQUIPMENT **BUCKET AUGER** BY: **RA**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					**ARTIFICIAL FILL (af)** Silty Sand, well-graded, medium dense, dry, light yellowish brown, large rock fragments (to 8").			
2								
4					Clayey Sand, well-graded, loose to medium dense, slightly moist, dark brown, fine- to medium-grained, some coarse-grained, large rock fragments (to 5"), some brick fragments.			
6	B6@6'					2	111.3	9.9
					Silty Sand, poorly graded, loose to medium dense, slightly moist, yellowish brown, very fine- to medium-grained, trace clay. - 7.5' large pockets of clay and rock debris			
8								
					- 9.0' some coarse-grained sand			
10								
	B6@11'					3	--	--
12					- 12.0' yellowish brown, very fine- to fine-grained, friable			
14								
16	B6@16'				- 16.0' trace coarse-grained sand	3	91.6	9.4
18					- 18.0' large rounded cobbles (to 6")			
					- 19.0' trace rounded gravel (to 1.5"), some claystone fragments			
20	BULK 20-25'				**SAN PEDRO SAND (Qsp)** Sand, poorly graded, medium dense, slightly moist, yellowish brown, very fine- to fine-grained, trace coarse-grained, weakly bedded, friable.			
	B6@21'				- 21.5' increase in coarse-grained sand content, oxidation staining in bedding plane	3	88.9	4.4
22								
24				SP				
					- 25.0' dense, trace rounded coarse-grained sand, micaceous, friable			
26	B6@26'					9		
28								

A9201-06-01C BORING LOGS.GPJ

Figure A6,
Log of Boring 6, Page 1 of 2

SAMPLE SYMBOLS

☐ ... SAMPLING UNSUCCESSFUL
◧ ... STANDARD PENETRATION TEST
■ ... DRIVE SAMPLE (UNDISTURBED)
⊠ ... DISTURBED OR BAG SAMPLE
◪ ... CHUNK SAMPLE
▼ ... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

BORING 6

ELEV. (MSL.) **189'** DATE COMPLETED **7/16/15**

EQUIPMENT **BUCKET AUGER** BY: **RA**

DEPTH IN FEET	SAMPLE NO.		LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
30									
	B6@31'	■			SP	- 31.5' some cobbles (to 2"), large chunk of claystone (1"), thinly bedded, slightly to moderately weathered, moderately fractured	7	98.8	5.0
32									
34									
36	B6@36'	■				- 35.5' trace medium-grained sand, slightly oxidized	10	93.8	3.5

Total depth of boring: 36.5 feet
Fill to 20 feet.
No groundwater encountered.
Backfilled with soil cuttings and tamped.

*Penetration resistance for Kelly Bar falling 12 inches.
Kelly Bar weights: 0-25 feet, 4,900 lbs, 25-50 feet, 3,400 lbs, 50-75 feet, 2,200 lbs, 70-100 feet, 1,200 lbs

Figure A6,
Log of Boring 6, Page 2 of 2

SAMPLE SYMBOLS

☐ ... SAMPLING UNSUCCESSFUL
◧ ... STANDARD PENETRATION TEST
■ ... DRIVE SAMPLE (UNDISTURBED)
⊠ ... DISTURBED OR BAG SAMPLE
◪ ... CHUNK SAMPLE
▼ ... WATER TABLE OR SEEPAGE

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PROJECT NO. A9201-06-01C

BORING 7

ELEV. (MSL.) **191'** DATE COMPLETED **7/20/15**

EQUIPMENT **BUCKET AUGER** BY: **RA**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					**ARTIFICIAL FILL (af)** Sandy Silt, very soft, slightly moist, light brown, fine- to medium-grained, some coarse-grained, some fine gravel and rock debris (to 5"), some diatomaceous silstone fragments (to 7").			
2								
	B7@3'				Silty Sand, well-graded, medium dense, slightly moist, some fine gravel and rock fragments (to 3").	4	108.6	19.0
4					- 4.0' some claystone and siltstone fragments (to 12")			
6	B7@6'				Sand, well-graded, medium dense, slightly moist, some silt, highly oxidized, yellowish brown, friable.	4	103.4	11.6
					Clay, firm, slightly moist, rock fragments (to 6"), organic odor.			
8	B7@8'					3	120.8	8.8
10								
	B7@11'					4	97.1	22.9
12					Clayey Sand with Silt, medium dense, slightly moist, grayish brown, medium-grained, some coarse-grained. - 13.0' some metal wire debris			
14					Clay, firm, slightly moist, some silt, rock fragments (to 3.5"), organic odor. - 14.0' some slate fragments (to 8"), wood debris (to 10"), metal scrap (to 5"), paper			
16	B7@16'					4	112.6	12.2
18					Clayey Sand with Silt, medium dense, slightly moist, medium-grained, some coarse-grained, grayish brown, metal debris (to 9"). - 18.0' rock fragments (to 11")			
20					Sand, poorly graded, medium dense, slightly moist, grayish brown, medium-grained, some coarse-grained, some fine gravel and rock debris (to 5"), friable. - 21.5' some gravel (to 3"), rounded to subrounded, some claystone fragments (to 6")			
	B7@21'					3	122.0	12.9
22								
24								
26	B7@26'			SP	**SAN PEDRO SAND (Qsp)** Sand, poorly graded, medium dense to dense, slightly moist, yellowish brown, fine- to medium-grained, some coarse-grained, massive, friable.	6		
28								

Figure A7,
Log of Boring 7, Page 1 of 2

A9201-06-01C BORING LOGS.GPJ

SAMPLE SYMBOLS
☐ ... SAMPLING UNSUCCESSFUL
◧ ... STANDARD PENETRATION TEST
■ ... DRIVE SAMPLE (UNDISTURBED)
☒ ... DISTURBED OR BAG SAMPLE
◪ ... CHUNK SAMPLE
▼ ... WATER TABLE OR SEEPAGE

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GEOCON

PROJECT NO. A9201-06-01C

BORING 7

ELEV. (MSL.) **191'** DATE COMPLETED **7/20/15**

EQUIPMENT **BUCKET AUGER** BY: **RA**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
30								
31	B7@31' ■				- 31.0' increase in coarse-grained sand	5		
32					- 32.5' subrounded cobble (to 5")			
34								
35				SP				
36	B7@36' ■					4		
38					- 38.0' trace subrounded cobbles (to 4")			
40					Sand, well-graded, medium dense, slightly moist, yellowish brown, some silt, friable, rounded to subrounded coarse-grained, friable.			
41	B7@41' ■					4		
42				SW				
44								
45				SP	Sand, poorly graded, slightly moist, yellowish brown, coarse-grained, some medium-grained, massive, friable.			
46	B7@46' ■					4		

Total depth of boring: 46.5 feet
Fill to 25 feet.
No groundwater encountered.
Backfilled with soil cuttings and tamped.

*Penetration resistance for Kelly Bar falling 12 inches.
Kelly Bar weights: 0-25 feet, 4,900 lbs, 25-50 feet, 3,400 lbs, 50-75 feet, 2,200 lbs, 70-100 feet, 1,200 lbs

Figure A7,
Log of Boring 7, Page 2 of 2

SAMPLE SYMBOLS

☐ ... SAMPLING UNSUCCESSFUL
◧ ... STANDARD PENETRATION TEST
■ ... DRIVE SAMPLE (UNDISTURBED)
▨ ... DISTURBED OR BAG SAMPLE
◪ ... CHUNK SAMPLE
▼ ... WATER TABLE OR SEEPAGE

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PROJECT NO. A9201-06-01C

BORING 8

ELEV. (MSL.) 213' DATE COMPLETED 7/20/15

EQUIPMENT BUCKET AUGER BY: RA

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0	BULK 0-5'				**ARTIFICIAL FILL (af)** Sandy Silt, soft, dry, light grayish brown, some diatomaceous siltstone fragments, brick fragments (to 3").			
2								
	B8@3'					2	110.5	4.3
4								
					- 5.0' some gravel (to 3")			
6	B8@6'					3	87.3	23.0
8	B8@8'				Silty Sand, poorly graded, medium dense, slightly moist, yellowish brown, fine- to medium-grained, trace coarse-grained, friable. - 9.5' some rock fragments (to 3")	3	108.1	2.9
10								
	B8@11'					3	114.8	12.6
12								
14					Clay, firm, slightly moist, dark brown, some coarse-grained sand, rock fragments (to 5"), organic odor.			
					Sand with Clay, poorly graded, medium dense, grayish brown, fine- to medium-grained, some rock fragments (to 3").			
16	B8@16'					4	118.5	12.1
18					- 17.5' rock fragments (to 5")			
					- 19.0' rock fragments (to 6"), increase in clay content			
20								
	B8@21'					3	112.6	14.0
22					- 21.5' rock fragments (to 1.5") - 22.5' some plastic piping			
24					- 23.5' rock fragments (to 5"), organic odor			
26	B8@26'					5	115.0	9.4
					Silty Sand, poorly graded, slightly moist, yellowish brown, fine- to medium-grained, trace coarse-grained, friable. - 27.5' some very soft claystone fragments (to 8")			
28								

A9201-06-01C BORING LOGS.GPJ

Figure A8,
Log of Boring 8, Page 1 of 2

SAMPLE SYMBOLS			
	☐ ... SAMPLING UNSUCCESSFUL	◧ ... STANDARD PENETRATION TEST	■ ... DRIVE SAMPLE (UNDISTURBED)
	☒ ... DISTURBED OR BAG SAMPLE	◩ ... CHUNK SAMPLE	▼ ... WATER TABLE OR SEEPAGE

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PROJECT NO. A9201-06-01C

BORING 8

ELEV. (MSL.) **213'** DATE COMPLETED **7/20/15**

EQUIPMENT **BUCKET AUGER** BY: **RA**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
30								
	B8@31' ■					2		
32								
34					- 34.0' very soft claystone fragments (to 7")			
36	B8@36' ■				**SAN PEDRO SAND (Qsp)** Sand, poorly graded, medium dense, slightly moist, light yellowish brown, very fine- to fine-grained, trace silt, friable.	3		
38								
40	BULK 40-45' ▨							
	B8@41' ■			SP	- 41.0' trace coarse-grained sand	5		
42								
44								
					- 45.0' increase in coarse-grained sand content, subrounded, trace rounded gravel (to 1.5")			
46	B8@46' ■					6		

Total depth of boring: 46.5 feet
Fill to 36 feet.
No groundwater encountered.
Backfilled with soil cuttings and tamped.

*Penetration resistance for Kelly Bar falling 12 inches.
Kelly Bar weights: 0-25 feet, 4,900 lbs, 25-50 feet, 3,400 lbs, 50-75 feet, 2,200 lbs, 70-100 feet, 1,200 lbs

Figure A8,
Log of Boring 8, Page 2 of 2

SAMPLE SYMBOLS

☐ ... SAMPLING UNSUCCESSFUL
◧ ... STANDARD PENETRATION TEST
■ ... DRIVE SAMPLE (UNDISTURBED)
☒ ... DISTURBED OR BAG SAMPLE
◪ ... CHUNK SAMPLE
▼ ... WATER TABLE OR SEEPAGE

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PROJECT NO. A9201-06-01C

BORING 9

ELEV. (MSL.) **243'** DATE COMPLETED **7/23/15**

EQUIPMENT **BUCKET AUGER** BY: **RA**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0	BULK 0-5'				**ARTIFICIAL FILL (af)** Silt with Sand, soft to stiff, slightly moist, pale grayish brown, fine- to medium-grained, friable.			
2	B9@2'					7		
4					- 4.0' some paper fragments			
	B9@5'					10		
6					Sandy Silt with Clay, firm, slightly moist, grayish brown, medium- to coarse-grained, some diatomaceous siltstone fragments (to 2").			
	B9@7'					13		
8					- 8.5' rock fragments (to 5")			
10	B9@10'				- 10.5' some rock fragments (to 1")	16	66.4	38.6
12					- 11.5' rock fragments (to 3")			
14								
16	B9@15'				- 15.5' reworked diatomaceous siltstone, very soft (H1), laminated, intensely fractured, slightly weathered	20	82.6	20.2
18								
20	B9@20'				**SAN PEDRO SAND (Qsp)** Sand, poorly graded, dense, slightly moist, light grayish brown, medium- to coarse-grained, massive, friable.	21	128.5	2.7
22	BULK 21-23'			SP				

Total depth of boring: 23 feet
Fill to 20 feet.
No groundwater encountered.
Backfilled with soil cuttings and tamped.

*Penetration resistance for Kelly Bar falling 12 inches.
Kelly Bar weights: 0-15 feet, 1,770 lbs, 15-30 feet, 1,200 lbs, 30-45 feet, 760 lbs, 45-60 feet, 490 lbs

Figure A9,
Log of Boring 9, Page 1 of 1

A9201-06-01C BORING LOGS.GPJ

SAMPLE SYMBOLS

☐ ... SAMPLING UNSUCCESSFUL
◧ ... STANDARD PENETRATION TEST
■ ... DRIVE SAMPLE (UNDISTURBED)
▨ ... DISTURBED OR BAG SAMPLE
◪ ... CHUNK SAMPLE
▼ ... WATER TABLE OR SEEPAGE

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GEOCON

PROJECT NO. A9201-06-01C

BORING 10

ELEV. (MSL.) **236'** DATE COMPLETED **7/24/15**

EQUIPMENT **BUCKET AUGER** BY: **RA**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0		▩			**ARTIFICIAL FILL (af)** Sandy Silt, firm to stiff, slightly moist, light gray, fine- to medium-grained, some gravel (to 2").			
2	B10@2' ■	▩				8		
4		⁘			**SAN PEDRO SAND (Qsps)** Sand, poorly graded, dense, yellowish brown, fine- to medium-grained, some coarse-grained, friable.			
6	B10@5' ■	⁘			- 5.5' light gray to white	10		
8	B10@7' ■	⁘			- 7.5' increase in coarse-grained sand	13		
10		⁘		SP	- 9.0' trace rounded gravel (to 1")			
12		⁘						
14		⁘			- 14.0' medium- to coarse-grained sand			
	B10@15' ■	⁘				15		
					Total depth of boring: 15.5 feet Fill to 3 feet. No groundwater encountered. Backfilled with soil cuttings and tamped. *Penetration resistance for Kelly Bar falling 12 inches. Kelly Bar weights: 0-15 feet, 1,770 lbs, 15-30 feet, 1,200 lbs, 30-45 feet, 760 lbs, 45-60 feet, 490 lbs			

Figure A10,
Log of Boring 10, Page 1 of 1

A9201-06-01C BORING LOGS.GPJ

SAMPLE SYMBOLS			
	☐ ... SAMPLING UNSUCCESSFUL	◧ ... STANDARD PENETRATION TEST	■ ... DRIVE SAMPLE (UNDISTURBED)
	⊠ ... DISTURBED OR BAG SAMPLE	◩ ... CHUNK SAMPLE	▼ ... WATER TABLE OR SEEPAGE

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GEOCON

PROJECT NO. A9201-06-01C

BORING 11

ELEV. (MSL.) **240'** DATE COMPLETED **7/23/15**

EQUIPMENT **BUCKET AUGER** BY: **RA**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					**ARTIFICIAL FILL (af)** Silt with Sand, firm, slightly moist, pale grayish brown, fine- to medium-grained, friable.			
2	B11@2'				Sand with Silt, poorly graded, medium dense, slightly moist, dark brown, medium-grained, some coarse-grained, friable.	10		
4					- 4.0' rock fragments (to 2")			
	B11@5'				**SAN PEDRO SAND (Qsps)** Sand, poorly graded, dense, slightly moist, light gray to white, medium- to coarse-grained, some fine-grained, friable.	12	89.1	26.7
6								
	B11@7'					13		
8					- 9.0' coarse-grained sand			
10	B11@10'					16		
12								
14					- 13.5' trace fine-grained sand			
	B11@15'			SP		20		
16								
18								
20	B11@20'				- 20.5' coarse-grained sand, friable	18		
22								
24								
	B11@25'					20		

Total depth of boring: 25.5 feet
Fill to 5 feet.
No groundwater encountered.
Backfilled with soil cuttings and tamped.

*Penetration resistance for Kelly Bar falling 12 inches.
Kelly Bar weights: 0-15 feet, 1,770 lbs, 15-30 feet, 1,200 lbs, 30-45 feet, 760

A9201-06-01C BORING LOGS.GPJ

Figure A11,
Log of Boring 11, Page 1 of 2

SAMPLE SYMBOLS

☐ ... SAMPLING UNSUCCESSFUL	◧ ... STANDARD PENETRATION TEST	■ ... DRIVE SAMPLE (UNDISTURBED)
▨ ... DISTURBED OR BAG SAMPLE	◪ ... CHUNK SAMPLE	▼ ... WATER TABLE OR SEEPAGE

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PROJECT NO. A9201-06-01C

BORING 11

ELEV. (MSL.) **240'** DATE COMPLETED **7/23/15**

EQUIPMENT **BUCKET AUGER** BY: **RA**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					lbs, 45-60 feet, 490 lbs			

Figure A11,
Log of Boring 11, Page 2 of 2

A9201-06-01C BORING LOGS.GPJ

SAMPLE SYMBOLS			
	☐ ... SAMPLING UNSUCCESSFUL	◧ ... STANDARD PENETRATION TEST	■ ... DRIVE SAMPLE (UNDISTURBED)
	☒ ... DISTURBED OR BAG SAMPLE	◪ ... CHUNK SAMPLE	▼ ... WATER TABLE OR SEEPAGE

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GEOCON

PROJECT NO. A9201-06-01C

BORING 12

ELEV. (MSL.) **242'** DATE COMPLETED **7/23/15**

EQUIPMENT **BUCKET AUGER** BY: **RA**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					**ARTIFICIAL FILL (af)** Silt with Sand, stiff, slightly moist, pale grayish brown, fine- to medium-grained, friable.			
2	B12@2' ■					10		
					- 3.0' some glass and brick fragments (to 2")			
4					Silty Clay with Sand, stiff, slightly moist, brown, medium- to coarse-grained, some rock fragments (to 1"), organic odor.			
	B12@5' ■					13		
6					- 6.0' angular rock fragments (to 5")			
	B12@7' ■					14	89.7	22.3
					- 7.5' decrease in clay content, increase in silt content			
8								
10	B12@10' ■				- 10.0' subangular rock fragments (to 2")	15	93.1	25.9
					- 11.5' rock fragments (to 3")			
12								
14								
	B12@15' ■				Sand, poorly graded, dense, slightly moist, yellowish brown, fine- to medium-grained, trace silt, friable.	21		
16								
18								
20	B12@20' ■					23		
					- 21.0' subangular rock fragments (to 5")			
22								
24								
	B12@25' ■				Sandy Clay, stiff, slightly moist, dark brown, medium- to coarse-grained, angular rock fragments (to 1").	25		
26								
					- 27.0' angular rock fragments (to 4")			
28								
					- 29.0' decrease in sand content, increase in clay content			

Figure A12,
Log of Boring 12, Page 1 of 2

SAMPLE SYMBOLS

☐ ... SAMPLING UNSUCCESSFUL
◧ ... STANDARD PENETRATION TEST
■ ... DRIVE SAMPLE (UNDISTURBED)
⊠ ... DISTURBED OR BAG SAMPLE
◪ ... CHUNK SAMPLE
▼ ... WATER TABLE OR SEEPAGE

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GEOCON

BORING 12

ELEV. (MSL.) **242'** DATE COMPLETED **7/23/15**

EQUIPMENT **BUCKET AUGER** BY: **RA**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
30	B12@30' ■	▨				26	104.4	11.5
32		▨						
34		▨			- 33.5' subrounded cobbles (to 11")			
36	B12@36' ■	⁘			**SAN PEDRO SAND (Qsp)** Sand, poorly graded, dense, slightly moist, light gray, medium- to coarse-grained.	28		
38		⁘		SP				
40	B12@40' ■	⁘			- 40.5' heavy caving	27		

Total depth of boring: 41 feet
Fill to 36 feet.
No groundwater encountered.
Backfilled with soil cuttings and tamped.

*Penetration resistance for Kelly Bar falling 12 inches.
Kelly Bar weights: 0-15 feet, 1,770 lbs, 15-30 feet, 1,200 lbs, 30-45 feet, 760 lbs, 45-60 feet, 490 lbs

Figure A12, Log of Boring 12, Page 2 of 2

SAMPLE SYMBOLS

☐ ... SAMPLING UNSUCCESSFUL
◧ ... STANDARD PENETRATION TEST
■ ... DRIVE SAMPLE (UNDISTURBED)
☒ ... DISTURBED OR BAG SAMPLE
◪ ... CHUNK SAMPLE
▼ ... WATER TABLE OR SEEPAGE

PROJECT NO. A9201-06-01C

BORING 13

ELEV. (MSL.) **193'** DATE COMPLETED **7/24/15**

EQUIPMENT **BUCKET AUGER** BY: **RA**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0 2 4					**ARTIFICIAL FILL (af)** Sandy Silt, light brown, fine- to medium-grained, some coarse-grained, rock fragments (to 8").			
6	B13@5' ■					11		
8 10				SP	**SAN PEDRO SAND (Qsp)** Sand, poorly graded, dense, slightly moist, dark yellowish brown, very fine- to fine-grained, friable. - 8.5' trace subrounded cobbles (to 5")			

Total depth of boring: 11 feet
Fill to 7.5 feet.
No groundwater encountered.
Backfilled with soil cuttings and tamped.

*Penetration resistance for Kelly Bar falling 12 inches.
Kelly Bar weights: 0-15 feet, 1,770 lbs, 15-30 feet, 1,200 lbs, 30-45 feet, 760 lbs, 45-60 feet, 490 lbs

Figure A13,
Log of Boring 13, Page 1 of 1

A9201-06-01C BORING LOGS.GPJ

SAMPLE SYMBOLS

☐ ... SAMPLING UNSUCCESSFUL
◧ ... STANDARD PENETRATION TEST
■ ... DRIVE SAMPLE (UNDISTURBED)
☒ ... DISTURBED OR BAG SAMPLE
◩ ... CHUNK SAMPLE
▼ ... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

GEOCON

PROJECT NO. A9201-06-01C

BORING 14

ELEV. (MSL.) **217'** DATE COMPLETED **7/24/15**

EQUIPMENT **BUCKET AUGER** BY: **RA**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0	BULK 0-5'				**ARTIFICIAL FILL (af)** Sandy Silt, firm, slightly moist, pale brown, fine- to medium-grained, some coarse-grained, rock fragments (to 11"), some cobbles (to 4"). - 1.25' concrete fragment (22" in longest dimension) - 2.5' rock fragments (to 11")			
2								
4					Silty Sand, medium dense, well-graded, slightly moist, pale brown, rock fragments (to 3").			
	B14@5'					8	110.5	12.8
6								
8								
10	B14@10'				**SAN PEDRO SAND (Qsp)** Silty Sand, poorly graded, medium dense, slightly moist, pale brown, very fine- to fine-grained, friable, subrounded gravel (to 2").	13		
12				SM				
14								
	B15@15'						88.6	2.4

Total depth of boring: 15.5 feet
Fill to 9 feet.
No groundwater encountered.
Backfilled with soil cuttings and tamped.

*Penetration resistance for Kelly Bar falling 12 inches.
Kelly Bar weights: 0-15 feet, 1,770 lbs, 15-30 feet, 1,200 lbs, 30-45 feet, 760 lbs, 45-60 feet, 490 lbs

**Figure A15,
Log of Boring 14, Page 1 of 1**

SAMPLE SYMBOLS
- ☐ ... SAMPLING UNSUCCESSFUL
- ☒ ... DISTURBED OR BAG SAMPLE
- ◧ ... STANDARD PENETRATION TEST
- ◪ ... CHUNK SAMPLE
- ■ ... DRIVE SAMPLE (UNDISTURBED)
- ▼ ... WATER TABLE OR SEEPAGE

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PROJECT NO. A9201-06-01C

BORING 15

ELEV. (MSL.) **217'** DATE COMPLETED **7/24/15**

EQUIPMENT **BUCKET AUGER** BY: **RA**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					**ARTIFICIAL FILL (af)** Sand, well-graded, loose to medium dense, pale brown, some silt, some rock fragments (to 5").			
2								
4					- 4.0' trace subrounded cobbles (to 2")			
	B15@5'					11		
6					- 6.0' rock fragments (to 11")			
8	B15@7.5'					19		
					- 8.5' tree branch			
10	B15@10'				- 10.0' trace subrounded cobbles (to 3")	19		
	BULK 10-15'							
12					- 12.0' subrounded gravel (to 1")			
					- 13.0' subrounded cobbles (to 3")			
14								
	B15@15'					20		
16								
18								
	BULK 19-19.5'				- 19.0' decrease in cobbles			
20	B15@20'				**SAN PEDRO SAND (Qsp)** Sandy Silt to Silty Sand, dense, slightly moist, pale brown, fine- to medium-grained, trace coarse-grained, friable.	22		
22								
24				ML/SM	- 23.5' subrounded gravel (to 2") - 24.0' increase in coarse-grained sand			
	B15@25'					22		
26					- 26.5' trace subrounded cobbles (to 3")			
28								

A9201-06-01C BORING LOGS.GPJ

Figure A17,
Log of Boring 15, Page 1 of 2

SAMPLE SYMBOLS			
	☐ ... SAMPLING UNSUCCESSFUL	◧ ... STANDARD PENETRATION TEST	■ ... DRIVE SAMPLE (UNDISTURBED)
	☒ ... DISTURBED OR BAG SAMPLE	◩ ... CHUNK SAMPLE	▼ ... WATER TABLE OR SEEPAGE

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GEOCON

PROJECT NO. A9201-06-01C

BORING 15

ELEV. (MSL.) **217'** DATE COMPLETED **7/24/15**

EQUIPMENT **BUCKET AUGER** BY: **RA**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
30				ML/SM	- 30.0 to 32.5' cobble-rich zone			
32								
					- 33.0' sand with gravel, yellowish brown, very fine- to fine-grained			
34								
36	BULK 35-37'	▨						
	B15@37'	■				21		
38				SP-SM	Sand with Silt, poorly graded, dense, slightly moist, yellowish brown, very fine- to fine-grained, some coarse-grained, some gravel (to 1").			
40					- 40.0' trace clay, cobble-rich zone (to 1")			
	B15@41'	■				22		

Total depth of boring: 41.5 feet
Fill to 20 feet.
No groundwater encountered.
Backfilled with soil cuttings and tamped.

*Penetration resistance for Kelly Bar falling 12 inches.
Kelly Bar weights: 0-15 feet, 1,770 lbs, 15-30 feet, 1,200 lbs, 30-45 feet, 760 lbs, 45-60 feet, 490 lbs

Figure A17,
Log of Boring 15, Page 2 of 2

SAMPLE SYMBOLS

☐ ... SAMPLING UNSUCCESSFUL
◧ ... STANDARD PENETRATION TEST
■ ... DRIVE SAMPLE (UNDISTURBED)
☒ ... DISTURBED OR BAG SAMPLE
◪ ... CHUNK SAMPLE
▼ ... WATER TABLE OR SEEPAGE

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PROJECT NO. A9201-06-01C

BORING 13A

ELEV. (MSL.) 192' DATE COMPLETED 7/24/15

EQUIPMENT BUCKET AUGER BY: RA

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0 2		▒			**ARTIFICIAL FILL (af)** Silty Sand, medium dense, well-graded, slightly moist, rock fragments (to 6"). - 2.0' rock fragments (to 11")			
4 6 8	B13A@5' ■	▒			Sandy Clay, firm, slightly moist, brown, fine- to medium-grained, some coarse-grained, some siltstone fragments (to 1"). - 8.5' rock fragments (to 11") - 9.0' increase in sand content	9	100.8	11.0
10 12	B13A@10 ■	▒			Clayey Sand, poorly graded, medium dense to dense, slightly moist, olive brown, very fine- to fine-grained, some subrounded cobbles (to 5"), organic odor.	16	125.2	10.6
14		▒			Sand, well-graded, dark yellowish brown, coarse-grained, trace rounded gravel (to 1"). - 13.0' rock fragments (to 6")			
16	B13A@15 ■	░		SP	**SAN PEDRO SAND (Qsp)** Silty Sand, poorly graded, slightly moist, light grayish brown, very fine- to fine-grained, friable. - 16.0' trace subrounded cobbles (to 3")	12	94.0	7.8
18	B13A@18 ■	░				13	88.4	4.4

Total depth of boring: 18.5 feet
Fill to 15 feet.
No groundwater encountered.
Backfilled with soil cuttings and tamped.

*Penetration resistance for Kelly Bar falling 12 inches.
Kelly Bar weights: 0-15 feet, 1,770 lbs, 15-30 feet, 1,200 lbs, 30-45 feet, 760 lbs, 45-60 feet, 490 lbs

A9201-06-01C BORING LOGS.GPJ

Figure A14,
Log of Boring 13A, Page 1 of 1

SAMPLE SYMBOLS			
	☐ ... SAMPLING UNSUCCESSFUL	◧ ... STANDARD PENETRATION TEST	■ ... DRIVE SAMPLE (UNDISTURBED)
	☒ ... DISTURBED OR BAG SAMPLE	◩ ... CHUNK SAMPLE	▼ ... WATER TABLE OR SEEPAGE

PROJECT NO. A9201-06-01C

BORING 14A

ELEV. (MSL.) 218' DATE COMPLETED 7/24/15

EQUIPMENT BUCKET AUGER BY: RA

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					**ARTIFICIAL FILL (af)** Sand, poorly graded, medium dense, slightly moist, pale brown, very fine- to fine-grained, some claystone fragments (to 5"), friable.			
2					Sand, well-graded, slightly moist, yellowish brown, some rock fragments (to 5").			
4								
6								
8					Sandy Clay, firm, slightly moist, olive brown, fine- to medium-grained, organic odor.			
10	B14A@10				- 10.0' some rock fragments (to 4")	8	121.5	11.9
12								
14								
	B14A@15					12		
16					- 15.5' trace subrounded cobbles (to 3")			
18								
20	B14A@20				Silty Sand, poorly graded, medium dense, slightly moist, dark yellowish brown, fine- to medium-grained, trace clay, trace cobbles (to 3").	17	120.3	9.5
22					- 21.5' rock fragments (to 5")			
24					**SAN PEDRO SAND (Qsp)** Silty Sand, poorly graded, medium dense to dense, pale brown, very fine- to fine-grained, friable, micaceous.			
26	BULK 25-26' B14A@25			SM		16	98.4	3.4
	B14A@27					18	92.5	2.8
					Total depth of boring: 27.5 feet Fill to 23.5 feet. No groundwater encountered. Backfilled with soil cuttings and tamped.			

A9201-06-01C BORING LOGS.GPJ

Figure A16,
Log of Boring 14A, Page 1 of 2

SAMPLE SYMBOLS

☐ ... SAMPLING UNSUCCESSFUL
▨ ... DISTURBED OR BAG SAMPLE
◧ ... STANDARD PENETRATION TEST
◪ ... CHUNK SAMPLE
■ ... DRIVE SAMPLE (UNDISTURBED)
▼ ... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

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PROJECT NO. A9201-06-01C

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	**BORING 14A** ELEV. (MSL.) **218'** DATE COMPLETED **7/24/15** EQUIPMENT **BUCKET AUGER** BY: **RA**	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					MATERIAL DESCRIPTION			
					*Penetration resistance for Kelly Bar falling 12 inches. Kelly Bar weights: 0-15 feet, 1,770 lbs, 15-30 feet, 1,200 lbs, 30-45 feet, 760 lbs, 45-60 feet, 490 lbs			

Figure A16,
Log of Boring 14A, Page 2 of 2

SAMPLE SYMBOLS			
	☐ ... SAMPLING UNSUCCESSFUL	◧ ... STANDARD PENETRATION TEST	■ ... DRIVE SAMPLE (UNDISTURBED)
	⊠ ... DISTURBED OR BAG SAMPLE	◪ ... CHUNK SAMPLE	▼ ... WATER TABLE OR SEEPAGE

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PROJECT NO. A9201-06-01C

HAND AUGER 1

ELEV. (MSL.) **203'** DATE COMPLETED **7/22/15**

EQUIPMENT **HAND AUGER** BY: **RP**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0	BULK 0-3'				**ARTIFICIAL FILL (af)** Silty Sand, loose to medium dense, dry, brown, fine- to coarse-grained, trace rootlets.			
2					Sandy Silt, firm slightly moist, brown to olive brown, fine-grained, some coarse-grained.			
	HA1@3'			SM	**MARINE SAND (Qm)** Silty Sand, loose, slightly moist, brown to reddish brown, fine-grained, some coarse-grained, trace rootlets.			
4								
6								
8	HA1@7.5							
10	HA1@10'							
12								
				SP	**SAN PEDRO SAND (Qsp)** Sand, poorly graded, medium dense, slightly moist, pale brown to brown, fine-grained, trace medium-grained, trace silt.			
14	HA1@13.5							
16	HA1@15.5							
					Total depth of boring: 16 feet Fill to 2.5 feet. No groundwater encountered. Backfilled with soil cuttings and tamped.			

Figure A18,
Log of HAND AUGER 1, Page 1 of 1

A9201-06-01C HAND AUGER LOGS 0815.GPJ

SAMPLE SYMBOLS

☐ ... SAMPLING UNSUCCESSFUL	◧ ... STANDARD PENETRATION TEST	■ ... DRIVE SAMPLE (UNDISTURBED)
▩ ... DISTURBED OR BAG SAMPLE	◩ ... CHUNK SAMPLE	▼ ... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

PROJECT NO. A9201-06-01C

HAND AUGER 2

ELEV. (MSL.) **196'** DATE COMPLETED **7/22/15**

EQUIPMENT **HAND AUGER** BY: **RP**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0	BULK 0-3'				**ARTIFICIAL FILL (af)** Silty Sand, loose, slightly moist, dark brown, fine- to coarse-grained, some rounded gravel (to 1/2"), trace rootlets.			
2	HA2@2'				- 1.5' mottled brown and reddish brown, slightly porous, trace rootlets		93.4	10.9
4	HA2@3.5				**MARINE SAND (Qm)** Silty Sand, medium dense, slightly moist, reddish brown, fine-grained, trace coarse-grained, some silt.			
6								
8				SM	- 8.5' decrease in silt content			
10	HA2@10'							
12								
14								
16	HA2@15'				**SAN PEDRO SAND (Qsp)** Sand, poorly graded, medium dense, slightly moist, pale brown, fine-grained, some medium-grained.			
18				SP				
20	HA2@20'							

Total depth of boring: 20.5 feet
Fill to 3 feet.
No groundwater encountered.
Backfilled with soil cuttings and tamped.

Figure A19,
Log of HAND AUGER 2, Page 1 of 1

A9201-06-01C HAND AUGER LOGS 0815.GPJ

SAMPLE SYMBOLS
☐ ... SAMPLING UNSUCCESSFUL
◧ ... STANDARD PENETRATION TEST
■ ... DRIVE SAMPLE (UNDISTURBED)
☒ ... DISTURBED OR BAG SAMPLE
◪ ... CHUNK SAMPLE
▼ ... WATER TABLE OR SEEPAGE

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PROJECT NO. A9201-06-01C

HAND AUGER 3

ELEV. (MSL.) **208'** DATE COMPLETED **7/22/15**

EQUIPMENT **HAND AUGER** BY: **RP**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					**ARTIFICIAL FILL (af)** Silty Sand, loose to medium dense, slightly moist, brown, fine- to coarse-grained, trace rootlets.			
2	HA3@2.5'				- 2.0' mottled pale brown and reddish brown, trace silt, sand is sub-rounded			
4								
6	HA3@6'				**MARINE SAND (Qm)** Silty Sand, reddish brown, slightly moist, medium dense, fine- to coarse-grained.			
8								
10	HA3@10'			SM	- 9.5' decrease in silt content, trace rootlets			
12								
14								
16	HA3@15'				**SAN PEDRO SAND (Qsp)** Sand, poorly graded, medium dense, slightly moist, pale brown to yellowish brown, fine- to coarse-grained, trace fine rounded gravel (to 3/4").			
18								
20	HA3@20'			SP				
22								
24	HA3@23.5'				- 23.5' slightly porous, grading to mostly fine-grained			

Total depth of boring: 23.5 feet
Fill to 5.5 feet.
No groundwater encountered.
Backfilled with soil cuttings and tamped.

Figure A20,
Log of HAND AUGER 3, Page 1 of 1

SAMPLE SYMBOLS			
	☐ ... SAMPLING UNSUCCESSFUL	◧ ... STANDARD PENETRATION TEST	■ ... DRIVE SAMPLE (UNDISTURBED)
	☒ ... DISTURBED OR BAG SAMPLE	◪ ... CHUNK SAMPLE	▼ ... WATER TABLE OR SEEPAGE

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PROJECT NO. A9201-06-01C

HAND AUGER 4

ELEV. (MSL.) **232'** DATE COMPLETED **7/23/15**

EQUIPMENT **HAND AUGER** BY: **RP**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0 2 4					**ARTIFICIAL FILL (af)** Sand, poorly graded, loose, dry to slightly moist, pale brown to yellowish brown, fine-grained, large amount of siltstone fragments (to 4.5"), thinly bedded, highly weathered, massive, friable, some roots (to 2").			
6 8	■ HA4@5' ■ HA4@7.5			SP	**SAN PEDRO SAND (Qsp)** Sand, poorly graded, loose, slightly moist, pale brown to yellowish brown, fine-grained, some cemented fragments. - 6.5' some sandstone fragments			
					- 8.5' refusal			

Total depth of boring: 8.5 feet
Fill to 5 feet.
No groundwater encountered.
Backfilled with soil cuttings and tamped.

Figure A21,
Log of HAND AUGER 4, Page 1 of 1

SAMPLE SYMBOLS			
	☐ ... SAMPLING UNSUCCESSFUL	◧ ... STANDARD PENETRATION TEST	■ ... DRIVE SAMPLE (UNDISTURBED)
	☒ ... DISTURBED OR BAG SAMPLE	◪ ... CHUNK SAMPLE	▼ ... WATER TABLE OR SEEPAGE

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PROJECT NO. A9201-06-01C

HAND AUGER 5

ELEV. (MSL.) **227'** DATE COMPLETED **7/23/15**

EQUIPMENT **HAND AUGER** BY: **RP**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0 2	HA5@2' ■	▓			**ARTIFICIAL FILL (af)** Sand, poorly graded, very loose, dry, white to pale brown, fine- to coarse-grained, some cobbles, heavy amount of trash (from 1.5-2' below slope face).			
4 6 8	HA5@8' ■	░		SP	**SAN PEDRO SAND (Qsp)** Sand, poorly graded, very loose, dry, white to pale brown, fine- to coarse-grained, some rounded cobbles, caving sands. - 7.5' trace medium-grained sand, slightly oxidized - 8.5' caving sands			

Total depth of boring: 8.5 feet
Fill to 3 feet.
No groundwater encountered.
Backfilled with soil cuttings and tamped.

Figure A22,
Log of HAND AUGER 5, Page 1 of 1

A9201-06-01C HAND AUGER LOGS 0815.GPJ

SAMPLE SYMBOLS			
	☐ ... SAMPLING UNSUCCESSFUL	◧ ... STANDARD PENETRATION TEST	■ ... DRIVE SAMPLE (UNDISTURBED)
	☒ ... DISTURBED OR BAG SAMPLE	◩ ... CHUNK SAMPLE	▼ ... WATER TABLE OR SEEPAGE

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PROJECT NO. A9201-06-01C

HAND AUGER 6

ELEV. (MSL.) **253'** DATE COMPLETED **7/23/15**

EQUIPMENT **HAND AUGER** BY: **RP**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0 2					**ARTIFICIAL FILL (af)** Sand, poorly graded, loose, dry, pale brown, fine-grained, some coarse-grained, some rootlets, trash debris (to 1.5'), cobbles and boulders (to 2').			
4 6	HA6@3' ■ HA6@6' ■			SP	**SAN PEDRO SAND (Qsp)** Sand, poorly graded, medium dense, slightly moist, pale brown to yellowish brown, fine-grained.			

Total depth of boring: 7 feet
Fill to 2.5 feet.
No groundwater encountered.
Backfilled with soil cuttings and tamped.

A9201-06-01C HAND AUGER LOGS 0815.GPJ

Figure A23,
Log of HAND AUGER 6, Page 1 of 1

SAMPLE SYMBOLS

☐ ... SAMPLING UNSUCCESSFUL
◧ ... STANDARD PENETRATION TEST
■ ... DRIVE SAMPLE (UNDISTURBED)
▨ ... DISTURBED OR BAG SAMPLE
◪ ... CHUNK SAMPLE
▼ ... WATER TABLE OR SEEPAGE

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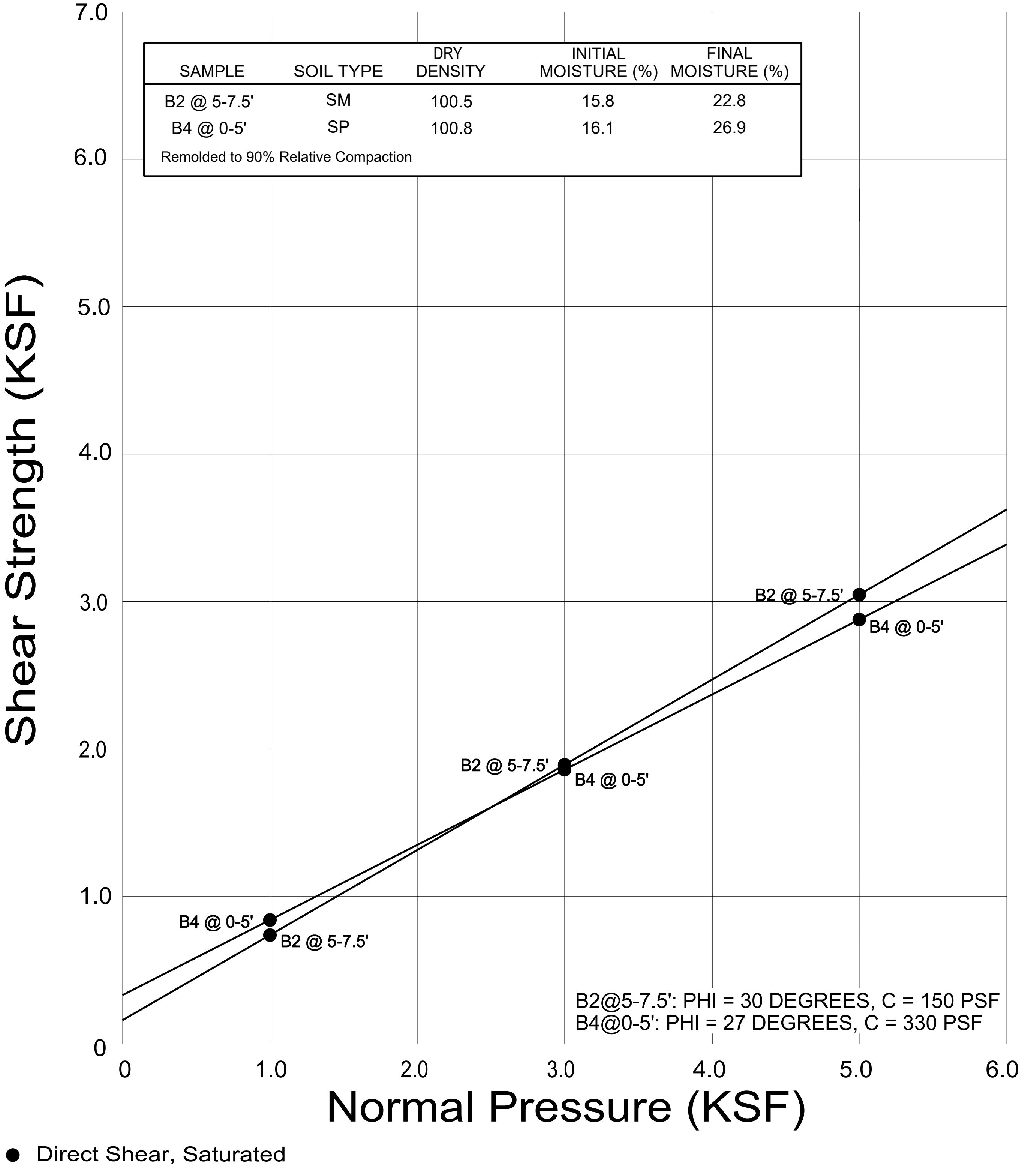
APPENDIX B

APPENDIX B

LABORATORY TESTING

Laboratory tests were performed in accordance with generally accepted test methods of the "American Society for Testing and Materials (ASTM)", or other suggested procedures. Selected samples were tested for direct shear strength, consolidation and expansion characteristics, corrosivity, in-place dry density and moisture content. The results of the laboratory tests are summarized in Figures B1 through B15. The in-place dry density and moisture content of the samples tested are presented on the boring logs, Appendix A.



SAMPLE	SOIL TYPE	DRY DENSITY	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B2 @ 5-7.5'	SM	100.5	15.8	22.8
B4 @ 0-5'	SP	100.8	16.1	26.9

Remolded to 90% Relative Compaction

B2@5-7.5': PHI = 30 DEGREES, C = 150 PSF
B4@0-5': PHI = 27 DEGREES, C = 330 PSF

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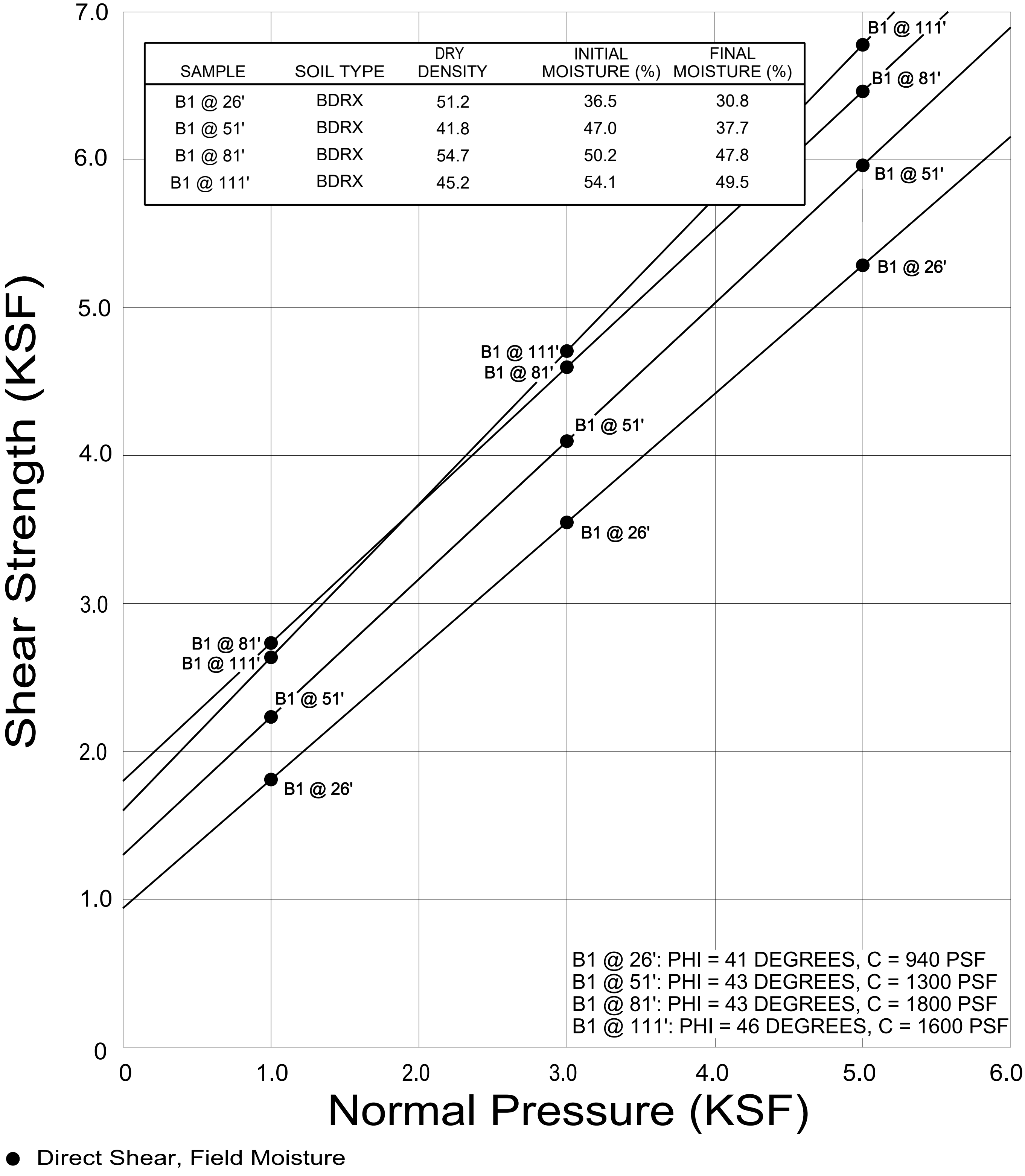
Drafted by: JMT | Checked by: NDB

DIRECT SHEAR TEST RESULTS

REYLENN PROPERTIES, LLC
HAWTHORNE & VIA VALMONTE
TORRANCE, CA 90505

MARCH 2016 | PROJECT NO. A9201-06-01C | FIG. B1



SAMPLE	SOIL TYPE	DRY DENSITY	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B1 @ 26'	BDRX	51.2	36.5	30.8
B1 @ 51'	BDRX	41.8	47.0	37.7
B1 @ 81'	BDRX	54.7	50.2	47.8
B1 @ 111'	BDRX	45.2	54.1	49.5

B1 @ 26': PHI = 41 DEGREES, C = 940 PSF
B1 @ 51': PHI = 43 DEGREES, C = 1300 PSF
B1 @ 81': PHI = 43 DEGREES, C = 1800 PSF
B1 @ 111': PHI = 46 DEGREES, C = 1600 PSF

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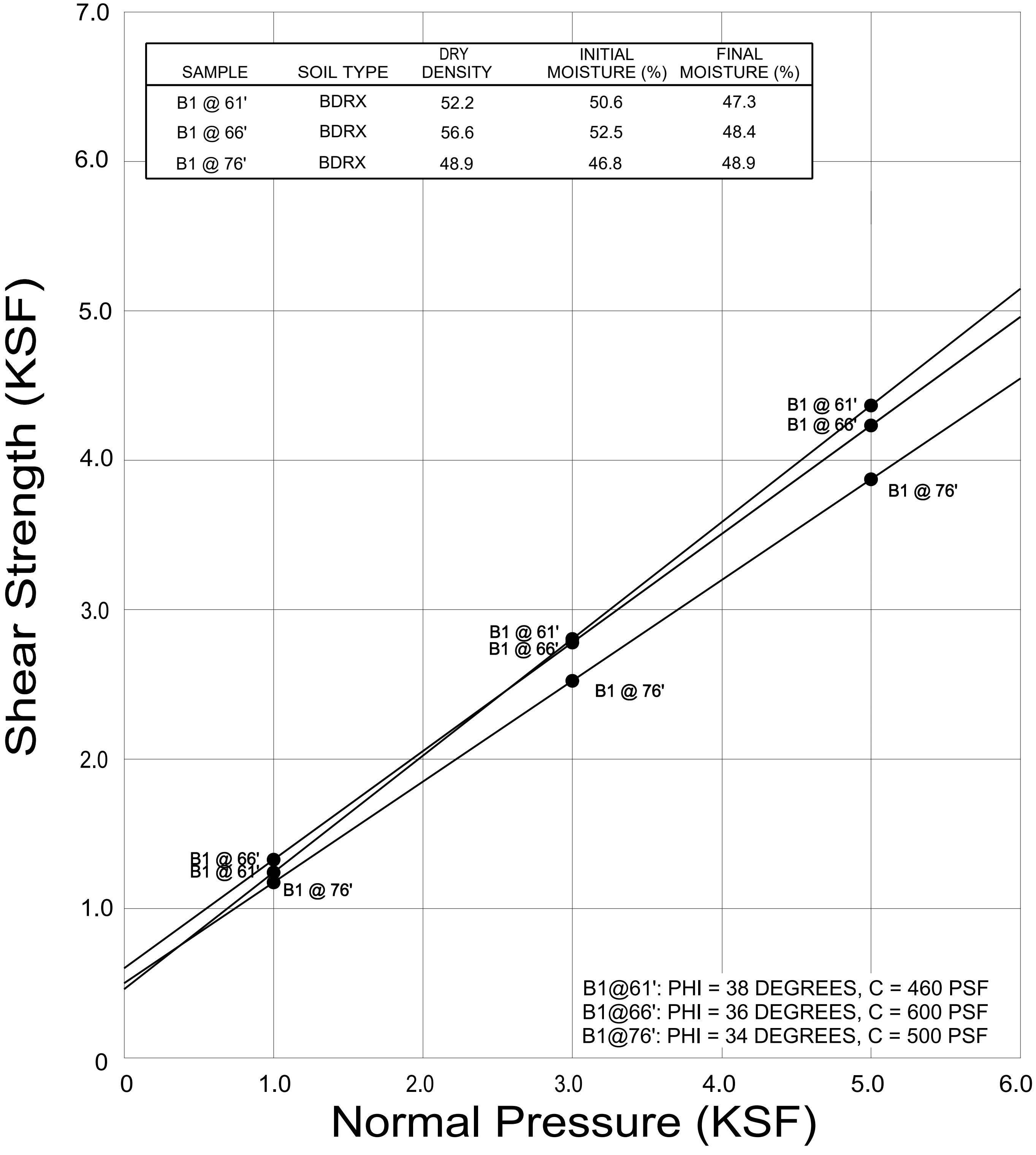
DIRECT SHEAR TEST RESULTS

REYLENN PROPERTIES, LLC
HAWTHORNE & VIA VALMONTE
TORRANCE, CA 90505

MARCH 2016 | PROJECT NO. A9201-06-01C | FIG. B2



SAMPLE	SOIL TYPE	DRY DENSITY	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B1 @ 61'	BDRX	52.2	50.6	47.3
B1 @ 66'	BDRX	56.6	52.5	48.4
B1 @ 76'	BDRX	48.9	46.8	48.9

B1@61': PHI = 38 DEGREES, C = 460 PSF
B1@66': PHI = 36 DEGREES, C = 600 PSF
B1@76': PHI = 34 DEGREES, C = 500 PSF

● Direct Shear, Field Moisture, Residual

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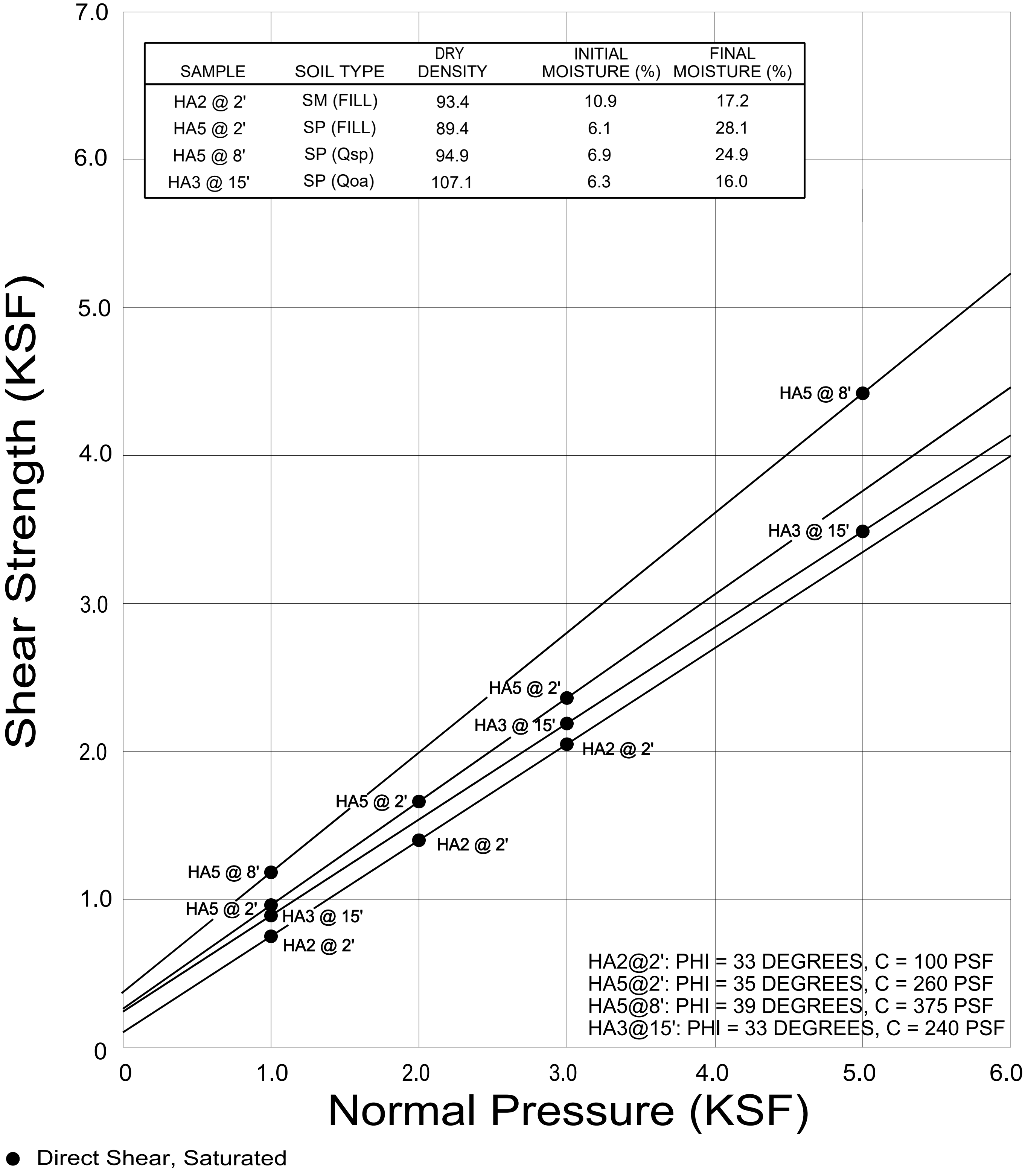
DIRECT SHEAR TEST RESULTS

REYLENN PROPERTIES, LLC
HAWTHORNE & VIA VALMONTE
TORRANCE, CA 90505

MARCH 2016 | PROJECT NO. A9201-06-01C | FIG. B3



SAMPLE	SOIL TYPE	DRY DENSITY	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
HA2 @ 2'	SM (FILL)	93.4	10.9	17.2
HA5 @ 2'	SP (FILL)	89.4	6.1	28.1
HA5 @ 8'	SP (Qsp)	94.9	6.9	24.9
HA3 @ 15'	SP (Qoa)	107.1	6.3	16.0

HA2@2': PHI = 33 DEGREES, C = 100 PSF
HA5@2': PHI = 35 DEGREES, C = 260 PSF
HA5@8': PHI = 39 DEGREES, C = 375 PSF
HA3@15': PHI = 33 DEGREES, C = 240 PSF

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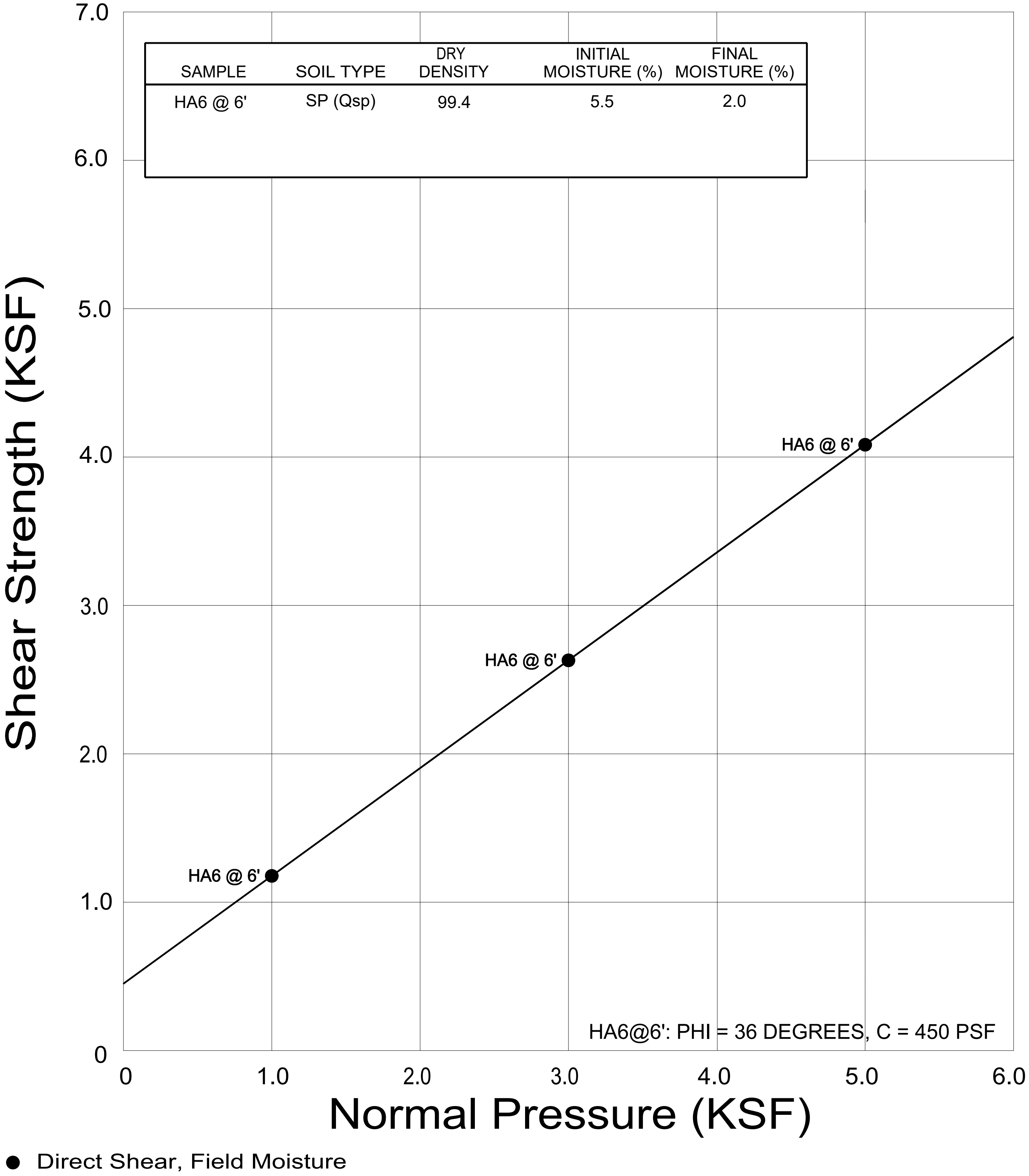
DIRECT SHEAR TEST RESULTS

REYLENN PROPERTIES, LLC
HAWTHORNE & VIA VALMONTE
TORRANCE, CA 90505

MARCH 2016 | PROJECT NO. A9201-06-01C | FIG. B4



SAMPLE	SOIL TYPE	DRY DENSITY	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
HA6 @ 6'	SP (Qsp)	99.4	5.5	2.0

HA6@6': PHI = 36 DEGREES, C = 450 PSF

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DIRECT SHEAR TEST RESULTS

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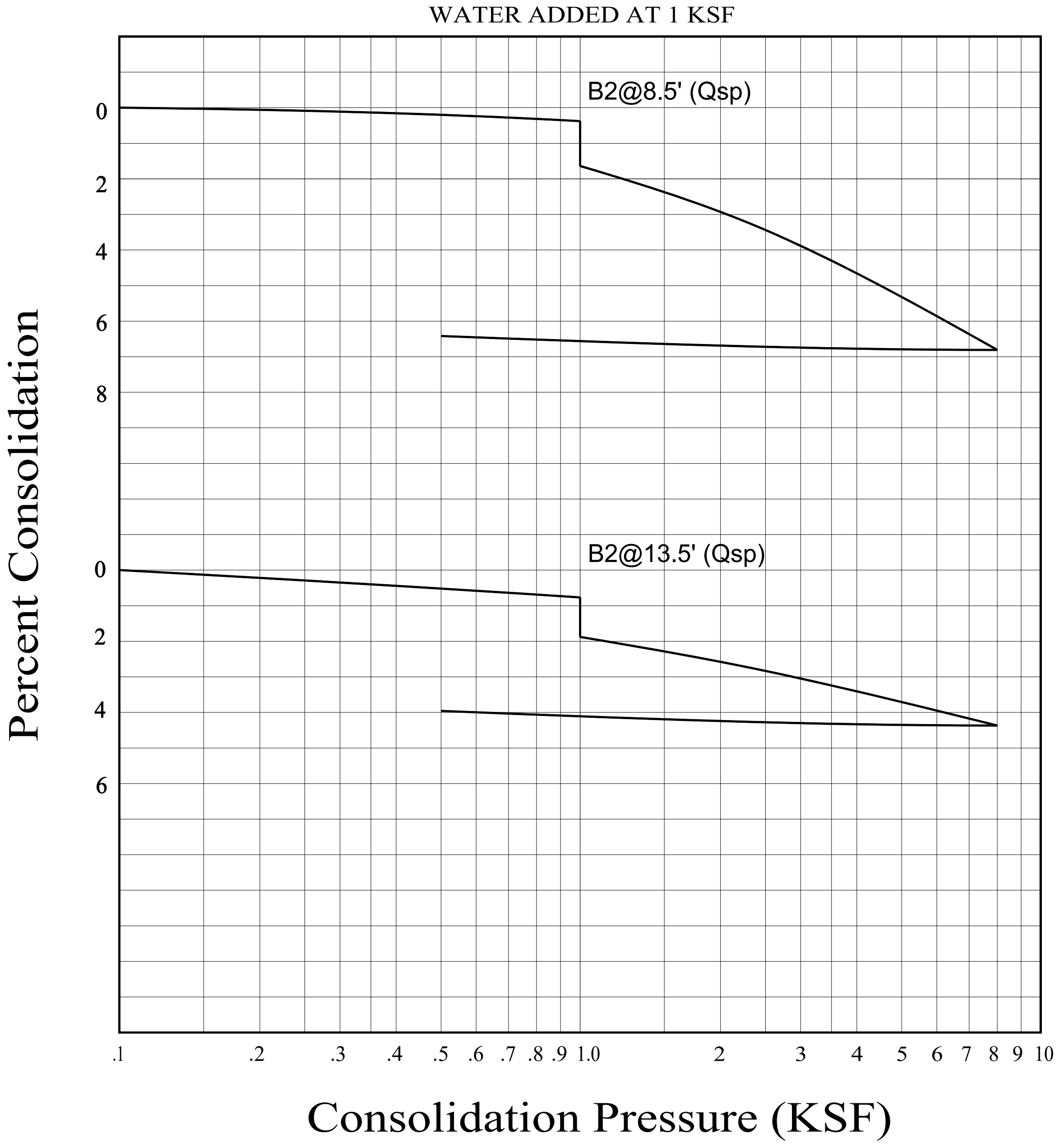
TORRANCE, CA 90505

MARCH 2016 | PROJECT NO. A9201-06-01C | FIG. B5



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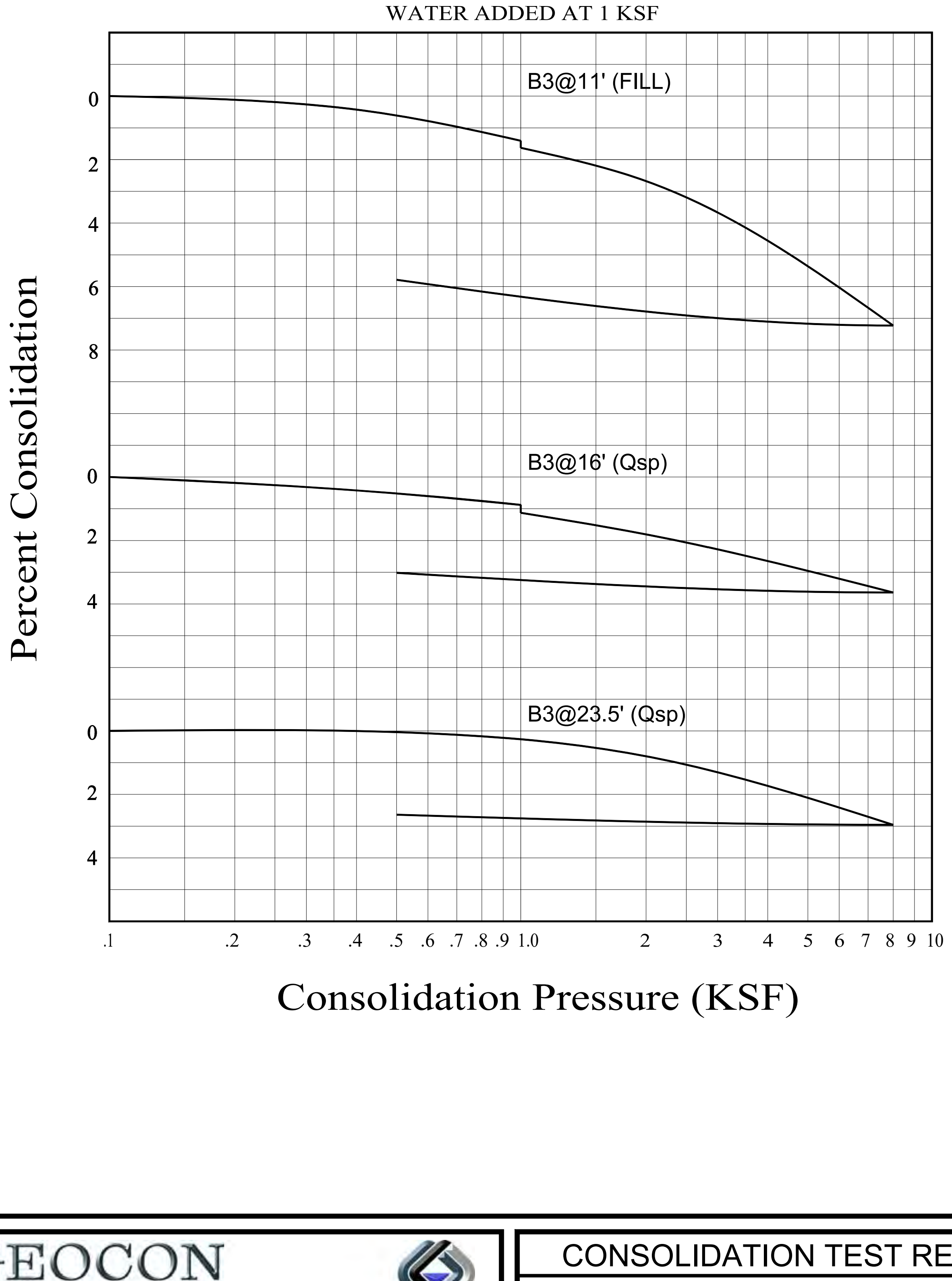
CONSOLIDATION TEST RESULTS

REYLENN PROPERTIES, LLC
HAWTHORNE & VIA VALMONTE
TORRANCE, CA 90505

MARCH 2016 | PROJECT NO. A9201-06-01C | FIG. B6



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CONSOLIDATION TEST RESULTS

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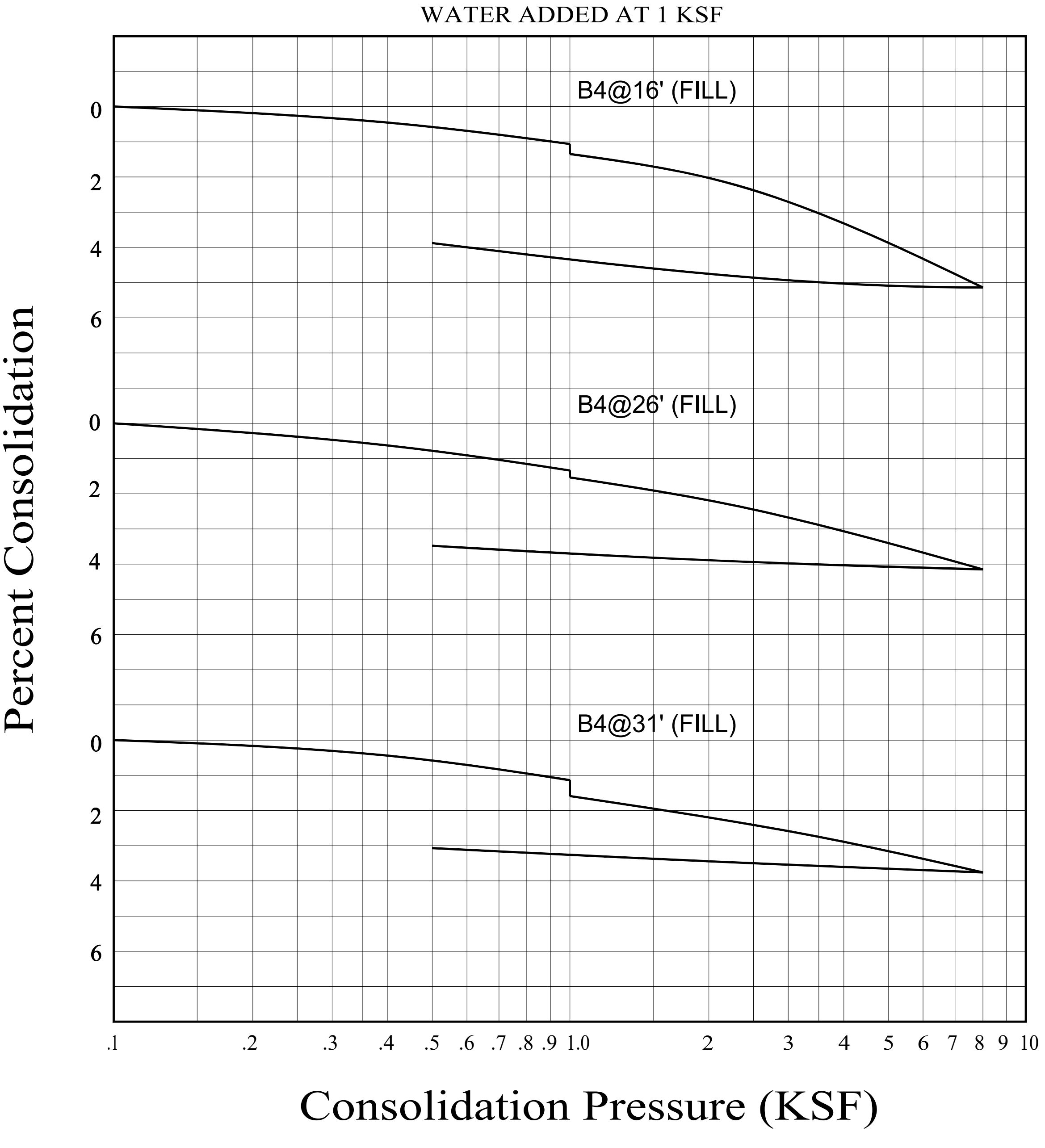
HAWTHORNE & VIA VALMONTE
TORRANCE, CA 90505

MARCH 2016 | PROJECT NO. A9201-06-01C | FIG. B7



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CONSOLIDATION TEST RESULTS

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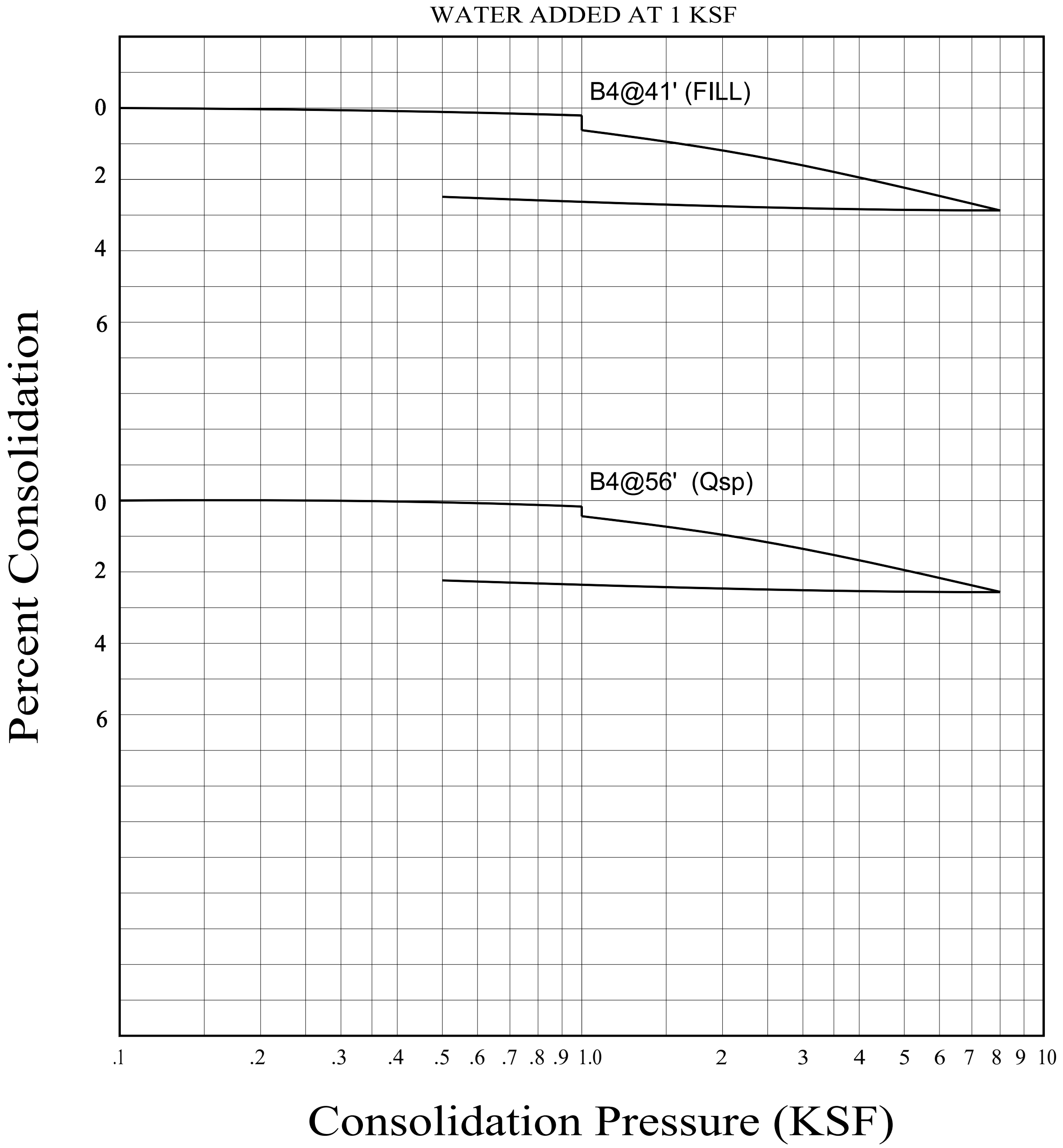
TORRANCE, CA 90505

MARCH 2016
PROJECT NO. A9201-06-01C
FIG. B8



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CONSOLIDATION TEST RESULTS

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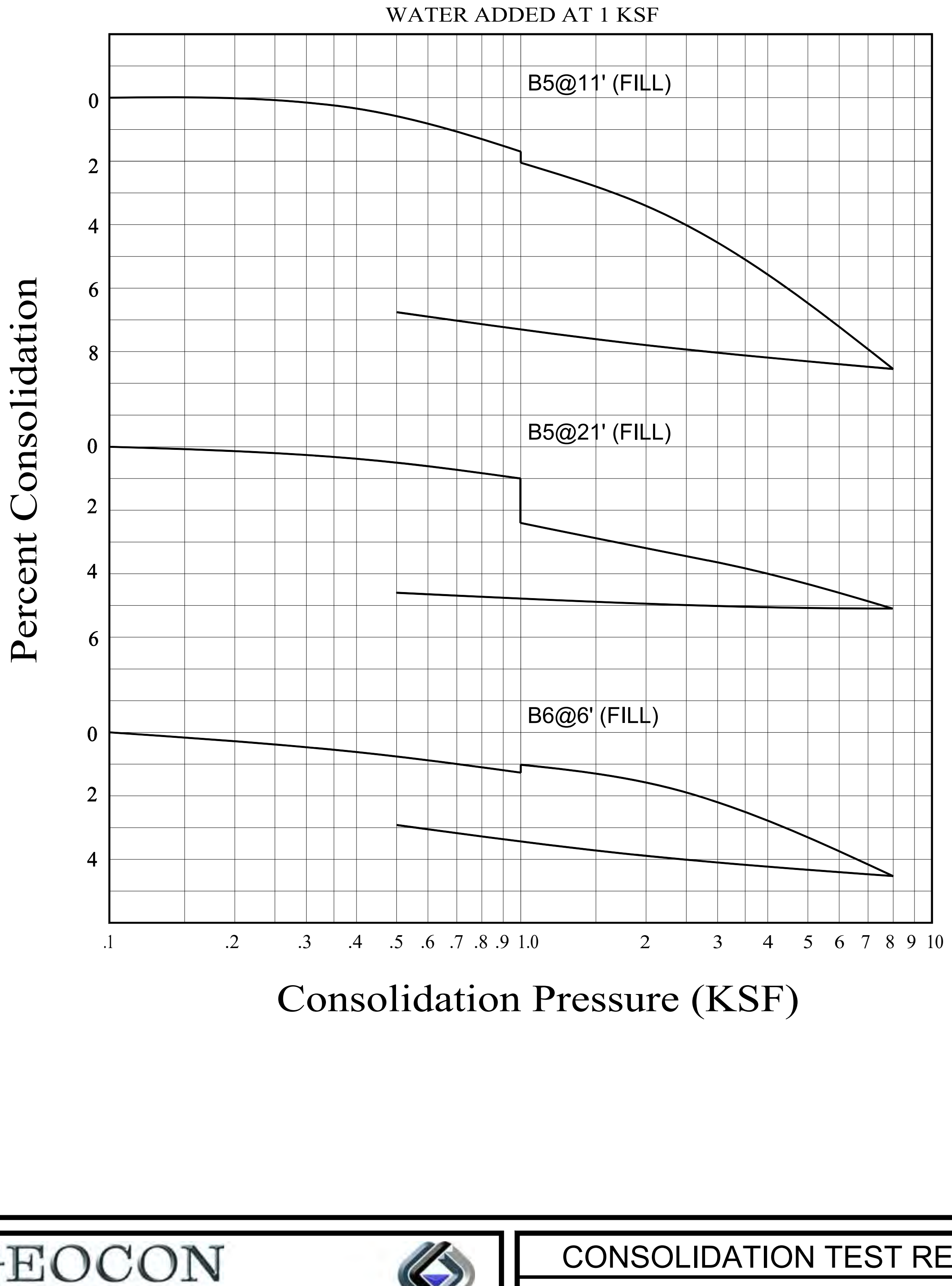
MARCH 2016 | PROJECT NO. A9201-06-01C | FIG. B9



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CONSOLIDATION TEST RESULTS

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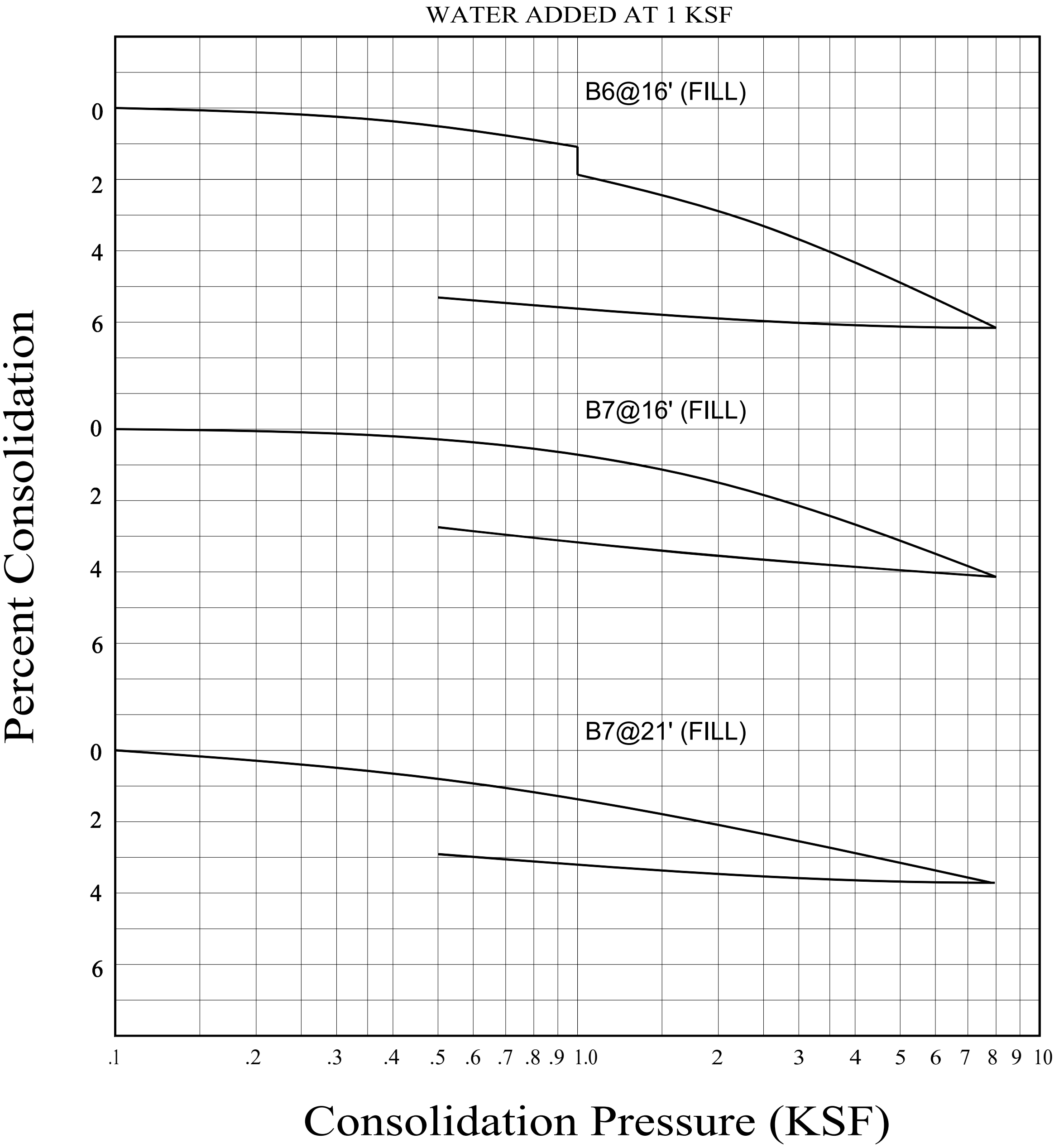
MARCH 2016 | PROJECT NO. A9201-06-01C | FIG. B10



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CONSOLIDATION TEST RESULTS

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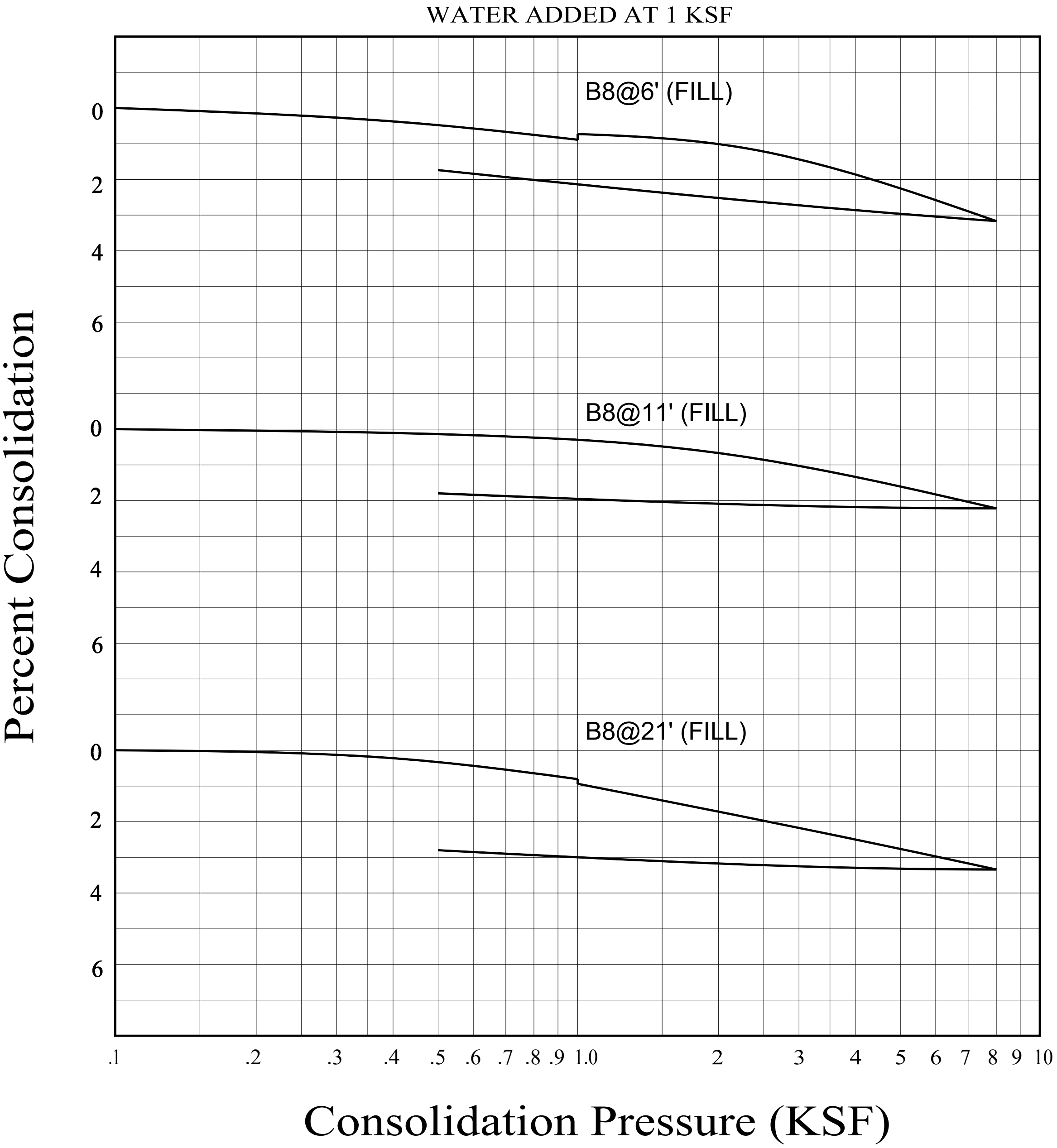
MARCH 2016 | PROJECT NO. A9201-06-01C | FIG. B11



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CONSOLIDATION TEST RESULTS

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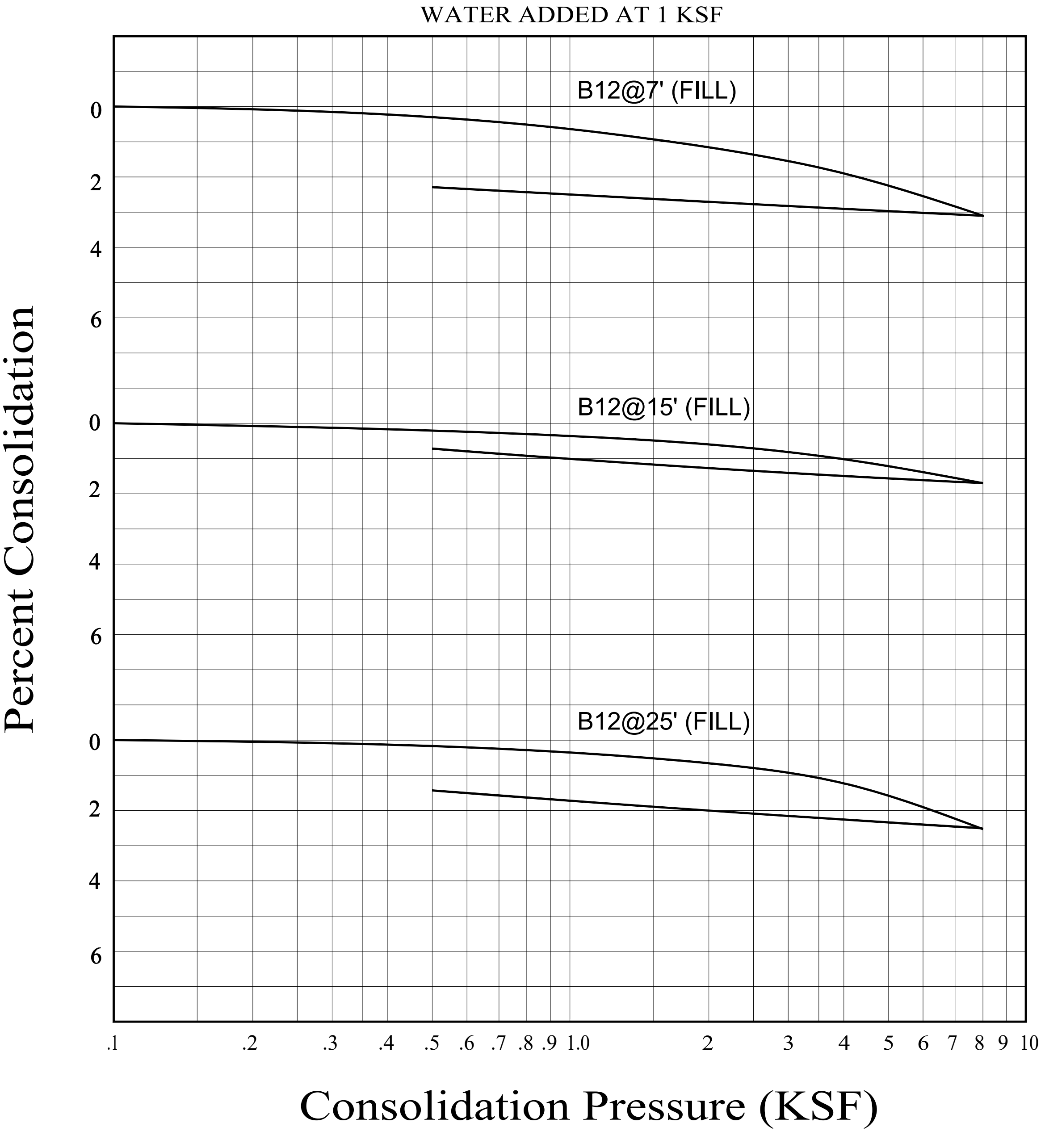
TORRANCE, CA 90505

MARCH 2016 | PROJECT NO. A9201-06-01C | FIG. B12



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CONSOLIDATION TEST RESULTS

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MARCH 2016 | PROJECT NO. A9201-06-01C | FIG. B13

SUMMARY OF LABORATORY EXPANSION INDEX TEST RESULTS
ASTM D 4829-11

Sample No.	Moisture Content (%)		Dry Density (pcf)	Expansion Index	*UBC Classification	**CBC Classification
	Before	After				
B2 @ 5-7.5'	12.3	24.1	101.8	33	Low	Expansive

* Reference: 1997 Uniform Building Code, Table 18-I-B.

** Reference: 2013 California Building Code, Section 1803.5.3

SUMMARY OF LABORATORY MAXIMUM DENSITY AND AND OPTIMUM MOISTURE CONTENT TEST RESULTS
ASTM D 1557-12

Sample No.	Soil Description	Maximum Dry Density (pcf)	Optimum Moisture (%)
B2 @ 5-7.5'	Brown Silty Sand	112.5	14.5
B4 @ 0-5'	Light Brown Gravelly Sand	112.0	15.5
B4 @ 10-15'	Brown Silty Sand	121.2	12.1
B4 @ 23-30'	Light Gray Sand	120.0	11.0
B4 @ 45-50'	Light Gray Sand	116.0	11.0

ORGANIC CONTENT

Sample No.	Organic Content (%)
B4@0-5'	2.8
B5@6'	4.4
B7@8'	3.9
B7@16'	3.3
B9@10'	3.0
B12@5'	2.0

LABORATORY TEST RESULTS

REYLENN PROPERTIES, LLC
HAWTHORNE & VIA VALMONTE
TORRANCE, CA 90505

MARCH 2016 | PROJECT NO. A9201-06-01C | FIG. B14

SUMMARY OF LABORATORY POTENTIAL OF HYDROGEN (pH) AND RESISTIVITY TEST RESULTS
CALIFORNIA TEST NO. 643

Sample No.	**pH**	**Resistivity (ohm centimeters)**
B4 @ 10-15'	7.93	650 (Severely Corrosive)
B8 @ 40-45'	8.4	2200 (Moderately Corrosive)

SUMMARY OF LABORATORY CHLORIDE CONTENT TEST RESULTS
EPA NO. 325.3

Sample No.	Chloride Ion Content (%)
B4 @ 10-15'	0.043
B8 @ 40-45'	0.006

SUMMARY OF LABORATORY WATER SOLUBLE SULFATE TEST RESULTS
CALIFORNIA TEST NO. 417

Sample No.	Water Soluble Sulfate (% SO_4)	Sulfate Exposure*
B4 @ 10-15'	0.097	Negligible
B8 @ 40-45'	0.000	Negligible

* Reference: 2013 California Building Code, Section 1904.3 and ACI 318-11 Section 4.3.

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CORROSIVITY TEST RESULTS

REYLENN PROPERTIES, LLC
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TORRANCE, CA 90505

MARCH 2016
PROJECT NO. A9201-06-01C
FIG. B15

APPENDIX C

APPENDIX C

PRIOR EXPLORATION LOGS

GEOTECHNICAL BORING LOG

SHEET 1 OF 3

PROJECT NO. 102568
DATE STARTED 2/9/05
DATE FINISHED 2/10/05
DRILLER J & N Drilling
TYPE OF DRILL RIG 30" Bucket Auger

PROJECT NAME Butcher Hill
GROUND ELEV. 444
GW DEPTH (FT)
DRIVE WT. See Note
DROP 12 inches

BORING DESIG. B-101
LOGGED BY CRN
NOTE 0-24' 3548#; 24-47' 2577#; 47-73' 1648#; 73-90' 810#; 90-110' 1660

DEPTH (feet)	ELEV.	SAMPLE TYPE	BLOWS/FT	LITHOLOGY	ATTITUDES	GEOTECHNICAL DESCRIPTION	MOISTURE CONT. (%)	DRY (pcf) DENSITY	SAT-URATION (%)	OTHER TESTS
						MONTEREY FORMATION (Tm) Silty Sandstone; very fine- to fine-grained, yellowish brown, dry to moist, moderately hard, abundant rootlets top 1±. foot.				
						Diatomaceous Siltstone; pale yellowish brown to yellowish gray, dry, moderately hard to hard, abundant shells (clam, turritella), some rounded to subrounded pebbles, cobbles and boulders (up to 1± ft.) of vesicular basalt, vague bedding - slightly laminated, generally massive.				
	440									
5										
	435									
10					B: N65W, 54SW		60.9	37.5	47	
		R/B	8 P/R 4"		J: N50W, 63NE	Diatomaceous Claystone; very pale grayish orange, dry, moderately hard to hard, well bedded - laminated.				
	430									
15										
					F: N10E, 78SE	Fault: 1± ft. crush zone, highly jumbled fragments of Siltstone and Claystone, abundant white carbonate, soft.				
	425									
20		R/B	2 P/R 5"				60.6	38.2		
					B: N20E, 56SE					
	420									
25										
					B: N40W, 47NE	Silicified Shale lens; 3± inches thick, grayish orange, dry, very hard, slightly fractured/jointed. Diatomaceous Claystone; grayish orange, dry, moderately hard, well bedded - laminated.				
	415									
30		R	2				99.8	56.1	136	
	410									
35		R	8 for 10" P/R 4"		B: N10E, 75SE J: N85E, 72NW	Silicified Shale lens, 1-1/2± ft. thick, highly jointed/fractured	6.3	106.3	30	
						Diatomaceous Siltstone; grayish orange, dry, moderately hard, poorly bedded - slightly laminated, generally massive.				
	405									

SAMPLE TYPES:

- R RING (DRIVE) SAMPLE
- S SPT (SPLIT SPOON) SAMPLE
- B BULK SAMPLE
- T TUBE SAMPLE

- ▼ GROUNDWATER LEVEL
- ► WATER SEEP
- B BEDDING
- J JOINTING
- C CONTACT
- F FAULT
- S SHEAR

PACIFIC SOILS ENGINEERING, INC.

GEOTECHNICAL BORING LOG

SHEET 2 OF 3

PROJECT NO. 102568
DATE STARTED 2/9/05
DATE FINISHED 2/10/05
DRILLER J & N Drilling
TYPE OF DRILL RIG 30" Bucket Auger

PROJECT NAME Butcher Hill
GROUND ELEV. 444
GW DEPTH (FT)
DRIVE WT. See Note
DROP 12 inches

BORING DESIG. B-101
LOGGED BY CRN
NOTE 0-24' 3548#; 24-47' 2577#; 47-73' 1648#; 73-90' 810#; 90-110' 1660

DEPTH (feet)	ELEV.	SAMPLE TYPE	BLOWS/FT	LITHOLOGY	ATTITUDES	GEOTECHNICAL DESCRIPTION	MOISTURE CONT. (%)	DRY (pcf) DENSITY	SAT-URATION (%)	OTHER TESTS
		R/B	3		B: (Hanging Wall) N10E, 74SE F: N10W, 70SW B: (Footwall) N-S, 68E	Normal Fault: 6 to 8± inch crush zone, offsets Claystone 2± inches, soft.	53.1	39.2	44	
	400									
45		R	2 P/R 4"				7.0	95.3	25	
					F: N10W Vertical to 75NE	Fault: 8 to 10± inch crush zone/gouge, black staining along fault plane, undulatory.				
	395									
						Diatomaceous Claystone; grayish orange, dry, moderately hard, poorly to well bedded - laminated, some subrounded pebbles of well cemented Siltstone.				
50		R/B	6				37.5	60.0	57	
					B: N20W, 63NE	Silicified Shale layer; 6 to 8± inches thick, very hard, highly jointed/fractured.				
	390									
55		R	4				44.6	39.1	37	
					B: N20W, 67NE					
	385									
60		R/B	6				43.7	46.8	46	
					B: N30W, 59NE					
	380									
65		R	7				40.8	42.6	38	
					B: N15W, 76NE					
					B: N30W, 70NE	Silicified Shale layer; 4 to 6± inches thick, very hard, highly fractured/jointed.				
	375									
70		R/B	7				45.2	41.4	40	
						Silicified Shale layer;1± foot thick, very hard, moderately fractured/jointed.				
	370				B: N10W, 76NE					
75		R	30 for 6" P/R 2"				3.6	91.2	12	
					B: N-S, 73E					
	365									

SAMPLE TYPES:
[R] RING (DRIVE) SAMPLE
[S] SPT (SPLIT SPOON) SAMPLE
[B] BULK SAMPLE [T] TUBE SAMPLE

▼ GROUNDWATER LEVEL
➤ WATER SEEP
[B] BEDDING
[J] JOINTING
[C] CONTACT
[F] FAULT
[S] SHEAR

PACIFIC SOILS ENGINEERING, INC.

PLATE A-2

GEOTECHNICAL BORING LOG

SHEET 3 OF 3

PROJECT NO. 102568
DATE STARTED 2/9/05
DATE FINISHED 2/10/05
DRILLER J & N Drilling
TYPE OF DRILL RIG 30" Bucket Auger

PROJECT NAME Butcher Hill
GROUND ELEV. 444
GW DEPTH (FT)
DRIVE WT. See Note
DROP 12 inches

BORING DESIG. B-101
LOGGED BY CRN
NOTE 0-24' 3548#; 24-47' 2577#; 47-73' 1648#; 73-90' 810#; 90-110' 1660

DEPTH (feet)	ELEV.	SAMPLE TYPE	BLOWS/FT	LITHOLOGY	ATTITUDES	GEOTECHNICAL DESCRIPTION	MOISTURE CONT. (%)	DRY (pcf) DENSITY	SAT-URATION (%)	OTHER TESTS
		R/B	35 for 10" P/R 5*		B: N15W, 63NE	Silicified Shale layer; 1± foot thick, very hard, moderately fractured/jointed.	35.0	52.6	43	
	360									
85		R	31 for 6" P/R 5*				39.9	52.2	49	
					B: N10W, 66NE	Interbedded Claystone with Diatoms; grayish orange, dry, moderately hard, distinct bedding, 1± foot thick.				
					B: N10W, 58NE	Basalt layer; 6± inches thick, dark gray to black, very hard, moderately fractured, baked lower contact 2 to 3± inches thick.				
	355									
90		R	40 for 6" P/R 4*			Added 20-foot stem.	34.0	45.3	34	
					B: N30W, 65NE	Sandy Claystone with Diatoms; very fine- to fine-grained, light yellowish orange, dry, moderately hard, poorly to well bedded - laminated to massive.				
	350									
95		R/B	25				39.7	54.2	51	
					B: N15W, 59NE					
					B: N-S, 52NE	Diatomaceous Claystone; grayish orange, dry, moderately hard, poorly to well bedded - laminated to massive, some subrounded pebbles of well cemented Siltstone.				
	345									
100		R	28				38.5	40.1	33	
					B: N25W, 63NE					
	340									
105		R/B	30 for 7" P/R 5*				22.7	66.4	40	
					B: N20W, 64NE					
	335									
110		R	30 for 6" P/R 4*				36.2	39.4	30	
						Total Depth 111 feet. No water, No caving. Hole backfilled with cuttings and tamped.				
						*P/R 4 = Partial ring sample recovery and number of rings recovered.				

SAMPLE TYPES:
- R RING (DRIVE) SAMPLE
- S SPT (SPLIT SPOON) SAMPLE
- B BULK SAMPLE
- T TUBE SAMPLE

- GROUNDWATER LEVEL
- WATER SEEP
- B BEDDING
- J JOINTING
- C CONTACT
- F FAULT
- S SHEAR

PACIFIC SOILS ENGINEERING, INC.

GEOTECHNICAL BORING LOG

SHEET 1 OF 1

PROJECT NO. 102568
DATE STARTED 4/4/05
DATE FINISHED 4/5/05
DRILLER D & D Construction
TYPE OF DRILL RIG 24" Bucket

PROJECT NAME Butcher Hill
GROUND ELEV. 349
GW DEPTH (FT)
DRIVE WT.
DROP

BORING DESIG. B-102
LOGGED BY CRN
NOTE Track Mounted

DEPTH (feet)	ELEV.	SAMPLE TYPE	BLOWS/FT	LITHOLOGY	ATTITUDES	GEOTECHNICAL DESCRIPTION	MOISTURE CONT. (%)	DRY (pcf) DENSITY	SAT-URATION (%)	OTHER TESTS
						SOIL: Sandy Silt to Silty Sand; very fine- to fine-grained, dark yellowish brown, slightly moist, loose, abundant rootlets.				
	345					**MONTEREY FORMATION (Tm)**Conglomerate; matrix of Sandy Claystone to Clayey Sandstone, very fine- to fine-grained, olive gray, slightly moist, soft/loose, weathered.				
5						Silicified shale concretion, pale gray, laminated, very hard, up to 1-1/2 foot with some shell fragments, generally massive, some vague alignment of concretion pebbles and cobbles.				
	340				B: N20E 23SE					
10										
					AB: B50E 26SE	Cobbles and boulders throughout, refusal.				
	335	B								
15						Total Depth 15 feet, Refusal. No water. Hole backfilled with cuttings and tamped.				

SAMPLE TYPES:
R RING (DRIVE) SAMPLE
S SPT (SPLIT SPOON) SAMPLE
B BULK SAMPLE T TUBE SAMPLE

▼ GROUNDWATER LEVEL
➤ WATER SEEP
B BEDDING
J JOINTING
C CONTACT
F FAULT
S SHEAR

PACIFIC SOILS ENGINEERING, INC.

PLATE A-4

GEOTECHNICAL BORING LOG

SHEET 1 OF 2

PROJECT NO. 102568
DATE STARTED 9/21/05
DATE FINISHED 9/21/05
DRILLER Hillside Repair & Drilling, Inc.
TYPE OF DRILL RIG 24" Crawler Rig

PROJECT NAME Butcher Hill
GROUND ELEV. 285
GW DEPTH (FT)
DRIVE WT.
DROP

BORING DESIG. B-201
LOGGED BY CRN
NOTE

DEPTH (feet)	ELEV.	SAMPLE TYPE	BLOWS/FT	LITHOLOGY	ATTITUDES	GEOTECHNICAL DESCRIPTION	MOISTURE CONT. (%)	DRY (pcf) DENSITY	SAT-URATION (%)	OTHER TESTS
	285					**MONTEREY FORMATION (Tm)** Diatomaceous Siltstone; white to light brownish tan, slightly moist, firm, laminated.				
5	280									
					B: N10E, 44SE					
		B								
10	275									
					B: N30W, 35SW					
15	270									
					B: N30W, 48SW					
20	265									
					B: N35W, 45SW					
		B								
25	260									
					B: N35W, 45SW	1/2 inch thick concretion layer, brown to dark brown, dry, very hard.				
30	255									
					B: N20W, 42SW					
35	250									
					B: N25W, 34SW					
	245									

SAMPLE TYPES:
R RING (DRIVE) SAMPLE
S SPT (SPLIT SPOON) SAMPLE
B BULK SAMPLE
T TUBE SAMPLE

GROUNDWATER LEVEL
WATER SEEP
B BEDDING
J JOINTING
C CONTACT
F FAULT
S SHEAR

PACIFIC SOILS ENGINEERING, INC.

PLATE A-5

GEOTECHNICAL BORING LOG

SHEET 2 OF 2

PROJECT NO. 102568
DATE STARTED 9/21/05
DATE FINISHED 9/21/05
DRILLER Hillside Repair & Drilling, Inc.
TYPE OF DRILL RIG 24" Crawler Rig

PROJECT NAME Butcher Hill
GROUND ELEV. 285
GW DEPTH (FT)
DRIVE WT.
DROP

BORING DESIG. B-201
LOGGED BY CRN
NOTE

DEPTH (feet)	ELEV.	SAMPLE TYPE	BLOWS/FT	LITHOLOGY	ATTITUDES	GEOTECHNICAL DESCRIPTION	MOISTURE CONT. (%)	DRY (pcf) DENSITY	SAT-URATION (%)	OTHER TESTS
	245	B								
					B: N35W, 42SW J: N40W, 50NE					
45	240									
					B: N50W, 55SW F: N60W, 34SW B: N10W, 44SW B: N25W, 36SW	Fault 1/4 inch thick clay gouge, olive gray, slightly moist, firm, flowers into upper strata approx. 6-8 inches.				
50	235									
					B: N25W, 32SW					
		B								
55	230									
						Total Depth 57 feet. No water, No caving. Backfilled with cuttings and tamped.				

SAMPLE TYPES:
R RING (DRIVE) SAMPLE
S SPT (SPLIT SPOON) SAMPLE
B BULK SAMPLE
T TUBE SAMPLE

▼ GROUNDWATER LEVEL
➤ WATER SEEP
B BEDDING
J JOINTING
C CONTACT
F FAULT
S SHEAR

PACIFIC SOILS ENGINEERING, INC.

GEOTECHNICAL BORING LOG

SHEET 1 OF 2

PROJECT NO. 102568
DATE STARTED 9/21/05
DATE FINISHED 9/22/05
DRILLER Hillside Repair & Drilling, Inc.
TYPE OF DRILL RIG 24" Crawler Rig

PROJECT NAME Butcher Hill
GROUND ELEV. 275
GW DEPTH (FT)
DRIVE WT.
DROP

BORING DESIG. B-202
LOGGED BY CRN
NOTE

DEPTH (feet)	ELEV.	SAMPLE TYPE	BLOWS/FT	LITHOLOGY	ATTITUDES	GEOTECHNICAL DESCRIPTION	MOISTURE CONT. (%)	DRY (pcf) DENSITY	SAT-URATION (%)	OTHER TESTS
	275					**MONTEREY FORMATION (Tm)** Diatomaceous Siltstone; white to light brown tan, slightly moist, firm, highly fractured siliceous siltstone in upper 12 feet, becomes laminated, punky siltstone below.				
5	270	B								
10	265									
					B: N40W, 65NE J: N20E, 47SE N80E, 37SE	Abundant white carbonate along joints.				
15	260									
					J: N10E, 80NW N50W, 57NE N60W, 61NE	Highly jointed.				
		B								
20	255									
25	250									
					B; N60W, 80SW	Abundant joints.				
30	245									
					B: N65W, 67NE					
35	240	B			B: N50W, 55SW	Moderate belling from 35 to 37 feet. Highly jointed, abundant white carbonate.				
	235									

SAMPLE TYPES:
R RING (DRIVE) SAMPLE
S SPT (SPLIT SPOON) SAMPLE
B BULK SAMPLE T TUBE SAMPLE

▼ GROUNDWATER LEVEL
WATER SEEP
B BEDDING
J JOINTING
C CONTACT
F FAULT
S SHEAR

PACIFIC SOILS ENGINEERING, INC.

PLATE A-7

GEOTECHNICAL BORING LOG

PROJECT NO. 102568
DATE STARTED 9/21/05
DATE FINISHED 9/22/05
DRILLER Hillside Repair & Drilling, Inc.
TYPE OF DRILL RIG 24" Crawler Rig

PROJECT NAME Butcher Hill
GROUND ELEV. 275
GW DEPTH (FT)
DRIVE WT.
DROP

BORING DESIG. B-202
LOGGED BY CRN
NOTE

DEPTH (feet)	ELEV.	SAMPLE TYPE	BLOWS/FT	LITHOLOGY	ATTITUDES	GEOTECHNICAL DESCRIPTION	MOISTURE CONT. (%)	DRY (pcf) DENSITY	SAT-URATION (%)	OTHER TESTS
	235				B: N30W, 48SW					
					B: N50W, 57SW	Some white carbonates.				
45	230									
					B: N20W, 36SW					
50	225				B: N20W, 63SW					
		B								
55	220									
						Total Depth 57 feet. Severe caving in upper 12 feet on 9/21/05. Cased hole on 9/22/05 and logged to 57 feet. Bottom of boring at 57 feet. No water, minor belling at 35 to 37 feet. Boring backfilled with cuttings and tamped.				

SAMPLE TYPES:
- R RING (DRIVE) SAMPLE
- S SPT (SPLIT SPOON) SAMPLE
- B BULK SAMPLE
- T TUBE SAMPLE

▼ GROUNDWATER LEVEL
◄ WATER SEEP
- B BEDDING
- J JOINTING
- C CONTACT
- F FAULT
- S SHEAR

PACIFIC SOILS ENGINEERING, INC.

PLATE A-8

TABLE II

LOG OF EXPLORATORY PITS

Logged: 12/17/04
Equip: JD455E Trackhoe
Logged By: DO

Log No.	Depth (ft.)	Description
EP-1	0 to 7	FILL (af): Sand, tan brown, slightly moist, loose.
	@ 5	Trash
	7 to 15	SAN PEDRO SAND (Qsp): Sand, tan brown, slightly moist, loose to medium dense.
		Total Depth 15 feet. No groundwater was encountered. No caving observed. Hole backfilled and wheel-rolled in-place. Bulk sample at 5 feet.
EP-2	0 to 9	COLLUVIUM (Qcol): Diatomaceous Silt, light brown, dry, soft, angular clasts to 12 inches in size, caving.
	@ 9	Vegetation
	9 to 10	SAN PEDRO SAND (Qsp): Sand, tan brown, slightly moist, loose to medium dense.
		Total Depth 10 feet. No groundwater was encountered. Caving observed throughout. Hole backfilled and wheel-rolled in-place. Bulk sample at 6 feet.

TABLE II

LOG OF EXPLORATORY PITS

Logged: 12/17/04
Equip: JD455E Trackhoe
Logged By: DO

Log No.	Depth (ft.)	Description
EP-3	0 to 4	FILL (af): Silty Sand, dark brown, slightly moist, medium dense, fine-grained.
	4 to 9	SAN PEDRO SAND (Qsp): Sand, tan brown, slightly moist, loose to medium dense, caving.
		Total Depth 9 feet. No groundwater was encountered. Caving from 4 to 9 feet. Hole backfilled and wheel-rolled in-place.
EP-4	0 to 5	FILL (af): Sand, tan brown, slightly moist, loose, fine-grained, vegetation at 5 feet.
	5 to 11	ALLUVIUM (Qal): Sand, tan brown, slightly moist, loose, fine-grained, caving.
		Total Depth 11 feet. No groundwater was encountered. Caving from 5 to 11 feet. Hole backfilled and wheel-rolled in-place.
EP-5	0 to 10	ALLUVIUM (Qal): Sand, light brown, dry, loose, fine-grained, horizontal stratification, roots.
	10 to 16	SAN PEDRO SAND (Qsp): Sand, tan brown, slightly moist, loose, fine-grained.
		Total Depth 16 feet. No groundwater was encountered. Caving throughout. Hole backfilled and wheel-rolled in-place.

GEOLOGIC LOG - TEST BORING

Project No.: 1299 Date: 8-23-93 Boring No.: 1

Equipment: Bucketauger Boring Dimensions: Dia 2' D 101'

Elevation: 460-plan

Depth, ft.	Description	Unit
0-1	Mod firm damp brn cl silt	Colluvium
1-101	Firm damp gray & tan diatomaceous siltstone lt gr at 39-47' brn at 47-49' Strata 10'-N15W76N 31- N20W72N 36- N15W76S 38- N20W76N 42- N15W75N 48- NS70W 54- N5E70S 60- N5E70N 67- N5E80S 75- N30W86N 86- N75W60S 96- N75E74S Fault 82'-N70W67N Joints 17'-N70W88N 45'-N15W89N No Seepage	Valmonte Mem.

BORING 1

W.O.93-1605
Page 1 of 6

Client: MR. DAN E. BUTCHER
Project: BUTCHER HILL DEVELOPMENT
Date: 8-23-93

Depth ft.	Sample	Lab Test	Blows/ft.		BORING LOG 24" Dia. Bucket Rig w/3350 Lb. Kelly	Dry Unit Weight Lbs/Cu.Ft.	Moisture content PCT/Dry WT.	Notes
0				0.0-34.0'	NATURAL GROUND-Diatomaceous SILTSTONE(WH-WM)-Lt. Brown/White highly to moderately weathered, dry to moist			
	B	EX					57.6	EI=56
5	R		6			58.3	60.2	
10	R		4			66.4	58.1	
15	R	DS	2			60.6	49.2	
20	R		4			60.9	41.8	
25	R		4			46.5	63.5	
30	R		8		2045 Lb. Kelly	48.5	71.4	

Continued on following page

BORING 1

(Continued)

W.O 93-1605
Page 2 of 6

Client: MR. DAN E. BUTCHER
Project: BUTCHER HILL DEVELOPMENT
Date: 8-23-93

Depth ft.	Sample	Lab Test	Blows/ft.	BORING LOG 24" Dia. Bucket Rig w/2045 Lb. Kelly	Dry Unit Weight Lbs/Cu.Ft.	Moisture content PCT/Dry WT.	Notes
35	R	DS	7	34.0-39.0' Diatomaceous SILTSTONE(WM) Lt. Brown/White, moderately weathered moist	40.5	65.7	
40	R		7	39.0-68.0' Diatomaceous SILTSTONE(WM)- White, moderately weathered, moist	40.1	85.5	
45	R		5		49.4	56.0	
50	R		6		43.7	69.8	
55	R	DS	15		42.3	57.6	
60	R		14	1200 Lb. Kelly	50.8	51.6	
55	R		13		64.3	50.5	

Continued on following page

BORING 1

(Continued)

Client: MR. DAN E. BUTCHER
Project: BUTCHER HILL DEVELOPMENT Date: 8-23-93

Depth ft.	Sample	Lab Test	Blows/ft.		BORING LOG	Dry Unit Weight Lbs/Cu.Ft.	Moisture content PCT/Dry WT.	Notes
					24" Dia. Bucket Rig w/1200 Lb. Kelly			
				68.0–100'	Diatomaceous SILTSTONE(WM)– White, moderately weathered, moist			
70	R		20/ 6"			60.3	39.2	
75	R		35/ 11"			59.4	34.1	
80	R	DS	30			84.1	38.8	
85	R		34			63.2	40.3	
90	R		22		2200 Lb. Kelly	45.5	53.7	
95	R		25			51.8	57.7	
100	R		31		Borings Halted & Backfilled No Groundwater Encountered	58.5	37.3	

APPENDIX D

APPENDIX D

ROCK MECHANICS AND ROCKFALL ANALYSIS

Rock Mechanics and Rockfall Simulation Modeling for MKS Residential Torrance, CA, prepared by GeoStabilization International, dated February 23, 2016.

GeoStabilization International®

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Rockfall Division Engineer
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February 23, 2016

Jelisa Thomas, PE
Project Engineer Geocon West, Inc.
3303 N. San Fernando Blvd. Suite 100,
Burbank, CA 91504

Subject: Rock Mechanics and Rockfall Simulation Modeling for MKS Residential Torrrance, CA

Dear Jelisa,

GeoStabilization International was asked to provide rock mechanics and rockfall simulation modeling for a project for MKS Residential located in Torrance, CA. This letter presents the results of the modeling.

Information provided

Geocon West Inc. provided geological structural information, geologic cross-sections, topographic cross-sections, and field observations to conduct the analysis. The scope of the work included rock mechanics modeling based on the geologic structure data and develop factors of safety as well as recommendations for the rock slopes based on the analysis. The focus of this investigation was for slopes that dipped slightly to the east of north at a slope dip of approximately 50^{o}.

Furthermore, there was a concern about rockfall. To analyze this information geologic data and cross-sections were provided by Geocon West Inc. and computer simulations were performed using the Colorado Rockfall Simulation program (CRSP 4.0). Assessments of the hazard are presented as well as mitigation options.

Rock Mass Stability

The slopes in the project area are controlled by its geologic structure. Geologic structure data was provided by Geocon. The first step in this analysis was to analyze the four provided data sets to ascertain if the geology was similar throughout the site or if there were different geologic domains. This was performed using the Dips computer program by Rocscience. The data that was collected from down-hole logging and geologic mapping of surface exposures show similar geologic structure throughout the four data sets. This allows the four data sets to be combined into one stereonet for analysis.

The geologic structure is shown in Figure 1. This shows a predominance of four discontinuity sets throughout the project area. These four discontinuities are provided in the Table 1 below.

For large-scale rock slopes failures they generally occur along the discontinuities in the rock mass in the form of planar, wedge, or toppling failures. Of these three failure mechanisms planar and wedge failures are the most likely, where toppling failures occur generally in specific geologic situations and present themselves generally as rockfalls.

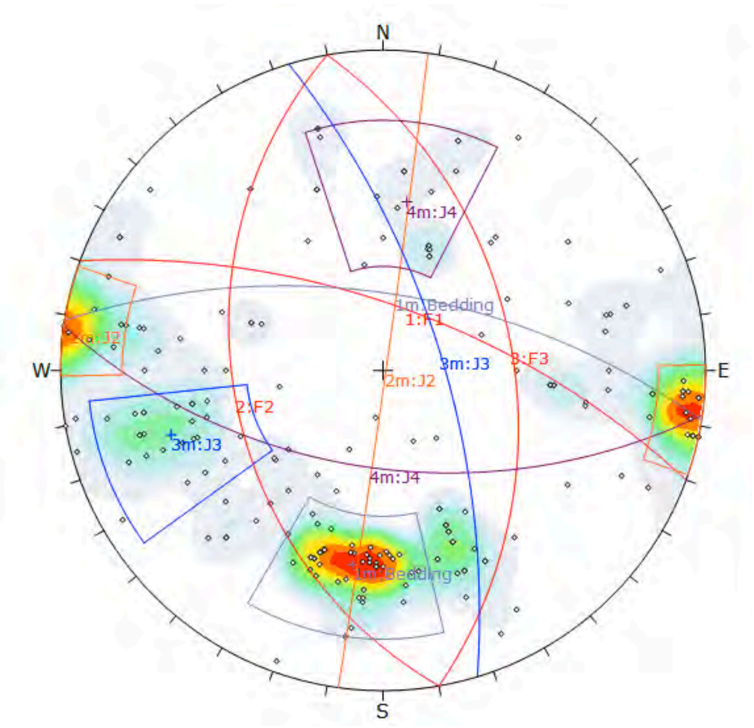


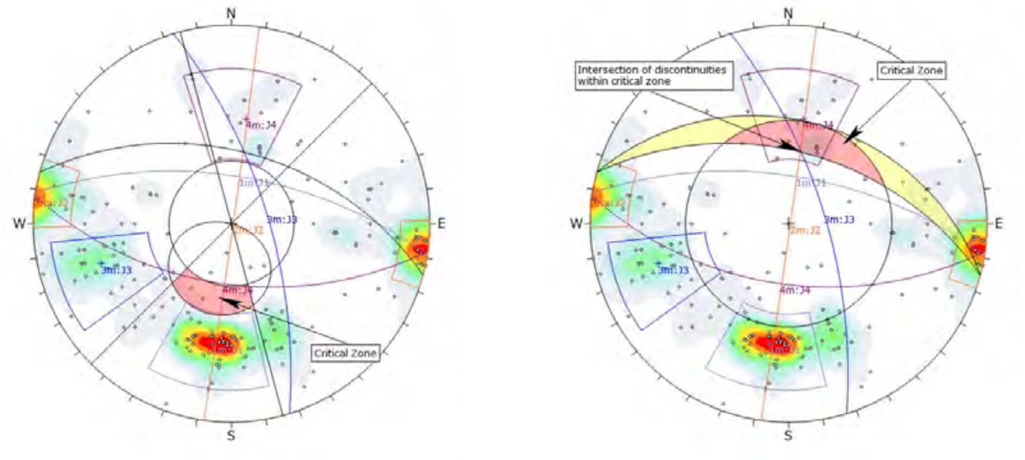
Figure 1. Combined Geologic Data used in the analysis.

Table 1. Summary of discontinuities found in the provided data set.

Discontinuity No.	Name	Avg. Dip	Avg. Dip Direction
1	J1	63	009
2	J2	89	098
3	J3	69	073
4	J4	56	188

To analyze the data to identify the potential modes of failure a kinematic analysis was performed using the Dips computer software. Figures 2 and 3 show the planar and wedge analyses that trend in the northeast direction (015 compass heading). In Figure 2 (planar sliding analysis) there is a pink area that is identified on the lower half of the stereonet. In this pole plot this is referred to as the “critical zone”. If poles of the discontinuities occur in this zone then they have the potential of “daylighting” and should be analyzed for stability. It should be noted that the trend of the rock slope extends from compass direction 008 to 022. Planar stability uses a lateral limit of +/- 20^{o} to define the critical zone. For this analysis to account for this variation in the slope stability the direction of the slope was assume to be in the 015 direction with an increased lateral limit to 30^{o}.

As shown in Figure 2 the closest discontinuity set to the critical zone is Discontinuity J1. However, this joint set dips (average 63^{o}) steeper than the angle for the slope (generally 50^{o}) and therefore, generally locked into place. There are two individual discontinuities that fall within this zone. Since they fall outside the normal trend of a discontinuity set their prevalence and effect on global stability is generally considered less. It is also less likely that, unless these discontinuities are identified in the field as a structure such as a fault, they will result in a large failure. In that light a stability analysis was conducted to analyze stability along these discontinuities.

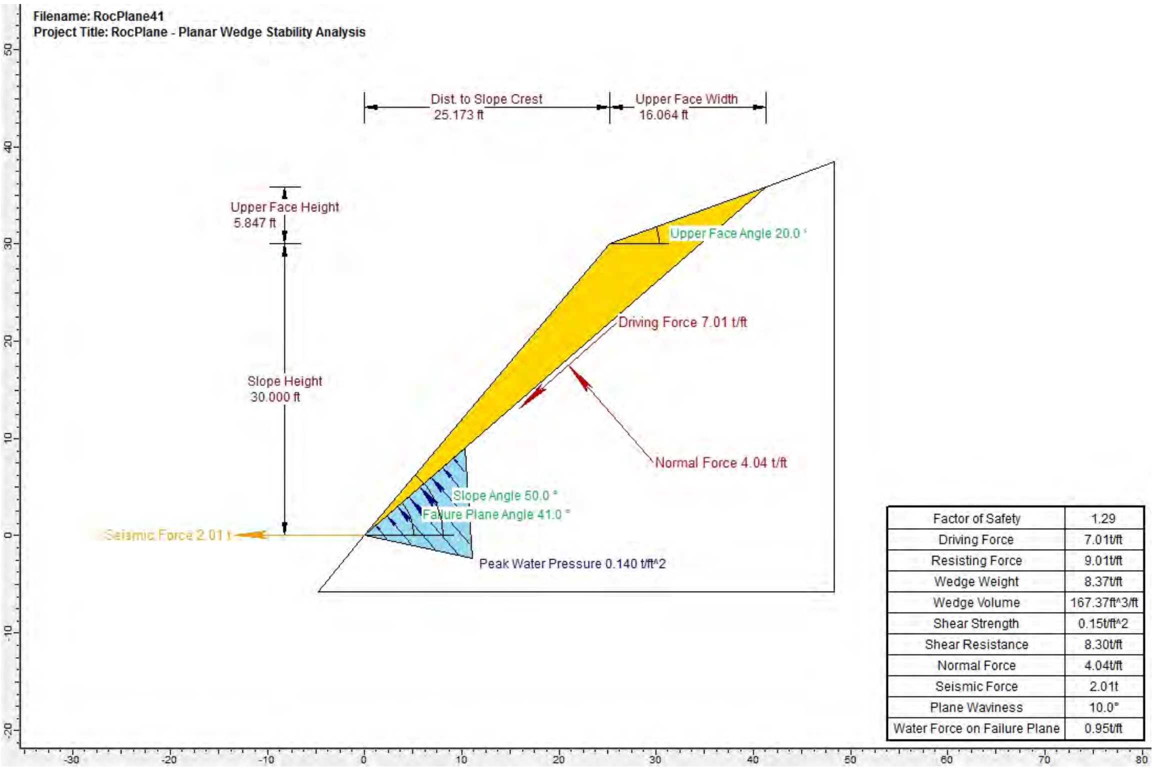


Figures 2 and 3. Figure 2 (left) pole plot stereonet showing kinematic analysis for planar failures, and Figure 3 (right) pole plot stereonet showing kinematic analysis for wedge failures.

This analysis was performed using the RocPlane computer program by Rocscience and is shown in Figure 4. The shear strength for the discontinuity is assumed to have a friction angle of 35 degrees and a cohesion value of 200 psf, based on test data provided by Geocon. Analysis included dry conditions, 25% saturated, and pseudo-static loads with 25% saturated conditions. Pseudo-static loads were provided by Geocon. It should be noted that in rock slope stability using 25% of the discontinuity filled with water to model saturated conditions more appropriately models conditions with some exceptions. These exceptions include very wet environments and closed discontinuities that don't allow the flow of water (e.g. ice covered slopes). The results of the planar analysis is provided below in Table 2.

Table 2. Summary of Planar Analysis

Condition	Safety Factor
Static- Dry Conditions	2
Static- 25% Saturated Conditions	1.8
Pseudo-Static and 25% Saturated	1.3



Filename: RocPlane41
Project Title: RocPlane - Planar Wedge Stability Analysis

Factor of Safety	1.29
Driving Force	7.01t/ft
Resisting Force	9.01t/ft
Wedge Weight	8.37t/ft
Wedge Volume	167.37ft^3/ft
Shear Strength	0.15t/ft^2
Shear Resistance	8.30t/ft
Normal Force	4.04t/ft
Seismic Force	2.01t
Plane Waviness	10.0°
Water Force on Failure Plane	0.95t/ft

Figure 4. Pseudo-static equilibrium analysis for identified potential planar mode of failure.

Planar analysis conclusions

Based on the geologic structure data the general trend of the main discontinuity is dipping steeper than the slope face and therefore, does not "daylight from the slope". There may be some isolated structures that dip more gently than the slope face and should be examined by a professional engineer or geologist to assess their individual stability. It is unlikely that these identified discontinuities will result in large-scale failure, but do present themselves as likely rockfalls that are discussed later in this letter.

Wedge Failure Analysis

In a similar manner to the planar failures, wedge failures (or failures along two or more discontinuities) can be identified using the stereonet. The Dips computer program was also used to perform this task. Figure 3 shows the kinematic analysis to identify wedge failures. In this stereonet one is looking for the intersection of great circles representing the discontinuity sets. If the intersection falls into the pink or yellow areas there is a kinematic potential for failure which should be examined. Looking in the north direction (015) the only intersection falling near the critical zone is the intersection of J2 and J3 which has a trend of 010 and plunge of 49 degrees. Since this was the only intersection to fall within the zone this was analyzed for stability. It should be noted however, that the average plunge is near the average slope angle that limits its ability to fail.

To analyze this potential wedge the SWedge computer program by Rocscience was utilized and is shown in Figure 5. As with the planar failure dry conditions, 25% saturated, pseudo-static with 25% saturated was analyzed. The results are provided in Table 3 below.

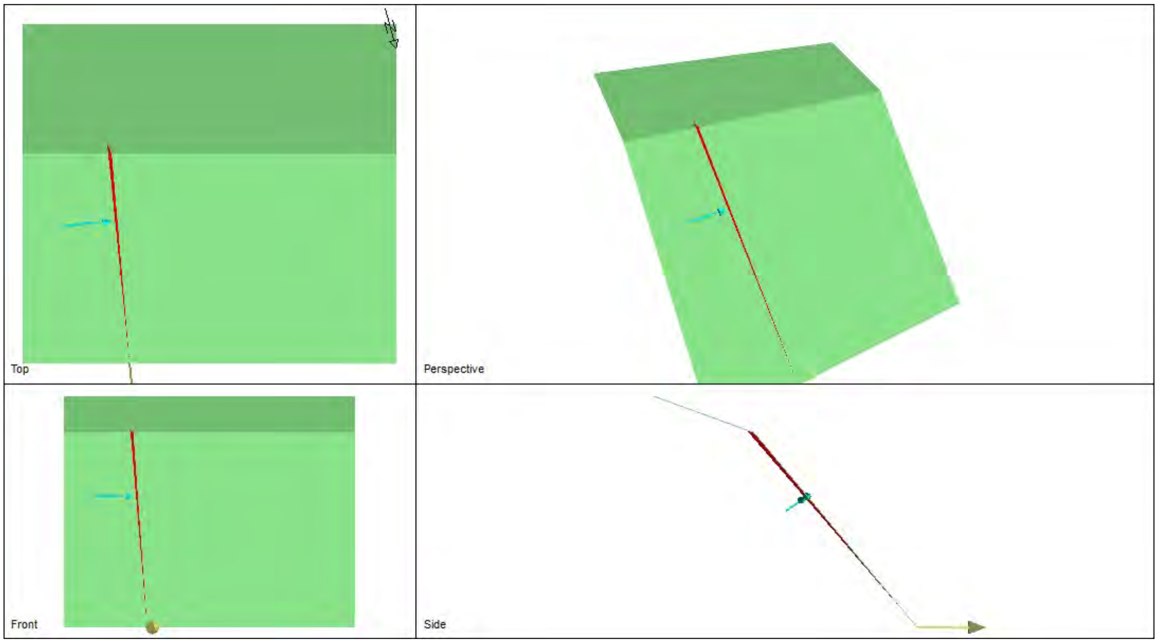


Figure 5. Wedge failure analysis identified in the stereonets. Note the limited geometry for this 200-foot tall slope.

Table 3. Summary of Wedge Analysis

Condition	Safety Factor
Static- Dry Conditions	+8
Static- 25% Saturated Conditions	8
Pseudo-Static and 25% Saturated	6

Wedge analysis conclusions

Similarly to the planar failures the potential wedge failure results in a very narrow, high angle wedge. Its geometry lends itself to smaller failures and presents itself more of a rockfall generator than a global failure.

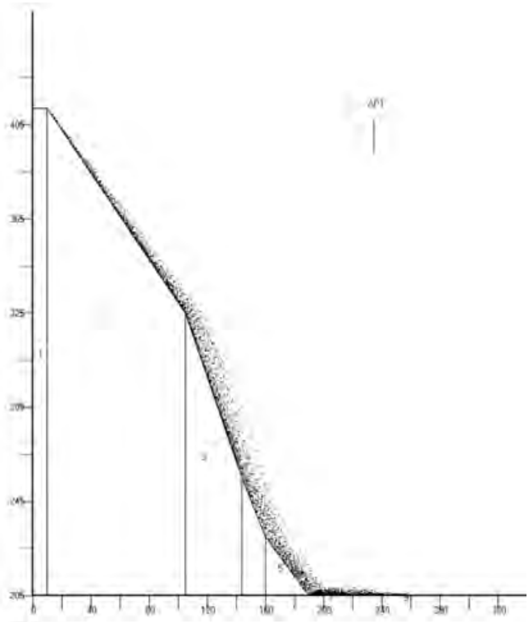
General comments

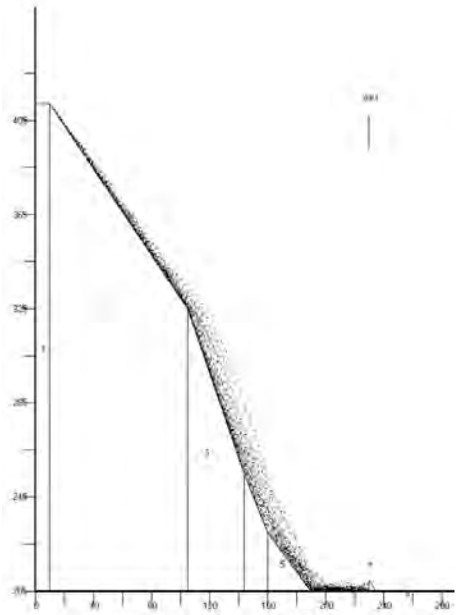
Geologic structure can change over a relatively short area. It is recommended that a professional engineer or professional geologist inspect the rock slope for larger blocks that have the potential for release since rockfall is the more likely mode of failure. If blocks are identified they can be mitigated by removal or minor amounts of reinforcement.

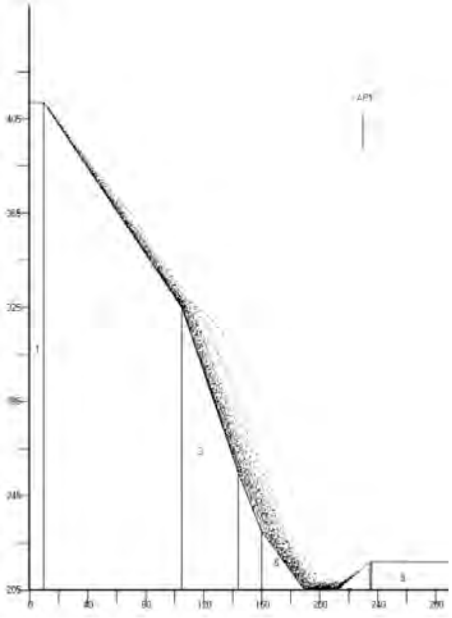
Rockfall simulation

A site civil plan was provided to GSI by Geocon. To conduct the rockfall simulation modeling the Colorado Rockfall Simulation Program (CRSP 4.0) was utilized. Cross-sections for the project were initially configured into the program and calibrated using the Oregon Catchment area guide (FHWA-OR-RD-02-04). This calibration was done on rockfalls generated at a height of 80-100 feet. After calibration rockfalls were generated from the full height of the slope.

For the majority of the project area structures are generally found to be at least 40 feet from the toe of the slope, an area that can be considered a rockfall catchment area. Three scenarios were modeled for rockfall simulation and include a flat catchment area, barrier placed 40 feet from the slope for a flat catchment area, and the construction of a rockfall berm. Results can be seen in Figures 6, 7, and 8.







Figures 6. Upper left figure showing results of the flat ditch rockfall simulation model.

Figure 7. Upper right figure showing results of a barrier placed 40 feet from toe of the slope.

Figure 8. Left figure showing results of berm placed 40 feet from toe of the slope.

The flat catchment area indicates the longest run out of rockfalls of all models presented (Figure 6). Based on the model rockfalls have the extreme potential of running out up to 70 feet. This is slightly greater than the expected rockfall run outs shown on the FHWA catchment guide due to the height of the slope.

Rockfall trajectories for on slopes that generally grade just steeper than a 1H: 1V grade, such as the subject slopes, the majority of rockfall trajectories are expected to be rolling with slight bouncing (Figure 9). Therefore, when a barrier is placed in the pathway of a rockfall the bounce heights are expected to be relatively low. A barrier was placed 40 feet from the toe of the slope and contained all the potential rockfalls (Figure 7). Based on the model the maximum bounce height at this location is 3 feet with kinetic energies of approximately 20 kJ. These are both relatively low values. In this case a barrier such as a jersey type barrier would suffice. This model can be seen in Figure 8.

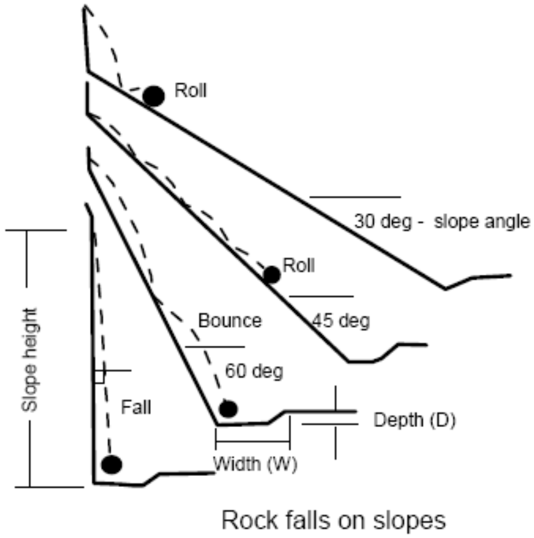
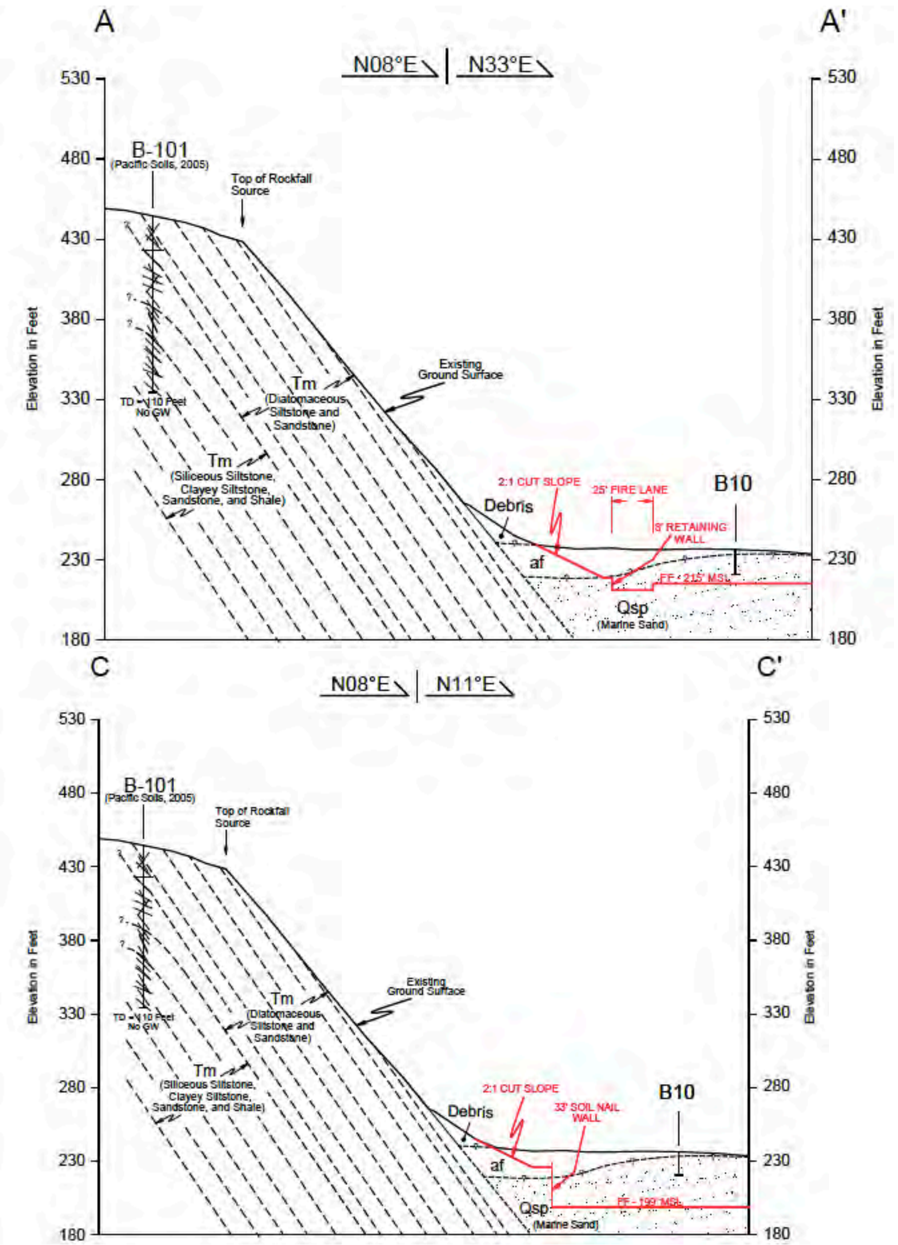


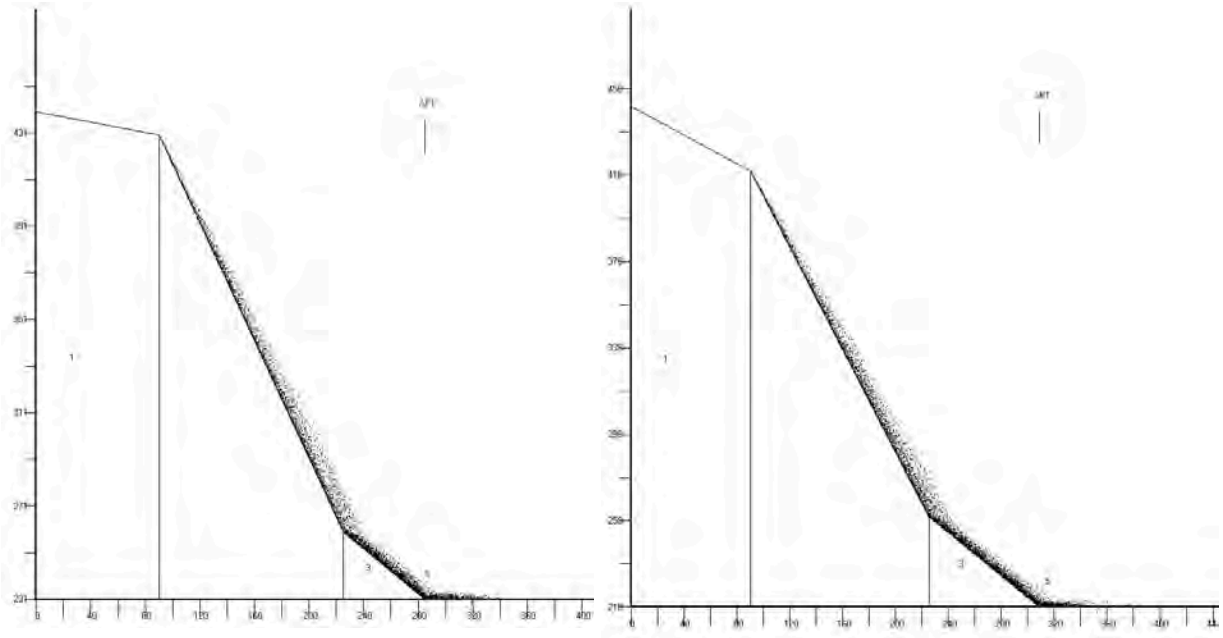
Figure 9. Illustration showing how rockfalls behave differently on different angled slopes.

To model a berm constructed at the toe of the slope a 12 foot high 2H:1V barrier was modeled with the maximum height of the catchment area located 40 feet from the toe of the slope. This model shows the berm satisfactorily contains all the rockfalls as well. This model can be seen in Figure 8.

There are two identified areas where structures are closer to the toe of the slope than for the typical areas. These two areas area identified by cross-sections named A-A’ and C-C’. These two cross-sections are shown in Figures 10 and 11. The results of the analysis shows that a significant amount of horizontal roll out should be expected in the location of the proposed soil nail wall. Results are shown in Figures 12 and 13.



Figures 10 and 11. Cross-provided to develop the CRSP rockfall simulations.



Figures 12 and 13. CRSP analyses for cross-sections A-A’ and C-C’.

The result of the analysis shows that down the constructed 2H: 1V slope bouncing and rolling of rocks can be expected, if generated. The bounce heights for both slopes ranged from 6-9 feet in height. It is recommended that for these areas a rockfall containment barrier be constructed to contain these rocks near the toe of the 2H: 1V slope. Options for this barrier include a rockfall barrier fence or a GCS wall barrier. A rockfall barrier fence is a flexible system that can be built at varying heights, but should be constructed at a height to contain all rockfalls. Since the energies of the rockfalls in the model indicate energies in the order of 20 kJ, many market fences can be used. The appearance of these fences is that it is a fence. Another aesthetic option would be to construct a barrier such as a stand along GCS wall. These can also be constructed at varying heights and since they have a considerably higher energy absorbing capacity would function very well. Facing of the GCS wall can also be varied for aesthetic purposes.

The performed analysis will be provided in an attached document. If you have any questions concerning this analysis feel free to contact me at marty@gsi.us or (540) 315-0270.

Sincerely,

Martin J. Woodard, PhD PG PE
Rockfall Division Engineer
GeoStabilization International
marty@gsi.us

Data Table for Rockfall Analysis

Data Table for Rock Fall Analysis

Section	Exploration Number	Rock Type	Discontinuity Type	Depth (ft)	Dip Direction (asmouth)	Dip Angle (degrees)
A	B-1 (Western Laboratories, 1993)	Diatomaceous Siltstone	Bedding	10	75	76
A	B-1 (Western Laboratories, 1993)	Diatomaceous Siltstone	Joint	17	20	88
A	B-1 (Western Laboratories, 1993)	Diatomaceous Siltstone	Bedding	31	70	72
A	B-1 (Western Laboratories, 1993)	Diatomaceous Siltstone	Bedding	36	255	76
A	B-1 (Western Laboratories, 1993)	Diatomaceous Siltstone	Bedding	38	70	76
A	B-1 (Western Laboratories, 1993)	Diatomaceous Siltstone	Bedding	42	75	75
A	B-1 (Western Laboratories, 1993)	Diatomaceous Siltstone	Joint	45	75	89
A	B-1 (Western Laboratories, 1993)	Diatomaceous Siltstone	Bedding	48	270	70
A	B-1 (Western Laboratories, 1993)	Diatomaceous Siltstone	Bedding	54	95	70
A	B-1 (Western Laboratories, 1993)	Diatomaceous Siltstone	Bedding	60	275	70
A	B-1 (Western Laboratories, 1993)	Diatomaceous Siltstone	Bedding	67	95	80
A	B-1 (Western Laboratories, 1993)	Diatomaceous Siltstone	Bedding	75	60	86
A	B-1 (Western Laboratories, 1993)	Diatomaceous Siltstone	Fault	82	20	67
A	B-1 (Western Laboratories, 1993)	Diatomaceous Siltstone	Bedding	86	195	60
A	B-1 (Western Laboratories, 1993)	Diatomaceous Siltstone	Bedding	96	165	74
A	Area 8 (Geocon West, 2016)	Cg-Fg Sand to Sandy Silt	Bedding	S	359	59
A	Area 8 (Geocon West, 2016)	Cg-Fg Sand to Sandy Silt	Bedding	S	340	68
A	Area 8 (Geocon West, 2016)	Cg-Fg Sand to Sandy Silt	Bedding	S	338	57
A	Area 8 (Geocon West, 2016)	Cg-Fg Sand to Sandy Silt	Bedding	S	338	57
A	Area 8 (Geocon West, 2016)	Cg-Fg Sand to Sandy Silt	Bedding	S	333	64
A	Area 8 (Geocon West, 2016)	Cg-Fg Sand to Sandy Silt	Bedding	S	358	60
A	Area 8 (Geocon West, 2016)	Cg-Fg Sand to Sandy Silt	Bedding	S	200	67
A	Area 8 (Geocon West, 2016)	Cg-Fg Sand to Sandy Silt	Bedding	S	355	60
A	Area 8 (Geocon West, 2016)	Cg-Fg Sand to Sandy Silt	Joint	S	331	81
A	Area 8 (Geocon West, 2016)	Cg-Fg Sand to Sandy Silt	Joint	S	302	68
A	Area 8 (Geocon West, 2016)	Cg-Fg Sand to Sandy Silt	Joint	S	294	78
A	Area 8 (Geocon West, 2016)	Cg-Fg Sand to Sandy Silt	Joint	S	111	44
A	Area 8 (Geocon West, 2016)	Cg-Fg Sand to Sandy Silt	Joint	S	98	90
A	Area 9 (Geocon West, 2016)	Diatomaceous Sandstone	Bedding	S	8	80
A	Area 9 (Geocon West, 2016)	Diatomaceous Sandstone	Bedding	S	5	72
A	Area 9 (Geocon West, 2016)	Diatomaceous Sandstone	Bedding	S	10	58
A	Area 9 (Geocon West, 2016)	Diatomaceous Sandstone	Bedding	S	6	71
A	Area 9 (Geocon West, 2016)	Diatomaceous Sandstone	Bedding	S	9	60
A	Area 9 (Geocon West, 2016)	Diatomaceous Sandstone	Bedding	S	10	60
A	Area 9 (Geocon West, 2016)	Diatomaceous Sandstone	Bedding	S	0	63
A	Area 9 (Geocon West, 2016)	Diatomaceous Sandstone	Bedding	S	10	67
A	Area 9 (Geocon West, 2016)	Diatomaceous Sandstone	Bedding	S	18	61
A	Area 9 (Geocon West, 2016)	Diatomaceous Sandstone	Bedding	S	20	64
A	Area 9 (Geocon West, 2016)	Diatomaceous Sandstone	Bedding	S	6	62
A	Area 9 (Geocon West, 2016)	Diatomaceous Sandstone	Joint	S	274	87
A	Area 9 (Geocon West, 2016)	Diatomaceous Sandstone	Joint	S	275	86
A	Area 9 (Geocon West, 2016)	Diatomaceous Sandstone	Joint	S	272	86
A	Area 9 (Geocon West, 2016)	Diatomaceous Sandstone	Joint	S	279	88
A	Area 9 (Geocon West, 2016)	Diatomaceous Sandstone	Joint	S	200	45
A	Area 9 (Geocon West, 2016)	Diatomaceous Sandstone	Joint	S	202	42
A	Area 9 (Geocon West, 2016)	Diatomaceous Sandstone	Joint	S	235	70
A	Area 9 (Geocon West, 2016)	Diatomaceous Sandstone	Joint	S	99	65
A	Area 9 (Geocon West, 2016)	Diatomaceous Sandstone	Joint	S	275	80
A	Area 10 (Geocon West, 2016)	Diatomaceous Sandstone	Bedding	S	332	68
A	Area 10 (Geocon West, 2016)	Diatomaceous Sandstone	Bedding	S	344	63
A	Area 10 (Geocon West, 2016)	Diatomaceous Sandstone	Joint	S	43	71

Data provided by Geocon for structural analysis

Data Table for Rock Fall Analysis

Section	Exploration Number	Rock Type	Discontinuity Type	Depth (ft)	Dip Direction (azimuth)	Dip Angle (degrees)
B	B-101 (Pacific Soils, 2005)	Diatomaceous Siltstone	Bedding	11	185	54
B	B-101 (Pacific Soils, 2005)	Diatomaceous Claystone	Joint	13	40	63
B	B-101 (Pacific Soils, 2005)	Diatomaceous Claystone	Fault	16	100	78
B	B-101 (Pacific Soils, 2005)	Diatomaceous Claystone	Bedding	21	110	56
B	B-101 (Pacific Soils, 2005)	Silicified Shale Lense	Bedding	27	50	47
B	B-101 (Pacific Soils, 2005)	Silicified Shale Lense	Bedding	35	100	75
B	B-101 (Pacific Soils, 2005)	Silicified Shale Lense	Joint	35	355	72
B	B-101 (Pacific Soils, 2005)	Diatomaceous Siltstone	Bedding	40	100	74
B	B-101 (Pacific Soils, 2005)	Diatomaceous Siltstone	Bedding	40	90	58
B	B-101 (Pacific Soils, 2005)	Diatomaceous Siltstone	Fault	40	260	70
B	B-101 (Pacific Soils, 2005)	Diatomaceous Siltstone	Fault	46	80	90-75
B	B-101 (Pacific Soils, 2005)	Silicified Shale Layer	Bedding	52	70	63
B	B-101 (Pacific Soils, 2005)	Silicified Shale Layer	Bedding	58	70	67
B	B-101 (Pacific Soils, 2005)	Silicified Shale Layer	Bedding	62	60	59
B	B-101 (Pacific Soils, 2005)	Silicified Shale Layer	Bedding	66	75	76
B	B-101 (Pacific Soils, 2005)	Silicified Shale Layer	Bedding	67	60	70
B	B-101 (Pacific Soils, 2005)	Silicified Shale Layer	Bedding	74	80	76
B	B-101 (Pacific Soils, 2005)	Silicified Shale Layer	Bedding	77	90	73
B	B-101 (Pacific Soils, 2005)	Silicified Shale Layer	Bedding	81	75	63
B	B-101 (Pacific Soils, 2005)	nterbedded Claystone w. Diatomit	Bedding	86	80	66
B	B-101 (Pacific Soils, 2005)	Basalt Layer	Bedding	87	80	58
B	B-101 (Pacific Soils, 2005)	Sandy Claystone w. Diatoms	Bedding	93	60	65
B	B-101 (Pacific Soils, 2005)	Sandy Claystone w. Diatoms	Bedding	96	75	59
B	B-101 (Pacific Soils, 2005)	Diatomaceous Claystone	Bedding	98	90	52
B	B-101 (Pacific Soils, 2005)	Diatomaceous Claystone	Bedding	103	65	52
B	B-101 (Pacific Soils, 2005)	Diatomaceous Claystone	Bedding	108	70	64
B	Area 6 (Geocon West, 2016)	Fossiliferous Sandstone	Joint	S	117	85
B	Area 6 (Geocon West, 2016)	Fossiliferous Sandstone	Joint	S	70	80
B	Area 6 (Geocon West, 2016)	Fossiliferous Sandstone	Joint	S	97	89
B	Area 6 (Geocon West, 2016)	Cherty Sandstone	Joint	S	180	45
B	Area 6 (Geocon West, 2016)	Cherty Sandstone	Joint	S	198	74
B	Area 6 (Geocon West, 2016)	Cherty Sandstone	Joint	S	186	64
B	Area 6 (Geocon West, 2016)	Cherty Sandstone	Joint	S	165	76
B	Area 6 (Geocon West, 2016)	Cherty Sandstone	Joint	S	275	56
B	Area 6 (Geocon West, 2016)	Cherty Sandstone	Joint	S	275	55
B	Area 6 (Geocon West, 2016)	Cherty Sandstone	Joint	S	294	84
B	Area 7 (Geoncon West, 2016)	Diatomaceous Sandstone	Bedding	S	338	68
B	Area 7 (Geoncon West, 2016)	Diatomaceous Sandstone	Bedding	S	338	50
B	Area 7 (Geoncon West, 2016)	Diatomaceous Sandstone	Bedding	S	338	55
B	Area 7 (Geoncon West, 2016)	Diatomaceous Sandstone	Bedding	S	4	62
B	Area 7 (Geoncon West, 2016)	Diatomaceous Sandstone	Bedding	S	2	63
B	Area 7 (Geoncon West, 2016)	Diatomaceous Sandstone	Bedding	S	20	64
B	Area 7 (Geoncon West, 2016)	Diatomaceous Sandstone	Bedding	S	18	61
B	Area 7 (Geoncon West, 2016)	Diatomaceous Sandstone	Bedding	S	19	62
B	Area 7 (Geoncon West, 2016)	Diatomaceous Sandstone	Bedding	S	338	60
B	Area 7 (Geoncon West, 2016)	Diatomaceous Sandstone	Bedding	S	21	64
B	Area 7 (Geoncon West, 2016)	Diatomaceous Sandstone	Joint	S	160	62
B	Area 7 (Geoncon West, 2016)	Diatomaceous Sandstone	Joint	S	80	74
B	Area 7 (Geoncon West, 2016)	Diatomaceous Sandstone	Joint	S	282	90
B	Area 7 (Geoncon West, 2016)	Diatomaceous Sandstone	Joint	S	274	86
B	Area 7 (Geoncon West, 2016)	Diatomaceous Sandstone	Joint	S	278	87
B	Area 7 (Geoncon West, 2016)	Diatomaceous Sandstone	Joint	S	80	62
B	Area 7 (Geoncon West, 2016)	Diatomaceous Sandstone	Joint	S	282	89
B	Area 7 (Geoncon West, 2016)	Diatomaceous Sandstone	Joint	S	275	89

Data Table for Rock Fall Analysis

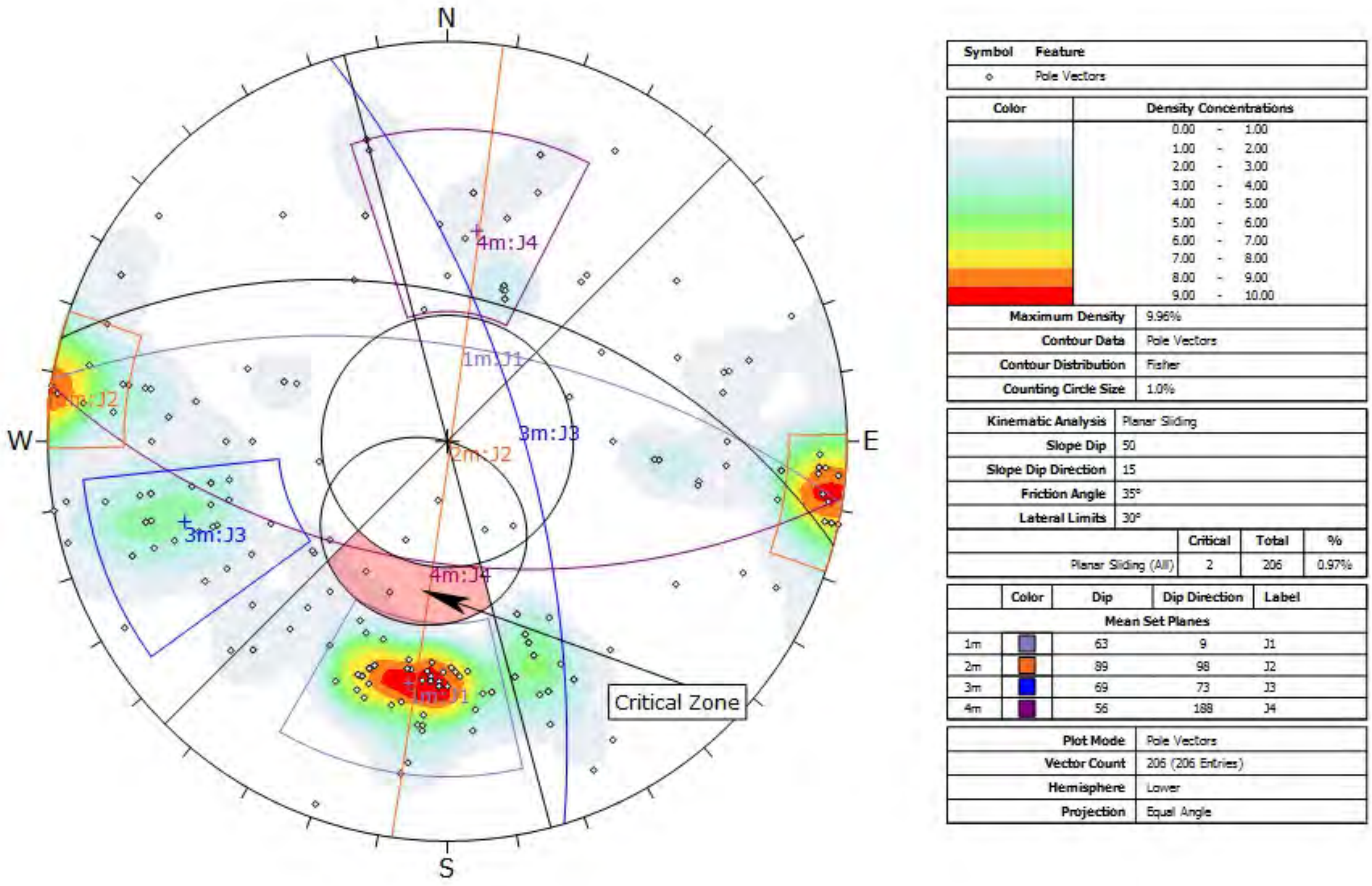
Section	Exploration Number	Rock Type	Discontinuity Type	Depth (ft)	Dip Direction (asmouth)	Dip Angle (degrees)
G	B-202 (Pacific Soils, 2005)	Diatomaceous Siltstone	Bedding	12	50	65
G	B-202 (Pacific Soils, 2005)	Diatomaceous Siltstone	Joint	12	110	47
G	B-202 (Pacific Soils, 2005)	Diatomaceous Siltstone	Joint	12	170	37
G	B-202 (Pacific Soils, 2005)	Diatomaceous Siltstone	Joint	17	280	80
G	B-202 (Pacific Soils, 2005)	Diatomaceous Siltstone	Joint	17	40	57
G	B-202 (Pacific Soils, 2005)	Diatomaceous Siltstone	Joint	17	30	61
G	B-202 (Pacific Soils, 2005)	Diatomaceous Siltstone	Bedding	26	210	80
G	B-202 (Pacific Soils, 2005)	Diatomaceous Siltstone	Bedding	31	25	67
G	B-202 (Pacific Soils, 2005)	Diatomaceous Siltstone	Bedding	35	220	55
G	B-202 (Pacific Soils, 2005)	Diatomaceous Siltstone	Bedding	40	240	48
G	B-202 (Pacific Soils, 2005)	Diatomaceous Siltstone	Bedding	44	220	57
G	B-202 (Pacific Soils, 2005)	Diatomaceous Siltstone	Bedding	48	250	36
G	B-202 (Pacific Soils, 2005)	Diatomaceous Siltstone	Bedding	50	250	63
G	Area 3 (Geocon West, 2016)	Silty Sand	Joint	S	5	60
G	Area 6 (Geocon West, 2016)	Fossiliferous Sandstone	Joint	S	117	85
G	Area 6 (Geocon West, 2016)	Fossiliferous Sandstone	Joint	S	70	80
G	Area 6 (Geocon West, 2016)	Fossiliferous Sandstone	Joint	S	97	89
G	Area 6 (Geocon West, 2016)	Cherty Sandstone	Joint	S	270	45
G	Area 6 (Geocon West, 2016)	Cherty Sandstone	Joint	S	198	74
G	Area 6 (Geocon West, 2016)	Cherty Sandstone	Joint	S	186	64
G	Area 6 (Geocon West, 2016)	Cherty Sandstone	Joint	S	165	76
G	Area 6 (Geocon West, 2016)	Cherty Sandstone	Joint	S	275	56
G	Area 6 (Geocon West, 2016)	Cherty Sandstone	Joint	S	275	55
G	Area 6 (Geocon West, 2016)	Cherty Sandstone	Joint	S	336	84
B	Area 7 (Geoncon West, 2016)	Diatomaceous Sandstone	Bedding	S	338	68
B	Area 7 (Geoncon West, 2016)	Diatomaceous Sandstone	Bedding	S	338	50
B	Area 7 (Geoncon West, 2016)	Diatomaceous Sandstone	Bedding	S	338	55
B	Area 7 (Geoncon West, 2016)	Diatomaceous Sandstone	Bedding	S	4	62
B	Area 7 (Geoncon West, 2016)	Diatomaceous Sandstone	Bedding	S	2	63
B	Area 7 (Geoncon West, 2016)	Diatomaceous Sandstone	Bedding	S	20	64
B	Area 7 (Geoncon West, 2016)	Diatomaceous Sandstone	Bedding	S	18	61
B	Area 7 (Geoncon West, 2016)	Diatomaceous Sandstone	Bedding	S	19	62
B	Area 7 (Geoncon West, 2016)	Diatomaceous Sandstone	Bedding	S	338	60
B	Area 7 (Geoncon West, 2016)	Diatomaceous Sandstone	Bedding	S	21	64
B	Area 7 (Geoncon West, 2016)	Diatomaceous Sandstone	Joint	S	160	62
B	Area 7 (Geoncon West, 2016)	Diatomaceous Sandstone	Joint	S	80	74
B	Area 7 (Geoncon West, 2016)	Diatomaceous Sandstone	Joint	S	282	90
B	Area 7 (Geoncon West, 2016)	Diatomaceous Sandstone	Joint	S	274	86
B	Area 7 (Geoncon West, 2016)	Diatomaceous Sandstone	Joint	S	278	87
B	Area 7 (Geoncon West, 2016)	Diatomaceous Sandstone	Joint	S	80	62
B	Area 7 (Geoncon West, 2016)	Diatomaceous Sandstone	Joint	S	282	89
B	Area 7 (Geoncon West, 2016)	Diatomaceous Sandstone	Joint	S	275	89
G	(Geocon West, 2016)	Diatomaceous Sandstone	Bedding	S	23	30
G	(Geocon West, 2016)	Diatomaceous Sandstone	Bedding	S	23	55
G	(Geocon West, 2016)	Diatomaceous Sandstone	Bedding	S	337	27
G	(Geocon West, 2016)	Diatomaceous Sandstone	Bedding	S	357	61
G	(Geocon West, 2016)	Diatomaceous Sandstone	Bedding	S	350	65
G	(Geocon West, 2016)	Diatomaceous Sandstone	Bedding	S	350	65
G	(Geocon West, 2016)	Diatomaceous Sandstone	Joint	S	144	70
G	(Geocon West, 2016)	Diatomaceous Sandstone	Joint	S	128	85
G	(Geocon West, 2016)	Diatomaceous Sandstone	Joint	S	282	88
G	(Geocon West, 2016)	Diatomaceous Sandstone	Joint	S	7	78
G	(Geocon West, 2016)	Diatomaceous Sandstone	Joint	S	110	47
G	(Geocon West, 2016)	Diatomaceous Sandstone	Joint	S	250	85
G	(Geocon West, 2016)	Diatomaceous Sandstone	Joint	S	275	80
G	(Geocon West, 2016)	Diatomaceous Sandstone	Joint	S	280	65
G	(Geocon West, 2016)	Diatomaceous Sandstone	Joint	S	150	50
G	(Geocon West, 2016)	Diatomaceous Sandstone	Joint	S	256	71

Data Table for Rockfall Analysis

Data Table for Rock Fall Analysis

Section	Exploration Number	Rock Type	Discontinuity Type	Depth (ft)	Dip Direction (asmouth)	Dip Angle (degrees)
F	B-1 (Geocon West, 2015)	Diatomaceous Siltstone	Bedding	10	322	30
F	B-1 (Geocon West, 2015)	Diatomaceous Siltstone	Fracture	10	102	85
F	B-1 (Geocon West, 2015)	Diatomaceous Siltstone	Bedding	13	50	42
F	B-1 (Geocon West, 2015)	Diatomaceous Siltstone	Bedding	19	51	47
F	B-1 (Geocon West, 2015)	Diatomaceous Siltstone	Fracture	19	96	85
F	B-1 (Geocon West, 2015)	Diatomaceous Siltstone	Bedding	26	330	54
F	B-1 (Geocon West, 2015)	Diatomaceous Siltstone	Bedding	30	21	44
F	B-1 (Geocon West, 2015)	Diatomaceous Siltstone	Fracture	30	109	84
F	B-1 (Geocon West, 2015)	Diatomaceous Siltstone	Bedding	37	18	55
F	B-1 (Geocon West, 2015)	Diatomaceous Siltstone	Fracture	37	81	88
F	B-1 (Geocon West, 2015)	Clayey Siltstone	Bedding	43	32	42
F	B-1 (Geocon West, 2015)	Clayey Siltstone	Bedding	44	354	68
F	B-1 (Geocon West, 2015)	Diatomaceous-rich Clayey Siltston	Bedding	52	322	67
F	B-1 (Geocon West, 2015)	Diatomaceous-rich Clayey Siltston	Bedding	57	352	65
F	B-1 (Geocon West, 2015)	Clayey Siltstone w. fg SS interbeds	Bedding	65	2	62
F	B-1 (Geocon West, 2015)	Clayey Siltstone w. fg SS interbeds	Bedding	69	178	57
F	B-1 (Geocon West, 2015)	Clayey Siltstone w. fg SS interbeds	Bedding	75	5	71
F	B-1 (Geocon West, 2015)	Clayey Siltstone w. fg SS interbeds	Bedding	83	38	51
F	B-1 (Geocon West, 2015)	Diatomaceous Siltstone Interbeds	Bedding	92	2	63
F	B-1 (Geocon West, 2015)	ed Clayey Siltstone & Diatomaceou	Bedding	99	46	74
F	B-1 (Geocon West, 2015)	ed Clayey Siltstone & Diatomaceou	Bedding	102	81	36
F	B-2 (Geocon West, 2015)	Diatomaceous Sandstone	Bedding	S	9	17
F	Area 10 (Geocon West, 2016)	Diatomaceous Sandstone	Bedding	S	332	68
F	Area 10 (Geocon West, 2016)	Diatomaceous Sandstone	Bedding	S	344	63
F	Area 10 (Geocon West, 2016)	Diatomaceous Sandstone	Fracture	S	43	71
F	Area 11 (Geocon West, 2016)	Diatomaceous Sandstone	Bedding	S	4	61
F	Area 11 (Geocon West, 2016)	Diatomaceous Sandstone	Bedding	S	26	53
F	Area 11 (Geocon West, 2016)	Diatomaceous Sandstone	Bedding	S	340	74
F	Area 11 (Geocon West, 2016)	Diatomaceous Sandstone	Joint	S	80	90
F	(Geocon West, 2016)	Diatomaceous Sandstone	Joint	S	100	79
F	(Geocon West, 2016)	Diatomaceous Sandstone	Bedding	S	5	69
F	(Geocon West, 2016)	Diatomaceous Sandstone	Bedding	S	4	58
F	(Geocon West, 2016)	Diatomaceous Sandstone	Bedding	S	12	68
F	(Geocon West, 2016)	Diatomaceous Sandstone	Bedding	S	10	60
F	(Geocon West, 2016)	Diatomaceous Sandstone	Bedding	S	1	60
F	(Geocon West, 2016)	Diatomaceous Sandstone	Bedding	S	18	68
F	(Geocon West, 2016)	Diatomaceous Sandstone	Bedding	S	18	65
F	(Geocon West, 2016)	Diatomaceous Sandstone	Joint	S	256	72
F	(Geocon West, 2016)	Diatomaceous Sandstone	Joint	S	200	45
F	(Geocon West, 2016)	Diatomaceous Sandstone	Joint	S	202	42
F	(Geocon West, 2016)	Diatomaceous Sandstone	Joint	S	200	44
F	(Geocon West, 2016)	Diatomaceous Sandstone	Joint	S	201	44
F	(Geocon West, 2016)	Diatomaceous Sandstone	Joint	S	279	65

Pole plot for planar analysis



Symbol	Feature
◇	Pole Vectors

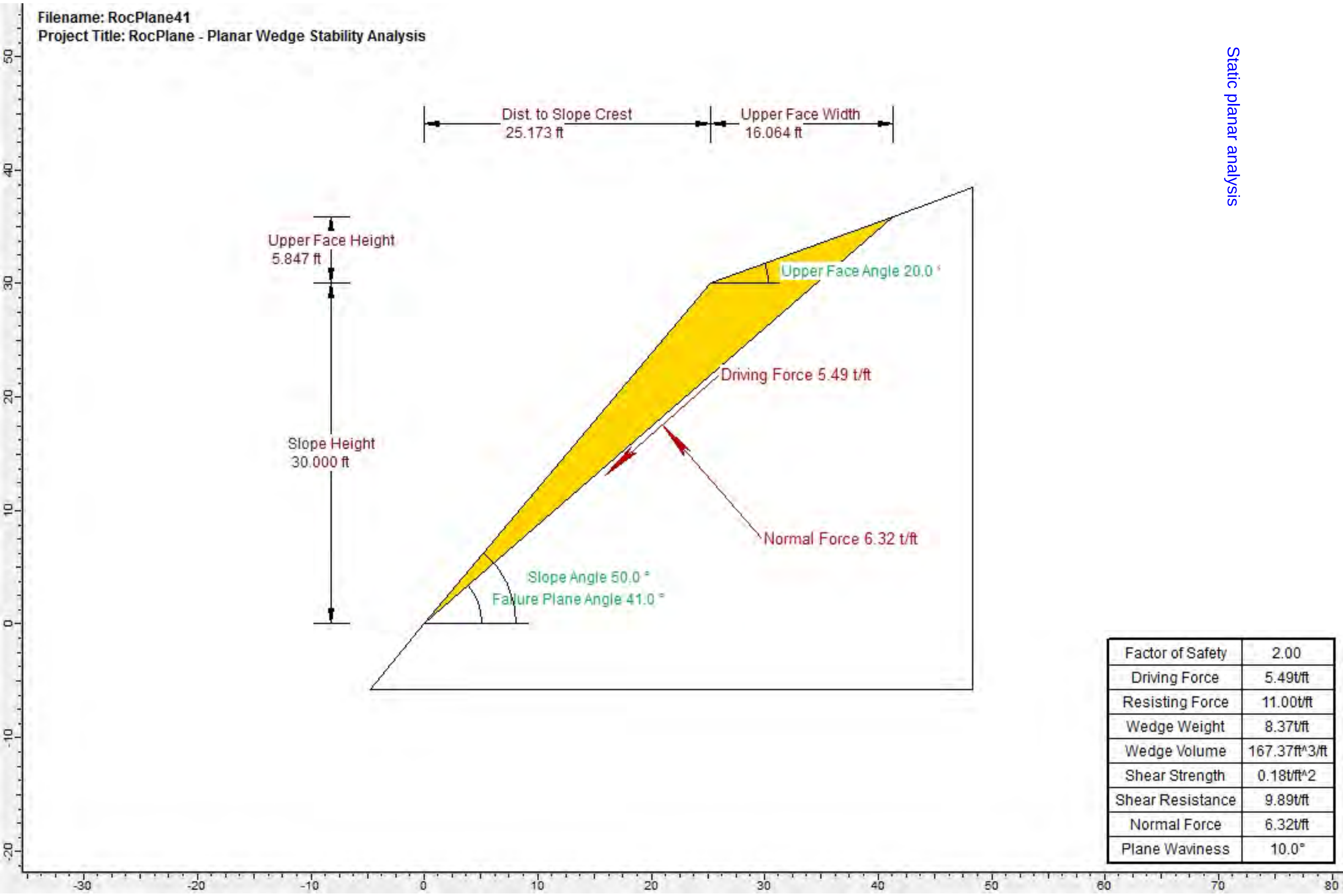
Color	Density Concentrations
	0.00 - 1.00
	1.00 - 2.00
	2.00 - 3.00
	3.00 - 4.00
	4.00 - 5.00
	5.00 - 6.00
	6.00 - 7.00
	7.00 - 8.00
	8.00 - 9.00
🟥	9.00 - 10.00
Maximum Density	9.96%
Contour Data	Pole Vectors
Contour Distribution	Fisher
Counting Circle Size	1.0%

Kinematic Analysis	Planar Sliding		
Slope Dip	50		
Slope Dip Direction	15		
Friction Angle	35°		
Lateral Limits	30°		
	Critical	Total	%
Planar Sliding (All)	2	206	0.97%

	Color	Dip	Dip Direction	Label
Mean Set Planes				
1m	🟪	63	9	J1
2m	🟧	89	98	J2
3m	🟦	69	73	J3
4m	🟪	56	188	J4

Plot Mode	Pole Vectors
Vector Count	206 (206 Entries)
Hemisphere	Lower
Projection	Equal Angle

Static planar analysis



Filename: RocPlane41
Project Title: RocPlane - Planar Wedge Stability Analysis

Factor of Safety	2.00
Driving Force	5.49t/ft
Resisting Force	11.00t/ft
Wedge Weight	8.37t/ft
Wedge Volume	167.37ft^3/ft
Shear Strength	0.18t/ft^2
Shear Resistance	9.89t/ft
Normal Force	6.32t/ft
Plane Waviness	10.0°

RocPlane Analysis Information

Document Name:
RocPlane1

Job Title:
RocPlane - Planar Wedge Stability Analysis with no additional forces

Analysis Results:

```
Analysis type  = Deterministic
Normal Force     = 6.31579 t/ft
Resisting Force  = 11 t/ft
Driving Force      = 5.49023 t/ft
Factor of Safety  = 2.00355
```

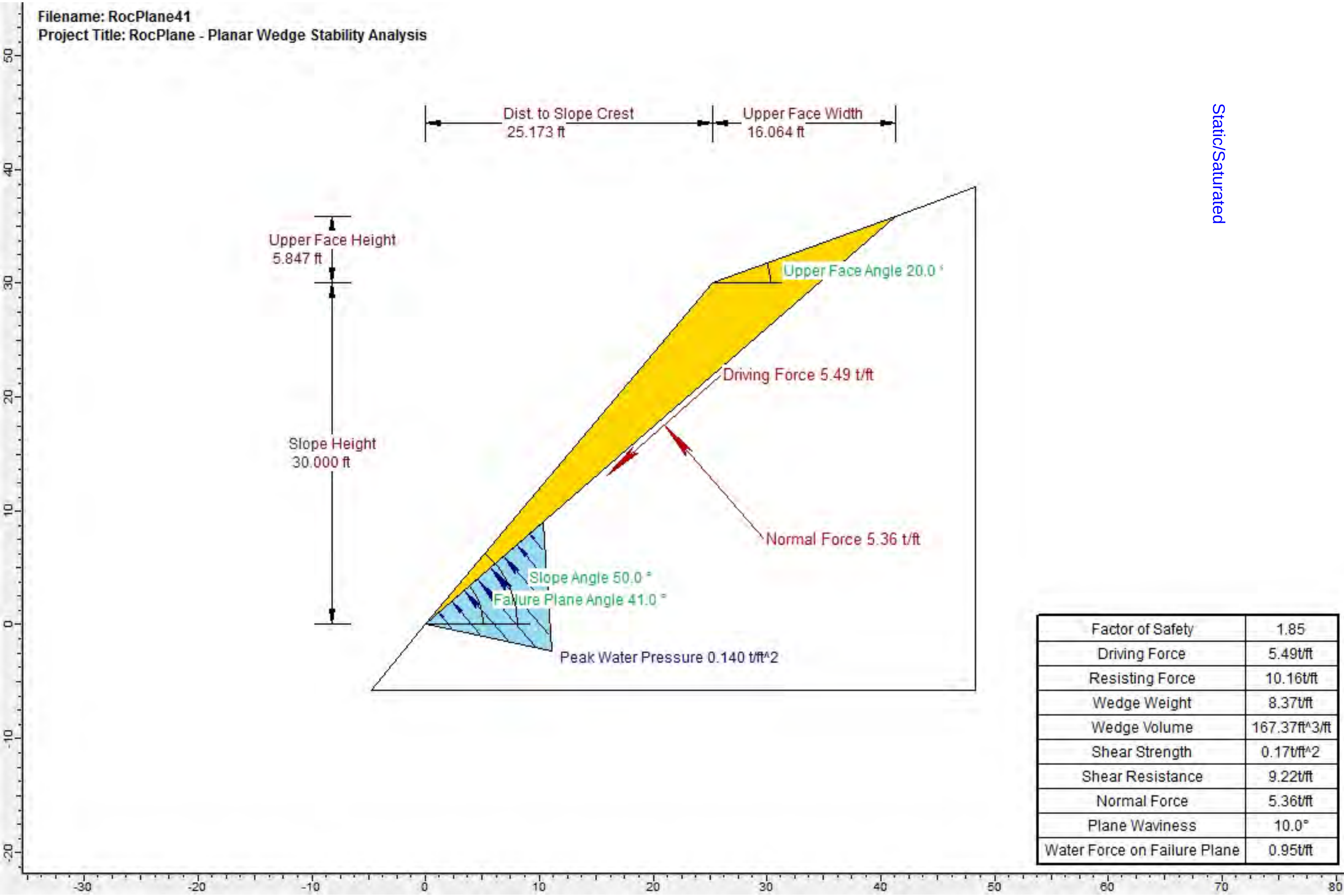
Geometry:

```
Slope Height     = 30 ft
Wedge Weight  = 8.3685 t/ft
Wedge Volume = 167.37 ft^3/ft
Rock Unit Weight = 0.05 t/ft^2
Slope Angle        = 50 °
Failure Plane Angle = 41 °
Upper Face Angle = 20 °
Bench Width : Not Present
Waviness           = 10 °

Intersection Point (B) of slope and upper face = ( 25.173 , 30 )
Intersection point (C) of failure plane and upper face = ( 41.2371 , 35.8469 )
Failure plane length ( Origin  -->  C ) = 54.6397 ft
Slope length ( Origin  -->  B ) = 39.1227 ft

Tension Crack : Not Present
```

Strength:

```
Shear Strength Model : Mohr-Coulomb
Friction Angle = 35 °
Cohesion = 0.1 t/ft^2
Shear Strength: 0.180937 t/ft^2
Shear Resistance: 9.88633 t/ft
```

External Forces : Not Present



Filename: RocPlane41
Project Title: RocPlane - Planar Wedge Stability Analysis

Static/Saturated

Factor of Safety	1.85
Driving Force	5.49t/ft
Resisting Force	10.16t/ft
Wedge Weight	8.37t/ft
Wedge Volume	167.37ft^3/ft
Shear Strength	0.17t/ft^2
Shear Resistance	9.22t/ft
Normal Force	5.36t/ft
Plane Waviness	10.0°
Water Force on Failure Plane	0.95t/ft

RocPlane Analysis Information

Document Name:
RocPlane41

Job Title:
RocPlane - Planar Wedge Stability Analysis with water pressure

Analysis Results:

Analysis type = Deterministic
Normal Force = 5.36094 t/ft
Resisting Force = 10.163 t/ft
Driving Force = 5.49023 t/ft
Factor of Safety = 1.85111

Geometry:

Slope Height = 30 ft
Wedge Weight = 8.3685 t/ft
Wedge Volume = 167.37 ft^3/ft
Rock Unit Weight = 0.05 t/ft^2
Slope Angle = 50 °
Failure Plane Angle = 41 °
Upper Face Angle = 20 °
Bench Width : Not Present
Waviness = 10 °

Intersection Point (B) of slope and upper face = (25.173 , 30)
Intersection point (C) of failure plane and upper face = (41.2371 , 35.8469)
Failure plane length (Origin --> C) = 54.6397 ft
Slope length (Origin --> B) = 39.1227 ft

Tension Crack : Not Present

Strength:

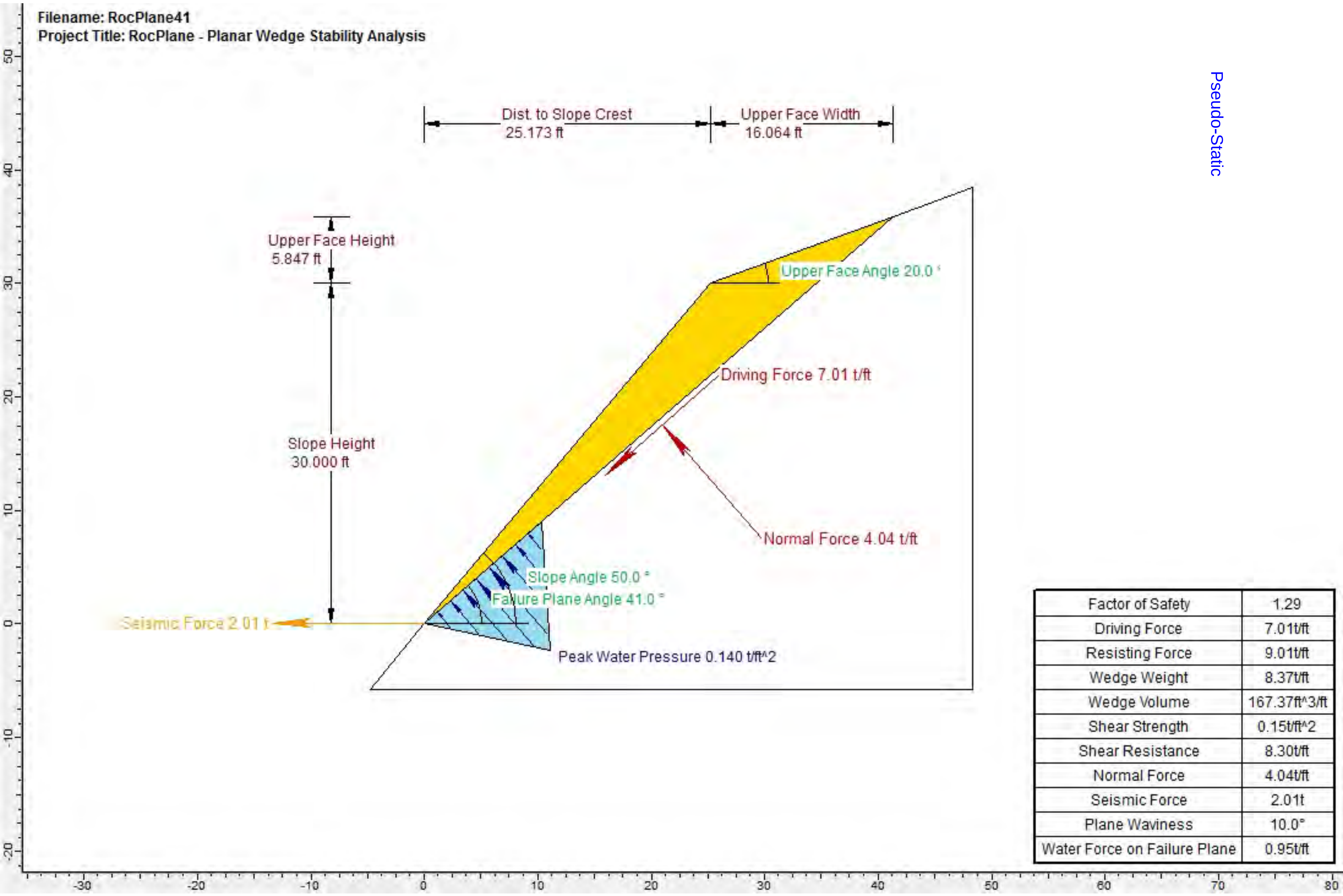
Shear Strength Model : Mohr-Coulomb
Friction Angle = 35 °
Cohesion = 0.1 t/ft^2
Shear Strength: 0.168701 t/ft^2
Shear Resistance: 9.21774 t/ft

Water Pressure:

Water Unit Weight = 0.0312 t/ft^2
Pressure Distribution Model : Peak Pressure - Mid Height
Percent Filled : 25 %
Water Force on Failure Plane = 0.954847 t/ft

External Forces : Not Present

Pseudo-Static



Filename: RocPlane41
Project Title: RocPlane - Planar Wedge Stability Analysis

Factor of Safety	1.29
Driving Force	7.01t/ft
Resisting Force	9.01t/ft
Wedge Weight	8.37t/ft
Wedge Volume	167.37ft^3/ft
Shear Strength	0.15t/ft^2
Shear Resistance	8.30t/ft
Normal Force	4.04t/ft
Seismic Force	2.01t
Plane Waviness	10.0°
Water Force on Failure Plane	0.95t/ft

RocPlane Analysis Information

Document Name:
RocPlane41

Job Title:
RocPlane - Planar Wedge Stability Analysis with water and seismic loads

Analysis Results:

Analysis type = Deterministic
Normal Force = 4.04329 t/ft
Resisting Force = 9.00805 t/ft
Driving Force = 7.00602 t/ft
Factor of Safety = 1.28576

Geometry:

Slope Height = 30 ft
Wedge Weight = 8.3685 t/ft
Wedge Volume = 167.37 ft^3/ft
Rock Unit Weight = 0.05 t/ft^2
Slope Angle = 50 °
Failure Plane Angle = 41 °
Upper Face Angle = 20 °
Bench Width : Not Present
Waviness = 10 °

Intersection Point (B) of slope and upper face = (25.173 , 30)
Intersection point (C) of failure plane and upper face = (41.2371 , 35.8469)
Failure plane length (Origin --> C) = 54.6397 ft
Slope length (Origin --> B) = 39.1227 ft

Tension Crack : Not Present

Strength:

Shear Strength Model : Mohr-Coulomb
Friction Angle = 35 °
Cohesion = 0.1 t/ft^2
Shear Strength: 0.151815 t/ft^2
Shear Resistance: 8.29511 t/ft

Water Pressure:

Water Unit Weight = 0.0312 t/ft^2
Pressure Distribution Model : Peak Pressure - Mid Height
Percent Filled : 25 %
Water Force on Failure Plane = 0.954847 t/ft

Seismic Force:

Direction : Horizontal
Seismic Coefficient : 0.24
Seismic Force : 2.00844 t/ft

External Forces : Not Present

0 5 10 20 30

SCALE: 1"=10'

DATE: 02-11-16

SOLANA TORRANCE

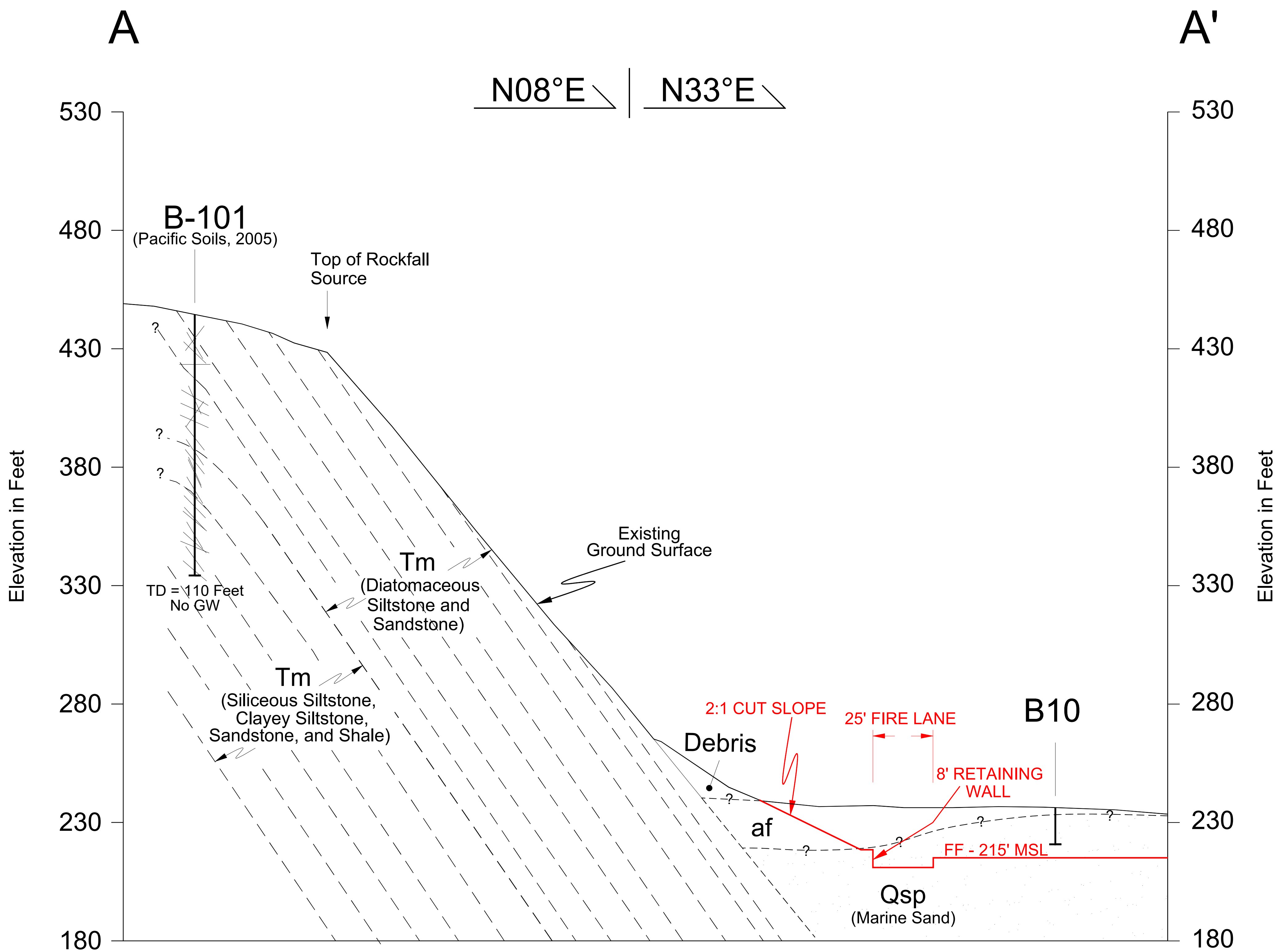
REYLENN PROPERTIES LLC

PARTIAL PRELIMINARY GRADING PLAN

TORRANCE , CALIFORNIA

KHR ASSOCIATES
CONSULTING ENGINEERS/SURVEYORS/PLANNERS
20411 SW Birch Street - Suite 310
Newport Beach, California 92660
Tel (949) 756-6440
Fax (949) 756-6444

R:\ReyLenn\ReyLenn-Torrance\CDs\Preliminary\Grading\RLT-GRAD-BASE.dwg February 11, 2016 – 10:50am Mike Thomas



NOTES:

1. THE SECTIONS ARE BASED ON GEOLOGIC CONDITIONS AT BORING LOCATIONS AND AT SURFACE EXPOSURES MAPPED DURING THE INVESTIGATION. THE GEOLOGIC CONDITIONS BETWEEN SUCH LOCATIONS HAVE BEEN INTERPOLATED. LOCALIZED VARIATIONS COULD OCCUR. THE SECTIONS ARE INTENDED FOR DESCRIPTIVE PURPOSES ONLY.
2. SEE PARTIAL PRELIMINARY GRADING PLAN (KHR ASSOCIATES) FOR APPROXIMATE LOCATION OF SECTIONS.
3. BEDDING DIPS STEEPER THAN SLOPE.

Scale (H&V): 1" = 50'

0 50' 100'

GEOCON
WEST, INC.

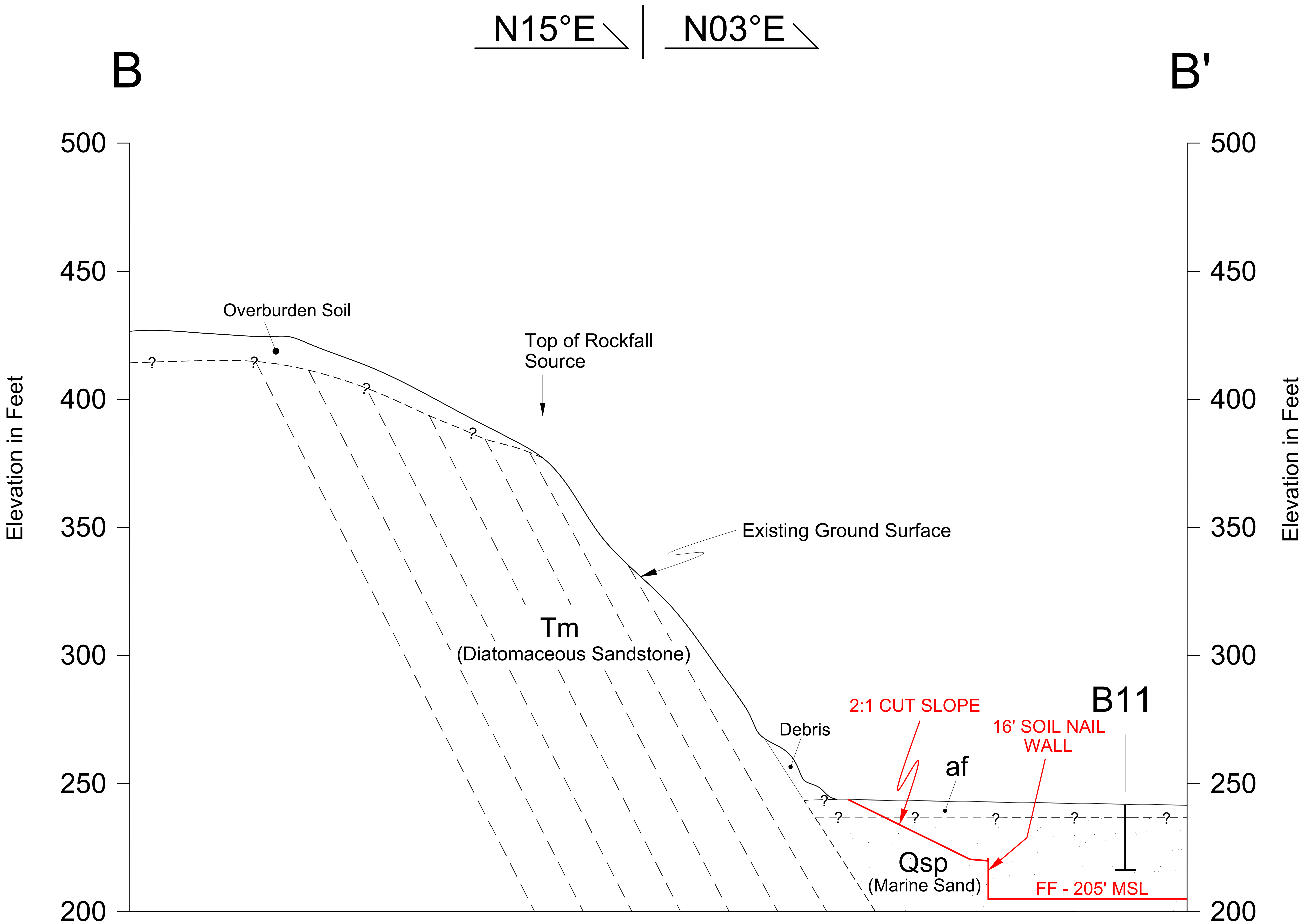
ENVIRONMENTAL GEOTECHNICAL MATERIALS
3303 N. SAN FERNANDO BLVD. - SUITE 100 - BURBANK, CA 91504
PHONE (818) 841-8388 - FAX (818) 841-1704

Drafted By: RMA
Checked By: SFK / GAK

CIVIL SECTION A-A'

MKS RESIDENTIAL
HAWTHORNE & VIA VALMONTE
TORRANCE, CA 90505

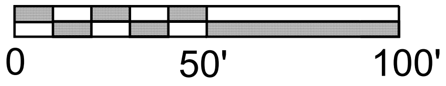
FEB 2016
PROJECT NO. A9201-06-01C
FIG. 1



NOTES:

1. THE SECTIONS ARE BASED ON GEOLOGIC CONDITIONS AT BORING LOCATIONS AND AT SURFACE EXPOSURES MAPPED DURING THE INVESTIGATION. THE GEOLOGIC CONDITIONS BETWEEN SUCH LOCATIONS HAVE BEEN INTERPOLATED. LOCALIZED VARIATIONS COULD OCCUR. THE SECTIONS ARE INTENDED FOR DESCRIPTIVE PURPOSES ONLY.
2. SEE PARTIAL PRELIMINARY GRADING PLAN (KHR ASSOCIATES) FOR APPROXIMATE LOCATION OF SECTIONS.
3. BEDDING DIPS STEEPER THAN SLOPE.

Scale (H&V): 1" = 50'



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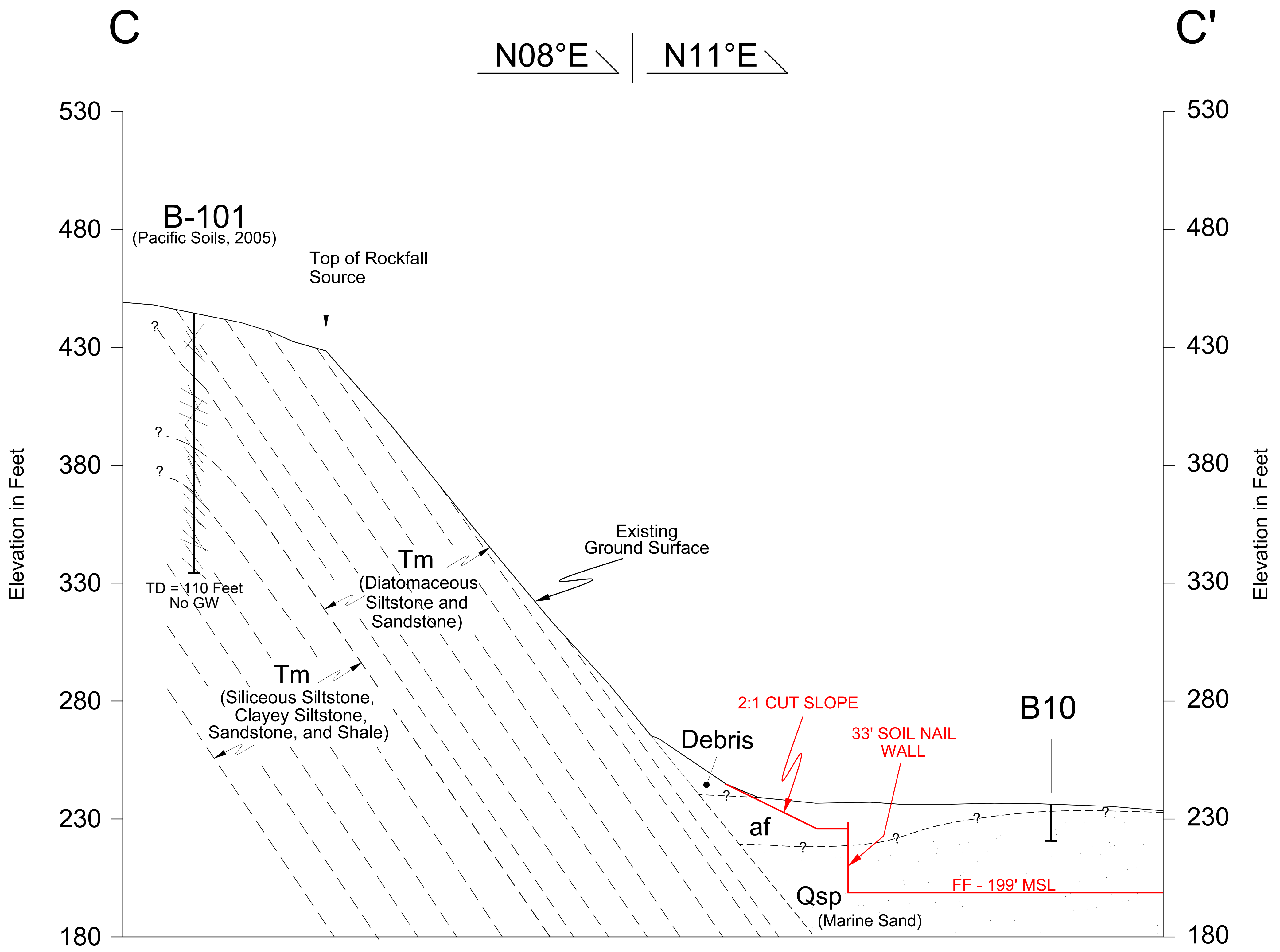
PHONE (818) 841-8388 - FAX (818) 841-1704

Drafted By: RMA | Checked By: SFK / GAK

CIVIL SECTION B-B'

MKS RESIDENTIAL
HAWTHORNE & VIA VALMONTE
TORRANCE, CA 90505

FEB 2016 | PROJECT NO. A9201-06-01C | FIG. 2



NOTES:

1. THE SECTIONS ARE BASED ON GEOLOGIC CONDITIONS AT BORING LOCATIONS AND AT SURFACE EXPOSURES MAPPED DURING THE INVESTIGATION. THE GEOLOGIC CONDITIONS BETWEEN SUCH LOCATIONS HAVE BEEN INTERPOLATED. LOCALIZED VARIATIONS COULD OCCUR. THE SECTIONS ARE INTENDED FOR DESCRIPTIVE PURPOSES ONLY.
2. SEE PARTIAL PRELIMINARY GRADING PLAN (KHR ASSOCIATES) FOR APPROXIMATE LOCATION OF SECTIONS.
3. BEDDING DIPS STEEPER THAN SLOPE.

Scale (H&V): 1" = 50'

0 50' 100'

GEOCON
WEST, INC.

ENVIRONMENTAL GEOTECHNICAL MATERIALS
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Drafted By: RMA | Checked By: SFK / GAK

CIVIL SECTION C-C'

MKS RESIDENTIAL
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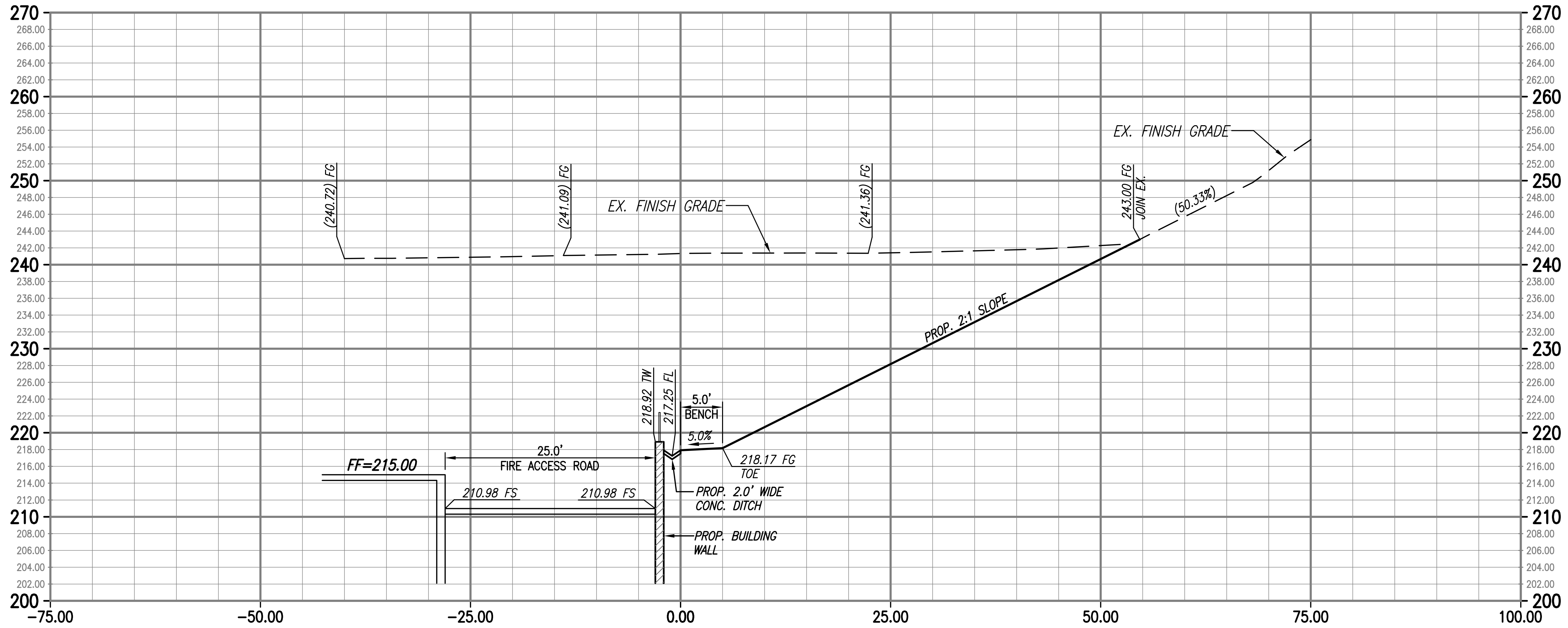
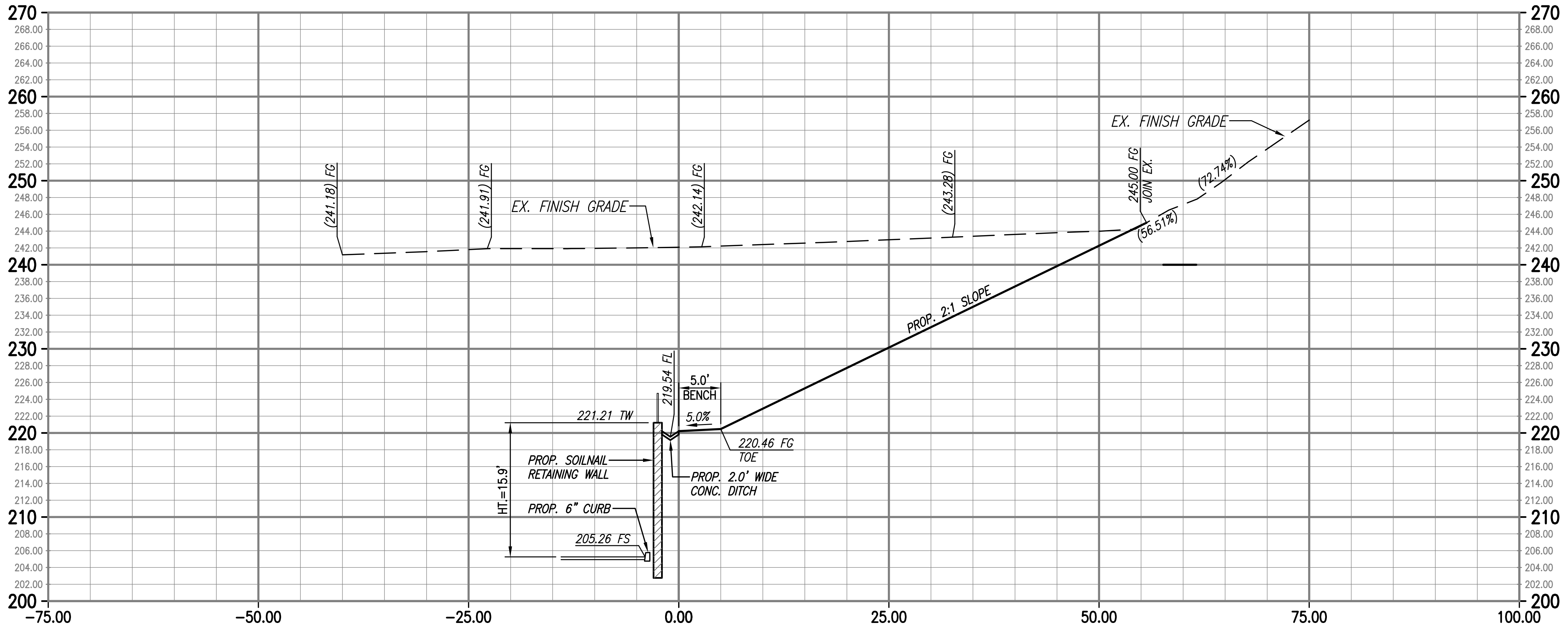
TORRANCE, CA 90505

FEB 2016 | PROJECT NO. A9201-06-01C | FIG. 3



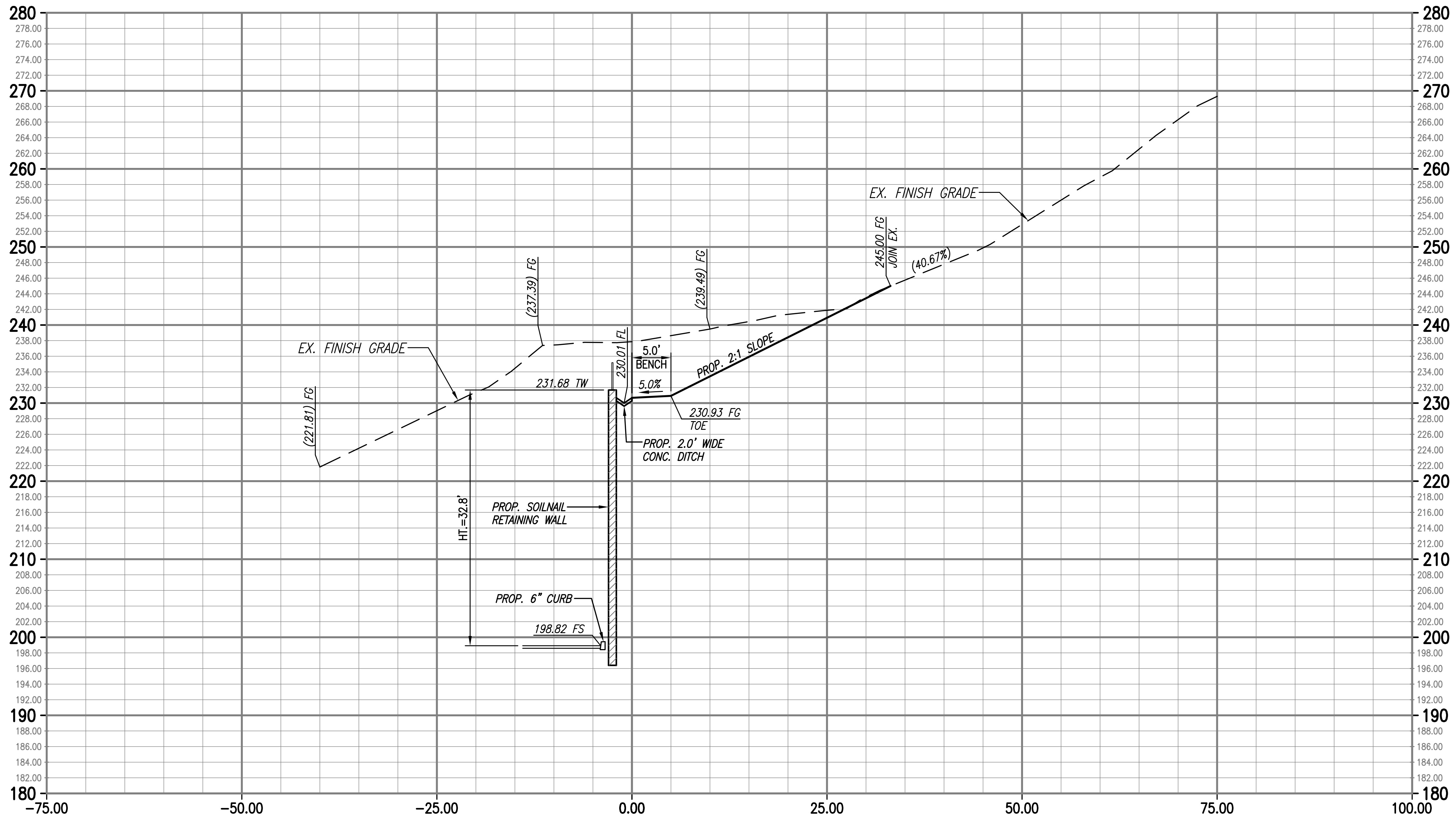
SECTION "A"-"A"

SCALE: 1"=10'



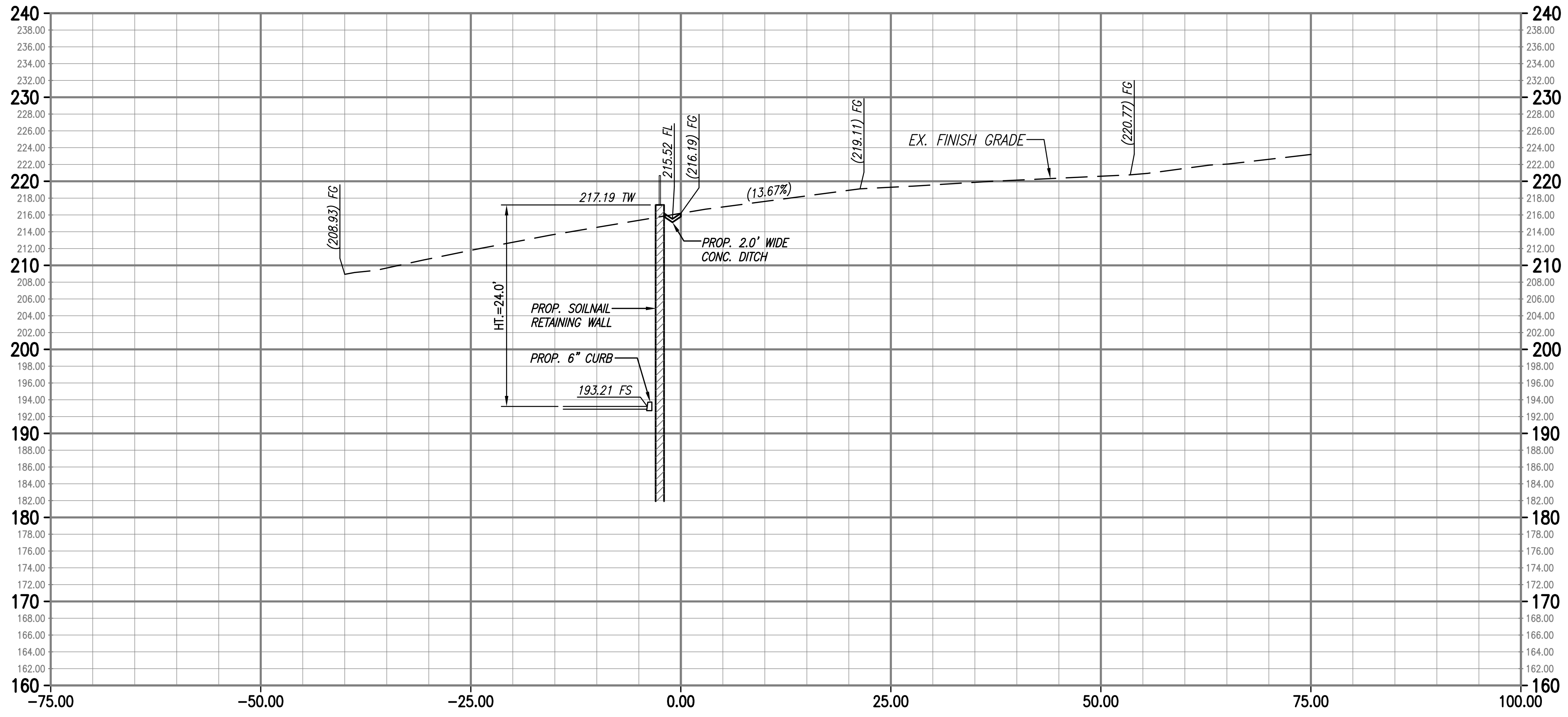
SECTION "B"-"B"

SCALE: 1"=10'



SECTION "C"-"C"

SCALE: 1"=10'



SECTION "D"-"D"

SCALE: 1"=10'

SOLANA TORRANCE

REYLENN PROPERTIES LLC

SITE SECTIONS

TORRANCE , CALIFORNIA

KHR ASSOCIATES
CONSULTING ENGINEERS/SURVEYORS/PLANNERS
20411 SW Birch Street - Suite 310
Newport Beach, California 92660
Tel (949) 756-6440
Fax (949) 756-6444

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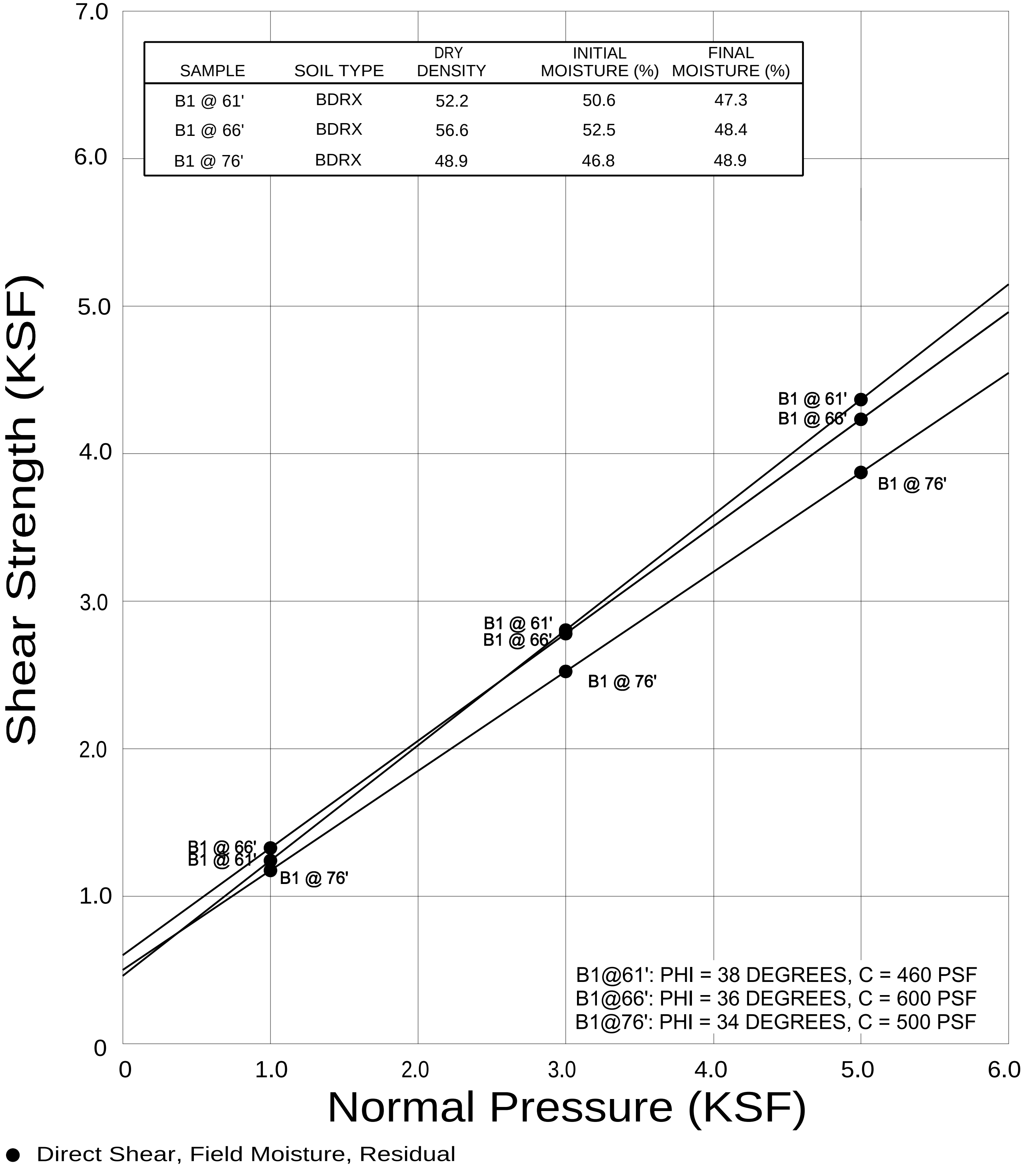

SAMPLE	SOIL TYPE	DRY DENSITY	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B1 @ 61'	BDRX	52.2	50.6	47.3
B1 @ 66'	BDRX	56.6	52.5	48.4
B1 @ 76'	BDRX	48.9	46.8	48.9

B1@61': PHI = 38 DEGREES, C = 460 PSF
B1@66': PHI = 36 DEGREES, C = 600 PSF
B1@76': PHI = 34 DEGREES, C = 500 PSF

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PHONE (818) 841-8388 - FAX (818) 841-1704

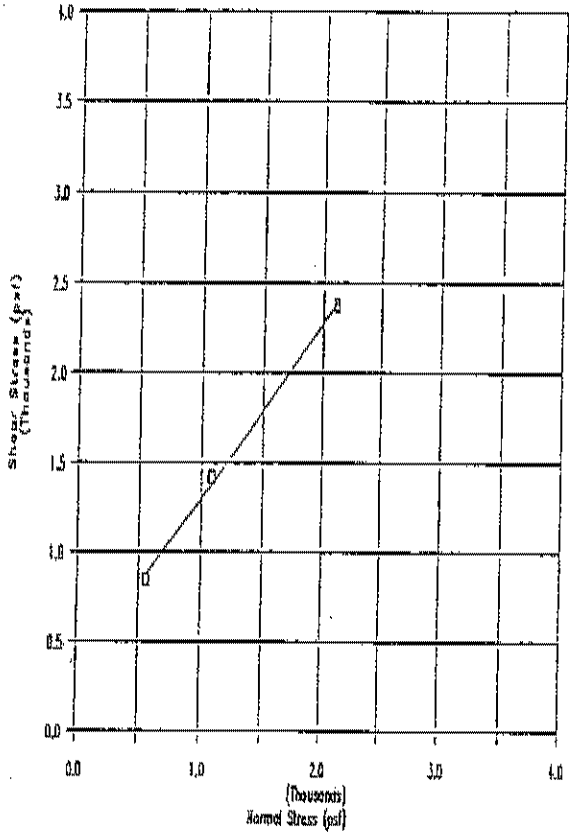
Drafted by: JMT | Checked by: NDB

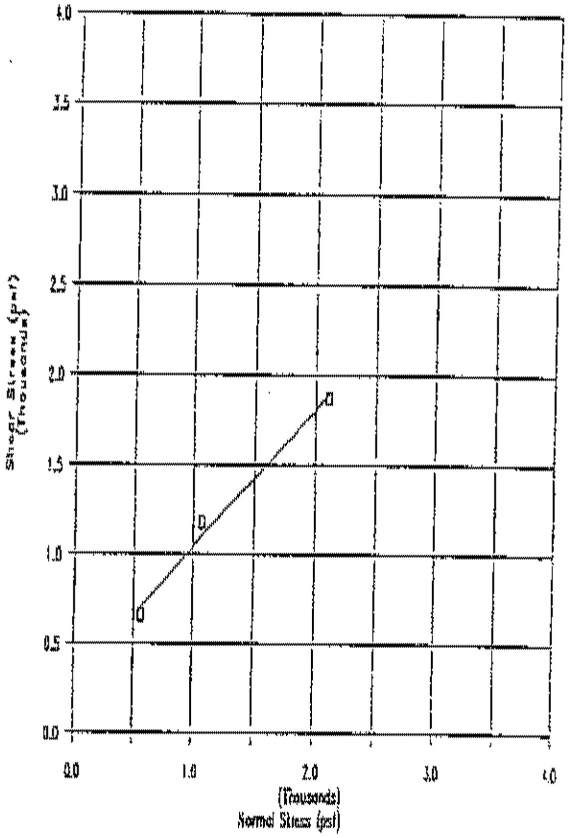
DIRECT SHEAR TEST RESULTS

MKS RESIDENTIAL
HAWTHORNE & VIA VALMONTE
TORRANCE, CA 90505

DRAFT | PROJECT NO. A9201-06-01C | FIG. B3

Work Order 93-1605



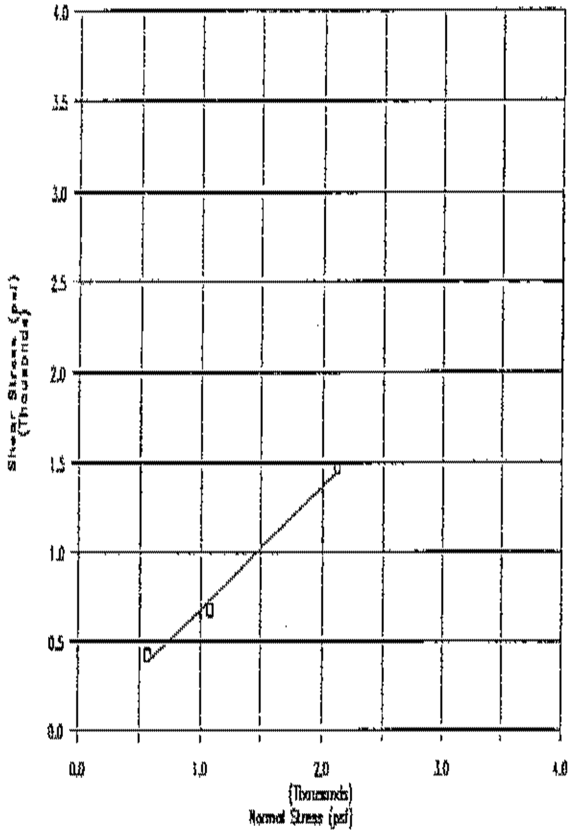


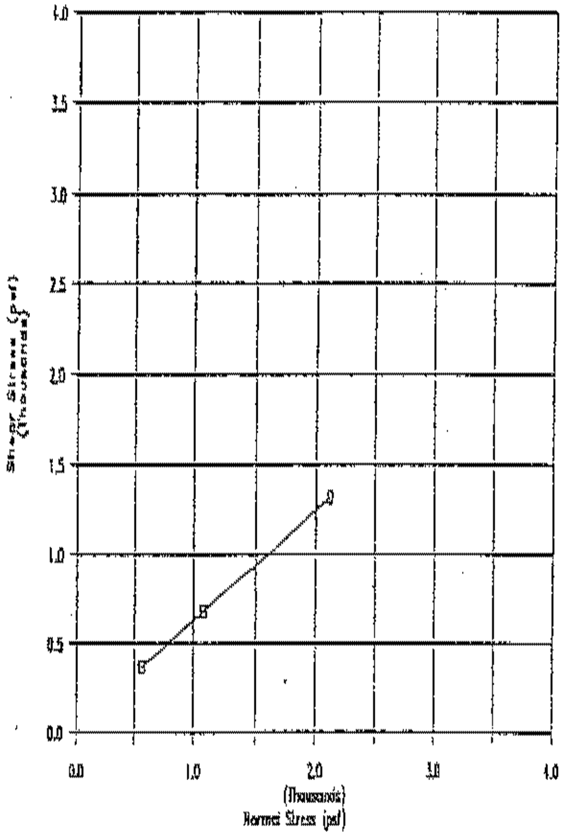
	B-1 AT 15.0 FEET	* B-1 AT 15.0 FEET
Angle of internal friction =	44 deg	37 deg
Cohesion =	320 psf	270 psf
R^2 =	.998	0.989

* Residual

DIRECT SHEAR TEST DATA

Prepared for: MR. DAN E. BUTCHER





	B-2 AT 15.0 FEET	* B-2 AT 15.0 FEET
Angle of internal friction =	34 deg	31 deg
Cohesion =	0 psf	20 psf
R^2 =	0.990	1.0

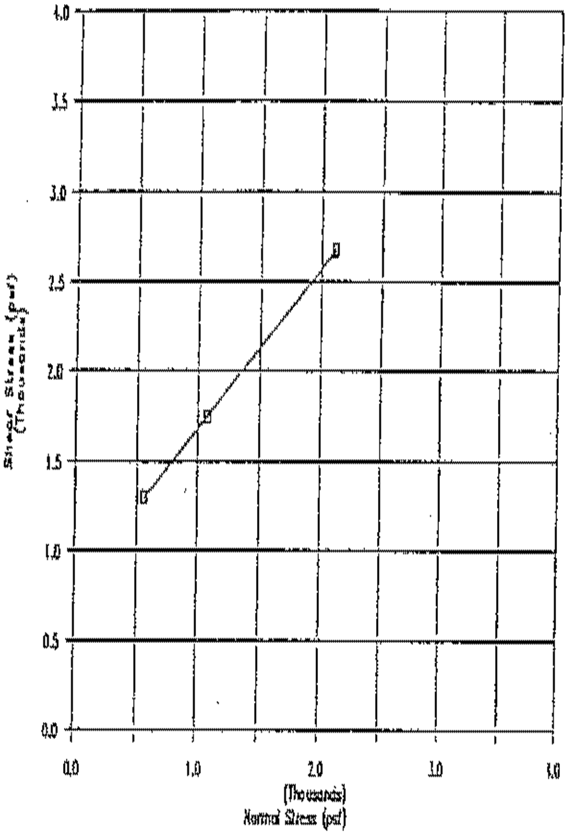
* Residual

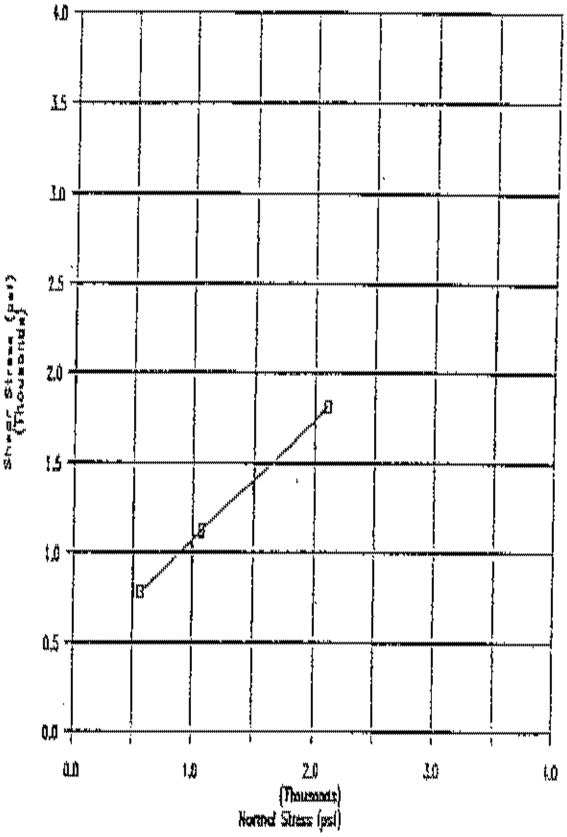
DIRECT SHEAR TEST DATA

Prepared for: MR. DAN E. BUTCHER

Western Laboratories
Geotechnical Engineering

PLATE E-6



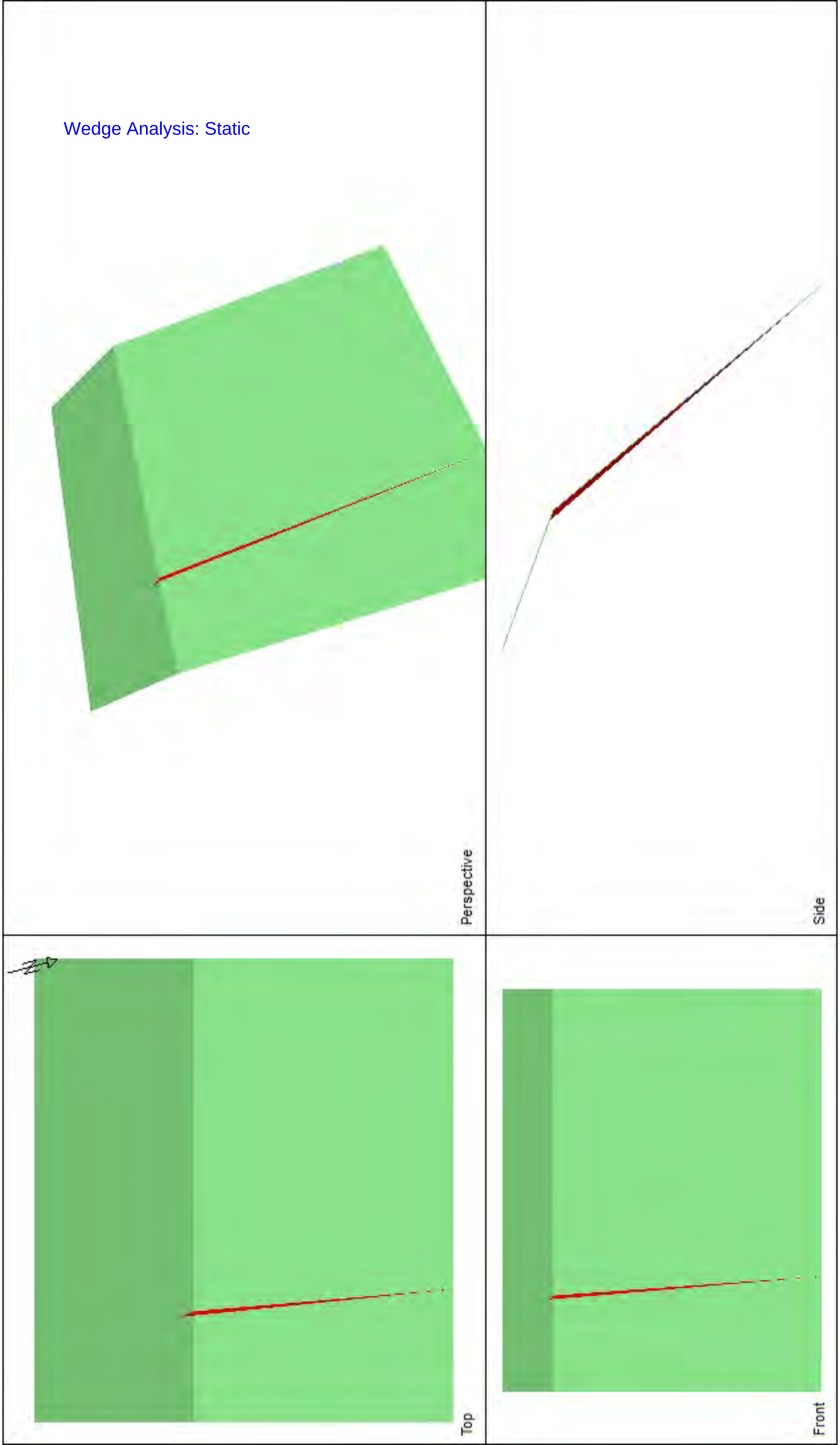


	B-1 AT 35.0 FEET	* B-1 AT 35.0 FEET
Angle of internal friction =	42 deg	34 deg
Cohesion =	790 psf	400 psf
R^2 =	1.0	1.0

* Residual

DIRECT SHEAR TEST DATA

Prepared for: MR. DAN E. BUTCHER


Wedge Analysis: Static
Perspective
Side
Top
Front

Swedge Analysis Information

Document Name:

- Wedge analysis with no additional forces

Project Summary:

- Job Title: SWEDGE - Surface Wedge Stability Analysis
- Date Created: 1/28/2016, 11:24:38 PM

Analysis Results:

- Analysis type: Deterministic
- Safety Factor: 8.7084
- Wedge height (on slope) [ft]: 200.00
- Bench width (on upper face) [ft]: 8.01
- Wedge volume [ft^3]: 510.045
- Wedge weight [tons]: 25.502
- Wedge area (joint1) [ft^2]: 560.32
- Wedge area (joint2) [ft^2]: 702.34
- Wedge area (slope) [ft^2]: 359.22
- Wedge area (upper face) [ft^2]: 11.72

Effective Normal and Strength Properties:

	Joint 1	**Joint 2**
Effective Normal force [tons]	26.83	32.01
Effective Normal stress [t/ft^2]	0.05	0.05
Shear Strength [t/ft^2]	0.13	0.13
Strength due to Waviness [t/ft^2]	0.00	0.00

- Driving force [tons]: 19.23
- Resisting force [tons]: 167.46

Failure Mode:

- Sliding on intersection line (joints 1&2)

Joint Sets 1&2 line of Intersection:

Plunge [deg]	**Trend [deg]**	**Length [ft]**
48.94	9.15	269.10

Trace Lengths:

	Slope Face [ft]	**Upper Face [ft]**
Joint 1	261.64	8.57
Joint 2	261.47	9.28

Persistence:

- Joint 1 [ft]: 269.10
- Joint 2 [ft]: 269.10

Intersection Angles:

	Slope Face	Upper Face
Joint 1 & Joint 2	0.60	17.14
Joint 1 & Crest	86.26	96.24
Joint 2 & Crest	93.14	66.62

Dip and Dip Direction:

	Dip [deg]	Dip Direction [deg]
Joint Set 1	89.00	98.00
Joint Set 2	69.00	73.00
Slope	50.00	15.00
Upper Face	20.00	15.00

Joint Set 1 Data:

- Cohesion [t/ft^2]: 0.10
- Friction Angle [deg]: 35.00

Joint Set 2 Data:

- Cohesion [t/ft^2]: 0.10
- Friction Angle [deg]: 35.00

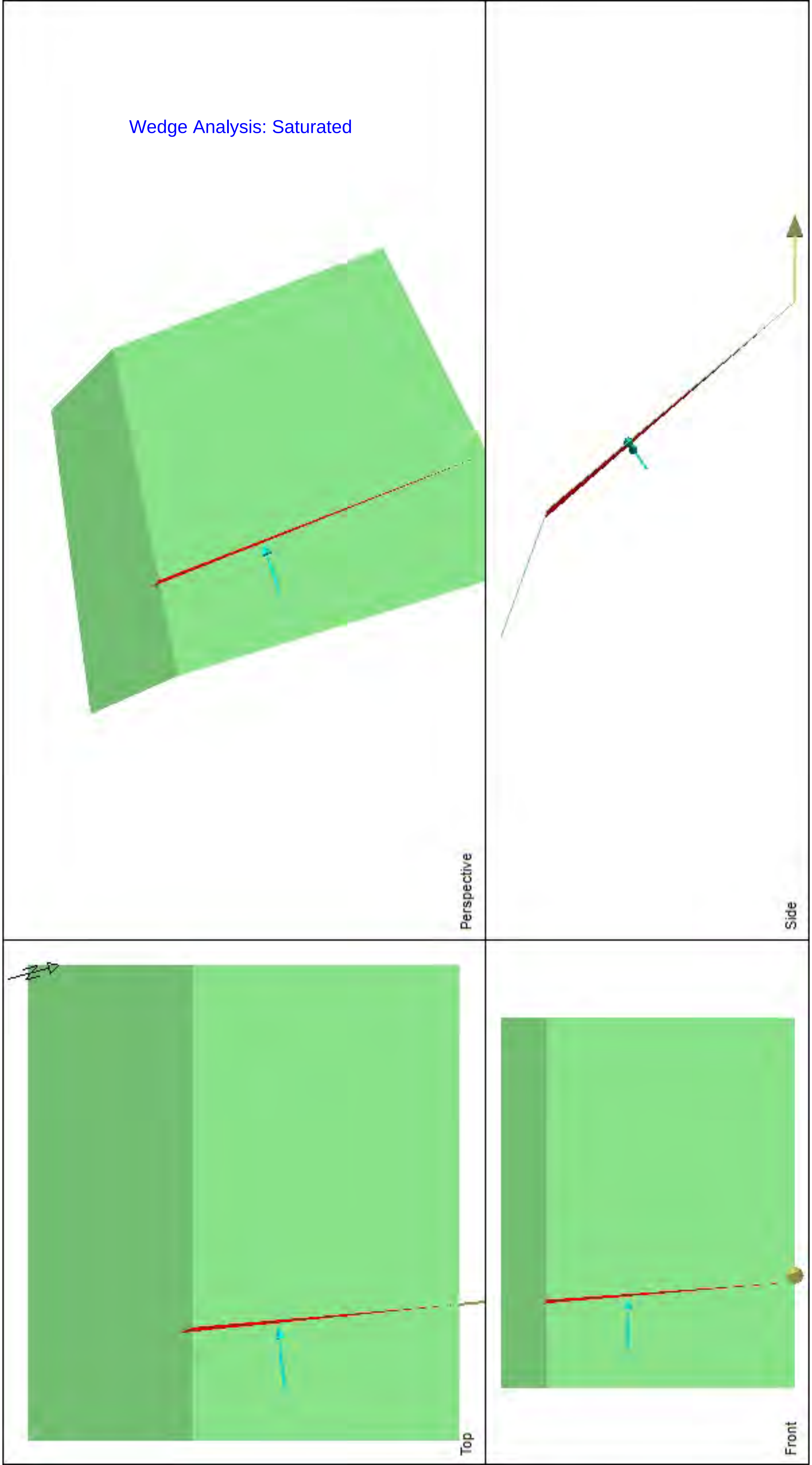
Slope Data:

- Slope height [ft]: 200.00
- Rock unit weight [t/ft^3]: 0.05
- Water pressures in the slope: NO
- Overhanging slope face: NO
- Externally applied force: NO
- Tension crack: NO

Wedge Vertices:

- Coordinates in Easting,Northing,Up Format
- 1=Joint1, 2=Joint2, 3=Upper Face, 4=Slope

Point	x	y	z
124	0.000	0.000	0.000
134	-26.929	-166.524	200.000
234	-29.587	-165.812	200.000
123	-28.101	-174.498	202.914


Wedge Analysis: Saturated
Perspective
Side
Top
Front

Swedge Analysis Information

Document Name:

- Wedge Analysis with water pressure

Project Summary:

- Job Title: SWEDGE - Surface Wedge Stability Analysis
- Date Created: 1/28/2016, 11:24:38 PM

Analysis Results:

- Analysis type: Deterministic
- Safety Factor: 7.9505
- Wedge height (on slope) [ft]: 200.00
- Bench width (on upper face) [ft]: 8.01
- Wedge volume [ft^3]: 510.045
- Wedge weight [tons]: 25.502
- Wedge area (joint1) [ft^2]: 560.32
- Wedge area (joint2) [ft^2]: 702.34
- Wedge area (slope) [ft^2]: 359.22
- Wedge area (upper face) [ft^2]: 11.72

Effective Normal and Strength Properties:

	Joint 1	**Joint 2**
Effective Normal force [tons]	17.59	20.43
Effective Normal stress [t/ft^2]	0.03	0.03
Shear Strength [t/ft^2]	0.12	0.12
Strength due to Waviness [t/ft^2]	0.00	0.00

- Driving force [tons]: 19.23
- Resisting force [tons]: 152.89

Water Pressures/Forces:

	Average pressure [t/ft^2]	**Water force [tons]**
Joint 1	N/A	9.24
Joint 2	N/A	11.58
Fissures	0.02	N/A

Failure Mode:

- Sliding on intersection line (joints 1&2)

Joint Sets 1&2 line of Intersection:

Plunge [deg]	**Trend [deg]**	**Length [ft]**
48.94	9.15	269.10

Trace Lengths:

	Slope Face [ft]	Upper Face [ft]
Joint 1	261.64	8.57
Joint 2	261.47	9.28

Persistence:

- Joint 1 [ft]: 269.10
- Joint 2 [ft]: 269.10

Intersection Angles:

	Slope Face	Upper Face
Joint 1 & Joint 2	0.60	17.14
Joint 1 & Crest	86.26	96.24
Joint 2 & Crest	93.14	66.62

Dip and Dip Direction:

	Dip [deg]	Dip Direction [deg]
Joint Set 1	89.00	98.00
Joint Set 2	69.00	73.00
Slope	50.00	15.00
Upper Face	20.00	15.00

Joint Set 1 Data:

- Cohesion [t/ft^2]: 0.10
- Friction Angle [deg]: 35.00

Joint Set 2 Data:

- Cohesion [t/ft^2]: 0.10
- Friction Angle [deg]: 35.00

Slope Data:

- Slope height [ft]: 200.00
- Rock unit weight [t/ft^3]: 0.05
- Water pressures in the slope: YES
- Overhanging slope face: NO
- Externally applied force: NO
- Tension crack: NO

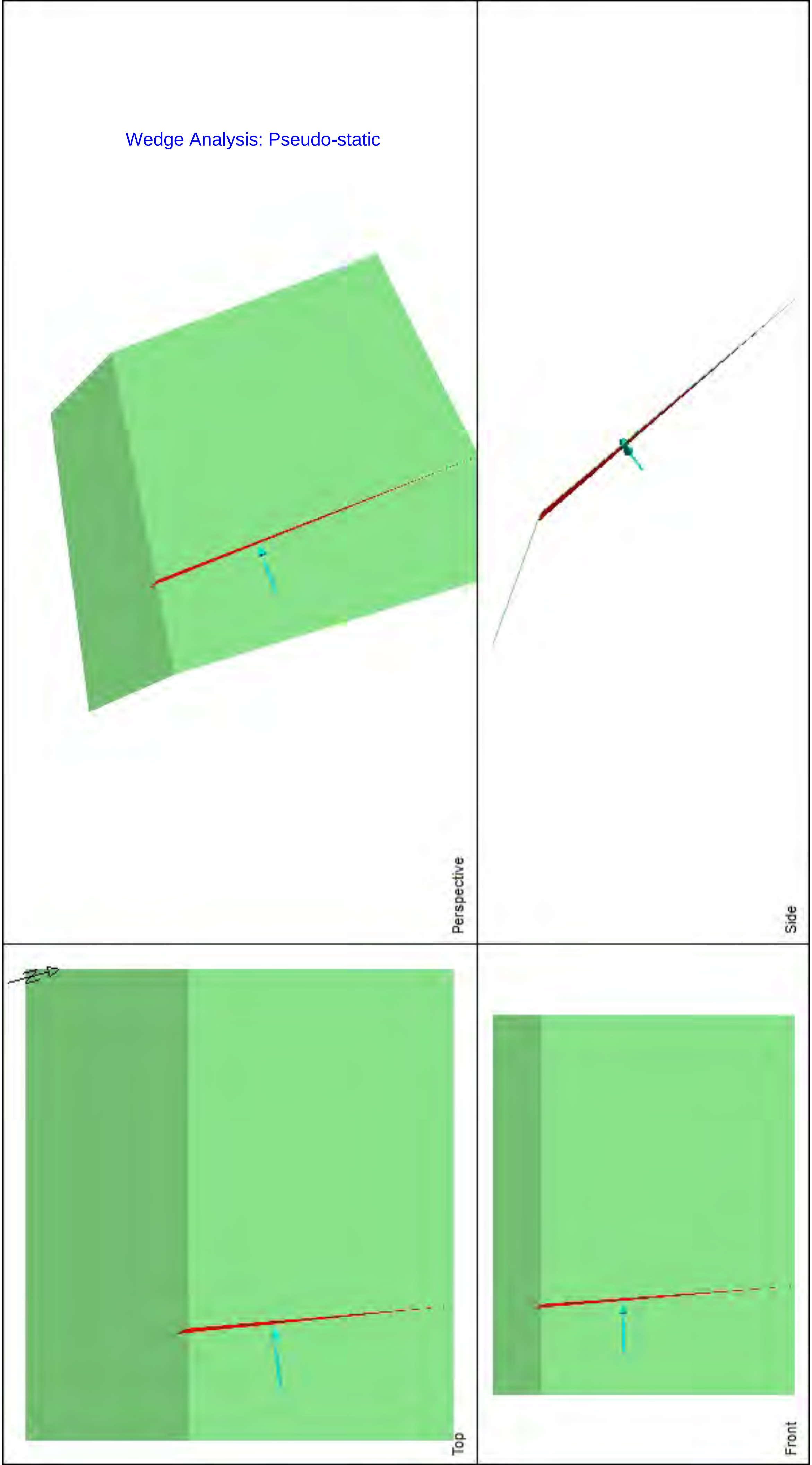
Water Pressure Data:

- Water unit weight [t/ft^3]: 0.031
- Pressure definition method: Percent Filled Fissures
- Percent Filled: 25.000 %

Wedge Vertices:

- Coordinates in Easting,Northing,Up Format
- 1=Joint1, 2=Joint2, 3=Upper Face, 4=Slope

Point	**x**	**y**	**z**
124	0.000	0.000	0.000
134	-26.929	-166.524	200.000
234	-29.587	-165.812	200.000
123	-28.101	-174.498	202.914


Wedge Analysis: Pseudo-static
Perspective
Side
Top
Front

Swedge Analysis Information

Document Name:

- Wedge with water and pseudo-static load

Project Summary:

- Job Title: SWEDGE - Surface Wedge Stability Analysis
- Date Created: 1/28/2016, 11:24:38 PM

Analysis Results:

- Analysis type: Deterministic
- Safety Factor: 6.0875
- Wedge height (on slope) [ft]: 200.00
- Bench width (on upper face) [ft]: 8.01
- Wedge volume [ft^3]: 510.045
- Wedge weight [tons]: 25.502
- Wedge area (joint1) [ft^2]: 560.32
- Wedge area (joint2) [ft^2]: 702.34
- Wedge area (slope) [ft^2]: 359.22
- Wedge area (upper face) [ft^2]: 11.72

Effective Normal and Strength Properties:

	Joint 1	**Joint 2**
Effective Normal force [tons]	10.20	11.61
Effective Normal stress [t/ft^2]	0.02	0.02
Shear Strength [t/ft^2]	0.11	0.11
Strength due to Waviness [t/ft^2]	0.00	0.00

- Driving force [tons]: 23.25
- Resisting force [tons]: 141.54

Water Pressures/Forces:

	Average pressure [t/ft^2]	**Water force [tons]**
Joint 1	N/A	9.24
Joint 2	N/A	11.58
Fissures	0.02	N/A

Seismic Force:

- Seismic force [tons]: 6.12

Failure Mode:

- Sliding on intersection line (joints 1&2)

Joint Sets 1&2 line of Intersection:

Plunge [deg]	Trend [deg]	Length [ft]
48.94	9.15	269.10

Trace Lengths:

	Slope Face [ft]	Upper Face [ft]
Joint 1	261.64	8.57
Joint 2	261.47	9.28

Persistence:

- Joint 1 [ft]: 269.10
- Joint 2 [ft]: 269.10

Intersection Angles:

	Slope Face	Upper Face
Joint 1 & Joint 2	0.60	17.14
Joint 1 & Crest	86.26	96.24
Joint 2 & Crest	93.14	66.62

Dip and Dip Direction:

	Dip [deg]	Dip Direction [deg]
Joint Set 1	89.00	98.00
Joint Set 2	69.00	73.00
Slope	50.00	15.00
Upper Face	20.00	15.00

Joint Set 1 Data:

- Cohesion [t/ft^2]: 0.10
- Friction Angle [deg]: 35.00

Joint Set 2 Data:

- Cohesion [t/ft^2]: 0.10
- Friction Angle [deg]: 35.00

Slope Data:

- Slope height [ft]: 200.00
- Rock unit weight [t/ft^3]: 0.05
- Water pressures in the slope: YES
- Overhanging slope face: NO
- Externally applied force: NO
- Tension crack: NO

Water Pressure Data:

- Water unit weight [t/ft^3]: 0.031
- Pressure definition method: Percent Filled Fissures

- Percent Filled: 25.000 %

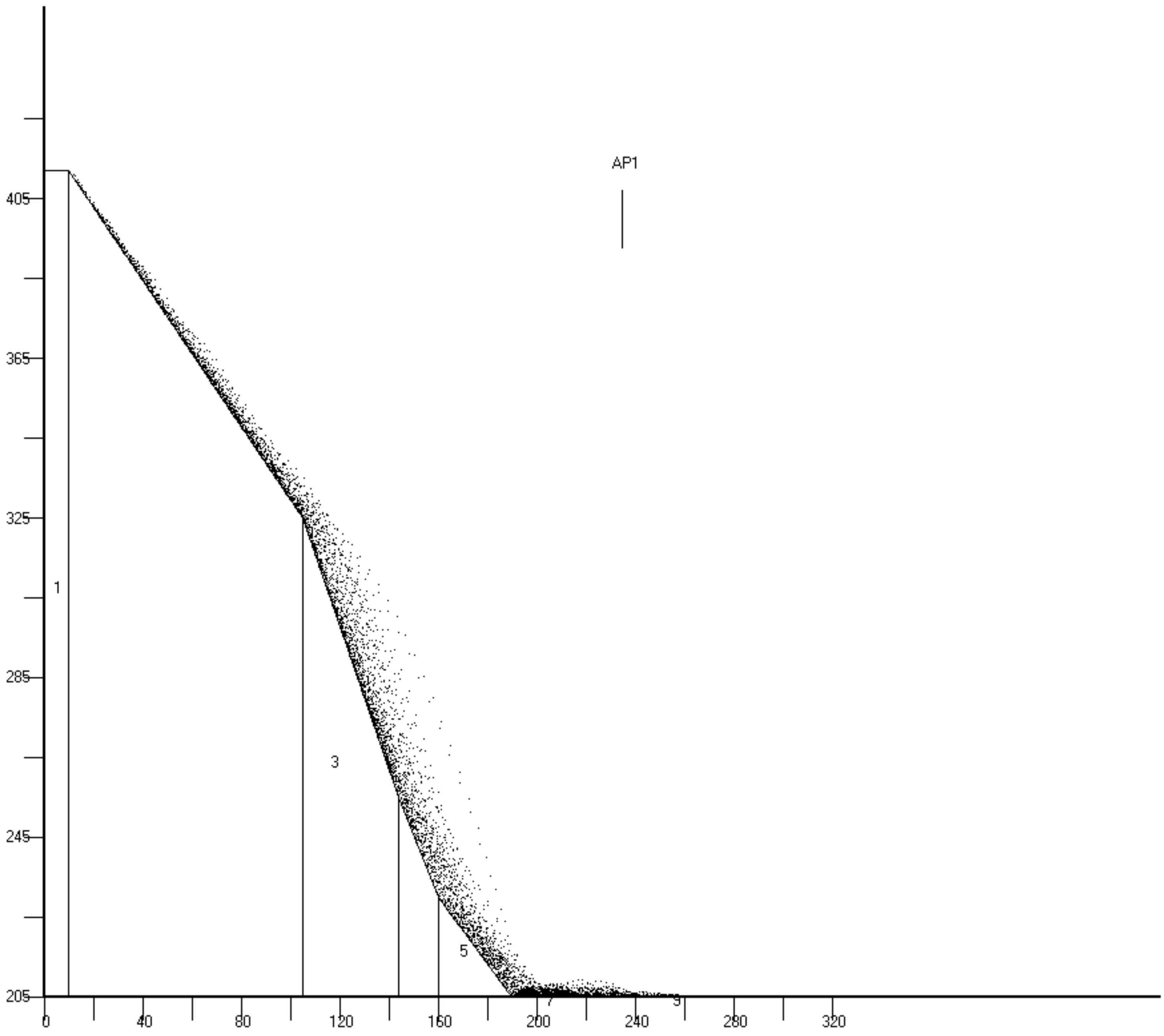
Seismic Data:

- Direction: line of intersection J1&J2 but horizontal
- Seismic coefficient: 0.240
- Trend [deg]: 9.15, Plunge [deg]: -0.00

Wedge Vertices:

- Coordinates in Easting,Northing,Up Format
- 1=Joint1, 2=Joint2, 3=Upper Face, 4=Slope

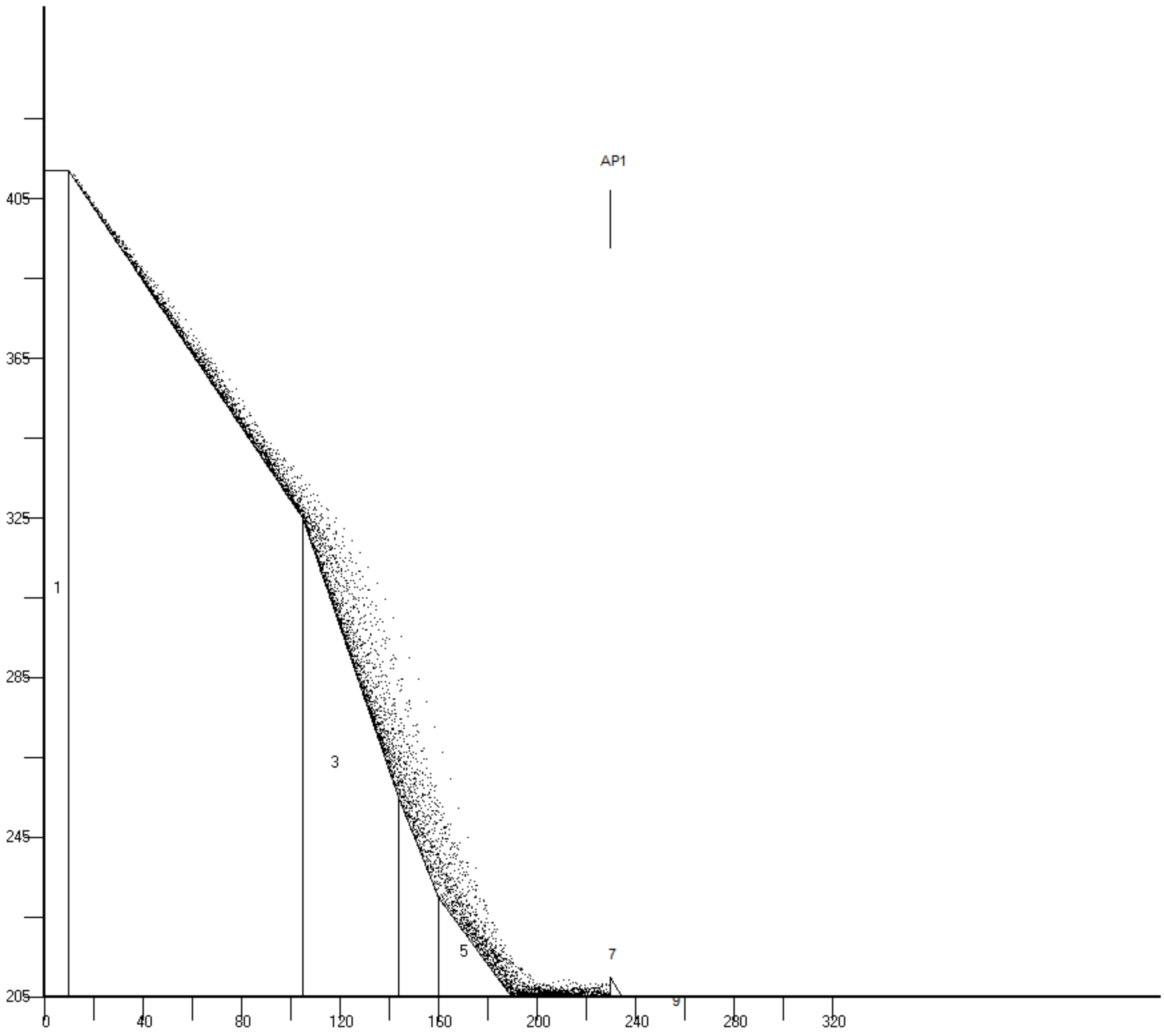
Point	x	y	z
124	0.000	0.000	0.000
134	-26.929	-166.524	200.000
234	-29.587	-165.812	200.000
123	-28.101	-174.498	202.914


AP1
405
365
325
285
245
205
0
40
80
120
160
200
240
280
320
1
3
5
7
9

```
CRSP Input File -\\psf\Home\Desktop\Projects\California Rock Slope\Flat Ditch
Results.doc


Input File Specifications

Units of Measure:  U.S.
Total Number of Cells:  9
Analysis Point 1 X-Coordinate:  235
Analysis Point 2 X-Coordinate:  0
Analysis Point 3 X-Coordinate:  0
Initial Y-Top Starting Zone Coordinate:  410
Initial Y-Base Starting Zone Coordinate:  265

Remarks:  Calibration run


Cell Data

Cell No.  S.R.  Tang. C.  Norm. C.   Begin X     Begin Y     End X     End Y

 1     1      .75           .25           0             412     10     412
 2     1      .85           .3            10            412     105     325
 3     1      .85           .3            105           325     144     255
 4     1.25   .85           .3            144           255     160     230
 5     1.25   .75           .2            160           230     190     205
 6     1.25   .75           .2            190           205     200     205
 7     1.25   .75           .2            200           205     210     205
 8     1.25   .75           .2            210           205     235     205
 9     1.25   .75           .2            235           205     310     205


CRSP Simulation Specifications:  Used with
\\psf\Home\Desktop\Projects\California Rock Slope\Flat Ditch Results.doc


Total Number of Rocks Simulated:  100
Starting Velocity in X-Direction:  1 ft/sec
Starting Velocity in Y-Direction:  -1 ft/sec
Starting Cell Number:  1
Ending Cell Number:  9
Rock Density:  155 lb/ft^3
Rock Shape:  Spherical
Diameter:  3 ft



CRSP Analysis Point 1 Data - \\psf\Home\Desktop\Projects\California Rock
Slope\Flat Ditch Results.doc


Analysis Point 1: X =  235, Y =  205

Total Rocks Passing Analysis Point:  21
```

```
Cumulative Probability      Velocity (ft/sec)      Energy (ft-lb)      Bounce
Ht. (ft)

         50%                  14.4         11582         0.12
         75%                  18.15        17221         5.73
         90%                  21.51        22293         10.78
         95%                  23.53        25338         13.81
         98%                  25.8         28755         17.21


Velocity (ft/sec)           Bounce Height (ft)          Kinetic Energy (ft-lb)

Maximum:  25.75             Maximum:  2.06                Maximum:  32672
Average:  14.4              Average:  .42   Average:  11582
Minimum:  3.96              G. Mean:  .12   Std. Dev.:  8351
Std. Dev.:  5.54            Std. Dev.:  8.31


Remarks:  Calibration run



CRSP Data Collected at End of Each Cell - \\psf\Home\Desktop\Projects\California
Rock Slope\Flat Ditch Results.doc


Velocity Units: ft/sec      Bounce Height Units: ft


Cell #   Max. Vel.   Avg. Vel.   S.D. Vel.   Max. Bounce Ht.   Avg. Bounce Ht.

 1            No rocks      past end of cell
 2             55            32            11.37         10            3
 3             88            53            16.35         44            9
 4             91            57            16.1          47            9
 5             98            46            13.16         13            4
 6             62            24            12.71         4             0
 7             36            18            8.18          3             0
 8             26            14            5.54          2             0
 9            No rocks      past end of cell



CRSP Rocks Stopped Data - \\psf\Home\Desktop\Projects\California Rock Slope\Flat
Ditch Results.doc

                              X Interval                   Rocks Stopped

                             0 To  10 ft                    0
                             10 To  20 ft                    0
                             20 To  30 ft                    0
```

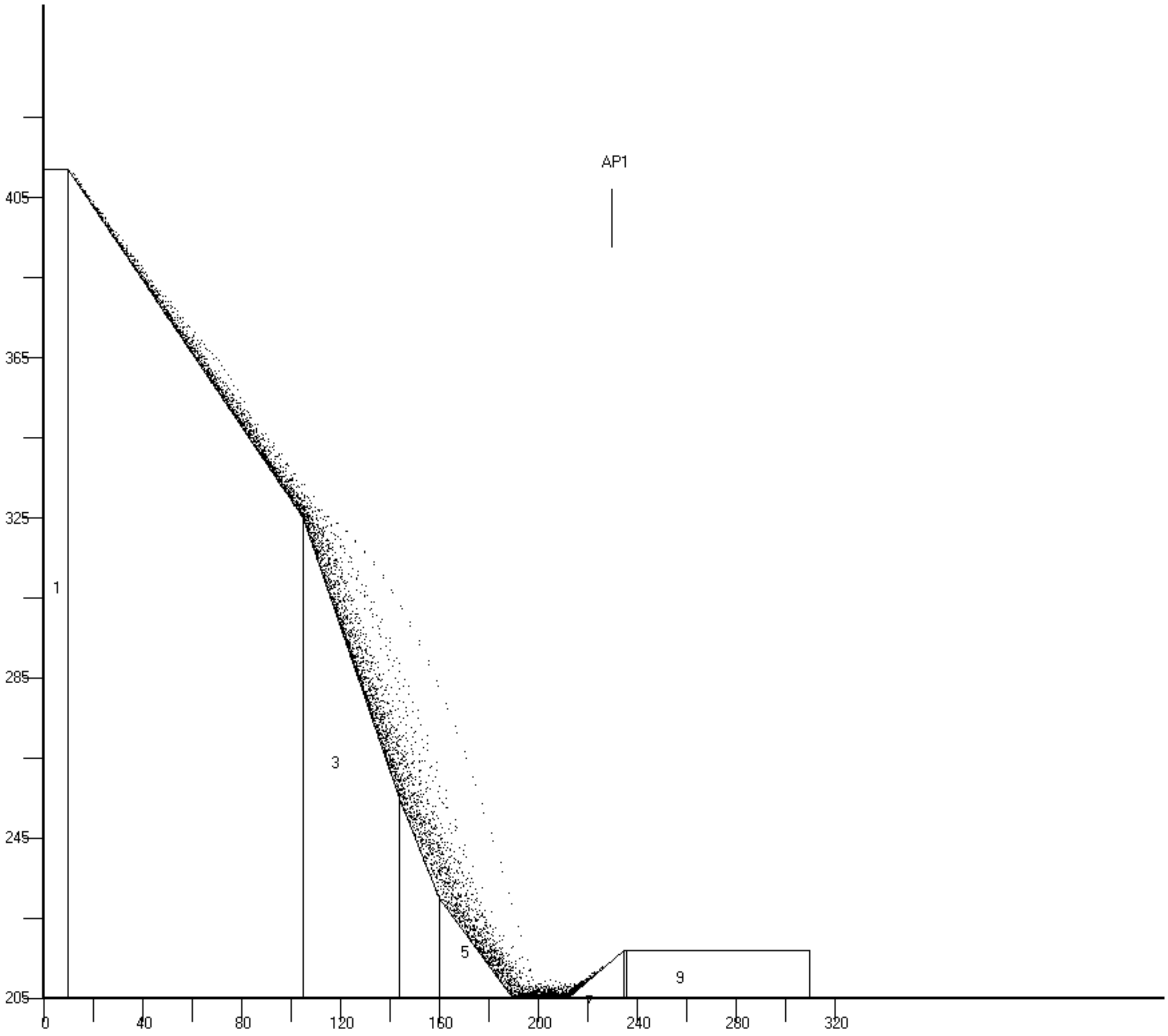
30 To 40 ft	0
40 To 50 ft	0
50 To 60 ft	0
60 To 70 ft	0
70 To 80 ft	0
80 To 90 ft	0
90 To 100 ft	0
100 To 110 ft	0
110 To 120 ft	0
120 To 130 ft	0
130 To 140 ft	0
140 To 150 ft	0
150 To 160 ft	0
160 To 170 ft	0
170 To 180 ft	0
180 To 190 ft	0
190 To 200 ft	10
200 To 210 ft	25
210 To 220 ft	26
220 To 230 ft	12
230 To 240 ft	10
240 To 250 ft	12
250 To 260 ft	3
260 To 270 ft	2
270 To 280 ft	0
280 To 290 ft	0
290 To 300 ft	0
300 To 310 ft	0


AP1
405
365
325
285
245
205
0
40
80
120
160
200
240
280
320
1
3
5
7
9

```
CRSP Input File -\\psf\Home\Desktop\Projects\California Rock Slope\CRSP Torrence
Barrier at 230.dat


Input File Specifications

Units of Measure:  U.S.
Total Number of Cells:  9
Analysis Point 1 X-Coordinate:  230
Analysis Point 2 X-Coordinate:  0
Analysis Point 3 X-Coordinate:  0
Initial Y-Top Starting Zone Coordinate:  410
Initial Y-Base Starting Zone Coordinate:  265

Remarks:  Berm Run


Cell Data

Cell No.  S.R.  Tang. C.  Norm. C.   Begin X     Begin Y     End X     End Y

 1     1      .75           .25           0             412     10     412
 2     1      .85           .3            10            412     105     325
 3     1      .85           .3            105           325     144     255
 4     1.25   .85           .3            144           255     160     230
 5     1.25   .75           .2            160           230     190     205
 6     1.25   .75           .2            190           205     230     205
 7     1.25   .75           .2            230           205     230.1     210
 8     1.25   .75           .2            230.1         210     235     205
 9     1.25   .75           .2            235           205     310     205



CRSP Simulation Specifications:  Used with
\\psf\Home\Desktop\Projects\California Rock Slope\CRSP Torrence Barrier at
230.dat


Total Number of Rocks Simulated:  100
Starting Velocity in X-Direction:  1 ft/sec
Starting Velocity in Y-Direction:  -1 ft/sec
Starting Cell Number:  1
Ending Cell Number:  9
Rock Density:  155 lb/ft^3
Rock Shape:  Spherical
Diameter:  3 ft



CRSP Analysis Point 1 Data - \\psf\Home\Desktop\Projects\California Rock
Slope\CRSP Torrence Barrier at 230.dat


Analysis Point 1: X =  230, Y =  205
```

```
Total Rocks Passing Analysis Point:  20


Cumulative Probability      Velocity (ft/sec)      Energy (ft-lb)      Bounce
Ht. (ft)

         50%                  14.05        11590         0.4
         75%                  17.92        17793         4.69
         90%                  21.4         23372         8.55
         95%                  23.49        26722         10.87
         98%                  25.83        30481         13.46



Velocity (ft/sec)           Bounce Height (ft)          Kinetic Energy (ft-lb)

Maximum:  28.79             Maximum:  2.96                Maximum:  39636
Average:  14.05             Average:  .89   Average:  11590
Minimum:  5.67              G. Mean:  .4    Std. Dev.:  9186
Std. Dev.:  5.73            Std. Dev.:  6.35


Remarks:  Berm Run



CRSP Data Collected at End of Each Cell - \\psf\Home\Desktop\Projects\California
Rock Slope\CRSP Torrence Barrier at 230.dat


Velocity Units: ft/sec      Bounce Height Units: ft


Cell #   Max. Vel.   Avg. Vel.   S.D. Vel.   Max. Bounce Ht.   Avg. Bounce Ht.

 1            No rocks      past end of cell
 2             51            32            10.99         10            2
 3             82            51            16.24         42           10
 4             87            55            15.07         40           10
 5             82            46            12.5          10            3
 6             29            14            5.73          3             0
 7            No rocks      past end of cell
 8            No rocks      past end of cell
 9            No rocks      past end of cell



CRSP Rocks Stopped Data - \\psf\Home\Desktop\Projects\California Rock Slope\CRSP
Torrence Barrier at 230.dat


                              X Interval                   Rocks Stopped

                             0 To  10 ft                    0
                             10 To  20 ft                    0
```

20 To 30 ft	0
30 To 40 ft	0
40 To 50 ft	0
50 To 60 ft	0
60 To 70 ft	0
70 To 80 ft	0
80 To 90 ft	0
90 To 100 ft	0
100 To 110 ft	0
110 To 120 ft	0
120 To 130 ft	0
130 To 140 ft	0
140 To 150 ft	0
150 To 160 ft	0
160 To 170 ft	0
170 To 180 ft	0
180 To 190 ft	0
190 To 200 ft	14
200 To 210 ft	26
210 To 220 ft	29
220 To 230 ft	24
230 To 240 ft	7
240 To 250 ft	0
250 To 260 ft	0
260 To 270 ft	0
270 To 280 ft	0
280 To 290 ft	0
290 To 300 ft	0
300 To 310 ft	0

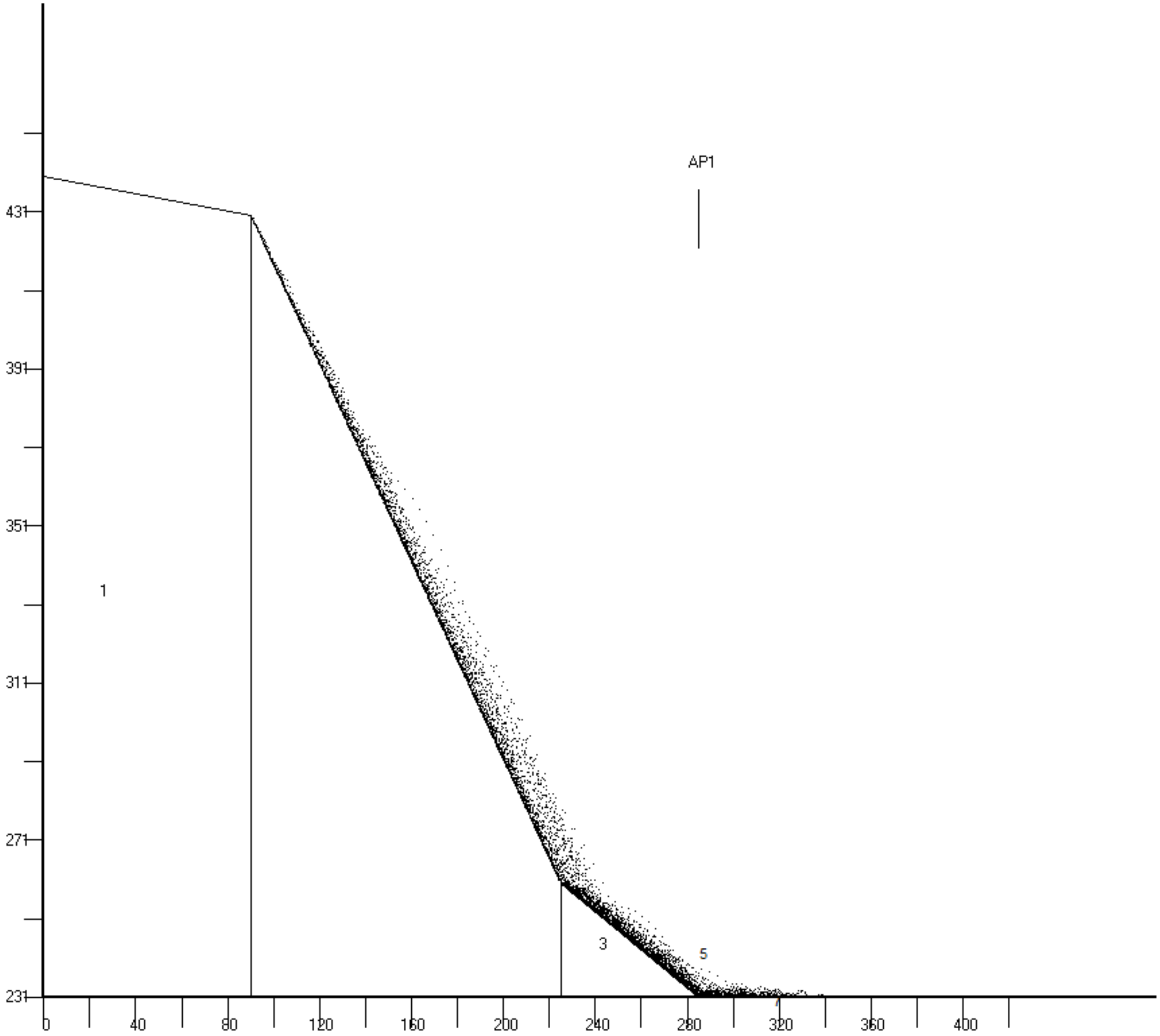

AP1
405
365
325
285
245
205
0
40
80
120
160
200
240
280
320
1
3
5
7
9

```
CRSP Input File -\\psf\Home\Desktop\Projects\California Rock Slope\10 foot 2 to
1 to el 215 from 205.dat


Input File Specifications

Units of Measure:  U.S.
Total Number of Cells:  9
Analysis Point 1 X-Coordinate:  230
Analysis Point 2 X-Coordinate:  0
Analysis Point 3 X-Coordinate:  0
Initial Y-Top Starting Zone Coordinate:  410
Initial Y-Base Starting Zone Coordinate:  265

Remarks:  Berm Run


Cell Data

Cell No.  S.R.  Tang. C.  Norm. C.   Begin X     Begin Y     End X     End Y

 1     1      .75           .25           0             412     10     412
 2     1      .85           .3            10            412     105     325
 3     1      .85           .3            105           325     144     255
 4     1.25   .85           .3            144           255     160     230
 5     1.25   .75           .2            160           230     190     205
 6     1.25   .75           .2            190           205     212     205
 7     1.25   .75           .2            212           205     235     217
 8     1.25   .75           .2            235           217     236     217
 9     1.25   .75           .2            236           217     310     217



CRSP Simulation Specifications:  Used with
\\psf\Home\Desktop\Projects\California Rock Slope\10 foot 2 to 1 to el 215 from
205.dat


Total Number of Rocks Simulated:  100
Starting Velocity in X-Direction:  1 ft/sec
Starting Velocity in Y-Direction:  -1 ft/sec
Starting Cell Number:  1
Ending Cell Number:  9
Rock Density:  155 lb/ft^3
Rock Shape:  Spherical
Diameter:  3 ft



CRSP Analysis Point 1 Data - \\psf\Home\Desktop\Projects\California Rock
Slope\10 foot 2 to 1 to el 215 from 205.dat


Analysis Point 1: X =  230, Y =  214
```

NO ROCKS PAST ANALSYSIS POINT 1

```
CRSP Data Collected at End of Each Cell - \\psf\Home\Desktop\Projects\California
Rock Slope\10 foot 2 to 1 to el 215 from 205.dat


Velocity Units: ft/sec      Bounce Height Units: ft


Cell #   Max. Vel.   Avg. Vel.   S.D. Vel.   Max. Bounce Ht.   Avg. Bounce Ht.

 1            No rocks      past end of cell
 2             50            33            11.42         9             2
 3             84            52            16.1          49            10
 4             89            56            15.64         53            11
 5             89            46            13.6          23            3
 6             30            17            6.68          4             0
 7            No rocks      past end of cell
 8            No rocks      past end of cell
 9            No rocks      past end of cell



CRSP Rocks Stopped Data - \\psf\Home\Desktop\Projects\California Rock Slope\10
foot 2 to 1 to el 215 from 205.dat

                              X Interval                   Rocks Stopped

                             0 To  10 ft                    0
                             10 To  20 ft                    0
                             20 To  30 ft                    0
                             30 To  40 ft                    0
                             40 To  50 ft                    0
                             50 To  60 ft                    0
                             60 To  70 ft                    0
                             70 To  80 ft                    0
                             80 To  90 ft                    0
                             90 To  100 ft                    0
                             100 To  110 ft                    0
                             110 To  120 ft                    0
                             120 To  130 ft                    0
                             130 To  140 ft                    0
                             140 To  150 ft                    0
                             150 To  160 ft                    0
                             160 To  170 ft                    0
                             170 To  180 ft                    0
                             180 To  190 ft                    0
                             190 To  200 ft                    8
                             200 To  210 ft                    23
```

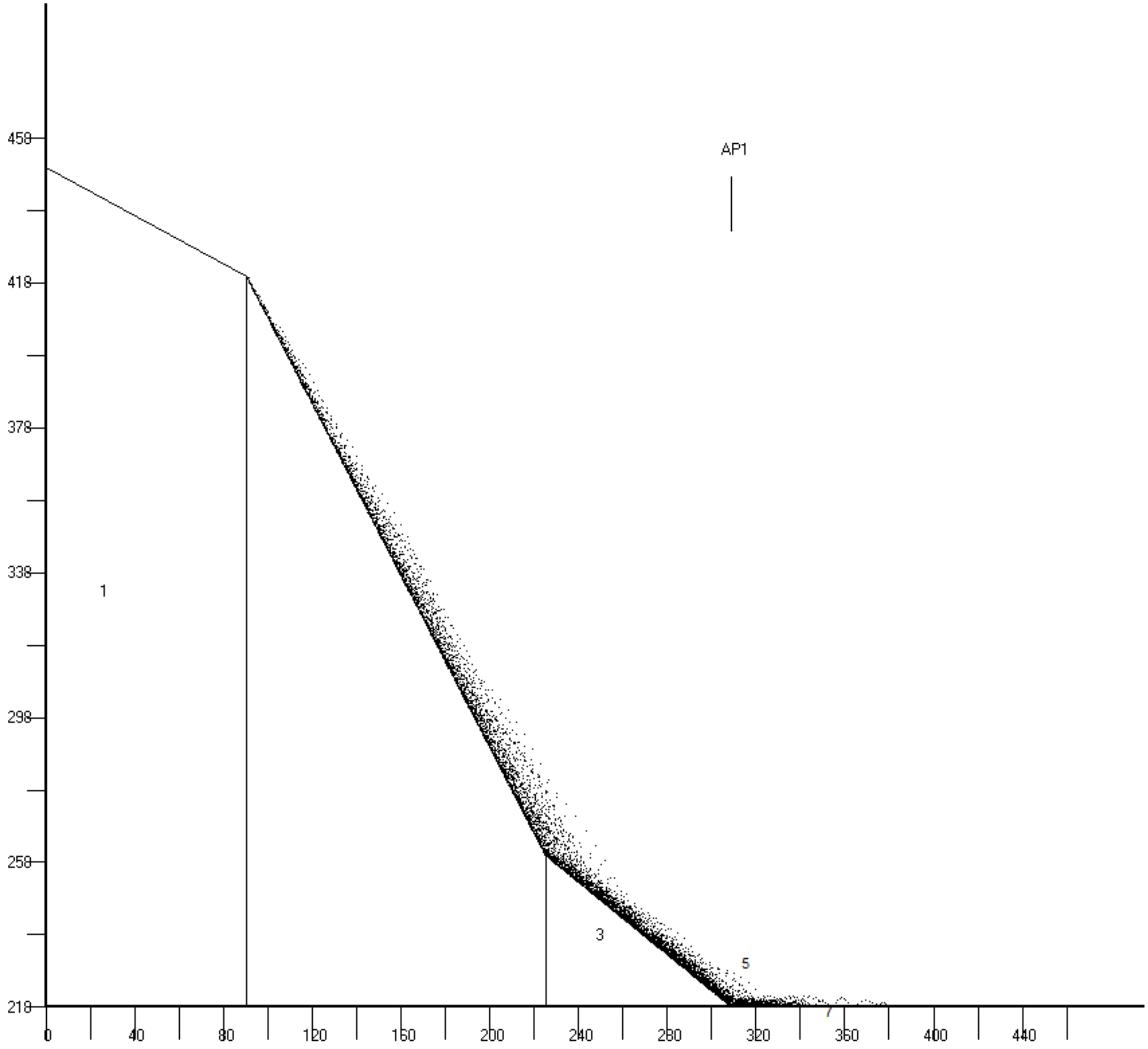
210 To 220 ft	58
220 To 230 ft	11
230 To 240 ft	0
240 To 250 ft	0
250 To 260 ft	0
260 To 270 ft	0
270 To 280 ft	0
280 To 290 ft	0
290 To 300 ft	0
300 To 310 ft	0


AP1
1
3
5
7
431
391
351
311
271
231
0
40
80
120
160
200
240
280
320
360
400

```
CRSP Input File -\\psf\Home\Desktop\Projects\California Rock Slope\Final
Push\Section C to C prime.dat


Input File Specifications

Units of Measure:  U.S.
Total Number of Cells:  7
Analysis Point 1 X-Coordinate:  285
Analysis Point 2 X-Coordinate:
Analysis Point 3 X-Coordinate:
Initial Y-Top Starting Zone Coordinate:  430
Initial Y-Base Starting Zone Coordinate:  260

Remarks:  Section C-C'


Cell Data

Cell No.  S.R.  Tang. C.  Norm. C.   Begin X     Begin Y     End X     End Y

 1     1      .75           .2            0             440     90     430
 2     1      .8            .25           90            430     225     260
 3     1.25   .75           .2            225           260     285     231
 4     1.25   .7            .2            285           231     286     231
 5     1.25   .7            .2            286           231     287     231
 6     1.25   .7            .2            287           231     288     231
 7     1.25   .7            .2            288           231     400     231



CRSP Simulation Specifications:  Used with
\\psf\Home\Desktop\Projects\California Rock Slope\Final Push\Section C to C
prime.dat


Total Number of Rocks Simulated:  100
Starting Velocity in X-Direction:  1 ft/sec
Starting Velocity in Y-Direction:  -1 ft/sec
Starting Cell Number:  1
Ending Cell Number:  7
Rock Density:  85 lb/ft^3
Rock Shape:  Spherical
Diameter:  2.5 ft



CRSP Analysis Point 1 Data - \\psf\Home\Desktop\Projects\California Rock
Slope\Final Push\Section C to C prime.dat


Analysis Point 1: X =  285, Y =  231

Total Rocks Passing Analysis Point:  87
```

```
Cumulative Probability      Velocity (ft/sec)      Energy (ft-lb)      Bounce
Ht. (ft)

         50%                  25.48        10865          0.79
         75%                  33.21        16401          4.37
         90%                  40.16        21379          7.58
         95%                  44.33        24368          9.51
         98%                  49.02        27723          11.68


Velocity (ft/sec)           Bounce Height (ft)          Kinetic Energy (ft-lb)

Maximum:  51.75             Maximum:  7.13                Maximum:  37182
Average:  25.48             Average:  1.61                Average:  10865
Minimum:  4.17              G. Mean:  .79   Std. Dev.:  8197
Std. Dev.:  11.45           Std. Dev.:  5.29


Remarks:  Section C-C'


CRSP Data Collected at End of Each Cell - \\psf\Home\Desktop\Projects\California
Rock Slope\Final Push\Section C to C prime.dat


Velocity Units: ft/sec      Bounce Height Units: ft


Cell #   Max. Vel.   Avg. Vel.   S.D. Vel.   Max. Bounce Ht.   Avg. Bounce Ht.

 1            No rocks      past end of cell
 2             82            45             16.2          20            6
 3             52            25             11.45         7             1
 4             48            24             12            7             1
 5             49            22             12.01         7             1
 6             49            20             11.12         7             0
 7            No rocks      past end of cell


CRSP Rocks Stopped Data - \\psf\Home\Desktop\Projects\California Rock
Slope\Final Push\Section C to C prime.dat

                              X Interval                     Rocks Stopped

                             0 To  10 ft                       1
                             10 To  20 ft                       0
                             20 To  30 ft                       0
                             30 To  40 ft                       0
                             40 To  50 ft                       0
                             50 To  60 ft                       0
```

60 To 70 ft	0
70 To 80 ft	0
80 To 90 ft	0
90 To 100 ft	0
100 To 110 ft	0
110 To 120 ft	0
120 To 130 ft	0
130 To 140 ft	0
140 To 150 ft	0
150 To 160 ft	0
160 To 170 ft	0
170 To 180 ft	0
180 To 190 ft	0
190 To 200 ft	0
200 To 210 ft	0
210 To 220 ft	0
220 To 230 ft	0
230 To 240 ft	0
240 To 250 ft	2
250 To 260 ft	3
260 To 270 ft	2
270 To 280 ft	1
280 To 290 ft	24
290 To 300 ft	32
300 To 310 ft	15
310 To 320 ft	11
320 To 330 ft	5
330 To 340 ft	2
340 To 350 ft	2
350 To 360 ft	0
360 To 370 ft	0
370 To 380 ft	0
380 To 390 ft	0
390 To 400 ft	0


AP1
458
418
378
338
298
258
218
0
40
80
120
160
200
240
280
320
360
400
440
1
3
5
7

```
CRSP Input File -\\psf\Home\Desktop\Projects\California Rock Slope\Final
Push\Section A to A prime.bmp


Input File Specifications

Units of Measure:  U.S.
Total Number of Cells:  7
Analysis Point 1 X-Coordinate:  309
Analysis Point 2 X-Coordinate:
Analysis Point 3 X-Coordinate:
Initial Y-Top Starting Zone Coordinate:  420
Initial Y-Base Starting Zone Coordinate:  260

Remarks:  Section A-A'


Cell Data

Cell No.  S.R.  Tang. C.  Norm. C.   Begin X     Begin Y     End X     End Y

 1     1      .75           .25           0             450     90     420
 2     1      .80           .3            90            420     225     260
 3     1      .7            .2            225           260     309     218
 4     1.25   .7            .2            309           218     314     218
 5     1.25   .7            .2            314           218     315     218
 6     1.25   .7            .2            315           218     316     218
 7     1.25   .7            .2            316           218     450     218



CRSP Simulation Specifications:  Used with
\\psf\Home\Desktop\Projects\California Rock Slope\Final Push\Section A to A
prime.bmp


Total Number of Rocks Simulated:  100
Starting Velocity in X-Direction:  1 ft/sec
Starting Velocity in Y-Direction:  -1 ft/sec
Starting Cell Number:  1
Ending Cell Number:  7
Rock Density:  85 lb/ft^3
Rock Shape:  Spherical
Diameter:  2.5 ft



CRSP Analysis Point 1 Data - \\psf\Home\Desktop\Projects\California Rock
Slope\Final Push\Section A to A prime.bmp


Analysis Point 1: X =  309, Y =  218

Total Rocks Passing Analysis Point:  99
```

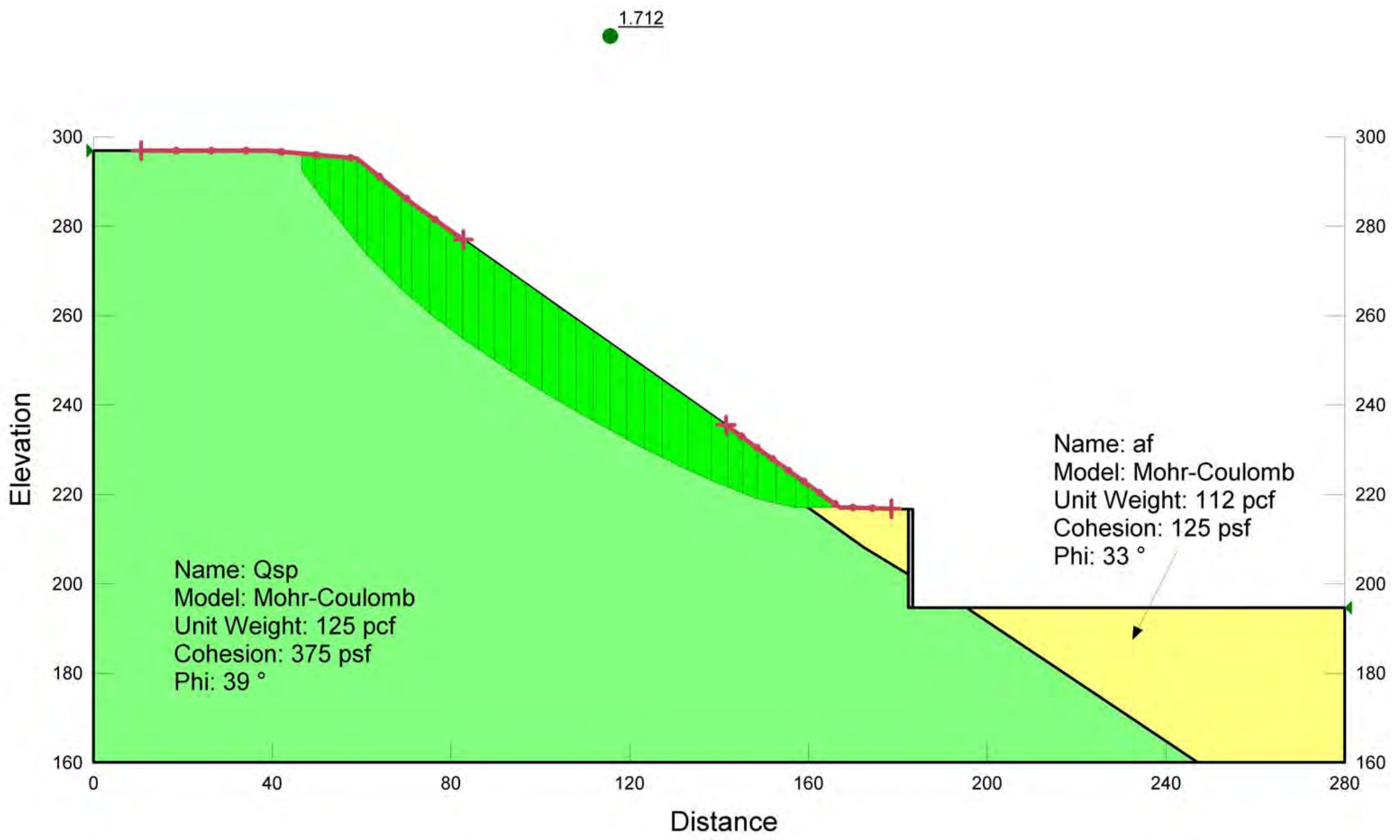
```
Cumulative Probability      Velocity (ft/sec)      Energy (ft-lb)      Bounce
Ht. (ft)

         50%                  26.89        11670         0.82
         75%                  33.59        17322         4.94
         90%                  39.61        22405         8.64
         95%                  43.23        25457         10.86
         98%                  47.29        28882         13.36


Velocity (ft/sec)           Bounce Height (ft)          Kinetic Energy (ft-lb)

Maximum:  55.02             Maximum:  9.53                Maximum:  44477
Average:  26.89             Average:  1.82                Average:  11670
Minimum:  8.11              G. Mean:  .82   Std. Dev.:  8370
Std. Dev.:  9.92            Std. Dev.:  6.1


Remarks:  Section A-A'


CRSP Data Collected at End of Each Cell - \\psf\Home\Desktop\Projects\California
Rock Slope\Final Push\Section A to A prime.bmp


Velocity Units: ft/sec      Bounce Height Units: ft


Cell #   Max. Vel.   Avg. Vel.   S.D. Vel.   Max. Bounce Ht.   Avg. Bounce Ht.

 1            No rocks      past end of cell
 2             74            45             16.34         21            5
 3             55            27             9.92          10            1
 4             54            20             11.64         8             0
 5             55            20             11.79         7             0
 6             45            20             10.69         7             0
 7            No rocks      past end of cell


CRSP Rocks Stopped Data - \\psf\Home\Desktop\Projects\California Rock
Slope\Final Push\Section A to A prime.bmp

                              X Interval                    Rocks Stopped

                             0 To  10 ft                     1
                             10 To  20 ft                     0
                             20 To  30 ft                     0
                             30 To  40 ft                     0
                             40 To  50 ft                     0
                             50 To  60 ft                     0
```

60 To 70 ft	0
70 To 80 ft	0
80 To 90 ft	0
90 To 100 ft	0
100 To 110 ft	0
110 To 120 ft	0
120 To 130 ft	0
130 To 140 ft	0
140 To 150 ft	0
150 To 160 ft	0
160 To 170 ft	0
170 To 180 ft	0
180 To 190 ft	0
190 To 200 ft	0
200 To 210 ft	0
210 To 220 ft	0
220 To 230 ft	0
230 To 240 ft	0
240 To 250 ft	0
250 To 260 ft	0
260 To 270 ft	0
270 To 280 ft	0
280 To 290 ft	0
290 To 300 ft	0
300 To 310 ft	0
310 To 320 ft	33
320 To 330 ft	31
330 To 340 ft	17
340 To 350 ft	7
350 To 360 ft	8
360 To 370 ft	1
370 To 380 ft	1
380 To 390 ft	1
390 To 400 ft	0
400 To 410 ft	0
410 To 420 ft	0
420 To 430 ft	0
430 To 440 ft	0
440 To 450 ft	0

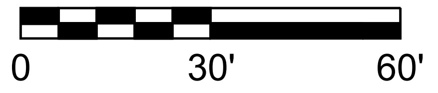
APPENDIX E

APPENDIX E

SLOPE STABILITY



SLOPE 1
SECTION C-C'
STATIC



GEOCON
WEST, INC.

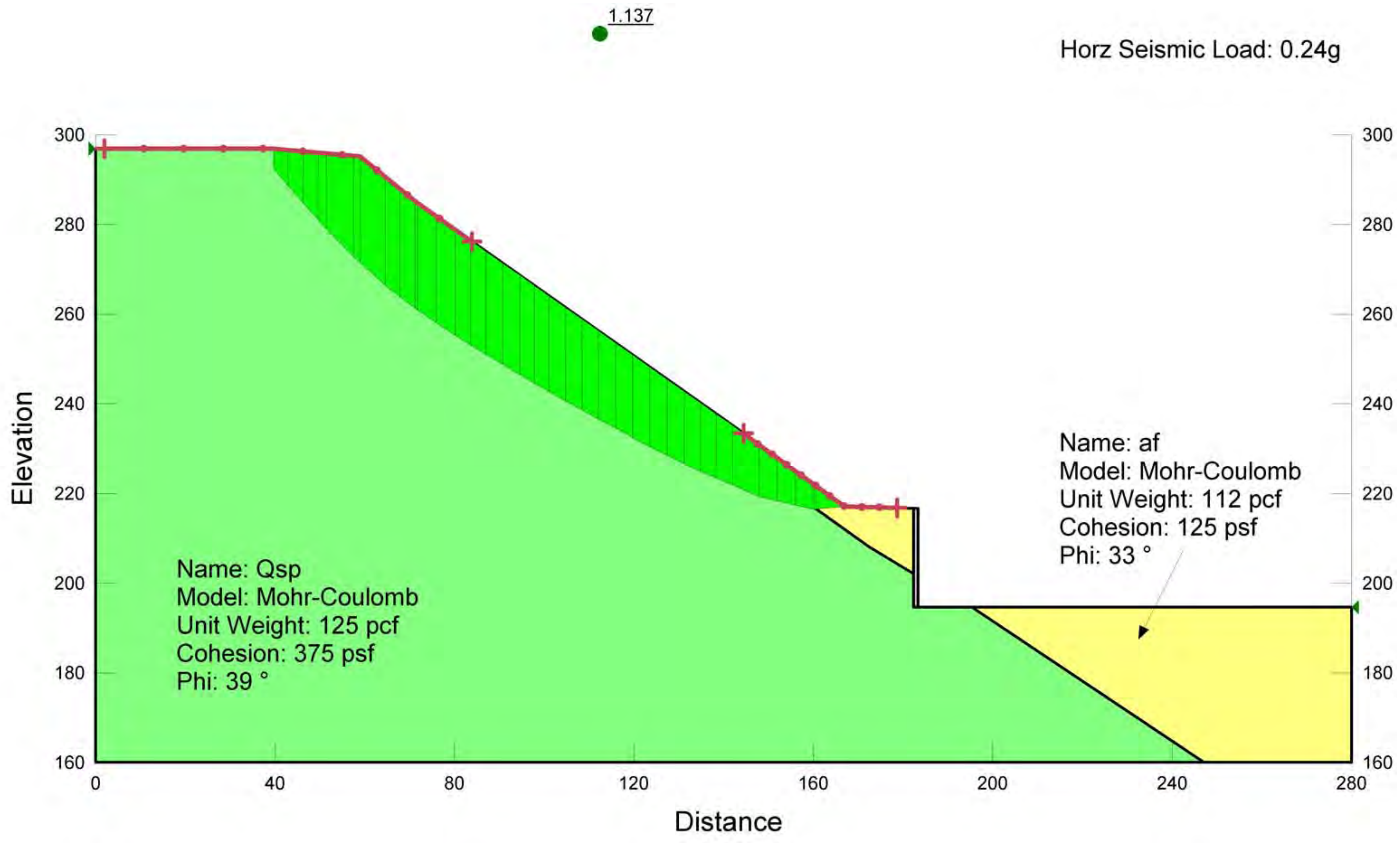
ENVIRONMENTAL GEOTECHNICAL MATERIALS
3303 N. SAN FERNANDO BLVD. - SUITE 100 - BURBANK, CA 91504
PHONE (818) 841-8388 - FAX (818) 841-1704

JMT		8000

SLOPE STABILITY ANALYSIS

MKS RESIDENTIAL
HAWTHORNE & VIA VALMONTE
TORRANCE, CA 90505

MARCH 2016	PROJECT NO. A9201-06-01C	FIG. E1



SLOPE 1
SECTION C-C'
PSUEDO-STATIC

0 30' 60'

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WEST, INC.

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3303 N. SAN FERNANDO BLVD. - SUITE 100 - BURBANK, CA 91504
PHONE (818) 841-8388 - FAX (818) 841-1704

JMT		8000

SLOPE STABILITY ANALYSIS

MKS RESIDENTIAL
HAWTHORNE & VIA VALMONTE
TORRANCE, CA 90505

MARCH 2016	PROJECT NO. A9201-06-01C	FIG. E2

PROJECT NO. A9201-06-01C

Slope 1 @ Section C-C’ - Artificial Fill (af)

ASSUMED CONDITIONS:

Slope Height	H	=	Infinite	
Depth of Saturation	Z	=	4	feet
Slope Inclination	2:1	(Horizontal:Vertical)		
Slope Angle	i	=	35	degrees
Unit Weight of Water	γ_w	=	62.4	pounds per cubic foot
Total Unit Weight of Soil	γ_t	=	112	pounds per cubic foot
Angle of Internal Friction	ϕ	=	33	degrees
Apparent Cohesion	C	=	125	pounds per square foot

Slope saturated to vertical depth Z below slope face.
Seepage forces parallel to slope face

ANALYSIS:

$$\text{FS} = \frac{C+(\gamma_t-\gamma_w)Z\cos^2 i\tan\phi}{\gamma_t Z\sin i\cos i} = 1.0$$

REFERENCES:

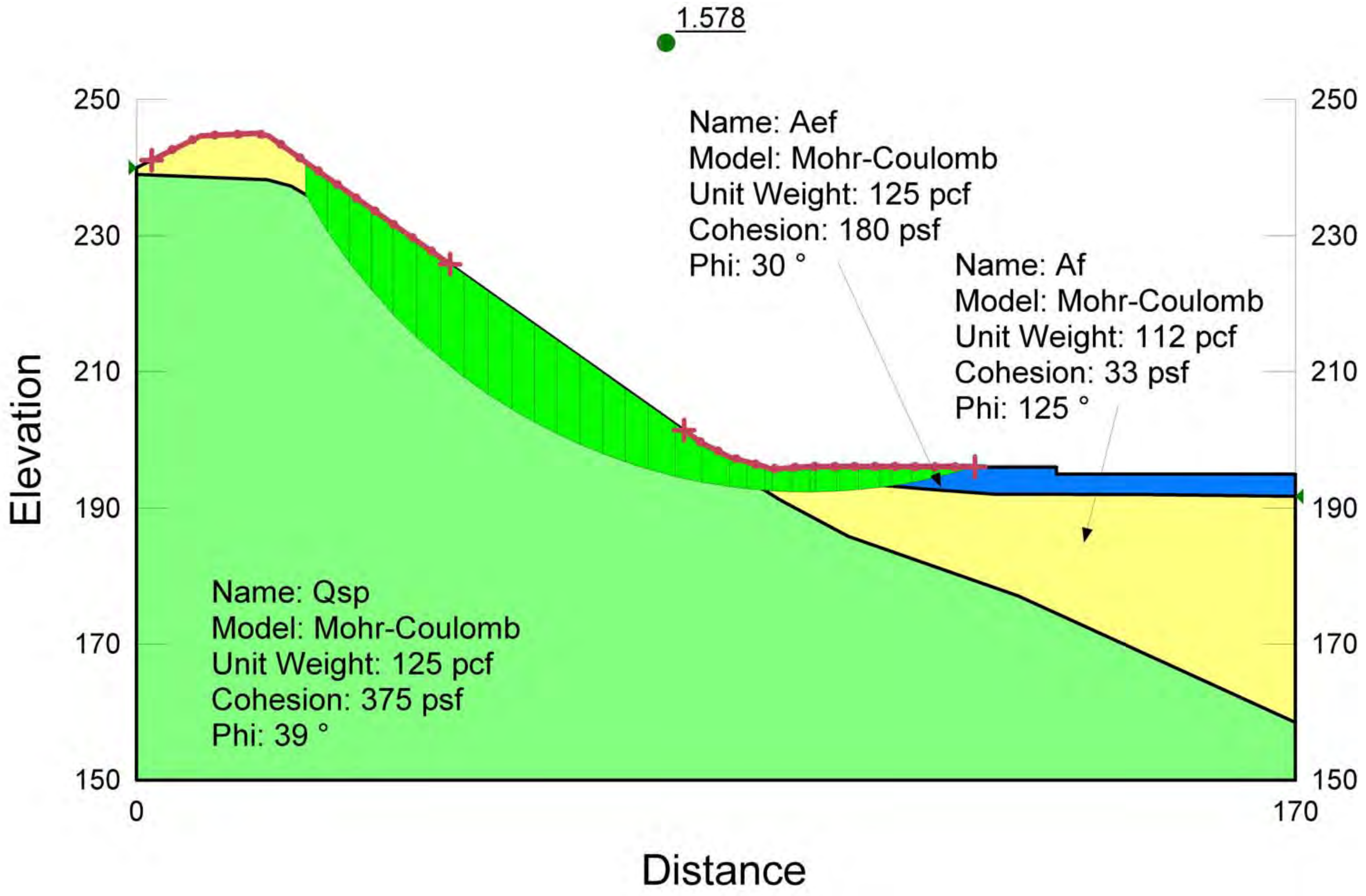
(1) Haefeli, R. *The Stability of Slopes Acted Upon by Parallel Seepage,* Proc. Second International Conference, SMFE, Rotterdam, 1948, 1, 57-62.

(2) Skempton, A. W., and F. A. Delory, *Stability of Natural Slopes in London Clay*, Proc. Fourth International Conference, SMFE, London, 1957, 2, 378-81.

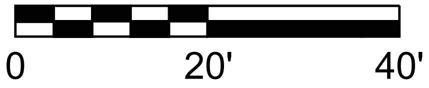
SURFICIAL SLOPE STABILITY ANALYSIS

MKS RESIDENTIAL
HAWTHORNE & VIA VALMONTE
TORRANCE, CALIFORNIA

FIGURE E3



SLOPE 1
SECTION D-D'
STATIC



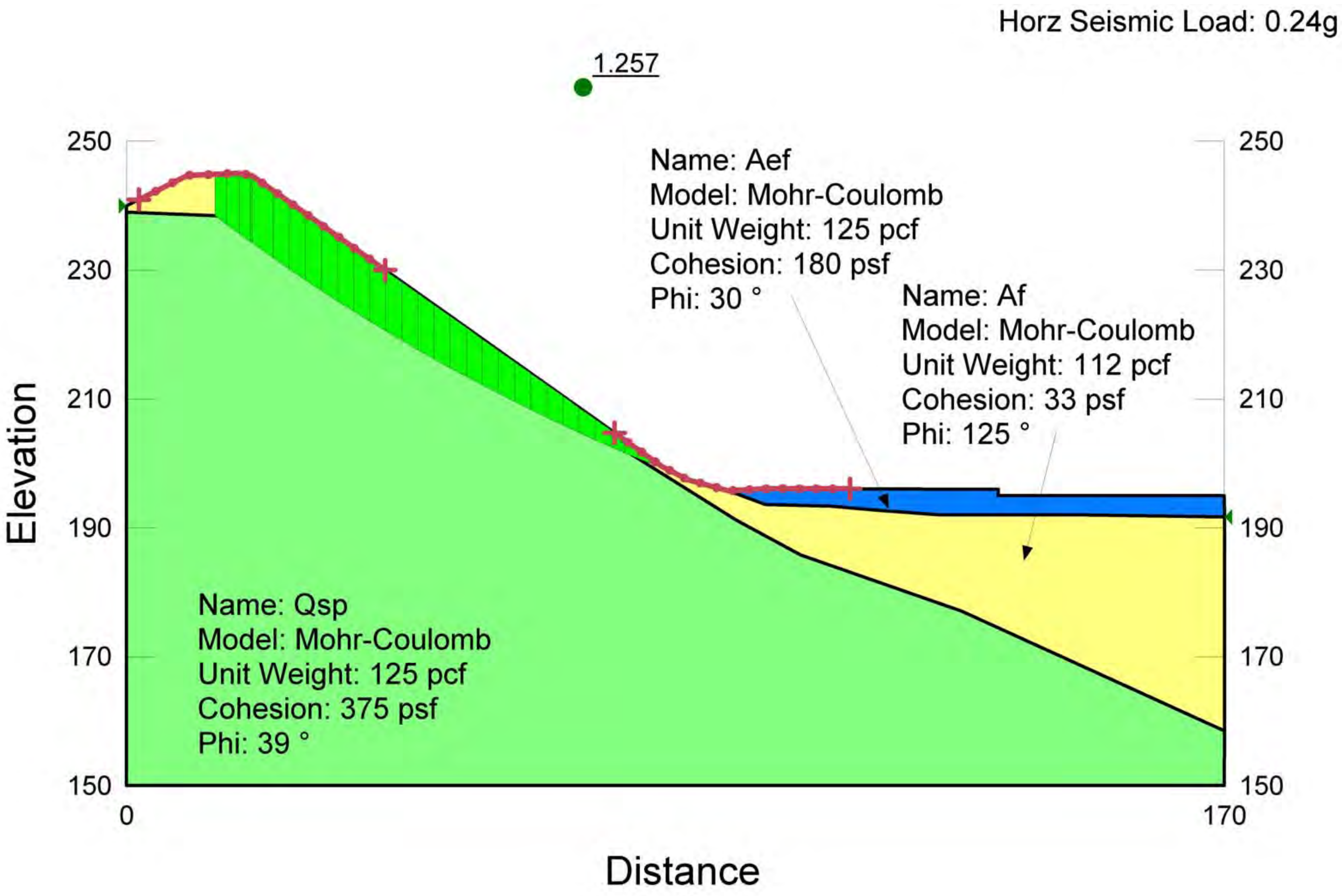
GEOCON
WEST, INC.
ENVIRONMENTAL GEOTECHNICAL MATERIALS
3303 N. SAN FERNANDO BLVD. - SUITE 100 - BURBANK, CA 91504
PHONE (818) 841-8388 - FAX (818) 841-1704

JMT		8000

SLOPE STABILITY ANALYSIS

MKS RESIDENTIAL
HAWTHORNE & VIA VALMONTE
TORRANCE, CA 90505

MARCH 2016	PROJECT NO. A9201-06-01C	FIG. E4



SLOPE 1
SECTION D-D'
PSEUDO-STATIC

0 20' 40'

GEOCON
WEST, INC.
ENVIRONMENTAL GEOTECHNICAL MATERIALS
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SLOPE STABILITY ANALYSIS

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TORRANCE, CA 90505

MARCH 2016	PROJECT NO. A9201-06-01C	FIG. E5

Slope 1 @ Section D-D’ - Artificial Fill (af)

ASSUMED CONDITIONS:

Slope Height	H	=	Infinite	
Depth of Saturation	Z	=	4	feet
Slope Inclination	2:1	(Horizontal:Vertical)		
Slope Angle	i	=	33.7	degrees
Unit Weight of Water	γ_w	=	62.4	pounds per cubic foot
Total Unit Weight of Soil	γ_t	=	112	pounds per cubic foot
Angle of Internal Friction	ϕ	=	33	degrees
Apparent Cohesion	C	=	125	pounds per square foot

Slope saturated to vertical depth Z below slope face.
Seepage forces parallel to slope face

ANALYSIS:

$$FS = \frac{C+(\gamma_t-\gamma_w)Z\cos^2 i\tan\phi}{\gamma_t Z\sin i\cos i} = 1.04$$

REFERENCES:

(1) Haefeli, R. *The Stability of Slopes Acted Upon by Parallel Seepage,* Proc. Second International Conference, SMFE, Rotterdam, 1948, 1, 57-62.

(2) Skempton, A. W., and F. A. Delory, *Stability of Natural Slopes in London Clay*, Proc. Fourth International Conference, SMFE, London, 1957, 2, 378-81.

SURFICIAL SLOPE STABILITY ANALYSIS

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FIGURE E6