GEOTECHNICAL INVESTIGATION

AVION
SAN DIEGO, CALIFORNIA

GEOCON
INCORPORATED

GEOTECHNICAL
ENVIRONMENTAL
MATERIALS

PREPARED FOR

LENNAR HOMES
SAN DIEGO, CALIFORNIA

AUGUST 24, 2018
PROJECT NO. G2213-32-01

Project No. G2213-32-01
August 24, 2018

Lennar Homes
16465 Via Esprillo, Suite 150
San Diego, California 92127

Attention: Mr. Alex Plishner

Subject: GEOTECHNICAL INVESTIGATION
AVION
SAN DIEGO, CALIFORNIA

Dear Mr. Plishner:

In accordance with your request and authorization of our Proposal No. LG-17423 dated November 20, 2017, we have performed a geotechnical investigation to address the tentative map for the subject project (previously referred to as the Debevoise Property). The accompanying report presents the findings of our study and our recommendations relative to the geotechnical aspects of developing the property as presently proposed.

The results of our study indicate that the site can be developed as planned, provided the recommendations of this report are followed. The presence of shallow hard rock in areas of planned excavation will be an important geotechnical consideration during project development.

Should you have any questions regarding this investigation, or if we may be of further service, please contact the undersigned at your convenience.

Very truly yours,

GEOCON INCORPORATED

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GEOTECHNICAL INVESTIGATION

1. PURPOSE AND SCOPE

The purpose of this geotechnical investigation was to evaluate the proposed grading for an 84-lot residential subdivision located in San Diego, California (see *Vicinity Map,* Figure 1). This report provides recommendations relative to the geotechnical engineering aspects of developing the property as proposed. In addition, this report is intended to update our previous report entitled *Geologic Reconnaissance, Avion, San Diego, California,* dated January 19, 2018 (Project No. G2213-32-01) and to address the plans entitled *Heritage Bluffs II, Rezone No. 1193243/Vesting Tentative Map No. 1193244/Planned Development Permit No. 1193245/Site Development Permit No. 1193246, Cover Sheet, PTS # 319435,* prepared by Project Design Consultants, revised August 14, 2018.

The scope of our recent work consisted of the following:

- Reviewing aerial photographs and readily available published and unpublished geologic literature.
- Reviewing the referenced plans prepared by Project Design Consultants for the subject property.
- Advancing twenty-three exploratory trenches using a track-mounted backhoe to evaluate the general extent and condition of surficial deposits (see Appendix A).
- Performing laboratory tests on selected soil samples to evaluate the physical characteristics for engineering analysis (see Appendix B).
- Performing slope stability analyses of slope areas that are likely to impact the proposed development area (see Appendix C).
- Performing two infiltration tests in select areas to be utilized during storm water management design and providing storm water management guidelines in accordance with the City of San Diego Storm Water Standards (Appendix D).
- Preparing this report, geologic cross sections, geologic map and our conclusions and recommendations regarding the geotechnical aspects of developing the property as presently proposed. In addition, we have included the six seismic traverses performed in August 2013 by Southwest Geophysics in Appendix E.

The approximate locations of the previous seismic traverses and recent exploratory trenches are shown on the *Geologic Map*, Figure 2. *Geologic Cross-Sections* A-A′ through D-D′ (Figures 3 and 4) represent our interpretation of the geologic conditions across the site and served as the basis for our slope stability analysis.

2. SITE AND PROJECT DESCRIPTION

The property consists of approximately 40 acres of undeveloped land that is located to the north of Black Mountain Open Space Park in San Diego, California. The site consists of a north-trending ridge with moderate to steep slopes along the flanks with elevations ranging from approximately 895 feet above Mean Sea Level (MSL) in the south to 680 feet MSL in the north. Vegetation consists of thick chaparral with wild artichoke thistle and low-lying grasses.

It is our understanding that the project will be developed to create approximately 84 single-family residential units and associated infrastructure. Retaining walls up to 44-feet in height are contemplated along the main roadway where it crosses the main drainage in the northern portion of the project. A detention basin is also planned in the northeastern portion of the site.

Based on our review of the referenced plans, grading quantities will consist of approximately 225,000 cubic yards of cut, 268,000 cubic yards of fill with an estimated 43,000 cubic yards of import material. We understand that these estimates do not account for bulking and shrinking of the materials. Maximum cuts and fills are on the order of 55 feet and 65 feet, respectively. Fill slopes are designed at 2:1 (horizontal:vertical) or flatter, with a maximum height of approximately 80 feet. Cut slopes are designed at 1.5:1 or flatter, with a maximum height of approximately 70 feet.

3. SOIL AND GEOLOGIC CONDITIONS

Five surficial soil types and one geologic formation were encountered during our field investigation. The surficial deposits consist of previously placed fill, undocumented fill, topsoil, alluvium and colluvium. The formational unit was the Jurassic-age Santiago Peak Volcanics. The approximate extent of the deposits, excluding topsoil, are presented on the *Geologic Map*. Each of the surficial soil types and geologic units encountered is described below in order of increasing age.

3.1 Previously Placed Fill (Qpf)

Previously placed fill soils associated with the adjacent Heritage Bluffs II project are mapped along the northeastern portion of the project. Geotechnical information associated with the placement of these fills is provided in Reference No. 8.

3.2 Undocumented Fill (Qudf)

Undocumented fill embankments that appear to be former dams are located along the main drainage that flanks the east side of the development. In addition, several relatively minor embankments, which were not mapped, are present in the northern portion of the development area where former structure foundations are present. These deposits, where present within the development footprint, will require remedial grading prior to placement of additional fill in areas planned to receive structural fill and/or settlement-sensitive structures.

3.3 Topsoil (Unmapped)

Topsoil blankets the majority of the site and varies in thickness from approximately ½ to 3 feet. The topsoils are characterized as predominately soft to stiff, dry to moist, sandy silts. Topsoil deposits are considered unsuitable in their present condition and will require removal and compaction in areas planned to receive structural fill and/or settlement-sensitive structures. The topsoils may exhibit a medium to high expansion potential and should be placed in deeper fill areas.

3.4 Alluvium (Qal)

Alluvial soils were encountered in Trench Nos. T-12, T-13 and T-15 within the drainage channel located along the eastern portion of the site. These deposits, where encountered, generally consisted of unconsolidated gravels, sands, silts and clays derived from the metavolcanic rock. The alluvial deposits typically are poorly consolidated and compressible, and will require remedial grading where encountered, including within the proposed abutments/walls and slopes along the main access road drainage crossing.

3.5 Colluvium (Qcol)

Colluvial deposits were encountered in several trenches in the eastern half of the property within the gentle to moderate slope areas of the project. Where observed, the maximum colluvium thickness is on the order of 7 feet (Trench Nos. T-17 and T-18). These deposits generally exhibit medium to high expansion potential, are poorly consolidated, and will require remedial grading in areas of planned development.

3.6 Santiago Peak Volcanics (Jsp)

The Santiago Peak Volcanics was encountered within our exploratory excavations throughout the property. This formation consists of weakly metamorphosed volcanic and sedimentary rocks that appear relatively dark-colored where exposed. The metavolcanic rock constitution ranges from rhyolite to basalt and commonly includes tuff, tuff-breccias, and andesites. Very fine-grained, silicified sandstones, slate, and other types of metasedimentary rocks can also be present.

The rippability characteristics of the Santiago Peak Volcanics is discussed in the *Rippability and Rock Considerations* portion of this report. The Santiago Peak Volcanics generally exhibit adequate bearing and slope stability characteristics. Cut slopes excavated at an inclination of 1.5:1 (horizontal:vertical) should be stable to the proposed heights, if free of adversely oriented joints or fractures. In this regard, structural measurements taken during this study indicate that adverse conditions do not exist.

4. SLOPE STABILITY EVALUATION

Four geologic cross-sections, A-A′ through D-D′ (Figures 3 and 4) were prepared to aid in evaluating the stability of proposed cut and fill slopes around the perimeter of the development area. Appendix C presents a summary of the slope stability analyses for the cross-sections studied.

The computer program SLOPE/W distributed by Geo-Slope International was utilized to perform the slope stability analyses. This program uses conventional slope stability equations and a two-dimensional limit-equilibrium method to calculate the factor of safety against deep-seated failure. For our analysis, Spencer's Method with a circular failure mode was used. Spencer's Method satisfies both moment and force equilibrium.

Table 4.1 presents the soil strength parameters that were utilized in the slope stability analyses. The values were derived from previous and recent laboratory test results and our experience with similar soil and geologic conditions. In addition, we have performed an evaluation using strength parameters selected from the *American Geological Institute (AGI) Data Sheets for Geology in the Field, Laboratory and Office, Third Addition*, Data Sheet 78.2, Table 1 (Physical Engineering Properties of Rocks) compiled by Lawrence C. Wood, Stanford University to evaluate the shear strength of the metavolcanic rock.

TABLE 4.1
SOIL STRENGTH PARAMETERS

Soil Condition	Angle of Internal Friction ϕ (degrees)	Cohesion (psf)
Alluvium	28	200
Colluvium	28	200
MSE Backfill	32	0
Compacted Fill	32	350
Santiago Peak Formation	45	570,000

In accordance with Special Publication 117 guidelines, site-specific seismic slope stability analyses are required for sites located within mapped hazard zones. Seismic Hazard Zone maps published by CDMG, including landslide hazard zones, have not been published for San Diego County due to the relatively low seismic risk compared with other jurisdictions in Southern California. Therefore, it is our opinion that seismic slope stability analyses are not required in San Diego County. However, as requested, seismic slope stability analyses on the most critical failure surfaces have been performed in accordance with *Recommended Procedures for Implementation of DMG Special Publication 117: Guidelines for Analyzing and Mitigating Landslide Hazards in California*, prepared by the Southern California Earthquake Center (SCEC), dated June 2002.

The seismic slope stability analysis was performed using an acceleration of 0.19g, corresponding to a 10 percent probability of exceedance in 50 years. In addition, a deaggregation analysis was performed. A modal magnitude and modal distance of 6.9 and 17.2 kilometers, respectively, was used in the analysis.

Using the parameters discussed herein, an equivalent site acceleration, k_{EQ}, of 0.11g was calculated to perform the screening analysis. The screening analysis was performed using an acceleration of 0.11g resulting in pseudo-static factors of safety greater than 1.4. A slope is considered acceptable by the screening analysis if the calculated factor of safety is greater than 1.0 using k_{EQ}; therefore, the most critical failure surfaces depicted on Cross-sections A-A′ through D-D′, pass the screening analysis for the seismic slope stability.

The output files and calculated factor of safety for the cross sections used for the stability analyses are presented in Appendix C and summarized in Table 4.2.

TABLE 4.2
SLOPE STABILITY SUMMARY

Cross Section	Figure Number	Condition Analyzed	Factor Of Safety
A-A’	C-1	Proposed Condition: Static – Circular Failure	266
	C-2/C-3	Proposed Condition: Seismic – Circular Failure	207
B-B’	C-4	Proposed Condition: Static – Circular Failure	1.7
	C-5/C-6	Proposed Condition: Seismic – Circular Failure	1.5
	C-7	Proposed Condition: Static – Circular Failure	1.9
	C-8/C-9	Proposed Condition: Seismic – Circular Failure	1.6
C-C’	C-10	Proposed Condition: Static – Circular Failure	1.9
	C-11/C-12	Proposed Condition: Seismic – Circular Failure	1.5
D-D’	C-13	Proposed Condition: Static – Circular Failure	2.0
	C-14/C-15	Proposed Condition: Seismic – Circular Failure	1.6

A groundwater table was conservatively incorporated into the analysis and generally placed within the alluvial drainage, although no groundwater was encountered during our field exploration.

Cross Section A-A′ was analyzed to demonstrate the stability of the proposed cut slopes founded in metavolcanic rock (Jsp). The cut slope depicted on Cross-Section A-A′ was found to possess static and pseudo-static factors of safety greater than 1.5 and 1.0, respectively (see Figures C-1 through C-3).

Cross-Section B-B′ was evaluated to demonstrate the global stability of the proposed MSE retaining walls. Two MSE retaining walls supporting the access road bridge abutments are shown. Both walls exhibited static and pseudo-static factors of safety greater than 1.5 and 1.0, respectively (see Figures C-4 through C-9).

Cross-Section C-C′ was analyzed to demonstrate the stability of the proposed fill slopes. The fill slope depicted on Cross-Section C-C′ exhibited static and pseudo-static factors of safety greater than 1.5 and 1.0, respectively (see Figures C-10 through C-12).

Cross-Section D-D′ was evaluated to demonstrate the stability of the proposed fill slope with MSE wall at the toe of slope. The fill slope depicted on Cross-Section D-D′ was found to possess static and pseudo-static factors of safety greater than 1.5 and 1.0, respectively (see Figures C-13 through C-15).

5. RIPPABILITY AND ROCK CONSIDERATIONS

To aid in evaluating the rippability characteristics of the rock in proposed cut areas, a subsurface exploration program consisting of a seismic refraction survey was performed. The results of the study indicate that very hard rock is present near the ground surface. Rock rippability is a function of natural weathering processes that can vary vertically and horizontally over short distances depending on jointing, fracturing, and/or mineralogic discontinuities within the bedrock.

Seismic traverses S-1 through S-6 (shown on Figure 2) were conducted in August 2013 by Southwest Geophysics. The results of their study presented in Appendix E. Based on our review of their study, it is expected that the majority of the significant excavations within the development will experience very difficult ripping and/or blasting conditions as excavations are extended beyond the rippable weathered mantle. Excavations, undercutting and blasting techniques can be expected to generate oversized rock (rocks greater than 12-inches in dimension), which will necessitate typical hard rock handling and placement procedures during grading operations.

Estimates of the anticipated volume of hard rock materials generated from proposed excavations should be evaluated based on the information provided and using the contractor’s judgement. Roadway/utility corridors and lot undercutting criteria should also be considered when calculating the volume of hard rock.

Earthwork construction should be carefully planned to efficiently utilize available rock placement areas. Oversize materials should be placed in accordance with rock placement procedures presented in Appendix F of this report, Oversize Rock Disposal zones (Figure 5), and governing jurisdictions.

6. GROUNDWATER/SEEPAGE

No groundwater or seepage was observed in the excavations performed during the field studies. However, groundwater levels in drainage areas can be expected to fluctuate seasonally and may affect grading. In this regard, grading may encounter wet soils causing excavation and compaction difficulty, particularly if construction is planned during the winter months.

Subdrain systems (i.e. canyon subdrain, toe drains) will be necessary for the proposed development to intercept and convey seepage migrating along fractures and impervious strata. The location of proposed underground improvements may result in modifications to the recommended subdrains shown on the *Geologic Map*.

7. GEOLOGIC HAZARDS

7.1 Faulting and Seismicity

Based on our previous observations during mass grading in adjacent areas, previous and recent geotechnical studies, and a review of published geologic maps and reports, the site is not located on any known “active,” “potentially active” or “inactive” fault traces as defined by the California Geological Survey (CGS).

The Rose Canyon Fault zone and the Newport-Inglewood Fault, located approximately 11 miles west of the site, are the closest known active faults. The CGS considers a fault seismically active when evidence suggests seismic activity within roughly the last 11,000 years. The CGS has included portions of the Rose Canyon Fault zone within an Alquist-Priolo Earthquake Fault Zone.

We used the computer program *EZ-FRISK* (Version 7.65) to determine the distance of known faults to the site and to estimate ground accelerations at the site for the maximum anticipated seismic event. According to the results, 7 known active faults are located within a search radius of 50 miles from the property. We used acceleration attenuation relationships developed by Boore-Atkinson (2008) NGA USGS2008, Campbell-Bozorgnia (2008) NGA USGS, and Chiou-Youngs (2008) NGA in our analysis. The nearest known active faults are the Newport-Inglewood and Rose Canyon Fault Zones, located approximately 11 miles west of the site, respectively, and are the dominant sources of potential ground motion. Table 7.1.1 lists the estimated maximum earthquake magnitudes and PGA’s for the most dominant faults for the site location calculated for Site Class C as defined by Table 1613.3.2 of the 2016 California Building Code (CBC).

TABLE 7.1.1
DETERMINISTIC SEISMIC SITE PARAMETERS

Fault Name	Distance from Site (miles)	Maximum Earthquake Magnitude (Mw)	Peak Ground Acceleration		
			Boore-Atkinson 2008 (g)	Campbell-Bozorgnia 2008 (g)	Chiou-Youngs 2008 (g)
Newport-Inglewood	11	7.5	0.25	0.20	0.25
Rose Canyon	11	6.9	0.21	0.18	0.20
Elsinore	25	7.85	0.19	0.13	0.17
Coronado Bank	25	7.4	0.17	0.11	0.13
Palos Verdes Connected	25	7.7	0.18	0.12	0.15
Earthquake Valley	33	6.8	0.11	0.07	0.07
San Jacinto	47	7.88	0.13	0.09	0.11

We performed a site-specific probabilistic seismic hazard analysis using the computer program *EZ-FRISK*. Geologic parameters not addressed in the deterministic analysis are included in this analysis. The program operates under the assumption that the occurrence rate of earthquakes on each mappable Quaternary fault is proportional to the faults slip rate. The program accounts for fault rupture length as a function of earthquake magnitude, and site acceleration estimates are made using the earthquake magnitude and distance from the site to the rupture zone. The program also accounts for uncertainty in each of following: (1) earthquake magnitude, (2) rupture length for a given magnitude, (3) location of the rupture zone, (4) maximum possible magnitude of a given earthquake, and (5) acceleration at the site from a given earthquake along each fault. By calculating the expected accelerations from considered earthquake sources, the program calculates the total average annual expected number of occurrences of site acceleration greater than a specified value. We utilized acceleration-attenuation relationships suggested by Boore-Atkinson (2008) NGA USGS 2008, Campbell-Bozorgnia (2008) NGA USGS 2008, and Chiou-Youngs (2008) NGA in the analysis. Table 7.1.2 presents the site-specific probabilistic seismic hazard parameters including acceleration-attenuation relationships and the probability of exceedence.

TABLE 7.1.2
PROBABILISTIC SEISMIC HAZARD PARAMETERS

Probability of Exceedence	Peak Ground Acceleration		
	Boore-Atkinson, 2008 (g)	Campbell-Bozorgnia, 2008 (g)	Chiou-Youngs, 2008 (g)
2% in a 50 Year Period	0.46	0.37	0.43
5% in a 50 Year Period	0.36	0.28	0.31
10% in a 50 Year Period	0.28	0.22	0.24

While listing peak accelerations is useful for comparison of potential effects of fault activity in a region, other considerations are important in seismic design, including the frequency and duration of motion and the soil conditions underlying the site. Seismic design of the structures should be evaluated in accordance with the California Building Code (CBC) guidelines currently adopted by the City of San Diego.

7.2 Liquefaction

Liquefaction typically occurs when a site is located in a zone with seismic activity, onsite soils are cohesionless, groundwater is encountered within 50 feet of the surface, and soil densities are less than about 70 percent of the maximum dry densities. If all four criteria are met, a seismic event could result in a rapid increase in pore water pressure from the earthquake-generated ground accelerations. The potential for liquefaction at the site is considered to be negligible due to the dense formational material encountered, remedial grading recommended, and lack of a shallow groundwater condition.

7.3 Landslides

No evidence of ancient landslide deposits was encountered at the site during the geotechnical investigation.

7.4 Geologic Hazard Category

Based on our review of the 2008 City of San Diego Seismic Safety Study Map Sheets 43 and 44, the site is located within Geologic Hazard Category 53. Category 53 indicates *level or sloping terrain, unfavorable geologic structure, low to moderate risk.*

8. CONCLUSIONS AND RECOMMENDATIONS

8.1 General

8.1.1 No soil or geologic conditions were encountered that, in the opinion of Geocon Incorporated, would preclude the development of the property as proposed, provided the recommendations of this report are followed.

8.1.2 The site is underlain by surficial units that include undocumented fill, topsoil, alluvial and colluvial deposits. These deposits are unsuitable in their present condition and will require remedial grading where improvements are planned.

8.1.3 The presence of hard rock at or near the existing ground surface will require special consideration during site grading. It is anticipated that the majority of the proposed excavations will encounter moderate to heavy ripping at shallow depths with conventional heavy-duty equipment. Based on the seismic refraction data, blasting is expected at shallow depths throughout the site. In addition, heavy ripping and blasting will generate oversize materials that will require special handling and fill placement procedures. Oversize materials should be placed in accordance with Figure 5 and Appendix F of this report.

8.1.4 An earthwork analysis should be performed to determine if there is an adequate volume of fill area available to accommodate the anticipated volume of blasted/oversize materials. This study should consider the proposed grading, capping requirements, proposed pad and utility corridor undercutting, rippability information contained in this report, and rock placement requirements. Crushing may be necessary to meet the project grading specifications with respect to capping and particle size restriction zones.

8.1.5 Cut slopes should be observed by an engineering geologist during grading to verify that the soil and geologic conditions do not differ significantly from those anticipated. Additional recommendations will be provided in the event that adverse conditions are encountered. Scaling of loose rock fragments from proposed cut slopes should also be anticipated.

8.2 Settlement Considerations

8.2.1 Fill soil, even though properly compacted, may experience settlement over the lifetime of the improvements that it supports. The ultimate settlement potential of the fill is a function of the soil classification, placement relative compaction, and subsequent increases in the soil moisture content and the geometry of the fill embankment.

8.2.2 Due to the variable fill thicknesses, a potential for differential settlement across the proposed buildings and underground improvements exists. Selective undercutting of the formational materials may be required to reduce the estimated differential settlement beneath the building and underground utilities. To consider the thicker fills, we recommend a minimum relative compaction of 93 percent at above optimum moisture content for fills deeper than 50 feet.

8.2.3 To reduce the effects of differential settlement for buildings spanning a cut/fill transition, the cut portion should be undercut a distance of H/5, where H is the maximum fill thickness beneath the building. For example, if the fill thickness across a building varies from 0 to 25 feet, the cut portion of the building pad should be undercut approximately 5 feet.

8.2.4 Some of the proposed buildings will be underlain by a fill thickness of approximately 70 feet. Due to its granular nature, the settlement of compacted fill is expected to occur over a relatively short time period. The need for a settlement monitoring program will be evaluated during grading and post-grading operations. If a settlement monitoring program is necessary, it should be initiated immediately upon completion of grading. Further recommendations in this regard will be provided during grading. We estimate that a potential area requiring settlement monitoring may include Lots 77 through 81.

8.3 Soil and Excavation Characteristics

8.3.1 The soil conditions encountered during our study consist of “low” to “medium” expansive sandy silt and silty/clayey sand and sandy clay with abundant angular rock fragments. However, highly expansive clays were encountered in the colluvial and alluvial deposits within the adjacent Heritage Bluffs II development.

8.3.2 Excavation of the surficial deposits (undocumented fill, topsoil, alluvium, colluvium) should generally require light to moderate effort using conventional heavy-duty grading equipment.

8.3.3 Excavating within the metavolcanic rock materials will generally vary in difficulty with the depth of excavation depending on the degree of weathering. Based on the seismic lines, blasting will likely be required for most of the excavations. Depending on the blasting pattern and overburden thickness, the generation of oversize rock could impact project development. Oversize rock should be placed in accordance with *Recommended Grading Specifications* (Appendix F), Oversize Rock Disposal exhibit (Figure 5), and the requirements of the City of San Diego. Oversize rock may require breakage/crushing to acceptable sizes or exportation from the property. Placement of oversize rock within the area of proposed underground utilities should not be permitted.

8.3.4 Surficial deposits (topsoil and alluvium/colluvium) may be very moist to saturated during the winter or early spring depending on preceding precipitation. Overly wet soils will require drying or mixing with drier material prior to their use as compacted fill.

8.4 Corrosion

8.4.1 We performed laboratory tests on a sample of the site materials to evaluate the percentage of water-soluble sulfate content. Results from the laboratory water-soluble sulfate content tests are presented in Appendix B and indicate that the on-site materials at the locations tested possess a “Not Applicable” and “S0” sulfate exposure to concrete structures as defined by 2016 CBC Section 1904 and ACI 318-14 Chapter 19. The presence of water-soluble sulfates is not a visually discernible characteristic; therefore, other soil samples from the site could yield different concentrations. Additionally, over time landscaping activities (i.e., addition of fertilizers and other soil nutrients) may affect the concentration. Table 8.4 presents a summary of concrete requirements set forth by 2016 CBC Section 1904 and ACI 318.

TABLE 8.4
REQUIREMENTS FOR CONCRETE EXPOSED TO SULFATE-CONTAINING SOLUTIONS

Sulfate Severity	Exposure Class	Water-Soluble Sulfate (SO_4) Percent by Weight	Cement Type (ASTM C 150)	Maximum Water to Cement Ratio by Weight[1]	Minimum Compressive Strength (psi)
Not Applicable	S0	SO_4<0.10	No Type Restriction	n/a	2,500
Moderate	S1	0.10≤SO_4<0.20	II	0.50	4,000
Severe	S2	0.20≤SO_4≤2.00	V	0.45	4,500
Very Severe	S3	SO_4>2.00	V+Pozzolan or Slag	0.45	4,500

[1] Maximum water to cement ratio limits do not apply to lightweight concrete.

8.4.2 Geocon Incorporated does not practice in the field of corrosion engineering. Therefore, if improvements that could be susceptible to corrosion are planned, it is recommended that further evaluation by a corrosion engineer be performed.

8.5 Slope Stability

8.5.1 Slope stability analyses for fill slopes were performed utilizing average drained direct shear strength parameters from the laboratory test results. These analyses indicate that the proposed 2:1 fill slopes, constructed of on-site materials, should have calculated factors of safety of at least 1.5 and 1.0 under static and pseudo-static conditions, respectively, for both deep-seated failure and shallow sloughing conditions to heights of at least 100 feet.

8.5.2 Cut slopes in rock materials (Santiago Peak Volcanics) do not lend themselves to conventional slope stability analyses. Based on experience with similar rock conditions, 1.5:1 cut slopes to the planned heights of up to 70 feet should possess a factor of safety of at least 1.5 with respect to slope instability, if free of adversely oriented joints or fractures. We did not encounter any of these features during our study. To satisfy agency requirements, we have performed a quantitative evaluation of the primary rock slope using strength parameters selected from the *American Geological Institute (AGI) Data Sheets for Geology in the Field, Laboratory and Office, Third Addition*, Data Sheet 78.2, Table 1 (Physical Engineering Properties of Rocks) compiled by Lawrence C. Wood, Stanford University. Based on the results of the analysis, the factor of safety for slopes excavated in metavolcanic rock to the design heights will possess static and pseudo-static factors of safety greater than 1.5 and 1.0, respectfully, for gross and surficial stability.

8.5.3 Although rare, the most common mode of instability for rock slopes are shallow wedge failures from intersecting fault planes or clay filled joints/fractures dipping out of slope. In this regard, the structural measurements obtained during our study did not reveal such conditions. The data indicates a random joint pattern with joint spacing ranging from a few inches up to approximately three or four feet apart. It is recommended, however, that all slope excavations proposed on the site be observed during grading by an engineering geologist to confirm that geologic conditions do not differ significantly from those anticipated. In the event that adverse conditions are observed, stabilization recommendations can be provided.

8.5.4 Fill slopes should 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 soils are uniformly compacted to at least 90 percent relative compaction to the face of the finished sloped. Alternatively, the fill slope may be over-built at least 3 feet and cut back to yield a properly compacted slope face.

8.5.5 Where fill slopes and fill-over-cut slopes are planned, following removal of the surficial soils, a 15-foot-wide, 2-foot-deep, undrained keyway should be constructed prior to placing compacted fill. The keyway should be constructed with a minimum 5 percent inclination away from the toe of slope.

8.5.6 All slopes should be landscaped with drought-tolerant vegetation, having variable root depths and requiring minimal landscape irrigation. In addition, all slopes should be drained and properly maintained to reduce erosion.

8.6 Stability Fills

8.6.1 Cut slopes in the interior of the project, which will expose highly fractured Santiago Peak Volcanics, should be evaluated during grading. These rock slopes, upwards of 15-feet high, may require stability fills. Our experience with nearby projects indicates that a potential to encounter moderately to intensely jointed/fractured rock exists. Cut slopes in this material may also readily transmit seepage and inhibit planned landscaping due to the hard rock.

8.7 Subdrains

8.7.1 The geologic units encountered on the site have permeability characteristics and/or fracture systems that could be susceptible under certain conditions to groundwater seepage. The use of canyon subdrains will be necessary to mitigate the potential for adverse impacts associated with seepage conditions. Appendix F depicts a typical canyon subdrain detail and the proposed locations are shown on the *Geologic Map*. In general, subdrains should be extended to within approximately 10 feet of the ultimate ground surface.

8.7.2 Prior to outletting, the final segment of subdrain should consist of non-perforated drainpipe. At the non-perforated/perforated interface, a seepage cutoff wall should be constructed on the downslope side of the junction in accordance with Appendix F. Subdrains that discharge into a natural drainage course or open space area should be provided with a permanent headwall structure in accordance with Appendix F.

8.7.3 The final grading plans should show the location of all proposed subdrains. Upon completion of remedial excavations and subdrain installation, the project civil engineer should survey the drain locations and prepare an “as-built” map depicting the existing conditions.

8.8 Toe Drains

8.8.1 Building pad areas adjacent to ascending slopes in the interior of the project may experience wet soil conditions due to water migration from natural or future irrigation sources. To reduce the potential for this to occur, consideration should be given to placing a toe drain along the base of the slopes to collect potential seepage and convey it to a suitable outlet. The drain should be sufficiently deep to intercept the seepage (on the order of 3 feet below finish grade) and constructed in accordance with Figure 6. The need for these drains can be evaluated during grading by your project superintendent.

8.8.2 A toe drain is recommended at the base of the cut slope behind Lots 35 through 44 (see Figure 2 for location). Prior to outletting, the toe drain should transition to non-perforated drainpipe with a seepage cut-off wall provided at this interface. The project civil engineer should be consulted for an appropriate outlet location.

8.8.3 The necessity for additional toe drains will be evaluated during grading. In addition, the project civil engineer should be consulted to evaluate the appropriate drain locations and necessary easements, building restriction zones or disclosure requirements that may be necessary. The drains should be surveyed for location and shown on the project as-built drawings.

8.9 Grading

8.9.1 All grading should be performed in accordance with the attached *Recommended Grading Specifications* (Appendix F). Where the recommendations of this section conflict with Appendix F, the recommendations of this section take precedence. All earthwork should be observed and all fills tested for proper compaction by Geocon Incorporated.

8.9.2 Prior to commencing grading, a preconstruction conference should be held at the site with the owner or developer, grading contractor, civil engineer, and geotechnical engineer in attendance. Special soil handling and/or the grading plans can be discussed at that time.

8.9.3 Site preparation should begin with the removal of all deleterious material and vegetation. The depth of removal should be such that material exposed in cut areas or soils to be used as fill are relatively free of organic matter. Material generated during stripping and/or site demolition should be exported from the site.

8.9.4 All compressible soil deposits, including undocumented fill, topsoil, alluvium and colluvium within areas where structural improvements are planned or where discussed herein, should be removed to firm natural ground and properly compacted prior to placing additional fill and/or structural loads. Deeper than normal benching and/or stripping operations for sloping ground surfaces will be required where the thickness of potentially compressible surficial deposits exceeds 3 feet. The actual extent of unsuitable soil removals will be determined in the field during grading by the geotechnical engineer and/or engineering geologist.

8.9.5 After removal of unsuitable materials is performed, the site should then be brought to final subgrade elevations with structural fill compacted in layers. In general, soils native to the site are suitable for re-use as fill if free from vegetation, debris and other deleterious material. Layers of fill should be no thicker than will allow for adequate bonding and compaction. All fill, including backfill and scarified ground surfaces, should be compacted to at least 90 percent of maximum dry density at or above optimum moisture content, as determined in accordance with ASTM Test Procedure D1557. Fill materials below optimum moisture content will require additional moisture conditioning prior to placing additional fill. **Fill <u>greater than 50 feet</u> thick should be compacted to at least <u>93 percent</u> of the laboratory maximum dry density at a moisture content above optimum moisture content.**

8.9.6 Grading operations should be scheduled to permit the placement of oversize material and expansive soils in the deeper fill areas and to cap the building pads with granular materials having a “very low” to “low” expansive potential. Oversize material should be placed at least 10 feet below finish grade or 2 feet below the deepest utilities, whichever is greater.

8.9.7 To reduce the potential for differential settlement, it is recommended that the cut portion of cut/fill transition building pads be undercut at least 3 feet and replaced with properly compacted “very low” to “low” expansive fill soils. Where the thickness of the fill below the building pad exceeds 15 feet, the depth of the undercut should be increased to one-fifth of the maximum fill thickness. The base of the undercuts should be sloped towards the front of the lots.

8.9.8 Oversize material (defined as material greater than 12 inches in nominal dimension) may be generated during ripping of formational materials. Placement of oversize material within fills should be conducted in accordance with the recommendations in Appendix F and Figure 5. Grading operations on the site should be scheduled such that oversize materials are placed in designated rock disposal areas and/or deeper fills.

8.9.9 Rock greater than 6 inches in maximum dimension should not be placed within 3 feet of finish grade in building pad areas or street subgrade. Rock greater than 12 inches in maximum dimension should not be placed within 10 feet of finish pad grade or within 2 feet of the deepest utility. Crushing may be required to achieve this placement criteria. The gradation of capping materials should conform to the project grading specifications.

8.9.10 Where practical, the upper 3 feet of all building pads (cut or fill) should be comprised of soil with a “very low” to “low” expansion potential. The more highly expansive fill soils should be placed in the deeper fill areas and properly compacted. “Very low” to “low” expansive soils are defined by the 2016 California Building Code (CBC) Section 1803.5.3 as those soils that have an Expansion Index of 50 or less.

8.9.11 Cut pads exposing metavolcanic rock should be undercut at least 3 feet and replaced with properly compacted “very low” to “low” expansive soil. The base of the undercuts should be sloped towards the front of the lots.

8.9.12 Undercutting of street areas should be considered to facilitate the excavation of underground utilities where the streets are located in cut areas composed of marginally to non-rippable hard rock. If subsurface improvements or landscape zones are planned outside these areas, consideration should be given to undercutting these areas as well. This can be evaluated during grading operations.

8.9.13 It is recommended that excavations be observed during grading by a representative of Geocon Incorporated to verify that soil and geologic conditions do not differ significantly from those anticipated.

8.9.14 It is the responsibility of the contractor and their competent person to ensure that all excavations, temporary slopes and trenches are properly constructed and maintained in accordance with applicable OSHA regulations in order to maintain safety and the stability of adjacent existing improvements.

8.9.15 Import materials (if required), should consist of “very low” to “low” expansive (Expansion Index of 50 or less) soils. Prior to importing the material, samples from proposed borrow areas should be obtained and subjected to laboratory testing to determine whether the material conforms to the recommended criteria. At least 3 working days should be allowed for laboratory testing of the soil prior to its importation. Import materials should be free of oversize rock and construction debris.

8.10 Seismic Design Criteria

8.10.1 We used the computer program *U.S. Seismic Design Maps*, provided by the USGS. Table 8.10.1 summarizes site-specific design criteria obtained from the 2016 California Building Code (CBC; Based on the 2015 International Building Code [IBC] and ASCE 7-10), Chapter 16 Structural Design, Section 1613 Earthquake Loads. The short spectral response uses a period of 0.2 second. The building structure and improvements should be designed using a Site Class C. We evaluated the Site Class based on the discussion in Section 1613.3.2 of the 2016 CBC and Table 20.3-1 of ASCE 7-10. The values presented in Table 8.10.1 are for the risk-targeted maximum considered earthquake (MCE_R).

TABLE 8.10.1
2016 CBC SEISMIC DESIGN PARAMETERS

Parameter	Value	2016 CBC Reference
Site Class	C	Section 1613.3.2
MCE_R Ground Motion Spectral Response Acceleration – Class B (short), S_S	0.921g	Figure 1613.3.1(1)
MCE_R Ground Motion Spectral Response Acceleration – Class B (1 sec), S_1	0.361g	Figure 1613.3.1(2)
Site Coefficient, F_A	1.032	Table 1613.3.3(1)
Site Coefficient, F_V	1.439	Table 1613.3.3(2)
Site Class Modified MCE_R Spectral Response Acceleration (short), S_{MS}	0.950g	Section 1613.3.3 (Eqn 16-37)
Site Class Modified MCE_R Spectral Response Acceleration (1 sec), S_{M1}	0.519g	Section 1613.3.3 (Eqn 16-38)
5% Damped Design Spectral Response Acceleration (short), S_{DS}	0.633g	Section 1613.3.4 (Eqn 16-39)
5% Damped Design Spectral Response Acceleration (1 sec), S_{D1}	0.346g	Section 1613.3.4 (Eqn 16-40)

8.10.2 Table 8.10.2 presents additional seismic design parameters for projects located in Seismic Design Categories of D through F in accordance with ASCE 7-10 for the mapped maximum considered geometric mean (MCE_G).

TABLE 8.10.2
2016 CBC SITE ACCELERATION PARAMETERS

Parameter	Value	ASCE 7-10 Reference
Mapped MCE_G Peak Ground Acceleration, PGA	0.345g	Figure 22-7
Site Coefficient, F_{PGA}	1.055	Table 11.8-1
Site Class Modified MCE_G Peak Ground Acceleration, PGA_M	0.364g	Section 11.8.3 (Eqn 11.8-1)

8.10.3 Conformance to the criteria in Tables 8.10.1 and 8.10.2 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.

8.11 Foundation and Concrete Slabs-On-Grade Recommendations

8.11.1 The following foundation recommendations are for proposed one- to three-story residential structures. The foundation recommendations have been separated into three categories based on either the maximum and differential fill thickness or Expansion Index. The foundation category criteria are presented in Table 8.11.1.

TABLE 8.11.1
FOUNDATION CATEGORY CRITERIA

Foundation Category	Maximum Fill Thickness, T (Feet)	Differential Fill Thickness, D (Feet)	Expansion Index (EI)
I	T<20	--	EI≤50
II	20≤T<50	10≤D<20	50<EI≤90
III	T≥50	D≥20	90<EI≤130

8.11.2 Final foundation categories for each building or lot will be provided after finish pad grades have been achieved and laboratory testing of the subgrade soil has been completed.

8.11.3 Table 8.11.2 presents minimum foundation and interior concrete slab design criteria for conventional foundation systems.

TABLE 8.11.2
CONVENTIONAL FOUNDATION RECOMMENDATIONS BY CATEGORY

Foundation Category	Minimum Footing Embedment Depth (inches)	Continuous Footing Reinforcement	Interior Slab Reinforcement
I	12	Two No. 4 bars, one top and one bottom	6 x 6 - 10/10 welded wire mesh at slab mid-point
II	18	Four No. 4 bars, two top and two bottom	No. 3 bars at 24 inches on center, both directions
III	24	Four No. 5 bars, two top and two bottom	No. 3 bars at 18 inches on center, both directions

8.11.4 The embedment depths presented in Table 8.11.2 should be measured from the lowest adjacent pad grade for both interior and exterior footings. The conventional foundations should have a minimum width of 12 inches and 24 inches for continuous and isolated footings, respectively. A typical wall/column footing detail is presented on Figure 7.

8.11.5 The concrete slabs-on-grade should be a minimum of 4 inches thick for Foundation Categories I and II and 5 inches thick for Foundation Category III. The concrete slabs-on-grade should be underlain by 4 inches and 3 inches of clean sand for 4-inch thick and 5-inch-thick slabs, respectively. Slabs expected to receive moisture sensitive floor coverings or used to store moisture sensitive materials should be underlain by a vapor inhibitor covered with at least 2 inches of clean sand or crushed rock. If crushed rock will be used, the thickness of the vapor inhibitor should be at least 10 mil to prevent possible puncturing.

8.11.6 As a substitute, the layer of clean sand (or crushed rock) beneath the vapor inhibitor recommended in the previous section can be omitted if a vapor inhibitor that meets or exceeds the requirements of ASTM E 1745-97 (Class A), and that exhibits permeance not greater than 0.012 perm (measured in accordance with ASTM E 96-95) is used. This vapor inhibitor may be placed directly on properly compacted fill or formational materials. The vapor inhibitor should be installed in general conformance with ASTM E 1643-98 and the manufacturer's recommendations. Two inches of clean sand should then be placed on top of the vapor inhibitor to reduce the potential for differential curing, slab curl, and cracking. Floor coverings should be installed in accordance with the manufacturer's recommendations.

8.11.7 As an alternative to the conventional foundation recommendations, consideration should be given to the use of post-tensioned concrete slab and foundation systems for the support of the proposed structures. The post-tensioned systems should be designed by a structural engineer experienced in post-tensioned slab design and design criteria of the Post-Tensioning Institute (PTI) DC 10.5-12 *Standard Requirements for Design and Analysis of Shallow Post-Tensioned Concrete Foundations on Expansive Soils* or *WRI/CRSI Design of Slab-on-Ground Foundations*, as required by the 2016 California Building Code (CBC Section 1808.6.2). Although this procedure was developed for expansive soil conditions, it can also be used to reduce the potential for foundation distress due to differential fill settlement. The post-tensioned design should incorporate the geotechnical parameters presented in Table 8.11.3 for the particular Foundation Category designated. The parameters presented in Table 8.11.3 are based on the guidelines presented in the PTI DC 10.5 design manual.

TABLE 8.11.3
POST-TENSIONED FOUNDATION SYSTEM DESIGN PARAMETERS

Post-Tensioning Institute (PTI), Third Edition Design Parameters	Foundation Category		
	I	II	III
Thornthwaite Index	-20	-20	-20
Equilibrium Suction	3.9	3.9	3.9
Edge Lift Moisture Variation Distance, e_M (feet)	5.3	5.1	4.9
Edge Lift, y_M (inches)	0.61	1.10	1.58
Center Lift Moisture Variation Distance, e_M (feet)	9.0	9.0	9.0
Center Lift, y_M (inches)	0.30	0.47	0.66

8.11.8 Foundation systems for the lots that possess a foundation Category I and a "very low" expansion potential (expansion index of 20 or less) can be designed using the method described in Section 1808 of the 2016 CBC. If post-tensioned foundations are planned, an alternative, commonly accepted design method (other than PTI DC 10.5) can be used. However, the post-tensioned foundation system should be designed with a total and differential deflection of 1 inch. Geocon Incorporated should be contacted to review the plans and provide additional information, if necessary.

8.11.9 If an alternate design method is contemplated, Geocon Incorporated should be contacted to evaluate if additional expansion index testing should be performed to identify the lots that possess a "very low" expansion potential (expansion index of 20 or less).

8.11.10 The foundations for the post-tensioned slabs should be embedded in accordance with the recommendations of the structural engineer. If a post-tensioned mat foundation system is planned, the slab should possess a thickened edge with a minimum width of 12 inches and extend below the clean sand or crushed rock layer.

8.11.11 If the structural engineer proposes a post-tensioned foundation design method other than PTI, Third Edition:

- The deflection criteria presented in Table 8.11.3 are still applicable.
- Interior stiffener beams should be used for Foundation Categories II and III.
- The width of the perimeter foundations should be at least 12 inches.
- The perimeter footing embedment depths should be at least 12 inches, 18 inches and 24 inches for foundation categories I, II, and III, respectively. The embedment depths should be measured from the lowest adjacent pad grade.

8.11.12 Our experience indicates post-tensioned slabs are susceptible to excessive edge lift, regardless of the underlying soil conditions. Placing reinforcing steel at the bottom of the perimeter footings and the interior stiffener beams may mitigate this potential. Current PTI design procedures primarily address the potential center lift of slabs but, because of the placement of the reinforcing tendons in the top of the slab, the resulting eccentricity after tensioning reduces the ability of the system to mitigate edge lift. The structural engineer should design the foundation system to reduce the potential of edge lift occurring for the proposed structures.

8.11.13 During the construction of the post-tension foundation system, the concrete should be placed monolithically. Under no circumstances should cold joints be allowed to form between the footings/grade beams and the slab during the construction of the post-tension foundation system.

8.11.14 Category I, II, or III foundations may be designed for an allowable soil bearing pressure of 2,000 pounds per square foot (psf) (dead plus live load). This bearing pressure may be increased by one-third for transient loads due to wind or seismic forces.

8.11.15 Isolated footings, if present, should have the minimum embedment depth and width recommended for conventional foundations for a particular foundation category. The use of isolated footings, which are located beyond the perimeter of the building and support structural elements connected to the building, are not recommended for Category III. Where this condition cannot be avoided, the isolated footings should be connected to the building foundation system with grade beams.

8.11.16 For Foundation Category III, consideration should be given to using interior stiffening beams and connecting isolated footings and/or increasing the slab thickness. In addition, consideration should be given to connecting patio slabs, which exceed 5 feet in width, to the building foundation to reduce the potential for future separation to occur.

8.11.17 Special subgrade presaturation is not deemed necessary prior to placing concrete; however, the exposed foundation and slab subgrade soil should be moisture conditioned, as necessary, to maintain a moist condition as would be expected in any such concrete placement.

8.11.18 Where buildings or other improvements are planned near the top of a slope steeper than 3:1 (horizontal:vertical), special foundations and/or design considerations are recommended due to the tendency for lateral soil movement to occur.

- For fill slopes less than 20 feet high, building footings should be deepened such that the bottom outside edge of the footing is at least 7 feet horizontally from the face of the slope.

- When located next to a descending 3:1 (horizontal:vertical) fill slope or steeper, the foundations should be extended to a depth where the minimum horizontal distance is equal to H/3 (where H equals the vertical distance from the top of the fill slope to the base of the fill soil) with a minimum of 7 feet but need not exceed 40 feet. The horizontal distance is measured from the outer, deepest edge of the footing to the face of the slope. An acceptable alternative to deepening the footings would be the use of a post-tensioned slab and foundation system or increased footing and slab reinforcement. Specific design parameters or recommendations for either of these alternatives can be provided once the building location and fill slope geometry have been determined.

- If swimming pools are planned, Geocon Incorporated should be contacted for a review of specific site conditions.

- Swimming pools located within 7 feet of the top of cut or fill slopes are not recommended. Where such a condition cannot be avoided, the portion of the swimming pool wall within 7 feet of the slope face be designed assuming that the adjacent soil provides no lateral support. This recommendation applies to fill slopes up to 30 feet in height, and cut slopes regardless of height. For swimming pools located near the top of fill slopes greater than 30 feet in height, additional recommendations may be required and Geocon Incorporated should be contacted for a review of specific site conditions.

- Although other improvements, which are relatively rigid or brittle, such as concrete flatwork or masonry walls, may experience some distress if located near the top of a slope, it is generally not economical to mitigate this potential. It may be possible, however, to incorporate design measures, which would permit some lateral soil movement without causing extensive distress. Geocon Incorporated should be consulted for specific recommendations.

8.11.19 The recommendations of this report are intended to reduce the potential for cracking of slabs due to expansive soil (if present), differential settlement of existing soil or soil with varying thicknesses. However, even with the incorporation of the recommendations presented herein, foundations, stucco walls, and slabs-on-grade placed on such conditions may still exhibit some cracking due to soil movement and/or 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.

8.11.20 Geocon Incorporated should be consulted to provide additional design parameters as required by the structural engineer.

8.12 Retaining Walls and Lateral Loads Recommendations

8.12.1 Retaining walls not restrained at the top and having a level backfill surface should be designed for an active soil pressure equivalent to the pressure exerted by a fluid with a density of 35 pounds per cubic foot (pcf). Where the backfill will be inclined at 2:1 (horizontal:vertical), an active soil pressure of 50 pcf is recommended. These soil pressures assume that the backfill materials within an area bounded by the wall and a 1:1 plane extending upward from the base of the wall possess an Expansion Index ≤50. Geocon Incorporated should be consulted for additional recommendations if backfill materials have an EI >50.

8.12.2 Retaining walls shall be designed to ensure stability against overturning sliding, excessive foundation pressure and water uplift. Where a keyway is extended below the wall base with the intent to engage passive pressure and enhance sliding stability, it is not necessary to consider active pressure on the keyway.

8.12.3 Where walls are restrained from movement at the top, an additional uniform pressure of 8H psf (where H equals the height of the retaining wall portion of the wall in feet) should be added to the active soil pressure where the wall possesses a height of 8 feet or less and 12H where the wall is greater than 8 feet. For retaining walls subject to vehicular loads within a horizontal distance equal to two-thirds the wall height, a surcharge equivalent to two feet of fill soil should be added (total unit weight of soil should be taken as 130 pcf).

8.12.4 Soil contemplated for use as retaining wall backfill, including import materials, should be identified in the field prior to backfill. At that time Geocon Incorporated should obtain samples for laboratory testing to evaluate its suitability. Modified lateral earth pressures may be necessary if the backfill soil does not meet the required expansion index or shear strength. City or regional standard wall designs, if used, are based on a specific active lateral earth pressure and/or soil friction angle. In this regard, on-site soil to be used as backfill may or may not meet the values for standard wall designs. Geocon Incorporated should be consulted to assess the suitability of the on-site soil for use as wall backfill if standard wall designs will be used.

8.12.5 Unrestrained walls will move laterally when backfilled and loading is applied. The amount of lateral deflection is dependent on the wall height, the type of soil used for backfill, and loads acting on the wall. The wall designer should provide appropriate lateral deflection quantities for planned retaining walls structures, if applicable. These lateral values should be considered when planning types of improvements above retaining wall structures.

8.12.6 Retaining walls should be provided with a drainage system adequate to prevent the buildup of hydrostatic forces and should be waterproofed as required by the project architect. The use of drainage openings through the base of the wall (weep holes) is not recommended where the seepage could be a nuisance or otherwise adversely affect the property adjacent to the base of the wall. The above recommendations assume a properly compacted granular (EI ≤50) free-draining backfill material with no hydrostatic forces or imposed surcharge load. A typical retaining wall drainage detail is presented on Figure 8. If conditions different than those described are expected, or if specific drainage details are desired, Geocon Incorporated should be contacted for additional recommendations.

8.12.7 In general, wall foundations having a minimum depth and width of one foot may be designed for an allowable soil bearing pressure of 2,000 psf, provided the soil within three feet below the base of the wall has an Expansion Index ≤ 90. The recommended allowable soil bearing pressure may be increased by 300 psf and 500 psf for each additional foot of foundation width and depth, respectively, up to a maximum allowable soil bearing pressure of 4,000 psf.

8.12.8 The proximity of the foundation to the top of a slope steeper than 3:1 could impact the allowable soil bearing pressure. Therefore, Geocon Incorporated should be consulted where such a condition is anticipated. As a minimum, wall footings should be deepened such that the bottom outside edge of the footing is at least seven feet from the face of slope when located adjacent and/or at the top of descending slopes.

8.12.9 The structural engineer should determine the Seismic Design Category for the project in accordance with Section 1613.3.5 of the 2016 CBC or Section 11.6 of ASCE 7-10. For structures assigned to Seismic Design Category of D, E, or F, retaining walls that support more than 6 feet of backfill should be designed with seismic lateral pressure in accordance with Section 1803.5.12 of the 2016 CBC. The seismic load is dependent on the retained height where H is the height of the wall, in feet, and the calculated loads result in pounds per square foot (psf) exerted at the base of the wall and zero at the top of the wall. A seismic load of 19H should be used for design. We used the peak ground acceleration adjusted for Site Class effects, PGA_M, of 0.364g calculated from ASCE 7-10 Section 11.8.3 and applied a pseudo-static coefficient of 0.33.

8.12.10 For resistance to lateral loads, a passive earth pressure equivalent to a fluid density of 300 pcf is recommended for footings or shear keys poured neat against properly compacted granular fill soils or undisturbed formational materials. The passive pressure assumes a horizontal surface extending away from the base of the wall at least five feet or three times the surface generating the passive pressure, whichever is greater. The upper 12 inches of material not protected by floor slabs or pavement should not be included in the design for lateral resistance.

8.12.11 An ultimate friction coefficient of 0.40 may be used for resistance to sliding between soil and concrete. This friction coefficient may be combined with the passive earth pressure when determining resistance to lateral loads.

8.12.12 The recommendations presented above are generally applicable to the design of rigid concrete or masonry retaining walls having a maximum height of 12 feet. In the event that walls higher than 12 feet are planned, Geocon Incorporated should be consulted for additional recommendations.

8.13 Mechanically Stabilized Earth (MSE) Retaining Walls

8.13.1 Mechanized stabilized earth (MSE) retaining walls are alternative walls that consist of modular block facing units with geogrid reinforced earth behind the block. The reinforcement grid attaches to the block units and is typically placed at specified vertical intervals and embedment lengths. The grid length and spacing will be determined by the wall designer. The designer should also check that sufficient horizontal distance exists to install the grids without having to excavate into the slope as the slope face consists of very strong rock material or rock fill.

8.13.2 The geotechnical parameters listed in Table 8.13 may be used for preliminary design of the MSE walls.

TABLE 8.13
GEOTECHNICAL PARAMETERS FOR MSE WALLS

Parameter	Reinforced Zone	Retained Zone	Foundation Zone
Angle of Internal Friction	32 degrees	32 degrees	32 degrees
Cohesion	0 psf	0 psf	0 psf
Wet Unit Density	125 pcf	125 pcf	125 pcf

8.13.3 The shear strength values used for the reinforced zone assume that predominately granular materials will be stockpiled for use as backfill. Geocon has no way of knowing whether these materials will actually be used as backfill behind the wall during construction. As such, once backfill materials have been selected and/or stockpiled, sufficient shear tests should be conducted on samples of the proposed backfill materials to verify they conform to actual design values. Results should be provided to the designer to re-evaluate stability of the walls. Dependent upon test results, the designer may require modifications to the original wall design (e.g., longer geogrid embedment lengths).

8.13.4 Wall foundations having a minimum depth and width of one foot may be designed for an allowable soil bearing pressure of 2,000 psf. This soil pressure may be increased by 300 psf and 500 psf for each additional foot of foundation width and depth, respectively, up to a maximum allowable soil bearing pressure of 4,000 psf.

8.13.5 Backfill materials within the reinforced zone should be compacted to a dry density of at least 90 percent of the laboratory maximum dry density near to slightly above optimum moisture content in accordance with ASTM D 1557. This is applicable to the entire embedment width of the reinforcement. Typically, wall designers specify no heavy compaction equipment within 3 feet of the face of the wall. However, smaller equipment (e.g., walk-behind, self-driven compactors or hand whackers) can be used to compact the materials without causing deformation of the wall. If the designer specifies no compactive effort for this zone, the materials are essentially not properly compacted and the reinforcement grid within the uncompacted zone should not be relied upon for reinforcement, and overall embedment lengths will have to be increased to account for the difference.

8.13.6 The wall designer should provide a drainage system sufficient to dissipate hydrostatic pressure behind the wall and to mitigate seepage through and beneath the wall. As such, a subdrain system consisting of a minimum 4-inch diameter, Schedule 40, perforated pvc pipe surrounded by at least 1 cubic foot of ¾-inch open-graded gravel and wrapped in filter fabric (Mirafi 140N or equivalent) should be incorporated into the wall design. In order to prevent soil piping into the open-graded gravel layer behind the wall, we recommend the filter fabric be extended to cover the entire gravel layer. The final segment of the subdrain should outlet into an approved drainage facility, such as storm drain or headwall structure. The final segment of the drain should consist of solid pvc pipe. At the transition between the solid and perforated pipe, a concrete cut-off wall should be added to direct the subsurface water into the solid pipe. Typical wall and drain details are presented herein (see Figure 8).

8.13.7 A peak ground acceleration adjusted for Site Class effects, PGA_M, of 0.399g was calculated from ASCE 7-10 Section 11.8.3.

8.13.8 Geosynthetic reinforcement must elongate to develop full tensile resistance. This elongation generally results in movement at the top of the wall. The amount of movement is dependent upon the height of the wall (e.g., higher walls rotate more) and the type of reinforcing grid used. In addition, over time the reinforcement grid has been known to exhibit creep (sometimes as much as 5 percent) and can undergo additional movement. Given this condition, the owner should be aware that structures and pavement placed within the reinforced and retained zones of the wall may undergo movement.

8.13.9 The MSE wall contractor should provide the estimated deformation of wall and adjacent ground in associated with wall construction. The calculated horizontal and vertical deformations should be determined by the wall designer. The estimated movements should be provided to the project structural engineer to determine if the planned improvements can tolerate the expected movements.

8.13.10 The MSE wall designer/contractor should review this report and incorporate our recommendations as presented herein. We should review the MSE wall plans to check if they are in conformance with our recommendations prior to issuance of a permit and construction.

8.14 Slope Maintenance

8.14.1 Slopes that are steeper than 3:1 (horizontal:vertical) may, under conditions that are both difficult to prevent and predict, be susceptible to near-surface (surficial) slope instability. The instability is typically limited to the outer 3 feet of a portion of the slope and usually does not directly impact the improvements on the pad areas above or below the slope. The occurrence of surficial instability is more prevalent on fill slopes and is generally preceded by a period of heavy rainfall, excessive irrigation, or the migration of subsurface seepage. The disturbance and/or loosening of the surficial soils, as might result from root growth, soil expansion, or excavation for irrigation lines and slope planting, may also be a significant contributing factor to surficial instability. It is, therefore, recommended that, to the maximum extent practical: (a) disturbed/loosened surficial soils be either removed or properly recompacted, (b) irrigation systems be periodically inspected and maintained to eliminate leaks and excessive irrigation, and (c) surface drains on and adjacent to slopes be periodically maintained to preclude ponding or erosion. It should be noted that although the incorporation of the above recommendations should reduce the potential for surficial slope instability, it will not eliminate the possibility, and, therefore, it may be necessary to rebuild or repair a portion of the project's slopes in the future.

8.15 Site Drainage and Moisture Protection

8.15.1 Adequate site drainage is critical to reduce the potential for differential soil movement, erosion and subsurface seepage. Under no circumstances should water be allowed to pond adjacent to footings. The site should be graded and maintained such that surface drainage is directed away from structures in accordance with 2016 CBC 1804.4 or other applicable standards. In addition, surface drainage should be directed away from the top of slopes into swales or other controlled drainage devices. Roof and pavement drainage should be directed into conduits that carry runoff away from the proposed structure.

8.15.2 Underground utilities should be leak free. Utility and irrigation lines should be checked periodically for leaks, and detected leaks should be repaired promptly. Detrimental soil movement could occur if water is allowed to infiltrate the soil for prolonged periods of time.

8.16 Grading and Foundation Plan Review

8.16.1 Geocon Incorporated should review the grading plans and foundation plans for the project prior to final design submittal to evaluate whether additional analyses and/or recommendations are required.

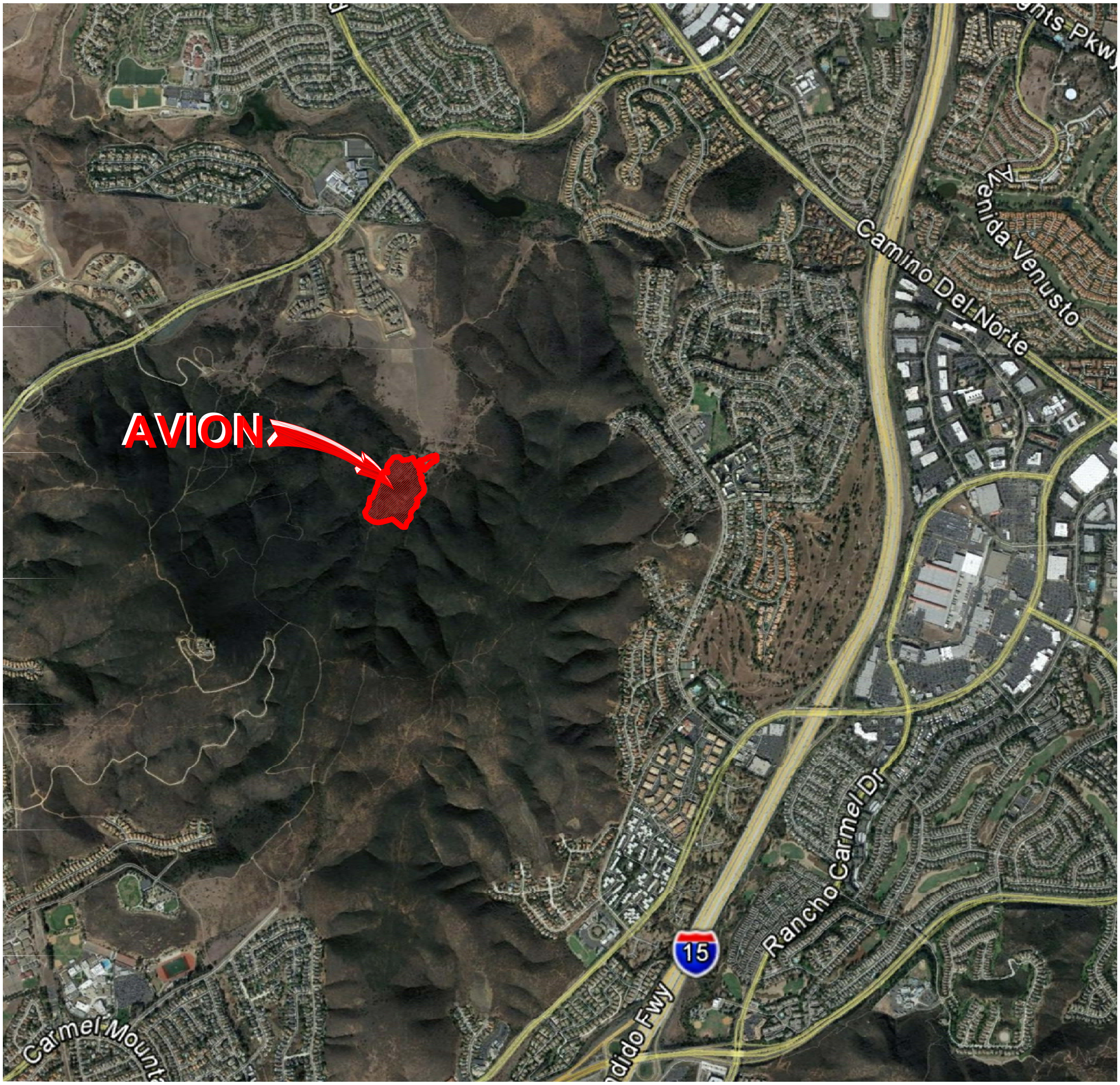
LIMITATIONS AND UNIFORMITY OF CONDITIONS

1. 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.

2. 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 Incorporated 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 Incorporated.

3. This report is issued with the understanding that it is the responsibility of the owner or 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.

4. The findings of this report are valid as of the present date. However, changes in the conditions of a property can occur with the passage of time, whether they be 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.



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N

NO SCALE

VICINITY MAP

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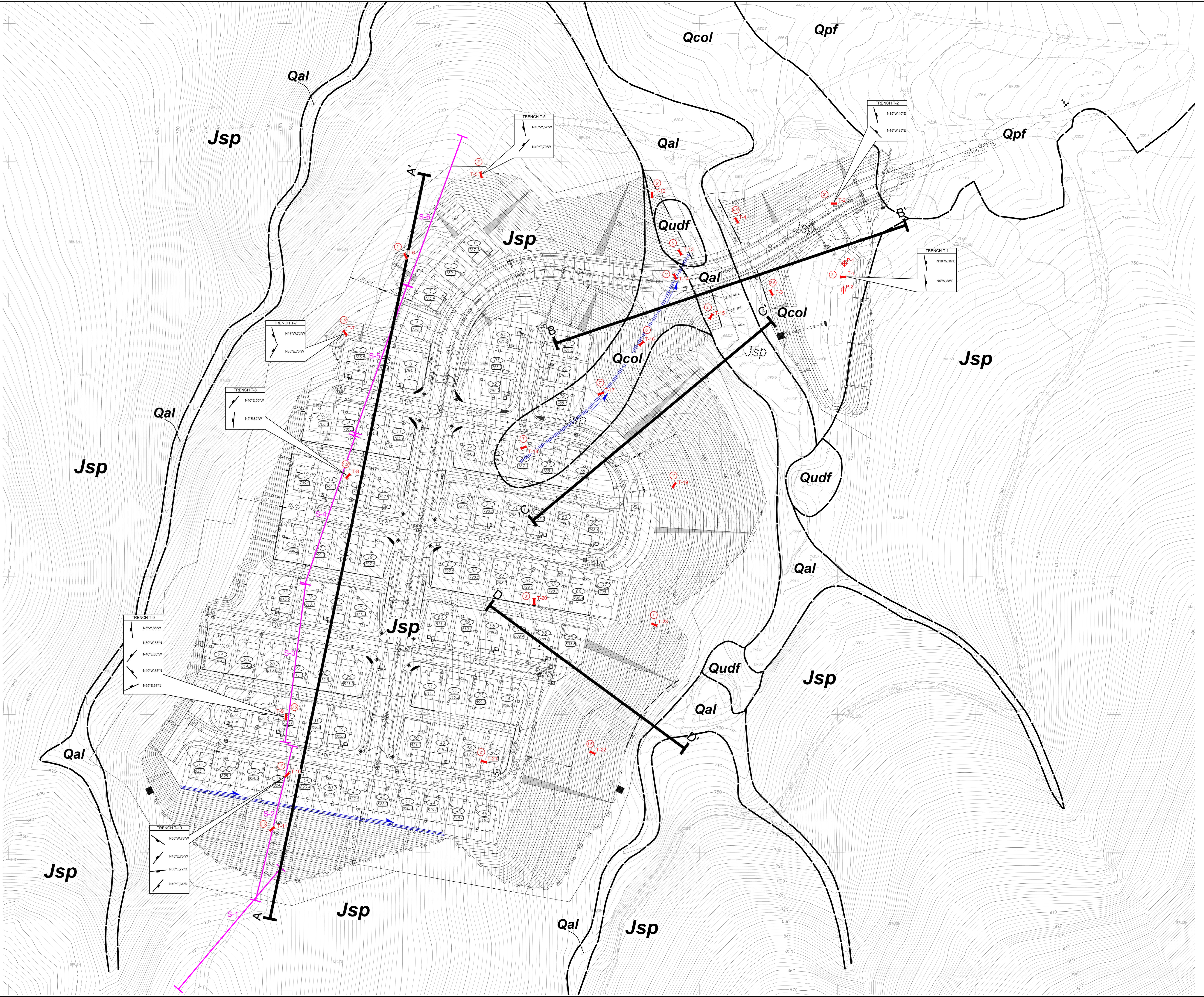
SAN DIEGO, CALIFORNIA

JP / RA		DSK/GTYPD	DATE 08 - 24 - 2018	PROJECT NO. G2213 - 32 - 01	FIG. 1



0' 50' 100' 150' 200'

SCALE 1"=50' (On 42x30)

GEOCON LEGEND

- **Qpf** ....PREVIOUSLY PLACED FILL
- **Qudf** ........UNDOCUMENTED FILL
- **Qal** ....ALLUVIUM
- **Qcol** ....COLLUVIUM
- **Jsp** ....SANTIAGO PEAK VOLCANICS (Dotted Where Buried)
- T-23 ....APPROX. LOCATION OF EXPLORATORY TRENCH
- P-2 ....APPROX. LOCATION OF INFILTRATION TEST
- 7' ....APPROX. THICKNESS OF REMEDIAL GRADING (In Feet)
- ....PROPOSED LOCATION OF CANYON SUBDRAIN
- ....PROPOSED LOCATION OF TOE DRAIN
- ....APPROX. LOCATION OF GEOLOGIC CONTACT
- S-6 ....APPROX. LOCATION OF SEISMIC REFRACTION TRAVERSE
- D D' ....APPROX. LOCATION OF GEOLOGIC CROSS-SECTION
- ....APPROX. STRIKE AND DIP OF JOINT/FRACTURE IN METAVOLCANIC ROCK

GEOLOGIC MAP

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SCALE 1" = 50'	DATE 08 - 24 - 2018	FIGURE 2
PROJECT NO. G2213 - 32 - 01		
SHEET 1 OF 1		

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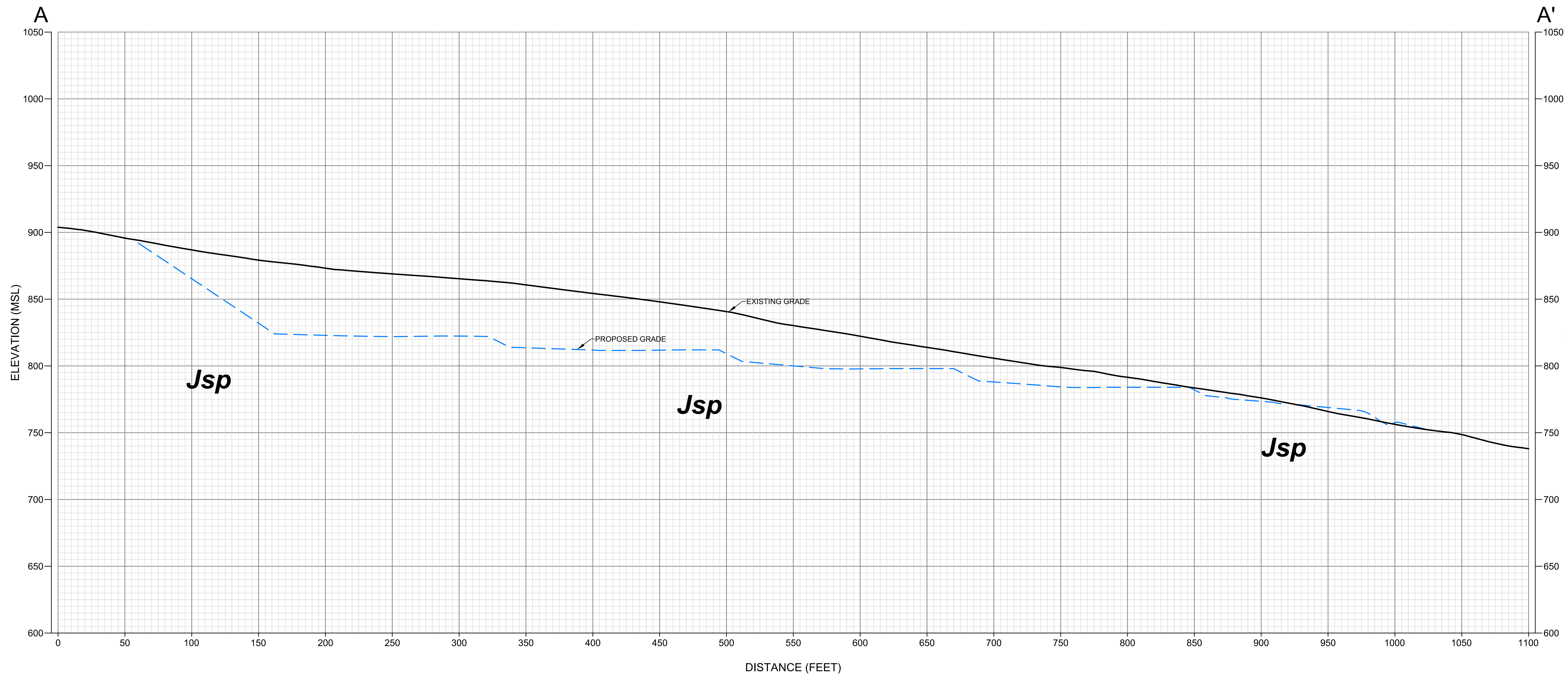
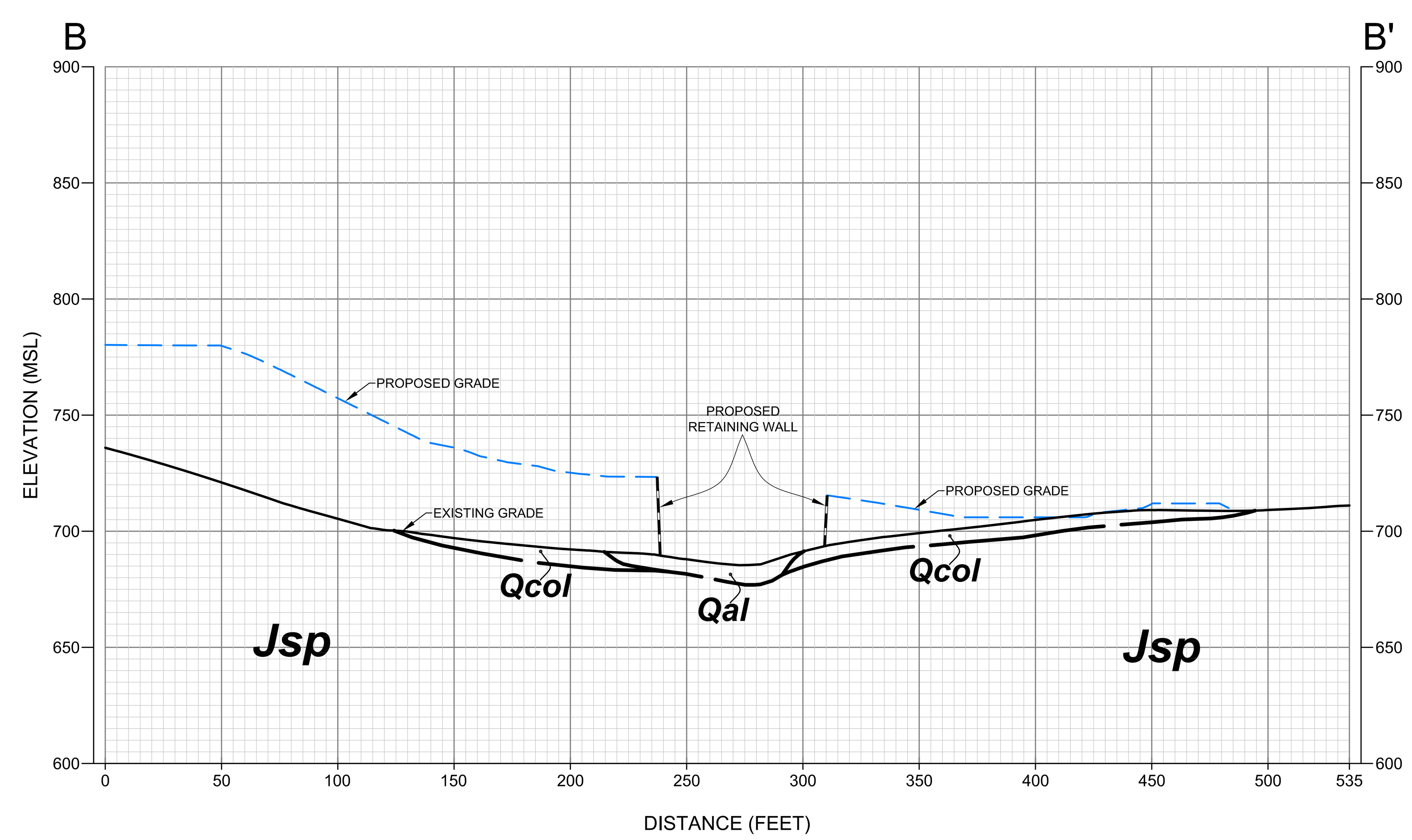


GEOLOGIC CROSS-SECTION A-A'

SCALE: 1" = 50' (Vert. = Horiz.)



GEOLOGIC CROSS-SECTION B-B'

SCALE: 1" = 50' (Vert. = Horiz.)

GEOCON LEGEND

Qal ....ALLUVIUM

Qcol ....COLLUVIUM

Jsp ....SANIAGO PEAK VOLCANICS

........APPROX. LOCATION OF GEOLOGIC CONTACT

GEOLOGIC CROSS - SECTION

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SCALE 1" = 50'	DATE 08 - 24 - 2018
PROJECT NO. G2213 - 32 - 01	FIGURE 3
SHEET 1 OF 2	

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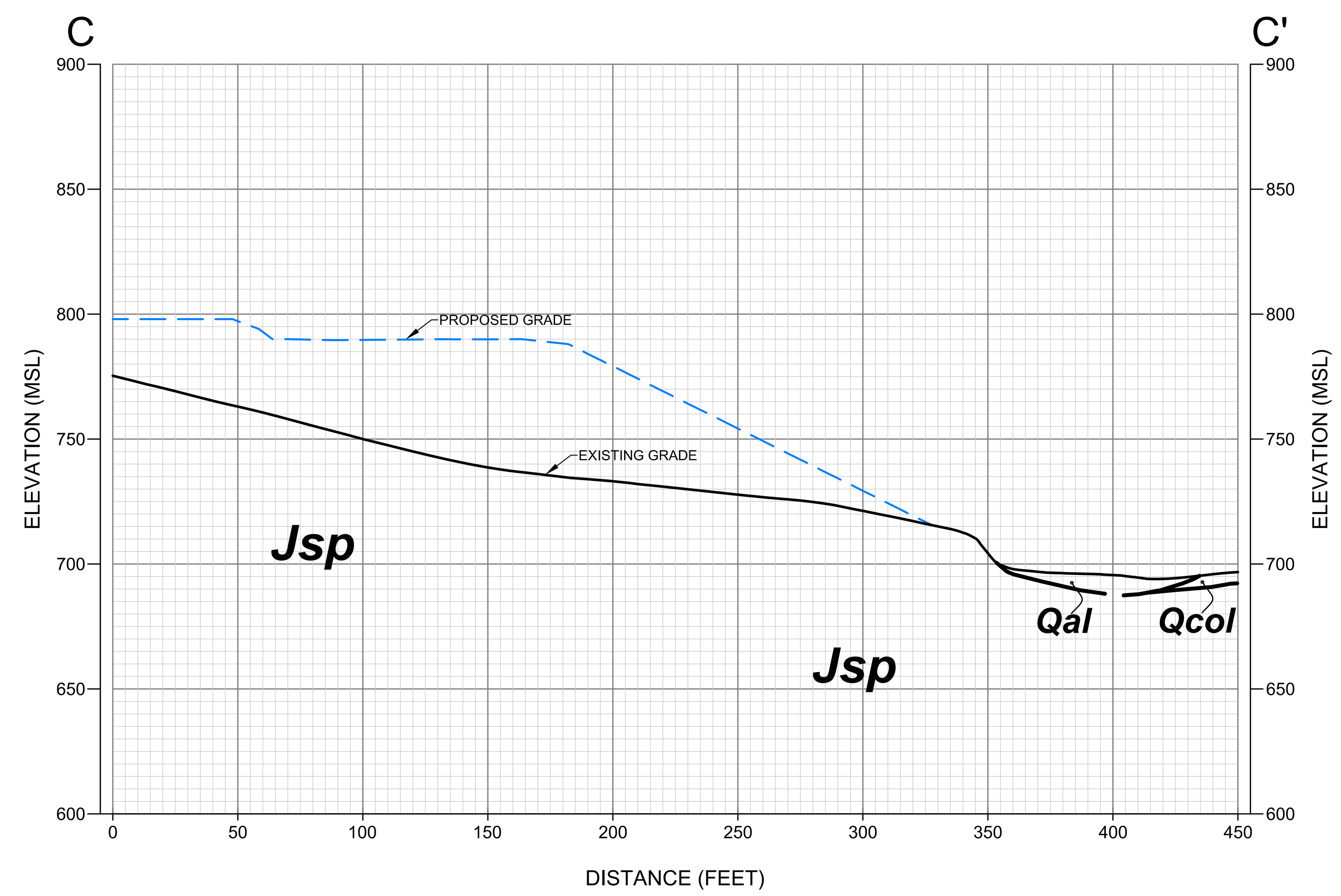
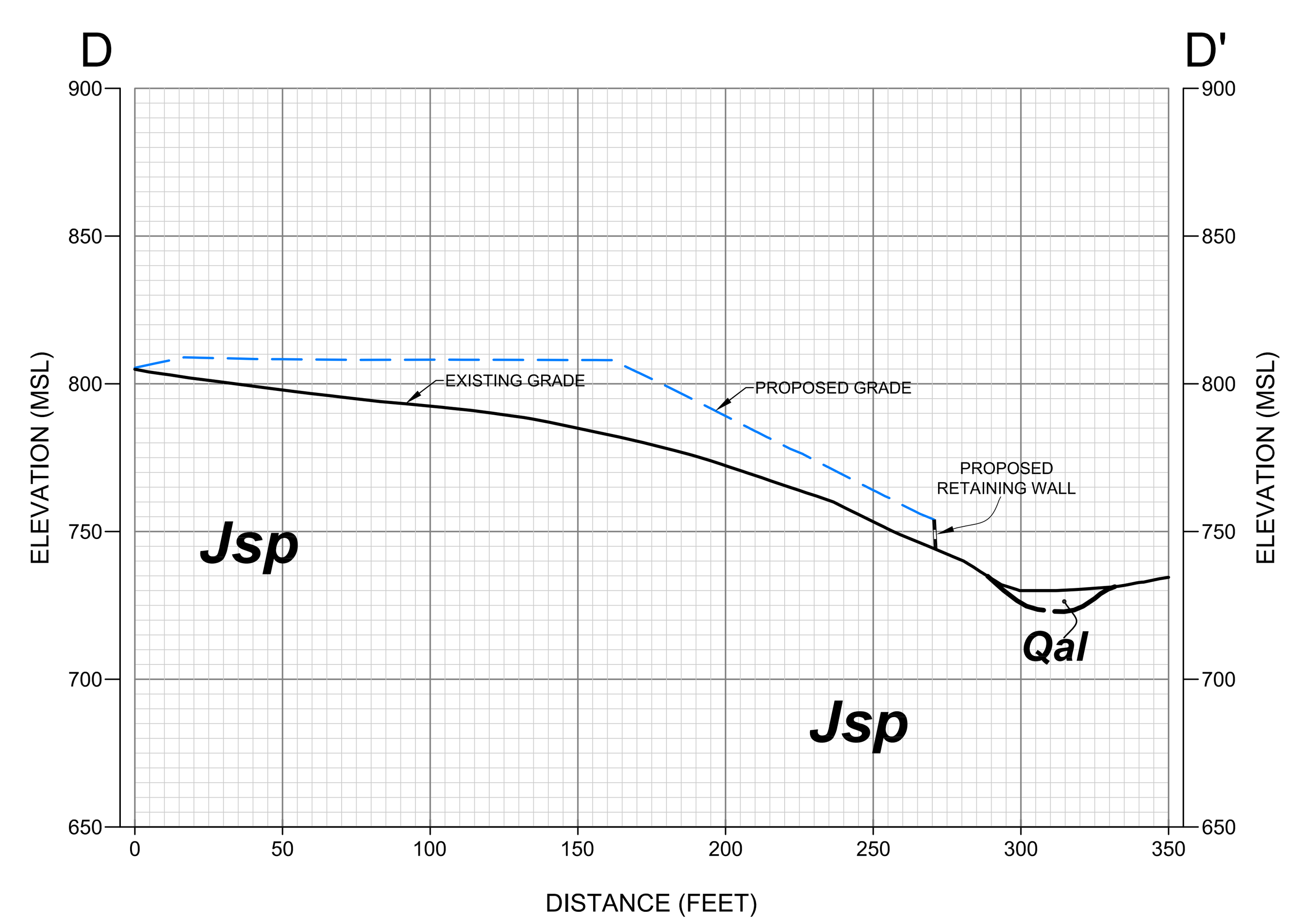


GEOLOGIC CROSS-SECTION C-C'

SCALE: 1" = 50' (Vert. = Horiz.)



GEOLOGIC CROSS-SECTION D-D'

SCALE: 1" = 50' (Vert. = Horiz.)

GEOCON LEGEND

Qal ....ALLUVIUM

Qcol ....COLLUVIUM

Jsp ....SANIAGO PEAK VOLCANICS

........APPROX. LOCATION OF GEOLOGIC CONTACT

GEOLOGIC CROSS - SECTION

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SCALE 1" = 50'	DATE 08 - 24 - 2018
PROJECT NO. G2213 - 32 - 01	FIGURE 4
SHEET 2 OF 2	

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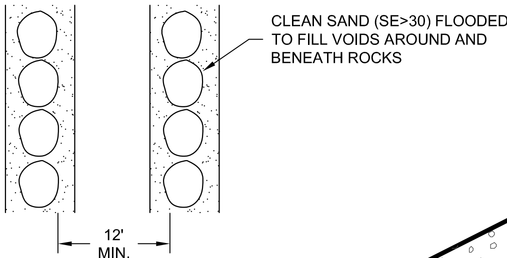
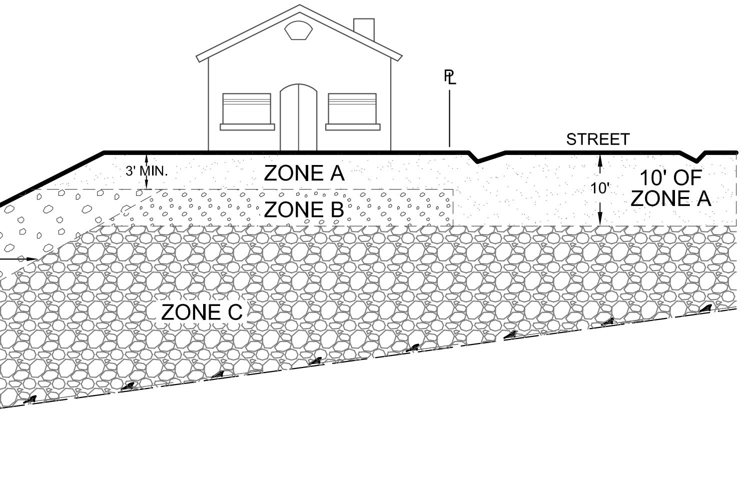
ZONE B

WINDROWS DETAIL
(PLAN VIEW)





NO SCALE

LEGEND

ZONE A: COMPACTED SOIL FILL. NO ROCK FRAGMENTS OVER 6 INCHES IN DIMENSION. 3 FEET THICK FOR RESIDENTIAL PADS.

ZONE B: ROCKS UP TO 1 FOOT IN MAXIMUM DIMENSION IN A MATRIX OF COMPACTED "SOIL FILL" WITHIN BUILDING PADS.

ZONE C: ROCK OR SOIL-ROCK FILL GENERALLY CONSISTING OF 2 FOOT MINUS MATERIAL WITH OCCASIONAL INDIVIDUAL FRAGMENTS UP TO 4 FEET MAXIMUM DIMENSION
ALTERNATE: ROCKS 2 TO 4 FEET IN MAXIMUM DIMENSION CAN BE PLACED IN WINDROWS IN COMPACTED SOIL FILL POSSESSING A SAND EQUIVALENT OF AT LEAST 30.

ZONE D: ROCKS UP TO 12 INCHES IN MAXIMUM DIMENSION IN A MATRIX OF COMPACTED "SOIL FILL".

NOTES

1. COMPACTED "SOIL FILL" IN UPPER 3 FEET SHALL CONTAIN AT LEAST 40 PERCENT SOIL PASSING THE 3/4 - INCH SIEVE (BY WEIGHT).

2. CONTINUOUS OBSERVATION REQUIRED BY GEOCON DURING ROCK PLACEMENT.

3. ROCK FILL (LESS THAN 40 PERCENT SOIL SIZES) MAY BE PERMITTED IN DESIGNATED AREAS UPON APPROVAL OF THE GEOTECHNICAL ENGINEER.

4. COMBINED DEPTH OF ZONE A AND B SHOULD BE AT LEAST 10 FEET AND SHOULD BE EXTENDED TO AT LEAST 2 FEET BELOW DEEPEST UTILITY WITHIN ROADWAYS.

5. SUBDRAINS PLACED AT THE BASE OF ROCK/SOIL-ROCK FILLS MAY BE NECESSARY. THE NECESSITY FOR THE DRAINS WILL BE EVALUATED BY THE GEOTECHNICAL ENGINEER DURING GRADING.

OVERSIZE ROCK DISPOSAL

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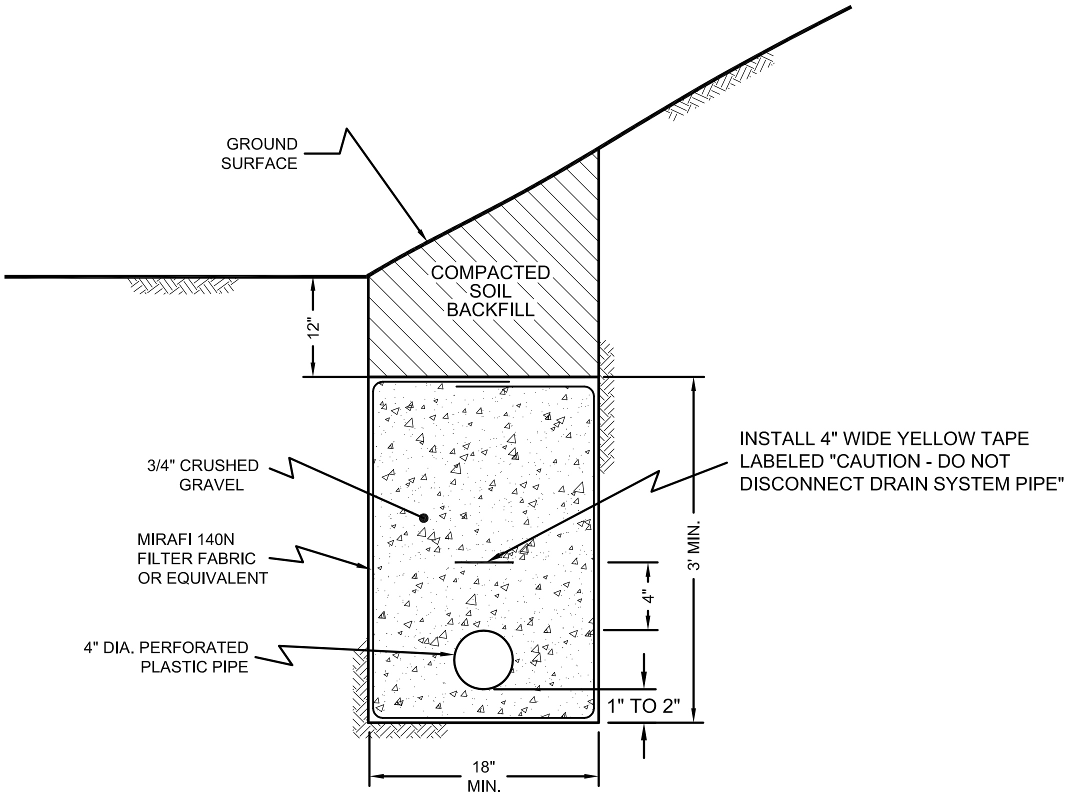
JP / RA		DSK/GTYPD	DATE 08 - 24 - 2018	PROJECT NO. G2213 - 32 - 01	FIG. 5



NOTES:

1. SOIL CAP COMPACTED TO 90% RELATIVE COMPACTION
2. PERMEABLE MATERIAL SHALL BE ¾" CRUSHED GRAVEL WRAPPED IN FILTER FABRIC (MIRAFI 140N OR EQUIVALENT)
3. 4" DIAMETER PERFORATED PIPE (SDR-35 OR EQUIVALENT) WITH PERFORATIONS DOWN
4. PIPE TO MAINTAIN A MINIMUM 1 PERCENT FALL
5. CONCRETE CUTOFF WALL TO BE PROVIDED AT TRANSITION TO SOLID OUTLET PIPE
6. SOLID OUTLET PIPE TO DRAIN TO APPROVED AREA
7. CLEANOUTS ARE REQUIRED AT EACH PROPERTY LINE

NO SCALE

RECOMMENDED SLOPE TOE DRAIN DETAIL

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JP / RA		DSK/GTYPD	DATE 08 - 24 - 2018	PROJECT NO. G2213 - 32 - 01	FIG. 6

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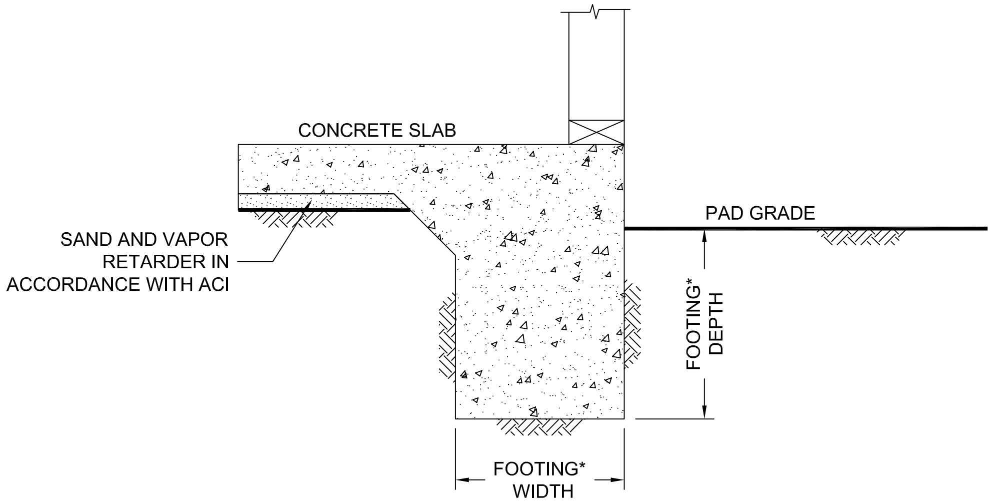

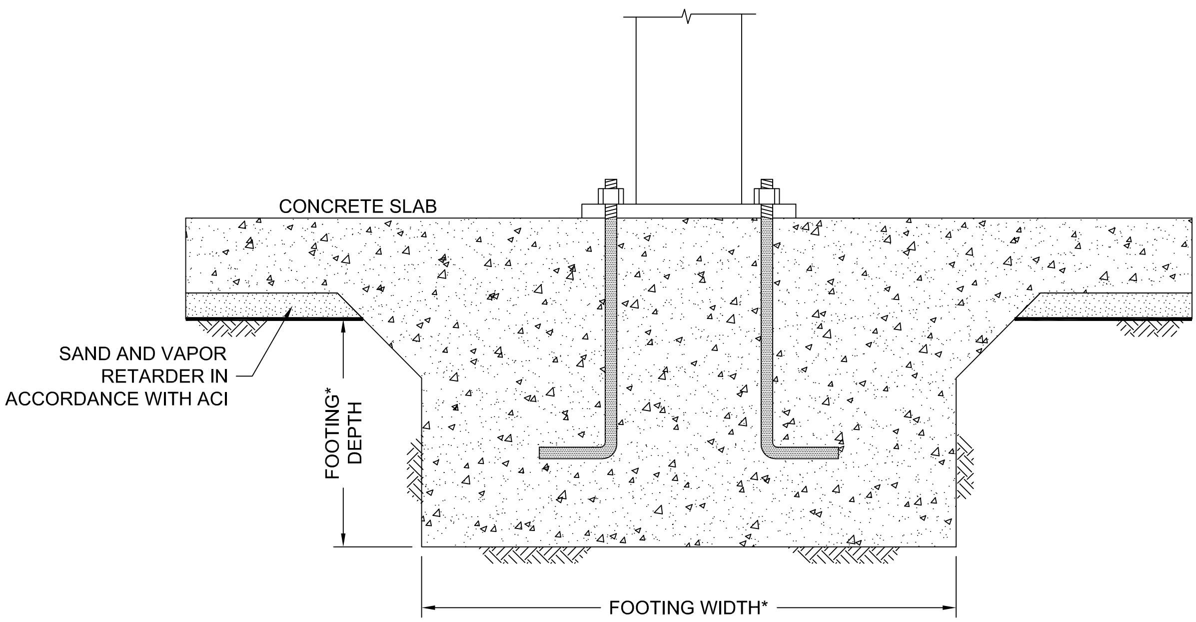

*....SEE REPORT FOR FOUNDATION WIDTH AND DEPTH RECOMMENDATION

NO SCALE

WALL / COLUMN FOOTING DIMENSION DETAIL

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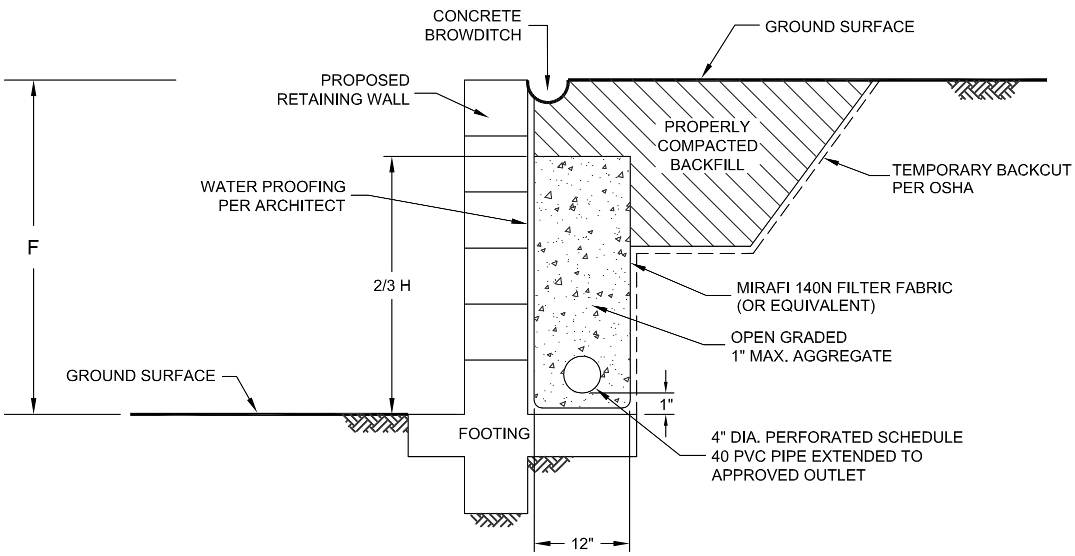
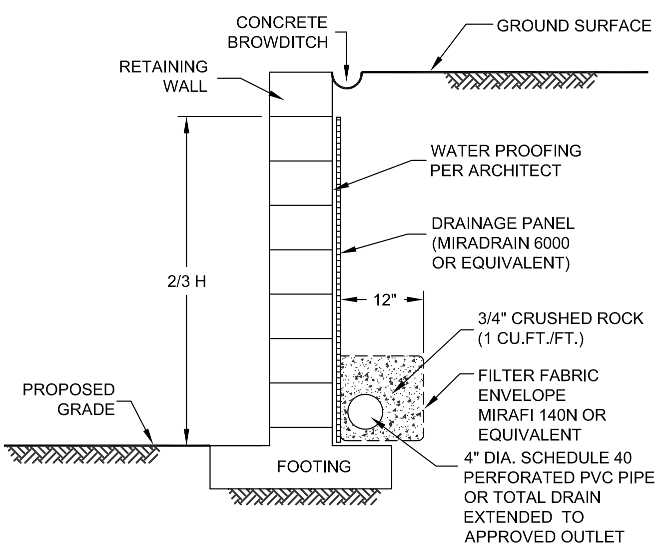
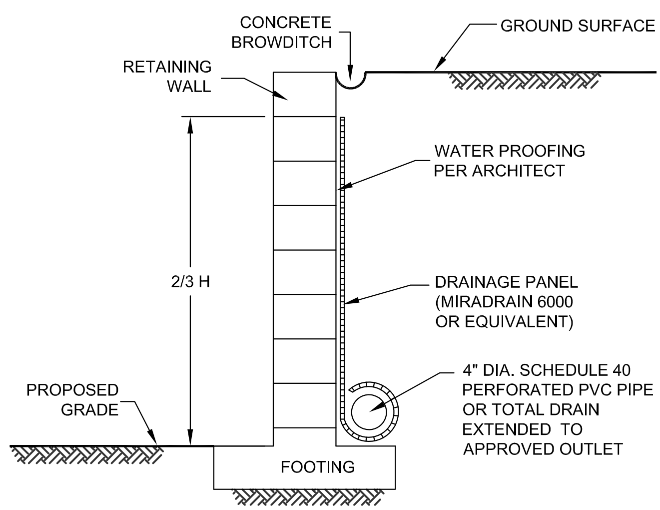
JP / RA		DSK/GTYPD	DATE 08 - 24 - 2018	PROJECT NO. G2213 - 32 - 01	FIG. 7







NOTE :

DRAIN SHOULD BE UNIFORMLY SLOPED TO GRAVITY OUTLET
OR TO A SUMP WHERE WATER CAN BE REMOVED BY PUMPING

NO SCALE

TYPICAL RETAINING WALL DRAIN DETAIL

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APPENDIX A

APPENDIX A

FIELD INVESTIGATION

The field investigation was performed in May 2018, and consisted of a visual site reconnaissance and excavating 23 exploratory trenches at various locations across the subject site. In addition, 2 infiltration tests were performed using an Aardvark constant head permeameter. The approximate locations of the trenches and infiltration tests are shown on the *Geologic Map*, Figure 2. The results and discussion of the infiltration testing is discussed in *Appendix D* of this report.

Exploratory trenches T-1 through T-23 were performed by Hillside Excavating and were advanced to depths of 2½ to 9½ feet using a John Deere 555 track-mounted backhoe equipped with a 24-inch-wide bucket. Bulk samples were obtained for laboratory testing. Logs of the trenches depicting the soil and geologic conditions encountered are presented on Figures A-1 through A-23.

The soils encountered in the excavations were visually classified and logged in general accordance with American Society for Testing and Materials (ASTM) practice for Description and Identification of Soils (Visual Manual Procedure D 2488).

TRENCH T 1

ELEV. (MSL.) **711'** DATE COMPLETED **05-21-2018**

EQUIPMENT **JD555 BACKHOE (W/24" BUCKET)** BY: **J. PAGNILLO**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0				ML	**COLLUVIUM (Qcol)** Soft, damp, reddish brown, fine, Sandy SILT			
	T1-1							
2					**SANTIAGO PEAK VOLCANICS (Jsp)** Slightly weathered, brownish gray, strong to very strong METAVOLCANIC ROCK; moderately jointed; excavates to Silty, fine to coarse SAND with angular rock fragments up to 8" size			
4					PRACTICAL REFUSAL AT 4 FEET Groundwater not encountered			

Figure A-1,
Log of Trench T 1, Page 1 of 1

G2213-32-01.GPJ

SAMPLE SYMBOLS			
	... SAMPLING UNSUCCESSFUL	... STANDARD PENETRATION TEST	... DRIVE SAMPLE (UNDISTURBED)
	... DISTURBED OR BAG SAMPLE	... 1 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.

TRENCH T 2

ELEV. (MSL.) 701' DATE COMPLETED 05-21-2018

EQUIPMENT JD555 BACKHOE (W/24" BUCKET) BY: J. PAGNILLO

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0				ML	**COLLUVIUM (Qcol)** Soft, damp, reddish brown, fine, Sandy SILT			
2					**SANTIAGO PEAK VOLCANICS (Jsp)** Moderately weathered, brownish gray, strong to very strong, METAVOLCANIC ROCK; moderately jointed; excavates to Silty/fine to coarse, Sandy GRAVEL with cobble sized rock fragments			
					REFUSAL ON FRESH ROCK AT 3.5 FEET Groundwater not encountered			

Figure A-2,
Log of Trench T 2, Page 1 of 1

G2213-32-01.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. G2213-32-01

TRENCH T 3

ELEV. (MSL.) **697'** DATE COMPLETED **05-21-2018**

EQUIPMENT **JD555 BACKHOE (W/24" BUCKET)** BY: **J. PAGNILLO**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0				ML	**COLLUVIUM (Qcol)** Soft, damp, reddish brown, fine, Sandy SILT			
2					-6" clay layer at base of topsoil			
					SANTIAGO PEAK VOLCANICS (Jsp) Slightly weathered, brownish gray, very strong, METAVOLCANIC ROCK; moderately jointed; excavates to Sandy GRAVEL			

REFUSAL AT 3 FEET
Groundwater not encountered

Figure A-3,
Log of Trench T 3, Page 1 of 1

G2213-32-01.GPJ

SAMPLE SYMBOLS			
	... SAMPLING UNSUCCESSFUL	... STANDARD PENETRATION TEST	... DRIVE SAMPLE (UNDISTURBED)
	... DISTURBED OR BAG SAMPLE	... CHUNK SAMPLE	... WATER TABLE OR SEEPAGE

TRENCH T 4

ELEV. (MSL.) **690'** DATE COMPLETED **05-21-2018**

EQUIPMENT **JD555 BACKHOE (W/24" BUCKET)** BY: **J. PAGNILLO**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0				CL	**COLLUVIUM (Qcol)** Soft, damp, dark brownish gray, Sandy CLAY with angular metavolcanic rock fragments up to 6" size			
2	T4-1							
4								
					SANTIAGO PEAK VOLCANICS (Jsp) Completely weathered, olive green, weak, METAVOLCANIC ROCK with calcium carbonate; excavates to Silty/Clayey SAND with rock fragments up to 6" size			
6	T4-2							
					-Becomes fresh rock, bluish green and very strong			
					REFUSAL ON FRESH ROCK AT 7.5 FEET Groundwater not encountered			

Figure A-4,
Log of Trench T 4, Page 1 of 1

G2213-32-01.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.

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PROJECT NO. G2213-32-01

TRENCH T 5

ELEV. (MSL.) 733' DATE COMPLETED 05-21-2018

EQUIPMENT JD555 BACKHOE (W/24" BUCKET) BY: J. PAGNILLO

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0	T5-1			ML	**TOPSOIL** Soft, damp, reddish brown, Sandy SILT			
2					**SANTIAGO PEAK VOLCANICS (Jsp)** Moderately weathered, bluish gray, very strong, METAVOLCANIC ROCK; moderately jointed; excavates to Sandy GRAVEL with angular rock fragments up to 8" size			
4					REFUSAL ON SLIGHTLY WEATHERED ROCK AT 4 FEET Groundwater not encountered			

Figure A-5,
Log of Trench T 5, Page 1 of 1

G2213-32-01.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

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	TRENCH T 6 ELEV. (MSL.) 760' DATE COMPLETED 05-21-2018 EQUIPMENT JD555 BACKHOE (W/24" BUCKET) BY: J. PAGNILLO	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					MATERIAL DESCRIPTION			
0				ML	**TOPSOIL** Soft, damp, reddish brown, Sandy SILT			
2					**SANTIAGO PEAK VOLCANICS (Jsp)** Highly weathered, brownish gray, moderately strong, METAVOLCANIC ROCK -Becomes slightly weathered, bluish gray and very strong; excavates to Sandy GRAVEL with angular rock fragments up to 8" size; slightly jointed			
4					REFUSAL AT 4 FEET Groundwater not encountered			

Figure A-6,
Log of Trench T 6, Page 1 of 1

G2213-32-01.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

TRENCH T 7

ELEV. (MSL.) **775'** DATE COMPLETED **05-21-2018**

EQUIPMENT **JD555 BACKHOE (W/24" BUCKET)** BY: **J. PAGNILLO**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0				ML	**TOPSOIL** Soft, damp, reddish brown, Sandy SILT			
2	T7-1				**SANTIAGO PEAK VOLCANICS (Jsp)** Moderately weathered, brownish gray, moderately strong, METAVOLCANIC ROCK; excavates to Sandy GRAVEL with rock fragments up to 6" size -Becomes slightly weathered, strong to very strong and moderately jointed			
4								
					REFUSAL AT 4.5 FEET Groundwater not encountered			

Figure A-7,
Log of Trench T 7, Page 1 of 1

G2213-32-01.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. G2213-32-01

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	TRENCH T 8 ELEV. (MSL.) 816' DATE COMPLETED 05-21-2018 EQUIPMENT JD555 BACKHOE (W/24" BUCKET) BY: J. PAGNILLO	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					MATERIAL DESCRIPTION			
0				ML	**TOPSOIL** Soft, damp, reddish brown, Sandy SILT			
2					**SANTIAGO PEAK VOLCANICS (Jsp)** Highly weathered, brownish gray, moderately strong, METAVOLCANIC ROCK			
4	T8-1				-Becomes moderately weathered, bluish gray, very strong METAVOLCANIC ROCK; moderately jointed; excavates to Sandy GRAVEL with angular rock fragments up to 8" size			
					REFUSAL ON SLIGHTLY WEATHERED ROCK AT 4.5 FEET Groundwater not encountered			

Figure A-8,
Log of Trench T 8, Page 1 of 1

G2213-32-01.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

TRENCH T 9

ELEV. (MSL.) **869'** DATE COMPLETED **05-21-2018**

EQUIPMENT **JD555 BACKHOE (W/24" BUCKET)** BY: **J. PAGNILLO**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0				ML	**TOPSOIL** Soft, damp, reddish brown, Sandy SILT			
2					**SANTIAGO PEAK VOLCANICS (Jsp)** Slightly weathered, brownish gray, very strong, METAVOLCANIC ROCK with moderately jointed, excavates to Sandy GRAVEL with angular rock fragments up to 8" size -Becomes bluish gray and extremely strong with angular rock fragments up to 14" size			
					REFUSAL AT 3.5 FEET Groundwater not encountered			

Figure A-9,
Log of Trench T 9, Page 1 of 1

G2213-32-01.GPJ

SAMPLE SYMBOLS			
	... SAMPLING UNSUCCESSFUL	... STANDARD PENETRATION TEST	... DRIVE SAMPLE (UNDISTURBED)
	... DISTURBED OR BAG SAMPLE	... CHUNK SAMPLE	... WATER TABLE OR SEEPAGE

TRENCH T 10

ELEV. (MSL.) **874'** DATE COMPLETED **05-21-2018**

EQUIPMENT **JD555 BACKHOE (W/24" BUCKET)** BY: **J. PAGNILLO**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0				ML	**TOPSOIL** Soft, damp, reddish brown, Sandy SILT			
					SANTIAGO PEAK VOLCANICS (Jsp) Highly weathered, reddish brown, moderately weak, METAVOLCANIC ROCK			
2	T10-1				-Becomes slightly weathered, bluish gray and very strong METAVOLCANIC ROCK; moderately jointed; excavates to Sandy GRAVEL; angular rock fragments up to 10" size with very little fines			
4					REFUSAL AT 4 FEET Groundwater not encountered			

Figure A-10,
Log of Trench T 10, Page 1 of 1

G2213-32-01.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.

TRENCH T 11

ELEV. (MSL.) 884' DATE COMPLETED 05-21-2018

EQUIPMENT JD555 BACKHOE (W/24" BUCKET) BY: J. PAGNILLO

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0	T11-1			ML	**TOPSOIL** Soft, damp, reddish brown, Sandy SILT			
2					**SANTIAGO PEAK VOLCANICS (Jsp)** Slightly weathered, bluish gray, very strong, METAVOLCANIC ROCK; excavates to Sandy GRAVEL with angular rock fragments up to 8" size			
					REFUSAL ON FRESH ROCK AT 2.5 FEET Groundwater not encountered			

Figure A-11,
Log of Trench T 11, Page 1 of 1

G2213-32-01.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. G2213-32-01

TRENCH T 12

ELEV. (MSL.) **680'** DATE COMPLETED **05-21-2018**

EQUIPMENT **JD555 BACKHOE (W/24" BUCKET)** BY: **J. PAGNILLO**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0				ML	**ALLUVIUM (Qal)** Soft, dry, grayish brown, Sandy SILT			
2	T12-1			SM	Medium dense, damp, Silty, fine to coarse SAND with angular rock fragments (metavolcanic rock) up to 6" size			
4								
6					**SANTIAGO PEAK VOLCANICS (Jsp)** Moderate to slightly weathered, bluish gray, very strong METAVOLCANIC ROCK; excavates to Sandy GRAVEL with angular rock fragments up to 14" size			
					REFUSAL AT 6.5 FEET Groundwater not encountered			

Figure A-12,
Log of Trench T 12, Page 1 of 1

G2213-32-01.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

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	**TRENCH T 13** ELEV. (MSL.) **684'** DATE COMPLETED **05-21-2018** EQUIPMENT **JD555 BACKHOE (W/24" BUCKET)** BY: **J. PAGNILLO**	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					MATERIAL DESCRIPTION			
0 - 3				SC	**UNDOCUMENTED FILL (Qudf)** Loose, damp, brown, Clayey, fine to coarse SAND with angular rock fragments up to 6" size and some trash debris			
3 - 5				SM	**ALLUVIUM (Qal)** Medium dense, damp, brown, Silty, fine to medium SAND with clay and some angular rock fragments			
5 - 7					**SANTIAGO PEAK VOLCANICS (Jsp)** Highly weathered, dark brownish gray, weak, METAVOLCANIC ROCK; excavates to Clayey SAND with rock fragments up to 6" size -Becomes slightly weathered and very strong			
					PRACTICAL REFUSAL AT 7 FEET Groundwater not encountered			

Figure A-13,
Log of Trench T 13, Page 1 of 1

G2213-32-01.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

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	TRENCH T 14 ELEV. (MSL.) 685' DATE COMPLETED 05-21-2018 EQUIPMENT JD555 BACKHOE (W/24" BUCKET) BY: J. PAGNILLO	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					MATERIAL DESCRIPTION			
0				ML	**COLLUVIUM (Qcol)** Soft, dry, brown, Sandy SILT			
					SANTIAGO PEAK VOLCANICS (Jsp) Highly weathered, brownish gray, moderately strong, METAVOLCANIC ROCK; excavates as Clayey/Sandy GRAVEL with angular rock fragments up to 10" size; intensely jointed			
2								
					-Rock fragments become very strong			
4	T14-1							
					-Becomes moderately weathered			
6					PRACTICAL REFUSAL AT 6 FEET Groundwater not encountered			

Figure A-14,
Log of Trench T 14, Page 1 of 1

G2213-32-01.GPJ

SAMPLE SYMBOLS			
	... SAMPLING UNSUCCESSFUL	... STANDARD PENETRATION TEST	... DRIVE SAMPLE (UNDISTURBED)
	... DISTURBED OR BAG SAMPLE	... CHUNK SAMPLE	... WATER TABLE OR SEEPAGE

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	**TRENCH T 15** ELEV. (MSL.) **694'** DATE COMPLETED **05-21-2018** EQUIPMENT **JD555 BACKHOE (W/24" BUCKET)** BY: **J. PAGNILLO**	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					MATERIAL DESCRIPTION			
0				ML	**ALLUVIUM (Qal)** Soft, damp, dark brown, Sandy SILT			
2					**SANTIAGO PEAK VOLCANICS (Jsp)** Moderately weathered, reddish brown, strong, METAVOLCANIC ROCK; excavates to Silty SAND with angular rock fragments -Becomes slightly weathered, bluish gray and very strong with rock fragments up to 15" size			
4								
					REFUSAL AT 5 FEET Groundwater not encountered			

Figure A-15,
Log of Trench T 15, Page 1 of 1

G2213-32-01.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.

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	**TRENCH T 16** ELEV. (MSL.) **705'** DATE COMPLETED **05-22-2018** EQUIPMENT **JD555 BACKHOE (W/24" BUCKET)** BY: **J. PAGNILLO**	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					MATERIAL DESCRIPTION			
0				ML	**COLLUVIUM (Qcol)** Soft, damp, reddish brown, Sandy SILT; abundant roots			
2	T16-1				-Becomes firm with fewer roots and some angular rock fragments up to 6" size			
4								
6					**SANTIAGO PEAK VOLCANICS (Jsp)** Highly weathered, brownish gray, weak, METAVOLCANIC ROCK; strong angular rock fragments up to 6" size; excavates to Sandy GRAVEL			
8					TRENCH TERMINATED AT 8 FEET Groundwater not encountered			

Figure A-16,
Log of Trench T 16, Page 1 of 1

G2213-32-01.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. G2213-32-01

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	TRENCH T 17 ELEV. (MSL.) 719' DATE COMPLETED 05-22-2018 EQUIPMENT JD555 BACKHOE (W/24" BUCKET) BY: J. PAGNILLO	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					MATERIAL DESCRIPTION			
0				SM	**COLLUVIUM (Qcol)** Loose, damp, reddish brown, Silty, fine SAND with abundant roots and trash debris at surface			
2					-Becomes medium dense with some angular rock fragments up to 6" size			
4	T17-1							
6								
					SANTIAGO PEAK VOLCANICS (Jsp) Highly weathered, brownish gray, weak, METAVOLCANIC ROCK; excavates to Sandy GRAVEL with strong angular rock fragments up to 6" size			
8					TRENCH TERMINATED AT 8 FEET Groundwater not encountered			

Figure A-17,
Log of Trench T 17, Page 1 of 1

G2213-32-01.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

TRENCH T 18

ELEV. (MSL.) **752'** DATE COMPLETED **05-22-2018**

EQUIPMENT **JD555 BACKHOE (W/24" BUCKET)** BY: **J. PAGNILLO**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0				SM	**COLLUVIUM (Qcol)** Loose, damp, reddish brown, Silty, fine SAND with abundant roots and trash debris at surface			
2								
4								
6								
7	T18-1				**SANTIAGO PEAK VOLCANICS (Jsp)** Completely weathered, olive green, weak, METAVOLCANIC ROCK (Saprolite); excavates to Sandy CLAY			
8								
					-Becomes highly weathered, greenish gray, weak, METAVOLCANIC ROCK; excavates to Sandy GRAVEL with moderately weak rock fragments up to 6" size			
					TRENCH TERMINATED AT 9.5 FEET Groundwater not encountered			

Figure A-18,
Log of Trench T 18, Page 1 of 1

G2213-32-01.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. G2213-32-01

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	TRENCH T 19 ELEV. (MSL.) 752' DATE COMPLETED 05-22-2018 EQUIPMENT JD555 BACKHOE (W/24" BUCKET) BY: J. PAGNILLO	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					MATERIAL DESCRIPTION			
0				SM	**TOPSOIL** Loose, dry, brown, Silty SAND			
					SANTIAGO PEAK VOLCANICS (Jsp) Slightly weathered, bluish gray, very strong, METAVOLCANIC ROCK; moderately fractured; excavates to Sandy GRAVEL with angular rock fragments up to 18" size			
2								
4					REFUSAL AT 4 FEET Groundwater not encountered			

Figure A-19,
Log of Trench T 19, Page 1 of 1

G2213-32-01.GPJ

SAMPLE SYMBOLS			
	... SAMPLING UNSUCCESSFUL	... STANDARD PENETRATION TEST	... DRIVE SAMPLE (UNDISTURBED)
	... DISTURBED OR BAG SAMPLE	... CHUNK SAMPLE	... WATER TABLE OR SEEPAGE

GEOCON

TRENCH T 20

ELEV. (MSL.) 793' DATE COMPLETED 05-22-2018

EQUIPMENT JD555 BACKHOE (W/24" BUCKET) BY: J. PAGNILLO

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0				ML	**TOPSOIL** Soft, damp, reddish brown, Sandy SILT; abundant roots			
2								
					SANTIAGO PEAK VOLCANICS (Jsp) Completely weathered, olive green, weak, METAVOLCANIC ROCK (Saprolite); excavates to Sandy CLAY			
4								
					-Becomes highly weathered, brownish gray, weak, METAVOLCANIC ROCK; excavates to Clayey/Sandy GRAVEL with moderately weak to moderately strong rock fragments; intensely jointed			
6								
	T20-1							
					TRENCH TERMINATED AT 7 FEET Groundwater not encountered			

Figure A-20,
Log of Trench T 20, Page 1 of 1

G2213-32-01.GPJ

SAMPLE SYMBOLS			
	... SAMPLING UNSUCCESSFUL	... STANDARD PENETRATION TEST	... DRIVE SAMPLE (UNDISTURBED)
	... DISTURBED OR BAG SAMPLE	... CHUNK SAMPLE	... WATER TABLE OR SEEPAGE

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	TRENCH T 21 ELEV. (MSL.) 800' DATE COMPLETED 05-22-2018 EQUIPMENT JD555 BACKHOE (W/24" BUCKET) BY: J. PAGNILLO	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					MATERIAL DESCRIPTION			
0				ML	**TOPSOIL** Soft, damp, reddish brown, Sandy SILT; abundant roots			
2					**SANTIAGO PEAK VOLCANICS (Jsp)** Moderately weathered, brownish gray, moderately weak, METAVOLCANIC ROCK -Becomes slightly weathered and strong; excavates to Sandy GRAVEL with angular rock fragments up to 15" size			
4					REFUSAL AT 4 FEET Groundwater not encountered			

Figure A-21,
Log of Trench T 21, Page 1 of 1

G2213-32-01.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.

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PROJECT NO. G2213-32-01

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	TRENCH T 22 ELEV. (MSL.) 760' DATE COMPLETED 05-22-2018 EQUIPMENT JD555 BACKHOE (W/24" BUCKET) BY: J. PAGNILLO	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					MATERIAL DESCRIPTION			
0				ML	**TOPSOIL** Soft, damp, reddish brown, Sandy SILT; abundant roots			
2					**SANTIAGO PEAK VOLCANICS (Jsp)** Slightly weathered, bluish gray, very strong, METAVOLCANIC ROCK; moderately jointed; excavates as very strong angular rock fragments up to 15" size			
					REFUSAL AT 3.5 FEET Groundwater not encountered			

Figure A-22,
Log of Trench T 22, Page 1 of 1

G2213-32-01.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

TRENCH T 23

ELEV. (MSL.) **765'** DATE COMPLETED **05-22-2018**

EQUIPMENT **JD555 BACKHOE (W/24" BUCKET)** BY: **J. PAGNILLO**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0				ML	**TOPSOIL** Soft, damp, reddish brown, Sandy SILT; abundant roots			
2					**SANTIAGO PEAK VOLCANICS (Jsp)** Slightly weathered, bluish gray, very strong, METAVOLCANIC ROCK; moderately jointed; excavates to Sandy GRAVEL with very strong angular rock fragments up to 15" size			
					REFUSAL AT 3.5 FEET Groundwater not encountered			

Figure A-23,
Log of Trench T 23, Page 1 of 1

G2213-32-01.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.

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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 soil samples were tested for maximum dry density and optimum moisture content, expansion index, shear strength, soluble sulfate content, and gradation. The results of our laboratory tests are summarized on Tables B-I through B-IV, and Figures B-1 and B-2.

TABLE B-I
SUMMARY OF LABORATORY MAXIMUM DRY DENSITY AND OPTIMUM MOISTURE CONTENT TEST RESULTS

Sample No.	Description	Maximum Dry Density (pcf)	Optimum Moisture Content (% dry wt.)
T4-1	Grayish brown, fine to coarse, Sandy CLAY	121.0	12.8
T8-1	Yellow, Clayey SILT with little gravel	122.8	13.6
T12-1	Dark reddish brown, Clayey SILT with sand and trace gravel	120.2	14.4
T17-1	Dark reddish brown, fine to coarse, Sandy SILT with little clay and gravel	127.5	11.2

TABLE B-II
SUMMARY OF LABORATORY EXPANSION INDEX TEST RESULTS

Sample No.	Moisture Content (%)		Dry Density (pcf)	Expansion Index
	Before Test	After Test		
T4-1	12.7	26.3	99.9	83
T12-1	12.3	22.3	101.9	12
T17-1	10.3	19.5	107.1	9

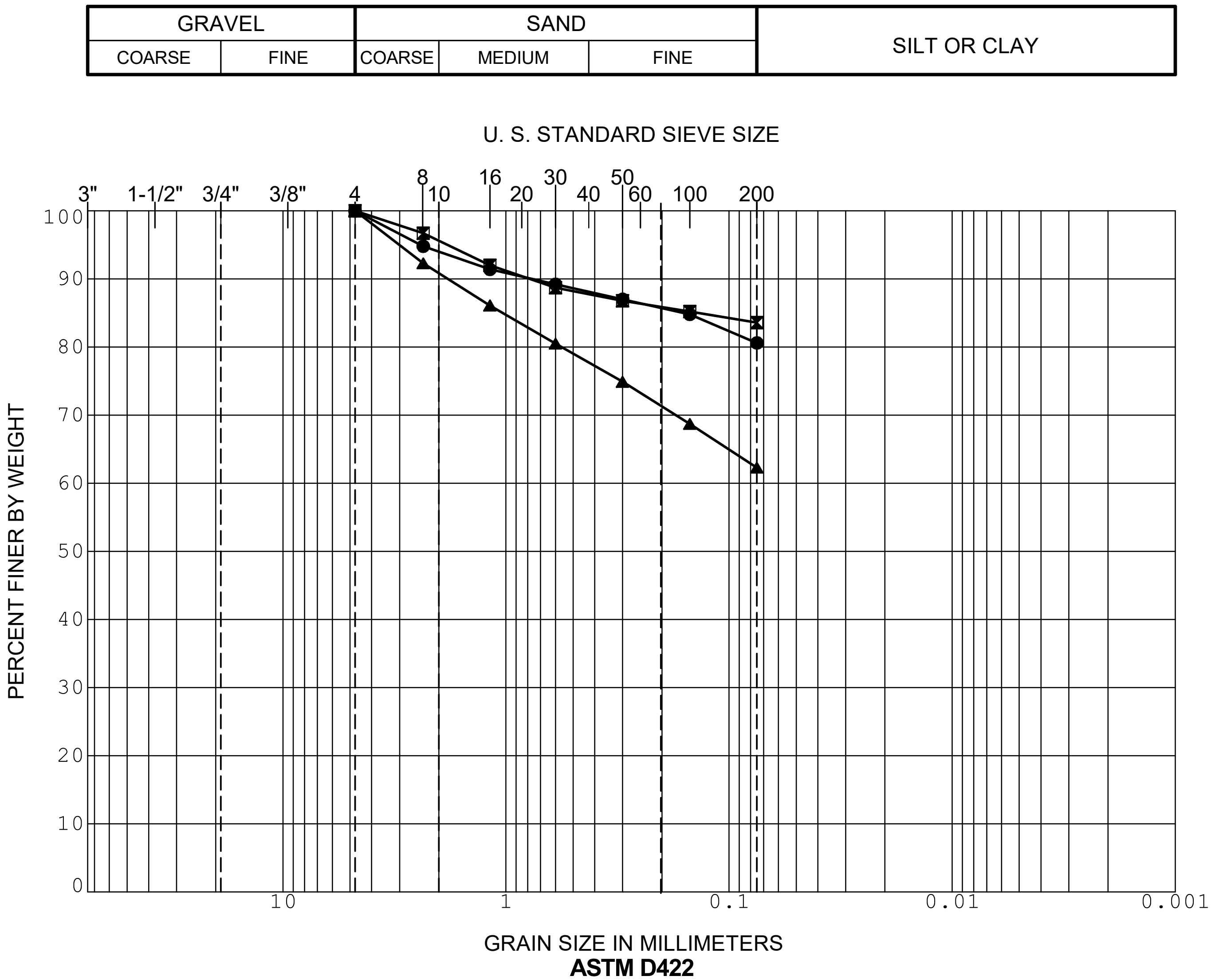
TABLE B-III
SUMMARY OF LABORATORY DIRECT SHEAR TEST RESULTS

Sample No.	Dry Density (pcf)	Moisture Content (%)	Unit Cohesion (psf)	Angle of Shear Resistance (degrees)
T4-1	108.3	13.8	240	29
T8-1	111.1	13.4	550	33
T12-1	108.0	14.8	860	24
T17-1	115.7	10.5	780	25

Samples were remolded to approximately 90 percent of maximum dry density at near optimum moisture content.

TABLE B-IV
SUMMARY OF LABORATORY WATER-SOLUBLE SULFATE TEST RESULTS

Sample No.	Water-Soluble Sulfate Content (%)	Exposure
T8-1	0.0030	Not Applicable
T17-1	0.0004	Not Applicable



	SAMPLE	DEPTH (ft)	CLASSIFICATION	NAT WC	LL	PL	PI
●	T4-1	2.0					
⊠	T8-1	3.5					
▲	T12-1	2.0					

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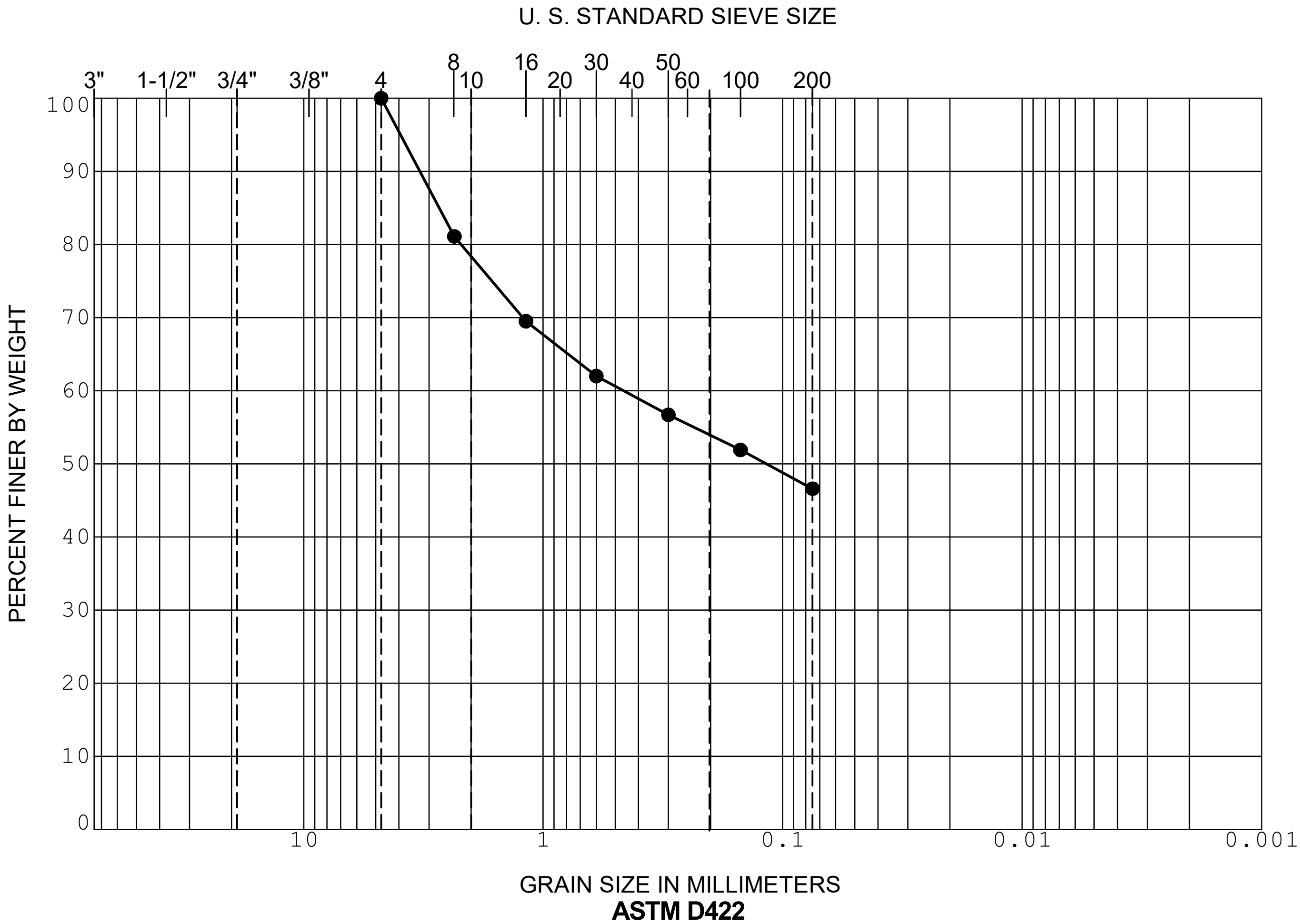
SAN DIEGO, CALIFORNIA

PROJECT NO. G2213-32-01

GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	



	SAMPLE	DEPTH (ft)	CLASSIFICATION	NAT WC	LL	PL	PI
●	T17-1	4.0					
⊠							
▲							

GRADATION CURVE

AVION

SAN DIEGO, CALIFORNIA

GEOCON

APPENDIX C

APPENDIX C

SLOPE STABILITY ANALYSES

FOR

AVION
SAN DIEGO, CALIFORNIA

PROJECT NO. G2213-32-01

Avion
Project No. G2213-32-01
Section A-A'
Name: A-A' Case 0 Static.gsz
Date: 08/20/2018 Time: 11:00:29 AM

Proposed Condition

Static Analysis

Color	Name	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
	Jsp - Santiago Peak Volcanics	150	570,000	45

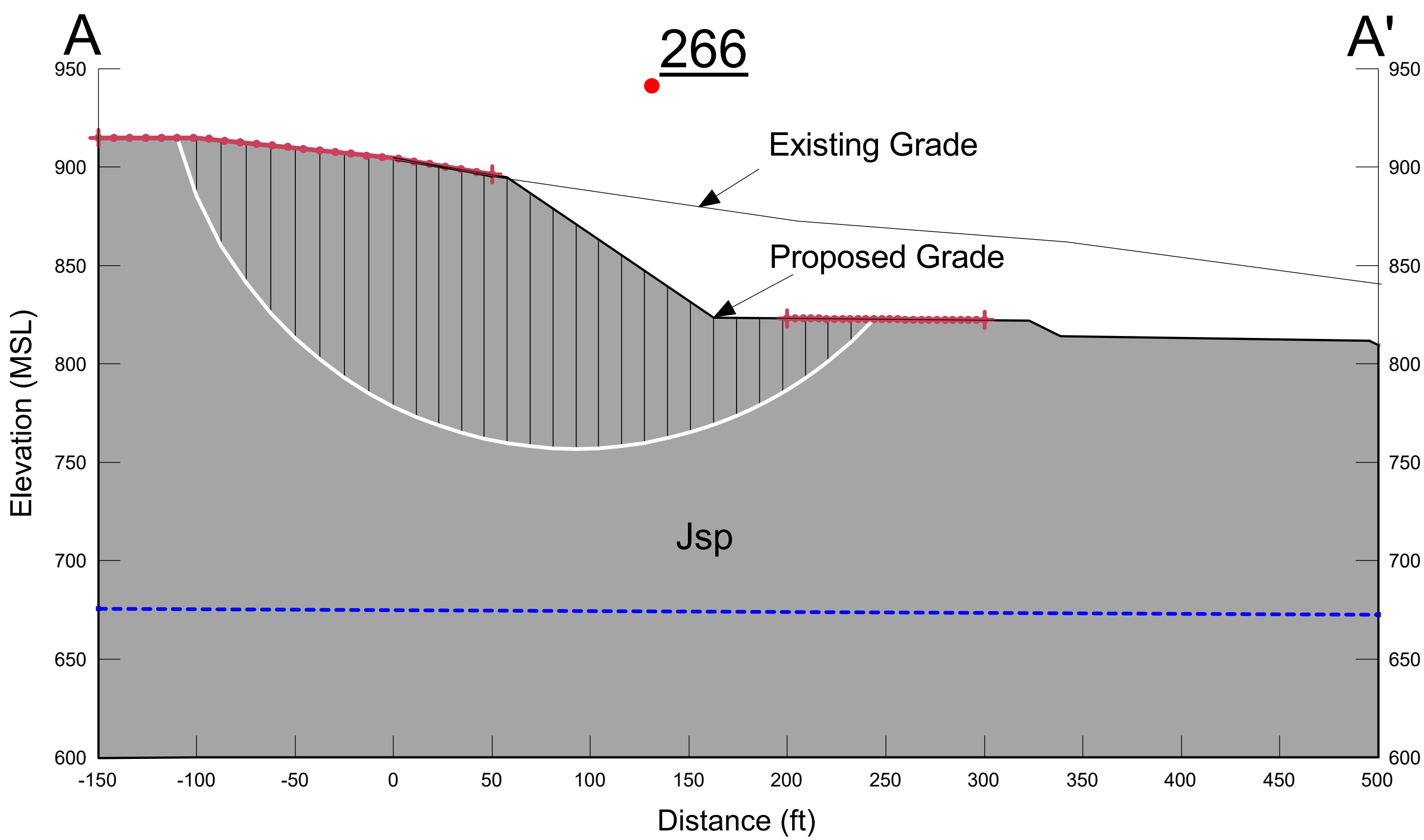


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Figure C-1

Avion
Project No. G2213-32-01
Section A-A'
Name: A-A' Case 0 Seismic.gsz
Date: 08/21/2018 Time: 09:15:05 AM

Proposed Condition

Seismic Analysis
keq=0.11g

Color	Name	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
	Jsp - Santiago Peak Volcanics	150	570,000	45

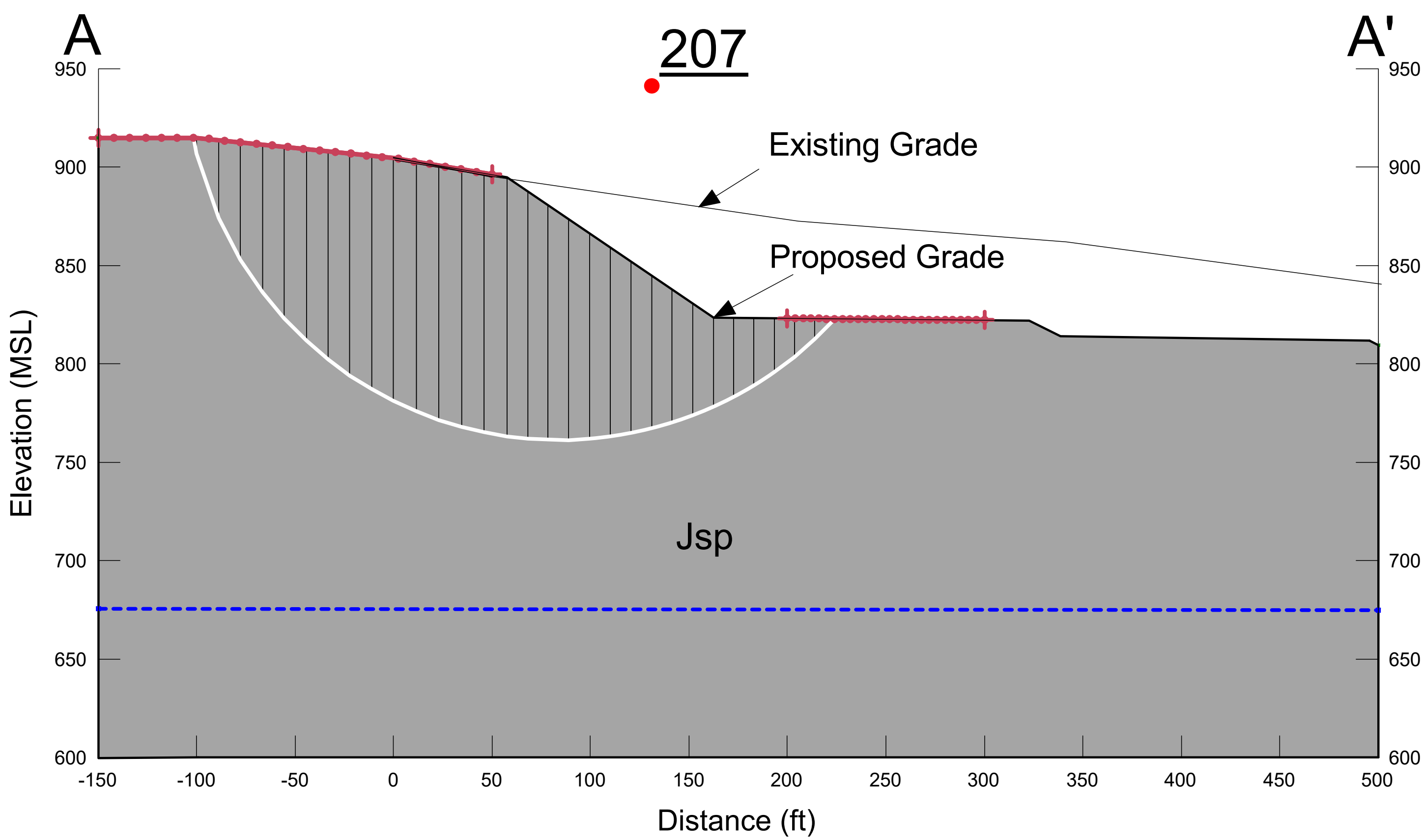

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Figure C-2

GEOCON

Seismic Slope Stability Evaluation

Input Data in Shaded Areas

Project	Avion	Computed By	TEM
Project Number	G2213-32-01		
Date	08/22/18		
Filename	AA-Case 0 seismic		

Peak Ground Acceleration (Firm Rock), MHA_r, g	0.19	10% in 50 years
Modal Magnitude, M	6.9	
Modal Distance, r, km	17.2	
Site Condition, S (0 for rock, 1 for soil)	0	
Yield Acceleration, k_y/g	NA	<-- Enter Value or NA for Screening Analysis
Shear Wave Velocity, V_s (ft/sec)	NA	<--
Max Vertical Distance, H (Feet)	NA	<--
Is Slide X-Area > 25,000ft^2 (Y/N)	N	<-- Use "N" for Buttress Fills
Correction for horizontal incoherence	1.0	
Duration, $D_{5\text{-}95}\|_{med}$, sec	13.850	
Coefficient, C_1	0.4110	
Coefficient, C_2	0.0837	
Coefficient, C_3	0.0021	
Standard Error, ε_T	0.437	
Mean Square Period, T_m, sec	0.522	

Initial Screening with MHEA = MHA = k_{max}g		Approximation of Seismic Demand	
k_y/MHA	NA	Period of Sliding Mass, $T_s = 4H/V_s$, sec	NA
f_{EQ}(u=5cm) = (NRF/3.477)*(1.87-log(u/((MHA_r/g)*NRF*$D_{5\text{-}95}$)))	0.5900	T_s/T_m	NA
k_{EQ} = feq(MHA_r)/g	0.112	MHEA/(MHA*NRF)	NA
Factor of Safety in Slope Analysis Using k_{EQ}	186.00	NRF = 0.6225+0.9196EXP(-2.25*MHA_r/g)	1.22
Passes Initial Screening Analysis		MHEA/g	NA
		$k_y/MHEA = k_y/k_{max}$	NA
		Normalized Displacement, Normu	NA
		Estimated Displacement, u (cm)	**NA**

FIGURE C-3

Avion
Project No. G2213-32-01
Section B-B'
Name: B-B' Case 0 Static.gsz
Date: 08/21/2018 Time: 09:39:17 AM

Proposed Condition

Static Analysis

Color	Name	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
	Jsp - Santiago Peak Volcanics	150	570,000	45
	Qal - Alluvium	120	200	28
	Qcf - Compacted Fill	125	350	32
	Qcol - Colluvium	120	200	28
	Qmse - MSE Wall Backfill	125	0	32

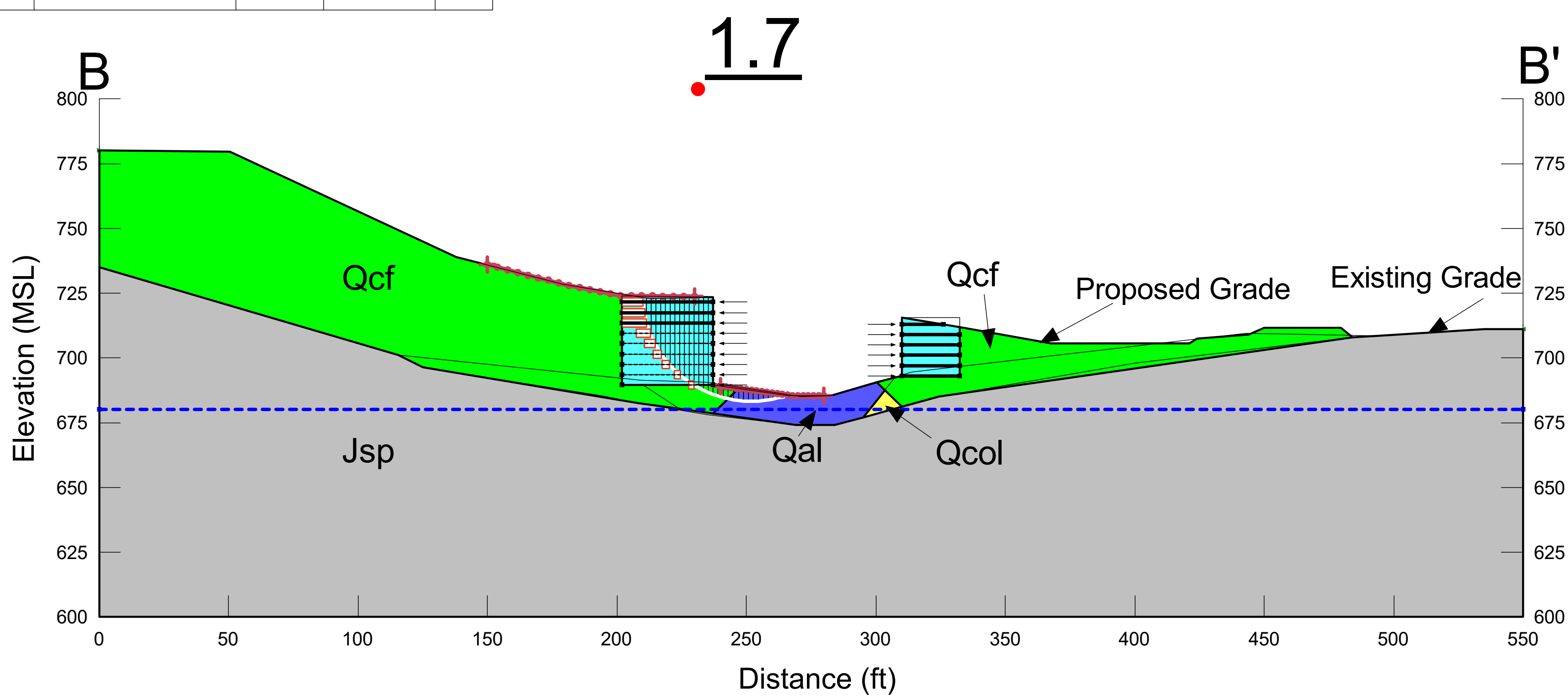

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Figure C-4

Avion
Project No. G2213-32-01
Section B-B'
Name: B-B' Case 0 Seismic.gsz
Date: 08/21/2018 Time: 09:45:20 AM

Proposed Condition

Seismic Analysis
keq = 0.11g

Color	Name	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
	Jsp - Santiago Peak Volcanics	150	570,000	45
	Qal - Alluvium	120	200	28
	Qcf - Compacted Fill	125	350	32
	Qcol - Colluvium	120	200	28
	Qmse - MSE Wall Backfill	125	0	32

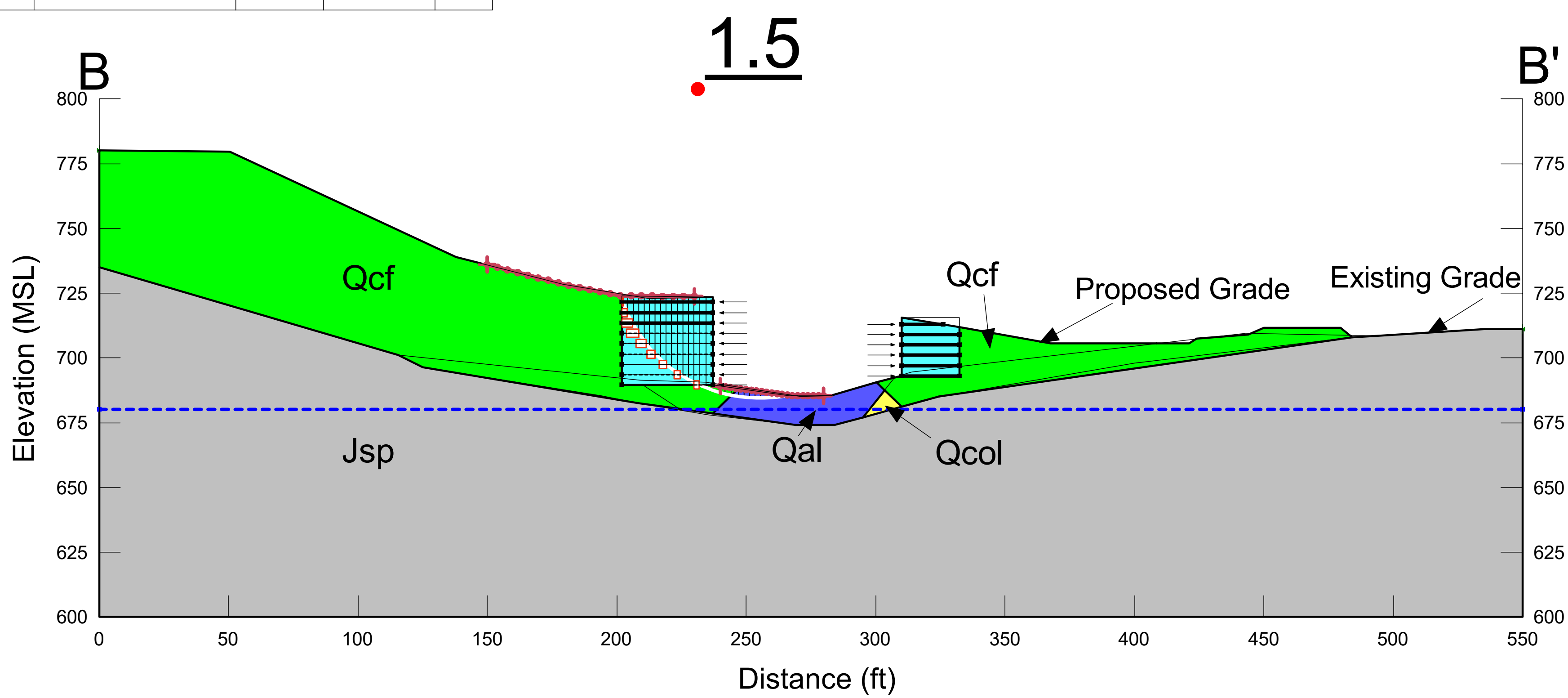

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Figure C-5

GEOCON

Seismic Slope Stability Evaluation

Input Data in Shaded Areas

Project	Avion	Computed By	TEM
Project Number	G2213-32-01		
Date	08/22/18		
Filename	BB-Case 0 seismic		

Peak Ground Acceleration (Firm Rock), MHA_r, g	0.19	10% in 50 years
Modal Magnitude, M	6.9	
Modal Distance, r, km	17.2	
Site Condition, S (0 for rock, 1 for soil)	0	
Yield Acceleration, k_y/g	NA	<-- Enter Value or NA for Screening Analysis
Shear Wave Velocity, V_s (ft/sec)	NA	<--
Max Vertical Distance, H (Feet)	NA	<--
Is Slide X-Area > 25,000ft^2 (Y/N)	N	<-- Use "N" for Buttress Fills
Correction for horizontal incoherence	1.0	
Duration, $D_{5\text{-}95}\|_{med}$, sec	13.850	
Coefficient, C_1	0.4110	
Coefficient, C_2	0.0837	
Coefficient, C_3	0.0021	
Standard Error, ε_T	0.437	
Mean Square Period, T_m, sec	0.522	

Initial Screening with MHEA = MHA = k_{max}g	
k_y/MHA	NA
f_{EQ}(u=5cm) = (NRF/3.477)*(1.87-log(u/((MHA_r/g)*NRF*$D_{5\text{-}95}$)))	0.5900
k_{EQ} = feq(MHA_r)/g	0.112
Factor of Safety in Slope Analysis Using k_{EQ}	1.50

Passes Initial Screening Analysis

Approximation of Seismic Demand	
Period of Sliding Mass, T_s = 4H/V_s, sec	NA
T_s/T_m	NA
MHEA/(MHA*NRF)	NA
NRF = 0.6225+0.9196EXP(-2.25*MHA_r/g)	1.22
MHEA/g	NA
k_y/MHEA = k_y/k_{max}	NA
Normalized Displacement, Normu	NA
Estimated Displacement, u (cm)	**NA**

FIGURE C-6

Avion
Project No. G2213-32-01
Section B-B'
Name: B-B' Case 1 Static.gsz
Date: 08/21/2018 Time: 11:32:11 AM

Proposed Condition

Static Analysis

Color	Name	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
	Jsp - Santiago Peak Volcanics	150	570,000	45
	Qal - Alluvium	120	200	28
	Qcf - Compacted Fill	125	350	32
	Qcol - Colluvium	120	200	28
	Qmse - MSE Wall Backfill	125	0	32

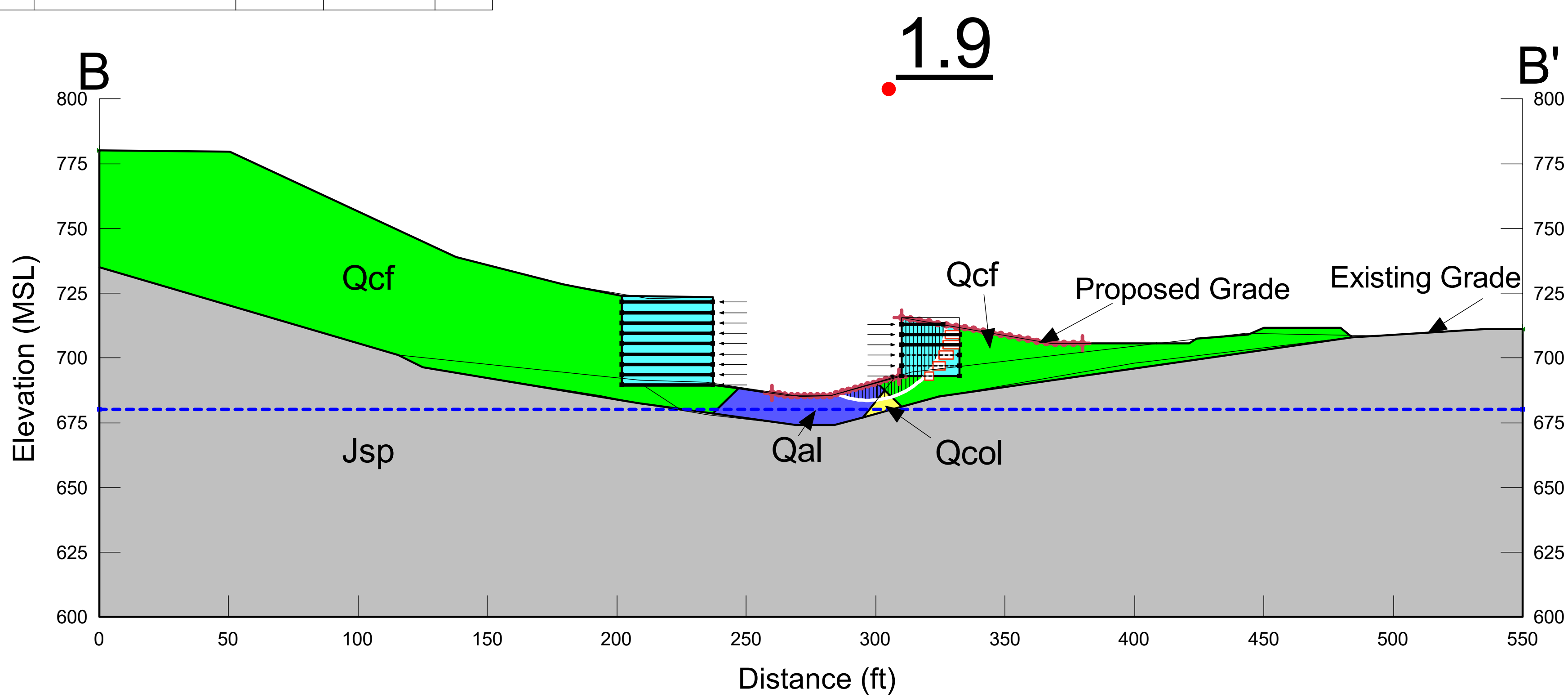

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Figure C-7

Avion
Project No. G2213-32-01
Section B-B'
Name: B-B' Case 1 Seismic.gsz
Date: 08/21/2018 Time: 11:33:23 AM

Proposed Condition

Seismic Analysis
keq = 0.11g

Color	Name	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
	Jsp - Santiago Peak Volcanics	150	570,000	45
	Qal - Alluvium	120	200	28
	Qcf - Compacted Fill	125	350	32
	Qcol - Colluvium	120	200	28
	Qmse - MSE Wall Backfill	125	0	32

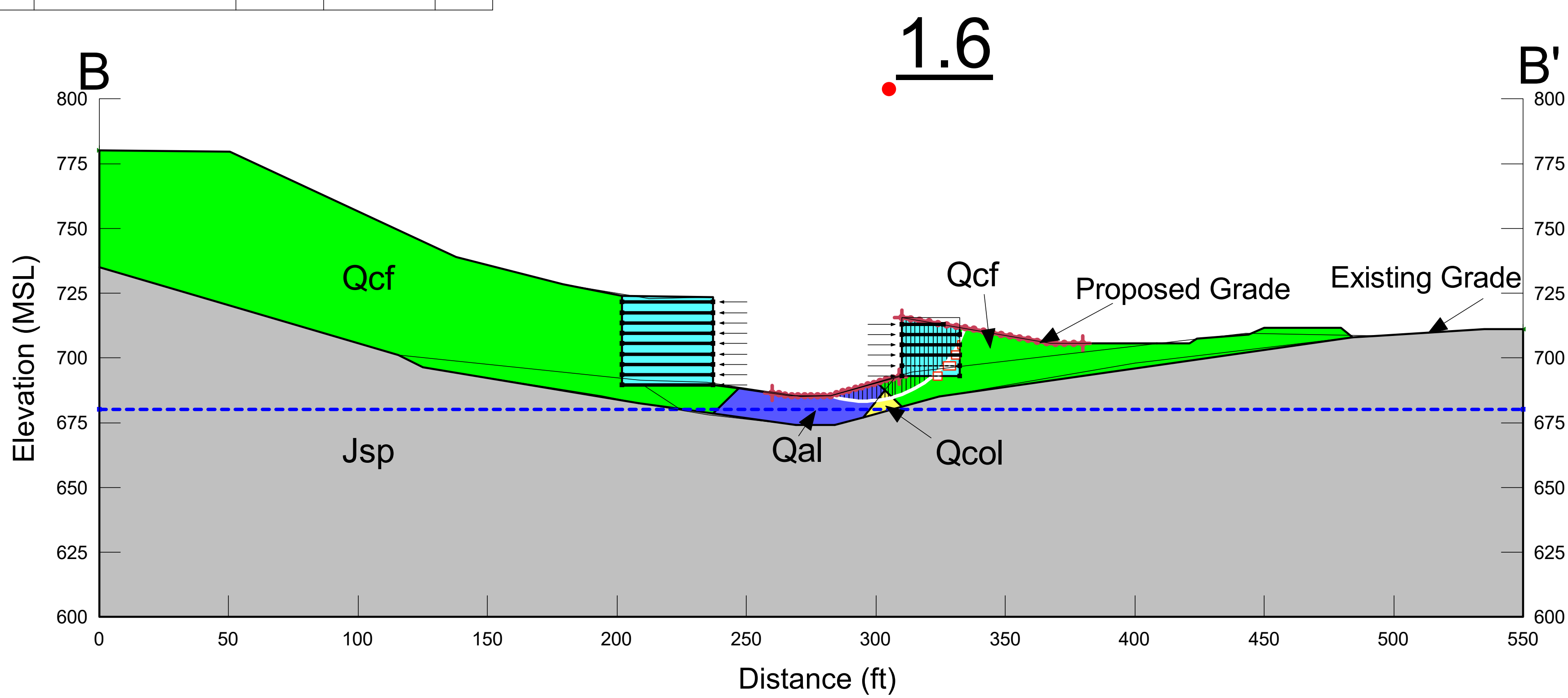

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Figure C-8

GEOCON

Seismic Slope Stability Evaluation

Input Data in Shaded Areas

Project	Avion	Computed By	TEM
Project Number	G2213-32-01		
Date	08/22/18		
Filename	BB-Case 1 seismic		

Peak Ground Acceleration (Firm Rock), MHA_r, g	0.19	10% in 50 years
Modal Magnitude, M	6.9	
Modal Distance, r, km	17.2	
Site Condition, S (0 for rock, 1 for soil)	0	
Yield Acceleration, k_y/g	NA	<-- Enter Value or NA for Screening Analysis
Shear Wave Velocity, V_s (ft/sec)	NA	<--
Max Vertical Distance, H (Feet)	NA	<--
Is Slide X-Area > 25,000ft^2 (Y/N)	N	<-- Use "N" for Buttress Fills
Correction for horizontal incoherence	1.0	
Duration, $D_{5\text{-}95}\|_{med}$, sec	13.850	
Coefficient, C_1	0.4110	
Coefficient, C_2	0.0837	
Coefficient, C_3	0.0021	
Standard Error, ε_T	0.437	
Mean Square Period, T_m, sec	0.522	

Initial Screening with MHEA = MHA = $k_{max}g$		Approximation of Seismic Demand	
k_y/MHA	NA	Period of Sliding Mass, $T_s = 4H/V_s$, sec	NA
f_{EQ}(u=5cm) = (NRF/3.477)*(1.87-log(u/((MHA_r/g)*NRF*$D_{5\text{-}95}$)))	0.5900	T_s/T_m	NA
k_{EQ} = feq(MHA_r)/g	0.112	MHEA/(MHA*NRF)	NA
Factor of Safety in Slope Analysis Using k_{EQ}	1.60	NRF = 0.6225+0.9196EXP(-2.25*MHA_r/g)	1.22
Passes Initial Screening Analysis		MHEA/g	NA
		k_y/MHEA = k_y/k_{max}	NA
		Normalized Displacement, Normu	NA
		Estimated Displacement, u (cm)	**NA**

FIGURE C-9

Avion
Project No. G2213-32-01
Section C-C'
Name: C-C' Case 0 Static.gsz
Date: 08/21/2018 Time: 11:35:50 AM

Proposed Condition

Static Analysis

Color	Name	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
	Jsp - Santiago Peak Volcanics	150	570,000	45
	Qal - Alluvium	120	200	28
	Qcf - Compacted Fill	125	350	32
	Qcol - Colluvium	120	200	28

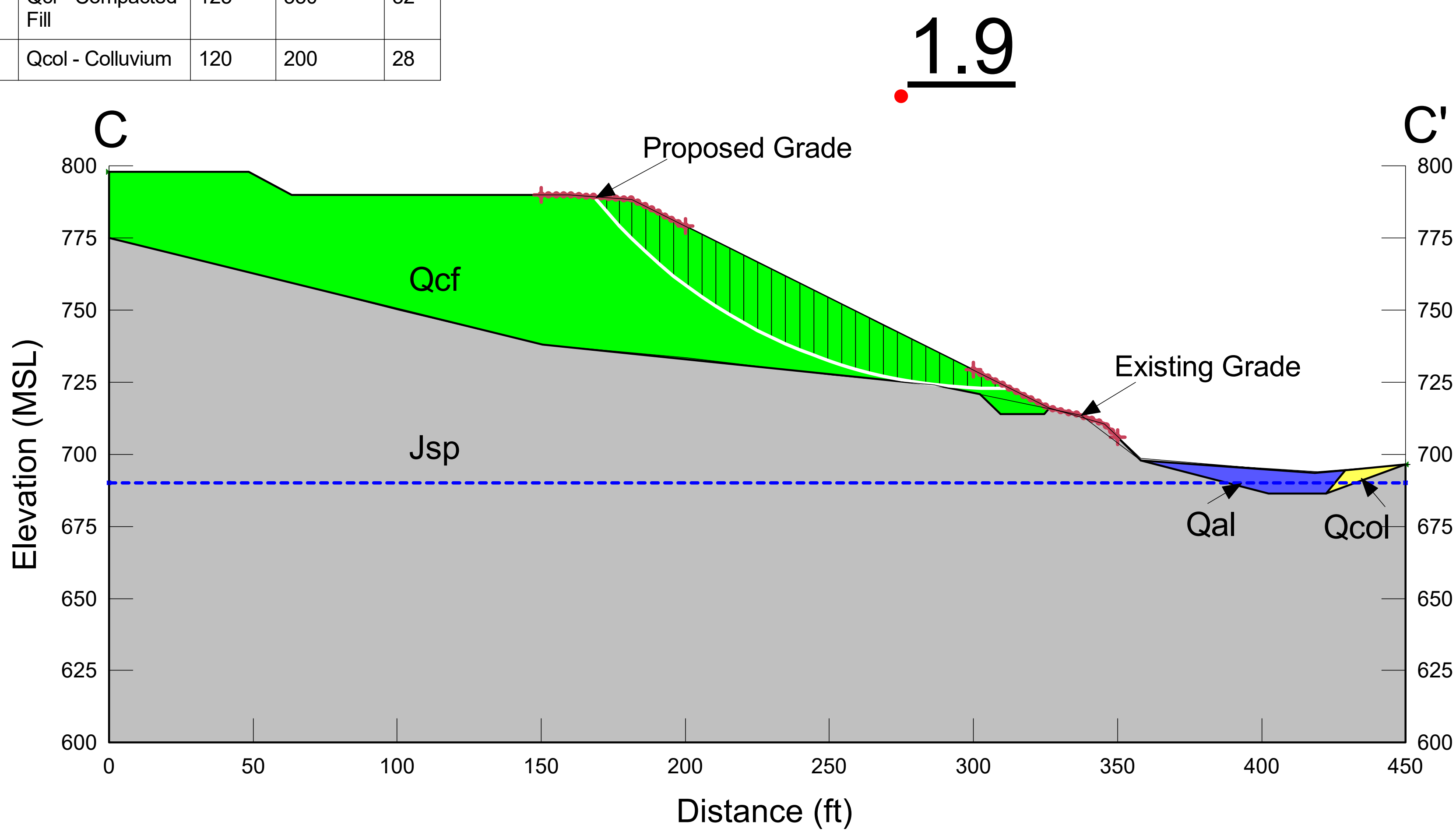

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Figure C-10

Avion
Project No. G2213-32-01
Section C-C'
Name: C-C' Case 0 Seismic.gsz
Date: 08/21/2018 Time: 11:39:03 AM

Proposed Condition

Seismic Analysis
keq = 0.11g

Color	Name	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
	Jsp - Santiago Peak Volcanics	150	570,000	45
	Qal - Alluvium	120	200	28
	Qcf - Compacted Fill	125	350	32
	Qcol - Colluvium	120	200	28

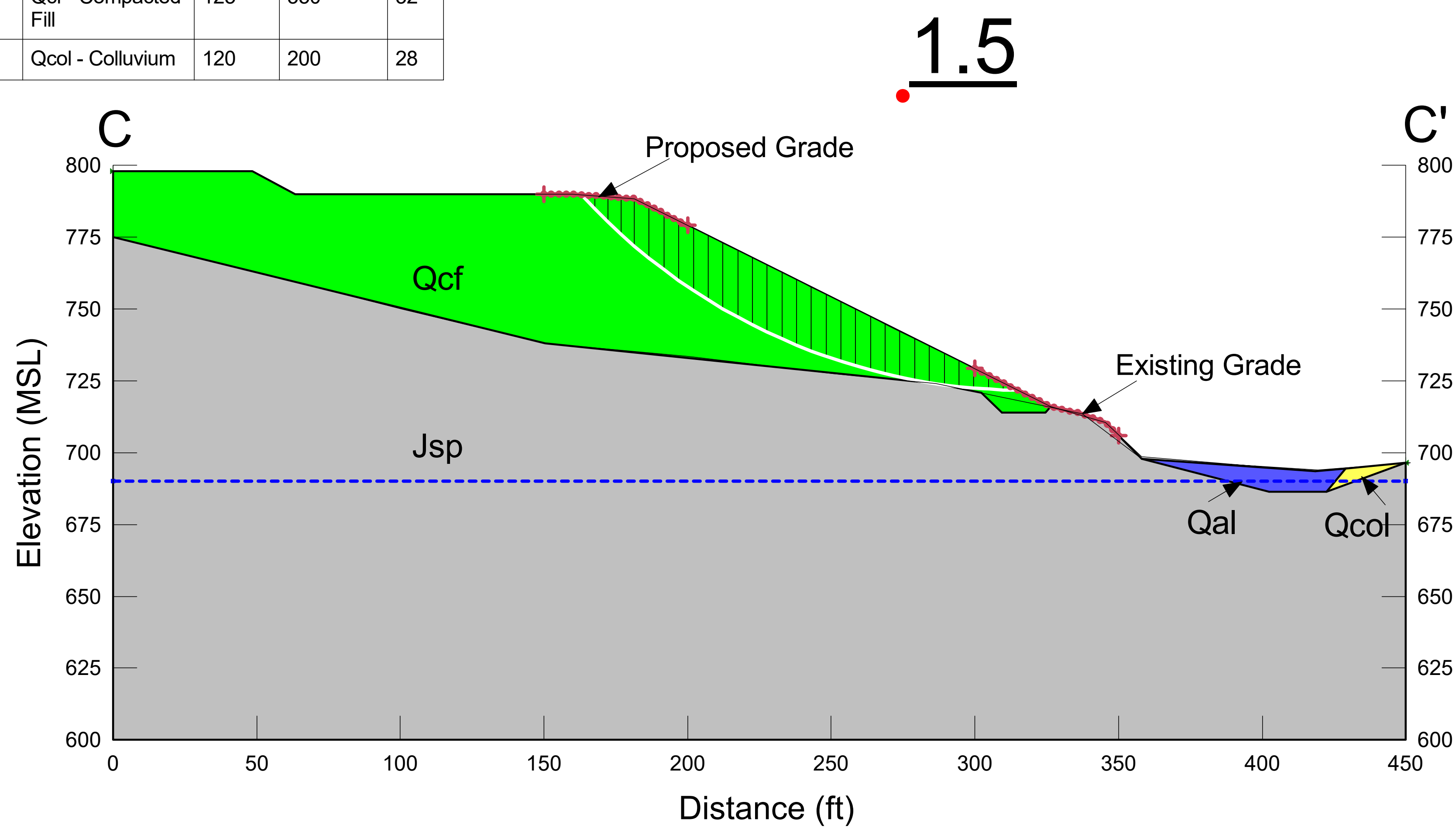

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Figure C-11

GEOCON

Seismic Slope Stability Evaluation

Input Data in Shaded Areas

Project	Avion	Computed By	TEM
Project Number	G2213-32-01		
Date	08/22/18		
Filename	CC-Case 0 seismic		

Peak Ground Acceleration (Firm Rock), MHA_r, g	0.19	10% in 50 years
Modal Magnitude, M	6.9	
Modal Distance, r, km	17.2	
Site Condition, S (0 for rock, 1 for soil)	0	
Yield Acceleration, k_y/g	NA	<-- Enter Value or NA for Screening Analysis
Shear Wave Velocity, V_s (ft/sec)	NA	<--
Max Vertical Distance, H (Feet)	NA	<--
Is Slide X-Area > 25,000ft^2 (Y/N)	N	<-- Use "N" for Buttress Fills
Correction for horizontal incoherence	1.0	
Duration, $D_{5\text{-}95}\|_{med}$, sec	13.850	
Coefficient, C_1	0.4110	
Coefficient, C_2	0.0837	
Coefficient, C_3	0.0021	
Standard Error, ε_T	0.437	
Mean Square Period, T_m, sec	0.522	

Initial Screening with MHEA = MHA = k_{max}g

k_y/MHA	NA
f_{EQ}(u=5cm) = (NRF/3.477)*(1.87-log(u/((MHA_r/g)*NRF*D_{5-95})))	0.5900
k_{EQ} = feq(MHA_r)/g	0.112
Factor of Safety in Slope Analysis Using k_{EQ}	1.50

Passes Initial Screening Analysis

Approximation of Seismic Demand

Period of Sliding Mass, T_s = 4H/V_s, sec	NA
T_s/T_m	NA
MHEA/(MHA*NRF)	NA
NRF = 0.6225+0.9196EXP(-2.25*MHA_r/g)	1.22
MHEA/g	NA
k_y/MHEA = k_y/k_{max}	NA
Normalized Displacement, Normu	NA
Estimated Displacement, u (cm)	**NA**

FIGURE C-12

Avion
Project No. G2213-32-01
Section D-D'
Name: D-D' Case 0 Static.gsz
Date: 08/21/2018 Time: 11:50:55 AM

Color	Name	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
	Jsp - Santiago Peak Volcanics	150	570,000	45
	Qal - Alluvium	120	200	28
	Qcf - Compacted Fill	125	350	32
	Qmse - MSE Wall Backfill	125	0	32

Proposed Condition

Static Analysis

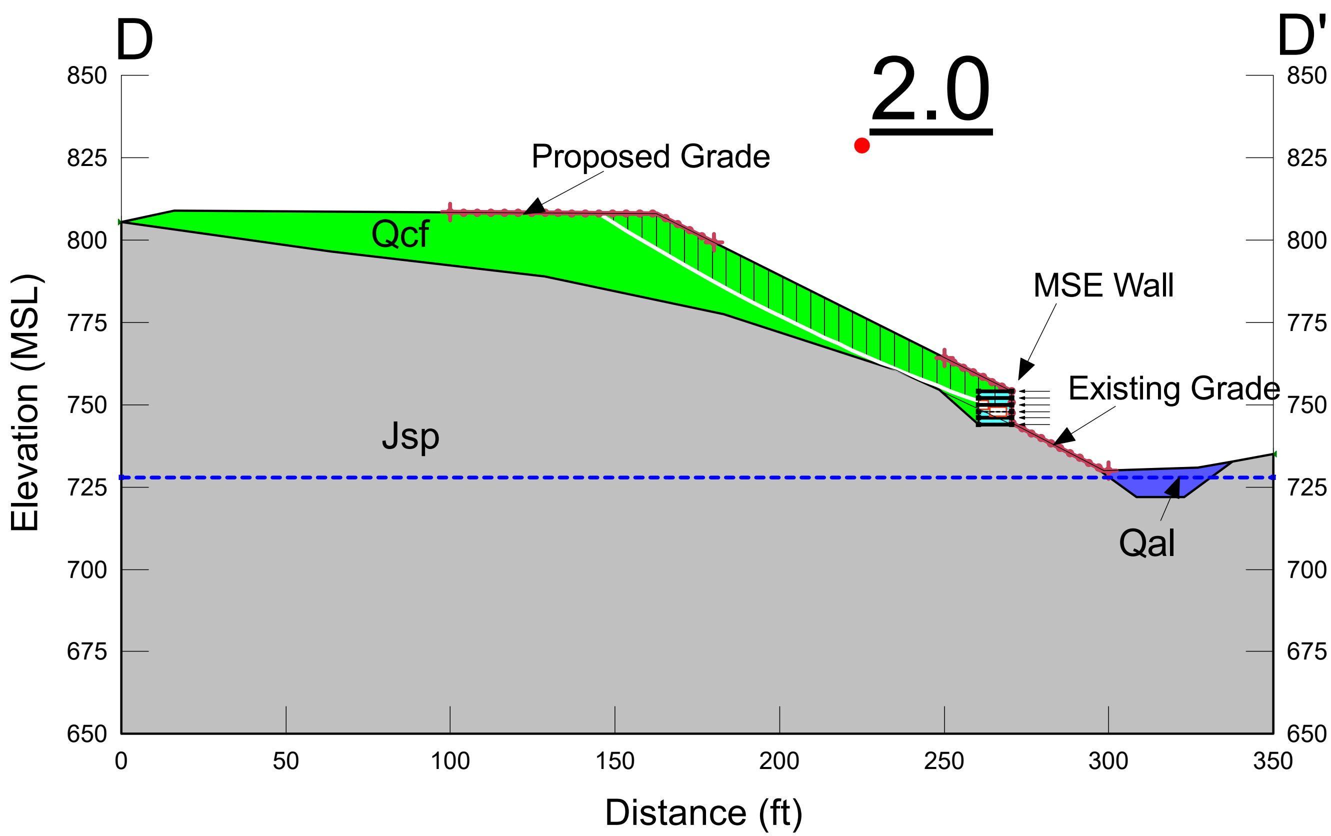

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Figure C-13

Avion
Project No. G2213-32-01
Section D-D'
Name: D-D' Case 0 Seismic.gsz
Date: 08/21/2018 Time: 11:52:41 AM

Proposed Condition

Seismic Analysis
keq = 0.11g

Color	Name	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
	Jsp - Santiago Peak Volcanics	150	570,000	45
	Qal - Alluvium	120	200	28
	Qcf - Compacted Fill	125	350	32
	Qmse - MSE Wall Backfill	125	0	32

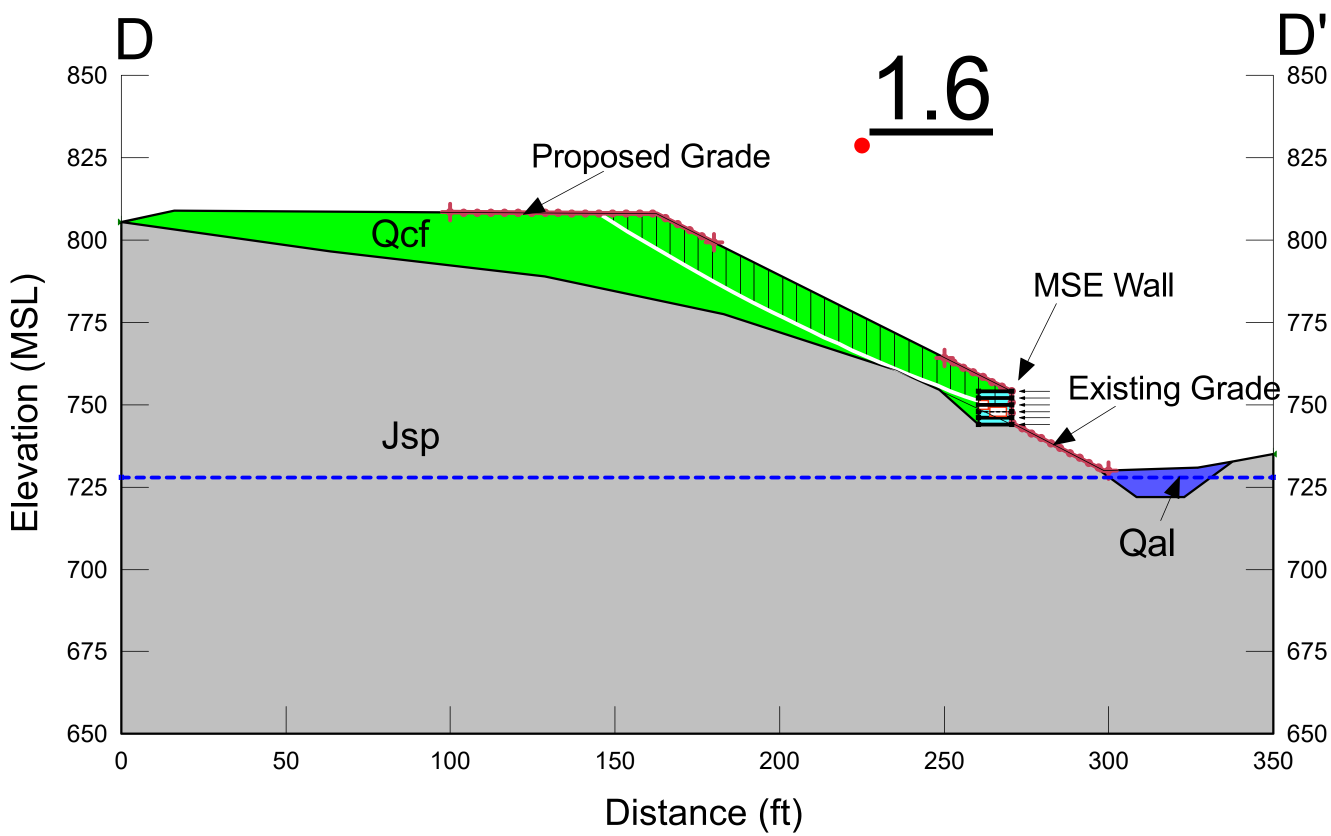

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Figure C-14

Seismic Slope Stability Evaluation

Input Data in Shaded Areas

Project	Avion	Computed By	TEM
Project Number	G2213-32-01		
Date	08/22/18		
Filename	DD-Case 0 seismic		

Peak Ground Acceleration (Firm Rock), MHA_r, g	0.19	10% in 50 years
Modal Magnitude, M	6.9	
Modal Distance, r, km	17.2	
Site Condition, S (0 for rock, 1 for soil)	0	
Yield Acceleration, k_y/g	NA	<-- Enter Value or NA for Screening Analysis
Shear Wave Velocity, V_s (ft/sec)	NA	<--
Max Vertical Distance, H (Feet)	NA	<--
Is Slide X-Area > 25,000ft^2 (Y/N)	N	<-- Use "N" for Buttress Fills
Correction for horizontal incoherence	1.0	
Duration, $D_{5\text{-}95}\|_{med}$, sec	13.850	
Coefficient, C_1	0.4110	
Coefficient, C_2	0.0837	
Coefficient, C_3	0.0021	
Standard Error, ε_T	0.437	
Mean Square Period, T_m, sec	0.522	

Initial Screening with MHEA = MHA = k_{max}g

k_y/MHA	NA
f_{EQ}(u=5cm) = (NRF/3.477)*(1.87-log(u/((MHA_r/g)*NRF*$D_{5\text{-}95}$)))	0.5900
k_{EQ} = feq(MHA_r)/g	0.112
Factor of Safety in Slope Analysis Using k_{EQ}	1.60

Passes Initial Screening Analysis

Approximation of Seismic Demand

Period of Sliding Mass, T_s = 4H/V_s, sec	NA
T_s/T_m	NA
MHEA/(MHA*NRF)	NA
NRF = 0.6225+0.9196EXP(-2.25*MHA_r/g)	1.22
MHEA/g	NA
k_y/MHEA = k_y/k_{max}	NA
Normalized Displacement, Normu	NA

Estimated Displacement, u (cm) NA

FIGURE C-15

APPENDIX D

APPENDIX D

STORM WATER MANAGEMENT INVESTIGATION

FOR

AVION
SAN DIEGO, CALIFORNIA

PROJECT NO. G2213-32-01

APPENDIX D

STORM WATER MANAGEMENT INVESTIGATION

We understand storm water management devices are being proposed in accordance with the *2017 City of San Diego Storm Water Standards for Permanent Site Design, Storm Water Treatment and Hydromodification Management,* commonly referred to as the Storm Water Standards (SWS). If not properly constructed, there is a potential for distress to improvements and properties located hydrologically down gradient or adjacent to these devices. Factors such as the amount of water to be detained, its residence time, and soil permeability have an important effect on seepage transmission and the potential adverse impacts that may occur if the storm water management features are not properly designed and constructed. We have not performed a hydrogeological study at the site. If infiltration of storm water runoff occurs, downstream properties may be subjected to seeps, springs, slope instability, raised groundwater, movement of foundations and slabs, or other undesirable impacts as a result of water infiltration.

Hydrologic Soil Group

The United States Department of Agriculture (USDA), Natural Resources Conservation Services, possesses general information regarding the existing soil conditions for areas within the United States. The USDA website also provides the Hydrologic Soil Group. Table D-1 presents the descriptions of the hydrologic soil groups. In addition, the USDA website also provides an estimated saturated hydraulic conductivity for the existing soil.

TABLE D-1
HYDROLOGIC SOIL GROUP DEFINITIONS

Soil Group	Soil Group Definition
A	Soils having a high infiltration rate (low runoff potential) when thoroughly wet. These consist mainly of deep, well drained to excessively drained sands or gravelly sands. These soils have a high rate of water transmission.
B	Soils having a moderate infiltration rate when thoroughly wet. These consist chiefly of moderately deep or deep, moderately well drained or well drained soils that have moderately fine texture to moderately coarse texture. These soils have a moderate rate of water transmission.
C	Soils having a slow infiltration rate when thoroughly wet. These consist chiefly of soils having a layer that impedes the downward movement of water or soils of moderately fine texture or fine texture. These soils have a slow rate of water transmission.
D	Soils having a very slow infiltration rate (high runoff potential) when thoroughly wet. These consist chiefly of clays that have a high shrink-swell potential, soils that have a high water table, soils that have a claypan or clay layer at or near the surface, and soils that are shallow over nearly impervious material. These soils have a very slow rate of water transmission.

The proposed storm water BMP's will be generally underlain by metavolcanic rock. The USDA Natural Resources Conservation Services (NRCS) Web Soil Survey indicates the property is underlain with one surficial unit identified as San Miguel-Exchequer rocky silt loams (SnG). This unit is classified as Soil Group D. Table D-2 presents the information from the USDA NRCS website.

TABLE D-2
USDA WEB SOIL SURVEY – HYDROLOGIC SOIL GROUP

Map Unit Name	Map Unit Symbol	Approximate Percentage of Property	Hydrologic Soil Group	k_{SAT} of Most Limiting Layer (inches/hour)
San Miguel-Exchequer rocky silt loam	SnG	100	D	0.00 – 0.06

In-Situ Testing

The infiltration rate, percolation rates and saturated hydraulic conductivity are different and have different meanings. Percolation rates tend to overestimate infiltration rates and saturated hydraulic conductivities by a factor of 10 or more. Table D-3 describes the differences in the definitions.

TABLE D-3
SOIL PERMEABILITY DEFINITIONS

Term	Definition
Infiltration Rate	The observation of the flow of water through a material into the ground downward into a given soil structure under long term conditions. This is a function of layering of soil, density, pore space, discontinuities and initial moisture content.
Percolation Rate	The observation of the flow of water through a material into the ground downward and laterally into a given soil structure under long term conditions. This is a function of layering of soil, density, pore space, discontinuities and initial moisture content.
Saturated Hydraulic Conductivity (k_{SAT}, Permeability)	The volume of water that will move in a porous medium under a hydraulic gradient through a unit area. This is a function of density, structure, stratification, fines content and discontinuities. It is also a function of the properties of the liquid as well as of the porous medium.

The degree of soil compaction or in-situ density has a significant impact on soil permeability and infiltration. Based on our experience and other studies we performed, an increase in compaction results in a decrease in soil permeability.

We performed 2 constant-head Aardvark Permeameter Tests, P-1 and P-2, at locations shown on the *Geologic Map*, Figure 2. The test borings were approximately 3 inches in diameter. The results of the tests provide parameters for the saturated hydraulic conductivity characteristics of on-site soil and geologic units. Table D-4 presents the results of the estimated field saturated hydraulic conductivity and estimated infiltration rates obtained from the Aardvark Permeameter tests. The field sheets are presented herein. We applied a feasibility factor of safety of 2 to the field results for use in preparation of Worksheet C.4-1. Based on a discussion in the County of Riverside *Design Handbook for Low Impact Development Best Management Practices*, the infiltration rate should be considered equal to the saturated hydraulic conductivity rate.

TABLE D-4
FIELD PERMEAMETER INFILTRATION TEST RESULTS

Test No.	Geologic Unit	Test Depth (feet)	Field-Saturated Hydraulic Conductivity, k_{sat} (inch/hour)	Worksheet[1] Saturated Hydraulic Conductivity, k_{sat} (inch/hour)
P-1	Jsp	1	0.017	0.0085
P-2	Jsp	1.5	0.008	0.0040

[1] Using a factor of safety of 2 for Worksheet C.4-1.

STORM WATER MANAGEMENT CONCLUSIONS

The *Geologic Map*, Figure 2, depicts the existing property, proposed development, and the locations of the field excavations and in-situ infiltration test locations.

Soil Types

Santiago Peak Volcanics – The Santiago Peak Volcanics Formation underlies the property. This formation consists of weakly metamorphosed volcanic and sedimentary rocks that appear relatively dark-colored where exposed. The metavolcanic rock constitution ranges from rhyolite to basalt and commonly includes tuff, tuff-breccias, and andesites. Very fine-grained, silicified sandstones, slate, and other types of metasedimentary rocks can also be present. The permeability characteristics of this metavolcanic unit are very low. Full and partial infiltration should be considered infeasible.

Infiltration Rates

The results of the factored infiltration rates for the Santiago Peak Volcanics ranged between 0.004 and 0.0085 inches per hour. Therefore, based on the results of the infiltration testing, full and partial infiltration should be considered infeasible.

Groundwater Elevations

We did not encounter groundwater during our field exploration. Groundwater is not expected to be a geotechnical constraint. We expect to encounter groundwater greater than 50 feet below the ground surface.

Soil or Groundwater Contamination

Based on review of the Geotracker website, soil or groundwater contamination is not expected.

New or Existing Utilities

No existing utilities are currently present. Proposed utilities are planned. Full or partial infiltration near existing or proposed utilities should be avoided to prevent lateral water migration into the permeable trench backfill materials.

Existing and Planned Structures

No existing structures are present. Proposed residential structures are not planned in the vicinity of the storm water basin, however a bridge will be constructed immediately down gradient.

Slopes

Topographically, the site is characterized by a north-trending ridge with moderate to steep slopes along the eastern flank. The ridge is comprised of metavolcanic rock and descends in a south to north direction. Drainage for the property generally flows to the east and north and is collected by a northwesterly trending canyon. The elevations within the proposed development consist of a topographic high of 890 feet Mean Sea Level (MSL) located in the northeast portion of the site and a low of approximately 680 feet MSL within the northern portion of the property.

Recommendations

Based on the above discussion, full and partial infiltration is infeasible and liners and subdrains should be incorporated into the design and construction of any planned storm water devices. The liners should be impermeable (e.g. High-density polyethylene, HDPE, with a thickness of about 30 mil or equivalent Polyvinyl Chloride, PVC) to prevent water migration. The subdrains should be perforated within the liner area, installed at the base and above the liner, be at least 4 inches in diameter and consist of Schedule 40 PVC pipe. The subdrains outside of the liner should consist of solid pipe. Seams and penetrations of the liners should be properly waterproofed. The subdrains should be connected to a proper outlet. The devices should also be installed in accordance with the manufacturer’s recommendations.

Storm Water Standard Worksheets

The SWS requests the geotechnical engineer complete the *Categorization of Infiltration Feasibility Condition* (Worksheet C.4-1 or I-8) worksheet information to help evaluate the potential for infiltration on the property. The attached Worksheet C.4-1 presents the completed information for the submittal process.

The regional storm water standards also have a worksheet (Worksheet C.5-1 or Form I-9) that helps the project civil engineer estimate the factor of safety based on several factors. Table D-5 describes the suitability assessment input parameters related to the geotechnical engineering aspects for the factor of safety determination.

TABLE D-5
SUITABILITY ASSESSMENT RELATED CONSIDERATIONS
FOR INFILTRATION FACILITY SAFETY FACTORS

Consideration	High Concern – 3 Points	Medium Concern – 2 Points	Low Concern – 1 Point
Assessment Methods	Use of soil survey maps or simple texture analysis to estimate short-term infiltration rates. Use of well permeameter or borehole methods without accompanying continuous boring log. Relatively sparse testing with direct infiltration methods	Use of well permeameter or borehole methods with accompanying continuous boring log. Direct measurement of infiltration area with localized infiltration measurement methods (e.g., Infiltrometer). Moderate spatial resolution	Direct measurement with localized (i.e. small-scale) infiltration testing methods at relatively high resolution or use of extensive test pit infiltration measurement methods.
Predominant Soil Texture	Silty and clayey soils with significant fines	Loamy soils	Granular to slightly loamy soils
Site Soil Variability	Highly variable soils indicated from site assessment or unknown variability	Soil boring/test pits indicate moderately homogenous soils	Soil boring/test pits indicate relatively homogenous soils
Depth to Groundwater/ Impervious Layer	<5 feet below facility bottom	5-15 feet below facility bottom	>15 feet below facility bottom

Based on our geotechnical investigation and the information in Table D-5, Table D-6 presents the estimated factor values for the evaluation of the factor of safety. This table only provides the suitability assessment safety factor (Part A) of the worksheet. The project civil engineer should evaluate the safety factor for design (Part B) and use the combined safety factor for the design infiltration rate.

TABLE D-6
FACTOR OF SAFETY WORKSHEET DESIGN VALUES – PART A[1]

Suitability Assessment Factor Category	Assigned Weight (w)	Factor Value (v)	Product (p = w x v)
Assessment Methods	0.25	2	0.50
Predominant Soil Texture	0.25	3	0.75
Site Soil Variability	0.25	2	0.50
Depth to Groundwater/ Impervious Layer	0.25	1	0.25
Suitability Assessment Safety Factor, $S_A = \sum p$			2.00

[1] The project civil engineer should complete Worksheet D.5-1 or Form I-9 using the data on this table. Additional information is required to evaluate the design factor of safety.

Categorization of Infiltration Feasibility Condition based on Geotechnical Conditions		Worksheet C.4-1: Form I- 8A[10]
Part 1 - Full Infiltration Feasibility Screening Criteria		
DMA(s) Being Analyzed:		**Project Phase:**
Avion		PLANNING
Criteria 1: Infiltration Rate Screening		
1A	Is the mapped hydrologic soil group according to the NRCS Web Soil Survey or UC Davis Soil Web Mapper Type A or B and corroborated by available site soil data[11]? ☐ Yes; the DMA may feasibly support full infiltration. Answer "Yes" to Criteria 1 Result or continue to Step 1B if the applicant elects to perform infiltration testing. ☐ No; the mapped soil types are A or B but is not corroborated by available site soil data (continue to Step 1B). ☒ No; the mapped soil types are C, D, or "urban/unclassified" and is corroborated by available site soil data. Answer "No" to Criteria 1 Result. ☐ No; the mapped soil types are C, D, or "urban/unclassified" but is not corroborated by available site soil data (continue to Step 1B).	
1B	Is the reliable infiltration rate calculated using planning phase methods from Table D.3-1? ☐ Yes; Continue to Step 1C. ☐ No; Skip to Step 1D.	
1C	Is the reliable infiltration rate calculated using planning phase methods from Table D.3-1 greater than 0.5 inches per hour? ☐ Yes; the DMA may feasibly support full infiltration. Answer "Yes" to Criteria 1 Result. ☐ No; full infiltration is not required. Answer "No" to Criteria 1 Result.	
1D	**Infiltration Testing Method.** Is the selected infiltration testing method suitable during the design phase (see Appendix D.3)? Note: Alternative testing standards may be allowed with appropriate rationales and documentation. ☐ Yes; continue to Step 1E. ☐ No; select an appropriate infiltration testing method.	

Note that it is not required to investigate each and every criterion in the worksheet, a single "no" answer in Part 1, Part 2, Part 3, or Part 4 determines a full, partial, or no infiltration condition.

[10] This form must be completed each time there is a change to the site layout that would affect the infiltration feasibility condition. Previously completed forms shall be retained to document the evolution of the site storm water design.

[11] Available data include site-specific sampling or observation of soil types or texture classes, such as obtained from borings or test pits necessary to support other design elements.

Categorization of Infiltration Feasibility Condition based on Geotechnical Conditions		Worksheet C.4-1: Form I- 8A[10]
1E	**Number of Percolation/Infiltration Tests.** Does the infiltration testing method performed satisfy the minimum number of tests specified in Table D.3-2? ☐ Yes; continue to Step 1F. ☐ No; conduct appropriate number of tests.	
IF	**Factor of Safety.** Is the suitable Factor of Safety selected for full infiltration design? See guidance in D.5; Tables D.5-1 and D.5-2; and Worksheet D.5-1 (Form I-9). ☐ Yes; continue to Step 1G. ☐ No; select appropriate factor of safety.	
1G	**Full Infiltration Feasibility.** Is the average measured infiltration rate divided by the Factor of Safety greater than 0.5 inches per hour? ☐ Yes; answer "Yes" to Criteria 1 Result. ☐ No; answer "No" to Criteria 1 Result.	
Criteria 1 Result	Is the estimated reliable infiltration rate greater than 0.5 inches per hour within the DMA where runoff can reasonably be routed to a BMP? ☐ Yes; the DMA may feasibly support full infiltration. Continue to Criteria 2. ☒ No; full infiltration is not required. Skip to Part 1 Result.	

Summarize infiltration testing methods, testing locations, replicates, and results and summarize estimates of reliable infiltration rates according to procedures outlined in D.5. Documentation should be included in project geotechnical report.

Two permeability tests using our constant-head Aardvark permeameter were performed, both within the top foot of metavolcanic rock. The unfactored infiltration rates for the metavolcanic rock was measured to be 0.014 and 0.007 inches/hour (iph). After applying a feasibility factor of safety of 2, the design infiltration rates for the metavolcanic rock are between 0.007 to 0.0035 iph. The Aardvark Permeameter test results are attached. In accordance with the Riverside County storm water procedures, which reference the United States Bureau of Reclamation Well Permeameter Method (USBR 7300), the saturated hydraulic conductivity is equal to the unfactored infiltration rate. The USDA NRCS Web Soil Survey of the proposed area indicated that 100% of the area belongs to Hydrologic Soil Group D (SnG). Based on the above information, full infiltration BMP's supported by the metavolcanic rock are not feasible. Please refer to the geotechnical investigation, Appendix C, for additional information. The locations of the borings and permeability tests are shown on the Geologic Map, Figure 2.

Categorization of Infiltration Feasibility Condition based on Geotechnical Conditions		Worksheet C.4-1: Form I- 8A[10]	
Criteria 2: Geologic/Geotechnical Screening			
2A	If all questions in Step 2A are answered "Yes," continue to Step 2B. For any "No" answer in Step 2A answer "No" to Criteria 2, and submit an "Infiltration Feasibility Condition Letter" that meets the requirements in Appendix C.1.1. The geologic/geotechnical analyses listed in Appendix C.2.1 do not apply to the DMA because one of the following setbacks cannot be avoided and therefore result in the DMA being in a no infiltration condition. The setbacks must be the closest horizontal radial distance from the surface edge (at the overflow elevation) of the BMP.		
2A–1	Can the proposed full infiltration BMP(s) avoid areas with existing fill materials greater than 5 feet thick below the infiltrating surface?	☒ Yes	☐ No
2A-2	Can the proposed full infiltration BMP(s) avoid placement within 10 feet of existing underground utilities, structures, or retaining walls?	☒ Yes	☐ No
2A-3	Can the proposed full infiltration BMP(s) avoid placement within 50 feet of a natural slope (>25%) or within a distance of 1.5H from fill slopes where H is the height of the fill slope?	☐ Yes	☒ No
2B	When full infiltration is determined to be feasible, a geotechnical investigation report must be prepared that considers the relevant factors identified in Appendix C.2.1. If all questions in Step 2B are answered "Yes," then answer "Yes" to Criteria 2 Result. If there are "No" answers continue to Step 2C.		
2B–1	**Hydroconsolidation.** Analyze hydroconsolidation potential per approved ASTM standard due to a proposed full infiltration BMP. Can full infiltration BMPs be proposed within the DMA without increasing hydroconsolidation risks?	☒ Yes	☐ No
2B-2	**Expansive Soils.** Identify expansive soils (soils with an expansion index greater than 20) and the extent of such soils due to proposed full infiltration BMPs. Can full infiltration BMPs be proposed within the DMA without increasing expansive soil risks?	☐ Yes	☒ No

Categorization of Infiltration Feasibility Condition based on Geotechnical Conditions		Worksheet C.4-1: Form I- 8A[10]	
2B-3	**Liquefaction**. If applicable, identify mapped liquefaction areas. Evaluate liquefaction hazards in accordance with Section 6.4.2 of the City of San Diego's Guidelines for Geotechnical Reports (2011 or most recent edition). Liquefaction hazard assessment shall take into account any increase in groundwater elevation or groundwater mounding that could occur as a result of proposed infiltration or percolation facilities. Can full infiltration BMPs be proposed within the DMA without increasing liquefaction risks?	☒ Yes	☐ No
2B-4	**Slope Stability**. If applicable, perform a slope stability analysis in accordance with the ASCE and Southern California Earthquake Center (2002) Recommended Procedures for Implementation of DMG Special Publication 117, Guidelines for Analyzing and Mitigating Landslide Hazards in California to determine minimum slope setbacks for full infiltration BMPs. See the City of San Diego's Guidelines for Geotechnical Reports (2011) to determine which type of slope stability analysis is required. Can full infiltration BMPs be proposed within the DMA without increasing slope stability risks?	☒ Yes	☐ No
2B-5	**Other Geotechnical Hazards.** Identify site-specific geotechnical hazards not already mentioned (refer to Appendix C.2.1). Can full infiltration BMPs be proposed within the DMA without increasing risk of geologic or geotechnical hazards not already mentioned?	☒ Yes	☐ No
2B-6	**Setbacks.** Establish setbacks from underground utilities, structures, and/or retaining walls. Reference applicable ASTM or other recognized standard in the geotechnical report. Can full infiltration BMPs be proposed within the DMA using established setbacks from underground utilities, structures, and/or retaining walls?	☒ Yes	☐ No

Categorization of Infiltration Feasibility Condition based on Geotechnical Conditions		Worksheet C.4-1: Form I- 8A[10]	
2C	**Mitigation Measures.** Propose mitigation measures for each geologic/geotechnical hazard identified in Step 2B. Provide a discussion of geologic/geotechnical hazards that would prevent full infiltration BMPs that cannot be reasonably mitigated in the geotechnical report. See Appendix C.2.1.8 for a list of typically reasonable and typically unreasonable mitigation measures. Can mitigation measures be proposed to allow for full infiltration BMPs? If the question in Step 2 is answered "Yes," then answer "Yes" to Criteria 2 Result. If the question in Step 2C is answered "No," then answer "No" to Criteria 2 Result.	☐ Yes	☒ No
Criteria 2 Result	Can infiltration greater than 0.5 inches per hour be allowed without increasing risk of geologic or geotechnical hazards that cannot be reasonably mitigated to an acceptable level?	☐ Yes	☒ No

Summarize findings and basis; provide references to related reports or exhibits.

Two permeability tests using our constant-head Aardvark permeameter were performed, both within the top foot of metavolcanic rock. The unfactored infiltration rates for the metavolcanic rock was measured to be 0.014 and 0.007 inches/hour (iph). After applying a feasibility factor of safety of 2, the design infiltration rates for the metavolcanic rock are between 0.007 to 0.0035 iph. The Aardvark Permeameter test results are attached. In accordance with the Riverside County storm water procedures, which reference the United States Bureau of Reclamation Well Permeameter Method (USBR 7300), the saturated hydraulic conductivity is equal to the unfactored infiltration rate. The USDA NRCS Web Soil Survey of the proposed area indicated that 100% of the area belongs to Hydrologic Soil Group D (SnG). Based on the above information, full infiltration BMP's supported by the metavolcanic rock are not feasible. Please refer to the geotechnical investigation, Appendix C, for additional information. The locations of the borings and permeability tests are shown on the Geologic Map, Figure 2.

The proposed storm water BMP will be founded in metavolcanic rock. The design infiltration rates do not support a full infiltration condition.

Part 1 Result – Full Infiltration Geotechnical Screening [12]	Result
If answers to both Criteria 1 and Criteria 2 are "Yes", a full infiltration design is potentially feasible based on Geotechnical conditions only. If either answer to Criteria 1 or Criteria 2 is "No", a full infiltration design is not required.	☐ Full infiltration Condition ☒ Complete Part 2

[12] To be completed using gathered site information and best professional judgement considering the definition of MEP in the MS4 Permit. Additional testing and/or studies may be required by City Engineer to substantiate findings.

Categorization of Infiltration Feasibility Condition based on Geotechnical Conditions		Worksheet C.4-1: Form I- 8A[10]
Part 2 – Partial vs. No Infiltration Feasibility Screening Criteria		
DMA(s) Being Analyzed:		**Project Phase:**
Avion		PLANNING
Criteria 3: Infiltration Rate Screening		
3A	**NRCS Type C, D, or "urban/unclassified":** Is the mapped hydrologic soil group according to the NRCS Web Soil Survey or UC Davis Soil Web Mapper is Type C, D, or "urban/unclassified" and corroborated by available site soil data? ☐ Yes; the site is mapped as C soils and a reliable infiltration rate of 0.15 in/hr. is used to size partial infiltration BMPS. Answer "Yes" to Criteria 3 Result. ☐ Yes; the site is mapped as D soils or "urban/unclassified" and a reliable infiltration rate of 0.05 in/hr. is used to size partial infiltration BMPS. Answer "Yes" to Criteria 3 Result. ☒ No; infiltration testing is conducted (refer to Table D.3–1), continue to Step 3B.	
3B	**Infiltration Testing Result:** Is the reliable infiltration rate (i.e. average measured infiltration rate/2) greater than 0.05 in/hr. and less than or equal to 0.5 in/hr? ☐ Yes; the site may support partial infiltration. Answer "Yes" to Criteria 3 Result. ☒ No; the reliable infiltration rate (i.e. average measured rate/2) is less than 0.05 in/hr., partial infiltration is not required. Answer "No" to Criteria 3 Result.	
Criteria 3 Result	Is the estimated reliable infiltration rate (i.e., average measured infiltration rate/2) greater than or equal to 0.05 inches/hour and less than or equal to 0.5 inches/hour at any location within each DMA where runoff can reasonably be routed to a BMP? ☐ Yes; Continue to Criteria 4. ☒ No: Skip to Part 2 Result.	
Summarize infiltration testing and/or mapping results (i.e. soil maps and series description used for infiltration rate). Two permeability tests using our constant-head Aardvark permeameter were performed, both within the top foot of metavolcanic rock. The unfactored infiltration rates for the metavolcanic rock member was measured to be 0.017 and 0.008 inches/hour (iph). After applying a feasibility factor of safety of 2, the design infiltration rates for the metavolcanic rock are between 0.0085 to 0.004 iph. The Aardvark Permeameter test results are attached. In accordance with the Riverside County storm water procedures, which reference the United States Bureau of Reclamation Well Permeameter Method (USBR 7300), the saturated hydraulic conductivity is equal to the unfactored infiltration rate. The USDA NRCS Web Soil Survey of the proposed area indicated that 100% of the area belongs to Hydrologic Soil Group D (SnG). Based on the above information, full infiltration BMP's supported by the metavolcanic rock are not feasible. Please refer to the geotechnical investigation, Appendix C, for additional information. The locations of the borings and permeability tests are shown on the Geologic Map, Figure 2.		

Categorization of Infiltration Feasibility Condition based on Geotechnical Conditions		Worksheet C.4-1: Form I- 8A[10]	
Criteria 4: Geologic/Geotechnical Screening			
4A	If all questions in Step 4A are answered “Yes,” continue to Step 2B. For any “No” answer in Step 4A answer “No” to Criteria 4 Result, and submit an “Infiltration Feasibility Condition Letter” that meets the requirements in Appendix C.1.1. The geologic/geotechnical analyses listed in Appendix C.2.1 do not apply to the DMA because one of the following setbacks cannot be avoided and therefore result in the DMA being in a no infiltration condition. The setbacks must be the closest horizontal radial distance from the surface edge (at the overflow elevation) of the BMP.		
4A–1	Can the proposed partial infiltration BMP(s) avoid areas with existing fill materials greater than 5 feet thick?	☐ Yes	☐ No
4A-2	Can the proposed partial infiltration BMP(s) avoid placement within 10 feet of existing underground utilities, structures, or retaining walls?	☐ Yes	☐ No
4A-3	Can the proposed partial infiltration BMP(s) avoid placement within 50 feet of a natural slope (>25%) or within a distance of 1.5H from fill slopes where H is the height of the fill slope?	☐ Yes	☐ No
4B	When full infiltration is determined to be feasible, a geotechnical investigation report must be prepared that considers the relevant factors identified in Appendix C.2.1 If all questions in Step 4B are answered “Yes,” then answer “Yes” to Criteria 4 Result. If there are any “No” answers continue to Step 4C.		
4B–1	**Hydroconsolidation.** Analyze hydroconsolidation potential per approved ASTM standard due to a proposed full infiltration BMP. Can partial infiltration BMPs be proposed within the DMA without increasing hydroconsolidation risks?	☐ Yes	☐ No
4B-2	**Expansive Soils.** Identify expansive soils (soils with an expansion index greater than 20) and the extent of such soils due to proposed full infiltration BMPs. Can partial infiltration BMPs be proposed within the DMA without increasing expansive soil risks?	☐ Yes	☐ No

Categorization of Infiltration Feasibility Condition based on Geotechnical Conditions		Worksheet C.4-1: Form I- 8A[10]	
4B-3	**Liquefaction**. If applicable, identify mapped liquefaction areas. Evaluate liquefaction hazards in accordance with Section 6.4.2 of the City of San Diego's Guidelines for Geotechnical Reports (2011). Liquefaction hazard assessment shall take into account any increase in groundwater elevation or groundwater mounding that could occur as a result of proposed infiltration or percolation facilities. Can partial infiltration BMPs be proposed within the DMA without increasing liquefaction risks?	☐ Yes	☐ No
4B-4	**Slope Stability**. If applicable, perform a slope stability analysis in accordance with the ASCE and Southern California Earthquake Center (2002) Recommended Procedures for Implementation of DMG Special Publication 117, Guidelines for Analyzing and Mitigating Landslide Hazards in California to determine minimum slope setbacks for full infiltration BMPs. See the City of San Diego's Guidelines for Geotechnical Reports (2011) to determine which type of slope stability analysis is required. Can partial infiltration BMPs be proposed within the DMA without increasing slope stability risks?	☐ Yes	☐ No
4B-5	**Other Geotechnical Hazards.** Identify site-specific geotechnical hazards not already mentioned (refer to Appendix C.2.1). Can partial infiltration BMPs be proposed within the DMA without increasing risk of geologic or geotechnical hazards not already mentioned?	☐ Yes	☐ No
4B-6	**Setbacks.** Establish setbacks from underground utilities, structures, and/or retaining walls. Reference applicable ASTM or other recognized standard in the geotechnical report. Can partial infiltration BMPs be proposed within the DMA using recommended setbacks from underground utilities, structures, and/or retaining walls?	☐ Yes	☐ No
4C	**Mitigation Measures.** Propose mitigation measures for each geologic/geotechnical hazard identified in Step 4B. Provide a discussion on geologic/geotechnical hazards that would prevent partial infiltration BMPs that cannot be reasonably mitigated in the geotechnical report. See Appendix C.2.1.8 for a list of typically reasonable and typically unreasonable mitigation measures. Can mitigation measures be proposed to allow for partial infiltration BMPs? If the question in Step 4C is answered “Yes,” then answer “Yes” to Criteria 4 Result. If the question in Step 4C is answered “No,” then answer “No” to Criteria 4 Result.	☐ Yes	☐ No

Categorization of Infiltration Feasibility Condition based on Geotechnical Conditions		Worksheet C.4-1: Form I- 8A[10]	
Criteria 4 Result	Can infiltration of greater than or equal to 0.05 inches/hour and less than or equal to 0.5 inches/hour be allowed without increasing the risk of geologic or geotechnical hazards that cannot be reasonably mitigated to an acceptable level?	☐ Yes	☐ No
Summarize findings and basis; provide references to related reports or exhibits.			

Part 2 – Partial Infiltration Geotechnical Screening Result[13]	Result
If answers to both Criteria 3 and Criteria 4 are "Yes", a partial infiltration design is potentially feasible based on geotechnical conditions only. If answers to either Criteria 3 or Criteria 4 is "No", then infiltration of any volume is considered to be infeasible within the site.	☐ Partial Infiltration Condition ☒ No Infiltration Condition

[13] To be completed using gathered site information and best professional judgement considering the definition of MEP in the MS4 Permit. Additional testing and/or studies may be required by City Engineer to substantiate findings.

GEOCON

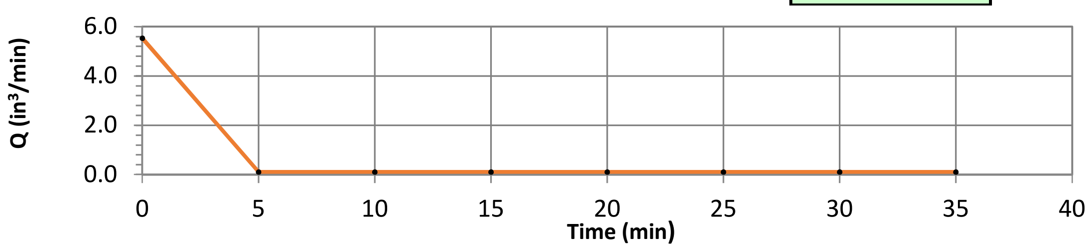
Aardvark Permeameter Data Analysis

Project Name:	Avion	Date:	5/21/2018
Project Number:	G2213-32-01	By:	DEG
Test Number:	P-1		

Borehole Diameter, d (in.):	4.00	Ref. EL (feet, MSL):	711.0
Borehole Depth, **H** (in):	12.00	Bottom EL (feet, MSL):	710.0
Distance Between Reservoir & Top of Borehole (in.)	41.00		
Estimated Depth to Water Table, **S** (feet):	100.00		
Height APM Raised from Bottom (in.):	1.00		
Pressure Reducer Used:	No		

Distance Between Reservoir and APM Float, **D** (in.):	44.75
Head Height Calculated, **h** (in.):	4.65
Head Height Measured, **h** (in.):	5.50
Distance Between Constant Head and Water Table, **L** (in.):	1193.50

Reading	Time Elapsed (min)	Water Weight Consumed (lbs)	Water Volume Consumed (in^3)	Q (in^3/min)
1	0.00	0.000	0.00	0.00
2	5.00	1.000	27.69	5.538
3	5.00	0.020	0.55	0.111
4	5.00	0.020	0.55	0.111
5	5.00	0.020	0.55	0.111
6	5.00	0.020	0.55	0.111
7	5.00	0.020	0.55	0.111
8	5.00	0.020	0.55	0.111
9	5.00	0.020	0.55	0.111
			Steady Flow Rate, Q (in^3/min):	0.111



Soil Matric Flux Potential, Φ_m

Φ_m= 0.00230 in^2/min

Field-Saturated Hydraulic Conductivity (Infiltration Rate)

K_{sat} = 2.34E-04 in/min 0.014 in/hr

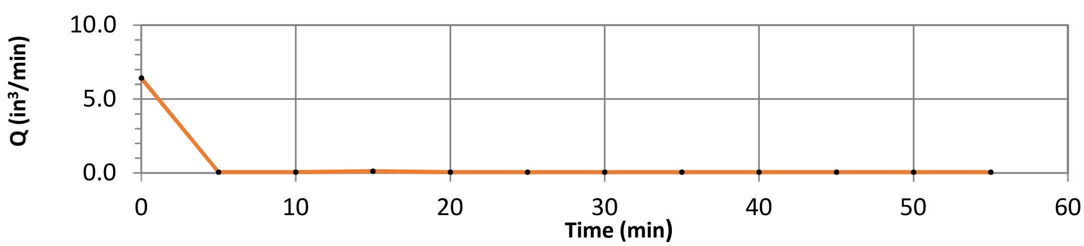
Aardvark Permeameter Data Analysis

Project Name:	Avion	Date:	5/21/2018
Project Number:	G2213-32-01	By:	DEG
Test Number:	P-2	Ref. EL (feet, MSL):	711.0
		Bottom EL (feet, MSL):	709.5

Borehole Diameter, d (in.):	4.00
Borehole Depth, **H** (in):	18.00
Distance Between Reservoir & Top of Borehole (in.)	31.00
Estimated Depth to Water Table, **S** (feet):	100.00
Height APM Raised from Bottom (in.):	1.00
Pressure Reducer Used:	No

Distance Between Reservoir and APM Float, **D** (in.):	40.75
Head Height Calculated, **h** (in.):	4.64
Head Height Measured, **h** (in.):	5.50
Distance Between Constant Head and Water Table, **L** (in.):	1187.50

Reading	Time Elapsed (min)	Water Weight Consumed (lbs)	Water Volume Consumed (in^3)	Q (in^3/min)
1	0.00	0.000	0.00	0.00
2	5.00	1.160	32.12	6.425
3	5.00	0.010	0.28	0.055
4	5.00	0.012	0.33	0.066
5	5.00	0.023	0.64	0.127
6	5.00	0.010	0.28	0.055
7	5.00	0.010	0.28	0.055
8	5.00	0.010	0.28	0.055
9	5.00	0.010	0.28	0.055
10	5.00	0.010	0.28	0.055
11	5.00	0.010	0.28	0.055
12	5.00	0.010	0.28	0.055
13	5.00	0.010	0.28	0.055
			Steady Flow Rate, Q (in^3/min):	0.055



Soil Matric Flux Potential, Φ_m

Φ_m= 0.0011 in^2/min

Field-Saturated Hydraulic Conductivity (Infiltration Rate)

K_{sat} = 1.17E-04 in/min 0.007 in/hr

United States Department of Agriculture

NRCS

Natural Resources Conservation Service

A product of the National Cooperative Soil Survey, a joint effort of the United States Department of Agriculture and other Federal agencies, State agencies including the Agricultural Experiment Stations, and local participants

Custom Soil Resource Report for

San Diego County Area, California

May 24, 2018

Preface

Soil surveys contain information that affects land use planning in survey areas. They highlight soil limitations that affect various land uses and provide information about the properties of the soils in the survey areas. Soil surveys are designed for many different users, including farmers, ranchers, foresters, agronomists, urban planners, community officials, engineers, developers, builders, and home buyers. Also, conservationists, teachers, students, and specialists in recreation, waste disposal, and pollution control can use the surveys to help them understand, protect, or enhance the environment.

Various land use regulations of Federal, State, and local governments may impose special restrictions on land use or land treatment. Soil surveys identify soil properties that are used in making various land use or land treatment decisions. The information is intended to help the land users identify and reduce the effects of soil limitations on various land uses. The landowner or user is responsible for identifying and complying with existing laws and regulations.

Although soil survey information can be used for general farm, local, and wider area planning, onsite investigation is needed to supplement this information in some cases. Examples include soil quality assessments (http://www.nrcs.usda.gov/wps/portal/nrcs/main/soils/health/) and certain conservation and engineering applications. For more detailed information, contact your local USDA Service Center (https://offices.sc.egov.usda.gov/locator/app?agency=nrcs) or your NRCS State Soil Scientist (http://www.nrcs.usda.gov/wps/portal/nrcs/detail/soils/contactus/?cid=nrcs142p2_053951).

Great differences in soil properties can occur within short distances. Some soils are seasonally wet or subject to flooding. Some are too unstable to be used as a foundation for buildings or roads. Clayey or wet soils are poorly suited to use as septic tank absorption fields. A high water table makes a soil poorly suited to basements or underground installations.

The National Cooperative Soil Survey is a joint effort of the United States Department of Agriculture and other Federal agencies, State agencies including the Agricultural Experiment Stations, and local agencies. The Natural Resources Conservation Service (NRCS) has leadership for the Federal part of the National Cooperative Soil Survey.

Information about soils is updated periodically. Updated information is available through the NRCS Web Soil Survey, the site for official soil survey information.

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How Soil Surveys Are Made

Soil surveys are made to provide information about the soils and miscellaneous areas in a specific area. They include a description of the soils and miscellaneous areas and their location on the landscape and tables that show soil properties and limitations affecting various uses. Soil scientists observed the steepness, length, and shape of the slopes; the general pattern of drainage; the kinds of crops and native plants; and the kinds of bedrock. They observed and described many soil profiles. A soil profile is the sequence of natural layers, or horizons, in a soil. The profile extends from the surface down into the unconsolidated material in which the soil formed or from the surface down to bedrock. The unconsolidated material is devoid of roots and other living organisms and has not been changed by other biological activity.

Currently, soils are mapped according to the boundaries of major land resource areas (MLRAs). MLRAs are geographically associated land resource units that share common characteristics related to physiography, geology, climate, water resources, soils, biological resources, and land uses (USDA, 2006). Soil survey areas typically consist of parts of one or more MLRA.

The soils and miscellaneous areas in a survey area occur in an orderly pattern that is related to the geology, landforms, relief, climate, and natural vegetation of the area. Each kind of soil and miscellaneous area is associated with a particular kind of landform or with a segment of the landform. By observing the soils and miscellaneous areas in the survey area and relating their position to specific segments of the landform, a soil scientist develops a concept, or model, of how they were formed. Thus, during mapping, this model enables the soil scientist to predict with a considerable degree of accuracy the kind of soil or miscellaneous area at a specific location on the landscape.

Commonly, individual soils on the landscape merge into one another as their characteristics gradually change. To construct an accurate soil map, however, soil scientists must determine the boundaries between the soils. They can observe only a limited number of soil profiles. Nevertheless, these observations, supplemented by an understanding of the soil-vegetation-landscape relationship, are sufficient to verify predictions of the kinds of soil in an area and to determine the boundaries.

Soil scientists recorded the characteristics of the soil profiles that they studied. They noted soil color, texture, size and shape of soil aggregates, kind and amount of rock fragments, distribution of plant roots, reaction, and other features that enable them to identify soils. After describing the soils in the survey area and determining their properties, the soil scientists assigned the soils to taxonomic classes (units). Taxonomic classes are concepts. Each taxonomic class has a set of soil characteristics with precisely defined limits. The classes are used as a basis for comparison to classify soils systematically. Soil taxonomy, the system of taxonomic classification used in the United States, is based mainly on the kind and character of soil properties and the arrangement of horizons within the profile. After the soil scientists classified and named the soils in the survey area, they compared the individual soils with similar soils in the same taxonomic class in other areas so that they could confirm data and assemble additional data based on experience and research.

The objective of soil mapping is not to delineate pure map unit components; the objective is to separate the landscape into landforms or landform segments that have similar use and management requirements. Each map unit is defined by a unique combination of soil components and/or miscellaneous areas in predictable proportions. Some components may be highly contrasting to the other components of the map unit. The presence of minor components in a map unit in no way diminishes the usefulness or accuracy of the data. The delineation of such landforms and landform segments on the map provides sufficient information for the development of resource plans. If intensive use of small areas is planned, onsite investigation is needed to define and locate the soils and miscellaneous areas.

Soil scientists make many field observations in the process of producing a soil map. The frequency of observation is dependent upon several factors, including scale of mapping, intensity of mapping, design of map units, complexity of the landscape, and experience of the soil scientist. Observations are made to test and refine the soil-landscape model and predictions and to verify the classification of the soils at specific locations. Once the soil-landscape model is refined, a significantly smaller number of measurements of individual soil properties are made and recorded. These measurements may include field measurements, such as those for color, depth to bedrock, and texture, and laboratory measurements, such as those for content of sand, silt, clay, salt, and other components. Properties of each soil typically vary from one point to another across the landscape.

Observations for map unit components are aggregated to develop ranges of characteristics for the components. The aggregated values are presented. Direct measurements do not exist for every property presented for every map unit component. Values for some properties are estimated from combinations of other properties.

While a soil survey is in progress, samples of some of the soils in the area generally are collected for laboratory analyses and for engineering tests. Soil scientists interpret the data from these analyses and tests as well as the field-observed characteristics and the soil properties to determine the expected behavior of the soils under different uses. Interpretations for all of the soils are field tested through observation of the soils in different uses and under different levels of management. Some interpretations are modified to fit local conditions, and some new interpretations are developed to meet local needs. Data are assembled from other sources, such as research information, production records, and field experience of specialists. For example, data on crop yields under defined levels of management are assembled from farm records and from field or plot experiments on the same kinds of soil.

Predictions about soil behavior are based not only on soil properties but also on such variables as climate and biological activity. Soil conditions are predictable over long periods of time, but they are not predictable from year to year. For example, soil scientists can predict with a fairly high degree of accuracy that a given soil will have a high water table within certain depths in most years, but they cannot predict that a high water table will always be at a specific level in the soil on a specific date.

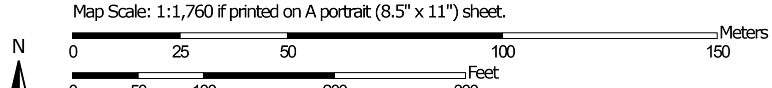
After soil scientists located and identified the significant natural bodies of soil in the survey area, they drew the boundaries of these bodies on aerial photographs and identified each as a specific map unit. Aerial photographs show trees, buildings, fields, roads, and rivers, all of which help in locating boundaries accurately.

Soil Map

The soil map section includes the soil map for the defined area of interest, a list of soil map units on the map and extent of each map unit, and cartographic symbols displayed on the map. Also presented are various metadata about data used to produce the map, and a description of each soil map unit.

Soil Map

Map Scale: 1:1,760 if printed on A portrait (8.5" x 11") sheet.



Map projection: Web Mercator Corner coordinates: WGS84 Edge tics: UTM Zone 11N WGS84

MAP LEGEND

Area of Interest (AOI)

Area of Interest (AOI)

Soils

Soil Map Unit Polygons

Soil Map Unit Lines

Soil Map Unit Points

Special Point Features

Blowout

Borrow Pit

Clay Spot

Closed Depression

Gravel Pit

Gravelly Spot

Landfill

Lava Flow

Marsh or swamp

Mine or Quarry

Miscellaneous Water

Perennial Water

Rock Outcrop

Saline Spot

Sandy Spot

Severely Eroded Spot

Sinkhole

Slide or Slip

Sodic Spot

Spoil Area

Stony Spot

Very Stony Spot

Wet Spot

Other

Special Line Features

Water Features

Streams and Canals

Transportation

Rails

Interstate Highways

US Routes

Major Roads

Local Roads

Background

Aerial Photography

MAP INFORMATION

The soil surveys that comprise your AOI were mapped at 1:24,000.

Warning: Soil Map may not be valid at this scale.

Enlargement of maps beyond the scale of mapping can cause misunderstanding of the detail of mapping and accuracy of soil line placement. The maps do not show the small areas of contrasting soils that could have been shown at a more detailed scale.

Please rely on the bar scale on each map sheet for map measurements.

Source of Map: Natural Resources Conservation Service
Web Soil Survey URL:
Coordinate System: Web Mercator (EPSG:3857)

Maps from the Web Soil Survey are based on the Web Mercator projection, which preserves direction and shape but distorts distance and area. A projection that preserves area, such as the Albers equal-area conic projection, should be used if more accurate calculations of distance or area are required.

This product is generated from the USDA-NRCS certified data as of the version date(s) listed below.

Soil Survey Area: San Diego County Area, California
Survey Area Data: Version 12, Sep 13, 2017

Soil map units are labeled (as space allows) for map scales 1:50,000 or larger.

Date(s) aerial images were photographed: Nov 3, 2014—Nov 22, 2014

The orthophoto or other base map on which the soil lines were compiled and digitized probably differs from the background imagery displayed on these maps. As a result, some minor shifting of map unit boundaries may be evident.

Map Unit Legend

Map Unit Symbol	Map Unit Name	Acres in AOI	Percent of AOI
SnG	San Miguel-Exchequer rocky silt loams, 9 to 70 percent slopes	12.3	100.0%
Totals for Area of Interest		**12.3**	**100.0%**

Map Unit Descriptions

The map units delineated on the detailed soil maps in a soil survey represent the soils or miscellaneous areas in the survey area. The map unit descriptions, along with the maps, can be used to determine the composition and properties of a unit.

A map unit delineation on a soil map represents an area dominated by one or more major kinds of soil or miscellaneous areas. A map unit is identified and named according to the taxonomic classification of the dominant soils. Within a taxonomic class there are precisely defined limits for the properties of the soils. On the landscape, however, the soils are natural phenomena, and they have the characteristic variability of all natural phenomena. Thus, the range of some observed properties may extend beyond the limits defined for a taxonomic class. Areas of soils of a single taxonomic class rarely, if ever, can be mapped without including areas of other taxonomic classes. Consequently, every map unit is made up of the soils or miscellaneous areas for which it is named and some minor components that belong to taxonomic classes other than those of the major soils.

Most minor soils have properties similar to those of the dominant soil or soils in the map unit, and thus they do not affect use and management. These are called noncontrasting, or similar, components. They may or may not be mentioned in a particular map unit description. Other minor components, however, have properties and behavioral characteristics divergent enough to affect use or to require different management. These are called contrasting, or dissimilar, components. They generally are in small areas and could not be mapped separately because of the scale used. Some small areas of strongly contrasting soils or miscellaneous areas are identified by a special symbol on the maps. If included in the database for a given area, the contrasting minor components are identified in the map unit descriptions along with some characteristics of each. A few areas of minor components may not have been observed, and consequently they are not mentioned in the descriptions, especially where the pattern was so complex that it was impractical to make enough observations to identify all the soils and miscellaneous areas on the landscape.

The presence of minor components in a map unit in no way diminishes the usefulness or accuracy of the data. The objective of mapping is not to delineate pure taxonomic classes but rather to separate the landscape into landforms or landform segments that have similar use and management requirements. The delineation of such segments on the map provides sufficient information for the development of resource plans. If intensive use of small areas is planned, however, onsite investigation is needed to define and locate the soils and miscellaneous areas.

An identifying symbol precedes the map unit name in the map unit descriptions. Each description includes general facts about the unit and gives important soil properties and qualities.

Soils that have profiles that are almost alike make up a *soil series*. Except for differences in texture of the surface layer, all the soils of a series have major horizons that are similar in composition, thickness, and arrangement.

Soils of one series can differ in texture of the surface layer, slope, stoniness, salinity, degree of erosion, and other characteristics that affect their use. On the basis of such differences, a soil series is divided into *soil phases*. Most of the areas shown on the detailed soil maps are phases of soil series. The name of a soil phase commonly indicates a feature that affects use or management. For example, Alpha silt loam, 0 to 2 percent slopes, is a phase of the Alpha series.

Some map units are made up of two or more major soils or miscellaneous areas. These map units are complexes, associations, or undifferentiated groups.

A *complex* consists of two or more soils or miscellaneous areas in such an intricate pattern or in such small areas that they cannot be shown separately on the maps. The pattern and proportion of the soils or miscellaneous areas are somewhat similar in all areas. Alpha-Beta complex, 0 to 6 percent slopes, is an example.

An *association* is made up of two or more geographically associated soils or miscellaneous areas that are shown as one unit on the maps. Because of present or anticipated uses of the map units in the survey area, it was not considered practical or necessary to map the soils or miscellaneous areas separately. The pattern and relative proportion of the soils or miscellaneous areas are somewhat similar. Alpha-Beta association, 0 to 2 percent slopes, is an example.

An *undifferentiated group* is made up of two or more soils or miscellaneous areas that could be mapped individually but are mapped as one unit because similar interpretations can be made for use and management. The pattern and proportion of the soils or miscellaneous areas in a mapped area are not uniform. An area can be made up of only one of the major soils or miscellaneous areas, or it can be made up of all of them. Alpha and Beta soils, 0 to 2 percent slopes, is an example.

Some surveys include *miscellaneous areas*. Such areas have little or no soil material and support little or no vegetation. Rock outcrop is an example.

San Diego County Area, California

SnG—San Miguel-Exchequer rocky silt loams, 9 to 70 percent slopes

Map Unit Setting

National map unit symbol: hbgl
Elevation: 400 to 3,300 feet
Mean annual precipitation: 15 inches
Mean annual air temperature: 61 to 64 degrees F
Frost-free period: 220 to 280 days
Farmland classification: Not prime farmland

Map Unit Composition

San miguel and similar soils: 45 percent
Exchequer and similar soils: 35 percent
Minor components: 20 percent
Estimates are based on observations, descriptions, and transects of the mapunit.

Description of San Miguel

Setting

Landform: Mountain slopes
Landform position (two-dimensional): Backslope
Landform position (three-dimensional): Mountainflank
Down-slope shape: Linear
Across-slope shape: Concave
Parent material: Residuum weathered from metavolcanics

Typical profile

H1 - 0 to 8 inches: silt loam
H2 - 8 to 18 inches: clay loam, silty clay loam, clay
H2 - 8 to 18 inches: gravelly clay loam, gravelly silty clay loam, gravelly clay
H2 - 8 to 18 inches: unweathered bedrock
H3 - 18 to 23 inches:
H3 - 18 to 23 inches:
H3 - 18 to 23 inches:
H4 - 23 to 27 inches:

Properties and qualities

Slope: 9 to 30 percent
Depth to restrictive feature: 20 to 34 inches to lithic bedrock
Natural drainage class: Well drained
Runoff class: Very high
Capacity of the most limiting layer to transmit water (Ksat): Very low to moderately low (0.00 to 0.06 in/hr)
Depth to water table: More than 80 inches
Frequency of flooding: None
Frequency of ponding: None
Sodium adsorption ratio, maximum in profile: 15.0
Available water storage in profile: Low (about 3.5 inches)

Interpretive groups

Land capability classification (irrigated): None specified
Land capability classification (nonirrigated): 7e
Hydrologic Soil Group: D

Ecological site: ACID CLAYPAN (Claypan Mesas - 1975) (R019XD062CA)
Hydric soil rating: No

Description of Exchequer

Setting

Landform: Mountain slopes
Landform position (two-dimensional): Backslope
Down-slope shape: Linear
Across-slope shape: Concave
Parent material: Metabasic residuum weathered from igneous and metamorphic rock

Typical profile

H1 - 0 to 10 inches: gravelly silt loam
H2 - 10 to 14 inches: unweathered bedrock

Properties and qualities

Slope: 30 to 70 percent
Depth to restrictive feature: 4 to 20 inches to lithic bedrock
Natural drainage class: Well drained
Runoff class: High
Capacity of the most limiting layer to transmit water (Ksat): Moderately high to high (0.57 to 1.98 in/hr)
Depth to water table: More than 80 inches
Frequency of flooding: None
Frequency of ponding: None
Available water storage in profile: Very low (about 1.5 inches)

Interpretive groups

Land capability classification (irrigated): None specified
Land capability classification (nonirrigated): 7e
Hydrologic Soil Group: D
Ecological site: SHALLOW LOAMY (1975) (R019XD060CA)
Hydric soil rating: No

Minor Components

Rock outcrop

Percent of map unit: 10 percent
Hydric soil rating: No

Escondido

Percent of map unit: 5 percent
Hydric soil rating: No

Friant

Percent of map unit: 5 percent
Hydric soil rating: No

References

American Association of State Highway and Transportation Officials (AASHTO). 2004. Standard specifications for transportation materials and methods of sampling and testing. 24th edition.

American Society for Testing and Materials (ASTM). 2005. Standard classification of soils for engineering purposes. ASTM Standard D2487-00.

Cowardin, L.M., V. Carter, F.C. Golet, and E.T. LaRoe. 1979. Classification of wetlands and deep-water habitats of the United States. U.S. Fish and Wildlife Service FWS/OBS-79/31.

Federal Register. July 13, 1994. Changes in hydric soils of the United States.

Federal Register. September 18, 2002. Hydric soils of the United States.

Hurt, G.W., and L.M. Vasilas, editors. Version 6.0, 2006. Field indicators of hydric soils in the United States.

National Research Council. 1995. Wetlands: Characteristics and boundaries.

Soil Survey Division Staff. 1993. Soil survey manual. Soil Conservation Service. U.S. Department of Agriculture Handbook 18. http://www.nrcs.usda.gov/wps/portal/nrcs/detail/national/soils/?cid=nrcs142p2_054262

Soil Survey Staff. 1999. Soil taxonomy: A basic system of soil classification for making and interpreting soil surveys. 2nd edition. Natural Resources Conservation Service, U.S. Department of Agriculture Handbook 436. http://www.nrcs.usda.gov/wps/portal/nrcs/detail/national/soils/?cid=nrcs142p2_053577

Soil Survey Staff. 2010. Keys to soil taxonomy. 11th edition. U.S. Department of Agriculture, Natural Resources Conservation Service. http://www.nrcs.usda.gov/wps/portal/nrcs/detail/national/soils/?cid=nrcs142p2_053580

Tiner, R.W., Jr. 1985. Wetlands of Delaware. U.S. Fish and Wildlife Service and Delaware Department of Natural Resources and Environmental Control, Wetlands Section.

United States Army Corps of Engineers, Environmental Laboratory. 1987. Corps of Engineers wetlands delineation manual. Waterways Experiment Station Technical Report Y-87-1.

United States Department of Agriculture, Natural Resources Conservation Service. National forestry manual. http://www.nrcs.usda.gov/wps/portal/nrcs/detail/soils/home/?cid=nrcs142p2_053374

United States Department of Agriculture, Natural Resources Conservation Service. National range and pasture handbook. http://www.nrcs.usda.gov/wps/portal/nrcs/detail/national/landuse/rangepasture/?cid=stelprdb1043084

United States Department of Agriculture, Natural Resources Conservation Service. National soil survey handbook, title 430-VI. http://www.nrcs.usda.gov/wps/portal/nrcs/detail/soils/scientists/?cid=nrcs142p2_054242

United States Department of Agriculture, Natural Resources Conservation Service. 2006. Land resource regions and major land resource areas of the United States, the Caribbean, and the Pacific Basin. U.S. Department of Agriculture Handbook 296. http://www.nrcs.usda.gov/wps/portal/nrcs/detail/national/soils/?cid=nrcs142p2_053624

United States Department of Agriculture, Soil Conservation Service. 1961. Land capability classification. U.S. Department of Agriculture Handbook 210. http://www.nrcs.usda.gov/Internet/FSE_DOCUMENTS/nrcs142p2_052290.pdf

APPENDIX E

APPENDIX E

SEISMIC REFRACTION STUDY
PREPARED BY SOUTHWEST GEOPHYSICS,
DATED AUGUST 8, 2013

FOR

AVION
SAN DIEGO, CALIFORNIA

PROJECT NO. G2213-32-01

SEISMIC REFRACTION SURVEY
DEBEVOISE PROPERTY
SAN DIEGO, CALIFORNIA

PREPARED FOR:

Geocon Incorporated
6960 Flanders Drive
San Diego, CA 92121-2974

PREPARED BY:

Southwest Geophysics, Inc.
8057 Raytheon Road, Suite 9
San Diego, CA 92111

August 8, 2013
Project No. 113283

SOUTHWEST
GEOPHYSICS, INC.
YOUR SUBSURFACE SOLUTION

August 8, 2013
Project No. 113283

Mr. Troy Reist
Geocon Incorporated
6960 Flanders Drive
San Diego, CA 92121-2974

Subject: Seismic Refraction Survey
Debevoise Property
San Diego, California

Dear Mr. Reist:

In accordance with your authorization, we have performed a seismic refraction survey pertaining to the Debevoise property located in the Rancho Bernardo area of San Diego, California. Specifically, our survey consisted of performing six seismic refraction traverses at the project site. The purpose of our services was to evaluate the apparent rippability of the subsurface materials and develop a subsurface velocity model of the areas surveyed for use in the design and construction of future improvements.

We appreciate the opportunity to be of service on this project. Should you have any questions related to this report, please contact the undersigned at your convenience.

Sincerely,
SOUTHWEST GEOPHYSICS, INC.

Patrick Lehrmann

Patrick Lehrmann, P.G., P.Gp.
Principal Geologist/Geophysicist

Hans van de Vrugt

Hans van de Vrugt, C.E.G., P.Gp.
Principal Geologist/Geophysicist

REGISTERED GEOPHYSICIST
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Exp. 2/28/14
STATE OF CALIFORNIA

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Distribution: Addressee (electronic)

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1. INTRODUCTION

In accordance with your authorization, we have performed a seismic refraction survey pertaining to the Debevoise property located in the Rancho Bernardo area of San Diego, California (Figure 1). Specifically, our survey consisted of performing six seismic refraction traverses at the project site. The purpose of our services was to evaluate the apparent rippability of the subsurface materials and develop a subsurface velocity model of the areas surveyed for use in the design and construction of future improvements.

2. SCOPE OF SERVICES

Our scope of services included:

- Performance of six seismic refraction lines at the project site.
- Compilation and analysis of the data collected.
- Preparation of this data report presenting our results and conclusions.

3. SITE AND PROJECT DESCRIPTION

The project site is located along the south side of Bernardo Center Drive and is accessed at the south side of the intersection of Dove Canyon Road and Bernardo Center Drive (Figures 1 and 2). The study area is located along a dirt road which generally follows a south to north trending ridge. Vegetation in the area of the lines primarily consists of sage brush, scattered trees, and annual grass. Fragments of metavolcanic rock were observed in and near the study area; however, no undisturbed outcrops were observed in the area of the lines. Figures 2 and 3 depict the general site conditions.

Based on our discussions with you, it is our understanding that a residential development is planned for the project site and that grading will likely include substantial cuts and fills. Details regarding the proposed cuts are not yet known.

4. SURVEY METHODOLOGY

A seismic P-wave (compression wave) refraction survey was conducted at the site to evaluate the characteristics of the subsurface materials and specifically the depth to bedrock. The seismic refraction method uses first-arrival times of refracted seismic waves to estimate the thicknesses and seismic velocities of subsurface layers. Seismic P-waves generated at the surface, using a hammer and plate, are refracted at boundaries separating materials of contrasting velocities. These refracted seismic waves are then detected by a series of surface vertical component geophones and recorded with a 24-channel Geometrics Geode seismograph. The travel times of the seismic P-waves are used in conjunction with the shot-to-geophone distances to obtain thickness and velocity information on the subsurface materials.

Six seismic lines (SL-1 through SL-6) were conducted in the study area. The general locations and lengths of the lines were selected by your office. Shot points (signal generation locations) were generally conducted at five equally spaced locations along the lines.

The seismic refraction theory requires that subsurface velocities increase with depth. A layer having a velocity lower than that of the layer above will not generally be detectable by the seismic refraction method and, therefore, could lead to errors in the depth calculations of subsequent layers. In addition, lateral variations in velocity, such as those caused by core stones, intrusions or boulders can also result in the misinterpretation of the subsurface conditions.

In general, seismic wave velocities can be correlated to material density and/or rock hardness. The relationship between rippability and seismic velocity is empirical and assumes a homogenous mass. Localized areas of differing composition, texture, and/or structure may affect both the measured data and the actual rippability of the mass. The rippability of a mass is also dependent on the excavation equipment used and the skill and experience of the equipment operator.

The rippability values presented in Table 1 are based on our experience with similar materials and assumes that a Caterpillar D-9 dozer ripping with a single shank is used. We emphasize that the cutoffs in this classification scheme are approximate and that rock characteristics, such as fracture spacing and orientation, play a significant role in determining rock rippability. These characteristics may also vary with location and depth. For trenching operations, the rippability values should be scaled downward. For example, velocities as low as 3,500 feet/second may indicate difficult ripping during trenching operations. In addition, the presence of boulders, which can be troublesome in a narrow trench, should be anticipated.

Table 1 – Rippability Classification

Seismic P-wave Velocity	Rippability
0 to 2,000 feet/second	Easy
2,000 to 4,000 feet/second	Moderate
4,000 to 5,500 feet/second	Difficult, Possible Blasting
5,500 to 7,000 feet/second	Very Difficult, Probable Blasting
Greater than 7,000 feet/second	Blasting Generally Required

It should be noted that the rippability cutoffs presented in Table 1 are slightly more conservative than those published in the Caterpillar Performance Handbook (Caterpillar, 2011). Accordingly, the above classification scheme should be used with discretion, and contractors should not be relieved of making their own independent evaluation of the rippability of the on-site materials prior to submitting their bids.

5. RESULTS

As previously indicated, six seismic traverses were conducted as part of our study. The collected data were processed using SIPwin (Rimrock Geophysics, 2003), a seismic interpretation program, and analyzed using SeisOpt Pro (Optim, 2008). SeisOpt Pro uses first arrival picks and elevation data to produce a subsurface velocity model through a nonlinear optimization technique called adaptive simulated annealing. The resulting velocity model provides a tomography image of the estimated geologic conditions. Both vertical and lateral velocity information is contained in the tomography model. Changes in layer velocity are revealed as gradients rather than discrete contacts, which typically are more representative of actual conditions.

The approximate locations of the seismic refraction traverses are shown on the Line Location Map (Figure 2). The velocity models are included in Figures 4a through 4c. In general, the effective depth of evaluation for a seismic refraction traverse is approximately one-third to one-fifth the length of the traverse.

6. CONCLUSIONS AND RECOMMENDATIONS

The results from our seismic survey revealed distinct layers/zones in the near surface that likely represent soil (topsoil and colluvium) overlying metavolcanic bedrock with varying degrees of weathering. Figures 4a through 4c provide the velocity gradient models calculated from SeisOpt Pro. Several feet of soil-like materials are present along portions of the lines, but in general weathered bedrock appears to be fairly shallow across the site. Based on the models, significant lateral variations in velocity are also present in the survey area. The cause of the velocity variations are likely related to remnant boulders, fracturing, and differential weathering of the bedrock materials. As a result, variability in the excavatability (including depth of rippability) of the subsurface materials should be expected across the project area.

Based on our results, very difficult conditions where blasting may be required will likely be encountered depending on the excavation depth, location, and desired rate of production. In addition, oversized materials should be expected. A contractor with excavation experience in similar difficult conditions should be consulted for expert advice on excavation methodology, equipment and production rate. In addition, once grading plans have been prepared we recommend that additional seismic lines be conducted in areas of proposed significant cuts.

7. LIMITATIONS

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

This document is intended to be used only in its entirety. No portion of the document, by itself, is designed to completely represent any aspect of the project described herein. Southwest Geophysics, Inc. should be contacted if the reader requires additional information or has questions regarding the content, interpretations presented, or completeness of this document. This report is intended exclusively for use by the client. Any use or reuse of the findings, conclusions, and/or recommendations of this report by parties other than the client is undertaken at said parties' sole risk.

8. SELECTED REFERENCES

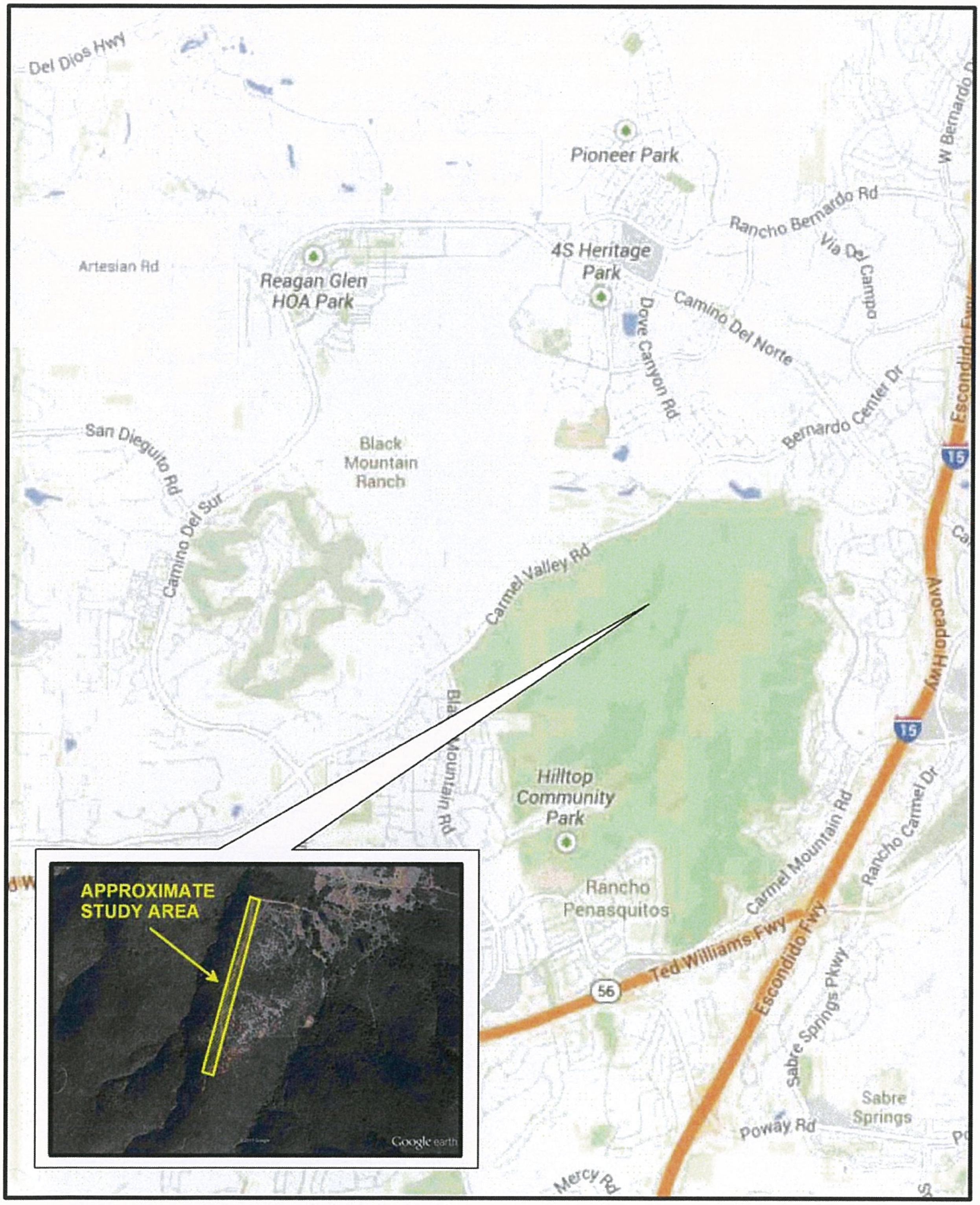
Caterpillar, Inc., 2011, Caterpillar Performance Handbook, Edition 41, Caterpillar, Inc., Peoria, Illinois.

Mooney, H.M., 1976, Handbook of Engineering Geophysics, dated February.

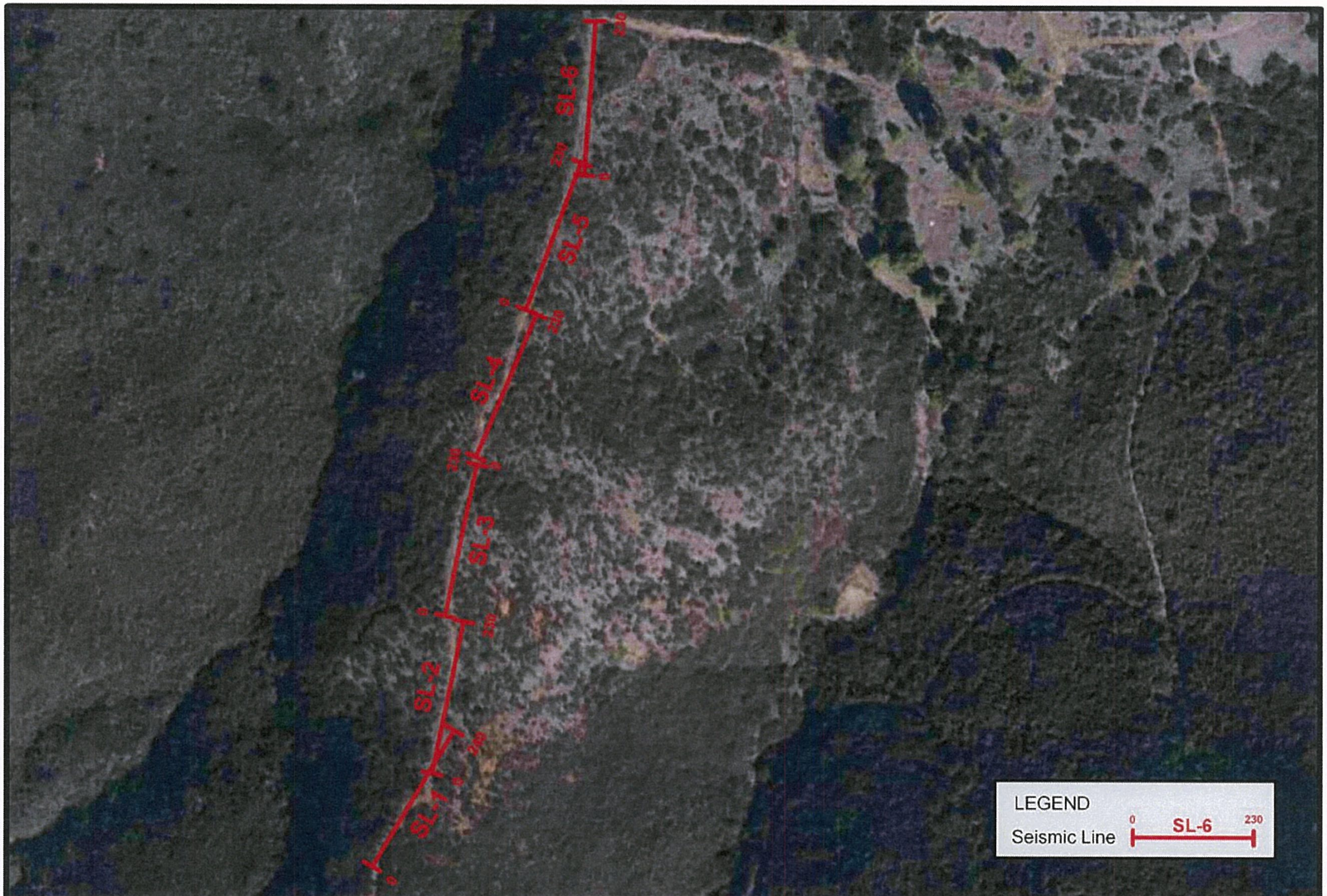
Optim, Inc., 2008, SeisOpt Pro, V-5.0.

Rimrock Geophysics, 2003, Seismic Refraction Interpretation Program (SIPwin), V-2.76.

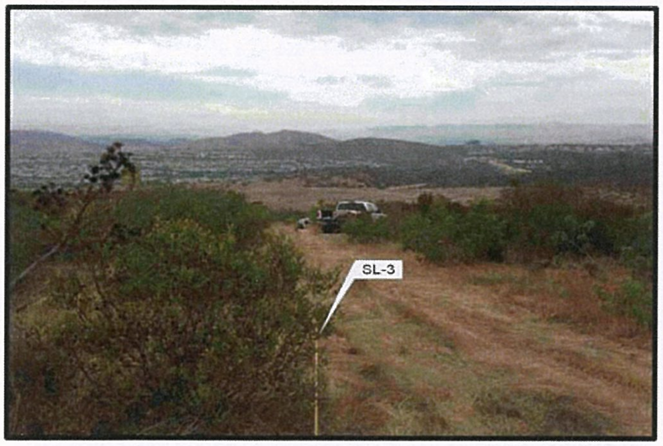
Telford, W.M., Geldart, L.P., Sheriff, R.E., and Keys, D.A., 1976, Applied Geophysics, Cambridge University Press.

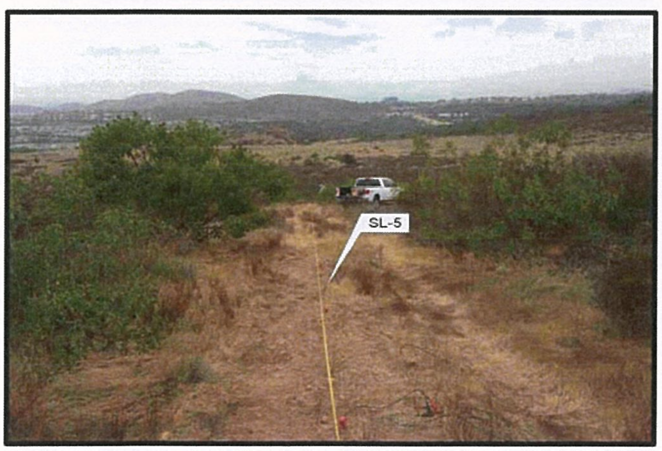


SITE LOCATION MAP	N	Debevoise San Diego, California	SOUTHWEST GEOPHYSICS INC.
		Project No.: 113283	Date: 08/13	Figure 1



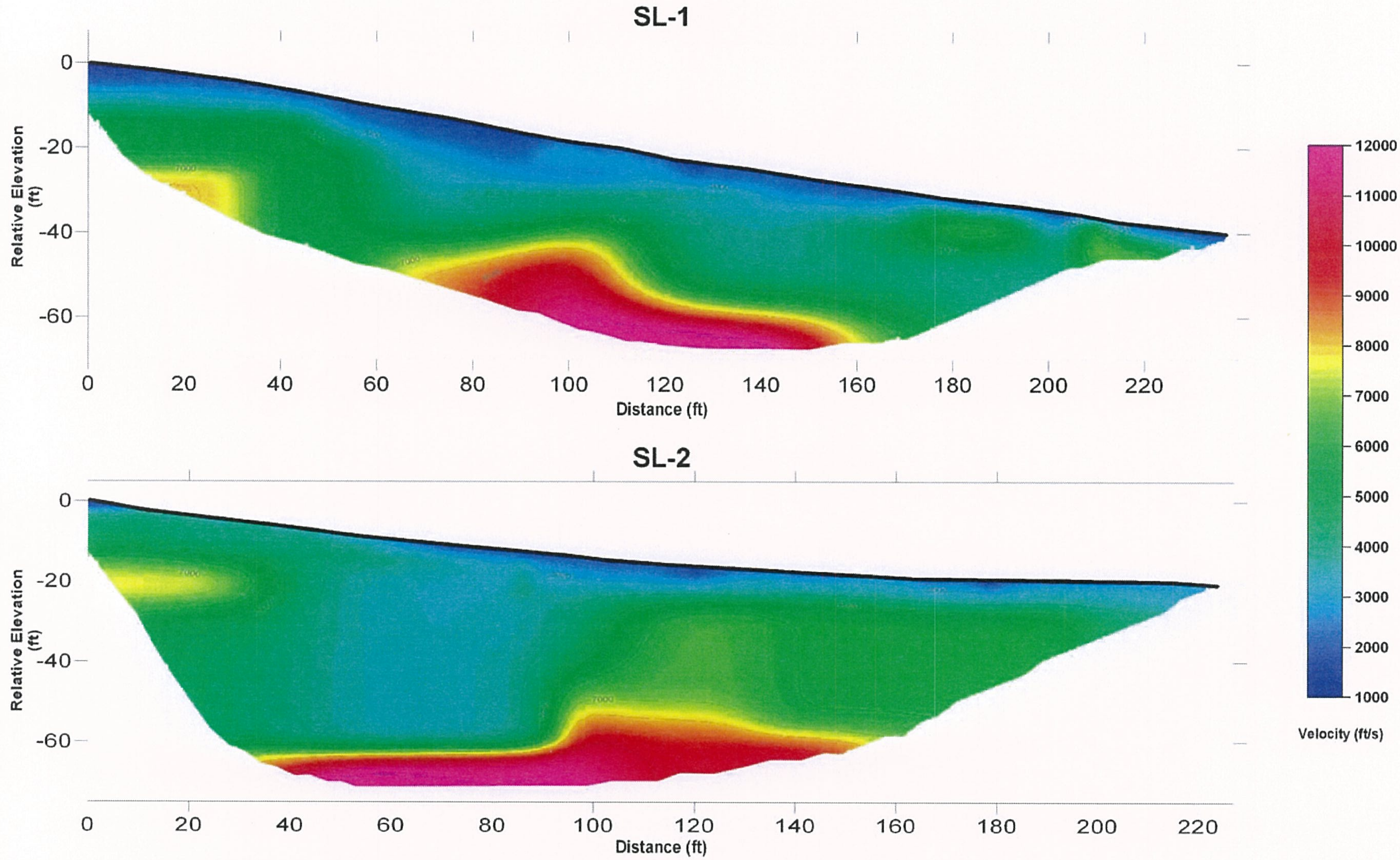
LINE LOCATION MAP	N	Debevoise Property San Diego, California Project No.: 113283 · Date: 08/13	SOUTHWEST GEOPHYSICS INC. Figure 2	0 · 200 · 400 approximate scale in feet





SITE PHOTOGRAPHS	Debevoise Property San Diego, California Project No.: 113283 Date: 08/13	SOUTHWEST GEOPHYSICS INC. Figure 3

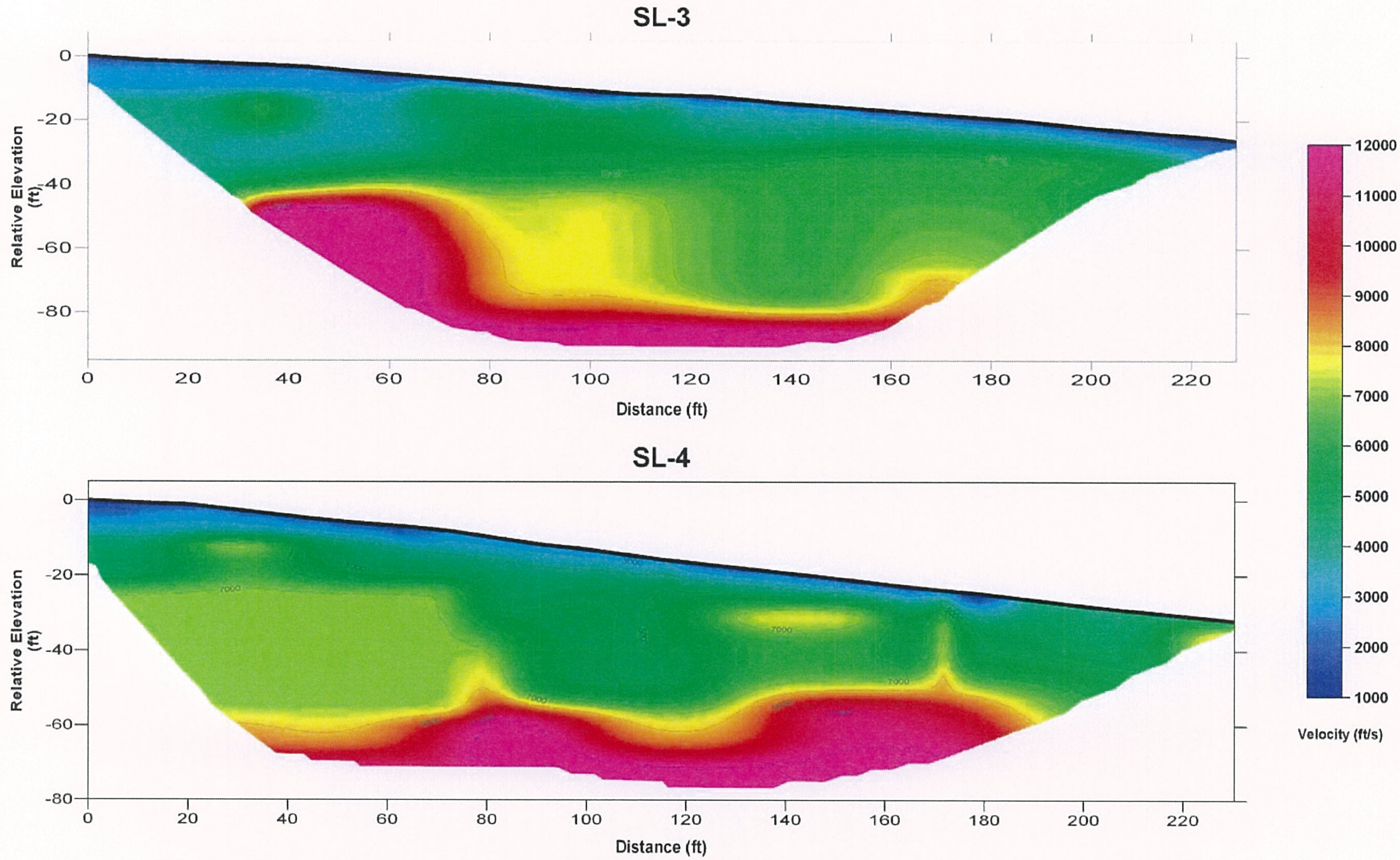
TOMOGRAPHY MODELS



SEISMIC PROFILES SL-1 & SL-2	Debevoise Property San Diego, California Project No.: 113283 Date: 08/13	SOUTHWEST GEOPHYSICS INC. Figure 4a

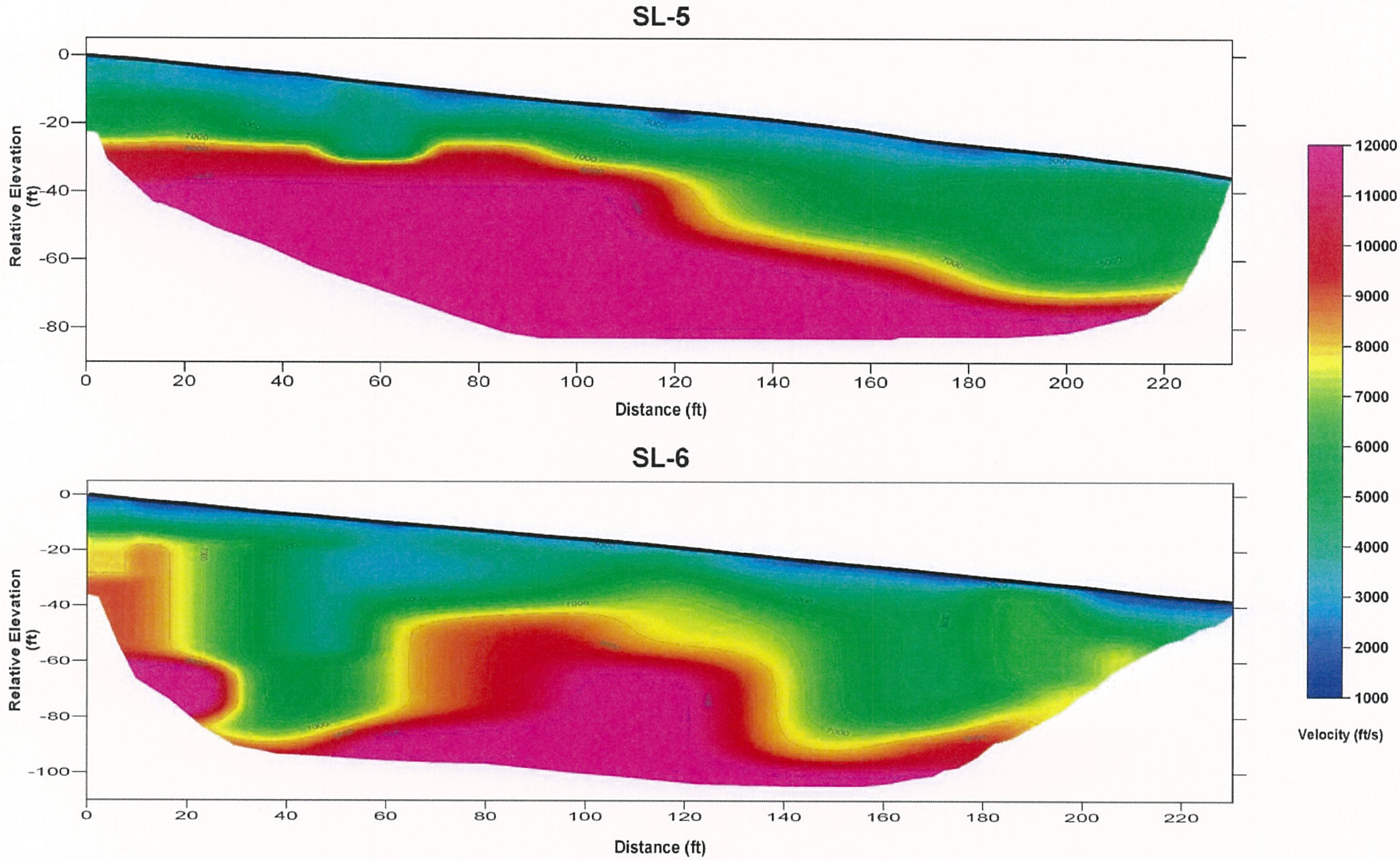
Note: Contour Interval = 1,000 feet per second

TOMOGRAPHY MODELS



SEISMIC PROFILES SL-3 & SL-4	Debevoise Property San Diego, California Project No.: 113283 — Date: 08/13	SOUTHWEST GEOPHYSICS INC. Figure 4b	**Note: Contour Interval = 1,000 feet per second**

TOMOGRAPHY MODELS



SEISMIC PROFILES SL-5 & SL-6	Debevoise Property San Diego, California Project No.: 113283 Date: 08/13	SOUTHWEST GEOPHYSICS INC. Figure 4c

Note: Contour Interval = 1,000 feet per second

APPENDIX F

APPENDIX F

RECOMMENDED GRADING SPECIFICATIONS

FOR

AVION
SAN DIEGO, CALIFORNIA

PROJECT NO. G2213-32-01

RECOMMENDED GRADING SPECIFICATIONS

1. GENERAL

1.1 These Recommended Grading Specifications shall be used in conjunction with the Geotechnical Report for the project prepared by Geocon. The recommendations contained in the text of the Geotechnical Report are a part of the earthwork and grading specifications and shall supersede the provisions contained hereinafter in the case of conflict.

1.2 Prior to the commencement of grading, a geotechnical consultant (Consultant) shall be employed for the purpose of observing earthwork procedures and testing the fills for substantial conformance with the recommendations of the Geotechnical Report and these specifications. The Consultant should provide adequate testing and observation services so that they may assess whether, in their opinion, the work was performed in substantial conformance with these specifications. It shall be the responsibility of the Contractor to assist the Consultant and keep them apprised of work schedules and changes so that personnel may be scheduled accordingly.

1.3 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, these specifications and the approved grading plans. If, in the opinion of the Consultant, unsatisfactory conditions such as questionable soil materials, poor moisture condition, inadequate compaction, and/or adverse weather result in a quality of work not in conformance with these specifications, the Consultant will be empowered to reject the work and recommend to the Owner that grading be stopped until the unacceptable conditions are corrected.

2. DEFINITIONS

2.1 **Owner** shall refer to the owner of the property or the entity on whose behalf the grading work is being performed and who has contracted with the Contractor to have grading performed.

2.2 **Contractor** shall refer to the Contractor performing the site grading work.

2.3 **Civil Engineer** or **Engineer of Work** shall refer to the California licensed Civil Engineer or consulting firm responsible for preparation of the grading plans, surveying and verifying as-graded topography.

2.4 **Consultant** shall refer to the soil engineering and engineering geology consulting firm retained to provide geotechnical services for the project.

2.5 **Soil Engineer** shall refer to a California licensed Civil Engineer retained by the Owner, who is experienced in the practice of geotechnical engineering. The Soil Engineer shall be responsible for having qualified representatives on-site to observe and test the Contractor's work for conformance with these specifications.

2.6 **Engineering Geologist** shall refer to a California licensed Engineering Geologist retained by the Owner to provide geologic observations and recommendations during the site grading.

2.7 **Geotechnical Report** shall refer to a soil report (including all addenda) which may include a geologic reconnaissance or geologic investigation that was prepared specifically for the development of the project for which these Recommended Grading Specifications are intended to apply.

3. MATERIALS

3.1 Materials for compacted fill shall consist of any soil excavated from the cut areas or imported to the site that, in the opinion of the Consultant, is suitable for use in construction of fills. In general, fill materials can be classified as *soil* fills, *soil-rock* fills or *rock* fills, as defined below.

3.1.1 **Soil fills** are defined as fills containing no rocks or hard lumps greater than 12 inches in maximum dimension and containing at least 40 percent by weight of material smaller than ¾ inch in size.

3.1.2 **Soil-rock fills** are defined as fills containing no rocks or hard lumps larger than 4 feet in maximum dimension and containing a sufficient matrix of soil fill to allow for proper compaction of soil fill around the rock fragments or hard lumps as specified in Paragraph 6.2. **Oversize rock** is defined as material greater than 12 inches.

3.1.3 **Rock fills** are defined as fills containing no rocks or hard lumps larger than 3 feet in maximum dimension and containing little or no fines. Fines are defined as material smaller than ¾ inch in maximum dimension. The quantity of fines shall be less than approximately 20 percent of the rock fill quantity.

3.2 Material of a perishable, spongy, or otherwise unsuitable nature as determined by the Consultant shall not be used in fills.

3.3 Materials used for fill, either imported or on-site, shall not contain hazardous materials as defined by the California Code of Regulations, Title 22, Division 4, Chapter 30, Articles 9 and 10; 40CFR; and any other applicable local, state or federal laws. The Consultant shall not be responsible for the identification or analysis of the potential presence of hazardous materials. However, if observations, odors or soil discoloration cause Consultant to suspect the presence of hazardous materials, the Consultant may request from the Owner the termination of grading operations within the affected area. Prior to resuming grading operations, the Owner shall provide a written report to the Consultant indicating that the suspected materials are not hazardous as defined by applicable laws and regulations.

3.4 The outer 15 feet of *soil-rock* fill slopes, measured horizontally, should be composed of properly compacted *soil* fill materials approved by the Consultant. *Rock* fill may extend to the slope face, provided that the slope is not steeper than 2:1 (horizontal:vertical) and a soil layer no thicker than 12 inches is track-walked onto the face for landscaping purposes. This procedure may be utilized provided it is acceptable to the governing agency, Owner and Consultant.

3.5 Samples of soil materials to be used for fill should be tested in the laboratory by the Consultant to determine the maximum density, optimum moisture content, and, where appropriate, shear strength, expansion, and gradation characteristics of the soil.

3.6 During grading, soil or groundwater conditions other than those identified in the Geotechnical Report may be encountered by the Contractor. The Consultant shall be notified immediately to evaluate the significance of the unanticipated condition.

4. CLEARING AND PREPARING AREAS TO BE FILLED

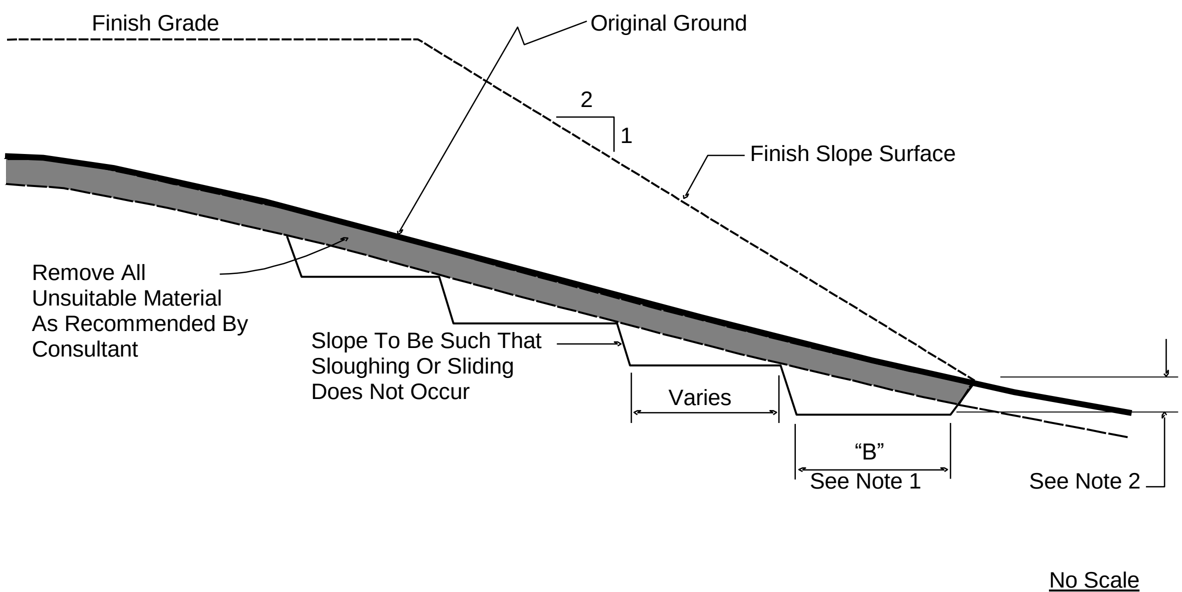
4.1 Areas to be excavated and filled shall be cleared and grubbed. Clearing shall consist of complete removal above the ground surface of trees, stumps, brush, vegetation, man-made structures, and similar debris. Grubbing shall consist of removal of stumps, roots, buried logs and other unsuitable material and shall be performed in areas to be graded. Roots and other projections exceeding 1½ inches in diameter shall be removed to a depth of 3 feet below the surface of the ground. Borrow areas shall be grubbed to the extent necessary to provide suitable fill materials.

4.2 Asphalt pavement material removed during clearing operations should be properly disposed at an approved off-site facility or in an acceptable area of the project evaluated by Geocon and the property owner. Concrete fragments that are free of reinforcing steel may be placed in fills, provided they are placed in accordance with Section 6.2 or 6.3 of this document.

4.3 After clearing and grubbing of organic matter and other unsuitable material, loose or porous soils shall be removed to the depth recommended in the Geotechnical Report. The depth of removal and compaction should be observed and approved by a representative of the Consultant. The exposed surface shall then be plowed or scarified to a minimum depth of 6 inches and until the surface is free from uneven features that would tend to prevent uniform compaction by the equipment to be used.

4.4 Where the slope ratio of the original ground is steeper than 5:1 (horizontal:vertical), or where recommended by the Consultant, the original ground should be benched in accordance with the following illustration.

TYPICAL BENCHING DETAIL



DETAIL NOTES:

(1) Key width "B" should be a minimum of 10 feet, 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 key should be below the topsoil or unsuitable surficial material and at least 2 feet into dense formational 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 Consultant.

4.5 After areas to receive fill have been cleared and scarified, the surface should be moisture conditioned to achieve the proper moisture content, and compacted as recommended in Section 6 of these specifications.

5. COMPACTION EQUIPMENT

5.1 Compaction of *soil* or *soil-rock* fill shall be accomplished by sheepsfoot or segmented-steel wheeled rollers, vibratory rollers, multiple-wheel pneumatic-tired rollers, or other types of acceptable compaction equipment. Equipment shall be of such a design that it will be capable of compacting the *soil* or *soil-rock* fill to the specified relative compaction at the specified moisture content.

5.2 Compaction of *rock* fills shall be performed in accordance with Section 6.3.

6. PLACING, SPREADING AND COMPACTION OF FILL MATERIAL

6.1 *Soil* fill, as defined in Paragraph 3.1.1, shall be placed by the Contractor in accordance with the following recommendations:

6.1.1 *Soil* fill shall be placed by the Contractor in layers that, when compacted, should generally not exceed 8 inches. Each layer shall be spread evenly and shall be thoroughly mixed during spreading to obtain uniformity of material and moisture in each layer. The entire fill shall be constructed as a unit in nearly level lifts. Rock materials greater than 12 inches in maximum dimension shall be placed in accordance with Section 6.2 or 6.3 of these specifications.

6.1.2 In general, the *soil* fill shall be compacted at a moisture content at or above the optimum moisture content as determined by ASTM D 1557.

6.1.3 When the moisture content of *soil* fill is below that specified by the Consultant, water shall be added by the Contractor until the moisture content is in the range specified.

6.1.4 When the moisture content of the *soil* fill is above the range specified by the Consultant or too wet to achieve proper compaction, the *soil* fill shall be aerated by the Contractor by blading/mixing, or other satisfactory methods until the moisture content is within the range specified.

6.1.5 After each layer has been placed, mixed, and spread evenly, it shall be thoroughly compacted by the Contractor to a relative compaction of at least 90 percent. Relative compaction is defined as the ratio (expressed in percent) of the in-place dry density of the compacted fill to the maximum laboratory dry density as determined in accordance with ASTM D 1557. Compaction shall be continuous over the entire area, and compaction equipment shall make sufficient passes so that the specified minimum relative compaction has been achieved throughout the entire fill.

6.1.6 Where practical, soils having an Expansion Index greater than 50 should be placed at least 3 feet below finish pad grade and should be compacted at a moisture content generally 2 to 4 percent greater than the optimum moisture content for the material.

6.1.7 Properly compacted *soil* fill shall extend to the design surface of fill slopes. To achieve proper compaction, it is recommended that fill slopes be over-built by at least 3 feet and then cut to the design grade. This procedure is considered preferable to track-walking of slopes, as described in the following paragraph.

6.1.8 As an alternative to over-building of slopes, slope faces may be back-rolled with a heavy-duty loaded sheepsfoot or vibratory roller at maximum 4-foot fill height intervals. Upon completion, slopes should then be track-walked with a D-8 dozer or similar equipment, such that a dozer track covers all slope surfaces at least twice.

6.2 *Soil-rock* fill, as defined in Paragraph 3.1.2, shall be placed by the Contractor in accordance with the following recommendations:

6.2.1 Rocks larger than 12 inches but less than 4 feet in maximum dimension may be incorporated into the compacted *soil* fill, but shall be limited to the area measured 15 feet minimum horizontally from the slope face and 5 feet below finish grade or 3 feet below the deepest utility, whichever is deeper.

6.2.2 Rocks or rock fragments up to 4 feet in maximum dimension may either be individually placed or placed in windrows. Under certain conditions, rocks or rock fragments up to 10 feet in maximum dimension may be placed using similar methods. The acceptability of placing rock materials greater than 4 feet in maximum dimension shall be evaluated during grading as specific cases arise and shall be approved by the Consultant prior to placement.

6.2.3 For individual placement, sufficient space shall be provided between rocks to allow for passage of compaction equipment.

6.2.4 For windrow placement, the rocks should be placed in trenches excavated in properly compacted *soil* fill. Trenches should be approximately 5 feet wide and 4 feet deep in maximum dimension. The voids around and beneath rocks should be filled with approved granular soil having a Sand Equivalent of 30 or greater and should be compacted by flooding. Windrows may also be placed utilizing an "open-face" method in lieu of the trench procedure, however, this method should first be approved by the Consultant.

6.2.5 Windrows should generally be parallel to each other and may be placed either parallel to or perpendicular to the face of the slope depending on the site geometry. The minimum horizontal spacing for windrows shall be 12 feet center-to-center with a 5-foot stagger or offset from lower courses to next overlying course. The minimum vertical spacing between windrow courses shall be 2 feet from the top of a lower windrow to the bottom of the next higher windrow.

6.2.6 Rock placement, fill placement and flooding of approved granular soil in the windrows should be continuously observed by the Consultant.

6.3 *Rock* fills, as defined in Section 3.1.3, shall be placed by the Contractor in accordance with the following recommendations:

6.3.1 The base of the *rock* fill shall be placed on a sloping surface (minimum slope of 2 percent). The surface shall slope toward suitable subdrainage outlet facilities. The *rock* fills shall be provided with subdrains during construction so that a hydrostatic pressure buildup does not develop. The subdrains shall be permanently connected to controlled drainage facilities to control post-construction infiltration of water.

6.3.2 *Rock* fills shall be placed in lifts not exceeding 3 feet. Placement shall be by rock trucks traversing previously placed lifts and dumping at the edge of the currently placed lift. Spreading of the *rock* fill shall be by dozer to facilitate *seating* of the rock. The *rock* fill shall be watered heavily during placement. Watering shall consist of water trucks traversing in front of the current rock lift face and spraying water continuously during rock placement. Compaction equipment with compactive energy comparable to or greater than that of a 20-ton steel vibratory roller or other compaction equipment providing suitable energy to achieve the required compaction or deflection as recommended in Paragraph 6.3.3 shall be utilized. The number of passes to be made should be determined as described in Paragraph 6.3.3. Once a *rock* fill lift has been covered with *soil* fill, no additional *rock* fill lifts will be permitted over the *soil* fill.

6.3.3 Plate bearing tests, in accordance with ASTM D 1196, may be performed in both the compacted *soil* fill and in the *rock* fill to aid in determining the required minimum number of passes of the compaction equipment. If performed, a minimum of three plate bearing tests should be performed in the properly compacted *soil* fill (minimum relative compaction of 90 percent). Plate bearing tests shall then be performed on areas of *rock* fill having two passes, four passes and six passes of the compaction equipment, respectively. The number of passes required for the *rock* fill shall be determined by comparing the results of the plate bearing tests for the *soil* fill and the *rock* fill and by evaluating the deflection variation with number of passes. The required number of passes of the compaction equipment will be performed as necessary until the plate bearing deflections are equal to or less than that determined for the properly compacted *soil* fill. In no case will the required number of passes be less than two.

6.3.4 A representative of the Consultant should be present during *rock* fill operations to observe that the minimum number of "passes" have been obtained, that water is being properly applied and that specified procedures are being followed. The actual number of plate bearing tests will be determined by the Consultant during grading.

6.3.5 Test pits shall be excavated by the Contractor so that the Consultant can state that, in their opinion, sufficient water is present and that voids between large rocks are properly filled with smaller rock material. In-place density testing will not be required in the *rock* fills.

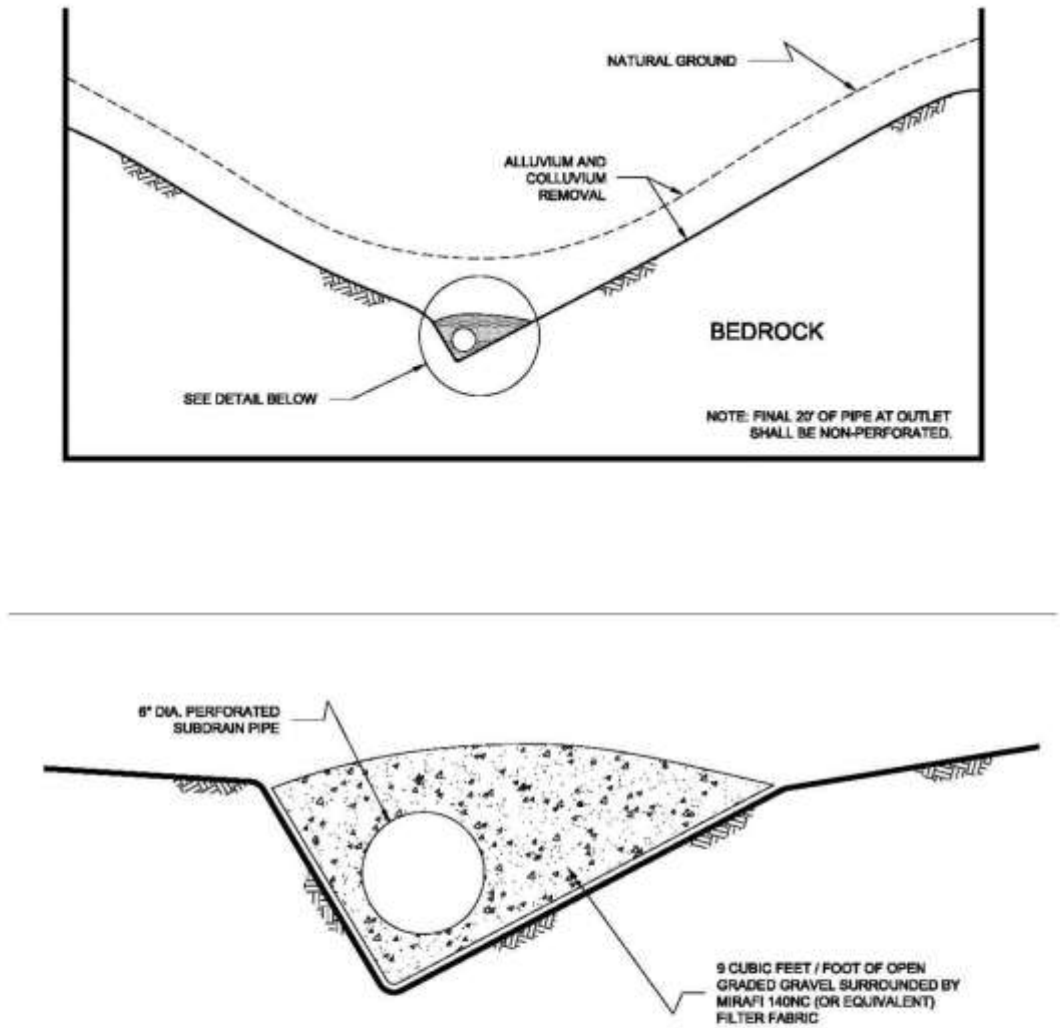
6.3.6 To reduce the potential for "piping" of fines into the *rock* fill from overlying *soil* fill material, a 2-foot layer of graded filter material shall be placed above the uppermost lift of *rock* fill. The need to place graded filter material below the *rock* should be determined by the Consultant prior to commencing grading. The gradation of the graded filter material will be determined at the time the *rock* fill is being excavated. Materials typical of the *rock* fill should be submitted to the Consultant in a timely manner, to allow design of the graded filter prior to the commencement of *rock* fill placement.

6.3.7 *Rock* fill placement should be continuously observed during placement by the Consultant.

7. SUBDRAINS

7.1 The geologic units on the site may have permeability characteristics and/or fracture systems that could be susceptible under certain conditions to seepage. The use of canyon subdrains may be necessary to mitigate the potential for adverse impacts associated with seepage conditions. Canyon subdrains with lengths in excess of 500 feet or extensions of existing offsite subdrains should use 8-inch-diameter pipes. Canyon subdrains less than 500 feet in length should use 6-inch-diameter pipes.

TYPICAL CANYON DRAIN DETAIL



NOTES:

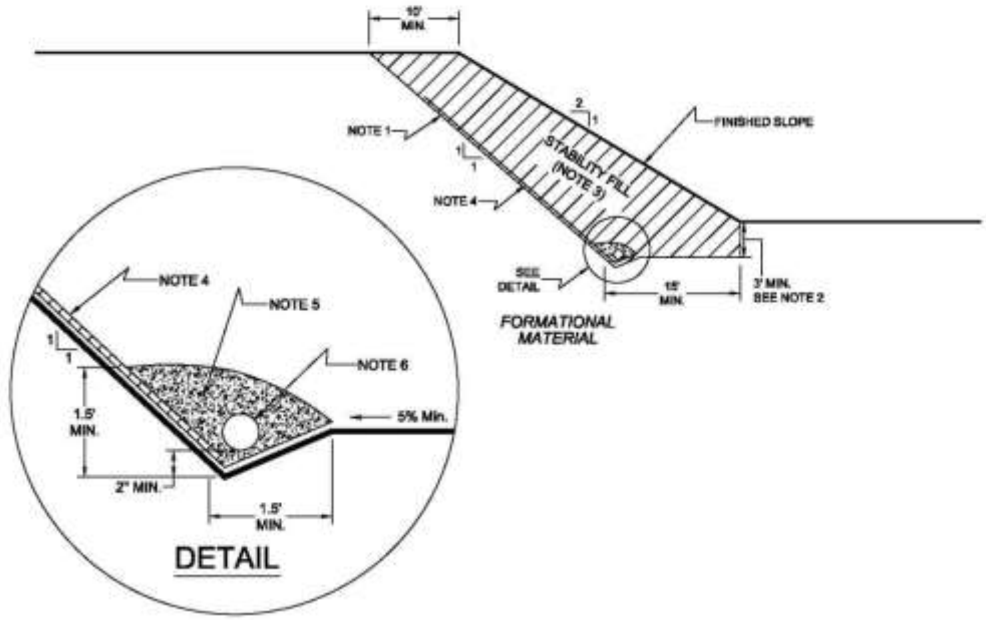
1.....6-INCH DIAMETER, SCHEDULE 80 PVC PERFORATED PIPE FOR FILLS IN EXCESS OF 100-FEET IN DEPTH OR A PIPE LENGTH OF LONGER THAN 500 FEET.

2.....6-INCH DIAMETER, SCHEDULE 40 PVC PERFORATED PIPE FOR FILLS LESS THAN 100-FEET IN DEPTH OR A PIPE LENGTH SHORTER THAN 500 FEET.

NO SCALE

7.2 Slope drains within stability fill keyways should use 4-inch-diameter (or lager) pipes.

TYPICAL STABILITY FILL DETAIL



NOTES:

1....EXCAVATE BACKCUT AT 1:1 INCLINATION (UNLESS OTHERWISE NOTED).

2....BASE OF STABILITY FILL TO BE 3 FEET INTO FORMATIONAL MATERIAL, SLOPING A MINIMUM 5% INTO SLOPE.

3....STABILITY FILL TO BE COMPOSED OF PROPERLY COMPACTED GRANULAR SOIL.

4....CHIMNEY DRAINS TO BE APPROVED PREFABRICATED CHIMNEY DRAIN PANELS (MIRADRAIN G200N OR EQUIVALENT) SPACED APPROXIMATELY 20 FEET CENTER TO CENTER AND 4 FEET WIDE. CLOSER SPACING MAY BE REQUIRED IF SEEPAGE IS ENCOUNTERED.

5....FILTER MATERIAL TO BE 3/4-INCH, OPEN-GRADED CRUSHED ROCK ENCLOSED IN APPROVED FILTER FABRIC (MIRAFI 140NC).

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.

NO SCALE

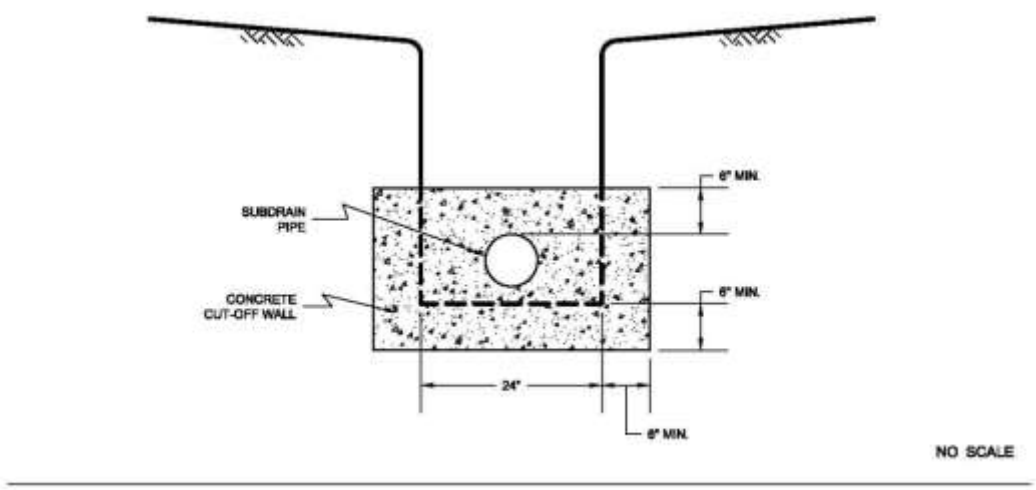
7.3 The actual subdrain locations will be evaluated in the field during the remedial grading operations. Additional drains may be necessary depending on the conditions observed and the requirements of the local regulatory agencies. Appropriate subdrain outlets should be evaluated prior to finalizing 40-scale grading plans.

7.4 *Rock* fill or *soil-rock* fill areas may require subdrains along their down-slope perimeters to mitigate the potential for buildup of water from construction or landscape irrigation. The subdrains should be at least 6-inch-diameter pipes encapsulated in gravel and filter fabric. *Rock* fill drains should be constructed using the same requirements as canyon subdrains.

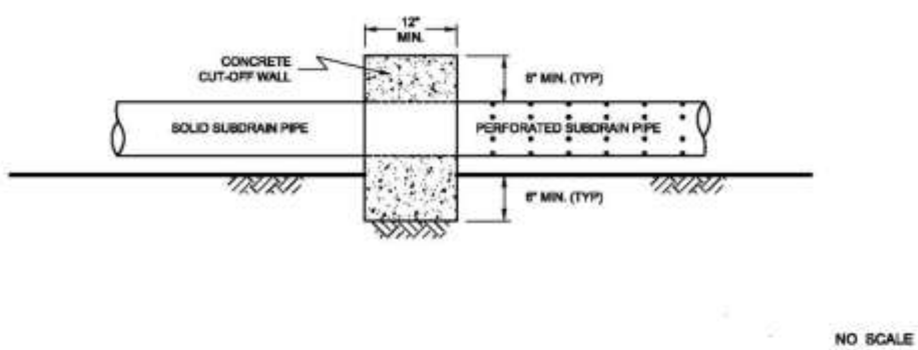
7.5 Prior to outletting, the final 20-foot segment of a subdrain that will not be extended during future development should consist of non-perforated drainpipe. At the non-perforated/ perforated interface, a seepage cutoff wall should be constructed on the downslope side of the pipe.

TYPICAL CUT OFF WALL DETAIL

FRONT VIEW

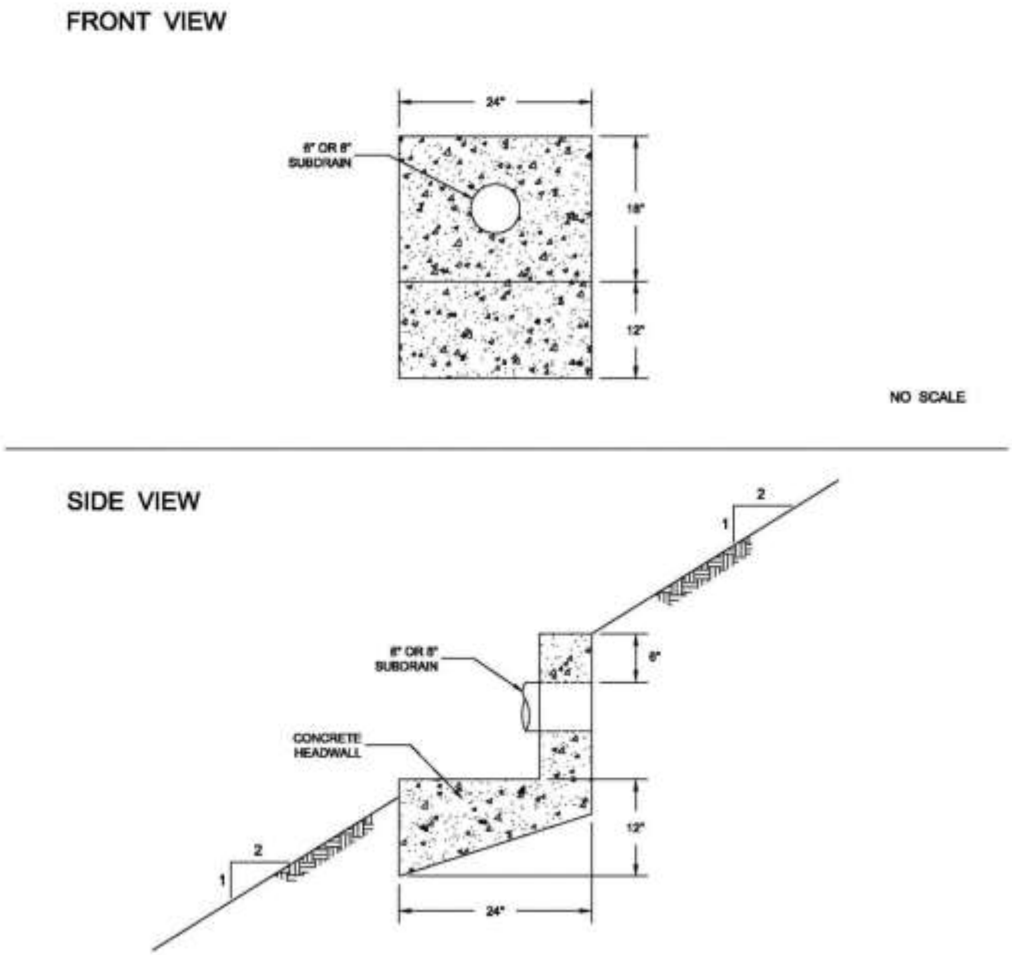


SIDE VIEW



7.6 Subdrains that discharge into a natural drainage course or open space area should be provided with a permanent headwall structure.

TYPICAL HEADWALL DETAIL



NOTE: HEADWALL SHOULD OUTLET AT TOE OF FILL SLOPE
OR INTO CONTROLLED SURFACE DRAINAGE

NO SCALE

7.7 The final grading plans should show the location of the proposed subdrains. After completion of remedial excavations and subdrain installation, the project civil engineer should survey the drain locations and prepare an “as-built” map showing the drain locations. The final outlet and connection locations should be determined during grading operations. Subdrains that will be extended on adjacent projects after grading can be placed on formational material and a vertical riser should be placed at the end of the subdrain. The grading contractor should consider videoing the subdrains shortly after burial to check proper installation and functionality. The contractor is responsible for the performance of the drains.

8. OBSERVATION AND TESTING

8.1 The Consultant shall be the Owner's representative to observe and perform tests during clearing, grubbing, filling, and compaction operations. In general, no more than 2 feet in vertical elevation of *soil* or *soil-rock* fill should be placed without at least one field density test being performed within that interval. In addition, a minimum of one field density test should be performed for every 2,000 cubic yards of *soil* or *soil-rock* fill placed and compacted.

8.2 The Consultant should perform a sufficient distribution of field density tests of the compacted *soil* or *soil-rock* fill to provide a basis for expressing an opinion whether the fill material is compacted as specified. Density tests shall be performed in the compacted materials below any disturbed surface. When these tests indicate that the density of any layer of fill or portion thereof is below that specified, the particular layer or areas represented by the test shall be reworked until the specified density has been achieved.

8.3 During placement of *rock* fill, the Consultant should observe that the minimum number of passes have been obtained per the criteria discussed in Section 6.3.3. The Consultant should request the excavation of observation pits and may perform plate bearing tests on the placed *rock* fills. The observation pits will be excavated to provide a basis for expressing an opinion as to whether the *rock* fill is properly seated and sufficient moisture has been applied to the material. When observations indicate that a layer of *rock* fill or any portion thereof is below that specified, the affected layer or area shall be reworked until the *rock* fill has been adequately seated and sufficient moisture applied.

8.4 A settlement monitoring program designed by the Consultant may be conducted in areas of *rock* fill placement. The specific design of the monitoring program shall be as recommended in the Conclusions and Recommendations section of the project Geotechnical Report or in the final report of testing and observation services performed during grading.

8.5 We should observe the placement of subdrains, to check that the drainage devices have been placed and constructed in substantial conformance with project specifications.

8.6 Testing procedures shall conform to the following Standards as appropriate:

8.6.1 Soil and Soil-Rock Fills:

8.6.1.1 Field Density Test, ASTM D 1556, *Density of Soil In-Place By the Sand-Cone Method.*

8.6.1.2 Field Density Test, Nuclear Method, ASTM D 6938, *Density of Soil and Soil-Aggregate In-Place by Nuclear Methods (Shallow Depth).*

8.6.1.3 Laboratory Compaction Test, ASTM D 1557, *Moisture-Density Relations of Soils and Soil-Aggregate Mixtures Using 10-Pound Hammer and 18-Inch Drop*.

8.6.1.4. Expansion Index Test, ASTM D 4829, *Expansion Index Test*.

9. PROTECTION OF WORK

9.1 During construction, the Contractor shall properly grade all excavated surfaces to provide positive drainage and prevent ponding of water. Drainage of surface water shall be controlled to avoid damage to adjoining properties or to finished work on the site. The Contractor shall take remedial measures to prevent erosion of freshly graded areas until such time as permanent drainage and erosion control features have been installed. Areas subjected to erosion or sedimentation shall be properly prepared in accordance with the Specifications prior to placing additional fill or structures.

9.2 After completion of grading as observed and tested by the Consultant, no further excavation or filling shall be conducted except in conjunction with the services of the Consultant.

10. CERTIFICATIONS AND FINAL REPORTS

10.1 Upon completion of the work, Contractor shall furnish Owner a certification by the Civil Engineer stating that the lots and/or building pads are graded to within 0.1 foot vertically of elevations shown on the grading plan and that all tops and toes of slopes are within 0.5 foot horizontally of the positions shown on the grading plans. After installation of a section of subdrain, the project Civil Engineer should survey its location and prepare an *as-built* plan of the subdrain location. The project Civil Engineer should verify the proper outlet for the subdrains and the Contractor should ensure that the drain system is free of obstructions.

10.2 The Owner is responsible for furnishing a final as-graded soil and geologic report satisfactory to the appropriate governing or accepting agencies. The as-graded report should be prepared and signed by a California licensed Civil Engineer experienced in geotechnical engineering and by a California Certified Engineering Geologist, indicating that the geotechnical aspects of the grading were performed in substantial conformance with the Specifications or approved changes to the Specifications.

LIST OF REFERENCES

1. Boore, D. M., and G. M Atkinson (2008), *Ground-Motion Prediction for the Average Horizontal Component of PGA, PGV, and 5%-Damped PSA at Spectral Periods Between 0.01 and 10.0 S*, Earthquake Spectra, Volume 24, Issue 1, pages 99-138, February 2008.

2. California Department of Conservation, Division of Mines and Geology, *Probabilistic Seismic Hazard Assessment for the State of California,* Open File Report 96-08, 1996.

3. California Geological Survey, *Seismic Shaking Hazards in California,* Based on the USGS/CGS Probabilistic Seismic Hazards Assessment (PSHA) Model, 2002 (revised April 2003). 10% probability of being exceeded in 50 years.
http://redirect.conservation.ca.gov/cgs/rghm/pshamap/pshamain.html

4. Campbell, K. W. and Y. Bozorgnia, *NGA Ground Motion Model for the Geometric Mean Horizontal Component of PGA, PGV, PGD and 5% Damped Linear Elastic Response Spectra for Periods Ranging from 0.01 to 10 s*, Preprint of version submitted for publication in the NGA Special Volume of Earthquake Spectra, Volume 24, Issue 1, pages 139-171, February 2008.

5. Chiou, Brian S. J. and Robert R. Youngs, *A NGA Model for the Average Horizontal Component of Peak Ground Motion and Response Spectra*, preprint for article to be published in NGA Special Edition for Earthquake Spectra, Spring 2008.

6. City of San Diego, *Seismic Safety Study, Geologic Hazards and Faults,* 2008 edition, Map Sheets 43 and 44.

7. *Fault Activity Map of California and Adjacent Areas,* California Division of Mines and Geology, compiled by C. W. Jennings, 1994.

8. Geocon Incorporated, *Final Report of Testing and Observation Services Performed During Site Grading, Heritage Bluffs II, I.O. No. 24005722, PTS No. 416489, Drawing No. 37825, San Diego, California,* prepared by Geocon Incorporated, dated December 15, 2017 (Project No. 07339-32-05).

9. *http://www.water.ca.gov.*

10. *http://websoilsurvey.nrcs.usda.gov.*

11. *http://geotracker.waterboards.ca.gov.*

12. Jennings, C. W., 1994, California Division of Mines and Geology, *Fault Activity Map of California and Adjacent Areas,* California Geologic Data Map Series Map No. 6.

13. Kennedy, M. P. and S. S. Tan, *Geologic Map of the San Diego 30'x60' Quadrangle, California*, USGS Regional Map Series, Scale 1:100,000, 2005.

14. Risk Engineering (2015), *EZ-FRISK (version 7.65).*

15. United States Department of Agriculture, *1953 Stereoscopic Aerial Photographs, Flight AXN-3M,* Photos Nos. 112 and 113 (scale 1:20,000).

LIST OF REFERENCES (Concluded)

16. Unpublished reports and maps on file with Geocon Incorporated.

17. USGS computer program, *Seismic Hazard Curves and Uniform Hazard Response Spectra.*