Appendix E-1

Geotechnical Report

GEOCON
INCORPORATED

GEOTECHNICAL
ENVIRONMENTAL
MATERIALS

GEOTECHNICAL INVESTIGATION

PASEO MONTRIL
SAN DIEGO, CALIFORNIA

PREPARED FOR

PARDEE HOMES
SAN DIEGO, CALIFORNIA

JANUARY 5, 2018
PROJECT NO. G2209-42-01

GEOCON
INCORPORATED

GEOTECHNICAL ■ ENVIRONMENTAL ■ MATERIALS

Project No. G2209-42-01
January 5, 2018

Pardee Homes
13400 Sabre Springs Parkway, Suite 200
San Diego, California 92128

Attention: Mr. Allen Kashani

Subject: GEOTECHNICAL INVESTIGATION
PASEO MONTRIL
SAN DIEGO, CALIFORNIA

Dear Mr. Kashani:

In accordance with your request, we have performed a geotechnical investigation for the subject project. The accompanying report presents the findings of our study with our conclusions and recommendations pertaining to geotechnical aspects of developing the property as proposed. Based on the results of our investigation, it is our opinion that the site can be developed as proposed provided the recommendations of this report are followed.

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

Very truly yours,

GEOCON INCORPORATED

Rodney C. Mikesell
GE 2533

Garry W. Cannon
CEG 2201
RCE 56468

(e-mail) Addressee
(3/del) Civil Sense, Inc.
Attention: Mr. Inh Ling

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GEOTECHNICAL INVESTIGATION

1. PURPOSE AND SCOPE

This report presents the results of our geotechnical investigation for the proposed Paseo Montril project located in San Diego, California (see Vicinity Map, Figure 1). The purpose of the investigation is to provide an evaluation of subsurface soil and geologic conditions at the site and, based on the conditions encountered, provide recommendations pertaining to the geotechnical aspects of developing the property. The area of planned development, as presently proposed, is presented on the Geologic Map, Figure 2.

The scope of our investigation included geologic mapping; subsurface exploration; laboratory testing; engineering analyses; and the preparation of this report. As a part of our investigation, we have reviewed published geologic maps and geologic reports related to the property and surrounding site area. A summary of the background information reviewed for this study is presented in the *List of References*.

The field investigation included geologic mapping, excavating four test pits, and drilling six, air-percussion borings. A discussion of the field investigation and logs of the trenches and borings are presented in Appendix A. The approximate locations of the exploratory trenches and borings are presented on the Geologic Map (Figure 2). We performed laboratory tests on soil samples obtained from the exploratory excavations to evaluate pertinent physical and chemical properties for engineering analysis. The results of the laboratory testing are presented in Appendix B.

Civil Sense, Inc. provided the topographic information and the site plan used during the field investigation and preparation of the Geologic Map. References to elevations presented in this report are based on the referenced topographic information. Geocon does not practice in the field of land surveying and is not responsible for the accuracy of such topographic information.

2. SITE AND PROJECT DESCRIPTION

The project is located east of the terminus of Paseo Montril and west of Interstate 15 in San Diego, California (see Vicinity Map, Figure 1). The property to be graded is approximately 4.5 acres and consists of a natural hillside covered by coastal sage scrub and non-native grass. Site elevations across the area to be graded range from approximately 580 feet above mean sea level (MSL) at the northwest corner to approximately 440 feet MSL at the southwest corner. Residential homes lie north of the site. A commercial center exists west of the property.

We understand that the property will be graded to construct 10 multi-family apartment buildings and a recreation center. A paved access road with parking stalls is planned along the perimeter of the site.

Grading will result in cuts up to 60 feet within the central and northern portions of the site, and fills up to 30 feet in the southwest corner and along the eastern edge. Retaining walls with heights ranging from less than 5 feet to 30 feet are planned along the site perimeter. The walls in the cut area will likely be soil nail walls or concrete walls. Walls in the fill areas will likely be concrete masonry unit (CMU), concrete, or mechanically stabilized earth (MSE) walls. A 1:5:1 (horizontal to vertical) cut slope will be made above the retaining wall at the north end of the property. Fill slopes with an inclination of 2:1 are planned at the southwest corner and east side of the site. We understand underground storage vaults are planned for storm water management.

The locations and descriptions provided herein are based on a site reconnaissance, review of the site plan, and project information provided by Civil Sense, Inc.

3. GEOLOGIC SETTING

The site is located in the Peninsular Ranges geomorphic province of Southern California. The Peninsular Ranges extend from Imperial Valley to the Pacific Ocean and from the Transverse Ranges into Baja California. The Peninsular Ranges are generally composed of Cretaceous age granitic rock intruded into older metamorphic rock. The Peninsular Ranges are dissected by the Elsinore Fault Zone that is associated with and sub-parallel to the San Andreas Fault Zone.

4. SOIL AND GEOLOGIC CONDITIONS

Based on our field investigation, geologic mapping, and published geologic maps, the site is underlain by surficial deposits consisting of undocumented fill, topsoil and weathered Mesozoic age metamorphic rock. The estimated lateral extent of the geologic units within the project boundary is shown on the Geologic Map and Cross Sections (see Figures 2 and 3) descriptions of the soil and geologic conditions are shown on the trench logs located in Appendix A and described herein.

4.1 Undocumented Fill (Qudf)

Undocumented fill was encountered in Trench T-1 and mapped along the western edge of the property. The undocumented fill was found to be approximately 4 feet thick near Trench T-1. We expect the undocumented fill could be up to 10 feet thick in the southwest corner. The undocumented fill is potentially compressible and should be removed and replaced as compacted fill.

4.2 Topsoil (Unmapped)

Topsoils blanket the majority of the site and vary in thickness from approximately 1 to 3 feet. The topsoils are characterized as stiff, dry to moist, sandy clay. 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 topsoil exhibits a high expansion potential and should be placed in deeper fill areas.

4.3 Weathered Metamorphic Rock(Unmapped)

Deeply weathered metamorphic rock was encountered within the southwestern portion of the property. The weathered soils were found to depths of 8 feet and greater than 17 feet below the ground surface in trenches T-1 and T-2. The soils were found to be predominately lean to fat clay. Laboratory expansion index tests indicate the weathered soils are highly expansive. The weathered soils should be removed and replaced as compacted fill. The actual depth of required removals will be determined during grading, however, for budgetary purposes, complete removal and recompaction should be planned. The weathered soils are also sufficiently clayey and expansive that use of the soils is not recommended within the outer 15 feet of fill slopes, upper 5 feet of finish grade, or as backfill for retaining walls.

4.4 Undifferentiated Metamorphic Rock (Mzu)

Mesozoic-age Undifferentiated Metamorphic Rock is the underlying bedrock unit and is exposed at grade on the northern hillside and underlies the undocumented fill, topsoil, and the weathered metamorphic rock. This unit varies greatly in degree of weathering from highly weathered rippable materials to fresh, hard, non-rippable rock. Metamorphic rock is suitable for support of settlement sensitive structures and improvements.

To evaluate excavation and rippability characteristics, 6 air- percussion borings were performed in the northern cut area. The locations of air-percussion borings are shown on Figure 2. A discussion of rock rippability is provided below. Excavations into the metamorphic rock will require specialized rock breaking techniques and blasting to effectively excavate. It should be anticipated that excavations within this unit will generate boulders and oversize materials (rocks greater than 12 inches in dimension) that will require special handling and placement within structural fills.

5. RIPPABILITY AND ROCK CONSIDERATIONS

To aid in evaluating the rippability characteristics of the rock in proposed cut areas, 6 air-percussion borings were performed using an Ingersoll Rand ECM 370 equipped with a 4-inch bit. Drill penetration rates were used to evaluate rock rippability and to estimate the depth at which difficult excavation will occur. 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.

A frequently used guideline to compare rock rippability to drill penetration rate is that a penetration rate of approximately 0 to 20 seconds per foot (spf) generally indicates rippable material, 20 to 30 spf indicates marginally to non-rippable material, and greater than 30 spf indicates non-rippable rock. These general guidelines are typically based on drill rates using a rotary percussion drill rig similar to an Ingersoll Rand ECM 360 with a 3½-inch drill bit. The penetration rates (recorded in seconds per foot) for each air-track boring are presented in Appendix A.

The estimated thickness of rippable material for each air-track boring using 20 spf as the boundary between rippable and marginal to non-rippable rock is presented on the *Geologic Map*. The estimate is derived from a literal interpretation of the penetration rate from each boring log, based on the first occurrence where the penetration rate reaches 20 spf. Perspective contractors should use their own judgment to identify the penetration rate boundary between productive and non-productive ripping, and rippable and non-rippable rock.

Based on the discussion above and review of the subsurface information, it is expected that the majority of excavations within the development will experience very difficult ripping and/or blasting as excavations are extended beyond the rippable weathered mantle. Based on an air-track penetration rate of 20 spf, the thickness of the rippable rock mantle varies between 1 to 15 feet thick. 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 from each boring and drill penetration rate criteria acceptable to the contractor. Perspective contractors should evaluate the air-track and seismic refraction data and use their own judgment to identify the boundary between productive and non-productive ripping, and rippable and non-rippable rock. Roadway/utility corridors and lot undercutting criteria should also be considered when calculating the volume of hard rock. Proposed cuts in hard rock areas can be expected to generate oversized fragments.

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 D of this report and governing jurisdictions. Crushing of oversize materials may be necessary to satisfy the placement requirements of this report.

6. SOIL CAPPING AND WALL BACKFILL CONSIDERATIONS

Based on our field investigation, we expect topsoil and weathered metamorphic rock to be highly expansive and not suitable for use as capping or wall backfill. It is our opinion that soil cap and wall backfill will need to be imported to the site. Alternatively, rock crushing can be utilized to produce sufficient soil cap and wall backfill materials. If MSE type retaining walls will be utilized, the crushed product should meet wall designer specifications. Typically, MSE wall designers do not allow the use of angular rock within the backfill soil due to the potential for damage to the reinforcing grid. We expect most crushed products will be suitable for use behind conventional CMU or concrete type retaining walls. All backfill behind retaining walls should have an expansion index (EI) of 50 or less.

Capping material should be at least five feet thick within building pads and 3 feet within paved roadways. The capping material should consist of soil fill with an approximate maximum particle dimension of 6 inches with a minimum of 40 percent soil passing the ¾-inch sieve and should have at least 20 percent of the soil passing the No. 4 screen. Soils with an expansion potential (EI) of greater than 50 are not suitable for capping and should be placed in the deeper fill areas or at least 5 feet below design grade across the site and 15 feet from face of slopes. The grading contractor should take necessary steps to manage the available soils to cap the project.

7. GROUNDWATER

We did not encounter groundwater during our field investigation. Groundwater is not expected to adversely impact proposed project development. However, the Metamorphic rock has permeability characteristics and fracture systems that are conducive to water migration (natural or artificially induced by irrigation) that may result in seepage where none previously occurred. Surface drainage as well as implementation of a landscape irrigation-monitoring program can reduce this potential.

8. GEOLOGIC HAZARDS

8.1 Geologic Hazard Category

Based on the City of San Diego 2008 Seismic Safety Study, the site is located in Hazard Category 53 which is *Level or sloping terrain, unfavorable geologic structure, low to moderate risk*. It is our opinion, provided the recommendations of this report are followed, that the site will have a low risk to geologic hazards at the completion of grading.

8.2 Ground Rupture

No evidence of faulting was observed during our investigation. The USGS Fold and Fault database (USGS, 2016) shows that there are no mapped Quaternary faults crossing or trending toward the property. The site is not located within a currently established Alquist-Priolo Earthquake Fault Zone. The risk associated with ground rupture hazard due to earthquake faulting is low.

8.3 Seismicity

We performed a deterministic seismic hazard analysis using Risk Engineering (2015). Seven known active faults are located within a search radius of 50 miles from the property. We used the 2008 USGS fault database that provides several models and combinations of fault data to evaluate the fault information. Based on this database, the Newport-Inglewood/Rose Canyon and Rose Canyon Fault Zones, located approximately 11 miles west of the site, are the nearest known active faults and are the dominant source of potential ground motion. Earthquakes that might occur on the Newport-Inglewood/Rose Canyon and Rose Canyon Fault Zones or other faults within the southern California and northern Baja California area are potential generators of significant ground motion at the site. The estimated maximum earthquake magnitude and peak ground acceleration for the Newport-Inglewood/Rose Canyon Fault are 7.5 and 0.24g, respectively. Table 8.3.1 lists the estimated maximum earthquake magnitude and peak ground acceleration for the most dominant faults in relation to the site location. We calculated peak ground acceleration (PGA) using Boore-Atkinson (2008) NGA USGS2008, Campbell-Bozorgnia (2008) NGA USGS 2008, and Chiou-Youngs (2008) NGA acceleration-attenuation relationships.

TABLE 8.3.1
DETERMINISTIC SPECTRA 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/Rose Canyon	11	7.5	0.23	0.19	0.24
Rose Canyon	11	6.9	0.19	0.17	0.18
Coronado Bank	25	7.4	0.13	0.10	0.11
Palos Verdes/Coronado Bank	25	7.7	0.15	0.11	0.13
Elsinore	27	7.85	0.15	0.11	0.14
Earthquake Valley	34	6.8	0.08	0.06	0.05
San Jacinto	48	7.88	0.09	0.07	0.08

In the event of a major earthquake on the referenced faults or other significant faults in the southern California and northern Baja California area, the site could be subjected to moderate to severe ground shaking. With respect to this hazard, the site is considered comparable to others in the general vicinity.

We performed a probabilistic seismic hazard analysis for the site using Risk Engineering (2015). 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 mapped Quaternary fault is proportional to the faults slip rate. The program accounts for earthquake magnitude as a function of fault rupture length, 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), Campbell-Bozorgnia (2008) and Chiou-Youngs (2008) in the analysis. Table 8.3.2 presents the site-specific probabilistic seismic hazard parameters including acceleration-attenuation relationships and the probability of exceedence.

TABLE 8.3.2
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	0.36	0.35	0.39
5% in a 50 Year Period	0.27	0.26	0.27
10% in a 50 Year Period	0.21	0.20	0.20

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 frequency and duration of motion and the soil conditions underlying the site. Seismic design of the structures should be performed in accordance with the 2016 California Building Code (CBC) guidelines currently adopted by the County of San Diego.

8.4 Liquefaction

Due to the dense underlying bedrock soils and the lack of near surface groundwater, the risk associated with liquefaction is low.

8.5 Landslides

Our geologic reconnaissance and review of available geotechnical and geologic reports for the site vicinity indicate that landslides are not present at the property or at a location that could impact the site. The risk associated with landsliding hazard is low.

8.6 Tsunamis and Seiches

The site is approximately 9 miles from the Pacific Ocean at an approximate site elevation between 440 to 580 feet above MSL. The risk associated with inundation hazard due to tsunamis is very low.

The site is no located down stream of any large bodies or water or reservoirs. The risk associated with inundation hazard due to seiche is very low.

8.7 Flooding

Our review of FEMA (2012) shows that the site is not located within a FEMA designated 100-year Flood Zone. The risk associated with flooding is low.

9. CONCLUSIONS AND RECOMMENDATIONS

9.1 General

9.1.1 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.

9.1.2 The site is underlain by compressible surficial soil deposits consisting of undocumented fill, topsoil and weathered metamorphic rock. Surficial soils will require remedial grading in the form of removal and recompaction. The surficial soils are also highly expansive and will require placement in deeper fill areas, away from slope faces, and outside of retaining wall backfill zones.

9.1.3 Mesozoic-age metamorphic rock underlies the surficial soil deposits and is exposed at grade in the northwestern hillside area of the property. This geologic unit is suitable for support of planned improvements and compacted fills.

9.1.4 With the exception of possible strong seismic shaking, no significant geologic hazards were observed or are known to exist that could adversely affect the proposed project.

9.1.5 The presence of hard rock within proposed cut areas will require special consideration during site development. Based on our study, the majority of the proposed excavation will encounter heavy ripping conditions with conventional heavy-duty equipment and blasting to achieve finish grade. 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 Appendix D of this report.

9.1.6 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, rippability information contained in this report, rock placement requirements and include proposed undercutting of pads and streets. Consideration should be given to stockpiling select materials to be utilized for capping.

9.1.7 Based on our field investigation, we expect topsoil and weathered metamorphic rock to be highly expansive and not suitable for use as capping or wall backfill. Due to the lack of available on-site suitable soil for soil cap and wall backfill, it is our opinion that select import fill will need to be imported to the site. Alternatively, rock crushing can be utilized to produce soil cap and wall backfill materials. Specifications for soil cap and wall backfill is provided in the Grading and Retaining Wall sections of this report.

9.1.8 Cut slopes should be observed during grading by an engineering geologist to verify that the soil and geologic conditions do not differ significantly from those anticipated. Scaling of loose rock fragments from proposed cut slopes may also be necessary.

9.2 Soil and Excavation Characteristics

9.2.1 Excavation of the surficial deposits (undocumented fill, topsoil, and weathered metamorphic rock should generally require moderate to heavy effort using conventional heavy-duty grading equipment.

9.2.2 Excavating within the rock materials will generally vary in difficulty with the depth of excavation depending. Blasting will likely be required for depths below approximately 10 feet in rock cut areas. 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 D). Oversize rock may require breakage to acceptable sizes or exportation from the property. Placement of oversize rock within the area of proposed underground utilities should not be permitted.

9.2.3 The soil encountered in the field investigation is considered to be expansive (expansion index greater than 20 as defined by 2016 California Building Code (CBC) Section 1803.5.3. Table 9.2 presents soil classifications based on the expansion index.

TABLE 9.2
EXPANSION CLASSIFICATION BASED ON EXPANSION INDEX

Expansion Index (EI)	ASTM D 4829 Expansion Classification	2016 CBC Expansion Classification
0 – 20	Very Low	Non-Expansive
21 – 50	Low	Expansive
51 – 90	Medium	
91 – 130	High	
Greater Than 130	Very High	

9.2.4 On-site topsoil and weathered metamorphic rock consist predominately of fine grained clays. These materials have a high expansion potential. These soils are not expected to be suitable for capping or use as wall backfill and will require placement within deeper fill areas and away from slope faces.

9.3 Corrosion

9.3.1 We performed laboratory tests on samples of the site materials to evaluate the percentage of water-soluble sulfate content. Appendix B presents results of the laboratory water-soluble sulfate content tests. The test results indicate the on-site materials at the locations tested possess "S0" sulfate exposure to concrete structures as defined by 2016 CBC Section 1904 and ACI 318-14 Chapter 19. Table 9.3 presents a summary of concrete requirements set forth by 2016 CBC Section 1904 and ACI 318. 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 9.3
REQUIREMENTS FOR CONCRETE EXPOSED TO SULFATE-CONTAINING SOLUTIONS

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)
S0	SO_4<0.10	No Type Restriction	n/a	2,500
S1	0.10≤SO_4<0.20	II	0.50	4,000
S2	0.20≤SO_4≤2.00	V	0.45	4,500
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

9.3.2 Geocon Incorporated does not practice in the field of corrosion engineering; therefore, further evaluation by a corrosion engineer may be needed to incorporate the necessary precautions to avoid premature corrosion of underground pipes and buried metal in direct contact with the soils.

9.4 Slopes

9.4.1 Slope stability analyses were performed utilizing assumed shear strength parameters for low expansive compacted fill assuming imported soils. These analyses indicate that the proposed 2:1 fill slopes, constructed of soils that have a friction angle of at least 30 degrees and cohesion of 100 pounds per square foot (psf), should have calculated factor of safety of at least 1.5 under static conditions for both deep-seated failure and shallow sloughing conditions to proposed maximum project fill slope height of 50 feet. Slope stability calculations and graphical printouts for both deep-seated and surficial slope stability are presented on Figures 4 and 5.

9.4.2 Cut slopes in rock materials do not lend themselves to conventional slope stability analyses. However, Figure 6 summarizes a slope stability analysis assuming soil shear strength parameters for the rock and modeling assumed soil nails for the retaining wall. The strength parameters used are considered conservative for Metamorphic Rock. Based on our analysis and experience with similar rock conditions, 1.5:1 cut slopes to the planned heights of up to 80 feet (including the vertical wall) should possess a factor of safety of at least 1.5 with respect to global stability, if free of adversely oriented joints or fractures.

9.4.3 All cut slope excavations should be observed during grading by an engineering geologist to check that soil and geologic conditions do not differ significantly from those anticipated. In the event that adverse conditions are observed during grading such as intersecting faults planes or clay filled joints/fractures dipping out of slope, stabilization recommendations can be provided. Possible mitigation techniques such as tie-back anchors/rock bolts, rock blankets, geogrid reinforced embankments, or reducing the slope inclination may be utilized to improve the local stability of the slope. We anticipate that these remedial alternatives could be implemented within the development limits. We have observed and evaluated similar 1.5:1 (horizontal:vertical) slopes in metamorphic rock on other projects which did not require mitigation.

9.4.4 The outer 15 feet of fill slopes, measure horizontal to the slope face, should be composed of properly compacted granular "soil" fill (expansion index of 50 or less) to reduce the potential for surface sloughing.

9.4.5 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.

9.4.6 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.

9.5 Subdrains

9.5.1 If rock fill is utilized on the project, subdrains may be required along the perimeter of the rock fill and at toe of slopes (see Figure 8). The need for subdrains can be determined by Geocon during grading based on the type of material that will be utilized for fill. Subdrains are also required for retaining walls.

9.6 Grading

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

9.6.2 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.

9.6.3 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.

9.6.4 All compressible soil deposits, including undocumented fill, topsoil, and weathered metamorphic rock within areas where structural improvements and/or structural fill are planned, should be removed to expose firm competent Metamorphic Rock 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.

9.6.5. Removals at the toe of proposed fill slopes should extend horizontally beyond the edge of improvements a distance equal to the depth of removal. A typical detail of remedial grading beyond proposed grading is presented in Figure 7.

9.6.6 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.

9.6.7 Grading operations should be scheduled to permit the placement of oversize material and expansive soils in deeper fill areas and to cap building pads with granular materials having a “very low” to “low” expansive potential (EI of 50 or less).

9.6.8 Where practical, the upper 5 feet of all building pads (cut or fill) should be comprised of soil with a “very low” to “low” expansion potential. Highly expansive fill soils should be placed in the deeper fill areas. Cobbles, rock fragments, and concretions greater than 6 inches in maximum dimension should not be placed within 3 feet of finish grade in building pad areas.

9.6.9 Cut pads exposing rock and cut/fill transition building pads should be undercut at least 5 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.

9.6.10 Undercutting of street areas and utilities should be performed in cut areas or areas where utilities will extend through the fill into the Metamorphic Rock to facilitate excavation of underground utilities in areas of hard rock. If subsurface improvements or landscape zones are planned outside these areas, consideration should be given to undercutting these areas as well.

9.6.11 Oversize material (defined as material greater than 12 inches in nominal dimension) will be generated during ripping and blasting of Metamorphic rock. Placement of oversize material within fills should be conducted in accordance with the recommendations in Appendix D and the oversize rock disposal detail (Figure 8). Grading operations on the site should be scheduled such that oversize materials are placed in deeper fills and at least 10 feet below finish pad grade and 2 feet below the deepest utilities.

9.6.12 Capping material should be at least five feet thick. The capping material should consist of soil fill with an approximate maximum particle dimension of 6 inches with a minimum of 40 percent soil passing the ¾-inch sieve and should have at least 20 percent of the soil passing the No. 4 screen. Soils with an expansion potential (EI) greater than 50 are not suitable for capping and should be placed in the deeper fill areas or at least 5 feet below design grade and 15 feet from face of slopes. The grading contractor should take necessary steps to manage the available soils to cap the project.

9.6.13 Based on our field investigation, we do not expect the on-site surficial soils will be suitable for capping and use as wall backfill. Import fill will be required. As an alternative, or in conjunction with importing soil, rock crushing can be considered to produce sufficient soil cap and wall backfill materials. If MSE type retaining walls will be utilized, the crushed product should meet wall designer specifications. Typically, MSE wall designers do not allow the use of angular rock within the backfill soil due to the potential for damage to the reinforcing grid. We expect most crushed products will be suitable for use behind conventional CMU or concrete type retaining walls. All backfill behind retaining walls should have an expansion index (EI) of 50 or less.

9.6.14 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.

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

9.6.16 Imported materials 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 5 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.

9.7 Settlement Monitoring

9.7.1 Settlement monuments are not required.

9.8 Earthwork Grading Factors

9.8.1 Estimates of embankment shrink-swell factors are based on comparing laboratory compaction tests with the density of the material in its natural state and experience with similar soil and rock types. It should be emphasized that variations in natural soil density, as well as in compacted fill, render shrinkage value estimates very approximate. As an example, the contractor can compact fills to any relative compaction of 90 percent or higher of the laboratory maximum dry density. Thus, the contractor has at least a 10 percent range of control over the fill volume. Based on the work performed to date and considering the above discussion, the following earthwork factors may be used as a basis for estimating how much the on-site soils may shrink or swell when removed from their natural state and placed in compacted fills.

TABLE 9.8
ESTIMATED BULK AND SHRINK VALUES

Soils Unit	Shrink-Swell Factors
Undocumented Fill and Topsoil	5 to 10 Percent Shrink
Weathered Metamorphic Rock	0 to 5 percent Shrink
Metamorphic Rock	20 to 25 percent bulk

9.9 Seismic Design Criteria

9.9.1 We used the computer program *U.S. Seismic Design Maps*, provided by the USGS. Table 9.9.1 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 seconds. The values presented in Table 9.9.1 are for the risk-targeted maximum considered earthquake (MCE_R). Site Class C should be used for building pads underlain by compacted fills less 15 feet thick or less. Site Class D should be used for building pads underlain by compacted fill in excess of 15 feet. 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.

TABLE 9.9.1
2016 CBC SEISMIC DESIGN PARAMETERS

Parameter	Value		2016 CBC Reference
Site Class	C	D	Section 1613.3.2
Fill Thickness, T (feet)	T≤15	T>15	--
Spectral Response – Class B (short), S_S	0.097 g	0.097 g	Figure 1613.3.1(1)
Spectral Response – Class B (1 sec), S_1	0.355 g	0.355 g	Figure 1613.3.1(2)
Site Coefficient, F_a	1.037	1.137	Table 1613.3.3(1)
Site Coefficient, F_v	1.445	1.690	Table 1613.3.3(2)
Maximum Considered Earthquake Spectral Response Acceleration (short), S_{MS}	0.941 g	1.031 g	Section 1613.3.3 (Eqn 16-37)
Maximum Considered Earthquake Spectral Response Acceleration – (1 sec), S_{M1}	0.513 g	0.600 g	Section 1613.3.3 (Eqn 16-38)
5% Damped Design Spectral Response Acceleration (short), S_{DS}	0.627 g	0.688 g	Section 1613.3.4 (Eqn 16-39)
5% Damped Design Spectral Response Acceleration (1 sec), S_{D1}	0.342 g	0.400 g	Section 1613.3.4 (Eqn 16-40)

9.9.2 Table 9.9.2 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 9.9.2
2016 CBC SITE ACCELERATION PARAMETERS

Parameter	Value		ASCE 7-10 Reference
Site Class	C	D	--
Mapped MCE_G Peak Ground Acceleration, PGA	0.342 g	0.342 g	Figure 22-7
Site Coefficient, F_{PGA}	1.058	1.158	Table 11.8-1
Site Class Modified MCE_G Peak Ground Acceleration, PGA_M	0.362 g	0.396 g	Section 11.8.3 (Eqn 11.8-1)

9.9.3 Conformance to the criteria for seismic design does not constitute any guarantee or assurance that significant structural damage or ground failure will not occur in the event of a maximum level earthquake. The primary goal of seismic design is to protect life and not to avoid all damage, since such design may be economically prohibitive.

9.10 Foundation and Concrete Slab-On-Grade Recommendations

9.10.1 The foundation recommendations herein 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 9.10.1.

TABLE 9.10.1
FOUNDATION CATEGORY CRITERIA

Foundation Category	Maximum Fill Thickness, T (feet)	Differential Fill Thickness, D (feet)	Expansion Index (EI)
I	$T<20$	--	$EI\leq 50$
II	$20\leq T<50$	$10\leq D<20$	$50<EI\leq 90$
III	$T\geq 50$	$D\geq 20$	$90<EI\leq 130$

9.10.2 We will provide final foundation categories for each building after finish pad grades have been achieved and we perform laboratory testing of the subgrade soil.

9.10.3 Table 9.10.2 presents minimum foundation and interior concrete slab design criteria for conventional foundation systems.

TABLE 9.10.2
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

9.10.4 The embedment depths presented in Table 9.10.2 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 foundation dimension detail is provided on Figure 9.

9.10.5 The concrete slab-on-grade should be a minimum of 4 inches thick for Foundation Categories I and II and 5 inches thick for Foundation Category III.

9.10.6 A vapor retarder should underlie slabs that may receive moisture-sensitive floor coverings or may be used to store moisture-sensitive materials. The vapor retarder design should be consistent with the guidelines presented in the American Concrete Institute's (ACI) *Guide for Concrete Slabs that Receive Moisture-Sensitive Flooring Materials* (ACI 302.2R-06). The project architect or developer should specify the vapor retarder to be used based on the type of floor covering that will be installed and if the structure will possess a humidity- controlled environment.

9.10.7 The project foundation engineer, architect, and/or developer should determine the slab bedding sand thickness. We should be contacted to provide recommendations if the bedding sand is thicker than 6 inches.

9.10.8 The foundation design engineer should provide appropriate concrete mix design criteria and curing measures to assure proper curing of the slab by reducing the potential for rapid moisture loss and subsequent cracking and/or slab curl. We suggest that the foundation design engineer present the concrete mix design and proper curing methods on the foundation plans. It is critical that the foundation contractor understands and follows the recommendations presented on the foundation plans.

9.10.9 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 9.10.3 for the particular Foundation Category designated. The parameters presented in Table 9.10.3 are based on the guidelines presented in the PTI DC 10.5 design manual.

TABLE 9.10.3
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

9.10.10 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.

9.10.11 If the structural engineer proposes a post-tensioned foundation design method other than PTI DC 10.5:

- The deflection criteria presented in Table 9.10.3 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.

9.10.12 Our experience indicates post-tensioned slabs may be 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. The structural engineer should design the foundation system to reduce the potential of edge lift occurring for the proposed structures.

9.10.13 During the construction of the post-tension foundation system, the concrete should be placed monolithically. Under no circumstances should cold joints form between the footings/grade beams and the slab during the construction of the post-tension foundation system unless designed by the structural engineer.

9.10.14 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. The estimated maximum total and differential settlement for the planned structures due to foundation loads is 1-inch and ½ inch, respectively.

9.10.15 Isolated footings outside of the slab area, 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. 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.

9.10.16 Interior stiffening beams should be incorporated into the design of the foundation system in accordance with the PTI design procedures.

9.10.17 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.

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

- For fill slopes less than 20 feet high or cut slopes regardless of height, 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. A post-tensioned slab and foundation system or mat foundation system can be used to reduce the potential for distress in the structures associated with strain softening and lateral fill extension. 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.

9.10.19 The recommendations of this report are intended to reduce the potential for cracking of slabs and foundations due to expansive soil (if present), differential settlement of fill 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 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.

9.10.20 Concrete slabs should be provided with adequate crack-control joints, construction joints and/or expansion joints to reduce unsightly shrinkage cracking. The design of joints should consider criteria of the American Concrete Institute (ACI) when establishing crack-control spacing. Additional steel reinforcing, concrete admixtures and/or closer crack control joint spacing should be considered where concrete-exposed finished floors are planned.

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

9.11 Excavation Slopes, Shoring, and Tiebacks

9.11.1 A retaining wall will be constructed along the north side of the site. We expect the wall will incorporate soil nails or solider pile and tie-backs, or other similar type wall construction. Deflection of the wall system should be limited so as to not impact adjacent structures and improvements.

9.11.2 The recommendations herein are provided for stable excavations and are submitted to the shoring and structural engineers to design a wall system. The contractor should construct the wall system as designed by the project shoring engineer. The stability of the excavation is dependent on the design and construction of the shoring system. Therefore, Geocon Incorporated cannot be responsible for site safety and the stability of the proposed excavations. It is the responsibility of the contractor to provide a safe excavation during the construction of the proposed project.

9.11.3 Temporary slopes should be made in conformance with OSHA requirements. Metamorphic Rock can be considered Type A soil (Type B soil if groundwater seepage is encountered) in accordance with OSHA requirements. Weathered metamorphic rock and compacted fill can be considered Type B soil (Type C if seepage is encountered). In general, special shoring requirements will not be necessary if temporary excavations will be less than 4 feet high. Temporary excavation depths greater than 4 feet, however, should be laid back at an appropriate inclination. These excavations should not become saturated or allowed to dry. Surcharge loads should not be permitted within a distance equal to the depth of the excavation from the top of the excavation. The top of the excavation should be a minimum of 15 feet from the edge of existing improvements. Excavations steeper than those recommended or closer than 15 feet from an existing surface improvement should be shored in accordance with applicable OSHA codes and regulations.

9.11.4 The design of shoring is governed by soil and groundwater conditions, and by the depth and width of the excavated area. Continuous support of the excavation face can be provided by a system of soldier piles and wood lagging. Excavations exceeding 15 feet may require tieback anchors to provide additional wall restraint.

9.11.5 The excavation will be made in hard metamorphic rock. As such, drilling for soldier piles, tie-back anchors, or soil nails will encounter very difficult drilling conditions.

9.11.6 Permanent walls with a level backfill should be designed using a lateral pressure envelope acting on the back of the shoring and applying a pressure equal to 23H, 15H, or 19H, for a triangular, rectangular, or trapezoidal distribution, respectively, where H is the height of the shoring, in feet (resulting pressure in pounds per square foot) as shown in Figure 10. These values are based on an estimated maximum wall height of 30 feet. For a 1.5:1 slope behind the wall, a pressure equal to 35H, 23H, or 28H, for a triangular, rectangular, or trapezoidal distribution, respectively, should be used as shown on Figure 11. Triangular distribution should be used for cantilevered shoring and the trapezoidal and rectangular distribution should be used for multi-braced systems such as tieback anchors and rakers. The project shoring engineer should determine the applicable soil distribution for the design of the wall system. Additional lateral earth pressure due to the surcharging effects of adjacent structures or traffic loads should be considered, where appropriate, in the design of the wall.

9.11.7 Passive soil pressure resistance for embedded portions of soldier piles into native bedrock can be based upon an equivalent passive soil fluid weight of 400+400D, where D is the depth of embedment in feet (resulting in pounds per square foot) from the base of the excavation limits, as shown in Figure 12. The passive resistance can be assumed to act over a width of three pile diameters. The soldier piles should be embedded a minimum of 0.5 times the maximum height of the excavation (this depth is to include footing excavations) if tieback anchors are not employed. The project shoring engineer should determine the actual embedment depth.

9.11.8 Drilled shafts for the soldier piles should be observed by Geocon Incorporated prior to the placement of concrete reinforcement to check that the exposed soil conditions are similar to those expected and that footing excavations have been extended to the appropriate bearing strata, and design depths. If unexpected soil conditions are encountered, foundation modifications may be required.

9.11.9 Lateral movement of shoring is associated with vertical ground settlement outside of the excavation. Therefore, it is essential that the soldier pile and tieback system allow very limited amounts of lateral displacement. Earth pressures acting on a lagging wall can cause movement of the shoring toward the excavation and result in ground subsidence outside of the excavation. Consequently, horizontal movements of the shoring wall should be accurately monitored and recorded during excavation and anchor construction.

9.11.10 Survey points should be established at the top of the pile on at least 20 percent of the soldier piles. An additional point located at an intermediate point between the top of the pile and the base of the excavation should be monitored on at least 20 percent of the piles if tieback anchors will be used. These points should be monitored on a weekly basis during excavation work and on a monthly basis until the completion of the wall.

9.11.11 The wall should be designed to limit horizontal soldier pile movement so as to not impact surrounding properties and improvements. The amount of horizontal deflection can be assumed to be essentially zero along the Active Zone and Effective Zone boundary. The magnitude of movement for intermediate depths and distances from the wall can be linearly interpolated. The project civil and/or wall engineer should determine the allowable amount of horizontal movement associated with the wall system that could affect existing utilities and structures, if present. In addition, the project civil and/or wall engineer should evaluate the existing utilities and improvements and provide a conclusion regarding the ability of the utilities and improvements to withstand the expected lateral and vertical movement associated with the planned excavation.

9.11.12 Tieback anchors employed in shoring should be designed such that anchors fully penetrate the Active Zone behind the wall. The Active Zone can be considered the wedge of soil from the face of the wall to a plane extending upward from the base of the excavation at a 25-degree angle from vertical, as shown on Figure 13. Normally, tieback anchors are contractor-designed and installed, and there are numerous anchor construction methods available. Non-shrinkage grout should be used for the construction of the tieback anchors.

9.11.13 A wall drain system should be incorporated into the design. A typical wall drain detail is provided on Figure 14. Corrosion protection should be provided for the tiebacks.

9.11.14 Experience has shown that the use of pressure grouting during formation of the bonded portion of the anchor will increase the soil-grout bond stress. A pressure grouting tube should be installed during the construction of the tieback. Post grouting should be performed if adequate capacity cannot be obtained by other construction methods.

9.11.15 Anchor capacity is a function of construction method, depth of anchor, batter, diameter of the bonded section, and the length of the bonded section. Anchor capacity should be evaluated using the strength parameters shown in Table 9.11.

TABLE 9.11
SOIL STRENGTH PARAMETERS FOR WALL

Description	Cohesion	Friction Angle
Metamorphic Rock	0 psf	45 degrees

9.11.16 Grout should only be placed in the tieback anchor's bonded section prior to testing. Tieback anchors should be proof-tested to at least 130 percent of the anchor's design working load. Following a successful proof test, the tieback anchors should be locked off at 80 percent of the allowable working load. Tieback anchor test failure criteria should be established in project plans and specifications. The tieback anchor test failure criteria should be based upon a maximum allowable displacement at 130 percent of the anchor's working load (anchor creep) and a maximum residual displacement within the anchor following stressing. Tieback anchor stressing should only be conducted after sufficient hydration has occurred within the grout. Tieback anchors that fail to meet project specified test criteria should be replaced or additional anchors should be constructed.

9.11.17 Lagging should keep pace with excavation and tieback anchor construction. The excavation should not be advanced deeper than three feet below the bottom of lagging at any time. These unlagged gaps of up to three feet should only be allowed to stand for short periods of time in order to decrease the probability of soil instability and should never be unsupported overnight. Backfilling should be conducted when necessary between the back of lagging and excavation sidewalls to reduce sloughing in this zone and all voids should be filled by the end of each day. Further, the excavation should not be advanced further than four feet below a row of tiebacks prior to those tiebacks being proof tested and locked off.

9.11.18 If tieback anchors are employed, an accurate survey of existing utilities and other underground structures adjacent to the shoring wall should be conducted. The survey should include both locations and depths of existing utilities. Locations of anchors should be adjusted as necessary during the design and construction process to accommodate the existing and proposed utilities.

9.11.19 The condition of existing buildings, streets, sidewalks, and other structures/improvements around the perimeter of the planned excavation should be documented prior to the start of shoring and excavation work. Special attention should be given to documenting existing cracks or other indications of differential settlement within these adjacent structures, pavements and other improvements. Underground utilities sensitive to settlement should be videotaped prior to construction to check the integrity of pipes. In addition, monitoring points should be established indicating location and elevation around the excavation and on existing buildings. These points should be monitored on a weekly basis during excavation work and on a monthly basis thereafter. Inclinometers should be installed and monitored behind any shoring sections that will be excavated deeper than 30 feet below the existing ground surface.

9.12 Soil Nail Wall

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

9.12.2 The excavation for the wall will be made in hard metamorphic rock. As such, drilling for soil nails will encounter very difficult drilling conditions.

9.12.3 A wall drain system should be incorporated into the design. A typical wall drain detail for a soil nail wall is provided on Figure 15. Corrosion protection should be provided for the nails.

9.12.4 Geocon Incorporated should provide observation services during nail installation, grout and shotcrete strength testing, and nail testing.

9.12.5 Design and testing of soil nails should be conducted in conformance with FHWA guidelines presented in the *Manual for Design and Construction Monitoring of Soil Nail Walls, FHWA-SA-96-069.* In addition to verification and proof testing, we recommended ultimate strength tests be performed to verify ultimate bond strength assumptions.

9.12.6 All verification test nails should sacrificial and not incorporated into the wall.

9.12.7 The soil strength parameters listed in Table 9.12 can be used in design of the soil nails.

TABLE 9.12
SOIL STRENGTH PARAMETERS FOR SOIL NAIL WALLS

Description	Cohesion (psf)	Friction Angle (degrees)	Ultimate Bond Stress (psi)
Metamorphic Rock	0	45 degrees	40 psi

9.13 Conventional Retaining Walls

9.13.1 Retaining walls that are allowed to rotate more than 0.001H (where H equals the height of the retaining portion of the wall) at the top of the wall and having a level backfill surface should be designed for an active soil pressure equivalent to the pressure exerted by a fluid density of 35 pcf. Where the backfill will be inclined at 2:1 (horizontal:vertical), an active soil pressure of 50 pcf is recommended. Expansive soils should not be used as backfill material behind retaining walls. All soil placed for retaining wall backfill should have an Expansion Index less than 50.

9.13.2 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.

9.13.3 Where walls are restrained from movement at the top, an additional uniform pressure of 7H psf 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 2 feet of fill soil should be added.

9.13.4 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 of less than 50) backfill material with no hydrostatic forces or imposed surcharge load. Figure 16 presents a typical retaining wall drainage detail. If conditions different than those described are anticipated, or if specific drainage details are desired, Geocon Incorporated should be contacted for additional recommendations.

9.13.5 The structural engineer should determine the seismic design category for the project in accordance with Section 1613 of the CBC. If the project possesses a seismic design category of D, E, or F, retaining walls that support more than 6 feet of backfill should be designed with seismic lateral pressure in accordance with Section 18.3.5.12 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 0.396g calculated from ASCE 7-10 Section 11.8.3 and applied a pseudo-static coefficient of 0.33.

9.13.6 The recommendations assume a properly compacted granular backfill soil with no hydrostatic forces or imposed surcharge load. If the retaining walls are subject to surcharge loading within a horizontal distance equal to or less than the height of the wall, or if conditions different than those described are expected, Geocon Incorporated should be contacted for additional recommendations.

9.13.7 Footings near the top of slopes or within slopes should be extended in depth such that the outer bottom edge of the footing is at least 7 feet horizontally from the face of the finish slope.

9.13.8 In general, shallow conventional wall footings founded in properly compacted fill and 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 3 feet below the base of the wall has an Expansion Index of 50 or less. The recommended allowable soil bearing pressures 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.

9.14 Lateral Loading

9.14.1 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 formation 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. Where walls are planned adjacent to and/or on descending slopes, a passive pressure of 150 pcf should be used in design.

9.14.2 If friction is to be used to resist lateral loads, an allowable coefficient of friction between soil and concrete of 0.35 should be used for design.

9.14.3 The passive and frictional resistant loads can be combined for design purposes. The lateral passive pressures may be increased by one-third when considering transient loads due to wind or seismic forces.

9.15 MSE Retaining Wall Recommendations

9.15.1 We recommend the following geotechnical parameters be used for design of the MSE retaining walls.

TABLE 9.15
GEOTECHNICAL DESIGN PARAMETERS

Parameter	Reinforced Zone	Retained Zone	Foundation Zone
Angle of Internal Friction	30 degrees	30 degrees	30 degrees
Cohesion	100 psf	100 psf	100 psf
Moist Unit Weight	130 pcf	130 pcf	130 pcf

9.15.2 The shear strength values provided in Table 9.15 for the reinforced zone assume that granular materials will be used as backfill. Because importing or crushing of on-site materials will be required to generate wall backfill materials, we recommend proposed wall backfill soils be tested prior to importing and during grading to check that the soils meet the values listed on Table 9.11 and those used in the design of the MSE wall.

9.15.3 If crushing of on-site soils will be performed to generate backfill for MSE type walls, the crushed product should meet wall designer specifications. Typically, MSE wall designers do not allow the use of angular rock within the backfill soil due to the potential for damage to the reinforcing grid. All wall backfill should have an expansion index (EI) of 50 or less.

9.15.4 Once proposed backfill materials are imported or crushed product is made, sufficient samples should be collected and subjected to laboratory testing to assess the soils suitability for use as wall backfill. 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).

9.15.5 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 or slightly above optimum moisture content in accordance with ASTM D 1557. This is applicable to the entire embedment length of the geogrid reinforcement. Typically, wall designers specify that heavy compaction equipment be excluded from within 3 feet of the face of the wall;

however, smaller equipment (e.g., walk-behind, self-driven compactors or hand whackers) should 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 geogrid within the uncompacted zone should not be relied upon for reinforcement and overall embedment lengths should be increased to account for the difference.

9.15.6 The wall should be provided with drainage system sufficient enough to prevent excessive seepage through the wall and water at the base of the wall to prevent hydrostatic pressures behind the wall.

9.15.7 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), construction, and the type of geosynthetic used. In addition, over time reinforced-earth retaining walls have been known to exhibit creep 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 and should be designed to accommodate this movement.

9.16 Storm Water Management

9.16.1 If storm water management devices are not properly designed and constructed, there is a risk for distress to improvements and properties located hydrologically down gradient or adjacent to these devices. Factors such as the amount of water being 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 into the subsurface occurs, downstream improvements 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.

9.16.2 Storm water management recommendations are provided in Appendix C.

9.17 Site Drainage and Moisture Protection

9.17.1 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.

9.17.2 In the case of basement walls or building walls retaining landscaping areas, a waterproofing system should be used on the wall and joints, and a Miradrain drainage panel (or similar) should be placed over the waterproofing. The project architect or civil engineer should provide detailed specifications on the plans for all waterproofing and drainage.

9.17.3 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.

9.17.4 Landscaping planters adjacent to paved areas are not recommended due to the potential for surface or irrigation water to infiltrate the pavement's subgrade and base course. Area drains to collect excess irrigation water and transmit it to drainage structures or impervious above-grade planter boxes can be used. In addition, where landscaping is planned adjacent to the pavement, construction of a cutoff wall along the edge of the pavement that extends at least 6 inches below the bottom of the base material should be considered.

9.18 Slope Maintenance

9.18.1 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. 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.

9.19 Grading and Foundation Plan Review

9.19.1 The geotechnical engineer and engineering geologist should review the grading and foundation plans prior to final submittal to check their compliance with the recommendations of this report and to determine the need for additional comments, recommendations and/or analysis.

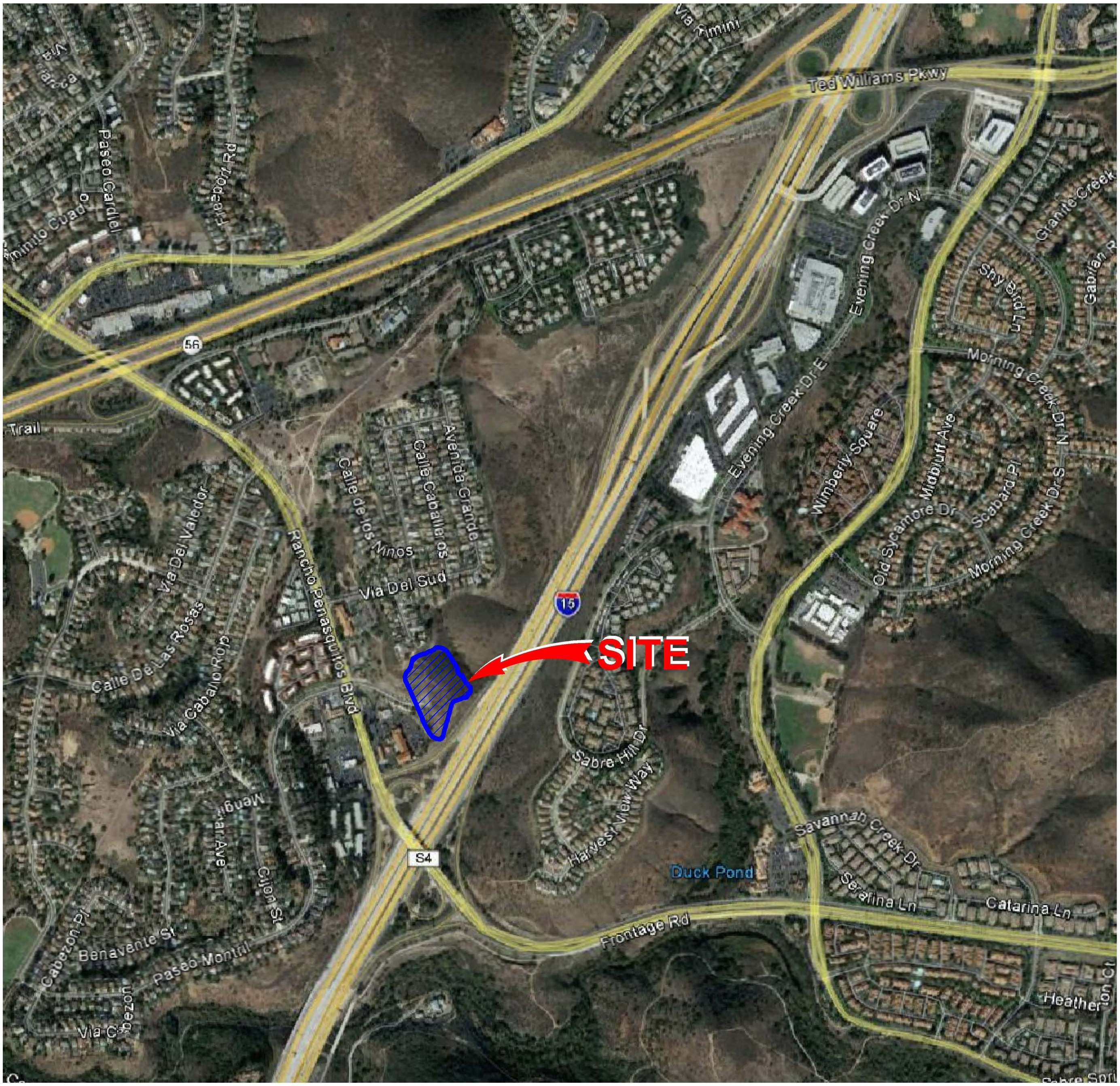
LIMITATIONS AND UNIFORMITY OF CONDITIONS

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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PASEO MONTRIL
SAN DIEGO, CALIFORNIA

RM / AML		DSK/GTYPD	DATE 01 - 05 - 2018	PROJECT NO. G2209 - 42 - 01	FIG. 1

Plotted:01/04/2018 3:40PM | By:ALVIN LADRILLONO | File Location:Y:\PROJECTS\G2209-42-01 (Paseo Montril)\DETAILS\G2209-42-01 Vic Map.dwg

GRAPHIC SCALE

0' 20' 40' 60' 80' 120' 160'

SCALE 1"=40' (on 36x24)

GEOCON LEGEND

Qudf ........UNDOCUMENTED FILL

Mzu ....METAMORPHIC ROCK

........APPROX. LOCATION OF GEOLOGIC CONTACT (Queried Where Uncertain)

T-4 ....APPROX. LOCATION OF TRENCH

AT-6 ....APPROX. LOCATION OF AIR TRACK BORING

(+17) ....APPROX. DEPTH TO BEDROCK

[15] ....APPROX. DEPTH OF RIPPABLE MATERIAL BASED ON PENETRATION RATE OF 20 SPF

D D' ....APPROX. LOCATION OF GEOLOGIC CROSS SECTION

GEOLOGIC MAP

PASEO MONTRIL
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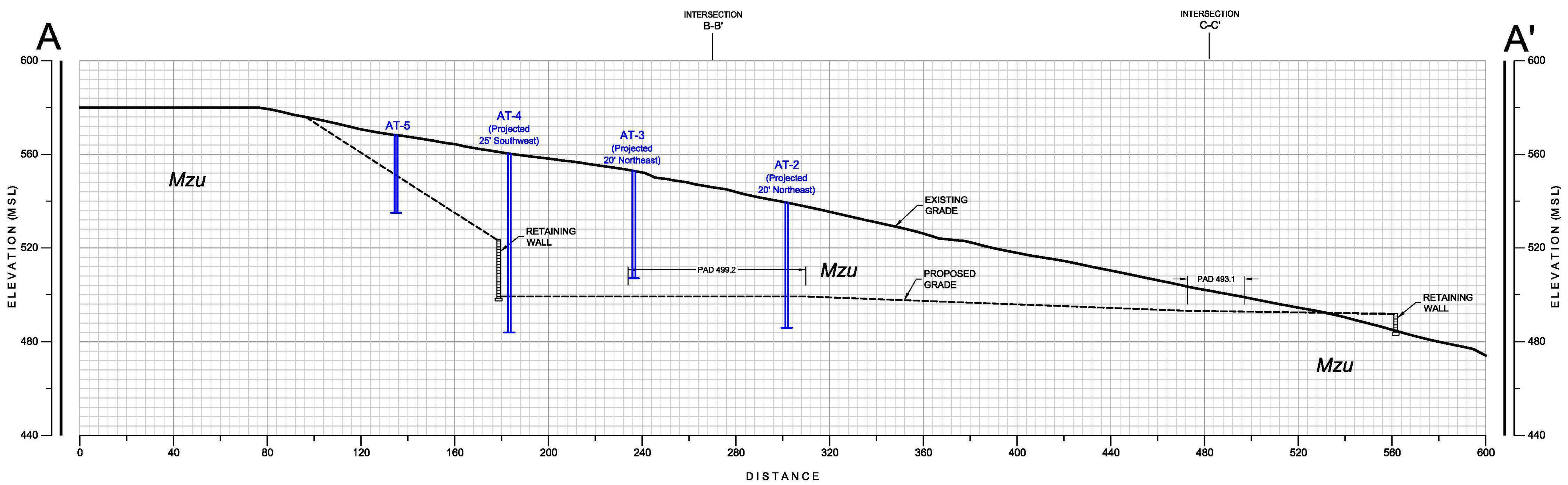
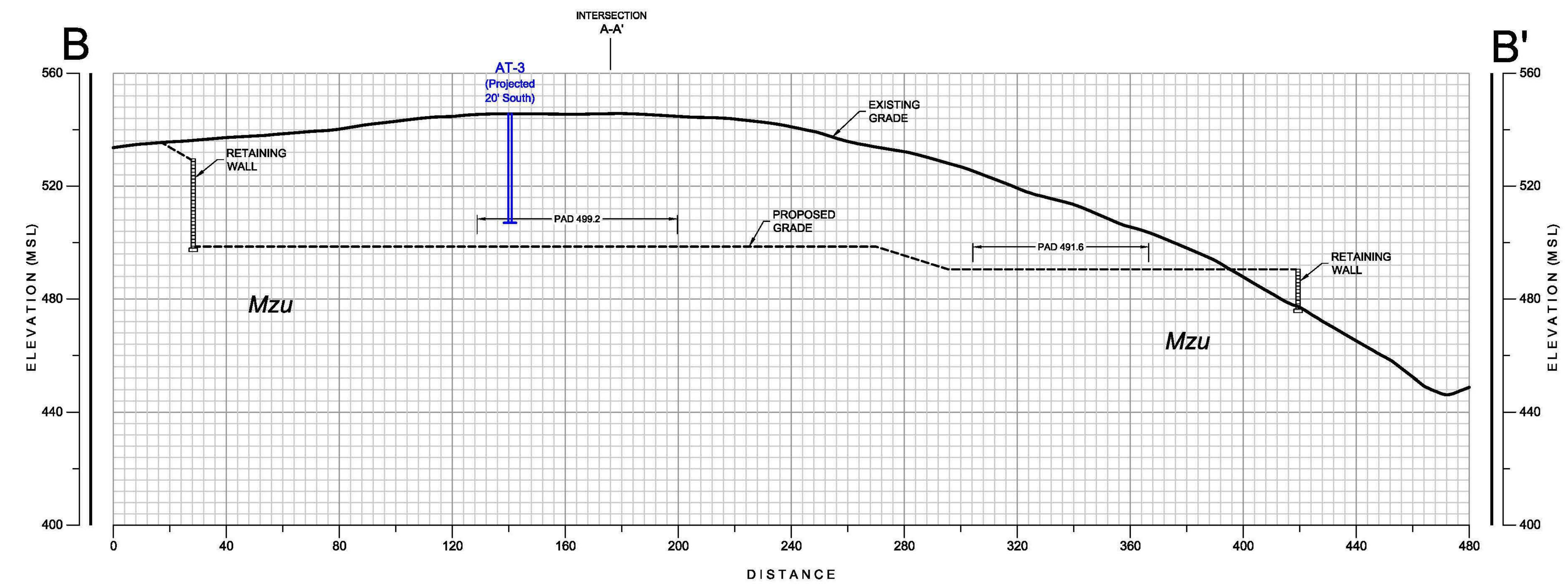
SCALE 1" = 40'	DATE 01 - 05 - 2018		
PROJECT NO. G2209 - 42 - 01		FIGURE 2	
SHEET 1 OF 1			

Plotted:01/04/2018 4:01PM | By:ALVIN LADRILLONO | File Location:Y:\PROJECTS\G2209-42-01 (Paseo Montril)\SHEETS\G2209-42-01 Geo Map.dwg



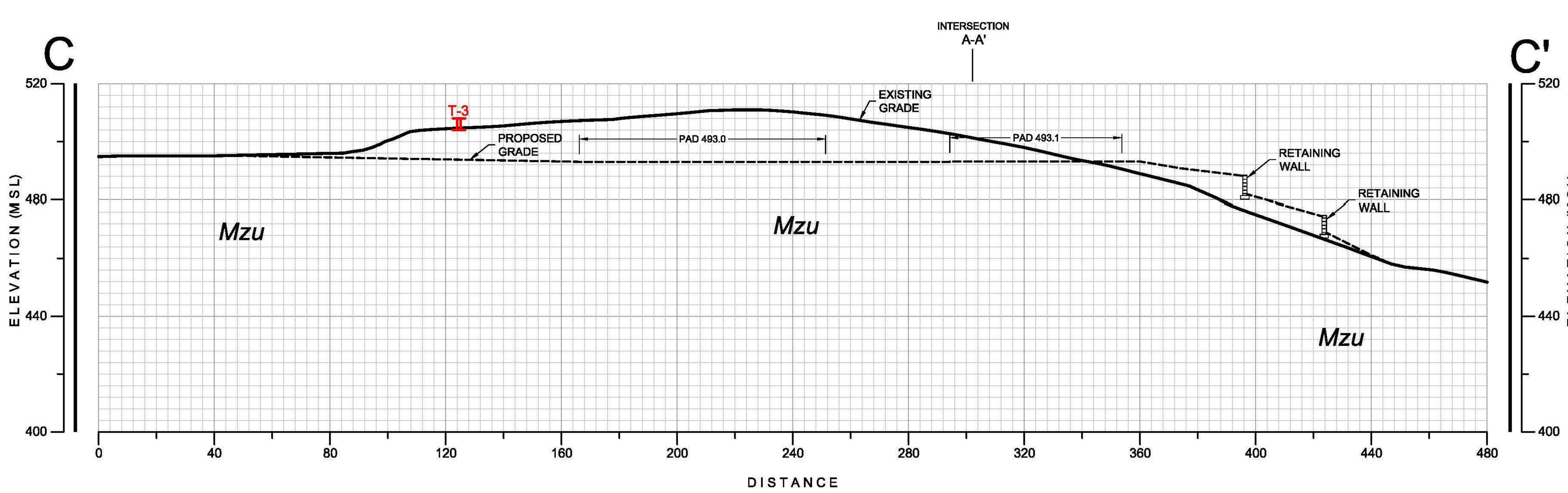
GEOLOGIC CROSS-SECTION A-A'

SCALE: 1" = 40' (Vert. = Horiz.)



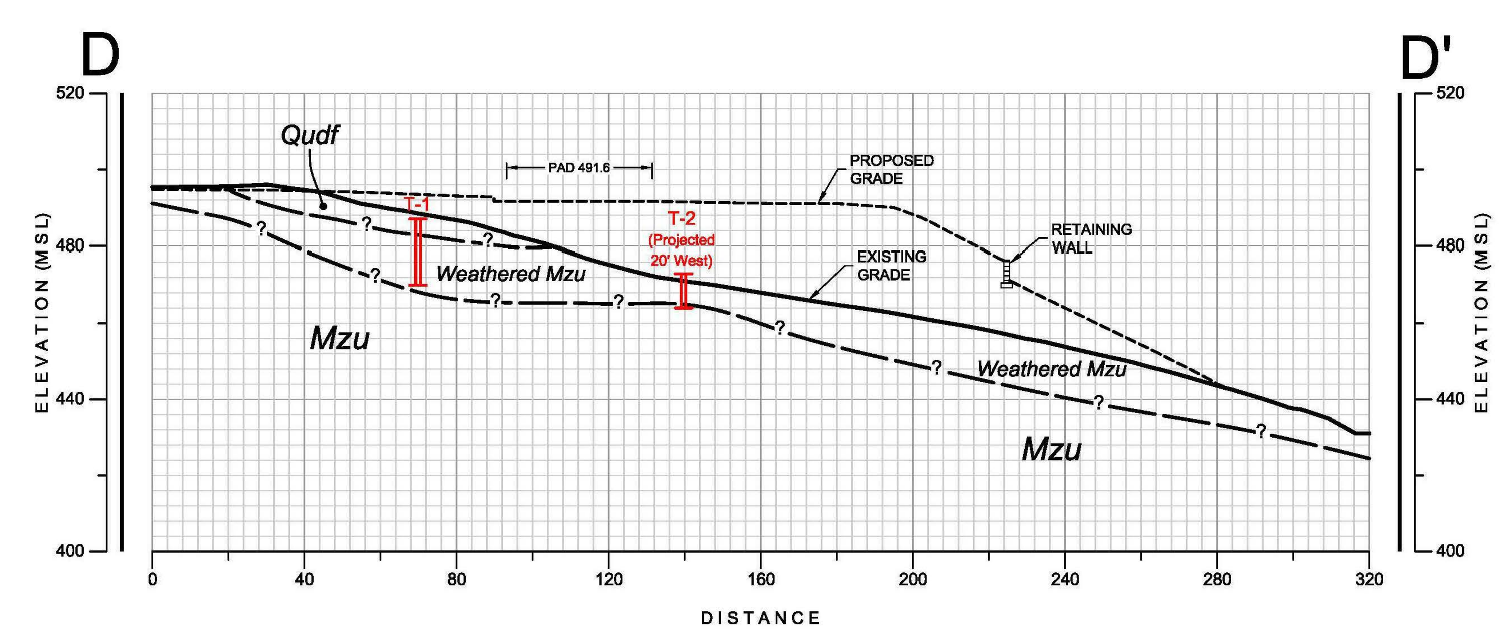
GEOLOGIC CROSS-SECTION A-A'

SCALE: 1" = 40' (Vert. = Horiz.)



GEOLOGIC CROSS-SECTION A-A'

SCALE: 1" = 40' (Vert. = Horiz.)



GEOLOGIC CROSS-SECTION A-A'

SCALE: 1" = 40' (Vert. = Horiz.)

GEOCON LEGEND

Qudf ........UNDOCUMENTED FILL

Mzu ....METAMORPHIC ROCK

T-4 ....APPROX. LOCATION OF TRENCH

AT-6 ....APPROX. LOCATION OF AIR TRACK BORING

........APPROX. LOCATION OF GEOLOGIC CONTACT (Queried Where Uncertain)

GEOLOGIC CROSS SECTION

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SCALE	1" = 40'	DATE	01 - 05 - 2018
PROJECT NO.	G2209 - 42 - 01	FIGURE	3
SHEET	1 OF 1		

Plotted:01/05/2018 8:50AM | By:RUBEN AGUILAR | File Location:Y:\PROJECTS\G2209-42-01 (Paseo Montril)\SHEETS\G2209-42-01 XSections.dwg

ASSUMED CONDITIONS :

SLOPE HEIGHT	H	= 50 feet
SLOPE INCLINATION	2 : 1	(Horizontal : Vertical)
TOTAL UNIT WEIGHT OF SOIL	γ_t	= 130 pounds per cubic foot
ANGLE OF INTERNAL FRICTION	ϕ	= 30 degrees
APPARENT COHESION	C	= 200 pounds per square foot
NO SEEPAGE FORCES		

ANALYSIS :

$\gamma_{c\phi} = \dfrac{\gamma_t H \tan\phi}{C}$		EQUATION (3-3), REFERENCE 1
$FS = \dfrac{N_{cf} C}{\gamma_t H}$		EQUATION (3-2), REFERENCE 1
$\gamma_{c\phi}$	= 18.8	CALCULATED USING EQ. (3-3)
N_{cf}	= 50	DETERMINED USING FIGURE 10, REFERENCE 2
FS	= 1.54	FACTOR OF SAFETY CALCULATED USING EQ. (3-2)

REFERENCES :

1......Janbu, N., Stability Analysis of Slopes with Dimensionless Parameters, Harvard Soil Mechanics, Series No. 46, 1954

2......Janbu, N., Discussion of J.M. Bell, Dimensionless Parameters for Homogeneous Earth Slopes, Journal of Soil Mechanics and Foundation Design, No. SM6, November 1967.

SLOPE STABILITY ANALYSIS

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RM / AML		DSK/GTYPD	DATE 01 - 05 - 2018	PROJECT NO. G2209 - 42 - 01	FIG. 4

ASSUMED CONDITIONS :

SLOPE HEIGHT	H	= Infinite
DEPTH OF SATURATION	Z	= 3 feet
SLOPE INCLINATION	2 : 1	(Horizontal : Vertical)
SLOPE ANGLE	i	= 26.6 degrees
UNIT WEIGHT OF WATER	γ_w	= 62.4 pounds per cubic foot
TOTAL UNIT WEIGHT OF SOIL	γ_t	= 130 pounds per cubic foot
ANGLE OF INTERNAL FRICTION	ϕ	= 30 degrees
APPARENT COHESION	C	= 200 pounds per square foot

SLOPE SATURATED TO VERTICAL DEPTH Z BELOW SLOPE FACE

SEEPAGE FORCES PARALLEL TO SLOPE FACE

ANALYSIS :

$$FS = \frac{C + (\gamma_t - \gamma_w)\ Z \cos^2 i \tan \phi}{\gamma_t\ Z \sin i \cos i} = 1.9$$

REFERENCES :

1......Haefeli, R. *The Stability of Slopes Acted Upon by Parallel Seepage*, Proc. Second International Conference, SMFE, Rotterdam, 1948, 1, 57-62

2......Skempton, A. W., and F.A. Delory, *Stability of Natural Slopes in London Clay*, Proc. Fourth International Conference, SMFE, London, 1957, 2, 378-81

SURFICIAL SLOPE STABILITY ANALYSIS

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RM / AML		DSK/GTYPD	DATE 01 - 05 - 2018	PROJECT NO. G2209 - 42 - 01	FIG. 5

Paseo Montril
Project No. G2209-42-01
Section A-A'
Name: Section A-A'.gsz
Date: 1/4/2018
Mzu: Unit Weight: 135 pcf: Cohesion: 500 psf: Phi: 45 °

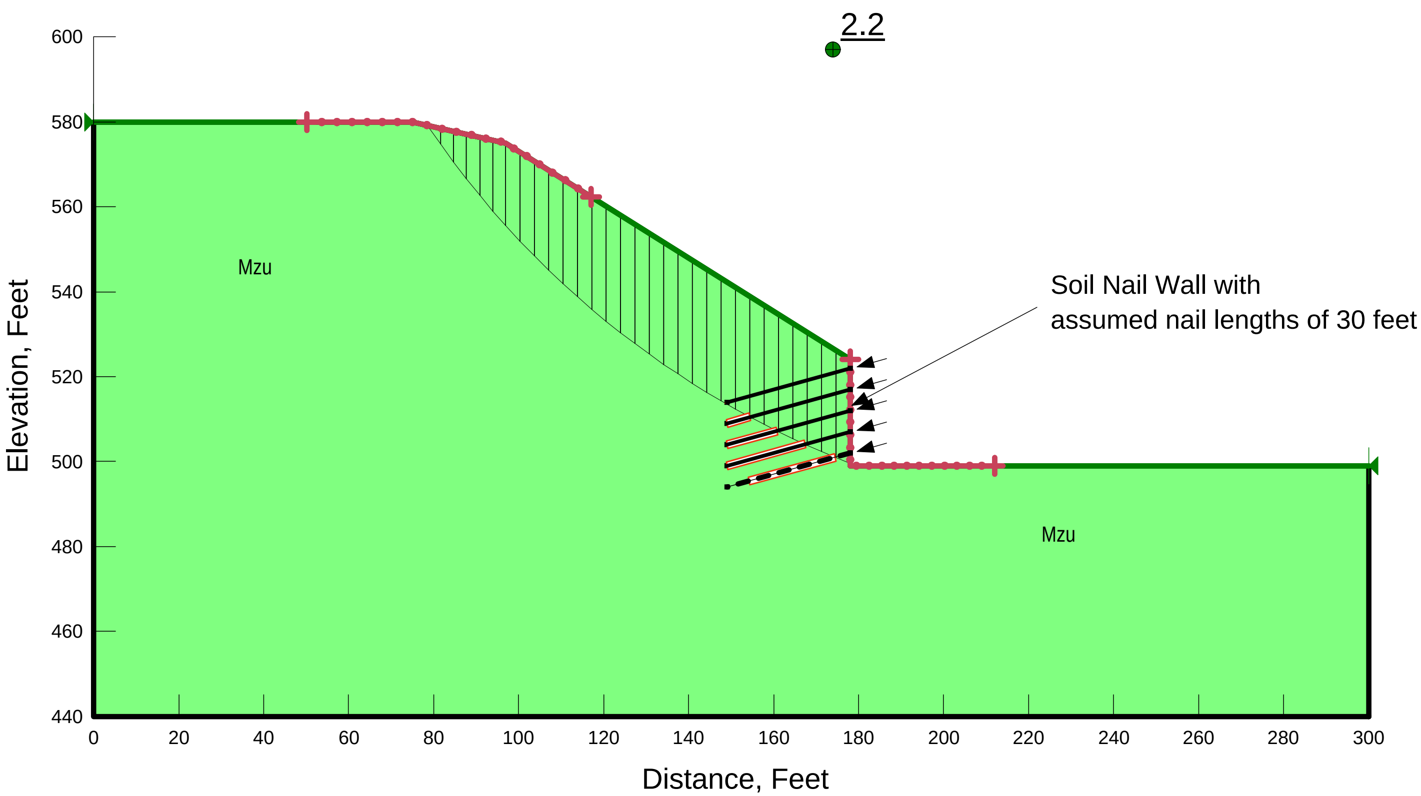
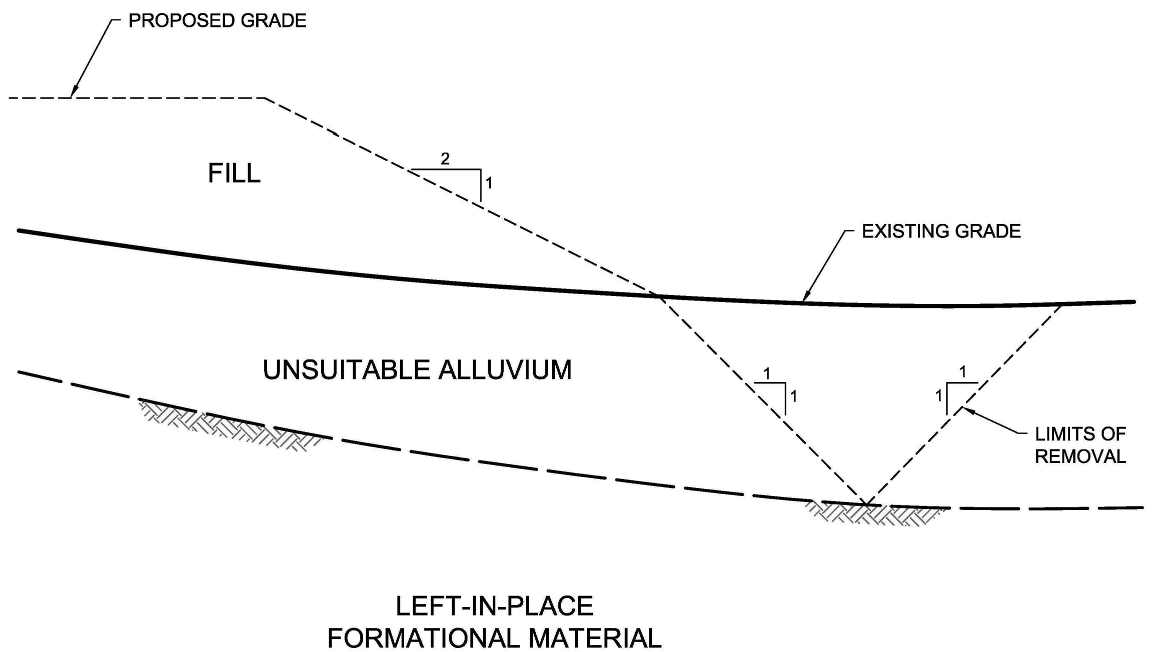


Figure 6



NOT TO SCALE

NOTE:

SLOPE OF BACKCUT MAY BE STEEPENED WITH THE APPROVAL OF THE SOILS ENGINEER WHERE BOUNDARY CONSTRAINTS LIMIT EXTENT OF REMOVALS

NO SCALE

CONSTRUCTION DETAIL FOR LATERAL EXTENT OF REMOVAL

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RM / AML		DSK/GTYPD	DATE 01 - 05 - 2018	PROJECT NO. G2209 - 42 - 01	FIG. 7

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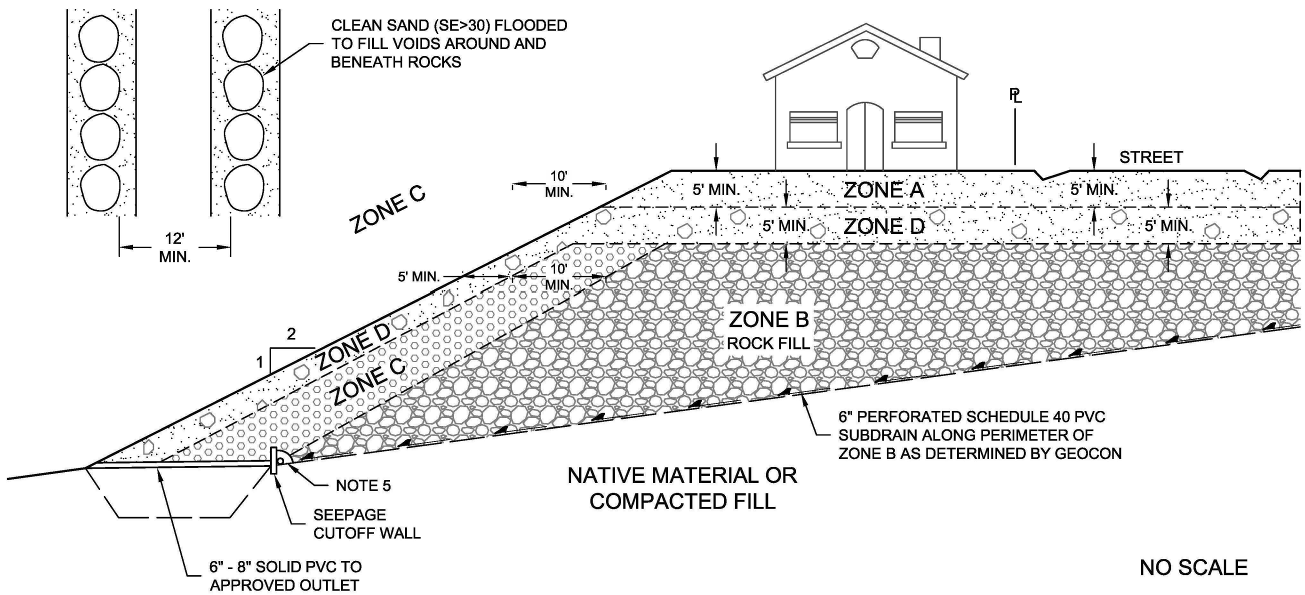
ZONE B

WINDROWS DETAIL
(PLAN VIEW)



LEGEND

ZONE A: COMPACTED SOIL FILL. NO ROCK FRAGMENTS OVER 6 INCHES IN DIMENSION.

ZONE B: BLASTED ROCK FILL GENERALLY CONSISTING OF 2 FOOT MINUS MATERIAL WITH OCCASIONAL INDIVIDUAL ROCK UP TO 4 FEET MAXIMUM DIMENSION. IN PARKWAY/STREETS, ZONE B SHOULD TERMINATE AT LEAST 2 FEET BELOW LOWEST UTILITY. 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 C: ROCKS UP TO 2 FEET IN MAXIMUM DIMENSION IN A MATRIX OF COMPACTED SOIL FILL WITHIN SLOPE AREAS ONLY.

ZONE D: ROCKS UP TO 1 FOOT IN MAXIMUM DIMENSION IN A MATRIX OF COMPACTED SOIL FILL.

NOTES

1. COMPACTED SOIL FILL IN UPPER 8 FEET SHALL CONTAIN AT LEAST 40 PERCENT SOIL PASSING THE 3/4 - INCH SIEVE (BY WEIGHT) AND IN THE UPPER 3 FEET OF PAD GRADE AT LEAST 20% SOIL PASSING THE NO. 4 SIEVE (BY WEIGHT) AND COMPACTED IN ACCORDANCE WITH SPECIFICATIONS FOR STRUCTURAL FILL.
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 THE RECOMMENDATION OF THE GEOTECHNICAL ENGINEER.
4. DEPTH OF ZONE D SHOULD EXTEND AT LEAST 2 FEET BELOW DEEPEST UTILITY WITHIN ROADWAYS.
5. 6" PERFORATED SCHEDULE 40 PVC SUBDRAIN ALONG THE TOE AND PORTIONS OF THE PERIMETER OF ZONE B.
6. BASE OF ZONE B SHOULD SLOPE A MINIMUM OF 2 PERCENT.

OVERSIZE ROCK DISPOSAL DETAIL

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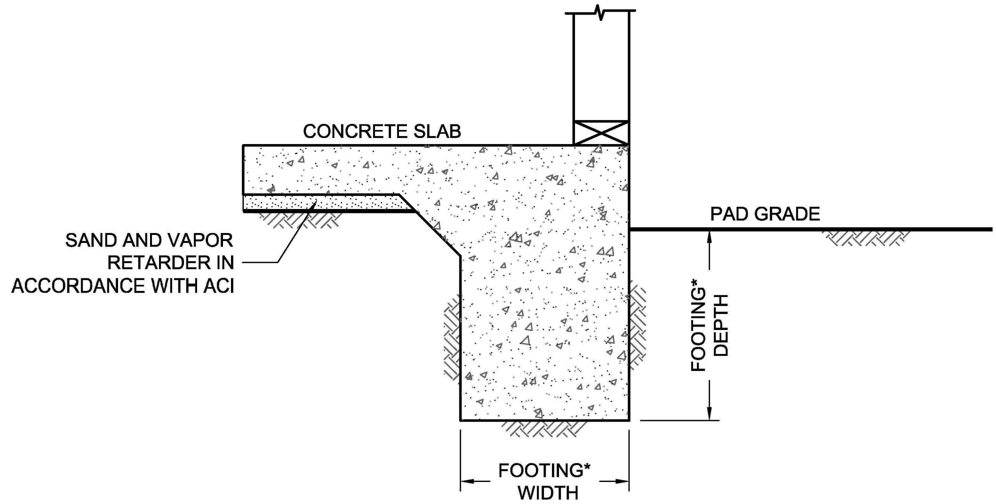
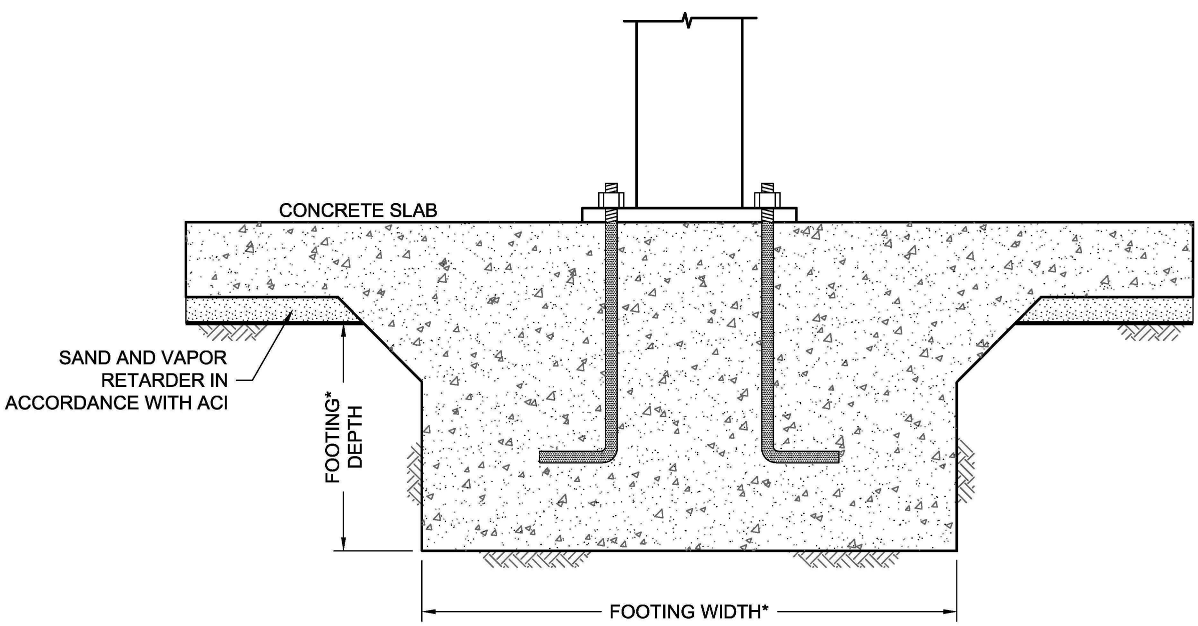
RM / AML		DSK/GTYPD	DATE 01 - 05 - 2018	PROJECT NO. G2209 - 42 - 01	FIG. 8





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

NO SCALE

WALL / COLUMN FOOTING DIMENSION DETAIL

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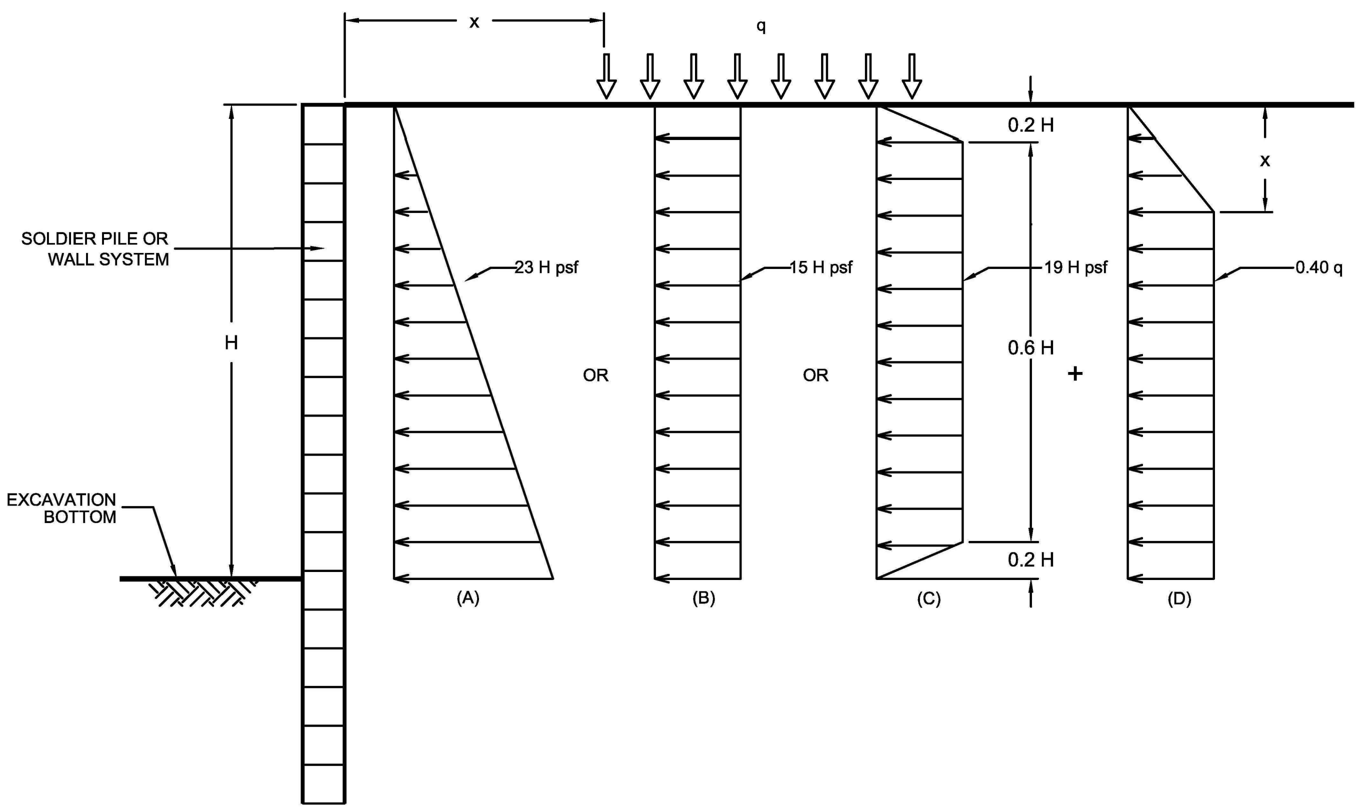
RM / AML		DSK/GTYPD	DATE 01 - 05 - 2018	PROJECT NO. G2209 - 42 - 01	FIG. 9

Plotted:01/04/2018 3:32PM | By:ALVIN LADRILLONO | File Location:Y:\PROJECTS\G2209-42-01 (Paseo Montril)\DETAILS\Wall-Column Footing Dimension Detail (COLFOOT2).dwg



(A)......TRIANGULAR DISTRIBUTION - CANTILEVERED SHORING

(B)......RECTANGULAR DISTRIBUTION - MULTI-BRACED SHORING

(C)......TRAPEZOIDAL DISTRIBUTION - MULTI BRACED SHORING

(D)......SURCHARGE DISTRIBUTION

NO SCALE

LATERAL ACTIVE PRESSURES FOR VERTICAL EXCAVATIONS

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RM / AML		DSK/GTYPD	DATE 01 - 05 - 2018	PROJECT NO. G2209 - 42 - 01	FIG. 10

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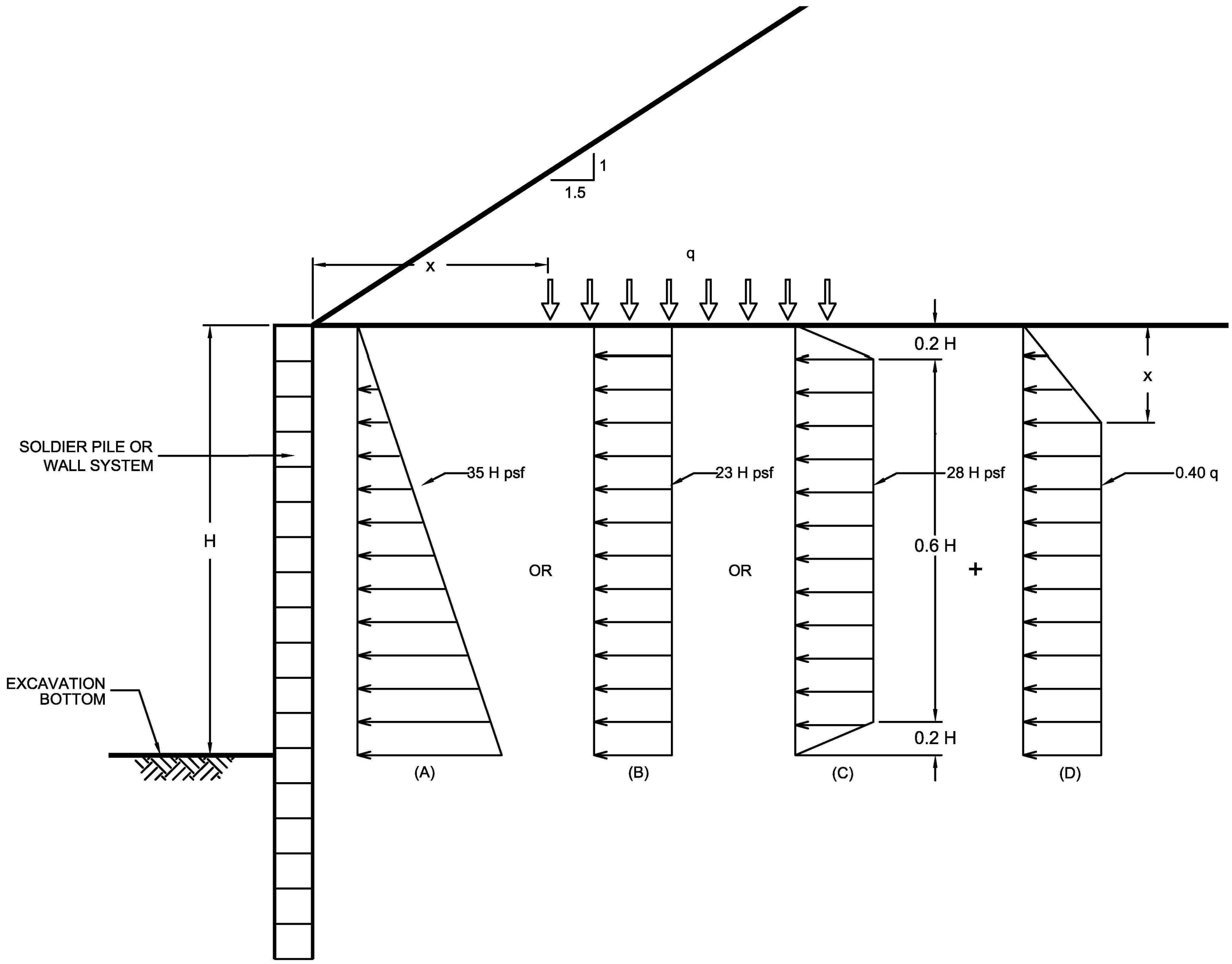

(A)......TRIANGULAR DISTRIBUTION - CANTILEVERED SHORING

(B)......RECTANGULAR DISTRIBUTION - MULTI-BRACED SHORING

(C)......TRAPEZOIDAL DISTRIBUTION - MULTI BRACED SHORING

(D)......SURCHARGE DISTRIBUTION

NO SCALE

LATERAL ACTIVE PRESSURES FOR VERTICAL EXCAVATIONS

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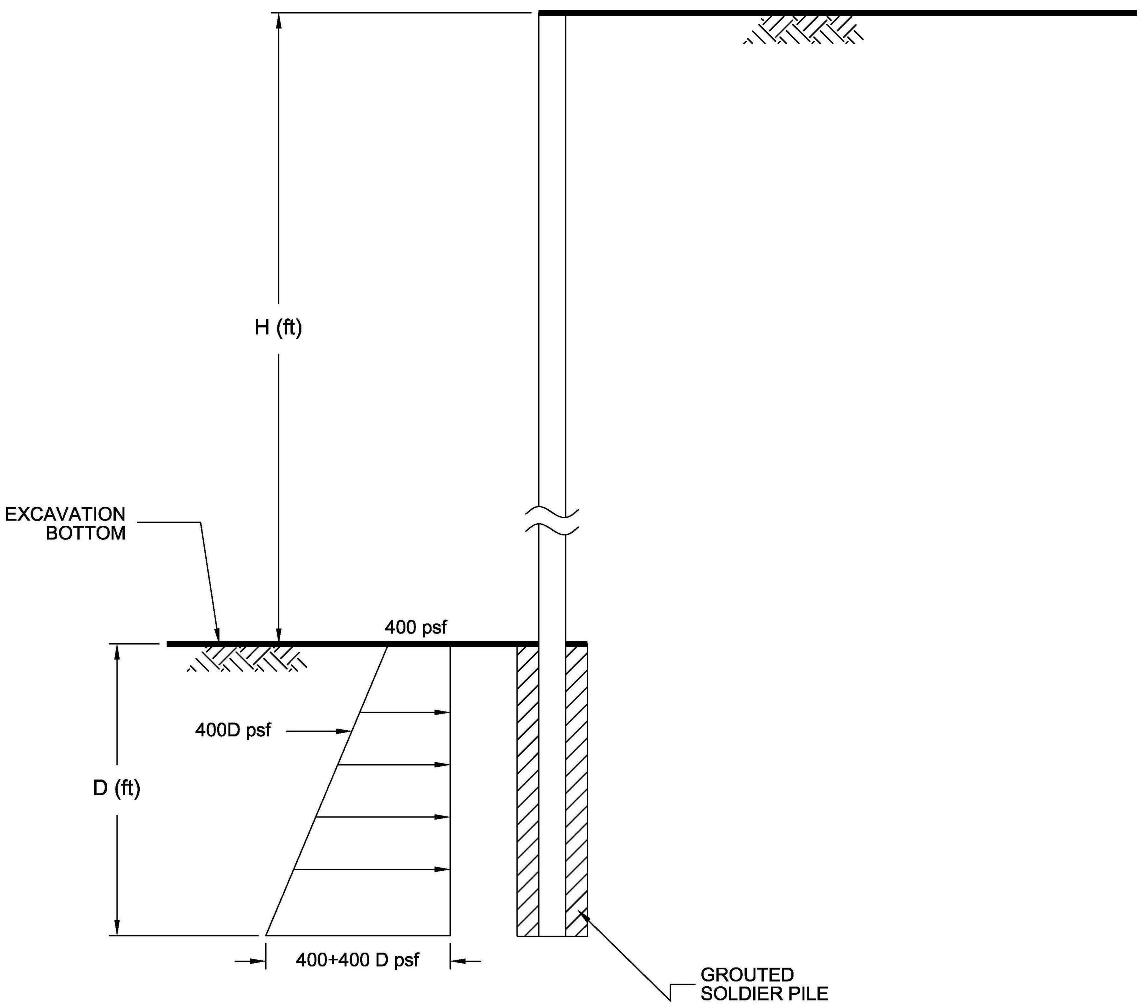
RM / AML		DSK/GTYPD	DATE 01 - 05 - 2018	PROJECT NO. G2209 - 42 - 01	FIG. 11



NO SCALE

RECOMMENDED GROUTED SOLDIER PILE PRESSURE DISTRIBUTION

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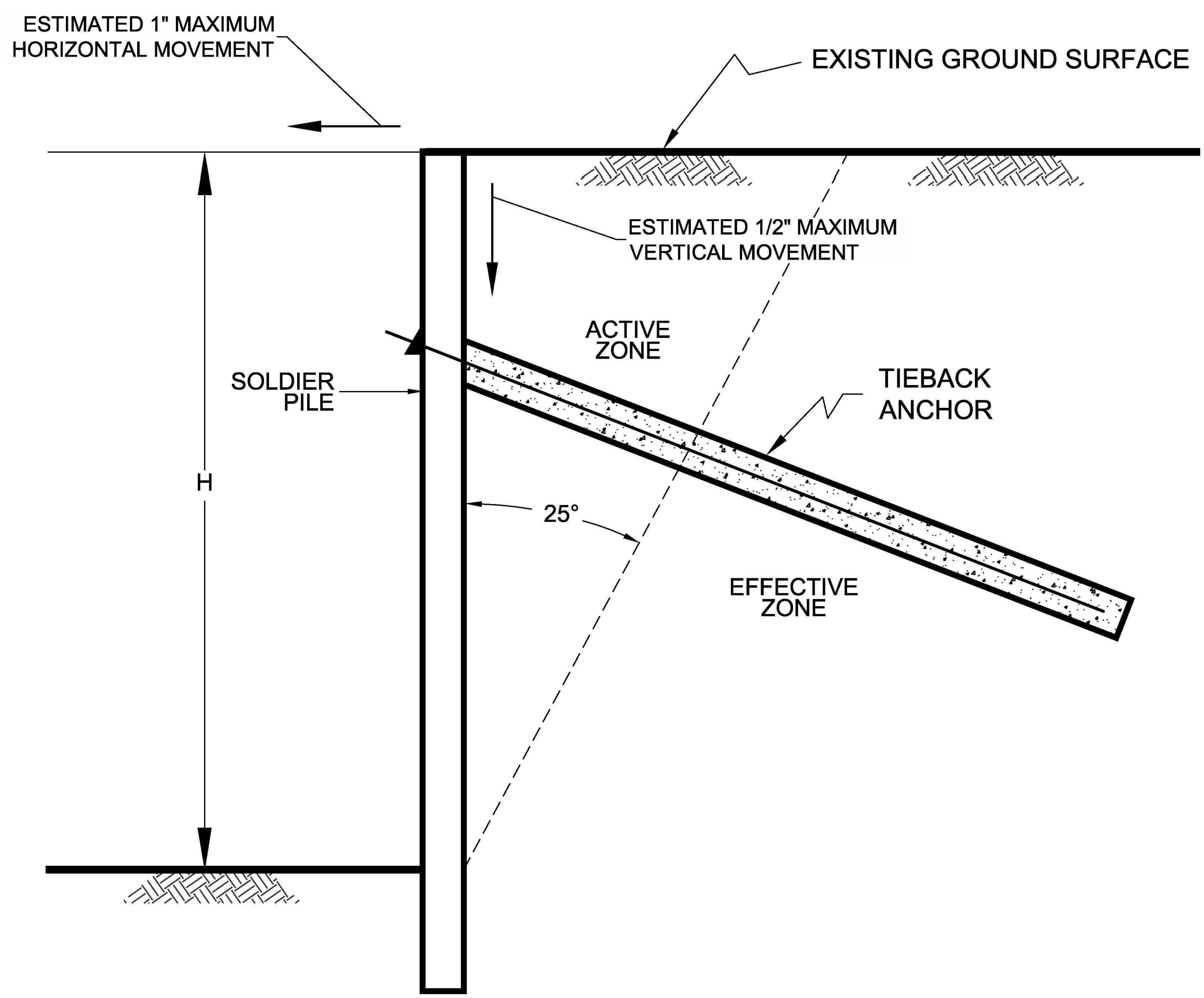
RM / AML		DSK/GTYPD	DATE 01 - 05 - 2018	PROJECT NO. G2209 - 42 - 01	FIG. 12



NOTE: NO ESTIMATED MOVEMENT AT EFFECTIVE ZONE

NO SCALE

RECOMMENDED EFFECTIVE ZONE FOR TIEBACK ANCHORS

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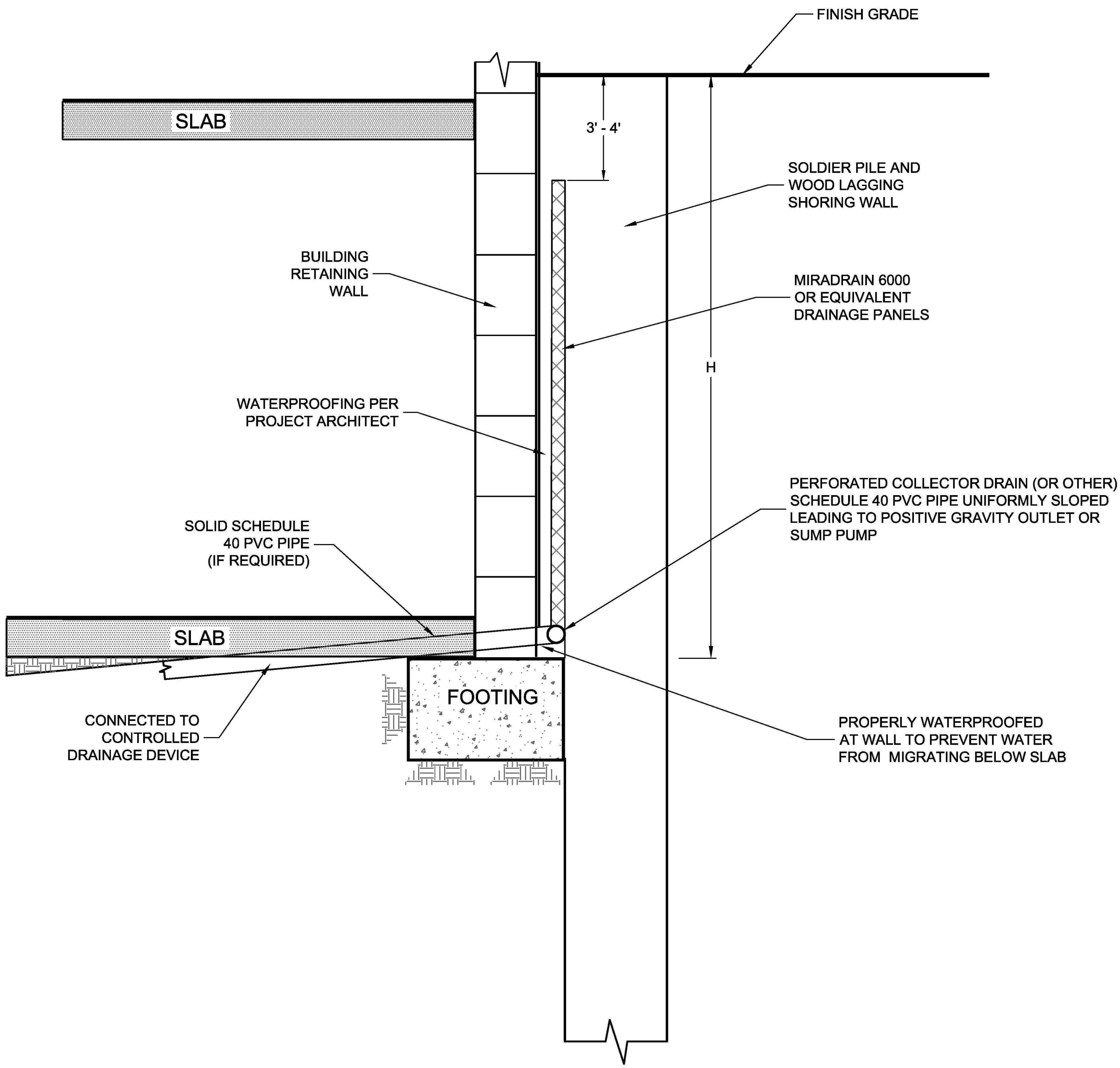
RM / AML		DSK/GTYPD	DATE 01 - 05 - 2018	PROJECT NO. G2209 - 42 - 01	FIG. 13



SOLDIER PILE WALL DRAINAGE DETAIL

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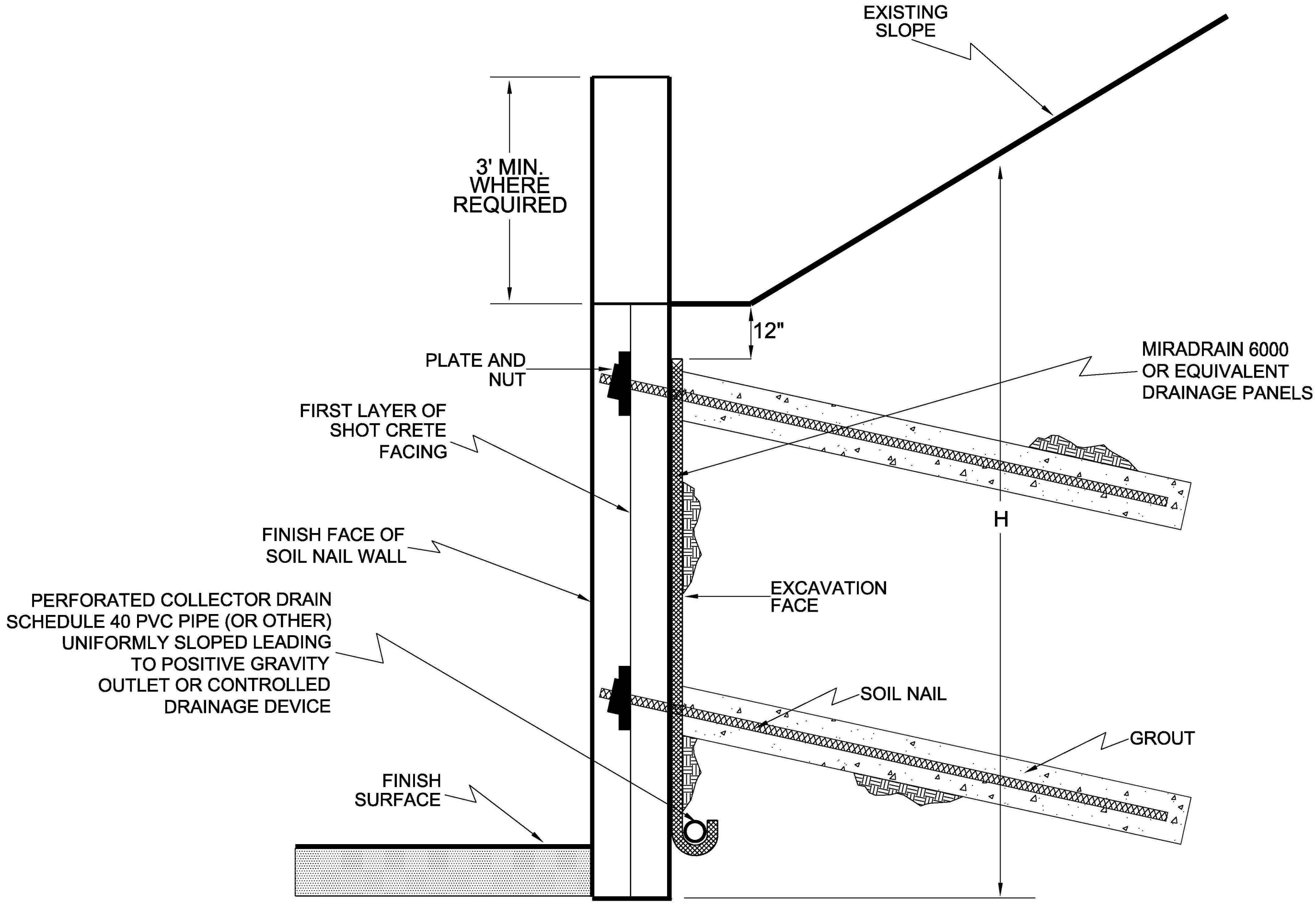
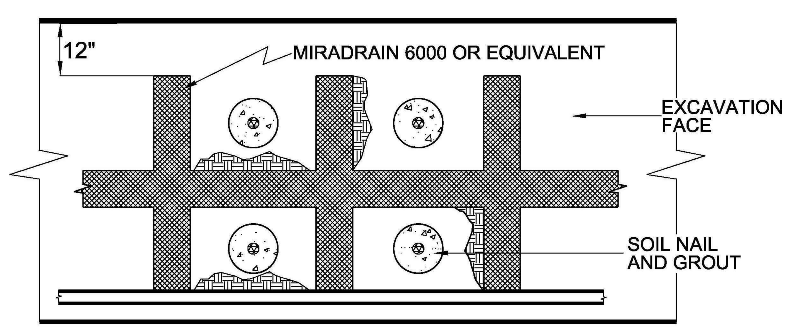
RM / AML		DSK/GTYPD	DATE 01 - 05 - 2018	PROJECT NO. G2209 - 42 - 01	FIG. 14



NO SCALE



DRAINAGE PANELS PER SOIL NAIL WALL ENGINEER

NO SCALE

SOIL NAIL WALL DRAINAGE DETAIL

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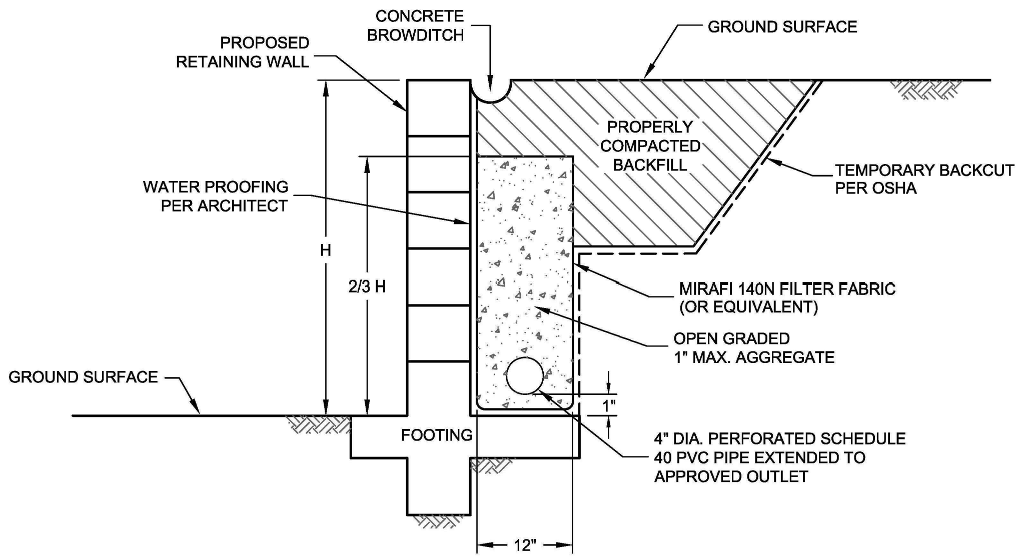
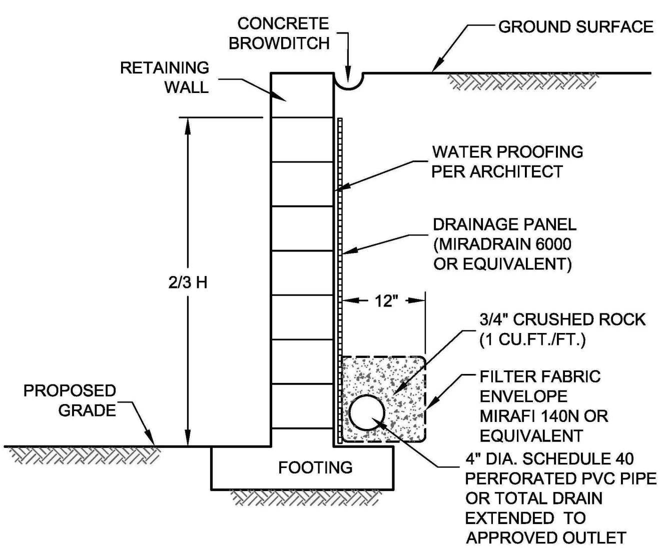
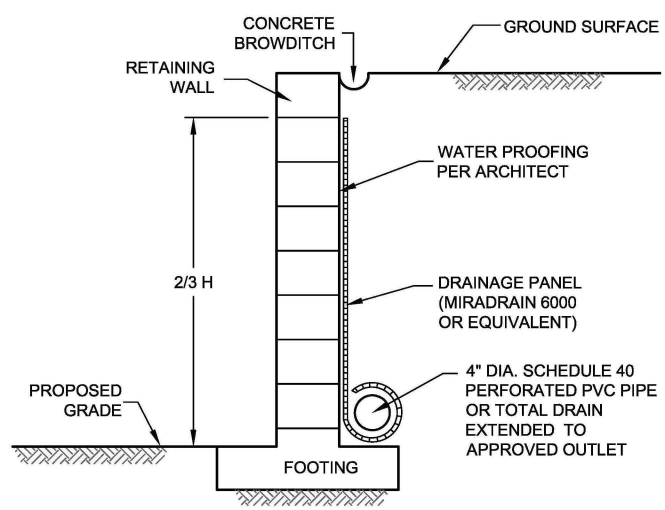
RM / AML		DSK/GTYPD	DATE 01 - 05 - 2018	PROJECT NO. G2209 - 42 - 01	FIG. 15







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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6960 FLANDERS DRIVE - SAN DIEGO, CALIFORNIA 92121 - 2974
PHONE 858 558-6900 - FAX 858 558-6159

PASEO MONTRIL
SAN DIEGO, CALIFORNIA

RM / AML		DSK/GTYPD	DATE 01 - 05 - 2018	PROJECT NO. G2209 - 42 - 01	FIG. 16

Plotted:01/04/2018 3:29PM | By:ALVIN LADRILLONO | File Location:Y:\PROJECTS\G2209-42-01 (Paseo Montril)\DETAILS\Typical Retaining Wall Drainage Detail (RWDD7A).dwg

APPENDIX A

APPENDIX A

FIELD INVESTIGATION

Fieldwork for our investigation was performed on November 15, 2017 and included a site reconnaissance and subsurface exploration. The subsurface exploration consisted of four backhoe test pits and six air-track percussion borings. The exploratory trenches were excavated using a John Deere 410G rubber tire backhoe with a 2-foot-wide bucket and extended to depths between 4 feet and 17 feet. The air-percussion borings were performed using an Ingersoll Rand ECM 370 equipped with a 4-inch bit. The borings extended to depths between 24 feet and 76 feet.

The approximate locations of trenches and borings are shown on the Geologic Map, Figure 2 (Map Pocket). The trenches and borings were located in the field based on visual reference points. Therefore, actual locations may deviate slightly.

The soil encountered in the borings were visually examined, 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). The logs depict the soil and geologic conditions observed and the depth at which samples were obtained. Logs of the trenches are presented on Figures A-1 through A-4. The logs depict the soil and geologic conditions encountered. Logs of the air-track borings are presented on Figures A-5 through A-10.

PROJECT NO. G2209-42-01

TRENCH T 1

ELEV. (MSL.) 487' DATE COMPLETED 11-15-2017

EQUIPMENT BY: G. CANNON

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/GW	**UNDOCUMENTED FILL (Qudf)** Loose, dry, brown, Clayey, fine to medium SAND and GRAVEL			
4				CH	**TOPSOIL (WEATHERED Mzu)** Stiff, moist, red brown, fine, FAT CLAY			
11	T1-1							
13					Dark olive - more Sand and Gravel (angular Mzu)			
15	T1-2							
17					TRENCH TERMINATED AT 17 FEET No groundwater encountered			

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

G2209-42-01.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. G2209-42-01

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	TRENCH T 2 ELEV. (MSL.) 473' DATE COMPLETED 11-15-2017 EQUIPMENT BY: G. CANNON	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					MATERIAL DESCRIPTION			
0				CH	**TOPSOIL (WEATHERED Mzu)** Stiff, moist, red brown, fine, FAT CLAY			
2								
4								
6								
8					**METAMORPHIC ROCK (Mzu)** Moderate to slightly weathered, dark gray, intensely fractured, META-SEDIMENTARY ROCK			
					TRENCH TERMINATED AT 9 FEET No groundwater encountered			

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

G2209-42-01.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 3** ELEV. (MSL.) **508'** DATE COMPLETED **11-15-2017** EQUIPMENT BY: **G. CANNON**	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					MATERIAL DESCRIPTION			
0				CH	**TOPSOIL** Stiff, moist, dark red brown, fine, FAT CLAY			
2					**METAMORPHIC ROCK (Mzu)** Moderate to slightly weathered, dark gray, intensely fracture, META-SEDIMENTARY ROCK			
4					TRENCH TERMINATED AT 4 FEET No groundwater encountered			

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

G2209-42-01.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 4

ELEV. (MSL.) **516'** DATE COMPLETED **11-15-2017**

EQUIPMENT BY: **G. CANNON**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0				CH	**TOPSOIL** Soft, dry, red brown, fine, FAT CLAY			
2					**METAMORPHIC ROCK (Mzu)** Moderate to slightly weathered dark gray, intensely fractured, META-SEDIMENTARY ROCK			
4	T4-1							
					TRENCH TERMINATED AT 5 FEET No groundwater encountered			

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

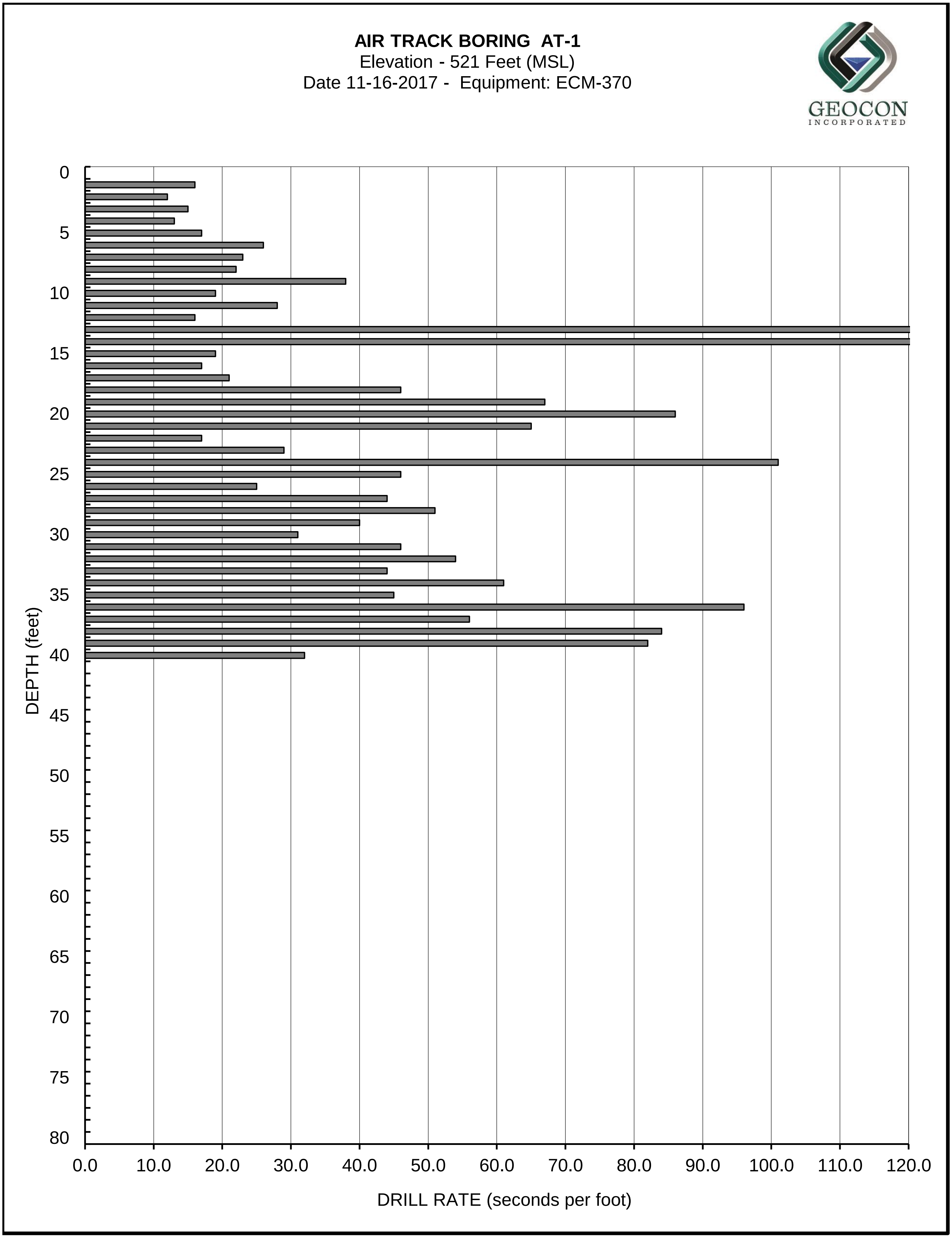
G2209-42-01.GPJ

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. G2209-42-01

PASEO MONTRIL



PASEO MONTRIL

AIR TRACK BORING AT-2
Elevation - 537 Feet (MSL)
Date 11-16-2017 - Equipment: ECM-370

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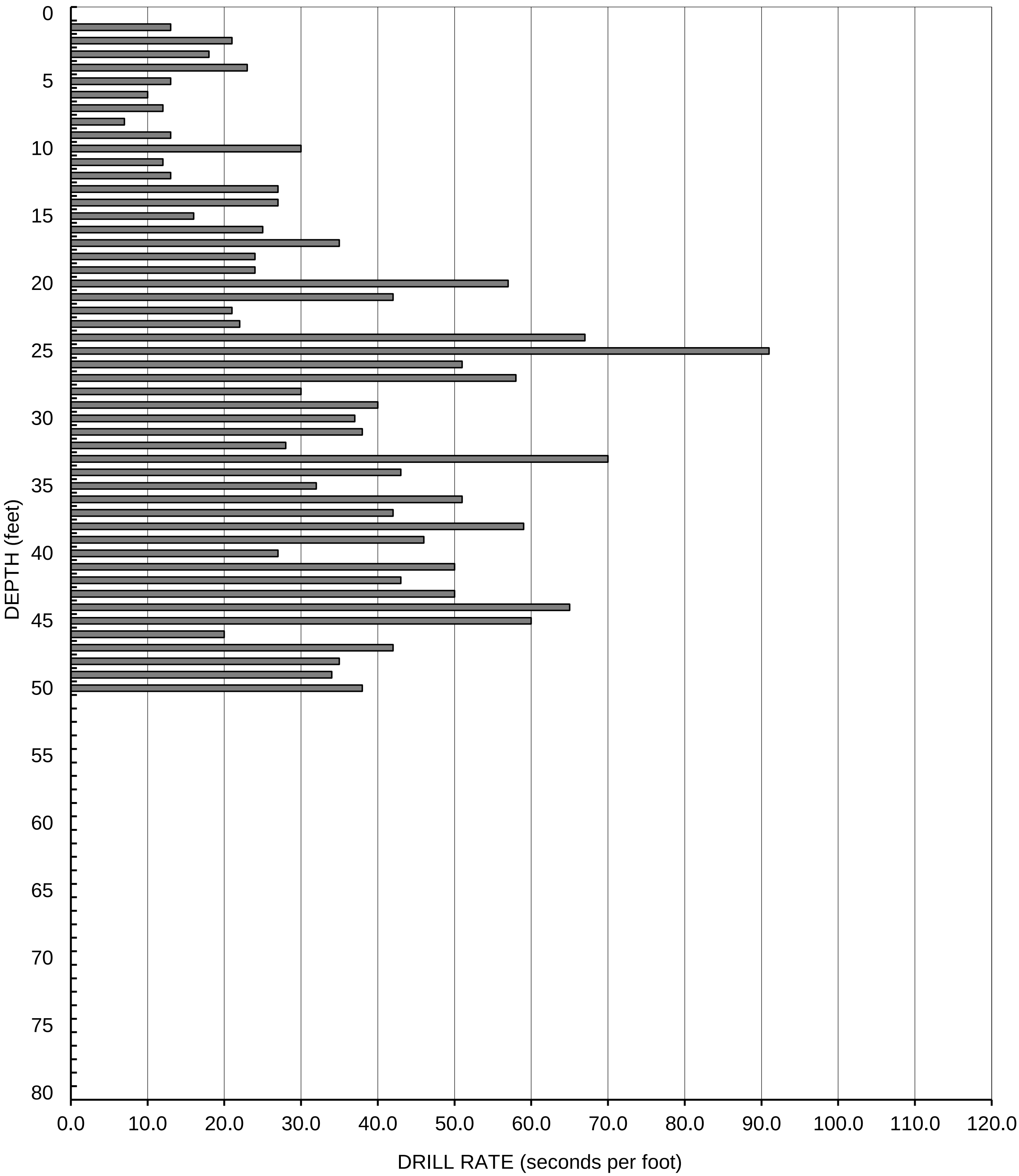
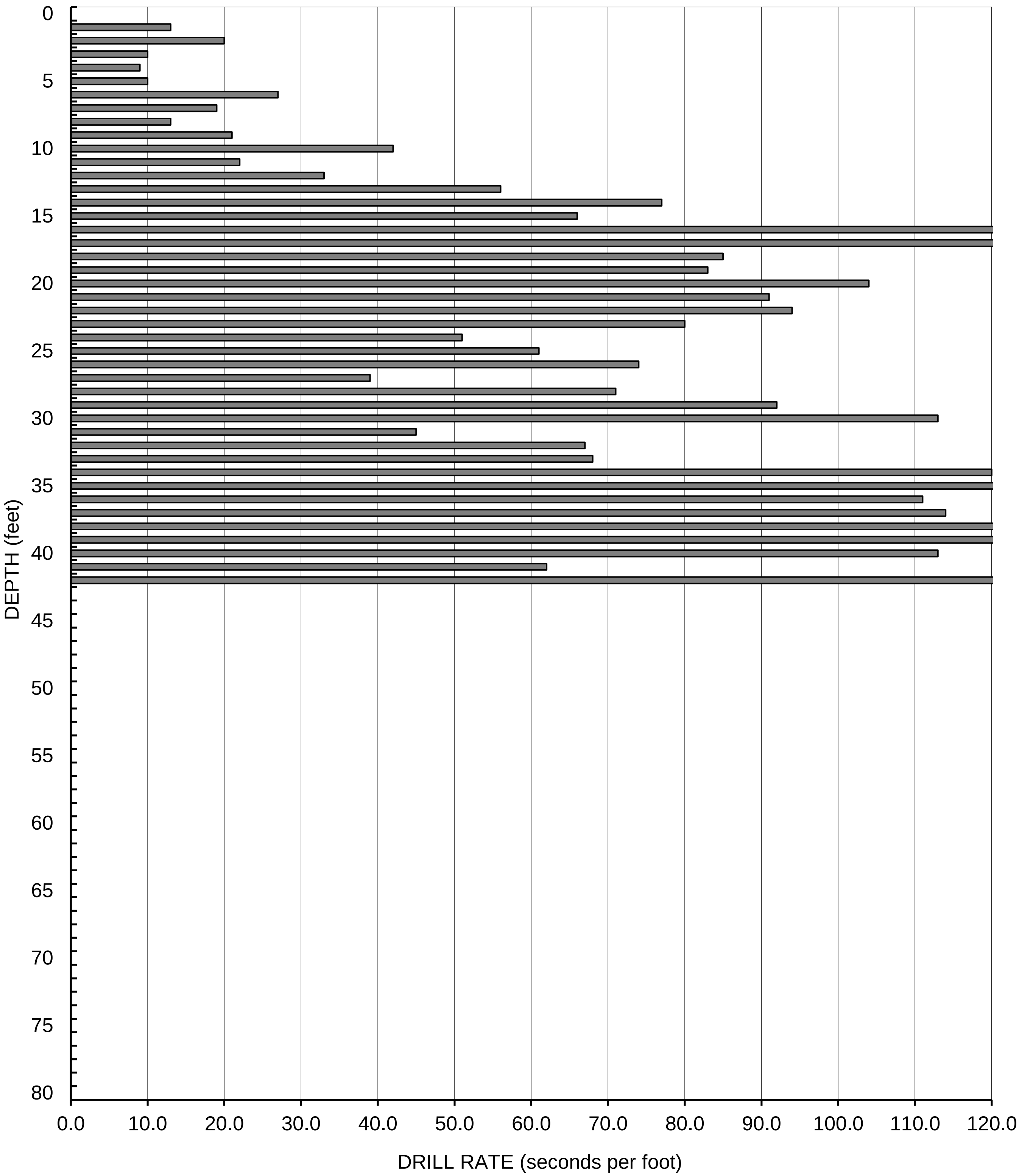


PASEO MONTRIL

AIR TRACK BORING AT-3
Elevation - 550 Feet (MSL)
Date 11-16-2017 - Equipment: ECM-370

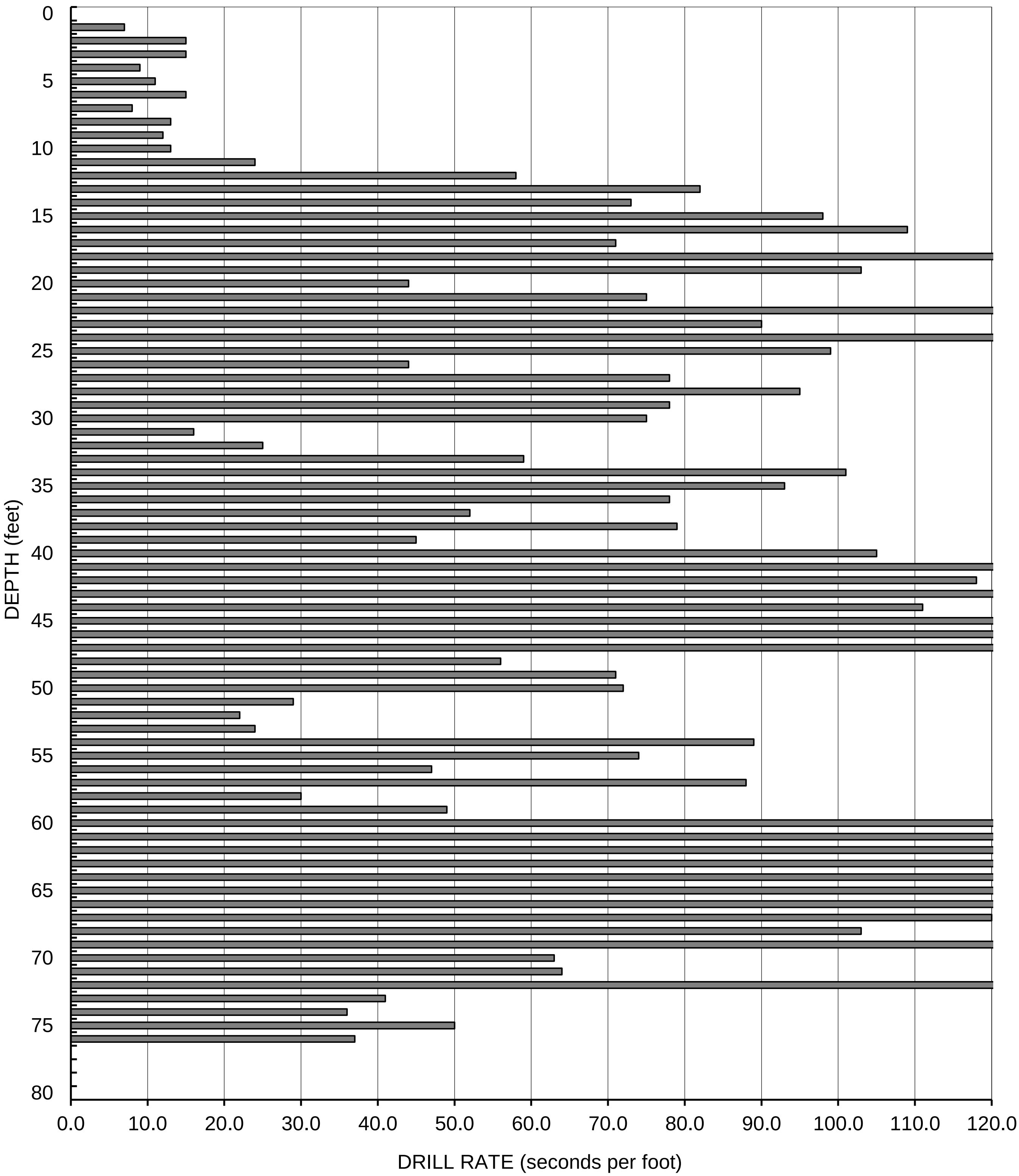
GEOCON INCORPORATED



PASEO MONTRIL

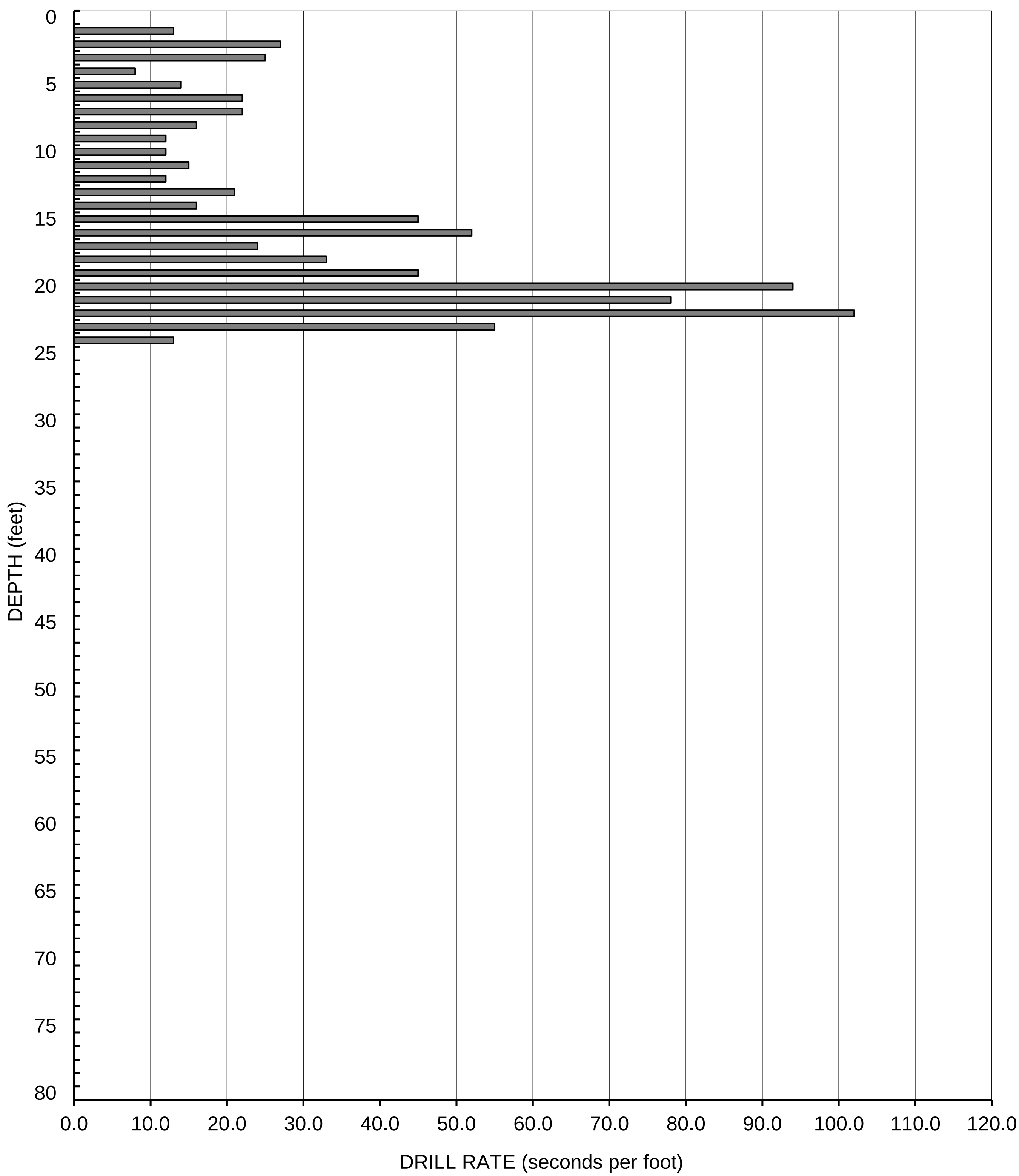
AIR TRACK BORING AT-4
Elevation - 560.5 Feet (MSL)
Date 11-15-2017 - Equipment: ECM-370

GEOCON INCORPORATED



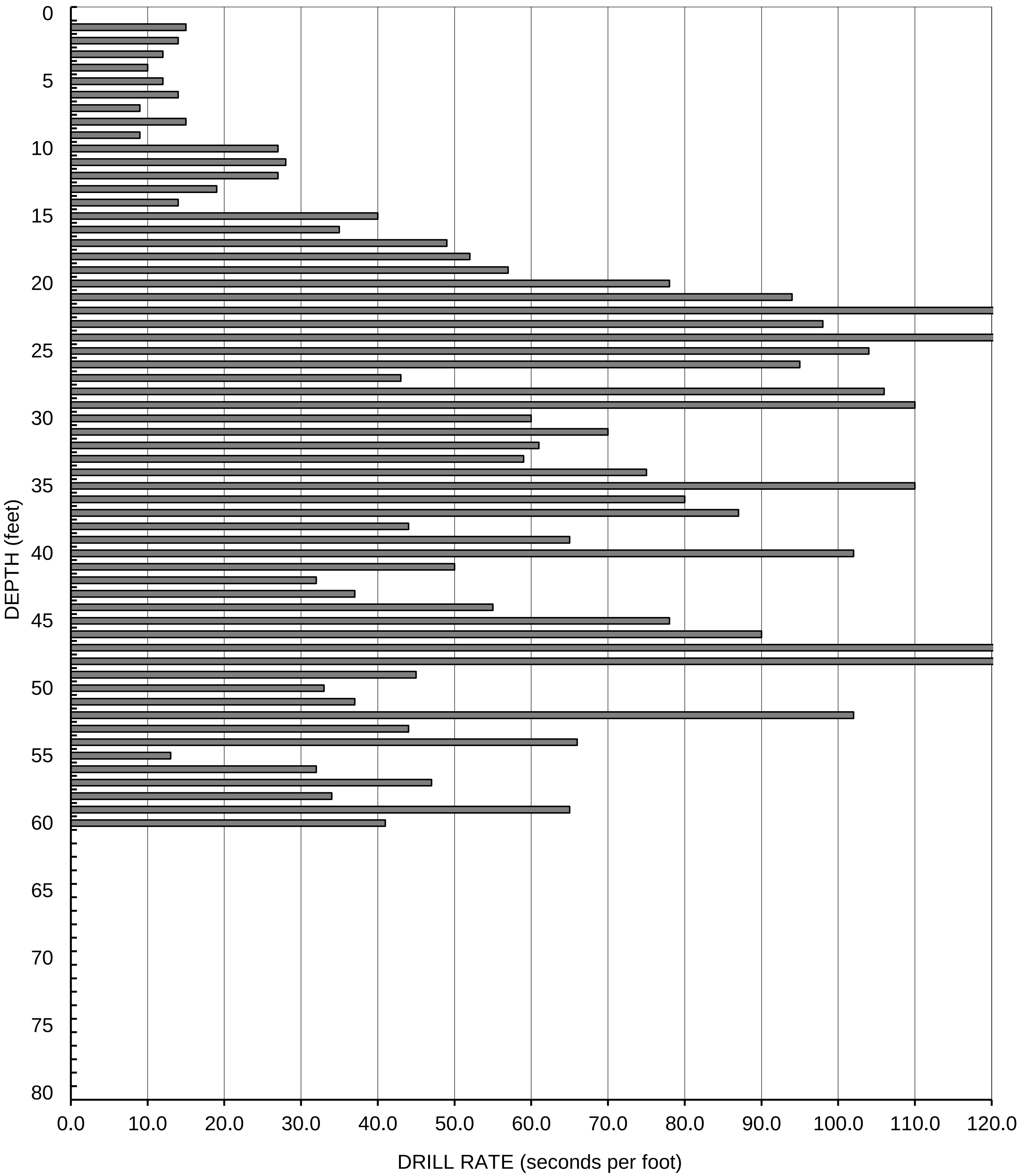
PASEO MONTRIL

AIR TRACK BORING AT-5
Elevation - 560 Feet (MSL)
Date 11-15-2017 - Equipment: ECM-370



PASEO MONTRIL

AIR TRACK BORING AT-6
Elevation - 549.5 Feet (MSL)
Date 11-15-2017 - Equipment: ECM-370



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 their maximum dry density and optimum moisture content, expansion characteristics, gradation, Atterberg limits, and water-soluble sulfate content. The results of our laboratory tests are summarized on the following tables and graphs.

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.)
T1-1	Dark brown CLAY with trace gravel and little sand	112.7	17.7
T1-2	Gray brown CLAY with trace gravel and sand	113.3	16.2

TABLE B-II
SUMMARY OF LABORATORY EXPANSION INDEX TEST RESULTS

Sample No.	Moisture Content (%)		Dry Density (pcf)	Expansion Index	Expansion Classification
	Before Test	After Test			
T1-1	14.7	34.9	93.7	107	High
T1-2	13.6	31.2	95.8	115	High

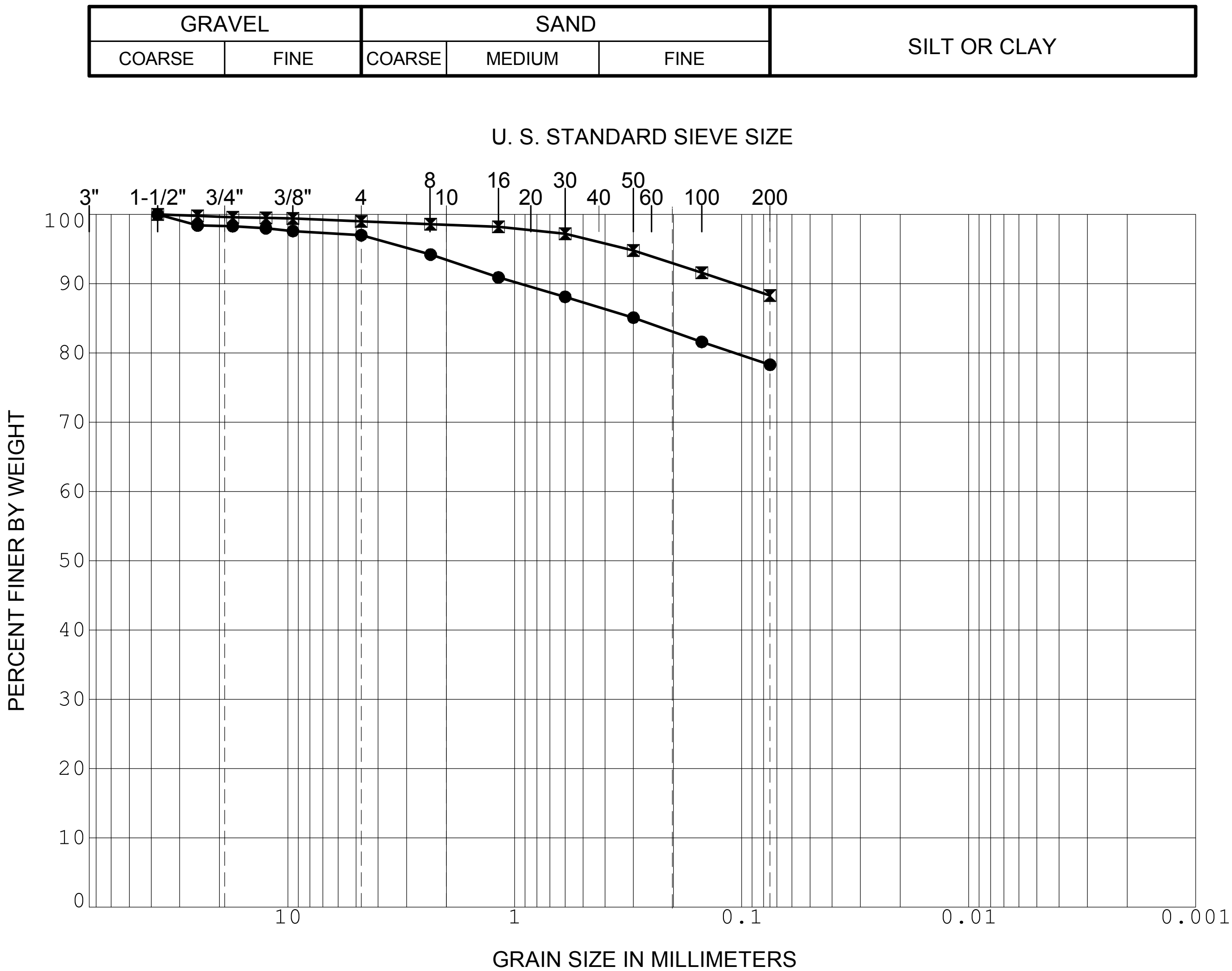
TABLE B-III
SUMMARY OF LABORATORY WATER-SOLUBLE SULFATE TEST RESULTS

Sample No.	Water-Soluble Sulfate Content (%)	Exposure
T1-1	0.034	Not Applicable
T1-2	0.038	Not Applicable

TABLE B-IV
SUMMARY OF LABORATORY ATTERBERG LIMITS TEST RESULTS
ASTM D 4318

Sample No.	Description	Liquid Limit (LL)	Plastic Limit (PL)	Plasticity Index (PI)	Unified Soil Classification (Group Symbol)
T1-1	Dark brown Fat CLAY	65	20	45	CH
T1-2	Gray Brown Fat CLAY	50	27	23	CH

PROJECT NO. G2209-42-01



	SAMPLE	DEPTH (ft)	CLASSIFICATION	NAT WC	LL	PL	PI
●	T1-1	11.0	(CH) FAT CLAY with Sand		65	20	45
⧗	T1-2	15.0	(CH) FAT CLAY		50	27	23
▲							

GRADATION CURVE

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SAN DIEGO, CALIFORNIA

G2209-42-01.GPJ

Figure B-1

GEOCON

APPENDIX C

APPENDIX C

STORM WATER MANAGEMENT

If storm water management devices are not properly designed and constructed, there is a risk for distress to improvements and properties located hydrologically down gradient or adjacent to these devices. Factors such as the amount of water being 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 into the subsurface occurs, downstream improvements 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, provides general information regarding soil conditions for areas within the United States. The USDA website also provides the Hydrologic Soil Group. Table C-1 presents the descriptions of the hydrologic soil groups. If a soil is assigned to a dual hydrologic group (A/D, B/D, or C/D), the first letter is for drained areas and the second is for undrained areas.

TABLE C-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 subject property is underlain by soil and geologic units consisting of undocumented fill, alluvium, terrace deposits, and granitic rock. The property falls within Hydraulic Soil Groups B, C, and D, which range from moderate infiltration characteristics to very slow infiltration. The majority of the site falls within Hydrologic Soil Group C. Table C-2 presents the information from the USDA website for the property.

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

Map Unit Name	Map Unit Symbol	Approximate Percentage of Property	Hydrologic Soil Group	Estimated Infiltration Rate (in/hr)
Diablo-Olivenhain complex, 9 to 30 percent slopes	DoE	7	D	0.06
Friant rocky fine sandy loam, 9 to 30 percent slopes	FxE	25	D	2
Olivenhain cobbly loam, 9 to 30 percent slopes	OhE	68	D	0.06

Summary of Existing and Future Graded Soil Conditions

Because the property is in an ungraded condition, the existing soil conditions do not reflect the soil conditions that will be present at the completion of grading. Currently, the site is underlain by undocumented fill, topsoil, weathered Metamorphic rock and Metamorphic Rock. Grading will result in cuts up to approximately 50 feet in northern portion of the property and fills along the eastern, southern and southwest portions of the property. At the completion of grading, the site will be underlain by compacted fill overlying Metamorphic Rock. Compacted fill depths are expected to range from 5 feet (bedrock undercut areas) to 30 feet in fill areas.

Infiltration Testing

Infiltration testing has not been performed as proposed grading will result in cuts and fills across the entire site and in-situ tests performed now will not reflect actual conditions at the completion of grading. Estimated infiltration rates from the USDA Web Soil Survey for each of the mapped soil units is shown on Table C-2.

STORM WATER MANAGEMENT CONCLUSIONS

Soil Types

At the completion of grading the site will be underlain by compacted fill and Metamorphic Rock. Compacted fill depths will range from approximately 5 feet in building pad undercut areas to 30 feet in fill areas. Infiltration into compacted fill is considered unfeasible due to the potential for settlement of structural improvements and lateral seepage migration into the retaining wall backfill along the perimeter of the project. Infiltration into the Metamorphic Rock is also considered infeasible due to its dense/hard nature and the potential to cause lateral water migration to structural improvements and slopes.

Infiltration Rates

Based on the USDA Web Soil Survey, we recommend an unfactored infiltration rate of 0.06 in/hr. The 2 in/hr indicated on the soil survey website for FxE is located in the hillside and drainage on the east side of the project. Grading along the eastern side of the property will result in compacted fill and walls up to 14 feet high.

Existing and Proposed Structures

There are no existing structures present on the property. However, at the completion of grading, residential multi-family structures and infrastructure be constructed across the property.

Groundwater

Groundwater was not encountered in our exploratory excavations. Groundwater is estimated to be at depths greater than 50 feet below proposed finish grades.

Soil or Groundwater Contamination

We are unaware of contaminated soil or groundwater on the property. Therefore, infiltration associated with this risk is considered feasible.

Slopes

New fill slopes are planned at the southwest and southeast corners of the site. A cut slope will be constructed along the northwest side of the property. An existing cut slopes that extends down to Interstate 15 exists on the south side of the site. Infiltration near slopes is not recommended due to the potential for lateral water migration.

Storm Water Management Devices

If basins are utilized, a liner with subdrains is recommended. The liner should be impermeable (e.g. High-density polyethylene, HDPE, with a thickness of about 30 mil or equivalent Polyvinyl Chloride**,** PVC). The subdrain should be perforated, be at least 4 inches in diameter and consist of Schedule 40 PVC pipe and surrounded in gravel. The subdrain should be connected to a proper outlet. If storage vaults are utilized, the vaults should be water-tight.

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. Worksheets C.4-1 have been attached.

The regional storm water standards also have a worksheet (Worksheet D.5-1 or Form I-9) that helps the project civil engineer estimate the factor of safety based on several factors. Table C-3 describes the suitability assessment input parameters related to the geotechnical engineering aspects for the factor of safety determination.

TABLE C-3
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

Table C-4 presents the estimated factor values for the evaluation of the factor of safety. The factor of safety is determined using the information contained in Table C-4 and the results of our geotechnical investigation. Table C-4 only presents the suitability assessment safety factor (Part A) of the worksheet. The project civil engineer should evaluate the safety factor for design (Part B of Worksheet D.5-1) and use the combined safety factor for the design infiltration rate.

TABLE C-4
FACTOR OF SAFETY WORKSHEET D.5-1 DESIGN VALUES – PART A[1]

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

1 The project civil engineer should complete Part B of Worksheet D.5-1 or Form I-9 to determine the overall factor of safety.

CONCLUSIONS AND RECOMMENDATIONS

It is our opinion that infiltration is infeasible due to expected low infiltration rates in the bedrock soils, as well as the presence of fill and retaining walls that will be constructed on the property. Our evaluation included the soil and geologic conditions, settlement and volume change of the underlying soil, slope stability, utility considerations, groundwater mounding, retaining walls, foundations, and existing groundwater elevations.

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:**
Overall Site	

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 includes 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.

Based on theUSDA Web Soil Survey, 75% of the site area has an infiltraiton rate of 0.06 in/hr or less. The other 25% of the site area is listed as having an estimated infiltration rate of 2 in/hr and is located along the eastern side of the site. However, based on field mapping, the area is underlain by hard metamorphic rock and is expected to have an infiltration rate of less than 0.5 in/hr. This area will recevie cuts to achieve proposed pad grade and fills in excess of 5 feet. In addition, in this area, retaining walls and building structures are planned. There is no reasonable area outside of the strucural improvements or compacted fill areas where an infiltraiton basin could be constructed due to the sloping hillside condition and sensitive habitat along the east side of the site.

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

<table>
<tr><th colspan="2">Categorization of Infiltration Feasibility Condition based on Geotechnical Conditions</th><th colspan="2">Worksheet C.4-1: Form I- 8A[10]</th></tr>
<tr><td>2C</td><td>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.</td><td>☐ Yes</td><td>☐ No</td></tr>
<tr><td>Criteria 2 Result</td><td>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?</td><td>☐ Yes</td><td>☐ No</td></tr>
<tr><td colspan="4">Summarize findings and basis; provide references to related reports or exhibits.</td></tr>
<tr><th colspan="2">Part 1 Result – Full Infiltration Geotechnical Screening [12]</th><th colspan="2">Result</th></tr>
<tr><td colspan="2">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.</td><td colspan="2">☐ Full infiltration Condition

☒ Complete Part 2</td></tr>
</table>

[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:**
Overall Site	

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).

Based on theUSDA Web Soil Survey, 75% of the site area has an infiltraiton rate of 0.06 in/hr or less. The other 25% of the site area is listed as having an estimated infiltration rate of 2 in/hr and is located along the eastern side of the site. However, based on field mapping, the area is underlain by hard metamorphic rock and is expected to have an infiltration rate of less than 0.05 in/hr. This area will recevie cuts to achieve proposed pad grade and fills in excess of 5 feet. In addition, in this area, retaining walls and building structures are planned. There is no reasonable area outside of the strucural improvements or compacted fill areas where an infiltraiton basin could be constructed due to the sloping hillside condition and sensitive habitat along the east side of the site.

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

<table>
<tr><th colspan="2">Categorization of Infiltration Feasibility Condition based on Geotechnical Conditions</th><th colspan="2">Worksheet C.4-1: Form I- 8A[10]</th></tr>
<tr><td>Criteria 4 Result</td><td>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?</td><td>☐ Yes</td><td>☐ No</td></tr>
<tr><td colspan="4">Summarize findings and basis; provide references to related reports or exhibits.</td></tr>
</table>

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.

APPENDIX D

APPENDIX D

RECOMMENDED GRADING SPECIFICATIONS

FOR

PASEO MONTRIL
SAN DIEGO, CALIFORNIA

PROJECT NO. G2209-42-01

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

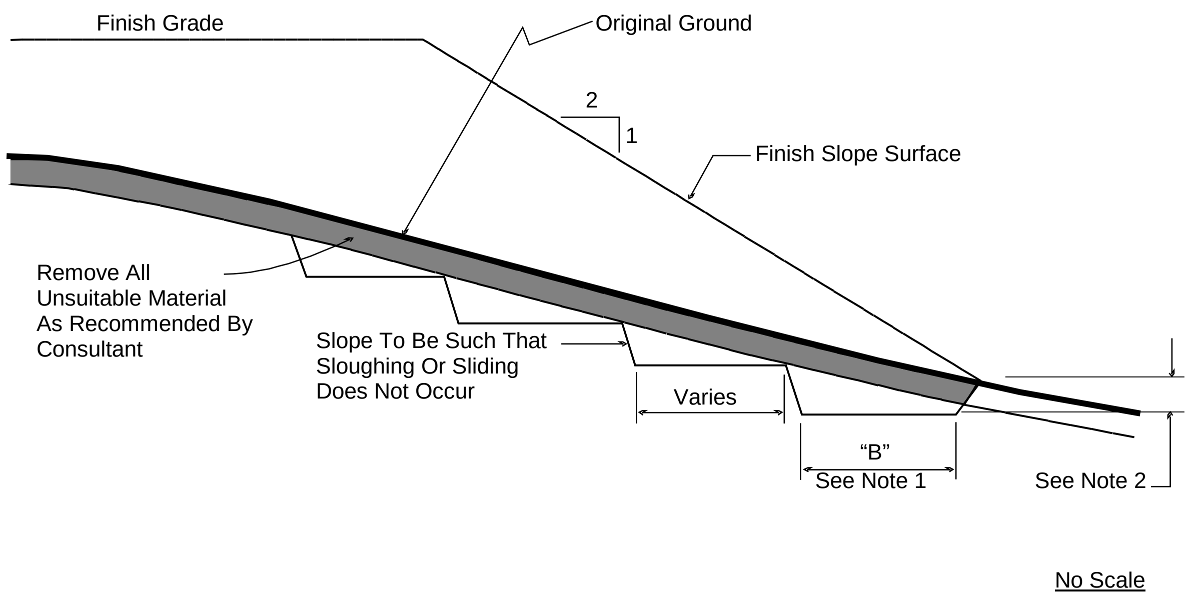
4.1 Areas to be excavated and filled shall be cleared and grubbed. Clearing shall consist of complete removal above the ground surface of trees, stumps, brush, vegetation, man-made structures, and similar debris. Grubbing shall consist of removal of stumps, roots, buried logs and other unsuitable material 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.

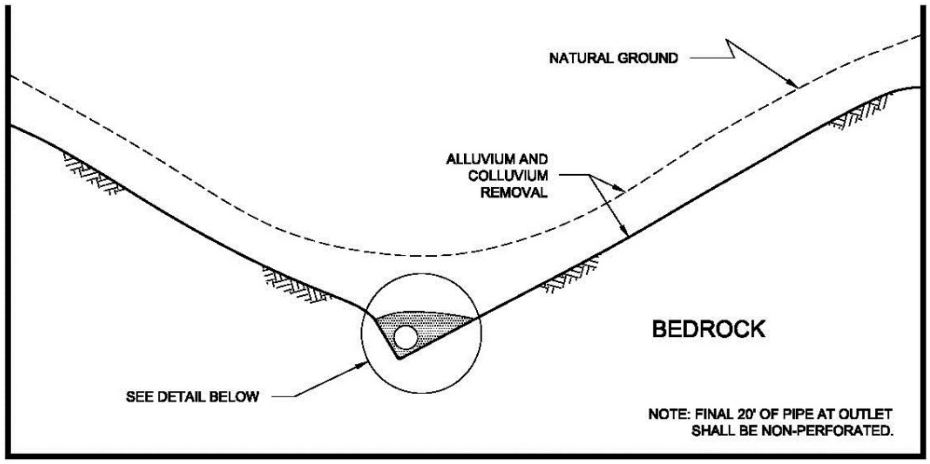
6.3.6 To reduce the potential for "piping" of fines into the *rock* fill from overlying *soil* fill material, a 2-foot layer of graded filter material shall be placed above the uppermost lift of *rock* fill. The need to place graded filter material below the *rock* should be determined by the Consultant 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.

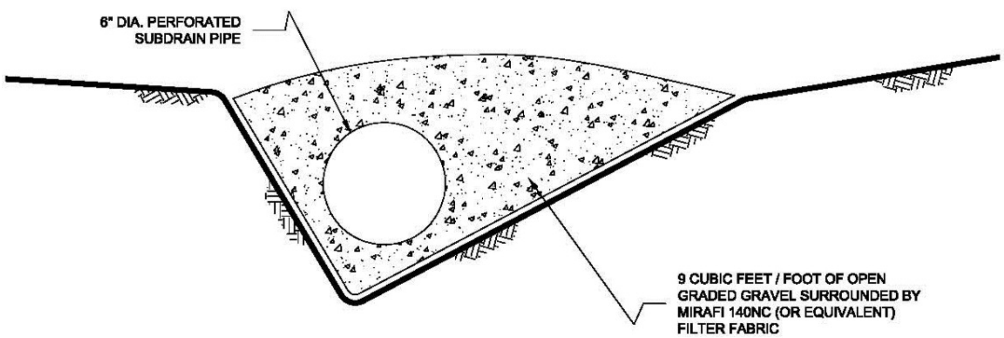
6.3.7 *Rock* fill placement should be continuously observed during placement by the Consultant.

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:

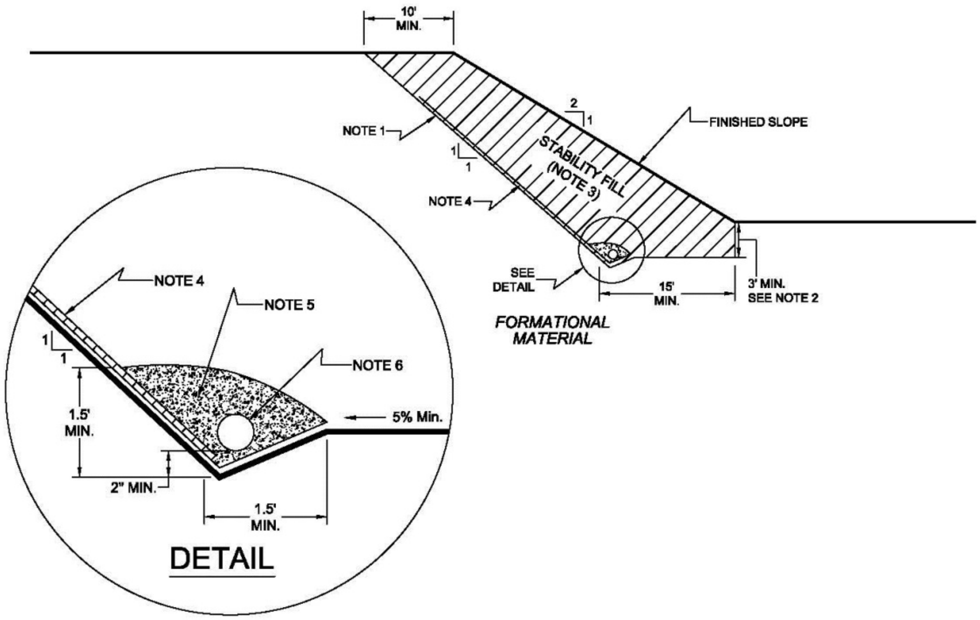
1......8-INCH DIAMETER, SCHEDULE 80 PVC PERFORATED PIPE FOR FILLS IN EXCESS OF 100-FEET IN DEPTH OR A PIPE LENGTH OF LONGER THAN 500 FEET.

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

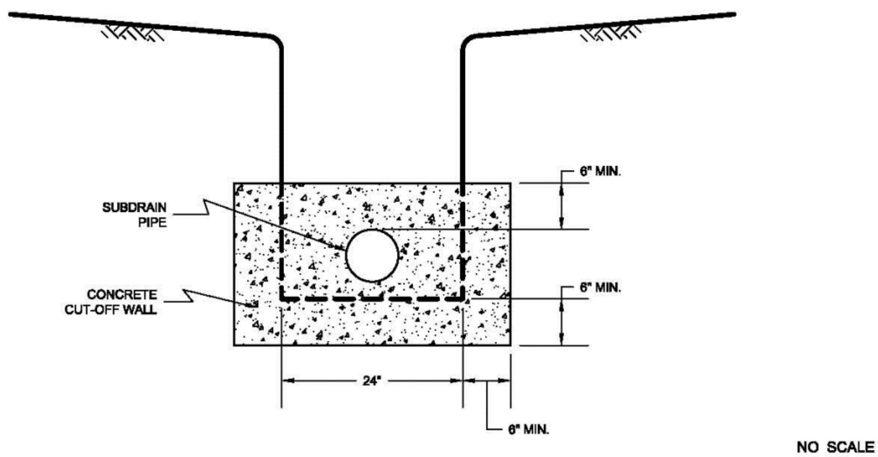
7.3 The actual subdrain locations will be evaluated in the field during the remedial grading operations. Additional drains may be necessary depending on the conditions observed and the requirements of the local regulatory agencies. Appropriate subdrain outlets should be evaluated prior to finalizing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

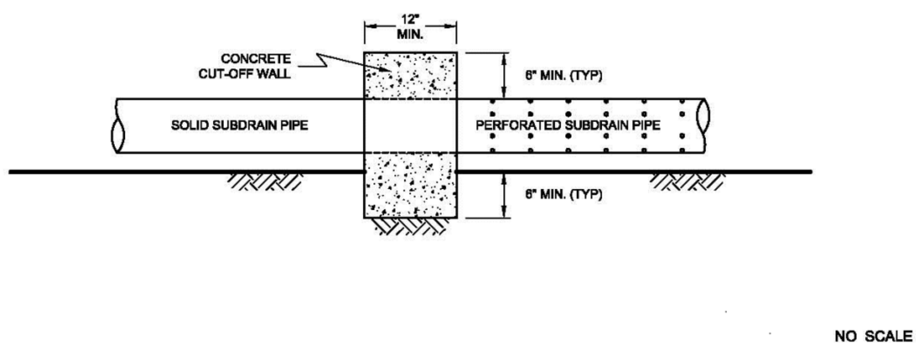
7.5 Prior to outletting, the final 20-foot segment of a subdrain that will not be extended during future development should consist of non-perforated drainpipe. At the non-perforated/ perforated interface, a seepage cutoff wall should be constructed on the downslope side of the pipe.

TYPICAL CUT OFF WALL DETAIL

FRONT VIEW

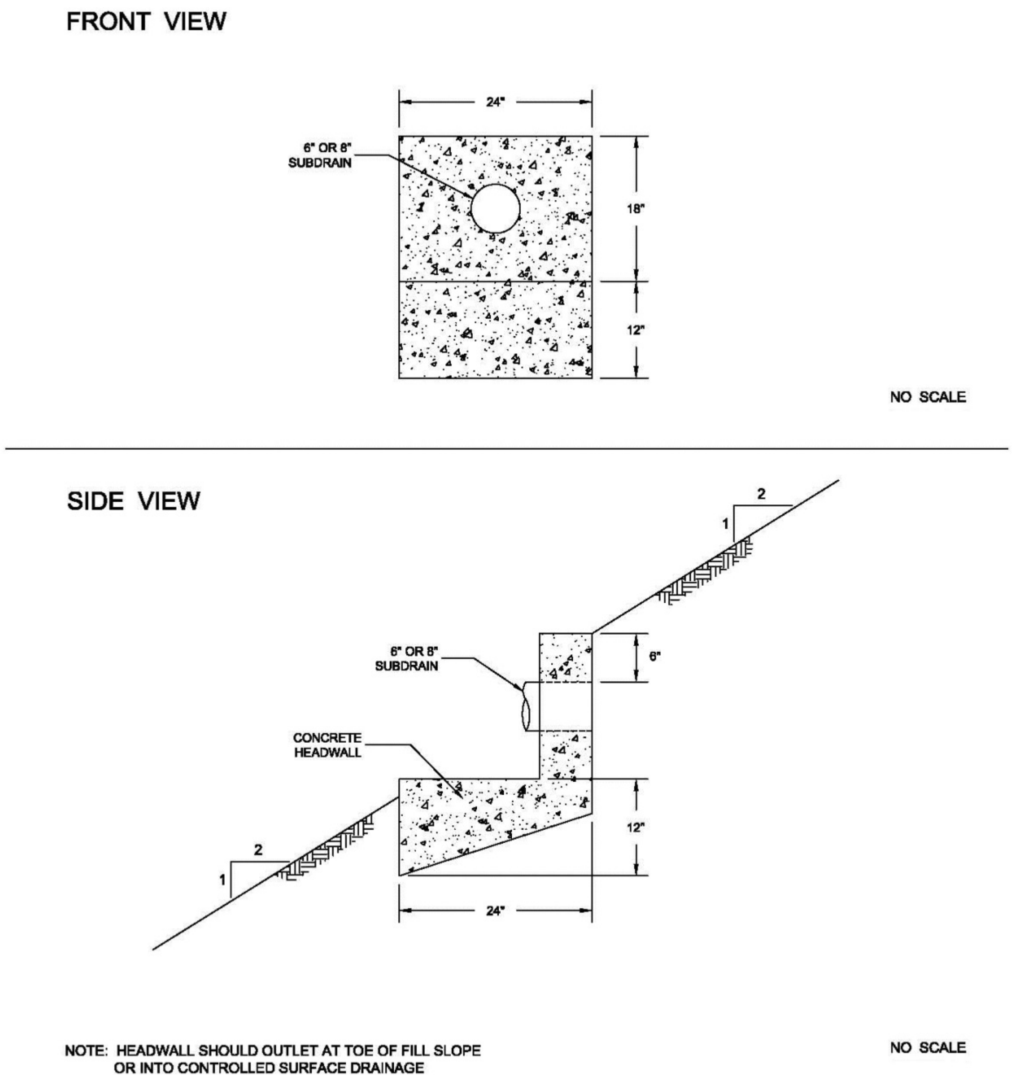


SIDE VIEW



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



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

Risk Engineering (2015), *EZ-FRISK (version 7.62)*, software package used to perform site-specific earthquake hazard analyses, accessed January 4, 2018;

FEMA (2012), *Flood Insurance Rate Map (FIRM) Map Number 06073C1353G, effective May 16, 2012*, http://www.fema.gov, accessed January 4, 2018;

Kennedy, M. P., and S. S. Tan, (2005), *Geologic Map of the San Diego 30' x 60' Quadrangle, California*, Californian Geological Survey, Regional Map Series, 1:100,000 Scale, Map No. 3;

USGS (2014), *U.S. Seismic Design Maps Web Application (version 3.1.0)*, http://earthquake.usgs.gov/designmaps/us/application.php. Accessed January 3, 2018;

USGS (2016), *Quaternary Fault and Fold Database of the United States*, http://earthquakes,usgs.gov/hazards/qfaults, accessed January 4, 2018.